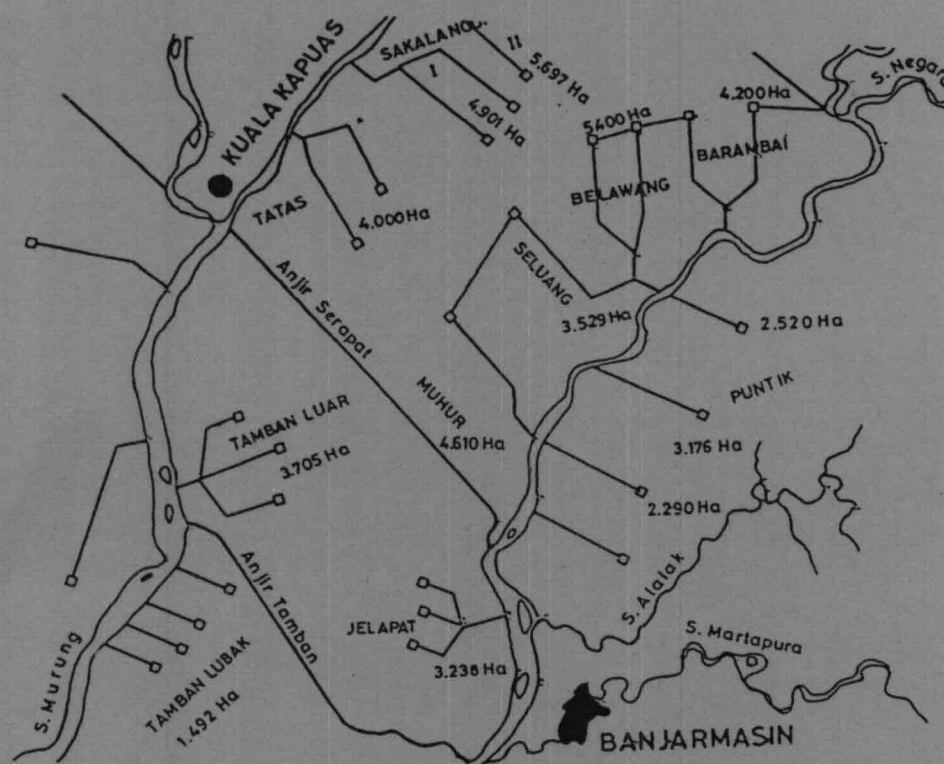A MATHEMATICAL MODELLING STUDY FOR UPGRADING WORK OF TIDAL
IRRIGATION/DRAINAGE SYSTEM OF THE BARAMBAI-SELUANG-BELAWANG-
MUHUR UNIT, SOUTH KALIMANTAN, INDONESIA

By : Suryadi
Delft, August 1987

INTERNATIONAL INSTITUTE FOR HYDRAULIC AND ENVIRONMENTAL
ENGINEERING, DELFT, THE NETHERLANDS

A MATHEMATICAL MODELLING STUDY FOR UPGRADING WORK OF TIDAL IRRIGATION/DRAINAGE SYSTEM OF THE BARAMBAI-SELUANG-BELAWANG-MUHUR UNIT, SOUTH KALIMANTAN, INDONESIA
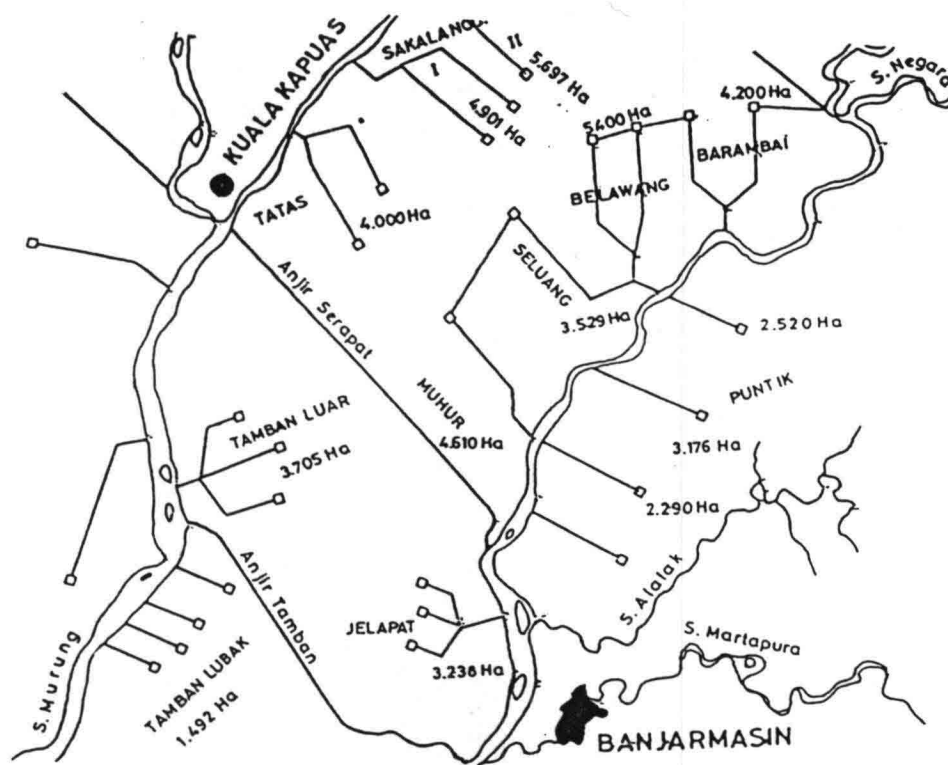

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A thesis submitted for awarding the degree of Master of Science of the International Institute for Hydraulic and Environmental Engineering, Delft, The Netherlands

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1 INTRODUCTION

1.1 History of lowland reclamation

In line with the execution of well balanced and optimal regional and national development, reclamation of tidal swamp areas in Indonesia is to be regarded as the effort to achieve people's welfare through both usage and conservation of natural resources.

Swamp areas have their own characteristics. Based on this and due to the lack of knowledge and experience in this field, these areas have been developed by the Indonesian Government following a so called step by step approach. This approach starts with reclamation, which will be followed by upgrading the schemes in various steps (see figure 1.1).

I	:	Reclamation
III	:	First upgrading step
II,IV,VI	:	Period of growth
V	:	Second upgrading step
VII	:	Third upgrading step
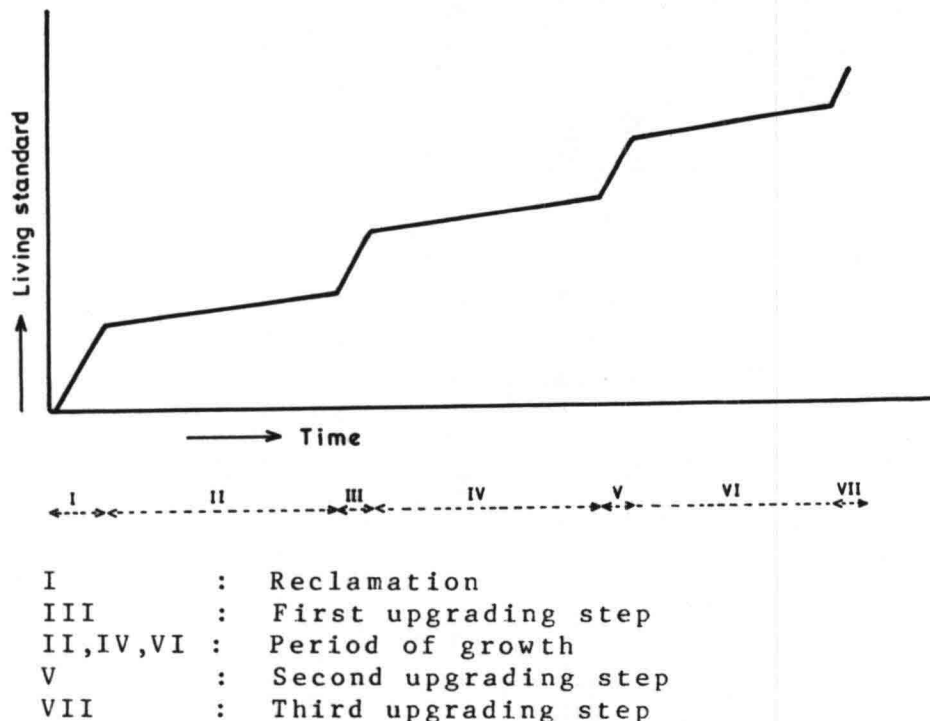
Figure 1.1 : Schematic presentation of reclamation, present state of development in existing tidal swamp schemes and step of upgrading

Because of the large area of tidal swamps that were to be reclaimed, the reclamation followed a so called low cost/simple technology approach. Most of the approximately 1 million ha of reclaimed areas are still in their first stage of development.
For the near future, activities will be focussed on the first upgrading.

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1.2 Lowland reclamation by the Indonesian Government

Data, collected up to 1977, showed that the swamp areas from Sumatra, Kalimantan, Sulawesi and Irian Jaya possess a high agricultural potential.
The total swamp area was estimated as 40 million ha, which can be divided into:
- Tidal swamp areas   :   7,000,000 ha
- Non tidal swamp areas : 33,000,000 ha
For location of these areas see figure 1.

With the start of the first Five Year Development Programme (PELITA I, 1969-1973), the Government of Indonesia reclaimed swamp areas in Sumatra and Kalimantan with the following objectives :
- to increase the national food production, especially rice, in order to be self-sufficient;
- to provide agricultural land to transmigrants, in order to support the Government transmigration programme;
- to support the regional development;
- to increasethe income per capita;
- to increase defence posts on coasts along the border line.

From 1970 until now, 1 million ha of swamp areas in Sumatra and Kalimantan have been reclaimed and have been made suitable for agriculture and settlement (phase I). Working hard according to the objectives, Indonesia finally succeeded in 1983 to be self sufficient in rice. Thus, at present Indonesia is able to feed its population of 160 million people. However, this situation has to be maintained in the future. With population growth of 2.3% per year this will ask for a continuous attention and can in principal be achieved in two ways:
- intensification of agricultural areas;
- extension of agricultural areas.

The agricultural intensification focusses mainly on the over-populated islands (Java and Bali) and in the existing tidal swamp schemes.
The extension programme can only be carried out in the less densily populated islands.
As far as swamp lands are concerned, because of Indonesia's present budgetary situation, the Government is now focussing more upon intensification following a low cost/simple technology approach. This means that in existing lowland schemes, where the hydraulic system is in its initial stage, measures can be carried out to improve these systems. These measures will form part of the measures needed to come to a fully controlled water management system, like a polder system.

In this context, the Government of Indonesia has included the Barambai-Seluang-Belawang-Muhur unit into an intensification and upgrading programme. This programme aims at diminishing the present limitations of the schemes by

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improving watermanagement, farming system, social structures and facilities, and by taking measures concerning the environment.
As these aspects are interrelated strongly, they have to be taken into consideration in a harmonious way.

1.3 Contents of the report

This report intends to contribute to the upgrading of the hydraulic infrastructive in existing lowland schemes, where water quality (acidity) problems occur. A summary of the contents of this report is given below:

Chapter II presents an overview of the already implemented lowland schemes in Indonesia and describes some alternatives for improving the Baramai-Seluang-Belawang-Muhur unit.
Chapter III is dealing with data collection and their intepretation. These data concern hydrometry, geometry and acidity.
Chapter IV discusses the set up of an unsteady flow model , which is based on the PENPAS programme. This is followed by the schematization of the study area, (defining nodes, branches, boundary conditions and initial conditions), the calibrations and the verification of the models, based on field data. This chapter is completed by the presentation of the results and an analysis of the alternatives.
Chapter V covers the interpretation of the results. Furthermore each alternative is discussed. From the computational results it can be concluded that the convective transport in the system is very dominant. By introducing the flushing canal without any regulating structure, it can create only very limited net outflow for each of the secondaries. It does not give significant differences between the system without flushing canal and with flushing canal (both with improved tertiary canals) or with interupted flushing canal. So, in order to have a better flushing effect, a system with a larger net outflow (wider secondary canals or with a regulating structure) is needed. This idea (wider secondary canals) can be applied for the secondary left of Barambai and the secondary of Seluang. For the wet season condition it will give an improved circulation in the system, but unfortunately for the dry season condition, a deadlock problem will occur in two of the secondary canals (the Barambai right and Muhur). So, in these canals sedimentation may occur besides poor refreshment of the canal water.
To avoid this problem, a regulating structure must be introduced for example at the end of those secondary canals. Then an optimalization study must be carried out in order to have an optimal solution for a water management strategy.
Chapter VI discusses erosion and sedimentation problems in the canal system, by means of sources of sediments and protection works against the local scouring.
Chapter VII covers a cost benefit analysis for this

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upgrading work based on the internal rate of return of 15%. (RUN 04).

Chapter VIII contains the conclusions and recommendations for improvement of the water management system of the Barambai-Seluang-Belawang-Muhur unit in particular, and for new lowland development and other upgrading activities in general.

References are given after chapter VIII.

Annex 1 briefly describes the theoretical background of long waves and the formulae used for calculation of transport of dissolved matter.

Annex 2 describes the PENPAS computer program and its facilities.

Annex 3 describes about Hydraulic Levelling in Swampy Areas which was also presented during the Third Congress of the Asian and Pacific Regional Devision (APD) of the IAHR (August 1982, Bandung, Indonesia).

Annex 4 gives examples of the input and output files of the alternative Run04.

Annex 5 presents Hjulstrom's graph related to the erosion and deposition of the particles.

2 PROBLEM DEFINITION AND ALTERNATIVES FOR THE KOLAM SYSTEM

2.1 General

Large parts of coastal areas in Indonesia, especially in Sumatra, Kalimantan and Irian Jaya are swampy and are subject to tidal influence. More than half a century ago, a few spots of swamps close to river mouths in Sumatra and Kalimantan were reclaimed by Banjarese and Buginese people, who originate from South Kalimantan and South Sulawesi. They can be regarded as lowland specialists. Through centuries they developed skills and experience for lowland reclamation and agriculture.
In 1950 the Government of Indonesia started reclamation of lowlands. The concept of the Banjarese and Buginese people was taken as starting point and adjusted towards reclamation on larger scale.
At present, four types of tidal irrigation/drainage systems in lowlands in Indonesia can be distinquished (see figure 2).

Type I:
This type is the oldest system, applied by the Banjarese and Buginese reclaimers. According to their experiences, it is possible to reclaim the backswamps (located behind the river levees) by connecting these swamps with a tidal river.
During low tides the canal drains stagnant toxic water to the river, while during high tides fresh water enters the system and can be conveyed to the field. However since the canals are narrow and shallow, the effect is only significant for a few kilometers from the river.
Each canal serves about 40 ha. farm land, so only a fringe of a few hundred meters along the rivers can be developed in this way.

Type II:
This type was built for the governmental reclamation projects in the period 1950-1970.
The main purpose for construction of the main canal was navigation. Therefor this canal connects two big rivers in South and Central Kalimantan. Later this navigation canal was also used for irrigation and drainage of the swampy areas between these rivers. Unfortunately, in the middle part deadlocks developed, caused by the tidal conditions at both ends of this canal. Here the tides almost have the same phase. In the deadlock area the current velocities are nearly zero, so sedimentation easily occurs.
At present speedboats are causing extra problems. Their waves erode the sides of these canals [7].

The person who made this design was the late Mr. Pangeran Noor, who at that time was the Minister of Public Works. Some 60,000 ha of lowlands were reclaimed in this way.

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Type III:
This rather sophisticated type was designed by the Bandung Institute of Technology.
Originally, the irrigation and drainage system were separate. Simple automatic gates were used to regulate the water flow. Practice however taught that the gates were often blocked by organic debris or logs and hampered a proper use of these structures. Hence these control structures are no longer applied. Also the idea of a separate irrigation/drainage system was left in this development phase.
This type was mainly applied in South Sumatra.

Type IV:
This type was designed by the Gajah Mada University, Yogyakarta.
In this system the design is based upon applying low cost technology, and was derived from the irrigation and drainage system of the Banjarese and Buginese people. The idea is that tidal irrigation can be realized by overtopping the embankments of the tertiary canal. Therefore the canals were dimensioned on basis of both tidal irrigation and tidal drainage requirements.
In this so called "fork system" a short primary canal splits up into two or more secondary canals, each ending in large ponds, called kolams. Tertiary canals run at regular distances (about 400 m) perpendicular form these secondary canals. There are no structures applied, so the entire system is in open connection with the river.

This type was especially designed for taking into account the problems related to the occurence of potential acid sulphate soils (acidification of the soil).The ponds (kolam) were thought to have the following functions:
- to store the leaching product during high tide;
- to increase the outward flow in the primary and secondary canals during the low tide.

This type was introduced in 1970 and up to now 200,000 ha of swamp areas in South/Central Kalimantan have been reclaimed (see figure 3).
Despite the construction of kolams, nowadays the inhabitants still face big acidity problems. Actually, only part of the acid water, which originate from the sawahs, discharges into the river. The remaining water stays at the tail of the secondary canal.
Research from the past, focussing at a proper functioning of the kolam system, show that a very large kolam area is needed ([6], [13] and [15]).

The above described situation about type IV is particularly valid for the Barambai-Seluang-Belawang-Muhur unit, one of the tidal irrigation and drainage systems in South Kalimantan. This unit has been choosen as project area for this study.

2.2 Acid sulphate soils

On many coastal plains in the tropics, for example in Indonesia, large tracts of land are poorly productive or entirely unsuitable for agriculture because of acid sulphate soils. Except for their adverse acidity (pH 3.5 - 4.5) and its related toxic effects, acid sulphate soils have many characteristics favourable to wet-rice cultivation.
The soils are naturally hydromorphic. The topographic and hydrologic setting is normally suitable for establishing paddies. Moreover, acid sulphate soils are generally well supplied with plant nutrients, partly because of relatively high contents of 2:1 clay minerals and organic matter. So, it is not surprising, therefore, that soils with moderate acid sulphate conditions are often used for rice growing.
On the other hand, the seemingly favourable land type for rice in most acid sulphate soils areas often led to injudicious reclamation projects that ended in total failure as a result of strong acidification.
Potential acid sulphate soils occur in tidal lowlands where they have high levels of pyrite, low levels of bases and produce strongly acid sulphate soils when pyrite oxidises into sulphuric acid after drainage.
Under this condition, the range of crops that can be grown is severely restricted and yield will be very low.

Development of acid sulphate soils following drainage.
Acid sulphate soils develop where the production of acid exceeds the neutralizing capacity of the soils, so the pH value drops to less than 4.
Potential acid soils become acid as a result of drainage. Pyrite is the most important sulfur mineral in marine sediments. The following overall reaction describes complete pyritization of ferric oxide :

$$Fe_2O_3 + 4\ SO_4^{--} + 8\ CH_2O + 1/2\ O_2 \longrightarrow 2\ FeS_2 + 8HCO_2 + 4H_2O$$
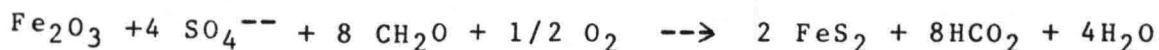
The main factors that influence the formation and accumulation of pyrite are :
-sulphate (SO_4^{--}) present in sea or brackish water;
-iron containing minerals present in the sediments;
-organic matter (CH_2O);
-sulphate reducing bacteria, which are practically always present;
-an anaerobic environment;
-limited aeration for oxidation of all sulfide to disulfide;
-time required for formation of pyrite, this time is in the order of magnitude of decades to centuries.

Pyrite is stable only under anaerobic conditions.
Drainage allows oxygen to enter the soil and pyrite is then oxidised,generating sulphuric acid. The reaction of pyrite with oxygen is a slow process.
Overall, the oxidation of pyrite can be represented by the equation:

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$$Fe\,S_2 + 15/4\,O_2 + 7/2\,H_2O \longrightarrow Fe\,(OH)^3 + 2\,SO_4^{--} + 4\,H^+$$
solid   dissolved   colloidal   sulphuric
pyrite   oxygen   iron   acid
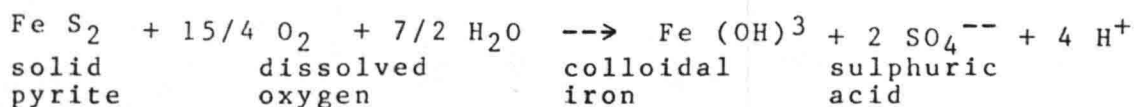
This oxidation of pyrite can take place only at pH less than 4. If it does occur, rice culture will be affected directly by H^+ ion.

The description of the oxidation of acid sulphate soils is probably available but the mathematical or chemical model most likely not. The undertaken research work is as yet inadequate for this purpose, especially with regard to the theoretical foundations. This is chiefly due to the fact that not only extremely diversified and complex chemical processes occur but also very many microbiological ones.

Chemical constraints

As the pH drops rice is affected directly by H^+ ion. Young rice seedlings and older plants growing in acid conditions suffer from small amounts of dissolved aluminium. The surface horizons of most acid sulphate soils contain a harmful concentration of soluble aluminium. Rice suffer also from iron toxicity if dissolved iron exceeds 300 to 500 ppm.

Improvements

Several measures can be mentioned for improving this acidity problem :
- application of substantial amounts of lime or other chemical materials in order to neutralize the sulphuric acid;
- intensive drainage of the soils to achieve maximum oxidation of pyrite combined with removel of toxic elements by leaching;
- limiting the pyrite oxidation and inactivating the existing acidity by maintaining the ground water table as high as possible;
- flushing the soils with fresh water to remove the noxious elements, by tidal irrigation on the sawahs or by infiltration in the subsurface soils.

From these measures it can be concluded that the improvement lies in the field of water management.

2.3 The Barambai-Seluang-Belawang-Muhur unit

This unit consists of four small schemes:
- Barambai (4,200   ha)
- Seluang   (3,500   ha)
- Belawang (5,400   ha)
- Muhur   (2,300   ha)

These projects are located in South Kalimantan, 30 km North of Banjarmasin, the capital of South Kalimantan, on the left bank of the Barito river between navigation canals (Anjir) Marabahan and (Anjir) Serapat, both connecting the Barito river with the Murung river (see figure 4).

The soils in this unit originate from marine sediments, which are acid sulphate or potentially acid sulphate soils.

All four schemes were constructed following the fork type design (type IV), with kolams at the upstream end of the secondary canals.

2.4 Problem analysis

In the project area (Barambai-Seluang-Belawang-Muhur) a major part of the rice field levels are higher then high water spring tide, it means that only a small part of the area can be irrigated by tidal movements. So, the main function of the canal system is for drainage.
In the existing condition, it is in open connection with the main river, without any regulating structure. Tidal damping is very small in the system (10-15%). It means tidal influence is large in the primary and secondary canals. But due to bad maintenance of the tertiary canals (friction and small effective internal storage area) there is only a small tidal intrusion in the tertiary canals.
This condition causes insufficient refreshment of the canal water.
Before reclamation there were no problems with toxicity or acidity in this area. After reclamation, however, due to the drainage of this area, acidity occured, because of the following process:
In the dry season, when there is a low groundwater table, air penetrates the soil. Thus there are favourable conditions for oxidation of pyrite, leading to a lowering of the pH of the soil. At the beginning of the rainy season stagnant water occurs on the fields, due to low field levels and bad maintenance of the tertiary canals, causing acidification and toxification of the soil and water.
This water, which is mainly acid, flows into the canal system. Because of an insufficient flushing capacity, accumulation of acidity will occur, leading to pH values of 2.5-3.4 for canal water.
The first yields after reclamation were satisfactory (about 3,000 kg/ha), but in later years yields dropped to as low as 500 kg/ha due to acidification of the soil. The cause of this is that the water inside the canal system is very acid.
Besides that problem, the canal systems (primary and secondary) at some places are decreasing in depth; then accessibility for local boat is limited to period of high tide. This is due to the difference in bed level between secondary and tertiary canals: the tide has eroded the downstream part of the tertiary canals. This eroded material has been deposited somewhere in the secondary canals. Also slips of the vertically excavated side slopes of the secondary canals have made the canals wider and reduced their depth.
For upgrading purposes, the following considerations have to

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be taken into account :
- low cost technology, if possible without any regulating
  structure;
- refreshment of the canal water in order to improve the
  quality of water;
- accessibility for local boats related to the development of
  the area;
- erosion problem in the canal system related to the
  maintenance cost and morphological changes.

2.5 Alternatives for the Barambai-Seluang-Belawang-Muhur unit

The kolam system was designed with the purpose of enlarging
the outflow of acid water from the canal system. As reality
shows that kolams are not so successful as was expected,
other measures are required. The general idea is to
construct a kind of flushing canal. Basic idea behind the
flushing canal is, that because of the natural gradient of
the Barito river, a net outflow from this canal via the fork
system into the river can be realized. In this way the acid
water will be pushed out gradually from the system.
For irrigation or leaching purposes a simple pumping system
may be used.
Besides, the flushing canal can also be used for navigation
purposes.

For the flushing canal several options can be mentioned:
1. A lateral flushing canal, which starts upstream of the
   project from Barito river, connecting the upper ends of
   the secondary canals of all four schemes. The canal will
   debouch into the Barito river at a downstream point via
   the right side secondary canal of the Muhur project.
2. The same lateral flushing canal, but through the
   alignment of the unconstructed leftside secondary
   canal of the Muhur project. This alternative will require
   a somewhat longer canal than alternative 1.
3. The same lateral flushing canal, but debouching into the
   middle part of the "Anjir Serapat" navigation canal. The
   basic idea of this alternative is to improve the deadlock
   problem in the "Anjir Serapat".

The lay out of alternatives 1,2 and 3 are presented in
figure 5.

4. A lateral flushing canal with interupted connections
   (see figure 6).
   The idea is that by introducing different tidal
   propagations in the system, a net flow pattern will be
   created as indicated in the following sketch:

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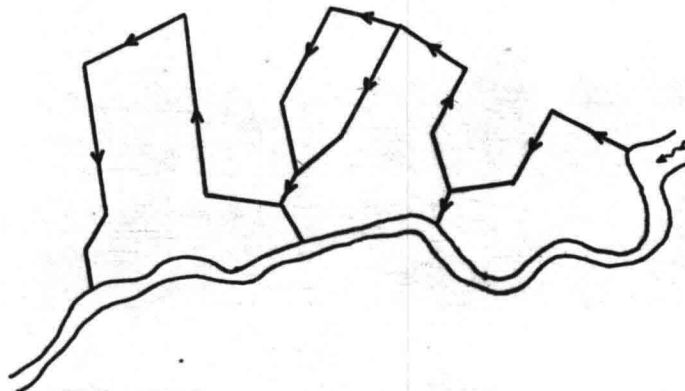
Average situation

Figure 2.1

Besides above alternatives, the following alternatives may be developed for leaching the soil:

5. A system where the irrigation and drainage canals are separated in a single unit or in the whole system.

- a single unit

irrigation canal

drainage canal

regulating structure
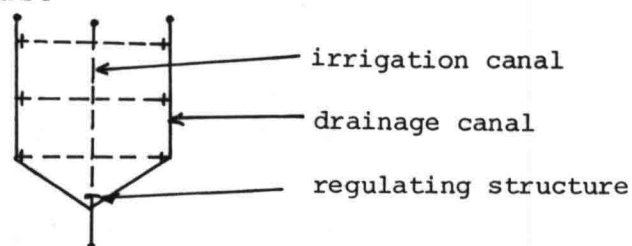
- the whole system
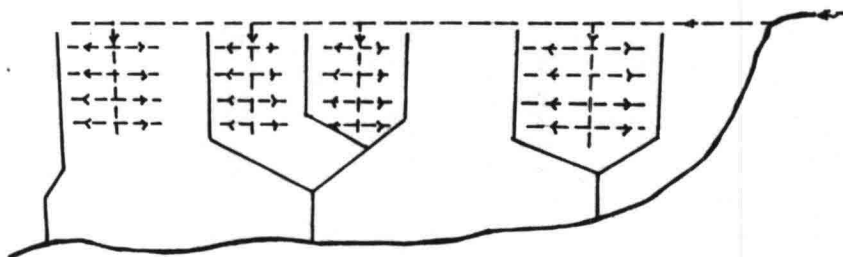
Because of limited time, alternative 1,4 and some modification of them can be studied in detail only.

2.6 Objectives of the study

The objectives of this study are:
- To learn about hydraulic phenomenae in the existing Barambai,Seluang-Belawang schemes, through a mathematical model (included calibration) that describes the water quantity and water quality (acidity) in the schemes;
- To study alternatives for the kolam system of the Barambai-Seluang-Belawang-Muhur unit, in order to improve the water-management system (reduce acidity). This improvement is related to the intensification and upgrading programme;
- To generate general alternatives related to the development of the whole area between the Anjir Serapat and Anjir Marabahan.

3 DATA COLLECTION AND EVALUATION

3.1 General

The kind of data needed for the calculations depend upon the model purpose. Related to the upgrading of existing lowland schemes and monitoring of the Barito river, several activities on data collection have been executed in the past. Unfortunately, for each river system these activities could not be conducted at the same level, due to shortage of funds.

For the Barito river, three data collecting surveys were executed in the past:
- A hydrometric survey, which only concerns hydrometric data collection of the main river. This survey was executed by the Institute of Hydraulic Engineering (Puslitbang Pengairan) Bandung in February and March 1985;
- A hydrometric survey of the Barambai scheme, which was executed in March 1986, in the framework of the preparatory research for the Lowland Development Symposium in Indonesia (August 1986);
- A hydrometric survey of the Seluang-Belawang-Muhur scheme related to the upgrading programme of the water management of this scheme.

Hydrometric data consist of measurement of water levels, discharges, cross-sections and acidity. Besides, topographical surveys were executed in the areas of Barambai, Seluang, Belawang and Muhur and in the alignment area of the proposed flushing canal. All levels are related to the Project Reference Level (PRL), which has been defined as +0.00 m.

However, in order to set up a mathematical model of an area, in this case the Barambai-Seluang-Belawang-Muhur unit, which includes part of the main Barito river, all levels have to refer to one reference level. There are actually two reference levels in this system, being:
- The Barito river system;
- The Barambai-Seluang-Belawang-Muhur system.
They can be translated into one reference level by using Bijker's method (see 3.7) and Annex 3.

3.2 Water levels

In figure 7 the various locations are given where water level observations were carried out. At each location observations were done by means of staff gauge readings with 0.5 hour time interval. For locations BAR2 and BAR3, continuous data collection were quoted from the Automatic Water Level Recorder (AWLR).

Tidal influences are very dominant. At the river mouth (km 0.0) the tidal range is 2.20 m. At station BAR 2 (km 37) the tidal range is 1.95 m, while at station BAR3 (km.82) it is 1.60 m. The water level data will be used for boundary conditions or as calibration parameters for the model.

3.3 Discharges

To collect discharge data in the system, velocity measurements were executed at several locations (see figure 7 and 8). The velocity data are also used for slope analyses. The measurements in Barambai unit were carried out in 1986 for one tidal cycle with an interval of 0.5 hours. These data will be applied for the boundary conditions and calibrations of the model, which will be discussed in the section about Mathematical Modelling.

3.4 Cross-section

Cross-sectional measurement locations are shown in figure 9. As explained before, not all cross-section levels were referring to one reference level. In order to have the same reference level in the whole project, a topographical survey has been done for Seluang-Belawang and Muhur schemes, then the results are refered to the Project Reference Level of the Barambai (see next part).

3.5 Levelling

As reference level the level as used by Euroconsult for the Barambai unit is applied [12], which is called the Project Reference Level (PRL).
Levelling was carried out by transfering all elevations of the Barambai-Seluang-Belawang-Muhur scheme and the staff gauge of BAR3 to the Project Reference Level. Cross-section elevations of the main river were levelled against the staff gauge of BAR3 by using Bijker's method (see 3.7).
The levels of the various benchmarks were correlated to each other (see figure 8 for the location of the benchmarks).
For observation of water levels and cross-sections, staff gauges were correlated with the nearest benchmark.

3.6 Acidity

Acidity observations were executed in Barambai, Seluang, and Muhur area and for the Barito river. pH values were measured at the locations shown in figure 8.
Since information on water quality will be used for input data as well as for calibration of the water quality model, the unit of acidity should be in accordance with the unit of the basic equations (eqs.3 and 4).

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For this purpose the pH is converted in H_2SO_4 content in gr/l.

The background is:
In general liquids are partly ionized. This means that a number of liquid molecules are split up into ions. This phenomenon also holds for water. So, in general in water there are ions, positive hydrogen H^+ and negative hydroxide OH^- ions.
It was found that if a kind of equilibrium exists, there is a kind of balance for the concentration of ions. The product of H^+ concentration times OH^- concentration is constant. Both for water and diluted solutions holds :

$$[H^+] * [OH^-] = 10^{-14} \qquad (\text{ for } 25^0 \text{ C })$$

Where : [H^-] = hydrogen ion concentration in gion/liter.
1 gion contains N ions ;
$N \approx 6.10^{-23}$ = Avogadro number

Because the atomic weight of H = 1, for hydrogen holds:
1 gion/liter = 1 gram/liter $\approx$ 1 %.

Furthermore it is defined that the hydrogen exponent:
$$pH = - \log [H^+]$$

When water is pure (destilled water) the amount of the positive ions is equal to the amount of the negative ions. From the ionic product can be learned that :

$$[H^+] = [OH^-] = 10^{-7}$$
$$\text{Thus } pH = -\log 10^{-7} = 7$$

In order to be able to compute acidity from the pH value, take for example sulphuric acid where acids are dissolved in the water and ionize :

$$H_2SO_4 \quad -----\rightarrow \quad 2 H^+ + SO_4^{--}$$

which is fully ionized in a solution.

