

REPUBLIC OF COLOMBIA

MINISTRY OF PUBLIC WORKS

ADENAVI

KINGDOM OF THE NETHERLANDS

DIRECTORATE OF INTERNATIONAL TECHNICAL ASSISTANCE

MISIÓN TÉCNICA COLOMBO-HOLANDESA

NEDECO

# **río magdalena and canal del dique survey project**

**1973**



## PREFACE

The Administrative Agreement between the Colombian and Netherlands Government for the execution of the Rfo Magdalena and Canal del Dique Survey Project was signed on 5th October, 1970.

The purpose of the project was to study a number of constraints to navigation on the rivers and to recommend technical solutions thereto. During the course of the investigations the study was given a somewhat wider scope and more scientific data were taken into account, because:

- a better knowledge and understanding of the hydrological and morphological regime of the rivers was essential, if satisfactory solutions to the navigational bottlenecks were to be found
- a full understanding of the regime of the rivers would be essential for the future development plans of the Magdalena River Basin
- the continuously changing conditions along the Rfo Magdalena imply that some of the recommended solutions may have only a temporary value; hence a knowledge of the background reasoning leading to the solutions would be of considerable value in later years.

The concept of the study was to a certain extent also influenced by the fact that during the investigation two other studies were initiated:

- Magdalena-Cauca River Flood Protection Study, and
- the Magdalena River Area Transport Study.

Both studies are inter-related with the river survey project and it is evident that a basic insight into the hydrological and morphological features of the river would provide valuable data for the flood protection as well as for the transport study.

The Report on the Rfo Magdalena and Canal del Dique Survey Project has been divided into four parts:

- Part I contains a description of the studies carried out and a summary of the recommendations made
- Part II contains the theoretical background of the recommended solutions to the navigational bottlenecks, together with all the relevant data that were collected. This Part may be considered as the scientific justification of Part III and provides deeper insight into the hydrological and morphological regime of the rivers

Part III contains the recommended technical solutions for the improvement of a number of navigational constraints on the rivers, whilst

Part IV contains a hydrographic manual.

NEDECO acknowledge with gratitude the excellent co-operation received from its Counterparts and from all Colombians in Government services as well as in private business, who assisted in providing the facts and details required for this study.

It is sincerely hoped that this Report may contribute to the economic development of the Río Magdalena Basin and, in particular, to the development of navigation along the Río Magdalena and the Canal del Dique.

## CONTENTS

	Page
<b>PREFACE</b>	
<b>PART I GENERAL</b>	
Chapter 1 THE INVESTIGATION (ADMINISTRATIVE)	1
1.1. The assignment and its antecedents	1
1.2. Terms of reference	2
1.3. The organization	3
1.4. Reports	7
Chapter 2 THE INVESTIGATION (TECHNICAL)	8
2.1. Defects of the river with relation to navigation	8
2.2. The execution of the project	11
Chapter 3 SUMMARY OF RECOMMENDATIONS	13
<b>PART II HYDROLOGY AND MORPHOLOGY</b>	
Chapter 1 GENERAL	19
Chapter 2 HYDROLOGY	20
2.1. Introduction	20
2.2. Rainfall and evaporation	20
2.3. Water-levels	22
2.3.1. Introduction	23
2.3.2. The hydrographs	24
2.3.3. Statistical elaboration of water-levels	25
2.3.4. Presentation of water-level data	30
2.3.5. Reduction-levels	46
2.4. Discharges	49
2.4.1. Introduction	49
2.4.2. Statistical elaboration of discharges	53
2.5. Relation-curves	54
2.6. Computation of backwater-curves	59
2.7. Prediction of water-levels	61
Chapter 3 MORPHOLOGY	64
3.1. General	64
3.2. Sediment transport	64
3.2.1. Bed-load	64
3.2.2. Suspended-load	65
3.2.3. Wash-load	67

3.3. Collected and elaborated data	67
3.3.1. Introduction	67
3.3.2. The discharge and sediment-transport measurements	68
3.3.3. The suspended-load measurements	78
3.3.4. The suspended-load concentrations	80
3.3.5. The wash-load concentrations	84
3.3.6. Bed-samples and grain-sizes	88
3.3.7. Longitudinal profiles	93
3.3.8. Local soundings	95
3.3.9. Miscellaneous information	96
3.4. Bed-forms, channel roughness	100
3.4.1. General	100
3.4.2. Grain roughness	100
3.4.3. Bed-form roughness	101
3.5. Selection of the transport-equation	110
3.5.1. Introduction	110
3.5.2. Selection of the transport-equation	111
3.5.3. The discharge, channel roughness, water-level gradient and sediment-transport relations	118
3.5.4. Procedures when no measurements are available	125
3.6. One-dimensional morphological computations	129
3.6.1. General	129
3.6.2. Mathematical base	130
3.6.3. The influence of the suspended-load on the behaviour of the river-bed	136
3.6.4. Application	137
3.6.5. Selection of a dominant discharge	142
3.7. Three-dimensional phenomena	142
3.7.1. Introduction	142
3.7.2. Plan-form	143
3.7.3. Flow in river bends	149
3.8. Applications	152
3.8.1. Introduction	152
3.8.2. Test case of the Río Sogamoso Confluence and the Río Regla Confluence	152

**PART III IMPROVEMENT FOR NAVIGATION OF THE RÍO MAGDALENA AND  
THE CANAL DEL DIQUE**

Chapter 1	GENERAL	157
Chapter 2	RIVER IMPROVEMENT	158
	2.1. Introduction	158
	2.2. Aids to navigation	158
	2.2.1. Measures taken by river operators	158
	2.2.2. Channel patrol service	160