So: 1 molecule H_2SO_4 $--\rightarrow$ 2 H^+ ions + 1 SO_4^{--} ion
or: 1 gmol H_2SO_4 $--\rightarrow$ 2 gion H^+ + 1 gion SO_4^{--}

Thus, for 1 gion H^+ , 1/2 gmol H_2SO_4 is required.
The molecule weight of H_2SO_4 = 2 + 32 + 4*16 = 98

Example:
pH = 4 ,
this means : [H^+] = 10^{-4} gion/liter
H_2SO_4 = 1/2 * 10^{-4} gmol/liter = 1/2 * 10^{-4} * 98 gram/liter
= 0.005 gram/liter

Based on this relationship, pH values were measured in the

system. By converting the pH number into H_2SO_4 concentrations in gram/liter, suitable data for the model have been obtained. Field data will be used as boundary conditions and for calibration of acidity computations.

3.7 Hydraulic levelling in swampy areas

All levels in a system are usually obtained through a topographycal survey. However for swampy areas, due to the soft and wet soils, it is hard to determine these elevations in this way, because the levels of the benchmarks are not 100% reliable.

Topographical data were available for the Barambai-Seluang-Belawang-Muhur scheme and for the proposed alignment of the flushing canal.These data have been referred to the local reference level: the Project Reference Level (PRL).
However, topographical data of the Barito river were not available.
The hardly accessible swamp areas caused several problems in the execution of topographical surveys, so the results can not be regarded as very accurate. Thus, although data are available they offer very poor information.

To overcome these difficulties, a hydraulic method has been applied in Indonesia since 1979 known as "Bijker's method".The accuracy of this method is sufficient enough for application in the model, as was proven during investigations at the Sebangau river in Central Kalimantan. Compared with the available topographical data, Bijker's method showed an inaccuracy of 10cm over a distance of 64 km [10].

Bijker's method is based on the characteristics of tidal waves. The length of tidal waves is usually very long (L†200 km for semi-diurnal tides), while the amplitude is only a few meters, thus causing a very gentle water surface slope.

Therefore a small part of this wave (1) can be considered as a straight line (see following sketch).
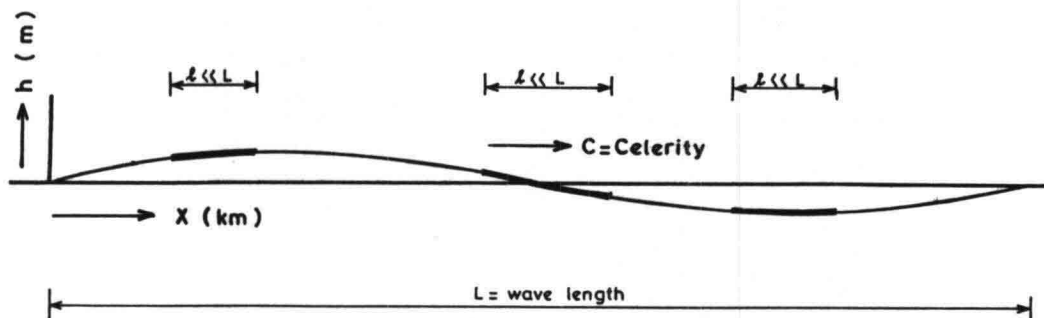
The river surface slopes change with tidal fluctuations. Whenever a surface slope is equal to zero, the elevations of

the river at point A will reach the same height as in point
B. This implies that when the height of benchmarks are
levelled against staff gauges in respective locations,
height differences of benchmarks can also be determined.
For long river sections the water surface can be considered
as a symetric curve.
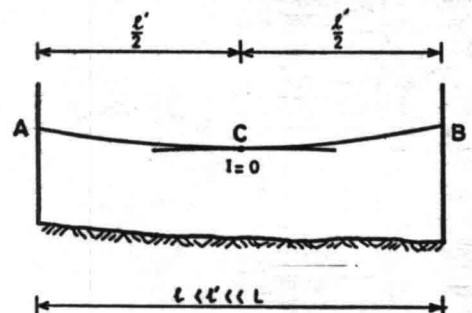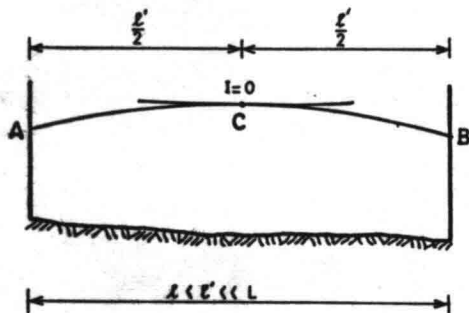
When I=0 at point C, the water level at point A is the same
as the water level at point B.

By using this principle the height differences of benchmarks
can be determined. By an increasing l the accuracy will
decrease. Besides the run-off discharge also will
influence the accuracy.

For more detailed information about this method see
reference [2] and Annex 3.

By applying this method, the following relation can be
obtained:

PRL   = + 0.00
MSL   = - 1.02
Zero staff gauge at location A = -2.58
Zero staff gauge at location B = -2.48

By using the above correlations all geometrical data from
the Barito river could be transferred into the Project
Reference Level.

4 UNSTEADY FLOW MATHEMATICAL MODELLING FOR WATER QUANTITY AND ACIDITY

4.1 General

The creation of a mathematical model which simulates water quantity and quality changes can be carried out on different levels. The aim of simulation should always be to pursue a maximum of simplicity consistent with the required degree of accuracy.
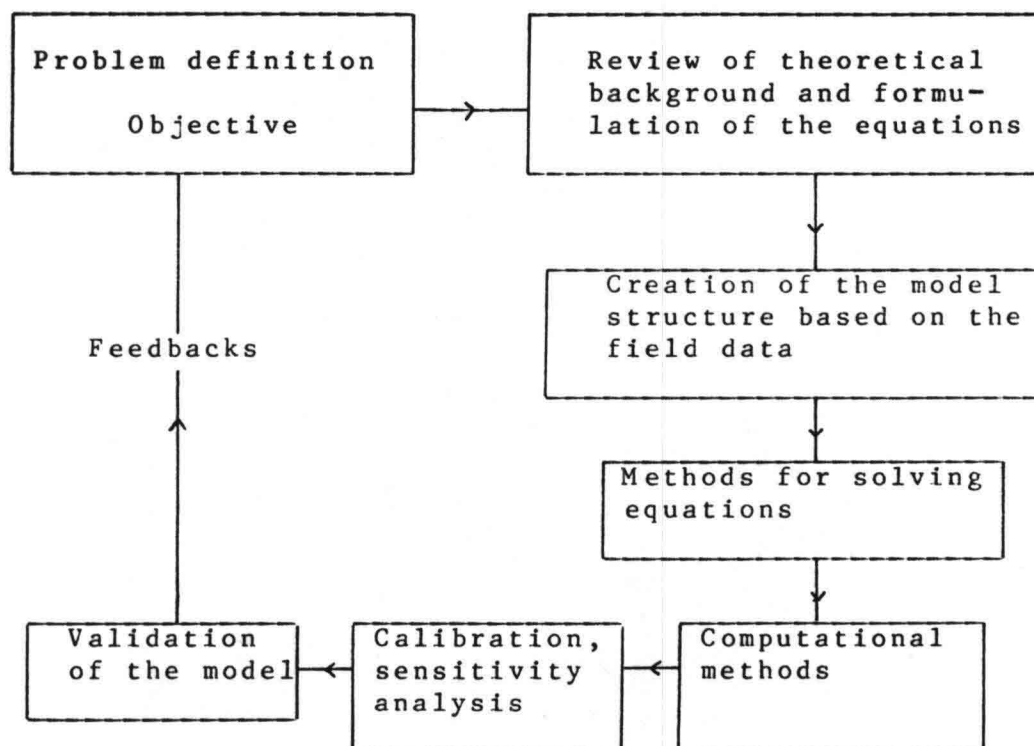The step in creating a simulation model is summarized in the following figure:

```
┌─────────────────────┐         ┌─────────────────────────┐
│ Problem definition  │         │ Review of theoretical   │
│                     │────────>│ background and formu-   │
│     Objective       │         │ lation of the equations │
└─────────────────────┘         └─────────────────────────┘
          │                                  │
          │                                  v
          │                      ┌─────────────────────────┐
          │                      │ Creation of the model   │
        Feedbacks                │ structure based on the  │
          │                      │ field data              │
          │                      └─────────────────────────┘
          │                                  │
          │                                  v
          ^                      ┌─────────────────────────┐
          │                      │ Methods for solving     │
          │                      │ equations               │
          │                      └─────────────────────────┘
          │                                  │
          │                                  v
┌─────────────┐  ┌──────────────┐  ┌─────────────────┐
│ Validation  │<─│ Calibration, │<─│ Computational   │
│ of the model│  │ sensitivity  │  │ methods         │
│             │  │ analysis     │  │                 │
└─────────────┘  └──────────────┘  └─────────────────┘
```

It shows that for a simple but realistic model, first the problem needs to be defined, followed by a clear formulation of the objectives.
The basic equations for water movement are :

Continuity :
$$b \frac{\partial h}{\partial t} + \frac{\partial(A\,v)}{\partial x} = 0 \quad \ldots\ldots(1)$$

Motion :

$$\frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + g \frac{\partial h}{\partial x} + g \frac{v\,|v|}{C\,R} = 0 \quad \ldots\ldots(2)$$

Where:

b	: width of the canal	[m]
h	: water level	[m]
t	: time	[s]
v	: flow velocity	[m/s]
x	; location	[m]
g	: gravity acceleration	[m/s2]
C	: Chezy coefficient for bottom roughness	[m1/2/s]
R	: hydraulic radius = A/P	[m]
A	: cross-sectional area	[m2]
P	: wet perimeter	[m]

For concentrations :
Continuity :
$$\frac{\partial T}{\partial x} + \frac{\partial(A\,c)}{\partial t} = 0 \quad \ldots\ldots\ldots(3)$$

Transport of matter :
For this phenomenon several transportmechanisms can be considered :
- Transportation with the average velocity, the advective or convective transport.
- Diffusion, molecular and turbulent transport
- Dispersion transport, due to the uneven velocity distribution, wind, mixing in tidal flow, etc.

Diffusion and dispersion are combined into the dispersive transport. Usually, dispersion dominates the diffusion, so the molecular diffusion can be neglected.
This leads to:

$$T = T_c + T_d$$
$$= Q\,c - A\,D \frac{\partial c}{\partial x} \quad \ldots\ldots\ldots\ldots(4)$$

Where:

T_c : convective transport
T_d : dispersive transport (negative sign because of a positive $\partial c/\partial x$; c is increasing in the positive x direction, which gives a transport in the negative x direction)

Combination of the equations (3) and (4) give:

$$\frac{\partial c}{\partial t} + v \frac{\partial c}{\partial x} - \frac{1}{A} \frac{\partial}{\partial x} \left(A\,D\,\frac{\partial c}{\partial x} \right) = 0 \qquad \ldots (5)$$

Where :
c   : concentration   [kg/m3]
D   : dispersion coeff.   [m2/s]

These equations are solved in a numerical way by a so called
finite difference method.
Annex 2 describes this numerical method and also the
linearization method, which is applied by the PENPAS
programme.

This chapter describes the mathematical modellings of water
and acidity concentrations in the hydraulic system of the
study area. For this purpose, the PENPAS program has been
used for the next situations :
- The mathematical modelling of the existing Barambai scheme,
  including model calibration on water quantity and quality
  (acidity);
- The mathematical modelling of the existing Seluang-
  Belawang scheme with some calibration on water levels;
- The mathematical modelling of the Barito-Barambai-Seluang-
  Belawang-Muhur system, as the basic model for the
  alternative with the flushing canal. Also including some
  calibrations on water levels and discharges in the main
  river;
- Mathematical modelling of several alternatives for the
  flushing canal.The results of these modellings will be
  discussed in chapter 5.

4.2 The mathematical modelling of the existing Barambai
    scheme

4.2.1 Network layout

The layout of a channel network must be specified in a
convenient form based on the following factors:
- Purpose of the computation;
- Geometry of the canal system;
- Hydrometrical data;
- Numerical factors;
- Boundary conditions
Based on these factors (mainly geometry of the canal system)

the Barambai system is schematized into a network, consisting of 21 nodes and 22 branches (see figure 9).

4.2.2 Schematization of cross-sections

For each branch an average cross-section is entered into the model. These average cross-sections are based on field data, which refer to the Project Reference Level (PRL). A maximum of 5 levels could be applied.
The tertiary canals and the field storage areas that are supplied by these canals are presented as storage points (PN).
The storage area is given as a function of the levels where the final values are obtained from calibration .
A constant inflow/outflow can be part of the input data of a storage point.
The connection between storage points and secondary canals can be realized by a hydraulic structure (weir).

4.2.3 Boundary conditions

Three types of boundary conditions are needed for the calculations of water movements and acidity:
1. Water levels, or
2. Discharges, and
3. Concentrations (for acidity distributions)

For Barambai the following boundary conditions are valid:

A Water movements
- Upstream boundary conditions:
  There is a constant inflow into the ponds (kolam) at node 706 and 806. The amount of these inflows are (Euroconsult [12]):
  0.5 m3/s (node 706)
  1.0 m3/s (node 806)
- Downstream boundary condition:
  At the mouth of the Barambai scheme the tidal fluctuation is given as boundary condition (one tidal cycle, see figure 10).
- Lateral flows:
  During the field measurement on 20-3-1986 a rainfall of 45 mm. was recorded at Station 2 from 18.00 to midnight.
  Based on this rainfall and the estimated effective area of Barambai (about 2400 ha.) lateral discharges were calculated and entered in the model.
  The final lateral discharges were obtained by calibrating the model (see Annex 4).

B Concentration
Acidity in the system was found by measuring the pH values. For the computations these pH values have to be converted into sulphuric acid contents.

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- Upstream boundary condition:
  As upstream boundary condition the measured sulphuric acid concentrations at node 706 and at node 806 are given (see figure 12).
- Downstream boundary condition:
  For the downstream boundary condition the sulphuric acid contents at the mouth of the Barambai scheme close to the Barito river is given, based on the field data (see figure 12).
  Besides, constant production perunit of time (q * c) of acid are given in the storage nodes, where q=1.0 1/s/ha and c= 0.016 g/1.

4.2.4 Initial conditions

For the start of numerical computations initial values have to be given for all nodes and branches. Normally these values are estimated. However, probable errors will disappear after some time, due to the friction and partial wave reflection in the system.

Horizontal water levels (h=0.87 m), small discharges (0.01 m3/s) and low concentrations (c=0.00005 g/1 or equivalent to pH= 6.0) in the system are used as the the initial conditions for the model whichis presented in Annex 4.

4.2.5 Time step and stability

One of the disadvantages of an explicit scheme is that the results become unstable in case the time step chosen becomes too large.

To avoid this instability it is sufficient to choose the time step in such a way that:

$$\Delta t < \frac{\Delta x}{c}$$

where c is the celerity and can be approximated by $\sqrt{ga}$.

Based on this criterion, the time step is calculated at 200 seconds.

4.2.6 Calibration of the model

In preparing the model, not all parameters are known, especially the bottom roughness and lateral storage areas for the water movement and the dispersion coefficients for concentration computations have to be estimated.

To check and improve these estimations, the model has to be calibrated. Calibration means comparing the computed results with the measured data.

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For the Barambai, the roughness and the storage areas were tried and changed during this process. The water levels at nodes 602 and 703 are compared with the computed results, besides the discharges at branch 622.

For the acidity computation, the dispersion coefficient served as a calibration parameter. The concentrations at nodes 703 and 805 have been compared with the computed results.

The results of the calibration are presented in figures 10, 11 and 13. The final storage areas, roughness and dispersion coefficients are presented in Annex 4.

When the calibration results are satisfactory, it can be concluded that the model parameters are representative for the existing situation.

From the calibration it can be concluded that the effects of the lateral storage areas and the bottom roughness play an important role.

From this existing scheme modelling results, the damping factor at the end of the secondary canals is about 10 - 12 % only. It can be seen that at the entrance the amplitude is 1.70 m and at the end of the secondary left is 1.48 m and at the secondary right is 1.47 m. The result is the average water slopes over the tidal cycle for both secondaries also very small (I= 1. - 1.5 * E-05).

The current velocities in the secondary canals are very low. Close to the kolam the current velocity is almost zero: the water moves up and down only (closed end). So, the result is that bad refreshment of concentrations will be around these areas.

Besides, due to the big difference in the bottom levels of tertiary and secondary canals, a high current velocity at the downstream part of the tertiary canals will be formed.

In fact at some locations around that confluence, erosion takes place.

So, it can be concluded that in order to have a better circulation, the tertiary canal conditions must be improved (internal storage areas) or very large kolams must be excavated [13] and [15], but around the kolams the acid still will be accumulated (closed end). It means, besides tertiary canal improvements, a circulation system must be introduced.

4.3 The mathematical modelling of the existing Seluang - Belawang schemes

4.3.1 Network layout

The layout of a channel network is presented in Figure 14 based on the field conditions. The Seluang-Belawang system

is schematized into a network, consisting of 79 nodes and 78 branches. In this model, only the water movements are computed. There are no data available on the concentration (acidity).

4.3.2 Schematization of cross-sections

The same with the Barambai scheme: an average cross-section is entered into the model for each branch which is based on field data and refered to the Project Reference Level (PRL). The same principle for the field storage areas is applied here. The connections with secondary canals are realized by a hydraulic structure (weir).

4.3.3 Boundary conditions

In this model, a water movement computation has been carried out, because no data is available on concentration/acidity distribution in the system.
For water movement, the following boundary conditions are valid:
The downstream boundary condition is the water level fluctuations at the entrance point and the closed boundary conditions are applied for the upstream ones (kolam).
There is no lateral inflows to the model , because the model is set up based on the dry season condition.

4.3.4 Time step

Based on the stability and accuracy criteria for water movement, the time step is calculated at 200 seconds.

4.3.5 Calibration of the model

The same procedures with the Barambai model for calibration has been used. The results on water level calibrations for points 112, 102, 121, 104, 128, and 138 are presented in figures 15 to 17.

From these results and from the field observations it can be stated that the same phenomenae occurs with the Barambai scheme: the damping of the tidal amplitude is small (about 15 %) and close to the kolam the current velocity is very small.
Besides, due to the big difference on the bottom level between tertiary and secondary canals, the current velocity around that confluences will be high. In fact from the field observation, it can be seen that at some confluences, local erosion occurs.
Again, for improving the water quantity and quality in the system in order to have a good circulation, tertiary canal

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conditions must be improved (as the first step).

4.4 The Muhur scheme

Based on the existing condition of the canal system, it is impossible to set up a mathematical model of this scheme. Most of the canals are not constructed yet to the design profile. So, during the tidal cycle, most of the tertiary canals are dry and during low water, upper part of the secondary canal is dry too.
In order to improve the system, the canal system must be reexcavated.
Then, in fact due to the high ground level condition, it has been decided that it is not necessary to continue the left part of the project which has not been constructed yet.

4.5 The basic mathematical modelling of the Barito-Barambai-Seluang-Belawang-Muhur unit

4.5.1 Schematization

The schematization of the present hydraulic system for the PENPAS model is presented in figure 18. The dotted line is referring to the flushing canal which will be proposed for the alternative runs.
First calculations are done for the present situation. To leave the flushing canal out of the calculations, its cross section is chosen very narrow, so that practically no flow occurs in this canal.
All the cross-section are refered to the Project Reference Level (PRL).
This is the basic model which will be used for all alternative runs of the kolam system.
This model has been run and calibrated for the water movement only, because in the main river it has a constant concentration (acidity) along the time (pH= 6.00). It is impossible to calibrate the model.

4.5.2 Boundary conditions

Based on the data of the hydrometric survey which is carried out in February 1985 [9], the following boundary conditions were applied for the analysis of the water movements:

A The upstream boundary condition
The upstream boundary condition is chosen 104 km from the river mouth where the discharges are given as a function of time during one tidal cycle.
This upstream boundary condition is presented in figure 19.

B The downstream boundary condition
The location of the downstream boundary is chosen at 36 km
from the river mouth, the place where the Barito river
confluences with the Anjir Serapat (Station BAR2) where the
water levels are given as a function of time during one
tidal cycle.
This boundary condition is presented in figure 19.

4.5.3 Lateral discharges

The field measurements show that the Negara river tributary
has an average discharge of about 200 m3/s. This lateral
constant discharge is entered in the model at node 201.

4.5.4 Roughness

The results from the slope analysis were taken for a first
estimation of the roughness. These results were converted
into the n-Manning roughness, which is applied in the model.
The first estimations were as follows:

Location	Chezy's (m1/2/s)	n Manning (-)
km.36 - km.80	70	0.022
km.82 - km.104	65	0.024

The final values of these roughness are presented in Annex
4.

4.5.5 Calibration of the model.

Due to the limited availability of the topographical data
for the Barito river, especially about lateral storage
areas, besides the calibration a sensitivity analysis is
needed to be carried out during model analysis. So, it means
that the roughness and the lateral storage areas were tried
and changed during calibration.
As a result of the calibration and sensitivity analyses,
these values were changed somewhat.
For calibration, the computed water levels at node 201 and
discharges of branch 132 were compared with the field data.
This comparison showed that some corrections for the storage
areas were necessary. The roughness and lateral storage
areas will create some effects on the damping of the tidal

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amplitude and phase of the tidal waves in the main river.
The result of the calibrations are presented in figure 20
and fig 21.

From this model, by comparing the discharges between the
main river and the irrigation/drainage schemes, it can be
stated that the effect of these irrigation/drainage schemes
on the main river is very small.

4.6 The mathematical modelling of alternatives for the kolam system of Barambai-Seluang-Belawang-Muhur unit

4.6.1 Schematization

For the schematization of this model the same approach as
in thebasic model is applied.
Several alternatives have been run with some
modifications.

4.6.2 Boundary conditions

A Water movement
The same boundary conditions with the basic model are
applied. See figure 19.
Besides, a lateral point flow has been introduced at each of
the storage points in the tidal irrigation/drainage scheme
(1.0 l/s/ha).

B Concentration
For the concentration, constant concentrations are applied
for the upstream and downstream boundary conditions (Points
100 and 612) with pH value equal to 6.0 , or the
concentration = 0.00005 g/l. Besides, a constant production
of acid perunit of time is given to the storage nodes (q*c),
where q= 1 l/s/ha and c= 0.016 g/l .

4.6.3 Initial conditions

As the initial conditions for the model, a horizontal water
levels (h=-0.40 m), small discharges (-0.01 m3/s) and low
concentrations (c=0.00005 g/l) have been given.

4.6.4 Time step

To avoid an instability and inaccuracy of the result, the
time step is calculated at 120 seconds.

About the result of each alternative run on water quantity
and water quality (acidity) will be discussed in chapter 5.

5. RESULT OF COMPUTATIONS

Based on alternatives which are described in Chapter 2, certain results can be discussed here: for all alternative, tertiary canals condition have to be improved.
So, with or without the flushing canal, these have to be improved. In the following computations, the cross-sectional areas of tertiary canals are two times of the existing areas.

5.1 The canal system without any regulating structure

-Run 01 :
The flushing canal debouches into the secondary canal of Muhur scheme. The Barambai and Seluang-Belawang do not have any connection with the flushing canal.
The layout of the model is presented in the following sketch:
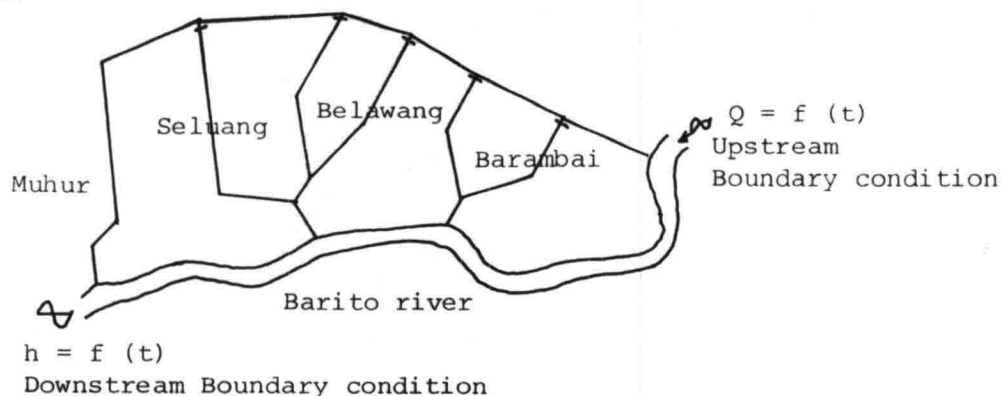
The boundary conditions

Water movement :
The same boundary conditions as was used with the basic model of the system are employed. The upstream boundary condition is the tidal discharge at branch 669 (see Figure 19).
The downstream boundary condition is the water levels fluctuation at point 100 (see Figure 19).
Besides that, the lateral drainage is defined, which is a flow into the storage areas (1 l/s/ha).

Concentration :
The upstream and downstream boundary conditions of the model (point 600 and 100) are defined as constant concentrations over the tidal cycle, where c= 0.00005 gr/1 , equivalent to pH= 6. Besides that, as internal boundary conditions are the constant production of acid perunit of time (q*c), where q= 1 l/s/ha and c=0.016 kg/m3 are given to the storage points. This assumption is based on the field measurement on the rice fields where the

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concentrations are almost constant during one tidal cycle
and the concentration is about 0.016 g/l, equivalent to pH
= 3.47 [6].
The same dispersion constants it were used with the basic
model are employed.

The schematization of the model
Based on the basic model which has been discussed, the
irrigation/drainage unit of the Barambai-Seluang-Belawang-
Muhur has been schematized (see Figure 18).
The cross-section of the secondary canals of the system
have been corrected, in order to access the local boat
traffic during low water, and all levels have been
related to the Project Reference Level (PRL).

The time step
For this run, Δt = 120 seconds has been chosen.

The stationary condition
By applying Δt= 120 seconds, the water movements inthe
system will be stationary after 4 tidal cycles, but for
concentrations, much longer periods are needed. Then, after
12 tidal cycles, the concentration computations are
stationary.This condition can be seen easily by comparing
the output of each tidal cycle.

From the computational results, the following can be
mentioned:

Water movement :
-Because of the increasing of the storage areas (improved
 tertiary canals by factor 2), the damping factor of
 the tidal amplitude at the end of each secondary canals
 arebigger than the existing condition.
 For example at the Barambai scheme, tidal amplitudes at
 the end of the secondary left is 0.77 m and at the end of
 the secondary right is 0.55 m.
 In the main river the tidal amplitude is 1.70 m. So, it
 can be stated that, the effect of the friction and the
 tertiary storage area is significant for damping the tidal
 amplitude.

-In the Muhur scheme the average discharge over the tidal
 cycle is 2.35 m3/s. The water from the flushing canal only
 flows into this secondary canal.

-Maximum current velocity
 At some places around the confluence of tertiary and
 secondary canals the current velocity is quite high. In
 the Barambai, the maximum current velocity in the primary
 canal is $\approx$ 0.60 m/s
 At some places in the Barambai, Seluang, Belawang and

Muhur, at the confluences of tertiary and secondary canals
, the current velocity is higher than 1.0 m/s.
This is due to the high differences in bottom levels
between secondary and tertiary canals. To avoid local
scouring around these locations, the tertiaries have to be
deeper.
In the secondary canals the current velocity is lower than
0.30 m/s and close to the kolams the current velocity
is almost zero.

Concentration :
-The Barambai: In the secondary canals, the concentrations
 vary between 0.0004 - 0.0050 g/l; then, in the tertiary
 canals, between 0.0007 - 0.0058 g/l.
-The Seluang-Belawang: in the secondary canals, the
 concentrations vary between 0.0062 - 0.0117 g/l and in the
 tertiary canals between 0.0089 - 0.013 g/l.
So, if they are converted into pH values, it means that pH
values in the secondary canals are 3.62 - 5.08 and in
the tertiary canals are 3.60 - 4.84.
The most important thing is in fact that at the end
of secondary canal (kolam) the water only moves up and
down, with the current velocity almost zero. So, around
these areas, bad refreshment of acid water will occur
(Point 304, 504, 518, 705, 805, 521 and 865). In the long
time scale, because of this condition, the same water
quality problem will reappear. In order to avoid that
problem, a circulation of the canal water is needed.
In the Seluang-Belawang and Muhur schemes, in the tertiary
canals, the concentration varies between 0.0013 - 0.0134
g/l or equal to pH 3.56 - 4.57. These high concentration
is due to the fact that the tertiary canals bottom
level are rather high compared to the tide level.
So, during one tidal cycle, the dilution processes in the
system are less intense. To improve this problem, the
tertiary canals have to be deeper.

-Run 02 :
A lateral flushing canal is introduced, connecting the
upper ends of the secondary canals of all four schemes:
This is an open canal system without any regulating
structure (see Figure 5). Tertiary canals are 1.0 m deeper
than Run 01.
The layout of the model is presented in the following
sketch:
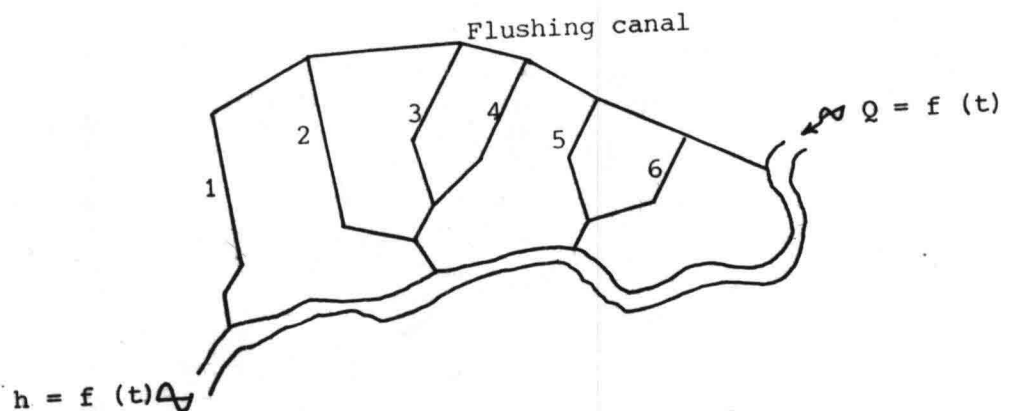
The boundary conditions
The same boundary conditions as with Run 01 have been
applied, both for water movement and concentration.

The schematization of the model
Based on the basic model and Run 01, the flushing canal is
designed with the constraint that the maximum bottom width
of the flushing canal is 20 m. This is due to the high
compensationcost for land. Besides that, the canalsystem
must be accessible for local boat transportation during
one tidal cycle; this requirement is related to the
development of the project area. In order to fulfil that
requirement, the minimum water depth is estimated at 1.5
m during low water.
The flushing canal has a constant cross-section with
the following dimensions :
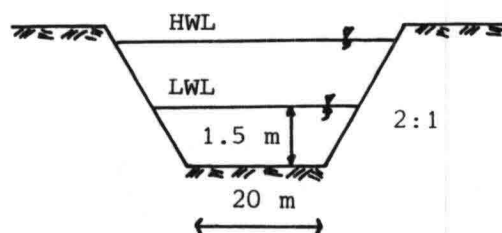
The Manning roughness n= 0.025 has been chosen for this
flushing canal. Then, the connection with each secondary
canal via a short channel with 10 m width and is located
between the existing kolam and the flushing canal.

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Flushing canal

Existing kolam
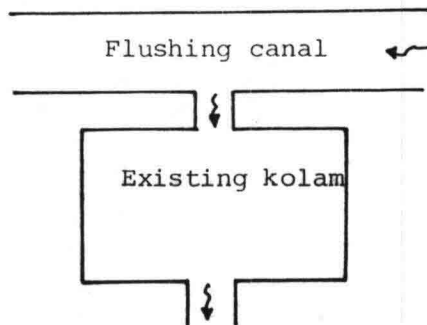
The idea behind that connection is that later in the second upgrading step, when some regulating structures may be introduced, the best places for these structures are available already.

The time step
The same time step with Run 01 is applied: $\Delta t = 120$ seconds.

The stationary condition
The same condition as with Run 01 is obtained, after 4 tidal cycles the water movement is stationary and the concentrationis obtained after 12 tidal cycles.

From the computational results, the following appear:

Water movement:
In order to have a better water quality in the canal system, a circulation of the canals water flow are needed. In this alternative, the average flow over the tidal cycle in the system is presented in the following sketch :

Q in m3/s
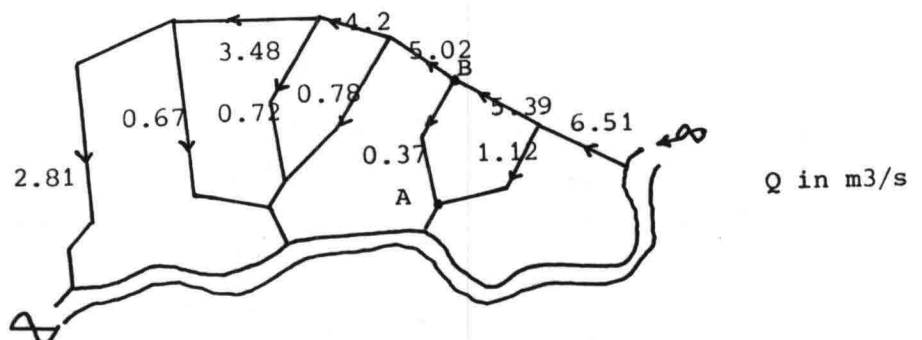
At the secondary Barambai right (5), the average water level over the tidal cycle at points A and B are almost the same. The result is the average discharge in this secondary is also small compared to other secondaries.It can be concluded that the part of the flushing canal

between (5) and (6) is useless.

Another problem is, in some secondary canals ((2),(3), (4) and (6)) a deadlock problem occur. This is due to the small phase difference between the water levels at both sides of these secondary canals.

For example at the secondary Barambai right (6), where the current velocities along the secondary canal has been plotted, it can be seen that during one tidal cycle the deadlock point remains in the same part of the canal. So, at that location, the current velocity will be very small, and sedimentation and acid accumulation will occur.
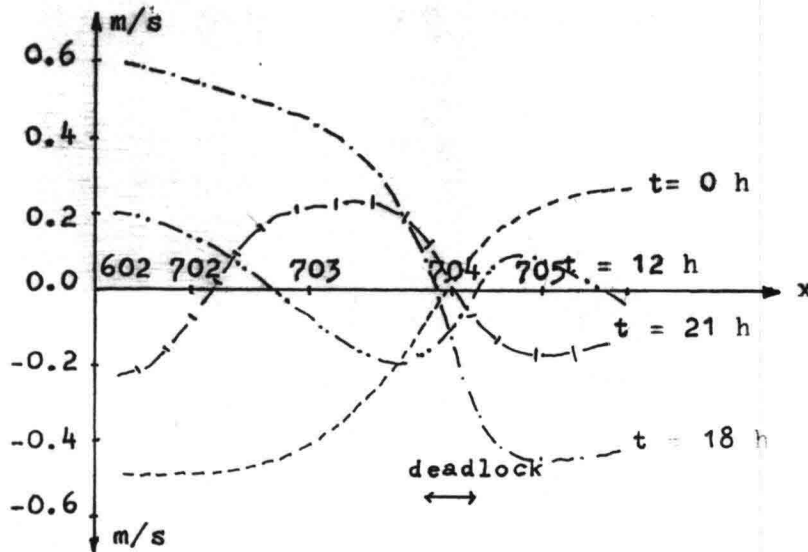
Erosion problem:

Around the entrance of the flushing canal, the maximum current velocity is about 0.70 m/s. By looking at the bottom material of the main river around this project, the mean diameter is 0.6 mm [9] and using the graph of H;ulstrom (see Annex 5), the critical current velocity for erosion is about 0.20 m/s. So, it can be stated that without any protection structure on the bottom and side banks around this entrance, it can be eroded. Finally a morphological change will take place, the canal becoming wider and wider.

The system will change completely.

To avoid this problem, a protection structure must be constructed. This protection structure will be discussed in the next chapter.

Concentration

The averaged concentration (over a tidal cycle) in the canals system are presented in the following table:

The number of the canal related to the model layout (see the previous sketch of layout):
1 : Muhur
2 : Seluang

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3 : Belawang left
4 : Belawang right
5 : Barambai left
6 : Barambai right
```

-secondary canals:

location	1	2	3	4	5	6
concent. (g/l)	0.0008 - 0.0082	0.0007 - 0.0020	0.0006 - 0.0021	0.0007 - 0.0016	0.0006 - 0.0024	0.0004 - 0.0010

-tertiary canals:

location	1	2	3	4	5	6
concent. (g/l)	0.0015 - 0.0135	0.0012 - 0.0022	0.0015 - 0.0022	0.0015 - 0.0019	0.0011 - 0.0038	0.0009 - 0.0022

note : c = 0.005 g/l is equivalent to pH = 4.0 (limit for
       pyrite oxydation).

The highest concentration is around the deadlock area.
From these concentrations, if they are converted into pH
values, it means in this system in the secondary canals pH
values vary between 4.34 - 5.08 , except Muhur and in
the tertiary canals between 3.56 - 4.73.
By comparing this result and result of Run 01, it can
be stated that the concentrations in the system are
not different much. In fact, a comparison can be made
between the mass of the fresh water which flows from the
flushing canal with the acid water which flows from the
storage nodes (simple mass balance).
In order to check that convective and dispersive transport,
the model has been run without the dispersive transport
(but unfortunately a numerical diffusion is still present).
From both computations, it can be concluded that the
physical dispersive transport gives only a very minor
effect. So, it can be concluded that, the convective
transport is dominant.
To analyse the effect of the numerical diffusion, a simple
comparison can be done between the result from the Penpas
model and the result of simple steady mass balance
computation.
For example for the secondary right Barambai:

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-The result from the Penpas model:
The maximum averaged concentration (over a tidal cycle) is
0.0010 g/l.

-The result from steady mass balance computation (without
dispersion) for dissolved matter:

$$c = \frac{c1 * Q1 + c2 * Q2}{Q1 + Q2}$$

Where: c1 : concentration of the acid water = 0.016 kg/m3
       Q1 : discharge of the acid water   = 0.68 m3/s
       c2 : concentration of the flushing canal water
            = 0.00005 kg/m3
       Q2 : discharge from the flushing canal = 1.12 m3/s

Then: c = 0.006 g/l

From this comparison, it can be concluded that the
numerical diffusion influences very much in the model.

Finally, it can be stated that the water quality in the
system will be improved, where the pH values   are higher
than 4.0 (except in the Muhur),   where below   that values
the oxydation of the pyrite will take place.
But, the problem is that by constructing that flushing
canal, in the long time scale, due to that deadlock
problem, the sedimentation and bad refreshment of the
canal water will reappear in the system.
To avoid that problem, some regulating structures must be
introduced.Then by applying a water management strategy,
deposited material and high acid concentrations can
be flushed out.But, in the case of this first upgrading
step, a low cost technology approach has to be used: this
means that using some   regulating structure will be
unfeasible. In other words we have to   study another
alternative which will fulfil that   requirement.

-Run 03 :
A lateral flushing canal with interupted connections (see
Figure 6).
The layout of this model is presented :

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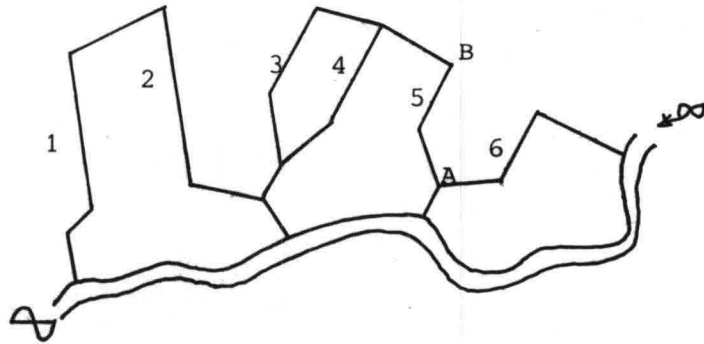
The idea behind this alternative is in fact, from the Run 02 result, that the average water levels over the tidal cycle at point A and B are almost the same. So, instead of the continuous flushing canal, an interupted flushing canal will give a beter flushing effect. It is hoped that it will create a circulation from point A to point B.

The boundary conditions
The same boundary conditions with Run 01 are applied both for water movement and concentration.

The schematization of the model
The network is the same as with the Run 01, only at the interupted parts of the flushing canal, a very high resistance or very narrow canals are introduced (0.01 m); in fact from practical point of view this is almost nothing.
The dimensions of the canal system, roughness coefficients and dispersion coefficients remain the same as with Run 02.
The time step
The same time step as was used with the previous Runs are applied; $\Delta t = 120$ seconds.

The stationary condition
The same conditions are obtained. For water movement the condition was obtained after 4 tidal cycles, and for concentration after 12 tidal cycles.
From the computational results, it can be said that:
Water movement :
The average flow distribution in the canal system is presented.

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0.53 0.29

0.39

0.82

1.99

Q in m3/s
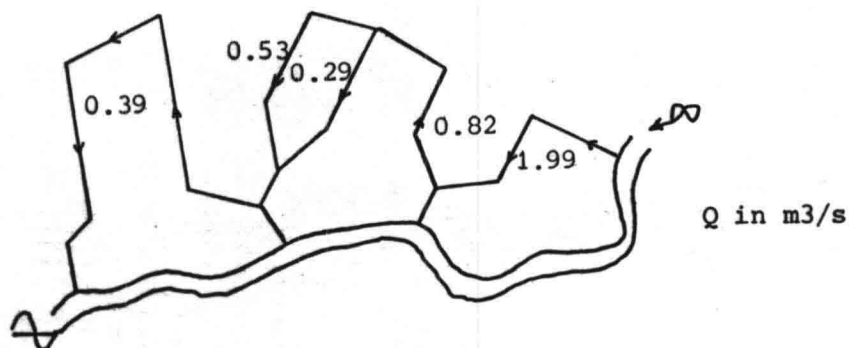
Unfortunatelya deadlock problem will occur also in the
secondary 4 (The Belawang secondary right). So, around
thatlocation, sedimentation and bad water refreshment may
occur.

Erosion problem
Themaximum current velocity around the entrance of the
flushing canal is about 0.43 m/s. From the previous
discussions, the critical current velocity for erosion is
0.20 m/s;this means that, without any protection structure
around that entrance, erosion will occur.

Concentration
Theaveraged concentration (over a tidal cycle) in the
canals system are presented in the following table:

-secondary canals:

location	1	2	3	4	5	6
concent. (g/1)	0.0012 - 0.0106	0.0007 - 0.0020	0.0007 - 0.0020	0.0008 - 0.0017	0.0006 - 0.0029	0.0004 - 0.0009

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-tertiary canals:

location	1	2	3	4	5	6
concent. (g/l)	0.0019 - 0.0135	0.0012 - 0.0024	0.0016 - 0.0023	0.0015 - 0.0020	0.0010 - 0.0054	0.0008 - 0.0020

The concentrations in the canals are almost the same with
the Run 02, except the secondary Barambai left.
Higher concentration in the Barambai left is caused
by the acid water from the right secondary.
By comparing this result and the result of Run02, it can
be stated that this alternative (Run03) will give a
much cheaper solution (less dredging cost).
But this alternative still gives a problem with adeadlock
point in one of the secondary canal.
To avoid or to minimize this problem, the next Run has to
be studied (to create bigger discharges in the canals) .

-Run 04 :
The same flushing canal layout as Run 03.
The different between Run 03 and Run 04 is in the secondary
canal dimensions of the Barambai left (5) and thesecondary
Seluang (2) and the part of the flushing canal which
debouches into the secondary canal of the Muhur. The
dimensions of these canals are increased.
The layout of this model is presented :
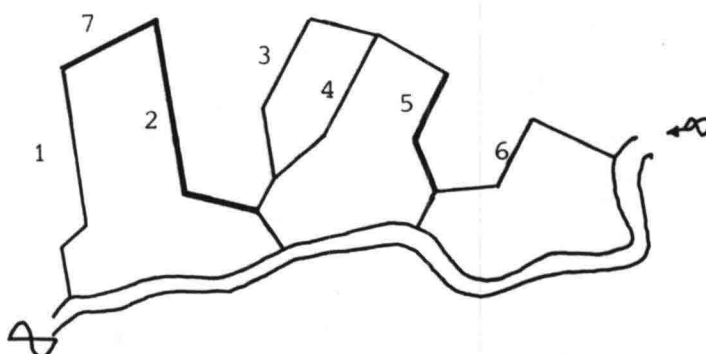
The boundary conditions
The same boundary conditions as obtained with Run 03 are
used both for water movement and concentration.

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The schematization of the model
The network of the model is the same as with the Run 03.
The width of the secondaries (2), (5) and segment (7) are
1.5 times its width in Run 03. So, some secondary canals
and part of the flushing canal are wider than was the case
in Run 03.
The time step
The same time step is used; Δt = 120 seconds.

The stationary condition
Just as with the previous runs, the stationary condition
for water movement is obtained after 4 tidal cycles and
for concentration after 12 tidal cycles.

From the computational results, the following appears:

Water movement :
The distribution of the average discharges over the tidal
cycle in the canal system is presented:
It can be seen that in the secondary (5) there is 1.61 m3/s
average flow from the main river.
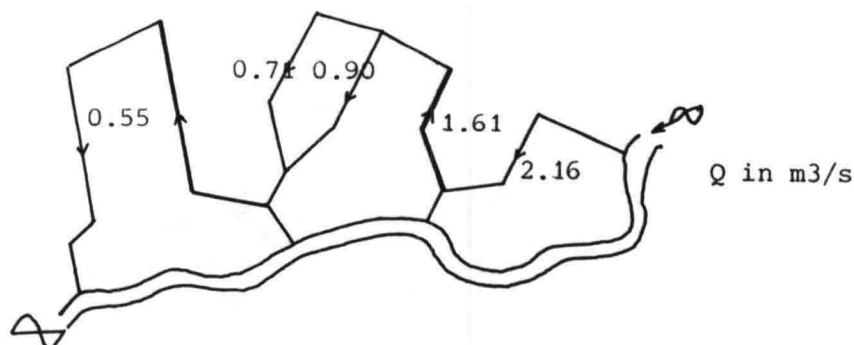
From the computation result it can be seen that along the
secondary canals, they do not have a deadlock problem
anymore. In the secondary canal no.4 the flow is almost in
one direction during one tidal cycle.
During low water, the water depth in the primaries,
secondaries and flushing canal are more than 2.0 m. So, it
can access the local boat transportation. For illustration,
the current velocities along the secondary Belawan right
have been plotted for several times during one tidal cycle:
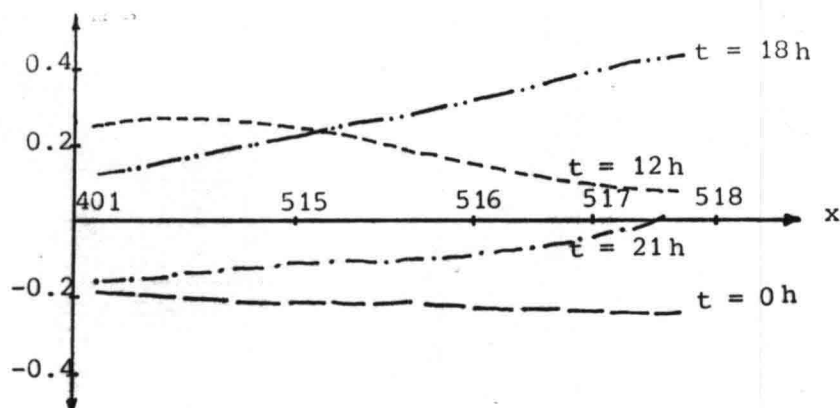
The maximum current velocity around the entrance of the flushing canal is 0.43 m/s. Again, the same erosion problem may occur. To avoid that problem, a protection structure must be constructed in order to provide against bottom and side bank erosion.

Concentration

In the secondary and tertiary canals of the Seluang and Belawang, concentrations are much lower than the result of Run 03. In Barambai, the results are almost the same as with Run 03, but the highest concentration is 0.0038 g/l or equivalent to pH=4.11

The averaged concentration (over a tidal cycle) in the canals syystem are presented in the following table:

-secondary canals:

location	1	2	3	4	5	6
concent. (g/l)	0.0011 - 0.0100	0.0002 - 0.0004	0.0006 - 0.0009	0.0008 - 0.0012	0.0006 - 0.0024	0.0004 - 0.0009

-tertiary canals:

location	1	2	3	4	5	6
concent. (g/l)	0.0013 - 0.0135	0.0003 - 0.0004	0.0008 - 0.0009	0.0011 - 0.0013	0.0010 - 0.0038	0.0008 - 0.0020

By comparing this result and the results of the previous runs, in my opinion, relatively, alternative Run 04 is

the best alternative for the first upgrading stage to
improve the water quality and water quantity in the
system (also for transportation purposes).
So, instead of the dredging work for the continuous
flushing canal, it is better to make wider secondary canals
(bigger netoutflow will give a better dilution process).

In order to have a good comparison study, some runs have
been done with some regulating structures in the system.
The results of that alternatives will be discussed in part
5.2

5.2 Some regulating structures

Here two runs will be discussed.
-Run 05
 The continuous flushing canal with a regulating structure
 (flap gate) at the intake of the flushing canal.
 The layout of the model is presented:
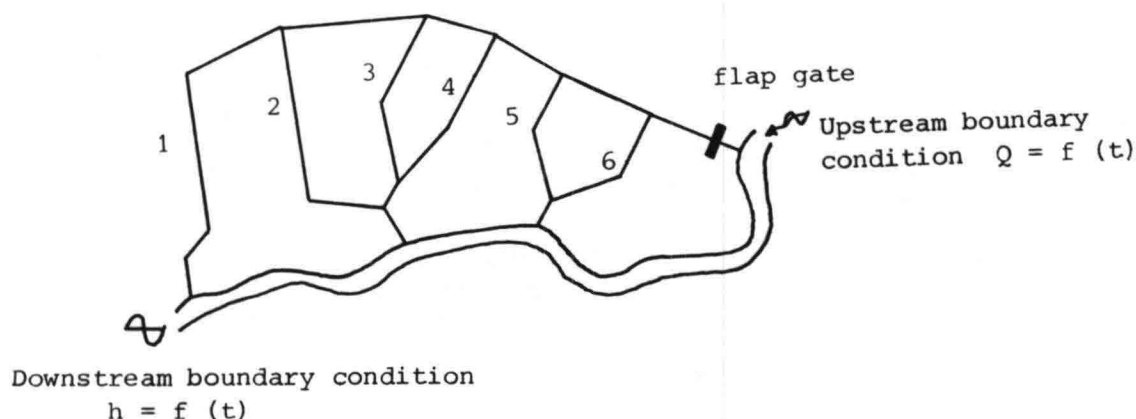
The flap gate can only operate if the upstream water level
is higher than the downstream water level.
The averaged concentration distribution (over a tidal
cycle) in the canals system are presented below:

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-secondary canals:

location	1	2	3	4	5	6
concent. (g/l)	0.0011 - 0.0054	0.0007 - 0.0015	0.0007 - 0.0015	0.0005 - 0.0013	0.0007 - 0.0013	0.0001 - 0.0008

-tertiary canals:

location	1	2	3	4	5	6
concent. (g/l)	0.0017 - 0.0110	0.0012 - 0.0019	0.0016 - 0.0018	0.0015 - 0.0017	0.0013 - 0.0043	0.0006 - 0.0016

Because of the flap gate effect, most of the influence is in the secondary right of the Barambai scheme, where the averaged outflow (over a tidal cycle) is larger than the previous runs.

For the rest of the area, the concentration distribution is more or less the same as with Run 04.

Again, it can be stated, that a higher discharge will give a better dilution processes.

From the construction point of view, this alternative is much more costly as compared with the alternative Run 04. Besides it is necessary to dredge the continuous flushing canal and an additional cost for a regulating structure is necessary.

-Run 06

The continuous flushing canal is used, but the primary canals of the Barambai and Seluang-Belawang are closed. The layout of the model is presented:
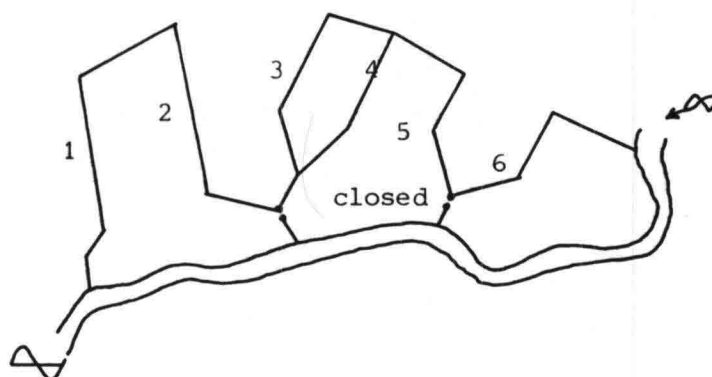
Actually, it is a parallel system with the main river.
Then, in fact the slope is very small, then the current
velocity in the secondary canals remains very low and the
concentration moves only in the upstream and downstream
directions around there. So, during high water the concen-
tration moves upstream ;then, during low water, it moves
downstream. Finally acid accumulation willoccur,espe-
cially in the upper parts.
From the concentration distibution it can be seen that
around the Barambai scheme, concentration will be high.
The maximum concentration distribution in the secondary
canals system are presented in the following sketch:

c in g/l
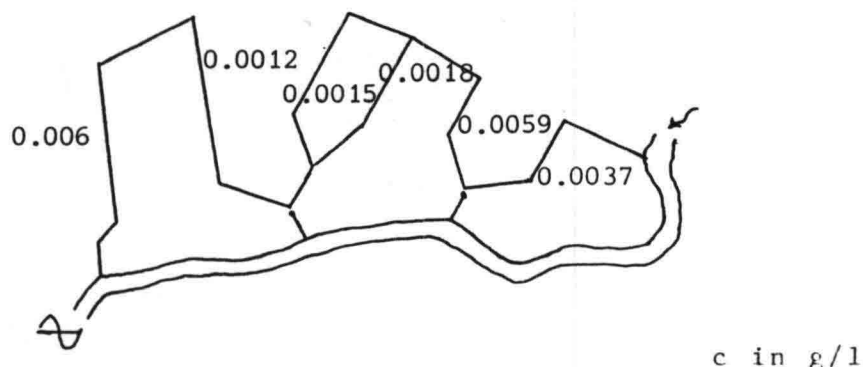
In the tertiary canals, the averaged concentrations over
atidal cycle are presented in the following table:

location	1	2	3	4	5	6
concent. (g/l)	0.0018	0.0021	0.0022	0.0017	0.0049	0.0024
	-	-	-	-	-	-
	0.0117	0.0035	0.0025	0.0021	0.0071	0.0077

Then, in the primary canals of the Barambai and the Seluang
- Belawang, the water will be fresh, because the length of
the canals are very short compared to the tidal wave length.
Another problem is that by closing these primary canals,
a discontinuity is created for local boat transportation.
So, it can be concluded that this alternative is not
feasible from the water refreshment and development of the
area points of view.

5.3 Dry season computation for Run 04A

By comparing all the alternative runs, relatively,it can be
concluded that the best alternative is Run 04 where the
interupted flushing canal is applied with some wider
secondary canals.

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The next step is to study the alternative Run 04 for other condition. The previous computation is based on the data which was collected in February 1985 . This is more orless representative for the wet season condition. So, this alternative has to be checked for another season condition (dry season).
Unfortunately, the hydrometric data on flows are not available up to now. The available data are the water levels registrations at both boundaries of the area.
Based on these data, the dry season model for the alternative has been run : Run 04A

-The boundary conditions
 Water levels fluctuation during one tidal cycle at both boundaries (see Annex 4).

-The schematization of the model
 The same as with Run 04

-The time step
 The same time step as with other runs, Δt = 120 seconds.

-The stationary condition
 Also after the same period as with the previous runs, the stationary condition is obtained.

The result of the computations are:
Water movement:
The average discharges over the tidal cycle in the canal system is presented below:
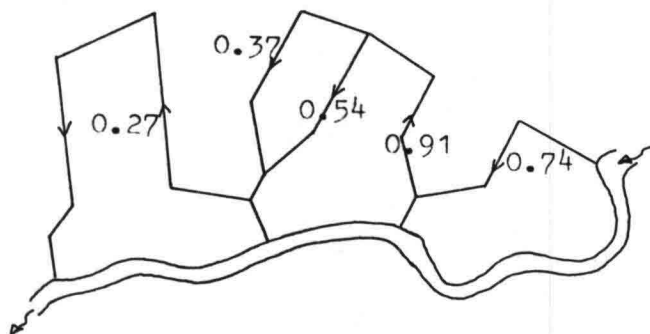
The deadlock problem occurs in the following secondary canals:
The Barambai: The secondary right between points 703 and 704;
The Belawang-Seluang : here no deadlock points occur;
The Muhur : The deadlock problem occurs between points 103 and 104.

The maximum current velocity around the entrance of the flushing canal is 0.36 m/s. Again, to avoid the erosion problem, a protection of bottom and side banks has to be

provided.

Concentration
-The Barambai scheme:
 In the secondary canals the concentrations vary between
 0.0004 - 0.0035 g/l and in the tertiary canals between
 0.0007 - 0.0043 g/l or pH values vary between 4.05 - 5.08

-The Belawang scheme:
 In the secondary canals, the concentrations vary between
 0.0004 - 0.0012 g/l and in the tertiary canals between
 0.0006 - 0.0013 g/l or pH values vary between 4.57 - 4.91

-The Seluang scheme:
 In the secondary canal the concentrations vary between
 0.0003 - 0.0005 g/l and in the tertiary canals between
 0.0004 - 0.0006 g/l or pH values vary between 4.91 - 5.21

-The Muhur scheme :
 In the secondary canal the concentrations vary between
 0.0012 - 0.002 g/l and in the tertiary canals between
 0.0012 - 0.0064 g/l or pH values vary between 3.88 - 4.61.
 Again this is due to the high level of the tertiary
 canal compared to the tide levels.

So, in order to avoid deadlock problems(sedimentation and
acid accumulation in the canals), a simple regulating
structure at the end of that secondary canals might be
constructed. The regulating structure can be simple gates
and in the dry season the regulating structure may be
operated in order to create flows in the canal (closed
partly during high water and opened during low water).
In order to have an optimal solution for flushing effect
and the deadlock problem, an optimization study must be
carried out. Because of the time limitation and availability
of data here, we can only mention this study as a subject
for further research.
So, monitoring works are strongly recommended as soon as
the first upgrading work is finished. The monitoring
works should cover the water quantity, water quality
and morphological changes. Based on these monitoring data,
an analysis can be carried out in order to have an optimal
solution for the second upgrading stage, etc.

5.4 The separate system of irrigation and drainage
This system can be applied only if the water level during
high tide is higher than the highest field level.

By using this system, besides a more reliable agriculture,
the irrigation water can be used for :
-leaching the acid soils;
-prevent further acidification of the soil.

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From the topographycal map which is available for the
Barambai scheme, it can be stated that only a small part
of the area can apply this system. For the other schemes,
from the field orientation, most of the areasare higher
than the highest tide level. Of course a detailed topogra-
phical measurement has to be done first.
Besides that, for this system some regulatingstructures
are needed and, by keeping the basic principle that in
this upgrading step a low cost technology must be applied
, this means that this alternative is not feasible for
the time being.

6. EROSION AND SEDIMENTATION IN THE CANAL SYSTEM

6.1 The sources of sediment transport

About the sources of sediment transport, no exact measurements are available in this area. Several suspended sediment loads have been measured and in fact the concentration is very low.

From the field survey and interviews with the local people, several possibilities can be mentioned related to the erosion and sedimentation problem in this area.

-Bank erosion

Due to the slips of the vertically excavated slopes of the secondary canals the canals have been made wider, which reduces their depth, so that access by boat is limited to periods of high tide.

Besides that, bank erosion is caused by waves of fast sailing speed boats (50 km/h).

In the existing condition and in the proposed system, from the numerical computations which have been discussed in the previous chapter, it is seen that the current velocity in thesecondary canals are quite low (Run 04 and Run 04A); theyare smaller than 0.3 m/s. It means that there is no erosion caused by the current velocity.

However, around the entrance of the primary canal and the flushing canal, rather high current velocity 0.60 m/s, occur. So, a local erosion will occur around the entrances and the eroded materials will be deposited in the canal system.

-Flocculation

Suspended clay in the canal system consists of flat or needle shape particles having a maximum dimension less than a few micrometer. Because of their form, large surface area and the crystal structure of the clay minerals, these particle are negatively charged on the surface. Since the particles are very small, the electrostatic forces is bigger than the gravity forces, which will keep the particle separated in suspension. From field measurement in the main river, the maximum concentration of the suspension material is 65 ppm. It is very small. But because of the chemical processes, finally these particles will be flocculated and will be deposited if the gravity forces become bigger than the electrostatic forces. This sedimentation can only occur as long as the current velocity in the canal system is very small or nearly zero. So, to avoid this problem, the current velocity must be kept not too small over tidal cycle.

-Due to the difference in bed level between tertiary canal and the secondary canal, the tide has eroded the downstreamparts of the tertiary canals (local scouring). This eroded material has been deposited in thesecondary canal.

So, it can be stated that the major part of the erosion

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and sedimentation sources are bank erosions, local scouring in the downstream part of the tertiary canals and waves of speedboats.

6.2 Bank protection and improved tertiary canals

To avoid erosion, sedimentation and morphological changes , several possibilities can be mentioned :

-To construct a protection structure at the place where the current velocity is high (> 0.3 m/s). The most important locations are around the entrance of the flushing canal and primary canals. A local material which is available can be used, for example wooden piles.

-Besides that, a suitable overgrowth (for example Rengas) may be applied also. This idea arises from the field orientation and interviews with the local people, in fact as long as that Rengas grows on the river banks, no erosion takes place. Of course the behaviour of the chosen plantation has to be studied very carefully with respect to hydraulic loads and the required maintenance.

-To reduce the speeds of the speedboats or local boats in order to reduce waves forces on the side banks.

6.3 Floating plants and debris

From the field visit and orientation it can be stated that most of the navigation canals in the southern part of South/Central Kalimantan are blocked by the floating plants or debris/logs. Of course they will influence the flow condition in the canal system and create some difficulties for navigation.
To avoid this problem, a structure or wooden guidewall can be constructed around the entrance of the primaries and the flushing canal.

7. COST BENEFIT ESTIMATION

7.1 Investment costs
Based on Run 04, about 3.969.050 m3 will be excavated. This amount consists of :

The flushing canal	=	2.983.750	m3
The Barambai scheme	=	165.000	m3
The Seluang-Belawang scheme	=	367.500	m3
The Muhur scheme	=	68.800	m3
Total excavated material	=	3.969.050	m3

An all-in price for excavation by a cutter dredger including clearing and small embankments on the boundaries of the spill area of US $ 1,-/m3 seems feasible.
Two gates are planned at the end of secondary canals Muhur and at a price of US $ 10.000,-per unit.

Construction costs :		US $
-Excavated canal : 3.969.050 m3	=	3.969.000,-
Gate 2 no.	=	20.000,-
Contingencies 10 %	=	400.000,-
Total cost	=	4.389.000,-

This investment gives benefits to a potential area of 15.400 ha. The cost is therefore approximately US $ 285,-/ha.
To improve the yields of the area, simple control structures (stoplogs) have to be constructed in order to maintain a proper ground water level. Construction of those structures, each serving about 40 ha. is estimated as Rp 2.000.000,- or US $ 1.200,- and equivalent to US $ 30,-/ha.
So, an investment cost for those structures will be US $ 462.000,-

To summarize the investment costs, the following total investment cost is obtained :

-Construction cost	= US $	4.389.000,-
-Special maintenance tertiary level	= US $	462.000,-
Total investment	= US $	4.851.000,-

7.2 Cost-Benefit analysis

In order to check if the proposed upgrading work will meet the required internal rate of return of 15 %, the following assumptions have been made.
-all four schemes will benefit from the provision of a flushing canal.
It has been assumed that the yields will increase from 0.5 t/ha to 1.5 t/ha over a period of 5 years and that during the same period land use will increase from 60 % to 80 %.
-Farmer income has been estimated as follows :
0.75 * 150 = US $ 112.5/ton.

The maintenance has been estimated as follows :
-earthworks : 10 % of investment during the first 3 years,
after that 5 %.
-structures : 10 % of investment.

Based on those estimations and assumptions, the estimated
costs and benefits are set out in the following tables 1 and
2.

Table 1. Benefits
Basis Present worth at 15 % , income US $ 112,5/ton

Project	Year	Agricultural area Total area	Increase in yield ton/ha	Factor	Present worth US $
Barambai	0	0.6	0	1.0	0
Seluang	1	0.6	0.3	.87	271.310
Belawang	2	0.65	0.6	.756	510.810
Muhur	3	0.70	0.9	.658	718.190
(15400ha)	4	0.75	1.20	.572	891.890
	5	0.80	1.50	.497	1033.260
	6-20	0.80	1.50	2.907	6043.650

Total   US $ 9.469.110

Table 2. Costs
Basis Present worth at 15 %

	Year	Investment or costs/year (maintenance)	Factor	Present worth US $
Investment -earthworks (including contingen- cies)		1.1 (3969000) = 4.366.000	1.0	4.366.000,-
-structures (including contingen- cies)		1.1 (480000) = 528.000	1.0	528.000,-
Maintenance -earthworks	1-3	437.000	2.284	998.108,-
	4-20	218.500	3.976	868.756,-
-structures	1-20	52.800	6.260	330.528,-

Total   US $ 7.091.392,-

From this cost-benefit estimation, it can be concluded this upgrading work is feasible based on the rate of internal return of 15 %.

8. CONCLUSIONS AND RECOMMENDATIONS

-To improve the tertiary canals condition is the most important thing to do (reduction of the friction), in order to have a sufficient refreshment of the water in the tertiary canal;

-A higher net outflow in the canal will give a lower concentration. This can be obtained by widening the secondary canals or to construct a flap gate in the secondary canal which will be closed during half a tidal cycle, but be careful: because of the gate the averaged flow can be reduced since half a tidal cycle the flow is interupted . An optimization study should still becarried out in order to have an optimal dilution processes.

-The interupted flushing canal with some wider secondary canals (Run04) is feasible for improvement the system of the Barambai-Seluang-Belawang-Muhur unit for this stage.
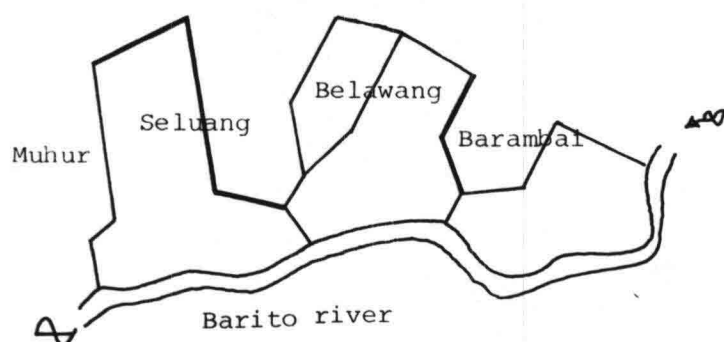
In the canal system the pH values are above 4.0 during the wet season (the limit for pyrite oxydation).

-To avoid the deadlock problem during the dry season which, will cause a sedimentation and bad water refreshment in two secondary canals, a regulating structure (gate) has to be installed at the end of the Barambai secondary right and the Muhur secondary.To optimize the gate operation if the field data on flows, sedimentation and accumulation rate are available but an optimization study must still be carried out.

-By constructing the flushing canal, support will be given to the development of the area. The canal system can access the local boats during the tidal cycle over the whole year.

-To avoid morphological change, some protection structures against erosion must be constructed, especially around the entrance of the flushing canal and primary canals.

From the side of recommendations, several things can be mentioned.

-The monitoring work is strongly recommended, as soon as the first upgrading work is finished. The monitoring work should cover the water quantity, water quality and morphological changes in the system.

-A pilot project in this unit is necessary. In this pilot project the effect of the first upgrading work can be studied in detail, included the rate of acid production in this area.

-Based on the data of the monitoring work and the pilot project results, a detail study about circulation in the canal system (water quantity and quality) should be carried out by means of a water management strategy in order to have an optimal solution for the next upgrading stage.

-To avoid that floating plants and debris enter the canal system, a structure like wooden guidewall can be applied around the entrance of the flushing canal and the primary canals.

-Related to the development of the whole area, a study about a possibility to use the secondary forest surrounding the project area as a fresh water resource should be considered.
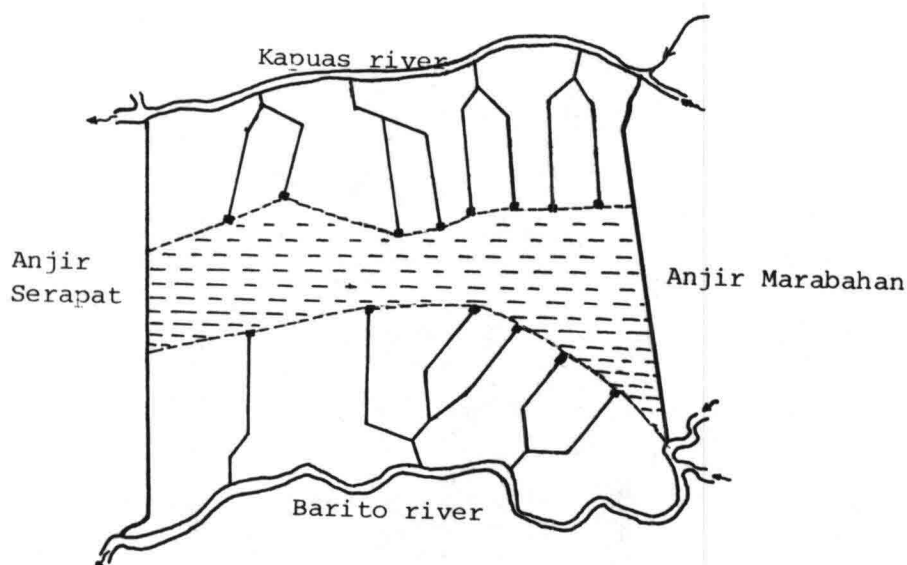
Finally, with this study result, a first insight about improvement of the tidal irrigation/drainage schemes is given and with the recommendations, it is clear that an improvement has to be done carefully based on the data with good quality.

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TABLE OF NOTATIONS

a   : water depth   [m]
A   : cross-sectional area   [m2]
b   : surface width of the canal   [m]
bt   : bottom width of the canal   [m]
C   : Chezy coefficient for bottom
       roughness   [m1/2/s]
c   : acid concentration   [g/1]
D   : dispersion coefficient   [m2/s]
Es   : modified estuary number   [-]
Frd : densimetric Froude number   [-]
g   : gravity acceleration   [m/s2]
h   : water level   [m]
L   : length of the estuary   [m]
p   : wetted perimeter of the cross-section   [m]
P   : tidal prism   [m3]
R   : hydraulic radius   [m]
t   : time   [s]
T   : tidal period   [s]
T   : total transport of acid   [kg/s]
Tc   : convective transport   [kg/s]
Td   : dispersive transport   [kg/s]
v   : current velocity, average over the
       cross-section   [m/s]
x   : location   [m]

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FIG.1 . TIDAL SWAMPS IN INDONESIA

OLD SYSTEM

Type I

CANAL SYSTEM

Type II

SEMI POLDER

CANAL

GREEN BELT · VILLAGE CENTRE · GREEN BELT

CANAL

—— Irrigation Canal
---- Drainage Canal
AERO – FLAPE Gate
1 Unit = 920 ha
Etective rice Field = 629 ha

Type III

FORK SYSTEM

1 Unit = ± 5.000 ha
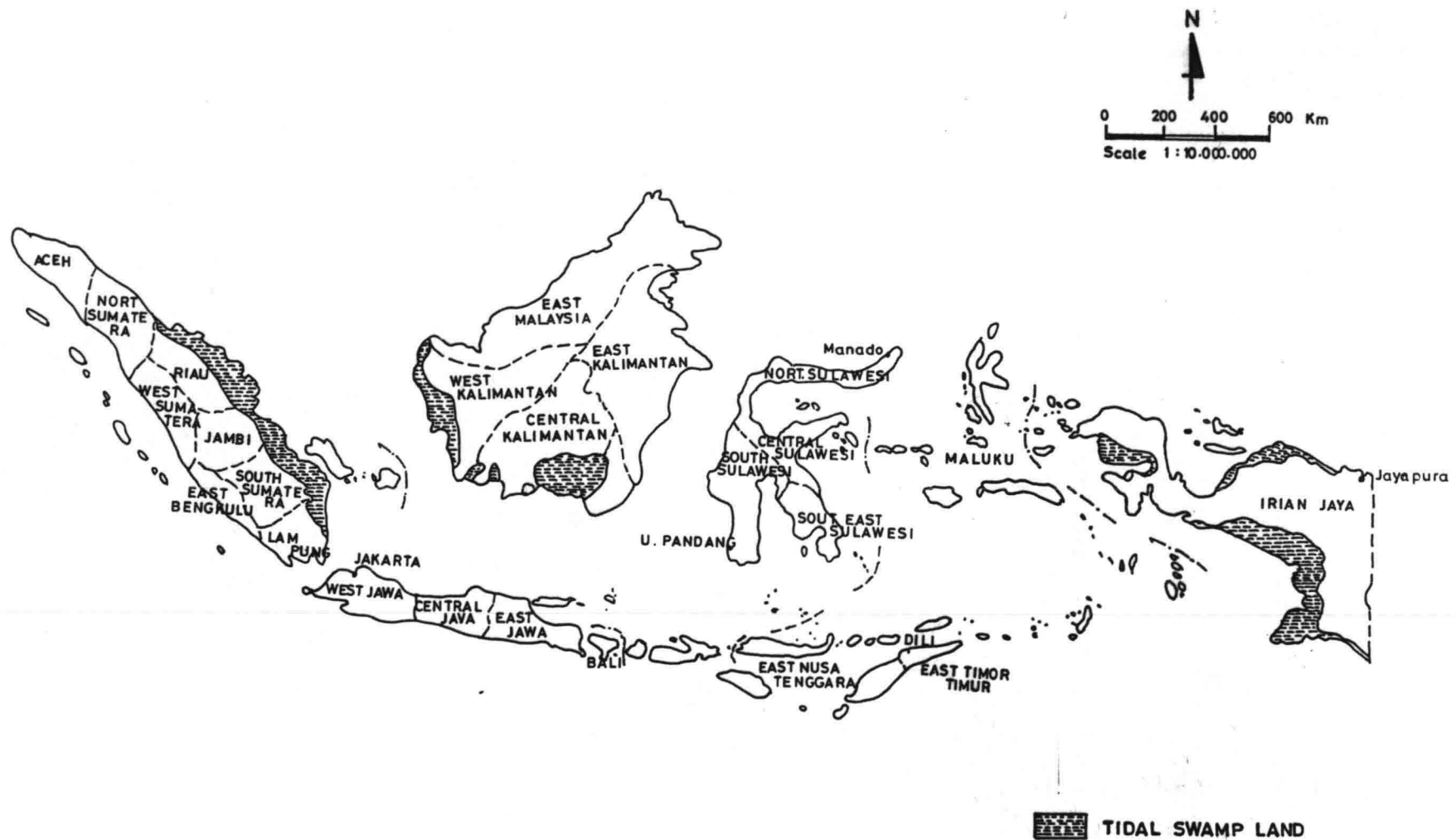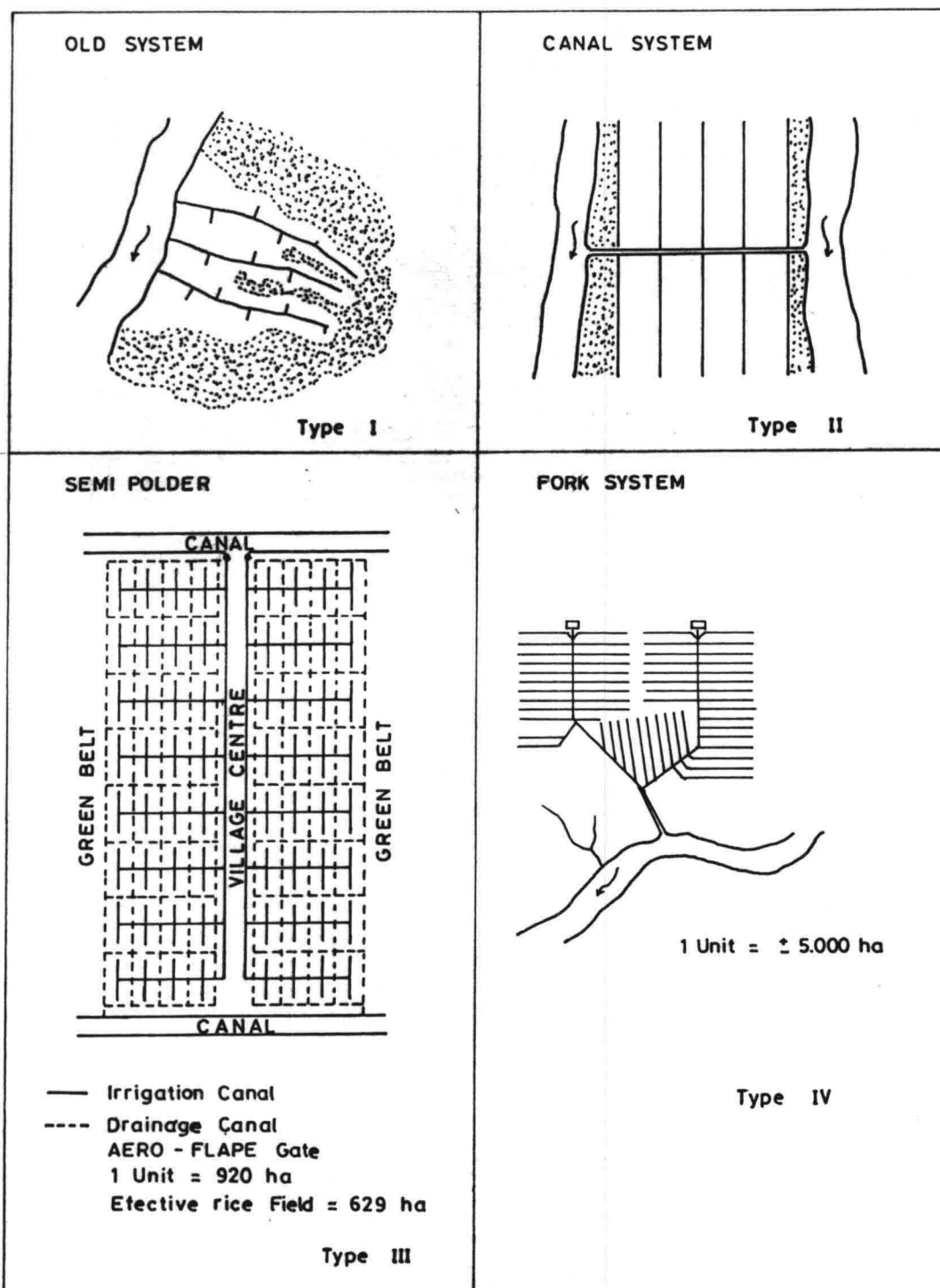
Type IV

FIG. 2 TYPES OF TIDAL IRRIGATION SYSTEM
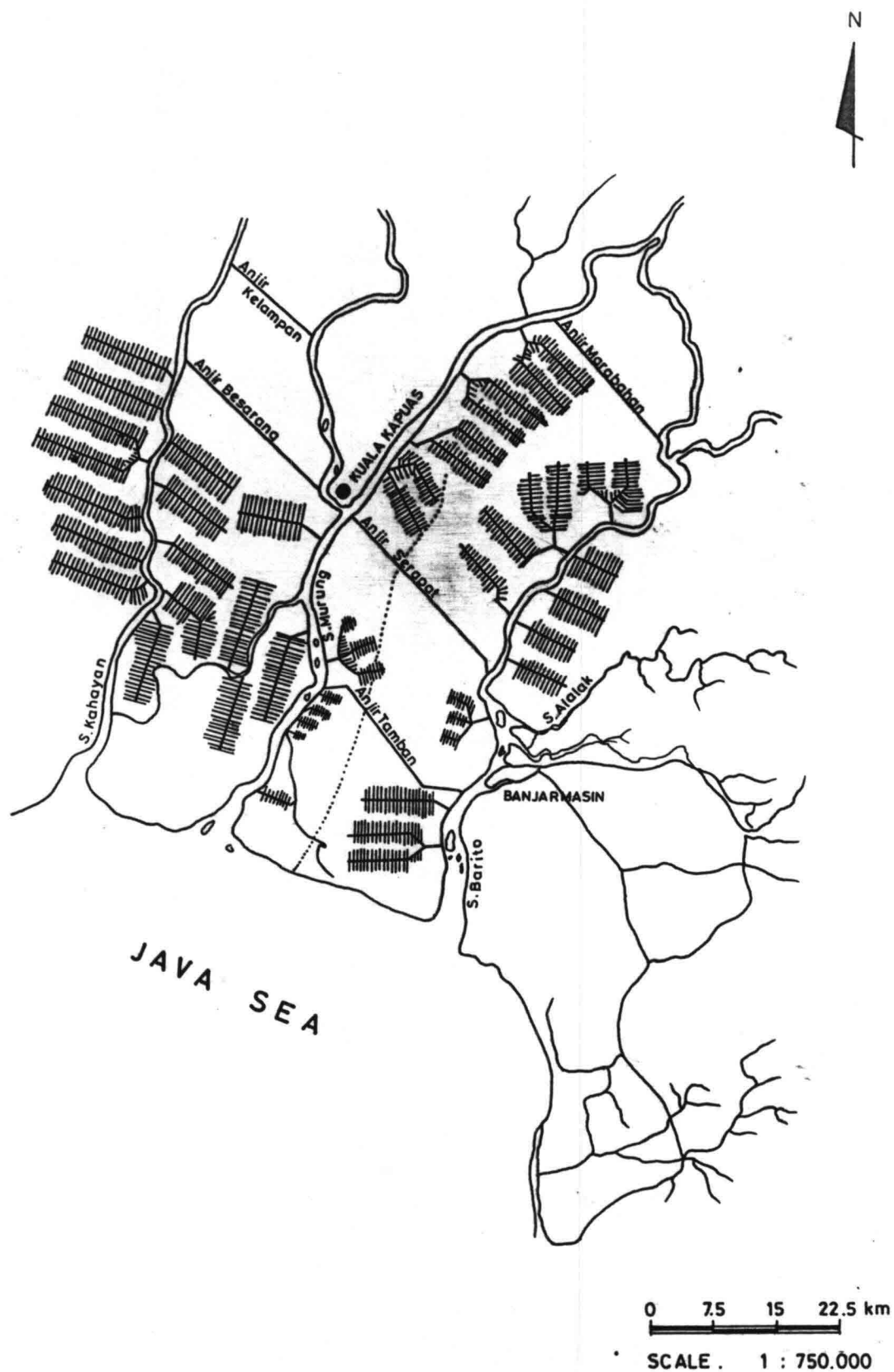
FIG.3 TYPE IV IN SOUTH KALIMANTAN
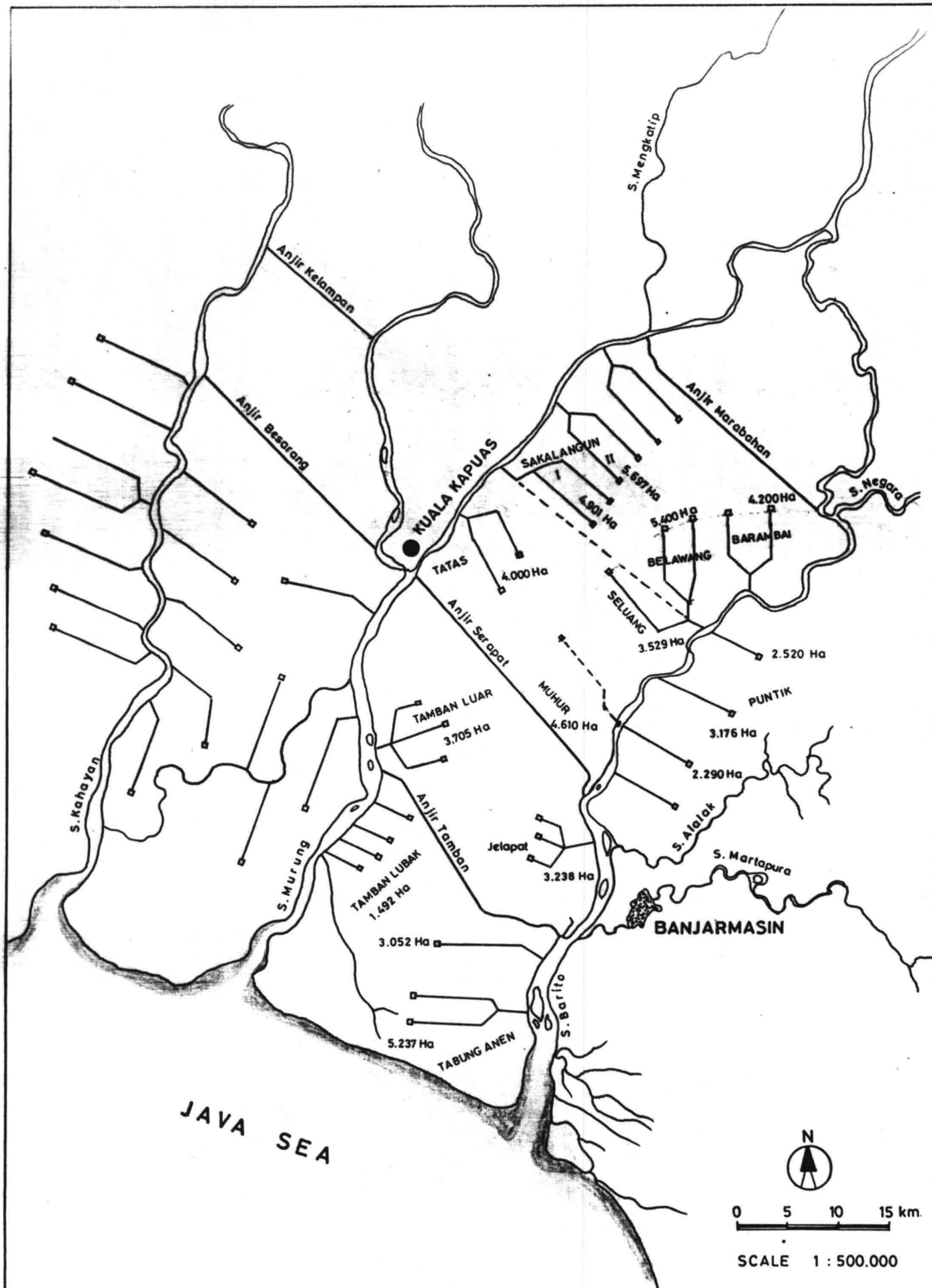
FIG. 4 LOCATION OF THE BARAMBAI – SELUANG – BELAWANG – MUHUR UNIT
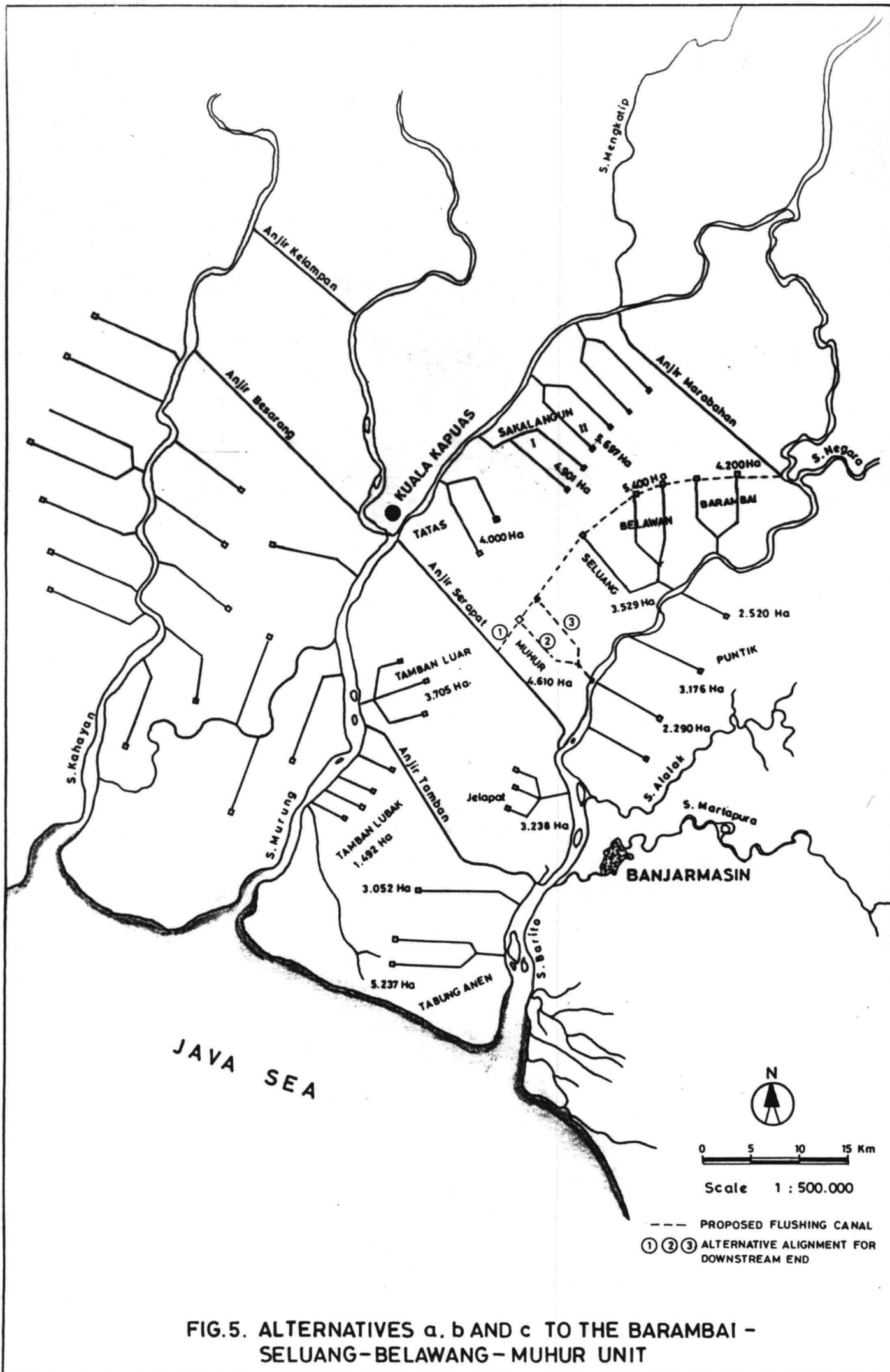
FIG.5. ALTERNATIVES a, b AND c TO THE BARAMBAI –
SELUANG – BELAWANG – MUHUR UNIT
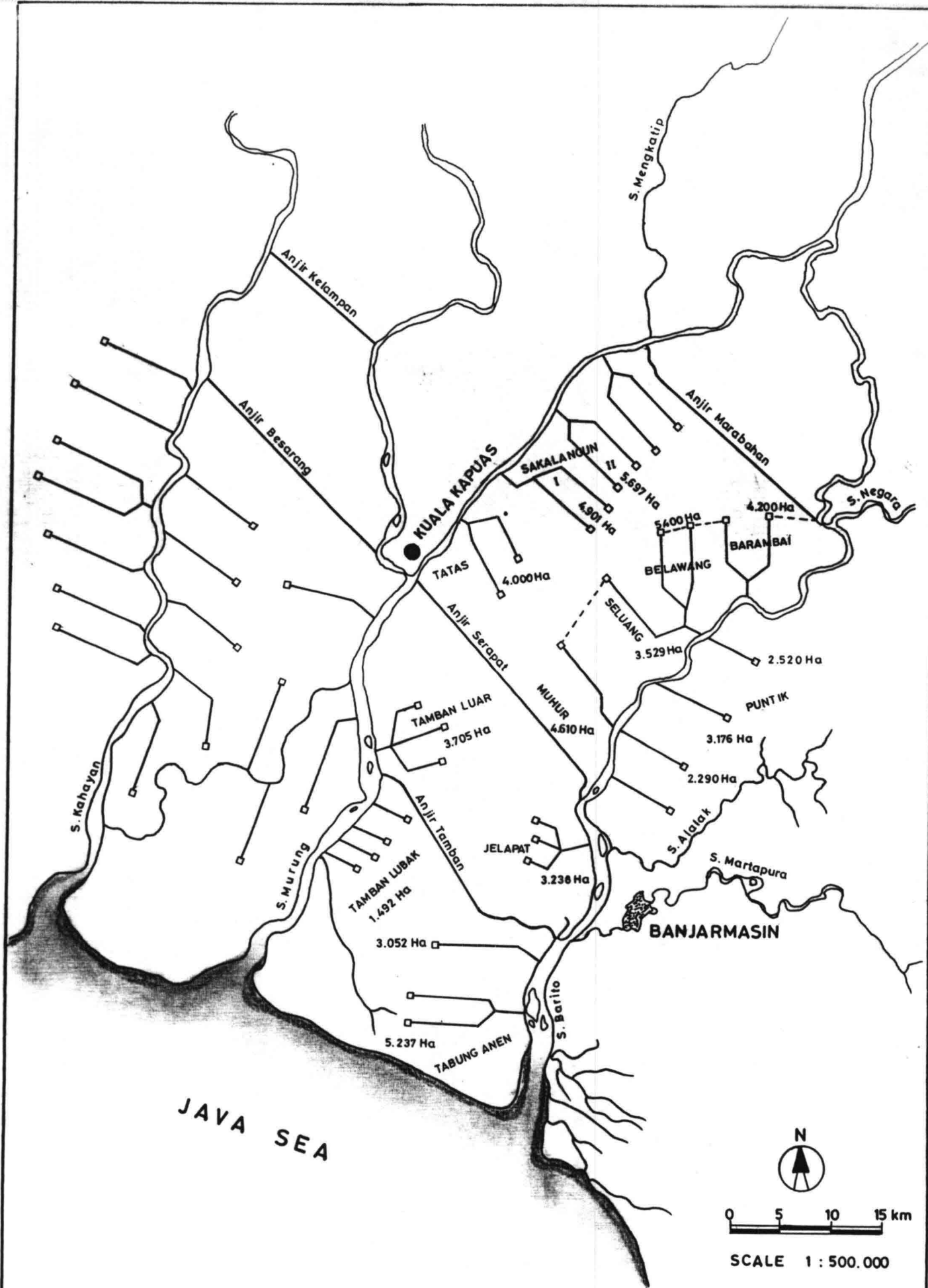
FIG.6. ALTERNATIVE D TO THE BARAMBAI −
SELUANG − BELAWANG − MUHUR UNIT
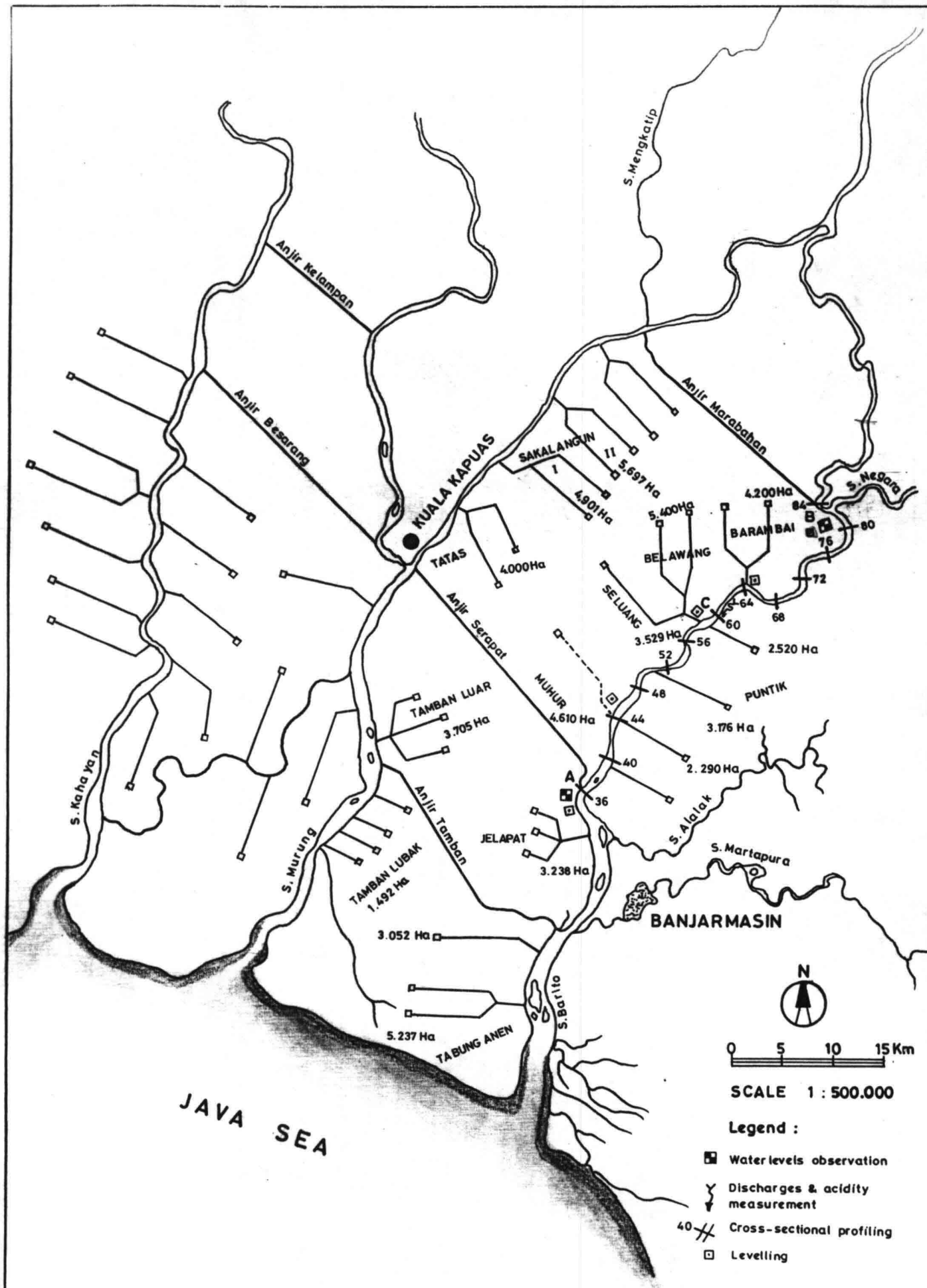
FIG.7. LOCATIONS OF WATER LEVELS, DISCHARGE AND
ACIDITY MEASUREMENTS IN THE BARITO RIVER

STATION	M
1	0
2	1400
3	5660
4	8120
5	4000
6	7700
10	8460
11	6860
12	6920
13	2160

LEGEND:

▣ = Waterlevel (staff-gauge) reading

Sta..╫ = Locations of velocity, acidity and electric conductivity measurements

▦ = Home-yards

Sta.1 = Locations of observation station with number

▣ = Levelling of benchmark

╫ = Cross-sectional measurement

Scale 1 : 40 000

0   0,4   0,8   1,2   1,6   2 Km

N
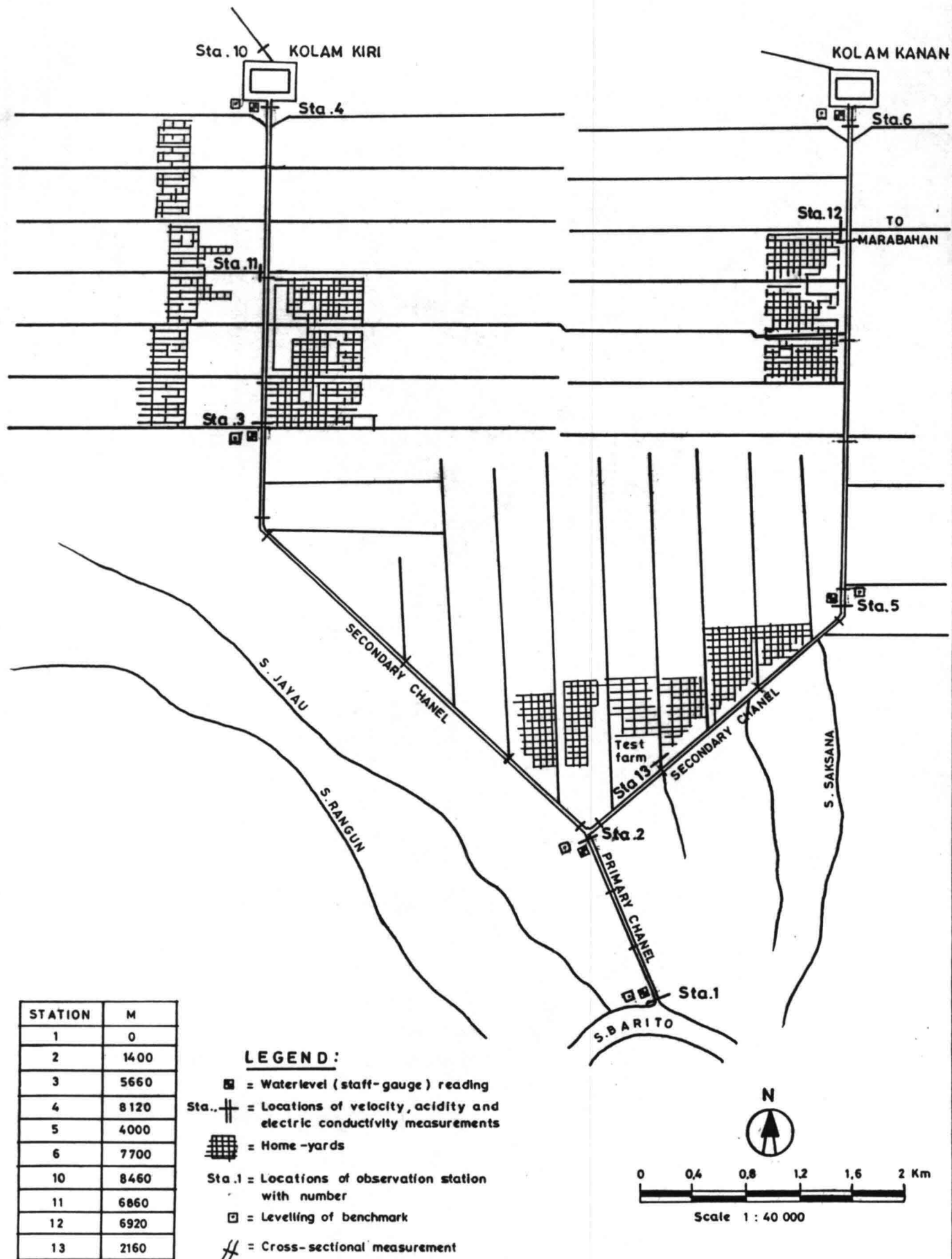
FIG.8. LOCATIONS OF WATERLEVELS, VELOCITY, LEVELLING ACIDITY AND CROSS-SECTIONAL MEASUREMENTS IN BARAMBAI
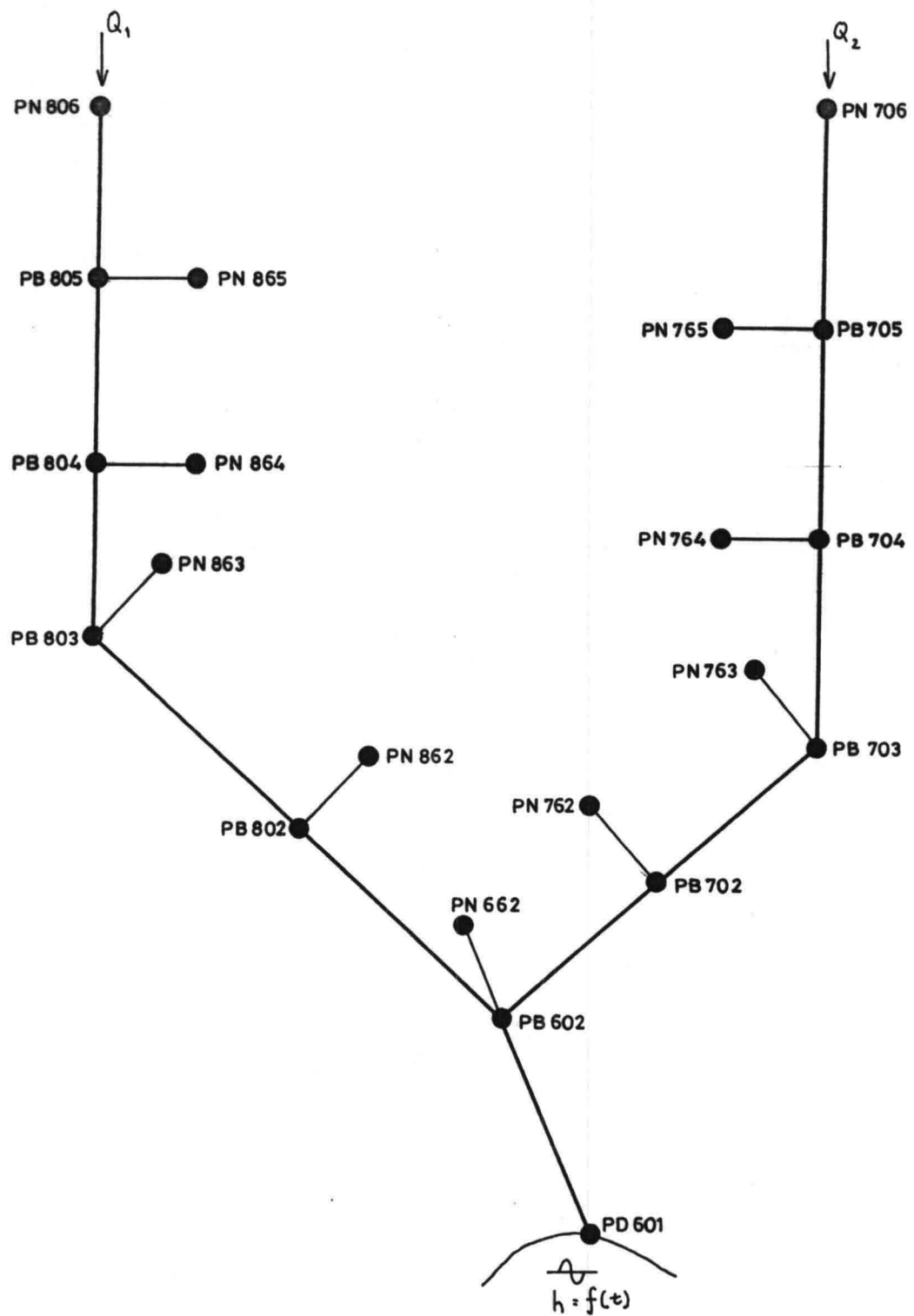
FIG. 9 . REPRESENTATION OF THE BARAMBAI MODEL
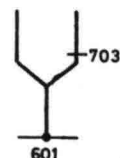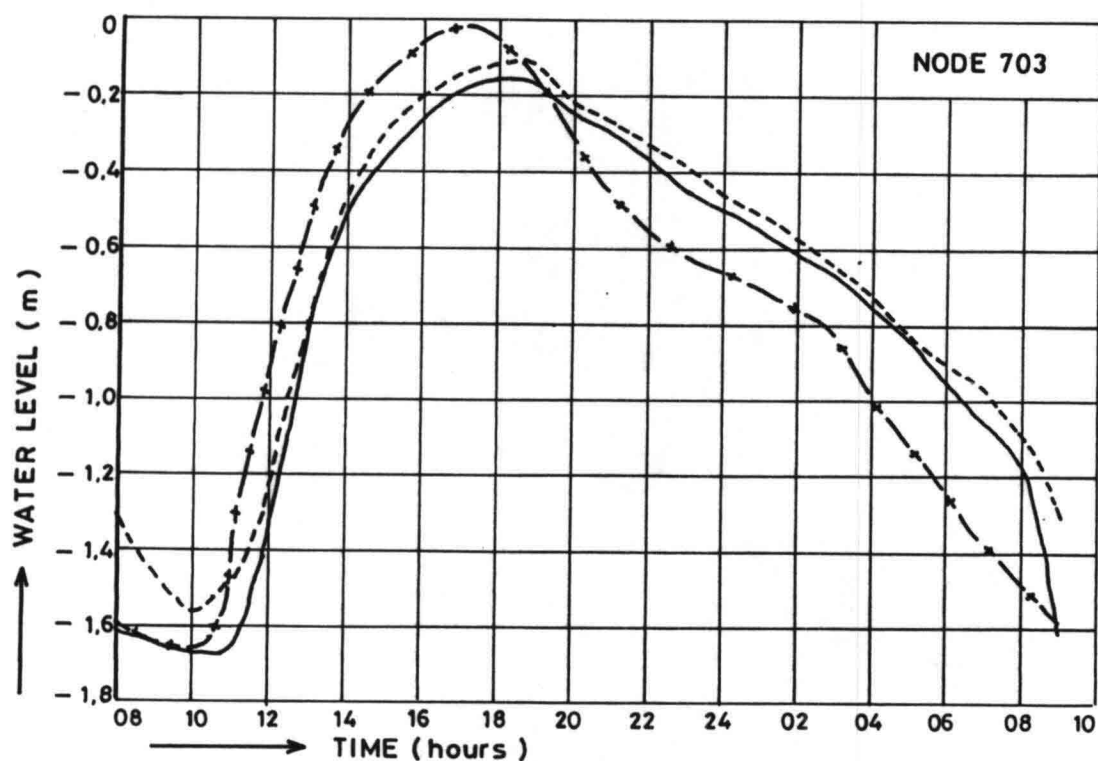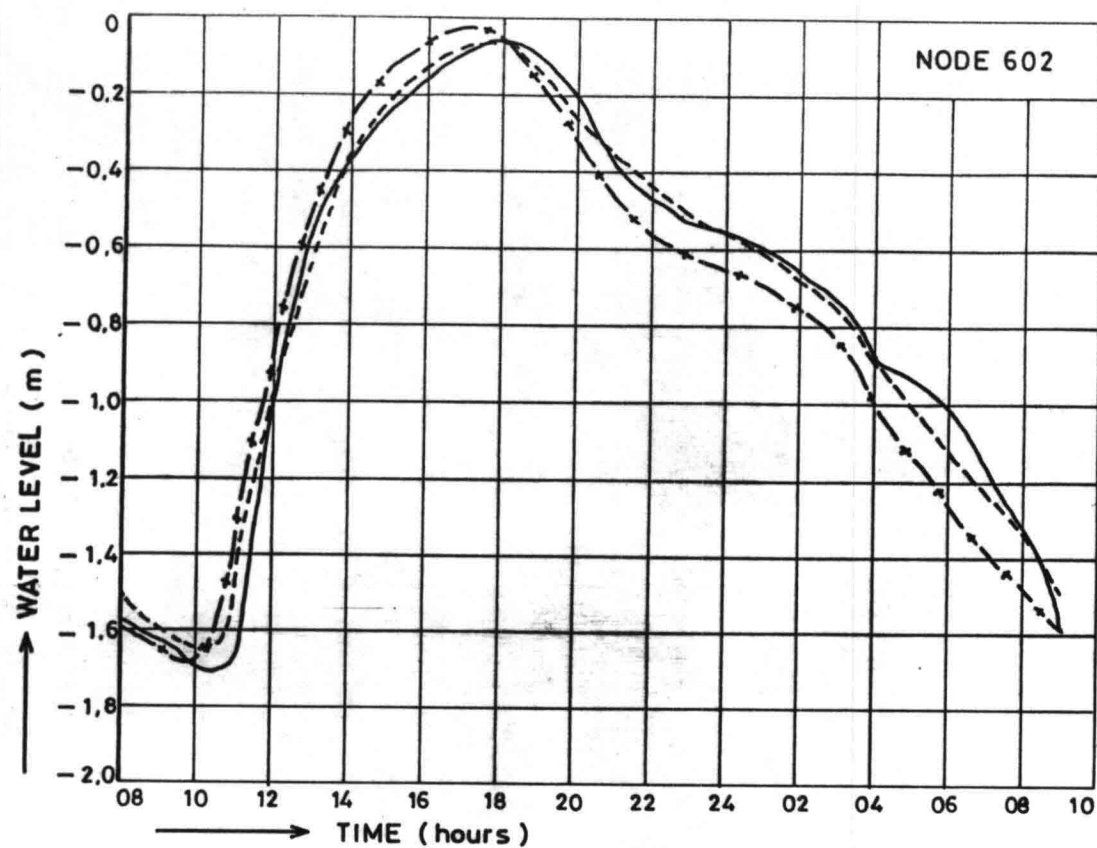
FIG.10. CALIBRATION OF WATER LEVELS AT NODE 602 AND NODE 703
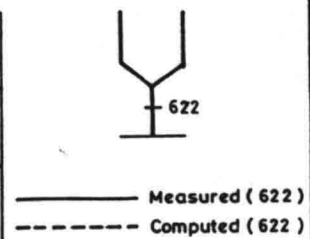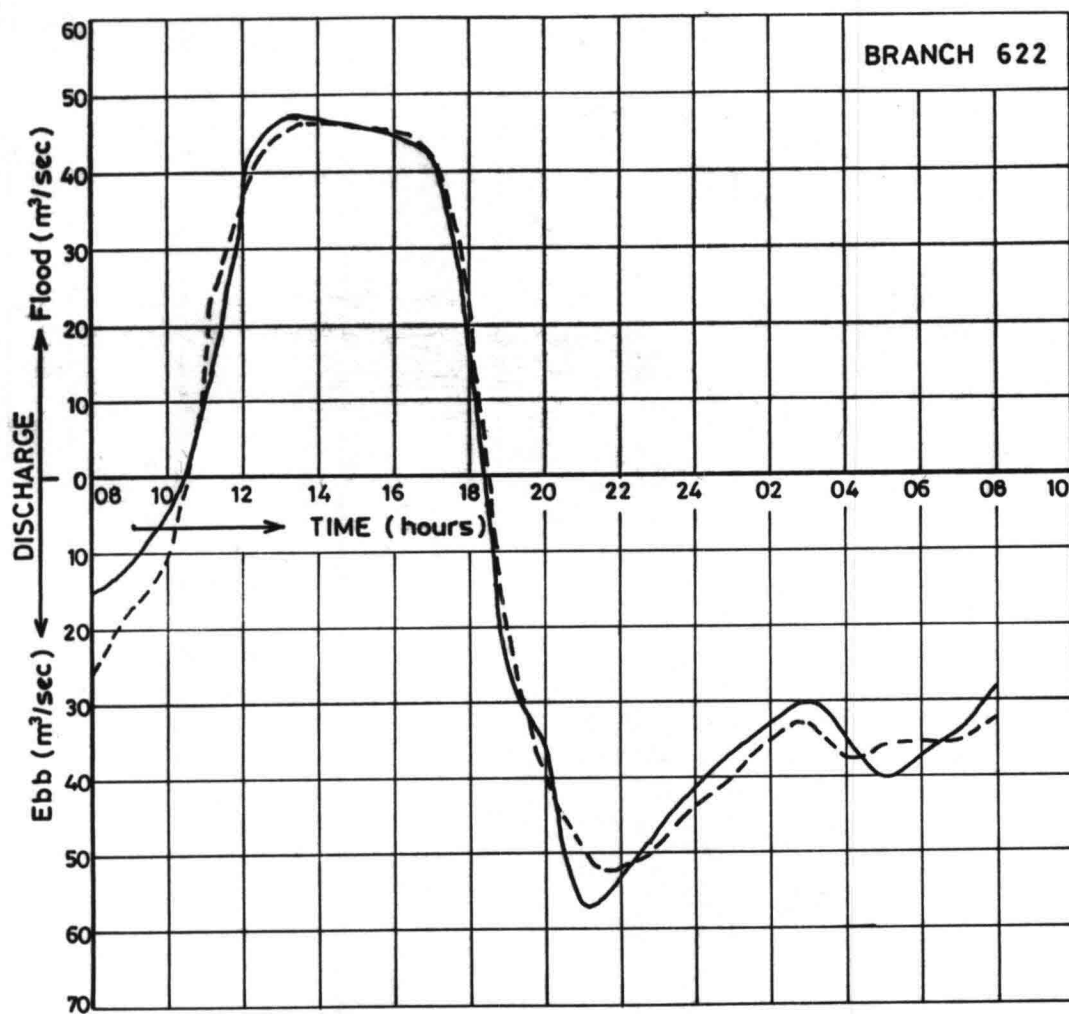
FIG.11 . CALIBRATION OF DISCHARGES AT BRANCH 622
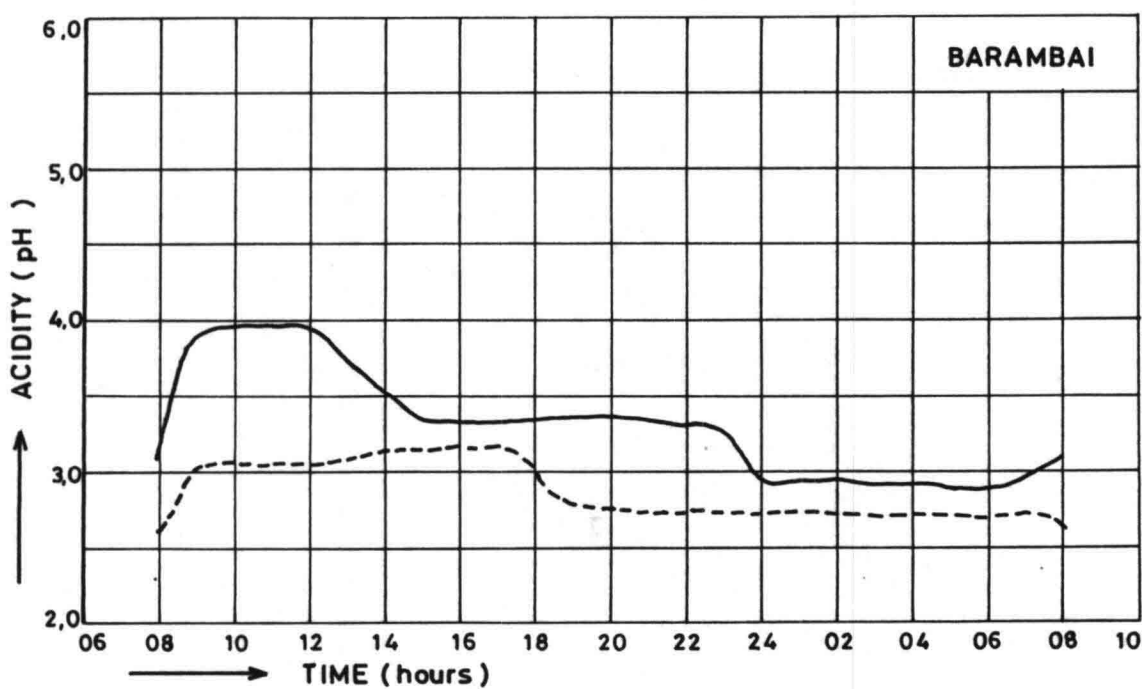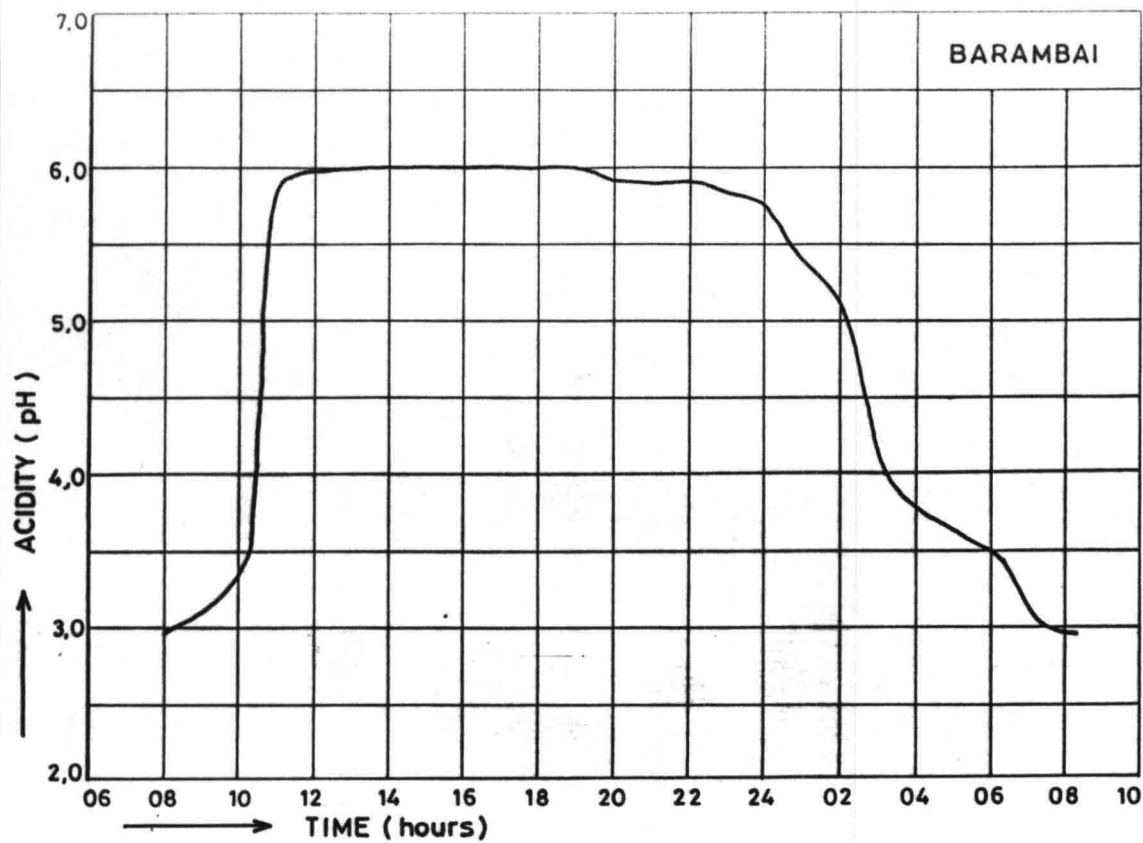
FIG.12. ACIDITY AT NODES 601,706 AND 806
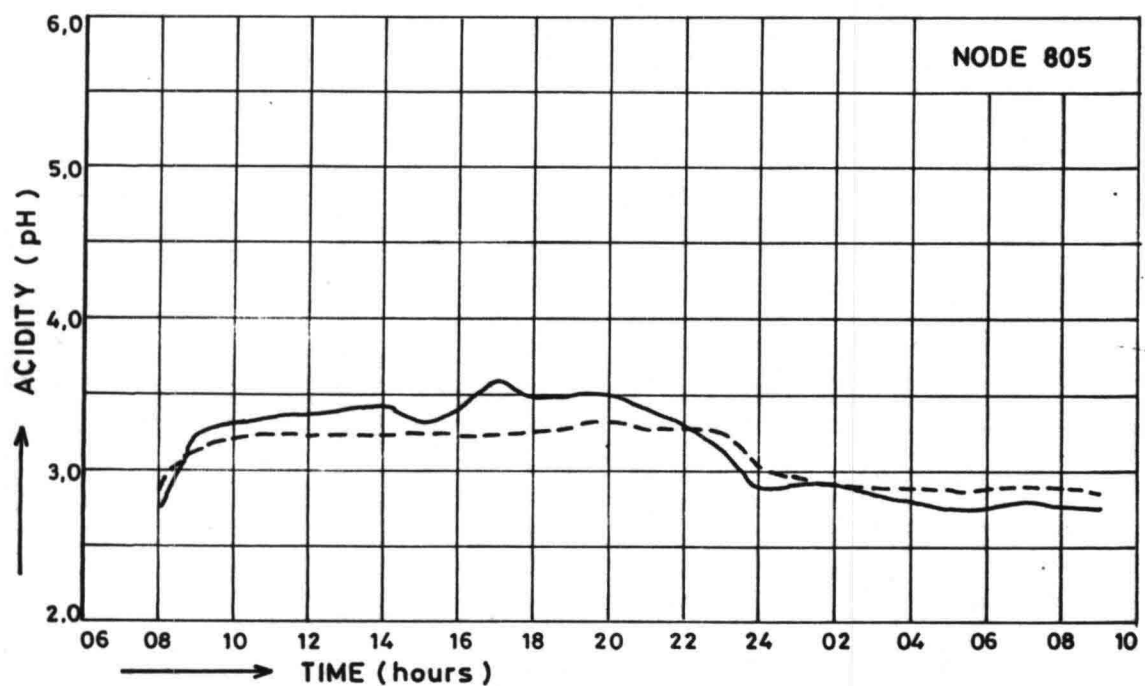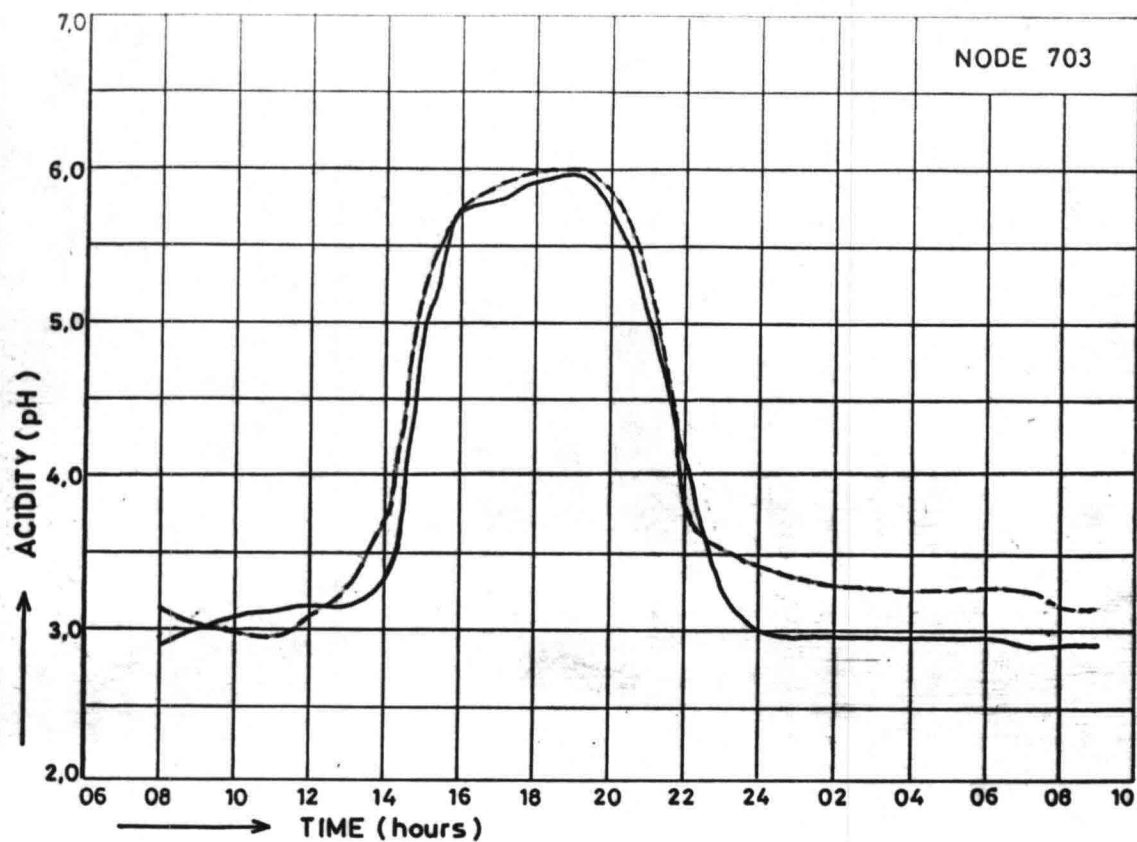
FIG. 13. ACIDITY CALIBRATION AT NODE 703 AND NODE 805
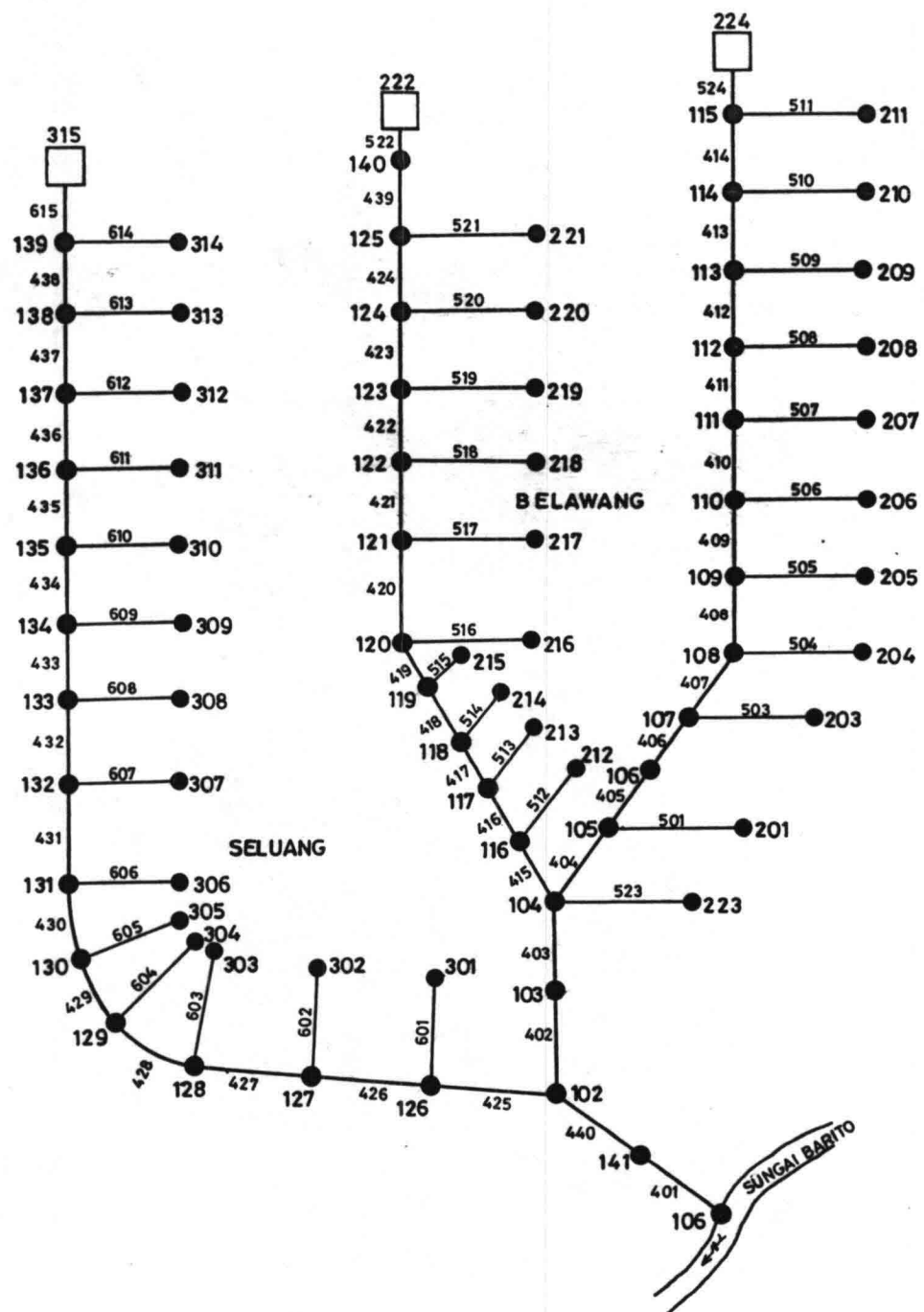
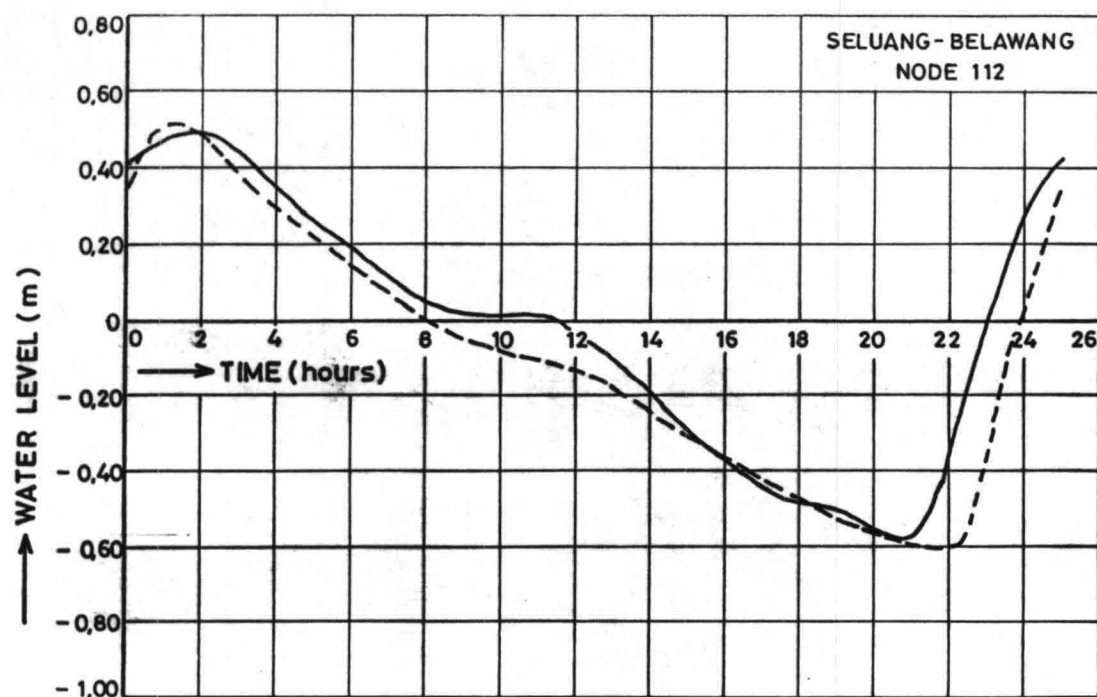FIG. 14. REPRESENTATION OF THE SELUANG – BELAWANG MODEL
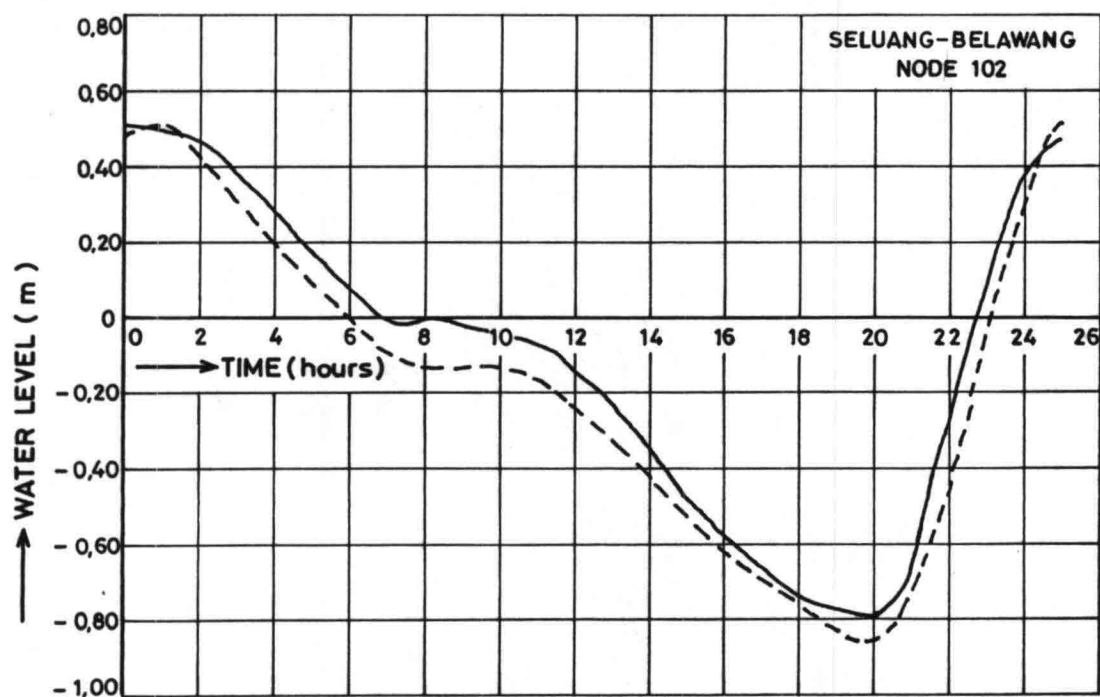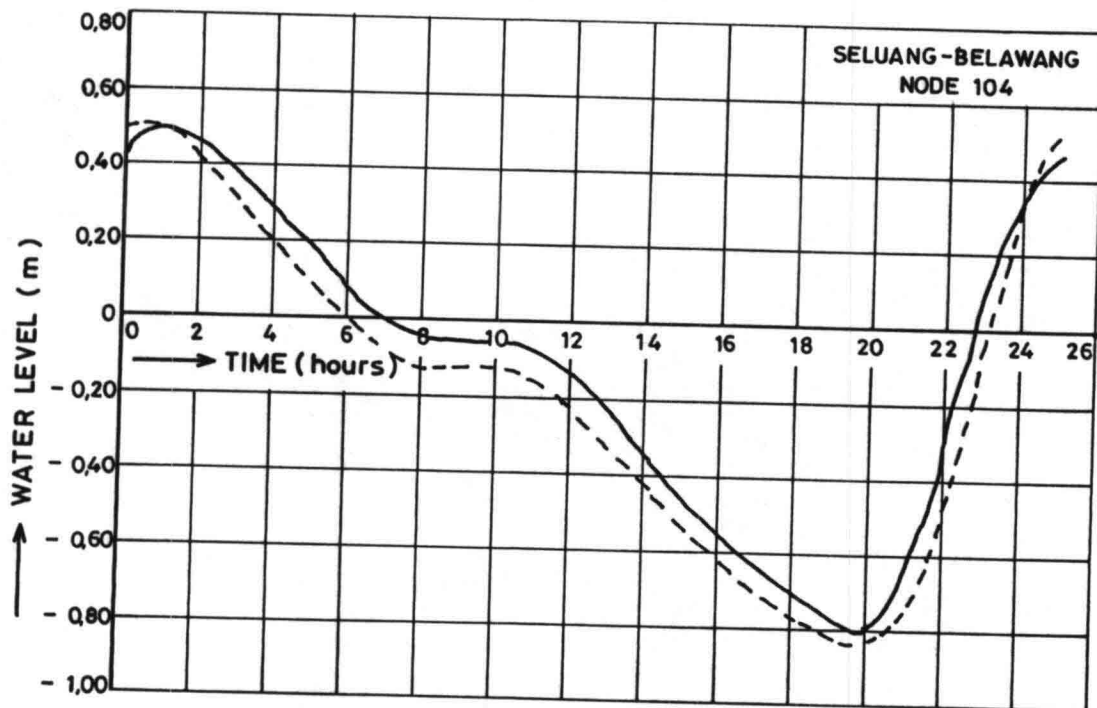
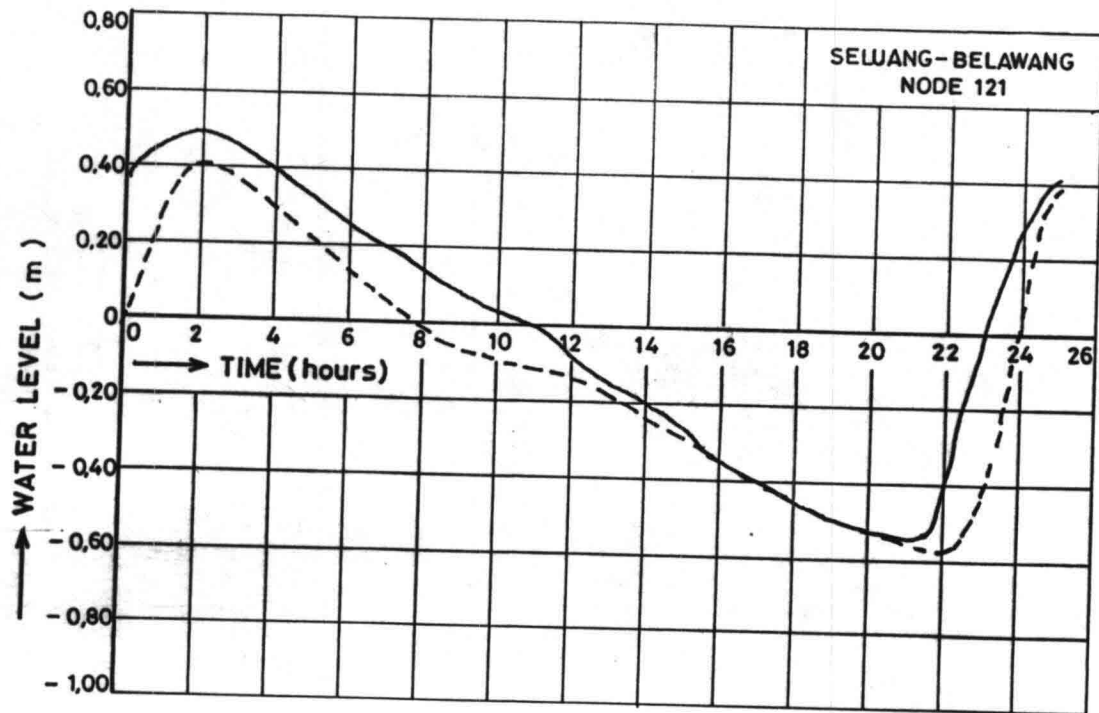FIG.15 . WATER LEVELS CALIBRATION AT NODE 112 & 102
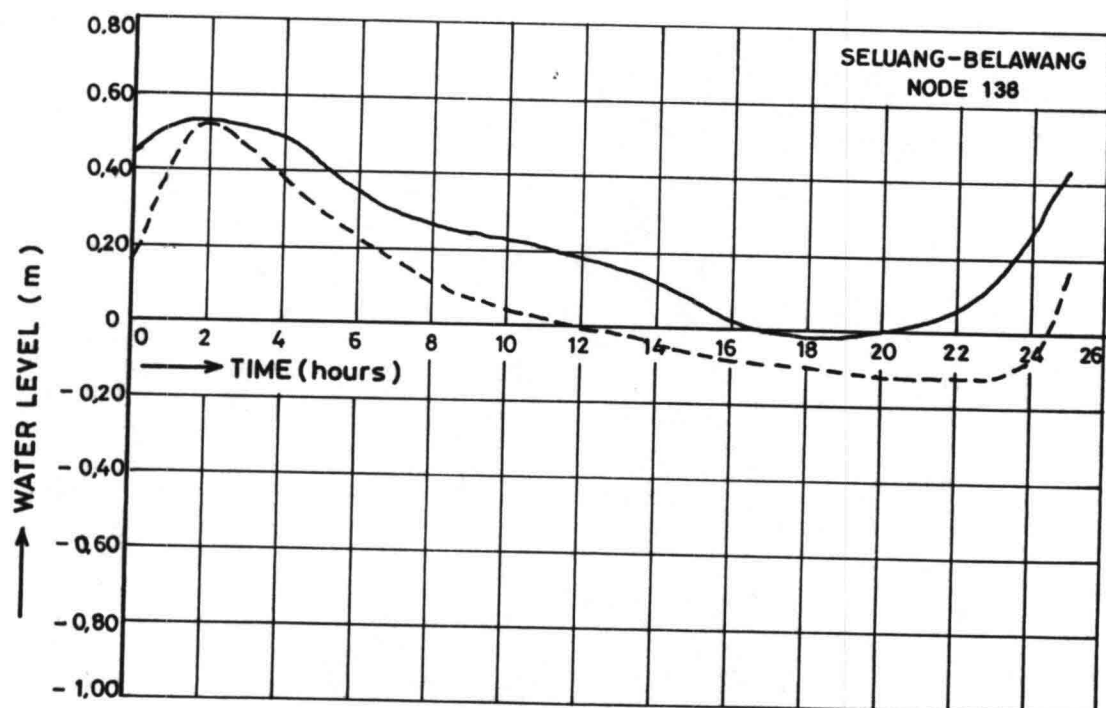
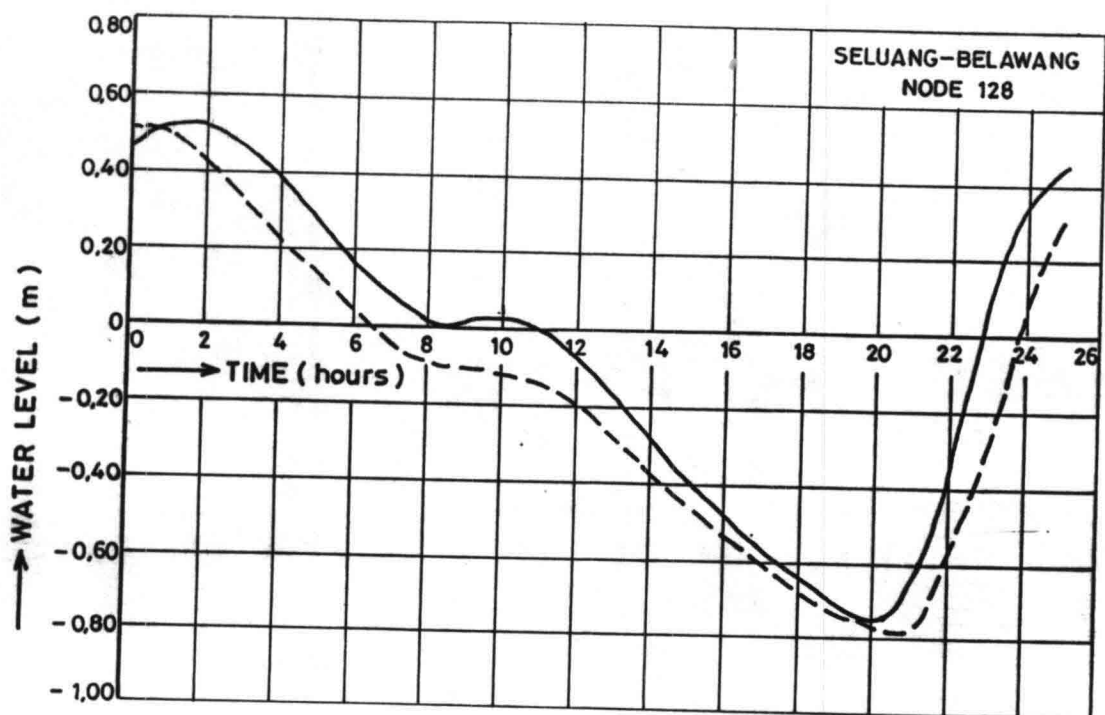FIG. 16. WATER LEVELS CALIBRATION AT NODE 121 & 104

FIG.17.WATER LEVELS CALIBRATION AT NODE 128 & 138
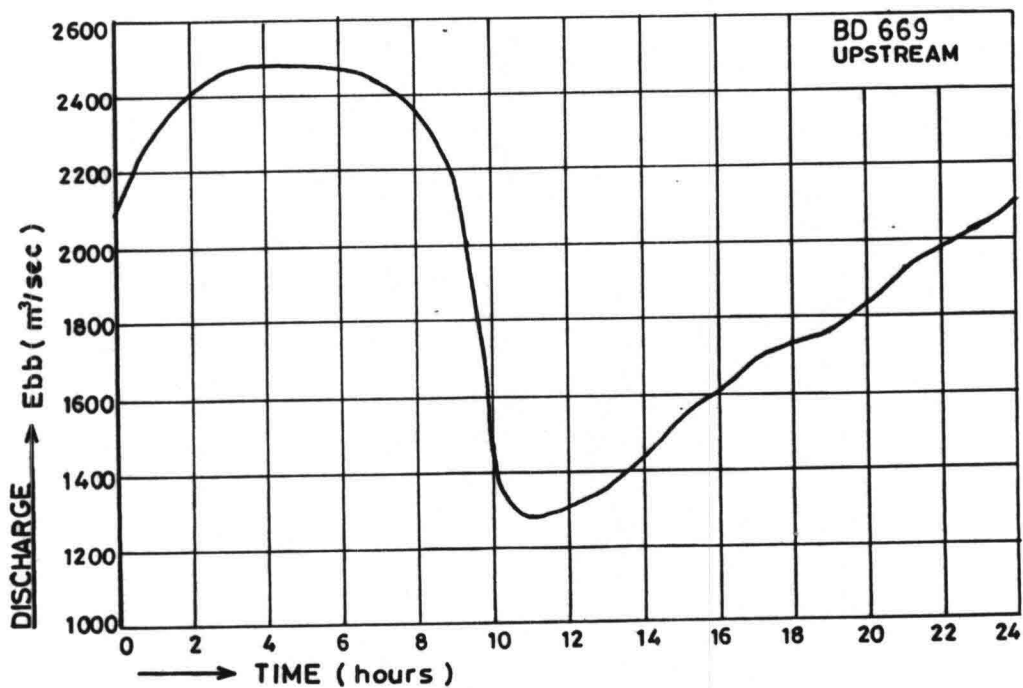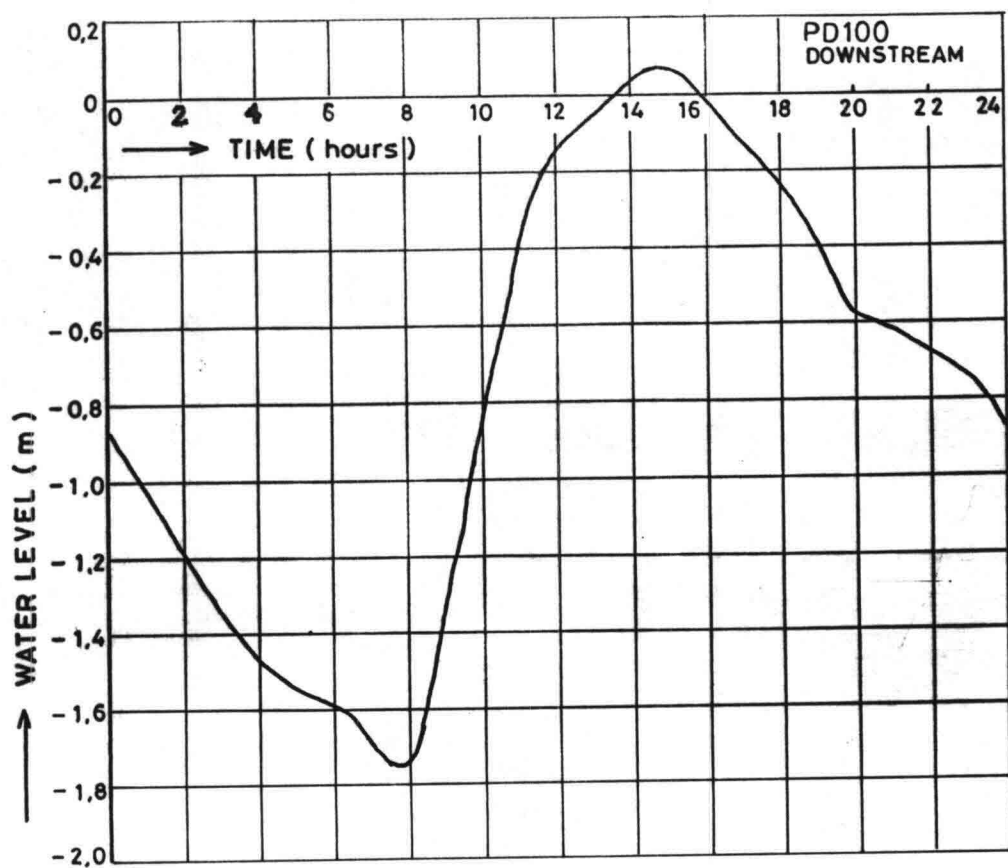
FIG 18. REPRESENTATION OF THE MODEL

FIG.19 . DOWNSTREAM AND UPSTREAM BOUNDARY CONDITIONS
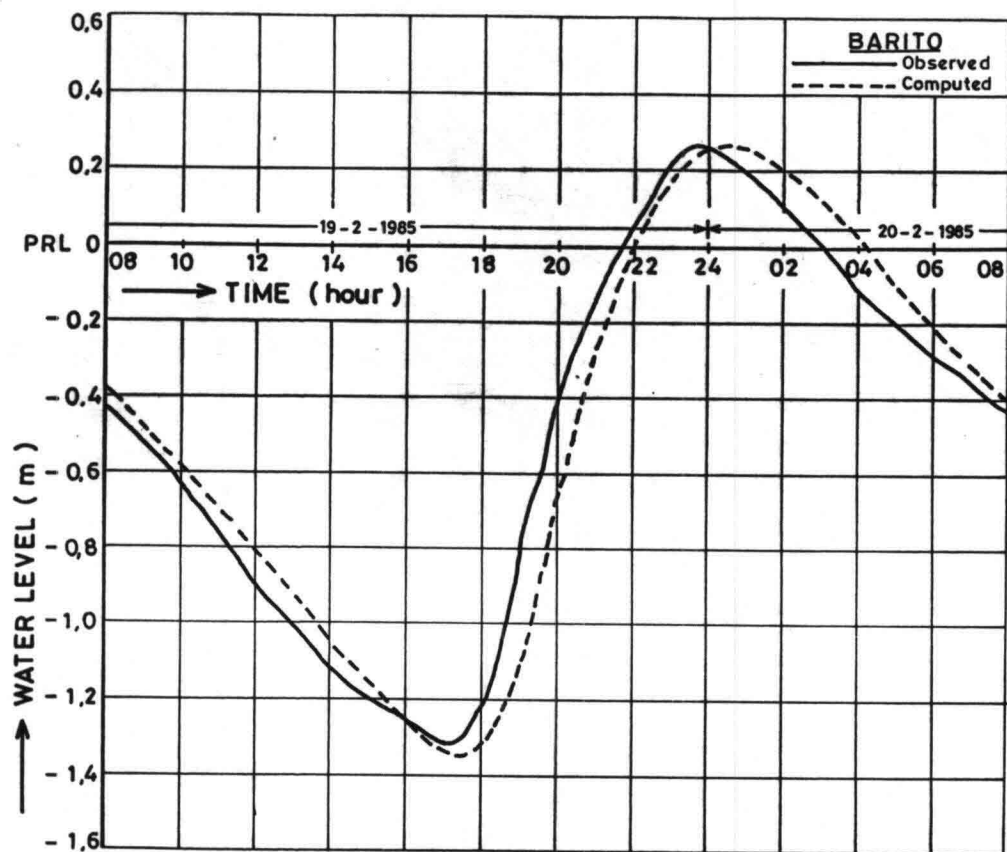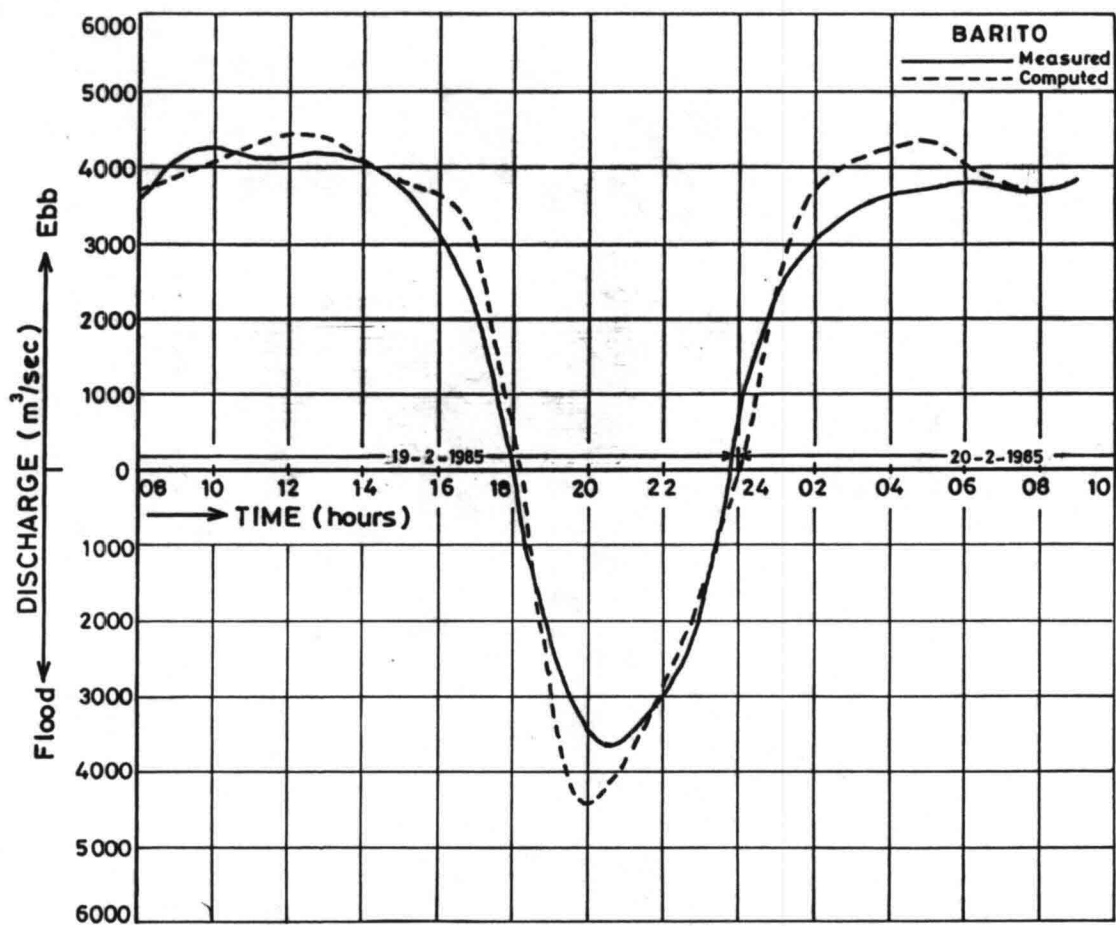
FIG.20.WATER LEVEL CALIBRATION AT NODE 611

FIG.21. DISCHARGE CALIBRATION AT BRANCH 536

ANNEX   1

THE THEORETICAL BACKGROUND OF LONGWAVES AND THE FORMULAE FOR TRANSPORT OF DISSOLVED MATTER

1.GENERAL

Looking at a body of water, water levels and velocities are a function of x, y, z and t.

The equations describing these movements, the so called Navier-Stokes equations, are already known for a long time. The solutions of these complete equations however is not possible up to now. Many simplifications have to be made before a solution is possible which is obtained numerically. In this report, the average water movement in one dimension is considered, leading to the so called one dimensional long waves equations.

Figure 1.1 gives a picture of the forces acting on the water. Wind and barometric pressure will not be considered. Their direct influence is normally small in Indonesian rivers. Also the Coriolis force is left out. Near the equator this force is very small and the effect in river with a maximum width of a few kilometers is negligible.
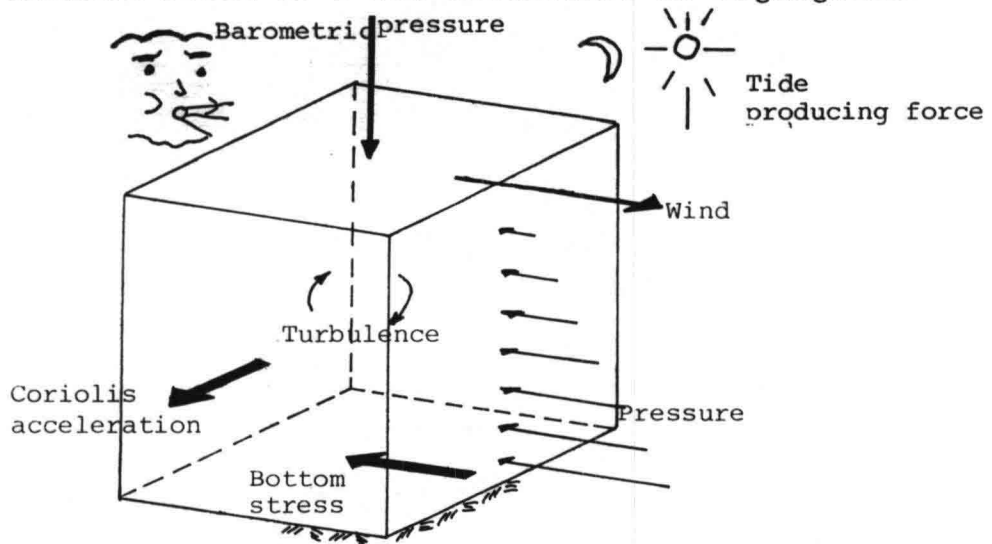

Fig.1.1 The forces acting on the water

The tide producing forces of sun and moon may seem very important, but in fact they can be neglected. How is possible ? Because the internal attraction forces of sun and moon are too small to cause any tidal movement in a river. The penetration of the tide in a river is affected via the boundary conditions. So, in these boundary conditions also the external influence of wind can be included (see figure 1.2).
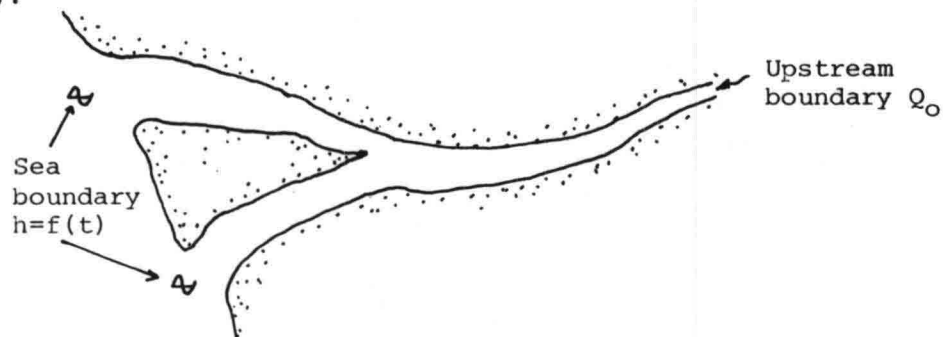

Fig.1.2 Tide penetration via the boundary conditions

1.1

2.BASIC EQUATIONS

The basic equations describing fluid motion are:
-equation of continuity
-equation of motion

The first equation is based on the principle that no mass of water is lost or produced (conservation of mass).
The second equation is based on the Newton's second law :

$$F = m \, a$$

This means that a fluid particle gets an acceleration which depend on its mass and the acting forces.

2.1 Equation of continuity

Water is considered imcompressible, by considering a channel of unit width, with Δx length (see figure 2.1):
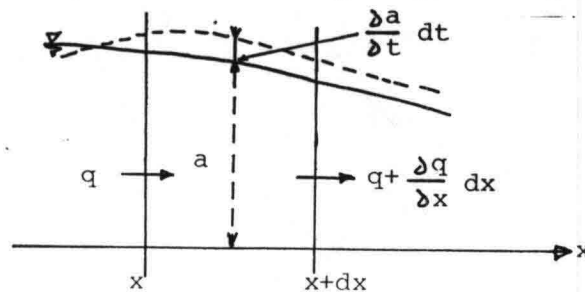

Fig.2.1 The continuity principle

The continuity principle :
For a certain time interval (dt) :

inflow - outflow = increase or decrease in volume

$$q \, dt - (q + \partial q / \partial x \; dx) \; dt = (a + \partial a / \partial t \; dt) \; dx - a \, dx$$

$$- \partial q / \partial x \quad dxdt = \partial a / \partial t \quad dxdt$$

$$\partial a / \partial t \; + \partial q / \partial x \; = \; 0 \qquad \ldots\ldots\ldots\ldots(2.1)$$

where :
a   = water depth [m]
t   = time   [s]
q   = discharge per unit width [m2/s]
x   = location [m]

2.2 Equation of motion

Further simplifications are that all energy losses due to friction, bends, turbulence, etc. are assumed to be concentrated along the wall of river. So, the external acting forces are pressure and wall friction.
According to Pascal s law the pressure in a fluid acts perpendicular to the planes of a particle dxdydz (see figure 2.2):
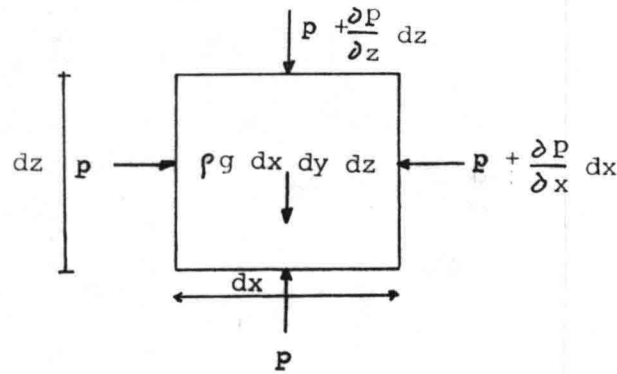
Fig.2.2 The Pascal´s law

Horizontal forces perpendicular to the flow direction are
not considered (y-direction).
In the z-direction the net force is :

p dxdy - (p + $\partial p/\partial z$ dz) dxdy - ρg dxdydz =
-($\partial p/\partial z$ + ρg) dxdydz

In longwaves, the vertical accelerations can be neglected
(in the momentum equation), this leads to :

-($\partial p/\partial z$ + ρg) dxdydz = 0

  $\partial p/\partial z$ + ρg = 0   ....(2.2)

In the x-direction, the net force is :

p dydz - (p + $\partial p/\partial x$ dx) dydz = - $\partial p/\partial x$ dxdydz

the acceleration is equal to dv/dt. Since v is a function
of x and z :

dv/dt = $\partial v/\partial x$ dx/dt + $\partial v/\partial t$ dv/dt

dv/dt = $\partial v/\partial t$ + v $\partial v/\partial x$
where :
 $\partial v/\partial t$ is called local acceleration
v $\partial v/\partial x$ is the convective acceleration

Now, the mass of the element, m = ρdxdydz
According to the Newton's second law :

F = m a

- $\partial p/\partial x$ dxdydz = dxdydz ($\partial v/\partial t$ + v $\partial v/\partial x$)

without friction, the force per unit volume :
- $\partial p/\partial x$ = ($\partial v/\partial t$ + v $\partial v/\partial x$)

Now, the friction force must be introduced. It is assumed
that the empirical formulae for steady flow (for example
Chezy's law) is also valid for non steady flow. This is a
theoretically weak point, but up to now there is nothing
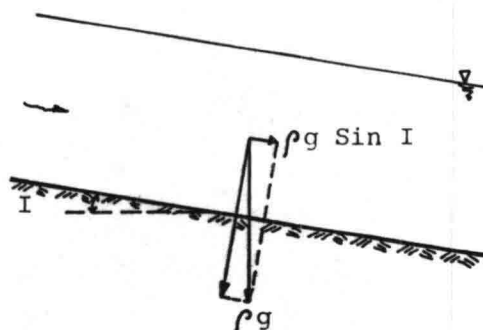
1.3

better. In practice this appears no problem

Fig.2.3 The friction force

Per unit volume, the total friction force is equal to the component in the slope direction (see figure 2.3).

$$F_f = \rho g \sin I. \approx \rho g I \qquad \text{,where I is small.}$$

Chezy's law : $v = C \sqrt{RI}$

$$I_f = V^2 / C^2 R$$

Then, the friction force per unit volume become :

$$F_f = \rho g V^2 / C^2 R$$

Since the direction of the friction force changes with the flow direction, this force has to be written as :

$$F_f = -\rho g v |v| / C^2 R$$

the equation of motion becomes :

$$-\partial p / \partial x - g v |v| / C^2 R = \rho (\partial v / \partial t + v \, \partial v / \partial x)$$

now, it is easier to work with waterlevels or depths instead of pressures.
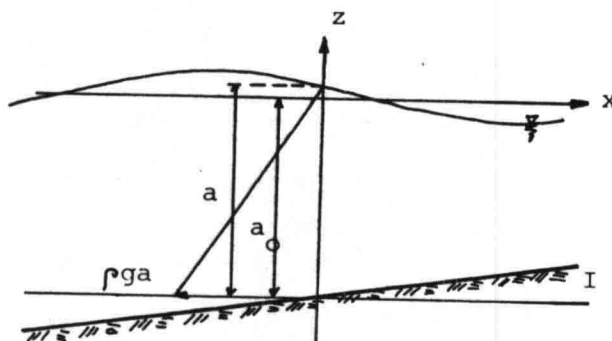Integrating eq.2.2 over the water depth (see figure 2.4)

Fig.2.4 Pressure distribution

1.4

$$\int \partial p / \partial z \, dz + \int \rho g \, dz = p + \rho g z + k = 0$$

$$p + \rho g z + k = 0$$

Boundary conditions :
-at the surface : $z = h \quad \longrightarrow \quad p = 0$
$k = - \rho g h$

so: $p + \rho g z - \rho g h = 0$
$p = - \rho g (z-h)$

Differentiate this to x, that gives :

$$\partial p / \partial x = \rho g \, \partial h / \partial x$$

Usually this is written as :

$$\rho g (\partial a / \partial x + I) ;$$

Finally, the equation of motion can be written as :

$$- \rho g (\partial a / \partial x + I) - \rho g \, v|v| / C^2 R = (\partial v / \partial t + v \, \partial v / \partial x)$$

in case of constant density :

$$\partial v / \partial t + v \, \partial v / \partial x + g(\partial a / \partial x + I) + g \, v|v|/C \quad R = 0$$
$$........(2.3)$$

where :
v = average flow velocity over the cross-section [m/s]
t = time [s]
x = location [m]
g = gravity acceleration [m/s2]
a = water depth [m]
I = bottom slope, positive in +x direction [-]
C = Chezy's coefficient [m1/2/s]
R = A/p =hydraulic radius [m]
A = cross-sectional area [m2]
p = wetted perimeter of cross-section [m]

3. TRANSPORT OF DISSOLVED MATTER
3.1 General
Water can transport a lot of material like salt, acid, oxygen, sand, mud, etc. Also physical properties like heat. In this report it describes only transportation of matter in solution where the movement of the water and the considered matter is more or less the same.

3.2 Basic equations
-continuity equation
Completely analogous with the continuity equation for water,

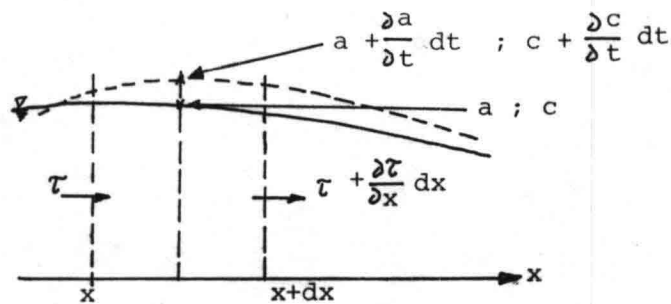this equation can be derived for the transport of matter
(see figure 3.1)

Fig.3.1 Continuity tansport of matter

Mass concentration:
inflow - outflow = increase or decrease of matter

$$T \, dt - (T + \partial T/\partial x \, dx) \, dt = (a + \partial a/\partial t \, dt)(c + \partial c/\partial t \, dt) dx - a \, c \, dx$$

by neglecting $\partial a/\partial t . \partial c/\partial t$, it gives :

$$\partial T/\partial x + \partial(a \, c)/\partial t = 0$$

integration over the storage width :

$$\partial T/\partial x + b \, \partial(a \, c)/\partial t = 0 \quad \ldots\ldots\ldots\ldots(3.1)$$

where :
T   = transport of matter perunit width [kg/s]
x   = location [m]
b   = storage width [m]
a   = water depth [m]
c   = concentration [kg/m3]
t   = time [s]

-transport equation
In a mixed system, again, several transport mechanism can be considered :
  -transportation with the average velocity (advective or convective transport)
  -diffusion (molecular and turbulent)
  -dispersion, due to the uneven velocity distribution, wind, mixing in tidal flow etc.
Diffusion and dispersion are combined into the dispersive transport. Usually the dispersion dominates the diffusion, the molecular diffusion is completely negligible.

The convective transport is simply :
$T_c = Q \, c$

1.6

The dispersive transport is assumed proportional with the concentration gradient :

$$T_D = - A D \delta c / \delta x$$

the negative sign is applied because a positive $\delta c / \delta x$ (c is increasing in the positive x-direction) gives a transport in the negative x-direction (see figure 3.2)
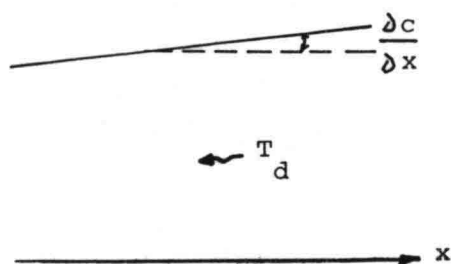

Fig.3.2 **The dispersive transport**

Now, the total transport is :

$$T = T_c + T_D$$

$$T = Q c - A D \delta c / \delta x \qquad \dots\dots\dots\dots\dots (3.2)$$

equation (3.1) and (3.2) are combined into :

$$\delta c / \delta t + v \; \delta c / \delta x = D \; \delta c / \delta x \qquad \dots\dots (3.3)$$

where   :
c   = concentration [kg/m3]
v   = average flow velocity over the cross-section [m/s]
A   = cross-sectional area [m2]
D   = dispersion coefficient [m2/s]

So, by using equations (2.1) and (2.3) water movement model can be set up (see Annex 2 about PENPAS model) with given input data (boundary conditions, initial conditions and model schematization).
The results are water movement parameters (h and v) can be computed.
Then, after having a satisfactory result of the water movement computations, concentration analysis can be carried out by applying the equations (3.1) and (3.2) based on the water movement parameters, boundary conditions and initial conditions for concentration model, besides the dispersion coefficients (see Annex 2 about PENPAS model).

3.3 The dispersion coefficient
In PENPAS, the dispersion coefficient can be estimated as:

$$D = D1 * a * v + D2 \frac{\delta c}{\delta x} \qquad \ldots \ldots [m2/s]$$

$$D2 = 0.002 * \frac{v * L}{co} * Es$$

Modified estuary number, Es is defined as:

$$Es = \frac{P * Frd}{Qf * T}$$

Densimetric Froude number, Frd is defined as:

$$Frd = \frac{v}{\sqrt{g * a * \Delta\rho/\rho}}$$

Where: D1 and D2 : dispersion constants
        co   : maximum concentration at the boundary
        L   : length of the estuary
        Qf   : upland flow
        P   : tidal prism
        T   : tidal period
In case of the acidity problem, there is no density
differences, $\Delta\rho = 0.0$, then D2= 0.0

ANNEX   2

THE NUMERICAL BACKGROUND OF PENPAS AND ITS FACILITIES

1. Numerical method used in PENPAS

PENPAS was developed by the Delft Hydraulics Laboratory, in cooperation with Rijkswaterstaat in 1978. It is especially designed for studying water management problems in tidal swamp reclamation project in Indonesia.

PENPAS uses the finite difference method. By using this method, the differential quotients are replaced by finite differences.
For example:

$$\frac{\partial h}{\partial t} \approx \frac{\Delta h}{\Delta t}$$

This can be done in several ways. The Penpas programme uses a central difference scheme, which can be vizualized as follows:
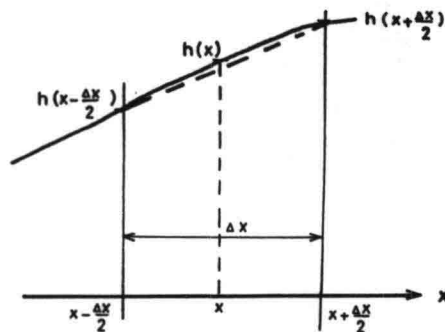
$$\frac{\partial h}{\partial x} \approx \frac{h(x + \Delta x/2) - h(x - \Delta x/2)}{\Delta x}$$

The other differential quotients are treated in a similar way. This implies that a river or a canal has to be devided into segments with a certain length, called branches. The points between the branches are called nodes.
In the nodes water levels and concentrations are computed, while at the centre of the branches the discharges and the transport of matter are computed. So, it is a staggered system in time and space.

To solve the linearized equations, PENPAS uses an explicit method for water movements. Based on the results the concentrations and their transport are computed by means of an implicit method.

To make the difference between explicit and implicit
methods clear, their operator systems are presented in the
figure 2.1:

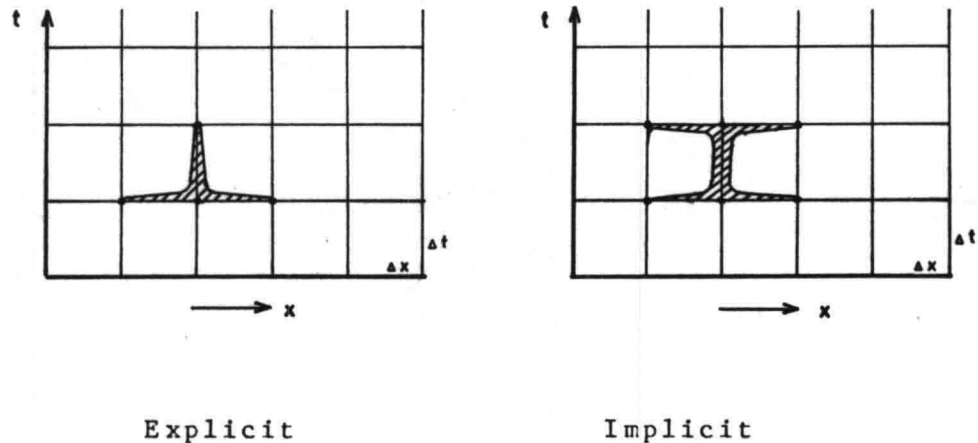<div align="center">Explicit   Implicit</div>

Figure 2.1 The operator system for explicit and implicit
          scheme

Explicit methods are more simple than implicit ones, but
they are less accurate and have stability constraints.
In PENPAS the so called Leap Frog method is applied.

2. The Leap Frog method in PENPAS

For tidal flow, the convective term is very small compared
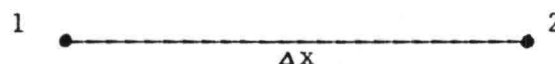to other terms. So, for the beginning this term is left out.

Water movements:
- Equation of motion :
   It is assumed that the bottom is horizontal and the
   convective term is neglected. Thus an integration of the
   momentum equation over the cross-section gives:

$$\frac{\partial Q}{\partial t} + g A \frac{\partial h}{\partial x} + g \frac{Q|Q|}{C^2 A R} = 0 \qquad \ldots\ldots(2.1)$$
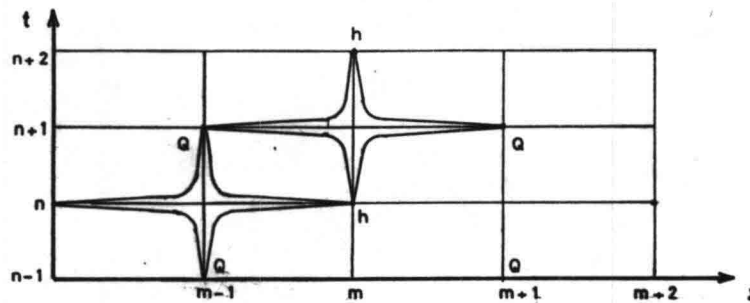

Integrating over the branch results in :

<div align="center">1 •————————————————• 2
Δx</div>

$$\int \frac{\partial Q}{\partial t} \, dx + \int g A \frac{\partial h}{\partial x} \, dx + \int g \frac{Q|Q|}{C^2 A R} \, dx = 0$$

2.2

$$\frac{\delta Q}{\delta t} + \frac{g\,A\;\Delta h}{\Delta x} + \frac{g\;Q\,|Q|}{C^2\,A\,R} = 0 \qquad \dots\dots(2.2)$$

The operator system can be presented as follows:

$$\frac{Q^{n+1}_{m-1} - Q^{n-1}_{m-1}}{\Delta t} + g\,A\;\frac{(h^n_m - h^n_{m-2})}{\Delta x} + g\;\frac{Q^{n+1}_{m-1}\,|Q^{n-1}_{m-1}|}{C^2_{m-1}\,A\,R} = 0$$

$$Q^{n+1}_{m-1} = \frac{Q^{n-1}_{m-1} + g\,A\,\Delta t\,(h^n_{m-2} - h^n_m)/\Delta x}{1 + \dfrac{g\;\Delta t\;|Q^{n-1}_{m-1}|}{C^2_{m-1}\,A\,R}} \qquad \dots(2.3)$$

- Equation of continuity:
  When integrated over a node, the result is:

$$\int b\,\frac{\delta h}{\delta t}\,dx + \int \frac{\delta Q}{\delta x}\,dx = 0$$

$$S\,\frac{\delta h}{\delta t} + \sum Q = 0$$

Or : $S\,\dfrac{\delta h}{\delta t} = \sum Q$

$$S^{n+1}_m\,\frac{(h^{n+2}_m - h^n_m)}{\Delta t} = \sum Q^{n+1}_m \qquad \dots\dots(2.4)$$

Inaccurate values will be the outcome of the calculations in

2.3

case S (storage area) increases strongly with a small increase of h (water level).

To prevent this, PENPAS uses the following procedures:
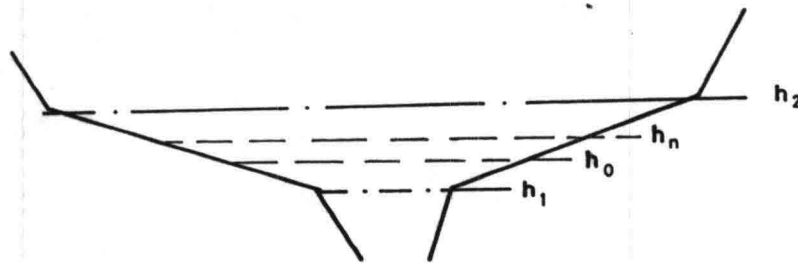
Known from input   :   h1, h2, S1, S2
Known from computation : h_o , S_o , $\sum Q$
Unknown   : h_n , S_n

If : $h_n - h_o = \Delta h$

Then: $\sum Q * \Delta t = \Delta h (S_o + S_n)/2$ ......(2.5)

From linear interpolation :

$S_n = S_o + \Delta h (S_2 - S_o)/(h_2 - h_o)$ ......(2.6)

This is in case of filling. When emptying, h_1 and S_1 are used.

Combining the equations (2.5) and (2.6) gives:

$$2 \sum Q \Delta t = 2 \Delta h S_o + \Delta h^2 (S_2 - S_o)/(h_2 - h_o)$$

$$\Delta h = \frac{- S_o \pm \sqrt{S_o^2 + 2 (S_2 - S_o)/(h_2 - h_o) \sum Q \Delta t}}{(S_2 - S_o)/(h_2 - h_o)}$$

........(2.7)

In all cases the positive sign has to be applied.

With given Δh, h_n and S_n can easily be computed.
By following this procedure at time n+1 all discharges will be computed. At time n+2 all water levels will be computed. At time n+3 again the discharges will be computed and so on.

In case the convective term is included, first v in the momentum equation is replaced by Q.
For the derivatives are found:

$$\frac{\partial v}{\partial t} = \frac{\partial (Q/A)}{\partial t} = \frac{A \, \partial Q/\partial t - Q \, \partial A/\partial t}{A^2}$$

2.4

$$= \frac{1}{A} \frac{\partial Q}{\partial t} - \frac{Q}{A^2} \frac{b}{} \frac{\partial a}{\partial t}$$

$$v \frac{\partial v}{\partial x} = \frac{Q}{A} \frac{\partial(Q/A)}{\partial x} = \frac{Q}{A} \frac{(A \, \partial Q/\partial x - Q \, \partial A/\partial x)}{A^3}$$

$$= \frac{Q}{A^2} \frac{\partial Q}{\partial x} - \frac{Q^2 b}{A^3} \frac{\partial a}{\partial x}$$

This is substituted in the momentum equation, leads to:

$$\frac{1}{A} \frac{\partial Q}{\partial t} - \frac{Q \, b}{A^2} \frac{\partial h}{\partial t} + \frac{Q}{A^2} \frac{\partial Q}{\partial x} - \frac{Q^2 \, b}{A^3} \frac{\partial h}{\partial x} + g \frac{\partial h}{\partial x} +$$

$$+ g \frac{Q \, |Q|}{C^2 \, A^2 \, R} = 0$$

From the continuity equation follows:°

$$b \frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = 0 \quad \longrightarrow \quad \frac{\partial Q}{\partial x} = - \, b \frac{\partial h}{\partial t}$$

b is also not known in a branch, this is approximated by b
The momentum equation finally becomes:

$$\frac{\partial Q}{\partial t} - \frac{2 \, Q \, b}{A} \frac{\partial h}{\partial t} + g \, A \, (1 - \frac{Q^2 \, b}{g \, A^3}) \frac{\partial h}{\partial x} + g \frac{Q \, |Q|}{C^2 \, A \, R} = 0$$

The term $\dfrac{2Q}{A} \dfrac{b}{} \dfrac{\partial h}{\partial t}$ has to be evaluated on a previous time level, because the water level at n+1 is not known. See the following:

Naming: $Q^{n+1}_{m+1} : Q_n$, $h^n_m : h_1$, $dh^{n-1}_m : dh_1$

$\qquad\qquad Q^{n-1}_{m+1} : Q_0$, $h^n_{m+2} : h_2$, $dh^{n-1}_{m+2} : dh_2$

The equation becomes:

$$\frac{Q_n - Q_o}{\Delta t} + g \frac{Q \, |Q|}{C^2 \, A \, R} =$$

$$= g \, A \, (1 - \frac{Q_o^2 \, b}{g \, A^3}) \, \frac{h_1 - h_2}{\Delta x} + \frac{2 \, Q_o \, b}{A} \, \frac{dh_1 + dh_2}{2 \, \Delta t}$$

From which follows:

$$Q_n = \frac{Q_o + g \, A \, \Delta t \, (1 - \frac{Q_o^2 \, b}{g \, A^3}) \, \frac{h_1 - h_2}{\Delta x} + \frac{Q_o \, b}{A} \, (dh_1 + dh_2)}{1 + \frac{g \, \Delta t \, |Q_o|}{C^2 \, A \, R}}$$

$$\ldots\ldots\ldots(2.8)$$

Concentration calculations

- Equation of continuity

$$\int \frac{\partial T}{\partial x} \, dx + \int b \, \frac{\partial (h * c)}{\partial t} \, dx = 0$$

$$T_2 - T_1 + \frac{\partial (V * C)}{\Delta t} = 0$$

$$\frac{\partial (V * C)}{\Delta t} = \sum T \qquad\qquad \ldots\ldots(2.9)$$

Where :
V : volume of the node = $\int b * a * dx$

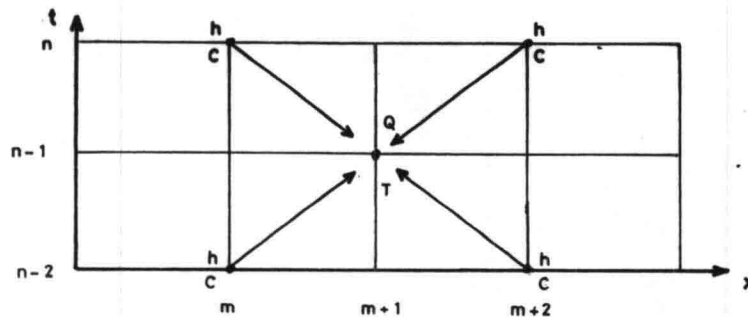$$\frac{\partial (V * C)}{\Delta t} \approx \frac{V * C \, (t + \Delta t / 2) - V * C \, (t - \Delta t / 2)}{\Delta t}$$

- Equation of transport :

$$\int T \, dx = \int Q*c*dx - \int A*D \frac{\partial c}{\partial x} \, dx$$

$$T = Q*c + A*D \frac{(c2 - c1)}{\Delta x} \quad\quad\quad \dots\dots(2.10)$$

An implicit scheme is used for solving these equations, because an explicit scheme would lead to a very small time step. The transport and concentrations are computed immediately after the discharge is computed.
The operator system of this implicit scheme is presented in the following figure:

$$T^{n-1}_{m+1} = Q^{n-1}_{m+1} \, \overline{\overline{c}} + A^{n-1}_{m+1} D^{n-1}_{m+1} (\overline{c}_2 - \overline{c}_1) / \Delta x \quad \dots(2.11)$$

Where :

$$\overline{\overline{c}} = (c^{n-2}_m + c^{n-2}_{m+2} + c^n_m + c^n_{m+2}) / 4$$

$$\overline{c}_1 = (c^{n-2}_m + c^n_m) / 2$$

$$\overline{c}_2 = (c^{n-2}_{m+2} + c^n_{m+2}) / 2$$

- Equation of continuity:

$$\frac{V^n_m c^n_m - V^{n-2}_m c^{n-2}_m}{\Delta t} = \sum T^{n-1}_m \quad\quad\quad \dots\dots(2.12)$$

Where : $V^n_m = V^{n-2}_m + \sum Q^{n-1}_m \Delta t$

The concentrations on the new time level are not known, so the transport can not be computed directly. In PENPAS it is done by using direct iteration. First, T is estimated by explicit computation, which only uses the concentrations at the old time level.

2.7

$$T_{m+1}^{n-1} = Q_{m+1}^{n-1} \frac{(c_m^{n-2} + c_{m+2}^{n-2})}{2} + A_{m+1}^{n-2} D_{m+1}^{n-2} \frac{(c_m^{n-2} - c_{m+2}^{n-2})}{\Delta x} \quad ...(2.13)$$

This transport is computed for all branches and used in the continuity equation to compute the concentration on the new time level. These new concentrations are then used to compute the transport again. This continues untill the differences in the computed concentrations at the new time level become sufficiently accurate.

- Convergence of the iteration
  This is a first order iteration process, which can be written as :

$$T = f(T)$$

Convergence is assured, when :

$$f'(T) < 1$$

$$f'(T) = \frac{A \ D \ \Delta t}{\Delta x \ V} < 1$$

$$\Delta t < \frac{\Delta x \ V}{A \ D}$$

Δt is minimal when there is no extra storage area, so:

$$V = \Delta x * A$$

$$\Delta t < \frac{\Delta x^2}{D} \quad ....(2.14)$$

This is of course only a first order approximation of the permittable time step. When it is too big, the iteration will diverge or may converge, but that happens very slowly.

3. Facilities in PENPAS

-water movement : Points and branches
 Points :
 PN   : A normal point, where the storage area is given as
           a function of the water level.

Also a constant inflow or outflow can be apart of the input data. This type is used when there is storage of water outside the flow section of the branches.

PB   : A point where the program itself computes the storage area from the branches that are concerned with the point. This type is used when there is no extra storage outside the flow section. Again a constant in or outflow can be apart of the input.

In PN and PB, the water level is computed with the continuity equation.

PC   : A boundary point, where the water level is given as a constant value.
PD   : A boundary point, where the water level is given in a data set. This type is used when the boundary is a measured water level.
PF   : A boundary point, where the water level is given as a sum of sine factors (Fourier components).
PP   : A boundary point, where the water level is a prediction, computed from the four main tidal components (M2, S2, K1, O1).

Branches :
BN   : A normal branch, where the width is given as a function of the water level. The roughness is given as a k value (Nikuradse).
BM   : The same as BN, but now the roughness is given as a n-value (Manning).
BW   : A weir, where the effective width is given as a function of the downstream water level.
BK   : A culvert, the effective width is again given as a function of the downstream water level.

In BN, BM, BW and BK, the discharge and velocity are computed with the equation of motion.

BC   : Same as PC, now the discharge is given.
BD   : Same as PD, now the discharge is given.
BF   : Same as PF, now the discharge is given.
BI   : A boundary branch of infinite length. Waves travel into this branch without being reflected.

-Concentration
Points:
PNSA   : A normal point where the salinity is computed with the continuity equation. This type is automatically assigned to PN and PB.
           The concentration of the in or outflow may have different values.
PCSA   : A boundary point, where the concentration is given as a constant value. Automatically assigned to PC, PD and PF.
PDSA   : A boundary point, where the concentration is given in a data set.

2.9

PSSA   : A boundary point where the concentration is com-
          puted during outflow, and a maximum value during
          inflow.

Branches:
BNSA   : A normal branch, where the concentration
          transport is computed with the transport
          equation. This is automatically assigned to BN
          and BM.
BASA   : A branch where only the advective transport is
          computed with the concentration of the upstream
          point. This type is automatically assigned to BK
          and BW.
BCSA   : A branch where the transport is computed from the
          discharge multiplied with a constant
          concentration. This type is automatically
          assigned to BC, BD and BF.

ANNEX   3

HYDRAULIC LEVELLING IN SWAMPY AREAS

PAPERS
Volume
B

Ba 5.

HYDRAULIC LEVELLING IN SWAMPY AREAS

by

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A B S T R A C T

Along various rivers in the tidal swamps in Indonesia many irrigation
schemes have been planned. The slopes of land, water and river (and other
hydro-topographic data) are important parameters to be known for the design
of gravitational irrigation and drainage systems. Already in the pre-feasi-
bility stage benchmarks are erected which have to be tied up to one reference
level.

In general the river banks are hardly accessible by which topographical
levelling is difficult and the accuracy of the results is often small. There-
fore "hydraulic levelling" is executed which means that levels of benchmarks
along a river are connected using a well-known*) hydraulic method.
    The method consists of simultaneous measurements of water levels (staff
gauges) at two places along the river and velocity measurements in between.
Applying the one-dimensional momentum equation for the river section considered,
the time the water level is horizontal in the middle of the section can be de-
termined and the levels of the staff gauges can be related. By topographical
levelling between staff gauges and nearby benchmarks, the benchmark levels are
determined.

This method is applied in various rivers in Indonesia. Experience is
gained with its possibilities and limitations. In some cases topographical
levelling data became available at a later stage. Comparison proves the valid-
ity of the method for the flat rivers in swampy areas.

*)   See a.o. E.W. Bijker : River Tide Measurements (Chapter 21) of
                            Coastal Engineering (Volume I);
                            Delft University of Technology.

1. INTRODUCTION

The slope of a river (bed slope or water surface slope) can be obtained by finding the relationship between levels of certain points along the river. Such relationship is usually obtained by traditional survey work, i.e. topographical work.

Tidal areas in Indonesia are usually found in the form of swampy areas with a dense vegetation. Such conditions cause difficulties in topographical levelling work, so that it seems to be an unappropriate approach.

To avoid difficulties a method of levelling was applied by making use of the tidal condition prevailing in the river. This paper deals with such a hydraulic method of levelling.

In low-lying areas, tidal waves along the coast enter the river through its mouth. This wave propagates along the river over tens, sometimes even several hundreds of kilometres before it is damped out. This distance depends on various conditions as tidal range, river-bed slope, roughness coefficient of the river, upstream discharge, etc.

In deep water, the diurnal tidal wave has a length of about 600 kilometres, while for the semidiurnal tidal wave the length is about 300 kilometres. Having a height of only several metres, the wave is considered as a mild slope wave. An assumption that an observed part of the wave which is much shorter than the wave length as a straight line can be accepted.
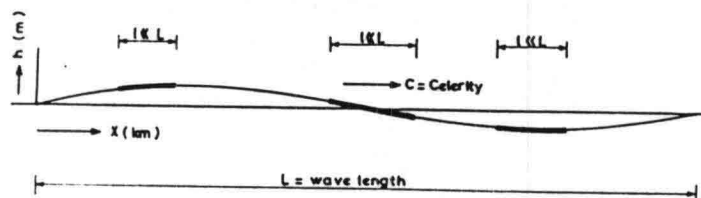
Fig. 1   Tidal wave

The slope of a certain river stretch of ℓ kilometres length will change in accordance with the part of the wave taking place in that stretch. At the moment the slope is zero, the water surface is at a same level at any point all over the river stretch. By relating at that moment the water levels at both ends of the river stretch with their nearby benchmarks, the level difference of the benchmarks can be determined.
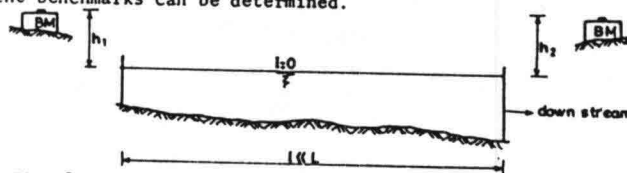

Fig. 2   River stretch of ℓ kilometres length ($\ell \ll L$).

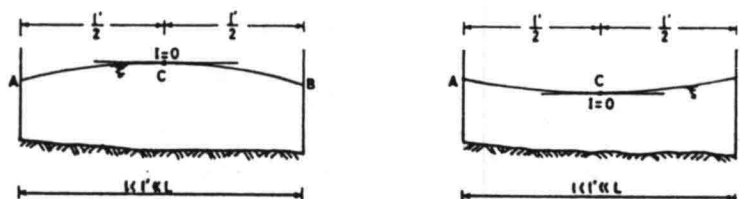For a longer river stretch, the water surface of the river stretch can be sketched as a curve (see Fig. 3).

Fig. 3   River stretch A-B of ℓ' kilometres length ($\ell < \ell' \ll L$)

As an approximation the points A and B will be taken at the same level (assuming symmetric condition).

When ℓ' is taken too distant, the levels of points A and B will not be the same. In this case the accuracy of this method lessens. On the other hand, taking a too short ℓ' makes this method inefficient. So the length of the river stretch must be chosen carefully.

Factors affecting the determination of the length of the river stretch will be discussed further in the other part of this paper.

A comparison of the result of this hydraulic method with topographic survey result is given.

2. THEORETICAL BACKGROUND

Consider the equation of motion :

$$v \frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} = - g\, I - g\, \frac{v\,|v|}{C^2 R} \qquad \dots \qquad (1)$$

where :

v = flow velocity
x = axis in longitudinal direction of the river
t = time
g = acceleration of gravity
I = water surface slope
C = Chezy coefficient
R = hydraulic radius.

At the moment $I = 0$ at point C in Fig. 3, equation (1) becomes :

$$v \frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} = - g\, \frac{v\,|v|}{C^2 R} \qquad ,$$

or: $\quad \dfrac{dv}{dt} = - g\, \dfrac{v\,|v|}{C^2 R} \qquad \dots \qquad (2)$

or: $\quad \dfrac{dv}{v\,|v|} = - g\, \dfrac{dt}{C^2 R}$

By integrating the last equation, we get :

$$\int \frac{dv}{v\,|v|} = - \int \frac{g}{C^2 R}\ dt$$

$$- \frac{1}{|v|} = - \frac{g}{C^2 R}\,(t - k)\ ; \quad k = \text{integration constant}$$

or : $\quad |v| \cdot (t - k) = \dfrac{C^2 R}{g} \qquad \dots \qquad (3)$

As R changes with the water level, whereas the water level at $I = 0$ still remains unknown at the moment, as an approximation R is determined for average water level.

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For a certain value of C and any value of k, equation (3) gives a curve showing the relationship between v and t as follows:
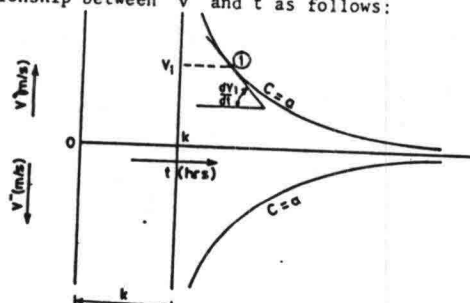

Fig. 4   v versus t curve

Any point of the curve indicates a pair of v and $\frac{dv}{dt}$ values at which I = 0 occurs. Accordingly another curve of v against t can be drawn by plotting survey results at point C (see Fig. 5).
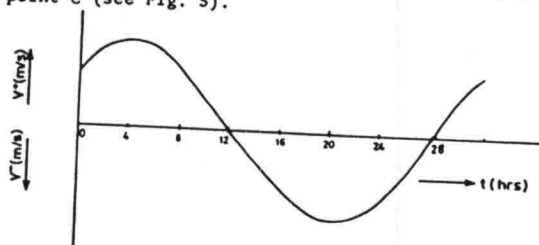

Fig. 5   Velocity measured at location C.

Any point of this curve with the same v and $\frac{dv}{dt}$ values as the values obtained from any point of previous curve, indicates the time of I = 0 phenomenon. Such points are the tangent points of both curves resulted by substituting an appropriate k value to the first curve.

Graphically such points can be obtained by moving the first curve (Fig.4) on the second curve (Fig.5) along its t axis. See Fig. 6 as follows :
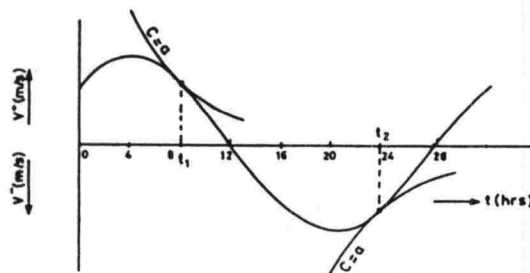
Fig. 6   The points indicate the time I = 0 phenomenon.

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At those moments (indicated by the tangent points), water level at points
A and B must be the same. As a consequence, the water-level curves of points
A and B should intersect each other at those moments.

By taking one of these intersection points as the basic of this graphical
solution, it can be seen whether other intersection points are in their right
positions in the t axis. Should the positions be incorrect, the C value
should be modified. By tidal and error the correct value of C in relation
with the approximation value of R is determined. Herewith the zero level dif-
ference, i.e. the difference between the two zero level of the staff gauges at
A and B, are obtained.
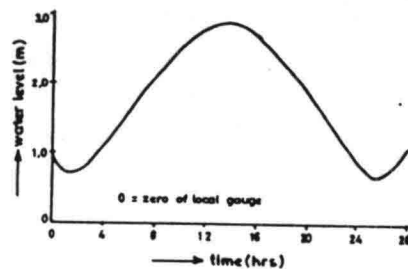
Fig. 7 Water level at point B   Fig. 8 Water level at point A
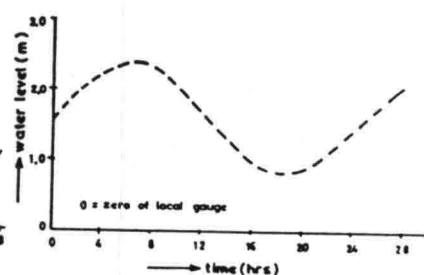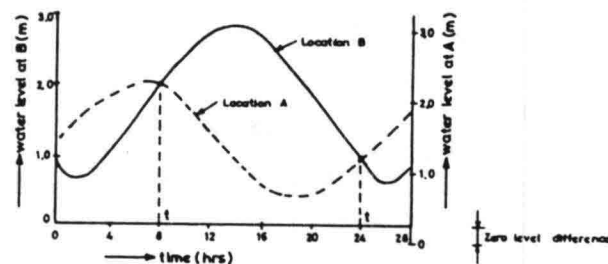
Fig. 9   Suitable position of zero levels of staff gauges at A and B

A better result of higher accuracy can be obtained by doing an iterative
process using R values related to the water levels at the moments I = 0 just
obtained by graphical solution.

3. CHECKING OF RESULT

Checking of result can be roughly done in 2 ways :

- Checking of the slope of the average water level.
  A negative slope of the average water level (the average downstream
  water level is higher than the average upstream water level), indicates
  incorrect result.

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- Using Chezy's formula
  For a constant water surface slope with respect to time (in the 16th and 28th hours in Fig. 9), $\frac{dv}{dt} = 0$ and hence equation (1) turns to Chezy's formula : $V = C\sqrt{RI}$.
  Checking is done by comparing the v values computed with this formula with the measured data.

4. FACTORS TO BE CONSIDERED IN RIVER STRETCH LENGTH DETERMINATION

- Upstream discharge
  Upstream discharge acts as a resistance to the propagation of the wave and in its turn decreases the length of the wave (L).
  To maintain $\ell' \ll L$ (ℓ' is the length of river stretch, in this case the distance between the water level stations), it is clear that ℓ' should be taken shorter when the upstream discharge increases.

- River dimension and roughness coefficient
  Effects similar to that caused by the upstream discharge appear in rivers with small dimension and high roughness.

- Tidal range
  As tidal range increases there will be an increase in $\frac{d^2h}{dx^2}$ (h and x represent wave height and wave length respectively). The part of the wave that can be assumed as a straight line is shorter than in case of a smaller tidal range. Again ℓ' must be taken shorter.

- River stretch location
  In the upstream reach of the river, the wave length decreases due to resistance during its propagation upstreamward. This is another consideration of taking a shorter ℓ'.

5. EXAMPLE AND COMPARISON

This hydraulic method has been used to determine benchmark levels along the downstream reach of Sebangau River in Central Kalimantan. See Figures 10, 11, 12, 13, 14 and 15.
The required measurements were executed in August/September 1980 (dry season conditions). The results have been compared with the results of accurate topographical work done in the same period.

Benchmark code	Benchmark location from river mouth (km)	Elevation (m)	
		Hydraulic method	Topographical survey
PS SB 0	4.6	+ 1.75	+ 1.75
SU 44	35.6	+ 1.53	+ 1.57
SK 7	63.5	+ 1.70	+ 1.71
DPMA 01	92.0	+ 2.44	+ 2.49
DPMA 02	113.5	+ 2.12	*)
DPMA X	132.3	+ 2.60	*)

*) no topographical work executed.

6. CONCLUSION

- The level differences between the results of the hydraulic and topographical method are very small especially in relation with the distance. From this it can be concluded that both methods have about the same accuracy.

- The topographical data used here are considered of exceptional high accuracy. In many other areas this appeared not to be the case. Hence usually the hydraulic method gives more accurate results than the topographical one.

- Compared with topographical work in such a difficult accessible area, the hydraulic method seems to be much easier and more practical.

7. ACKNOWLEDGEMENT

The authors wish to express their appreciation to Ir. P.van Groen, Team leader of the BTA-60 Project for his valuable assistance in the preparation of this paper.

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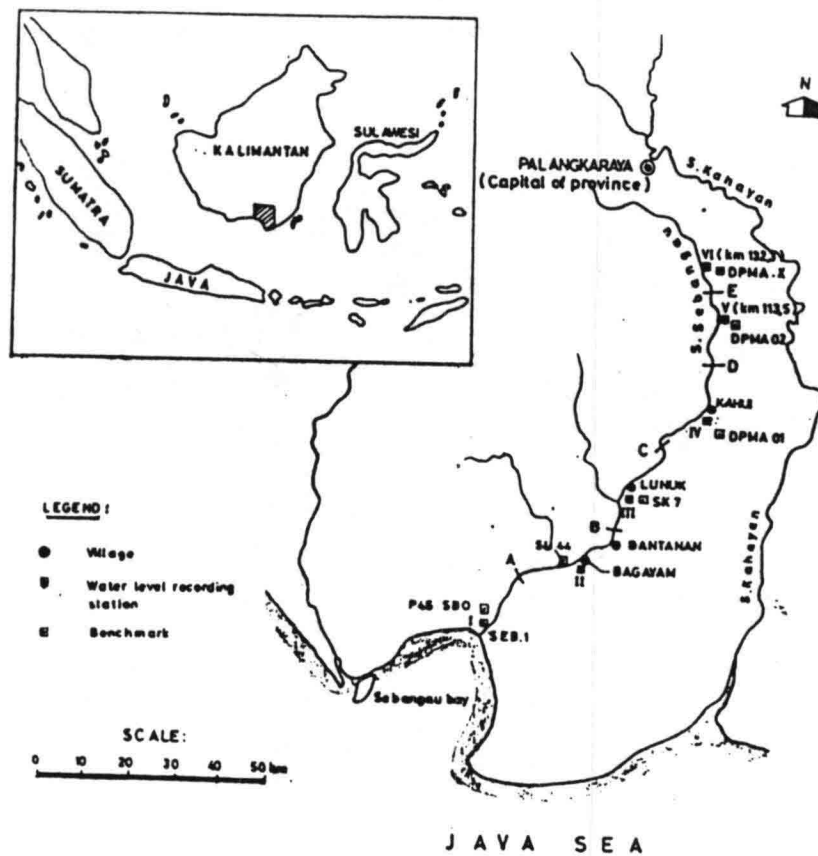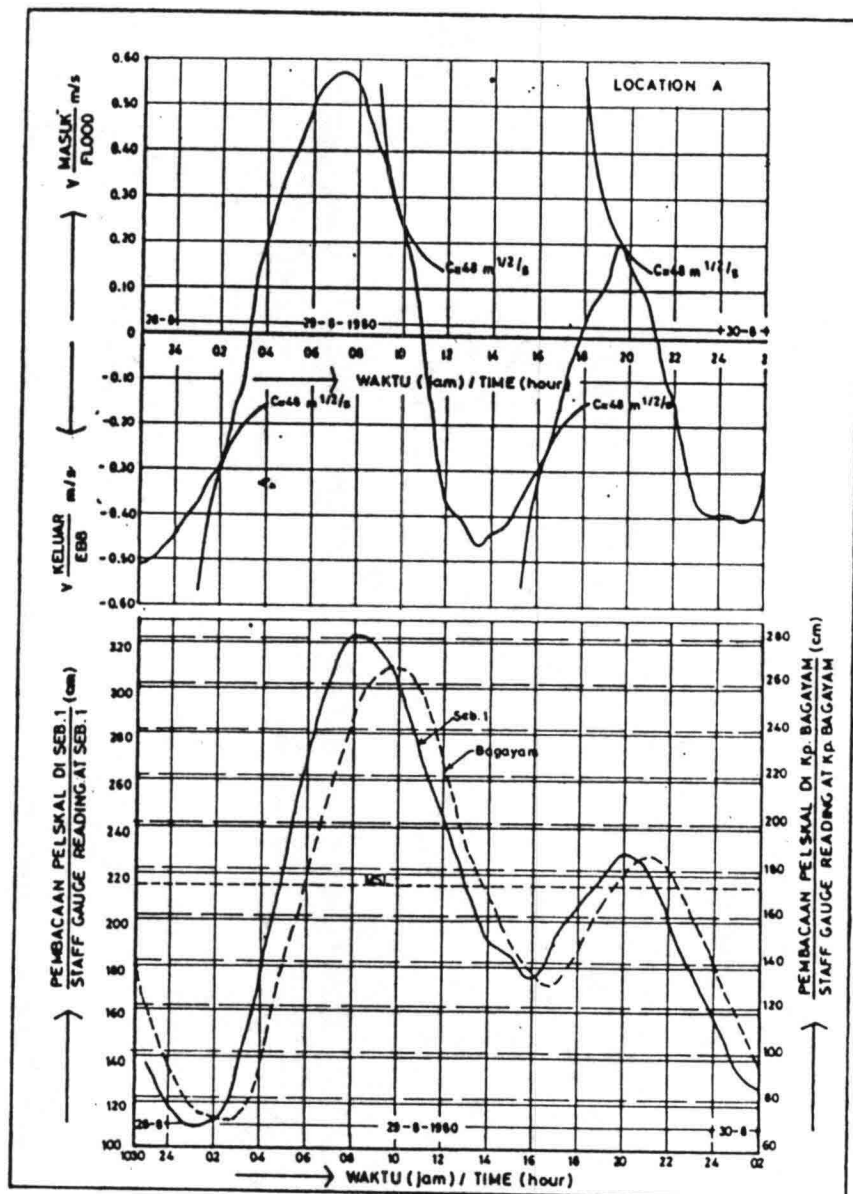
Fig. 10   Situation sketch

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Figure 11   Slope analysis of Sebangau River

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ANNEX   4

THE   INPUT   AND   OUTPUT   FILES   OF   RUN 04

Dimension of the canal system for Run 04
For the location see figure 18, all the levels are related
to the Project Reference Level (PRL)

1. The main river

Branch	x (km)		item: level,1(m)		; width,w(m)		n Manning
120	7.0	1	-15.02	-11.22	-11.02	-4.02	0.019
		w	115.0	260.0	350.0	505.	
132	11.0	1	-14.02	-11.02	-8.82	-1.82	0.020
		w	0.00	305.0	380.	590.	
536	8.80	1	-13.60	-11.00	-3.00	-1.00	0.022
		w	0.00	465.0	515.	535.	
671	11.0	1	-16.22	-16.12	-6.92	-5.92	0.024
		w	0.00	205.0	340.	370.	
672	10.0	1	-15.72	-13.62	-10.62	-6.42	0.023
		w	0.00	160.0	320.0	415.	
670	10.0	1	-13.72	-10.62	-6.42	-0.80	0.023
			0.00	320.0	415.0	415.	

2. The primary canals

Branch	x (km)						
121	2.40	1	-3.91	-3.41	-0.59	0.04	0.040
		w	0.00	18.1	35.0	40.0	
211	1.75	1	-3.66	-2.16	-1.26	-0.36	0.030
		w	15.0	80.9	86.4	89.0	
622	1.50	1	-4.65	-4.10	-2.00	-0.40	0.050
		w	0.00	24.0	39.0	53.1	

3. The secondary canals

Branch	x (km)						
122	2.00	1	-3.61	-2.21	-1.0	0.98	0.040
		w	10.0	15.8	28.8	38.7	
123	2.90	1	-3.65	-1.61	-1.09	-0.74	0.040
		w	10.0	18.0	24.5	28.7	
124	2.45	1	-3.61	-1.61	-1.11	0.39	0.040
		w	10.0	18.0	23.0	26.0	
125	2.45	1	-3.61	-1.61	-1.11	0.39	0.040
		w	10.0	18.0	23.0	26.0	

321	2.60	l	-4.40	-3.61	-2.11	-0.36	0.030
		w	0.00	3.00	73.4	75.0	
322	2.60	l	-3.61	-3.44	-1.94	-0.36	0.030
		w	0.00	3.00	73.2	75.0	
323	2.45	l	-3.30	-1.80	-1.26	-0.36	0.030
		w	3.00	22.5	45.5	46.5	
324	2.90	l	-3.22	-1.72	-1.26	-0.36	0.030
		w	3.00	18.2	36.2	37.3	
325	1.70	l	-3.07	-1.57	-1.26	-0.36	0.030
		w	3.00	12.0	16.4	31.8	
241	2.00	l	-3.66	-2.16	-1.26		0.030
		w	15.0	46.9	46.9		
424	2.12	l	-3.60	-2.10	-1.26		0.030
		w	2.00	26.3	26.4		
521	3.30	l	-3.47	-1.97	-1.26		0.030
		w	2.00	25.3	25.3		
522	1.80	l	-3.37	-1.87	-1.80	-0.36	0.030
		w	2.00	8.00	14.9	15.6	
523	1.38	l	-3.27	-1.77	-1.26	-0.36	0.030
		w	2.00	8.00	12.8	14.6	
528	2.96	l	-3.56	-2.09	-1.26	-0.36	0.030
		w	2.00	23.2	23.2	24.2	
529	2.40	l	-3.45	-1.95	-1.26		0.030
		w	2.00	19.2	19.2		
530	1.60	l	-3.39	-1.89	-1.26		0.030
		w	2.00	13.9	16.7		
531	1.60	l	-3.29	-1.79	-1.26	-0.36	0.030
		w	2.00	8.00	17.2	19.5	
822	1.40	l	-3.00	-2.80	-2.40	-0.40	0.040
		w	3.00	21.0	36.0	51.8	
823	2.05	l	-2.95	-2.75	-2.10	-0.40	0.040
		w	3.00	21.0	36.0	51.9	
824	1.22	l	-2.95	-2.75	-2.25	-0.40	0.045
		w	3.00	21.0	27.0	39.2	
825	1.17	l	-2.95	-2.45	-2.05	-0.40	0.045
		w	3.00	21.0	27.0	39.0	

826	1.00	1	-2.95	-2.15	-1.75	-0.40	0.045
		w	3.00	21.0	27.0	37.2	
722	1.27	1	-3.00	-2.80	-2.60	-0.40	0.035
		w	2.00	19.0	24.0	36.7	
723	1.35	1	-2.95	-2.45	-2.00	-0.40	0.030
		w	2.00	19.0	24.0	33.6	
724	1.84	1	-3.05	-2.90	-2.25	-0.40	0.045
		w	2.00	14.0	19.0	27.2	
725	1.24	1	-2.95	-2.25	-2.05	-0.40	0.045
		w	2.00	14.0	19.0	26.3	
726	0.95	1	-2.95	-2.25	-1.80	-0.40	0.045
		w	2.00	14.0	19.0	22.8	

---

4. The flushing canal

901	8.00	1	-3.58	0.37			0.025
		w	30.0	52.8			
902	7.00	1	-3.51	0.44			0.025
		w	0.01	0.01			
903	3.00	1	-3.46	0.49			0.025
		w	20.0	35.8			
904	3.50	1	-3.43	0.52			0.025
		w	20.0	35.8			
905	4.50	1	-3.39	0.56			0.025
		w	0.01	0.01			
906	5.00	1	-3.37	0.58			0.025
		w	20.0	35.8			

Tertiaries storage area
Levels are related to the Project Reference Level (PRL)

Node	Level (m)	Area (ha)	Level (m)	Area (ha)
112	-2.221	0.30	0.00	98.70
113	-1.410	0.58	0.00	507.00
114	-0.520	1.67	0.00	757.00
115	0.387	796.00	1.18	796.75
311	-2.112	4.06	-1.30	7.09

312	-3.131	4.08	-1.49	7.12
313	-2.029	6.05	-1.15	10.00
314	-2.118	2.17	-1.10	3.90
511	-1.720	3.70	-0.87	5.60
512	-2.040	1.70	-1.31	2.55
513	-1.820	4.73	-1.12	7.10
519	-2.470	2.74	-1.85	4.10
520	-2.200	3.50	-1.46	5.22
521	-2.090	5.87	-1.56	7.91
862	-2.500	0.00	-0.75	72.59
863	-2.400	0.00	0.00	66.00
864	-2.500	0.00	-0.25	53.00
865	-2.400	0.00	0.00	60.00
762	-2.400	0.00	-0.40	15.00
763	-2.400	0.00	-1.10	86.40
764	-2.400	0.00	-1.00	81.00
765	-2.400	0.00	0.00	15.00

Boundary conditions

-Downstream boundary condition, water levels at node 100:
t : time (hrs)   ; wl : water level (m)

----

t	wl	t	wl	t	wl	t	wl
0.0	-0.87	1.0	-1.02	2.0	-1.18	3.0	-1.35
4.0	-1.48	5.0	-1.56	6.0	-1.59	7.0	-1.70
8.0	-1.73	9.0	-1.33	10.	-0.83	11.	-0.38
12.	-0.13	13.	-0.05	14.	0.04	15.	0.07
16.	-0.03	17.	-0.13	18.	-0.24	19.	-0.38
20.	-0.58	21.	-0.62	22.	-0.68	23.	-0.74
24.	-0.87						

-Upstream boundary condition, discharges at branch 669:
t : time (hrs) ; Q : discharge (m3/s)

----

t	Q	t	Q	t	Q	t	Q
0.0	-2100.	1.0	-2300.	2.0	-2410.	3.0	-2470.
4.0	-2480.	5.0	-2480.	6.0	-2470.	7.0	-2440.
8.0	-2360.	9.0	-2180.	10.	-1500.	11.	-1300.
12.	-1320.	13.	-1360.	14.	-1440.	15.	-1570.
16.	-1600.	17.	-1700.	18.	-1740.	19.	-1760.
20.	-1840.	21.	-1930.	22.	-1970.	23.	-2030.
24.	-2100.						

-Internal boundary conditions:
Constant inflow Q (m3/s):

Node	Q	Node	Q	Node	Q
762	0.3	763	0.085	764	0.28
765	0.015	862	0.20	863	0.06
864	0.5	865	0.06	311	0.01
312	0.007	313	0.01	314	0.004
511	0.006	512	0.003	513	0.007
519	0.004	520	0.005	521	0.008
112	0.10	113	0.50	114	0.75
115	0.80				

-Concentration boundary conditions:

Downstream boundary condition , concentration at Node 100 =
0.00005 g/1;
Upstream boundary condition, concentration at Node 600 =
0.00005 g/1.
Internal boundary conditions:
Constant acid production perunit of time at storage node= Q*c
Q are as the above table for internal boundary flow and c=
0.016 g/1.

Initial conditions:

Water levels   = -0.40 m
Discharges   = -0.01 m3/s
Concentrations = 0.0001 g/1

Dispersion constant:
D1 = 20.0   D2= 0.0

Time step = 120 s

LAY-OUT OF OUTPUT

Q AND V IN BRANCHES

120	121	122	123	124	125	126	127	128	129
131	132	211	321	322	323	324	325	326	327
328	329	330	331	241	424	521	522	523	524
525	526	527	572	528	529	530	531	532	537
533	534	535	536	622	722	723	724	725	726
727	822	823	824	825	826	642	742	743	744
745	842	843	844	845	846	670	672	671	901
902	903	904	905	906	669				

H IN POINTS

600	612	611	601	602	662	702	703	704	705
706	707	762	763	764	765	802	803	804	805
806	807	862	863	864	865	201	202	301	302
303	304	305	306	311	312	313	314	315	401
501	502	503	504	511	512	513	514	541	515
516	517	518	519	520	521	522	523	101	102
103	104	105	106	107	112	113	114	115	100

S IN POINTS

600	612	611	601	602	662	702	703	704	705
706	707	762	763	764	765	802	803	804	805
806	807	862	863	864	865	201	202	301	302
303	304	305	306	311	312	313	314	315	401
501	502	503	504	511	512	513	514	541	515
516	517	518	519	520	521	522	523	101	102
103	104	105	106	107	112	113	114	115	100

Time:
/ 288.00
DISCHARGE IN BRANCHES

-4154.67	-28.16	-16.98	-4.72	-1.80	-.71	-10.14	-11.84	-2.22	-.
-.39	-3932.14	-51.29	-21.20	-15.08	-10.49	-6.85	-5.02	-1.64	-1.
-1.73	-.61	-4.71	-3.98	-24.18	-10.87	-8.44	-7.14	-6.14	-1.
-.41	-.75	-6.07	-5.71	-10.76	-9.03	-7.85	-6.71	-6.61	-6.
-.71	-.70	-.34	-3454.22	-51.16	-29.42	-18.80	-9.48	9.31	11.
13.57	-19.04	-8.69	-7.21	9.28	10.15	-2.26	-9.00	-8.84	-18.
-1.93	-9.65	-1.17	-16.31	-.68	10.69	-2280.26	-2811.90	-3264.32	1.
.00	-5.42	-11.03	.00	15.18	-2098.83				

VELOCITY IN BRANCHES

-.77	-.37	-.29	-.08	-.03	-.01	-.24	-.45	-.85	-.
-.01	-.74	-.25	-.12	-.09	-.15	-.12	-.15	-.04	-.
-.02	-.02	-.21	-.13	-.20	-.18	-.15	-.25	-.25	-.
-.02	-.01	-.25	-.19	-.20	-.21	-.23	-.23	-.22	-.
-.02	-.02	-.01	-.63	-.36	-.38	-.29	-.16	.19	-.
.45	-.19	-.09	-.09	.12	.15	-.09	-.09	-.10	-.
-.14	-.11	-.01	-.19	-.01	.35	-.47	-.30	-.36	.0
.00	-.07	-.14	.00	.19	.00				

WATERLEVEL IN POINTS

-.13	-.37	-.43	-.53	-.40	-.40	-.31	-.27	-.23	-.
-.33	-.34	-.31	-.26	-.23	-.27	-.37	-.35	-.34	-.
-.38	-.38	-.36	-.35	-.34	-.35	-.63	-.59	-.58	-.
-.54	-.51	-.48	-.48	-.58	-.57	-.54	-.51	-.48	-.
-.54	-.52	-.45	-.41	-.54	-.52	-.45	-.40	-.40	-.
-.49	-.45	-.40	-.53	-.49	-.45	-.40	-.40	-.80	-.
-.48	-.47	-.47	-.48	-.48	-.60	-.47	-.41	.46	-.

/ 288.00
ACIDITY IN POINTS

.0001	.0001	.0000	.0001	.0012	.0013	.0010	.0001	.0009	.000
.0004	.0003	.0017	.0007	.0010	.0012	.0005	.0003	.0004	.002
.0016	.0016	.0007	.0022	.0018	.0037	.0000	.0002	.0005	.000
.0004	.0004	.0003	.0000	.0005	.0004	.0004	.0003	.0003	.000
.0006	.0007	.0008	.0008	.0008	.0009	.0010	.0008	.0008	.000
.0009	.0011	.0011	.0013	.0012	.0013	.0011	.0012	.0001	.005
.0058	.0111	.0029	.0001	.0020	.0013	.0014	.0129	.0160	.000

DISPERSION COEFFICIENT IN BRANCHES:

219.3	24.1	18.1	7.0	4.4	1.	.0	.0	.0	
.0	196.3	15.3	9.4	5.8	8.	6.7	7.8	.0	
.0	.0	.0	.0	12.4	10.	8.7	14.6	14.5	
.0	.0	.0	.0	11.9	12.	13.2	13.1	.0	
.0	.0	.0	163.3	30.0	20	15.5	9.1	10.0	13.
.0	9.8	4.6	4.6	6.7	8.	.0	.0	.0	
.0	.0	.0	.0	.0	.0	125.4	91.6	114.7	
.2	4.2	8.5	.3	11.7	.0				

STATISTIC ELABORATION OF RESULTS

WATERLEVELS

POINT	AVERAGE	MAX	MIN
600	-.320	.503	-1.385
612	-.446	.399	-1.373
611	-.493	.356	-1.365
601	-.574	.292	-1.465
602	-.515	-.062	-1.044
662	-.514	-.062	-1.044
702	-.492	-.172	-.905
703	-.481	-.185	-.816
704	-.466	-.182	-.780
705	-.423	.002	-.905
706	-.439	.257	-1.208
707	-.435	.304	-1.229
762	-.492	-.172	-.905
763	-.481	-.185	-.816
764	-.467	-.182	-.780
765	-.422	.002	-.905
802	-.509	-.176	-.959
803	-.505	-.265	-.915
804	-.510	-.330	-.921
805	-.511	-.311	-.928
806	-.515	-.239	-.931
807	-.515	-.218	-.931
862	-.509	-.176	-.959
863	-.505	-.265	-.915
864	-.511	-.330	-.921
865	-.511	-.311	-.928
201	-.630	.190	-1.519
202	-.616	.158	-1.395
301	-.618	.154	-1.371
302	-.617	.147	-1.352
303	-.587	.101	-1.149
304	-.566	.002	-1.010
305	-.556	-.197	-.852
306	-.560	-.218	-.841
311	-.619	.154	-1.371
312	-.617	.147	-1.352
313	-.587	.101	-1.149
314	-.566	.007	-1.010
315	-.559	-.210	-.844
401	-.605	.133	-1.319
501	-.595	.110	-1.251
502	-.579	.068	-1.159
503	-.538	-.054	-.953
504	-.518	-.162	-.920
511	-.595	.110	-1.251
512	-.579	.068	-1.159
513	-.538	-.054	-.953
514	-.518	-.169	-.920
541	-.518	-.174	-.920
515	-.582	.076	-1.176
516	-.562	.003	-1.054

5:7	-.546	-.066	-.982
5:8	-.518	-.168	-.925
519	-.582	.076	-1.176
520	-.563	.003	-1.054
521	-.546	-.066	-.982
522	-.518	-.174	-.925
523	-.518	-.180	-.925
101	-.731	.111	-1.666
102	-.634	-.164	-1.125
103	-.583	-.420	-.783
104	-.574	-.369	-.790
105	-.567	-.293	-.803
106	-.562	-.229	-.834
107	-.562	-.227	-.835
112	-.638	-.164	-1.125
113	-.586	-.456	-.743
114	-.428	-.404	-.449
115	.457	.461	.453
100	-.770	.070	-1.730

DISCHARGES

BRANCH	AVERAGE	MAX	MIN
120	-2103.7870	5488.6290	-5112.6050
121	-2.2894	55.7420	-34.7867
122	-2.2118	31.7358	-22.2809
123	-1.7353	6.3014	-10.3203
124	-.9925	6.7451	-13.2875
125	-.5433	6.6507	-12.7218
126	-.0979	38.0941	-30.7816
127	-.4814	36.4690	-21.9195
128	-.7435	3.6028	-2.3151
129	-.4495	-.4117	-.4855
131	-.5434	6.4987	-12.8037
132	-2099.1460	5008.3520	-4820.0510
211	-1.1303	135.6033	-65.2907
321	.5182	65.3412	-27.9868
322	.5249	47.0141	-19.9295
323	.5328	34.2410	-13.8251
324	.5422	25.8280	-9.7363
325	.5551	22.8054	-8.1624
326	-.0070	6.3432	-3.0610
327	-.0069	6.2739	-2.9616
328	-.0098	7.3927	-3.8890
329	-.0039	2.5513	-1.3163
330	.5453	22.0800	-7.9603
331	.5450	20.4443	-7.4010
241	-1.6505	57.4986	-31.3258
424	-.7304	26.4255	-14.1943
521	-.7244	20.4337	-11.2047
522	-.7207	17.5493	-9.5786
523	-.7134	14.5124	-8.3667
524	-.0060	4.7472	-2.2747
525	-.0030	2.3863	-1.0158

526	-.0070	3.9441	-2.4708
527	-.7144	14.2104	-8.2776
572	-.7144	12.5461	-7.7317
528	-.9186	25.8729	-14.2723
529	-.9159	21.8953	-12.1929
530	-.9110	18.6251	-10.6932
531	-.9014	15.4229	-9.2222
532	-.9029	14.9966	-9.0864
537	-.9029	13.4973	-8.5473
533	-.0040	3.2374	-1.6318
534	-.0050	3.1108	-1.9836
535	-.0080	4.1823	-2.8224
536	-2098.2740	4152.3500	-4457.2130
622	-2.0792	121.4081	-67.4701
722	-2.8234	48.0145	-28.4760
723	-2.5231	34.2621	-23.6332
724	-2.4394	11.8908	-10.6362
725	-2.1628	12.8843	-31.6407
726	-2.1467	15.6073	-35.3774
727	-2.1454	17.6578	-37.9505
822	.8025	60.9839	-35.9102
823	1.0017	30.4116	-23.3539
824	1.0573	26.7529	-20.2544
825	1.5563	12.7725	-19.4619
826	1.6162	13.6966	-21.2142
642	-.0600	12.8828	-10.0940
742	-.3000	13.1016	-20.5882
743	-.0850	22.4236	-12.3840
744	-.2800	42.4737	-22.6532
745	-.0150	10.0711	-2.8918
842	-.1999	34.8950	-26.8085
843	-.0600	7.4770	-9.9317
844	-.4997	45.6856	-32.5143
845	-.0600	2.6673	-5.0911
846	1.6166	14.4723	-22.2218
670	-1938.9740	-540.1595	-2696.4930
672	-2096.4960	926.7441	-3281.1340
671	-2095.0110	2724.1010	-3894.6100
901	-.5458	6.3098	-16.0841
902	-.0001	.0001	-.0001
903	-.7140	11.4318	-7.3089
904	-1.6158	23.0306	-14.9475
905	-.0001	.0003	-.0005
906	-2.1615	20.0704	-40.9845
669	-1947.9160	-1300.3330	-2480.0000

ACIDITIES

POINT	AVERAGE	MAX	MIN
600	.0001	.0001	.0001
612	.0001	.0001	.0001
611	.0000	.0000	.0000
601	.0001	.0001	.0001
602	.0011	.0024	.0002
662	.0014	.0017	.0012
702	.0004	.0011	.0000
703	.0008	.0018	.0001
704	.0008	.0011	.0004
705	.0009	.0014	.0005
706	.0008	.0013	.0002
707	.0008	.0018	.0003
762	.0020	.0026	.0015
763	.0008	.0010	.0007
764	.0011	.0012	.0009
765	.0011	.0012	.0011
802	.0006	.0016	.0000
803	.0015	.0032	.0000
804	.0015	.0021	.0002
805	.0024	.0028	.0020
806	.0019	.0022	.0011
807	.0020	.0023	.0015
862	.0010	.0019	.0007
863	.0022	.0024	.0020
864	.0023	.0034	.0017
865	.0038	.0040	.0035
201	.0000	.0001	.0000
202	.0004	.0012	.0000
301	.0004	.0005	.0001
302	.0004	.0005	.0003
303	.0003	.0005	.0002
304	.0002	.0004	.0001
305	.0002	.0004	.0001
306	.0002	.0005	.0000
311	.0004	.0005	.0004
312	.0004	.0004	.0004
313	.0003	.0004	.0003
314	.0003	.0004	.0003
315	.0002	.0003	.0001
401	.0004	.0009	.0000
501	.0006	.0008	.0003
502	.0007	.0009	.0005
503	.0008	.0011	.0006
504	.0008	.0010	.0006
511	.0008	.0008	.0006
512	.0009	.0009	.0008
513	.0009	.0010	.0008
514	.0009	.0011	.0007

514	.0010	.0014	.0006
515	.0010	.0016	.0004
516	.0009	.0012	.0006
517	.0012	.0018	.0007
518	.0011	.0015	.0008
519	.0012	.0013	.0011
520	.0011	.0012	.0010
521	.0013	.0014	.0012
522	.0013	.0016	.0010
523	.0015	.0020	.0011
101	.0001	.0001	.0001
102	.0022	.0048	.0002
103	.0014	.0051	.0000
104	.0109	.0130	.0082
105	.0064	.0105	.0020
106	.0024	.0071	.0000
107	.0020	.0028	.0016
112	.0013	.0017	.0012
113	.0015	.0017	.0012
114	.0135	.0146	.0126
115	.0160	.0160	.0160
100	.0001	.0001	.0001

ACID TRANSPORT

BRANCH	AVERAGE	BRANCH	AVERAGE
120	-.1840	121	-.0254
122	-.0241	123	-.0168
124	-.0057	125	.0008
126	-.0015	127	-.0077
128	-.0118	129	-.0072
131	.0008	132	-.1588
211	-.0011	321	.0007
322	.0005	323	.0004
324	.0005	325	.0005
326	-.0001	327	-.0001
328	-.0001	329	-.0001
330	.0005	331	.0004
241	-.0019	424	-.0005
521	-.0006	522	-.0007
523	-.0007	524	.0000
525	.0000	526	.0000
527	-.0007	572	-.0009
528	-.0015	529	-.0016
530	-.0016	531	-.0017
532	-.0017	537	-.0018
533	.0000	534	.0000

535	.0000	536	-.1556
622	-.0204	722	-.0099
723	-.0051	724	-.0038
725	.0005	726	.0007
727	.0006	822	-.0096
823	-.0064	824	-.0056
825	.0022	826	.0031
642	-.0009	742	-.0048
743	-.0013	744	-.0043
745	-.0002	842	-.0032
843	-.0007	844	-.0078
845	-.0009	846	.0031
670	-.1326	672	-.1316
671	-.1355	901	-.0004
902	.0000	903	-.0010
904	-.0030	905	.0000
906	.0007	669	.0000

END OF CALCULATION

ANNEX   5
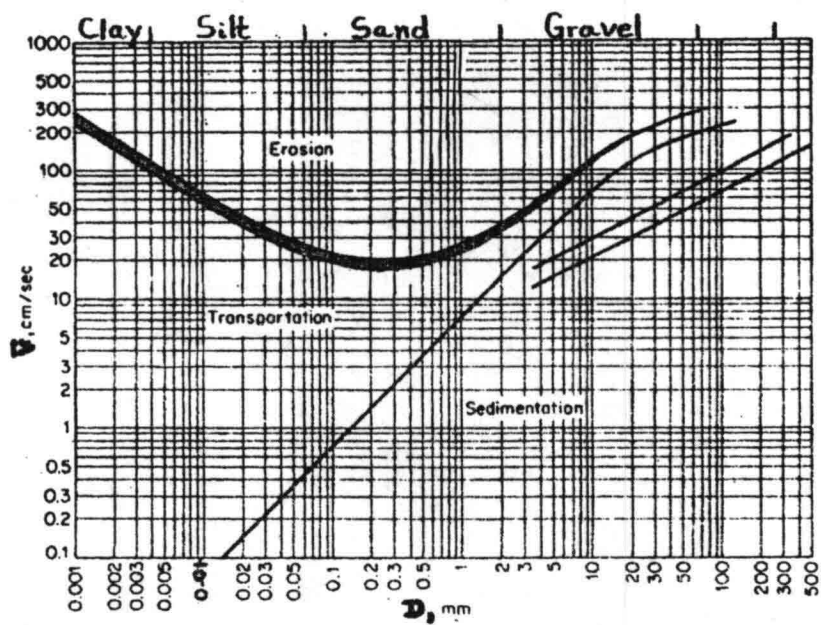

HJULSTROM´S GRAPH   (EROSION-DEPOSITION CRITERIA)

Fig. 5.1 Erosion-deposition criteria (Hjulström, 1935)