2.3. Temporary river-works	168
2.3.1. Introduction	168
2.3.2. Temporary constructions	169
2.3.3. Dredging equipment (general)	169
2.3.4. Dredging equipment at present in use on the Río Magdalena and the Canal del Dique	180
2.3.5. New equipment for the Río Magdalena	182
2.3.6. Organization of dredging	187
2.4. Permanent river improvement	187
2.4.1. Introduction	187
2.4.2. Regulation of the plan-form	190
2.4.3. Normalization of the river width	199
2.4.4. Regulation of the river-bed	202
2.5. Constructions	205
2.5.1. Introduction	205
2.5.2. Design criteria of a protection	206
2.5.3. Closure of secondary branches	216
2.5.4. Examples of constructions	216
Chapter 3    RÍO MAGDALENA	224
3.1. Introduction	224
3.2. La Dorada-Puerto Inmarco (km 884-773)	225
3.2.1. General description and design criteria	225
3.2.2. Temporary improvement by means of dredging	233
3.2.3. Improvement of La Dorada-Pto. Salgar	234
3.2.4. Confluences of the Río Magdalena and the Ríos Negro and La Miel	253
3.2.5. River-crossing km 833	256
3.2.6. Puerto Triunfo	257
3.2.7. Puerto Boyacá	260
3.2.8. River-crossings near km 780	263
3.2.9. Confluence of the Río Magdalena and the Río Nare	264
3.3. Puerto Inmarco-Puerto Berrío (km 773-730)	266
3.3.1. General description and design criteria	266
3.3.2. Temporary improvement by means of dredging	270
3.3.3. Río Magdalena upstream Pto. Berrío (km 735-730.5)	271
3.4. Puerto Berrío-Barrancabermeja (km 730-630)	274
3.4.1. General description and design criteria	274
3.4.2. Temporary improvement by means of dredging	278
3.4.3. The section between Pto. Berrío and the Río Regla Confluence (km 730-710)	278
3.4.4. River improvement near the Río Regla Confluence (km 711-706)	279

3.4.5. The Río Magdalena between the Río Carare Confluence and Chucurí (km 675-659)	285
3.5. Barrancabermeja-Gamarra (km 630-475)	288
3.5.1. General description and design criteria	288
3.5.2. Temporary improvement by means of dredging	294
3.5.3. The access to the Barrancabermeja Port	294
3.5.4. The "Hidro Electrica Lebrija" (km 626) and navigation between km 624 and km 621	295
3.5.5. The Río Sogamoso Confluence	296
3.5.6. The access to Pto. Wilckes	302
 Chapter 4 CANAL DEL DIQUE	 303
4.1. General	303
4.2. The canal	305
4.2.1. Present dimensions of the canal	305
4.2.2. Required dimensions (MITCH design-section)	306
4.2.3. Bends and bend-radius	310
4.3. Calamar (Canal del Dique bifurcation)	311
4.3.1. Introduction	311
4.3.2. Sedimentation in the present situation	311
4.3.3. Comparison of different solutions	313
4.4. Lower Canal del Dique (Pasacaballos)	318
4.4.1. Introduction	318
4.4.2. Division of discharge and sediment-load along the canal	318
4.4.3. Siltation near Pasacaballos	324
4.4.4. Dredging of the Lower Canal del Dique	331
 Chapter 5 DREDGING PROGRAMME AND PHASING OF RIVER-WORKS	 333
5.1. Introduction	333
5.2. Dredging programme	333
5.2.1. The organization	333
5.2.2. Amounts to be dredged and required capacity of the dredge fleet	334
5.3. Phasing of the river-works	336
 <b>PART IV HYDROGRAPHIC MANUAL</b>	
 Chapter 1 GENERAL	 337
 Chapter 2 INSTRUMENTS	 338
2.1. Introduction	338
2.2. Sextant	338
2.3. Theodolite	339
2.4. Levelling instrument	340
2.5. Rangefinder	341

2.6. Echo-sounder	342
2.7. Pendulum current meters	344
2.8. Propellor current meters	345
2.9. Water sampler	346
2.10. Delft bottle (DF)	347
2.11. Bed-load transport meter "Arnhem"	349
2.12. Bottom grab	349
2.13. Turbidity meter	350
2.14. Conductivity meter	351
2.15. Visual accumulation tube	352
 Chapter 3 LOCATION FIXING	 354
3.1. Triangulation	354
3.2. Local position fixing	356
3.2.1. With two sextants	356
3.2.2. With one sextant and a leading line	357
3.2.3. With one rangefinder and a leading line	358
3.2.4. With one sextant and a rangefinder	358
3.2.5. With one compass and a sextant	359
3.2.6. With one compass and a rangefinder	359
3.2.7. With one compass only	360
 Chapter 4 MEASUREMENTS AND ELABORATIONS	 362
4.1. Water-levels	362
4.1.1. Introduction	362
4.1.2. Gauges	362
4.1.3. Selection of gauge sites	364
4.2. Soundings and route maps	365
4.2.1. Introduction	365
4.2.2. Reduction-level	365
4.2.3. Preparations for soundings	366
4.2.4. Actual soundings	367
4.2.5. Elaborations	368
4.2.6. Charts involved	369
4.2.7. Route maps	371
4.2.8. Kilomatrage	371
4.2.9. Processing of route maps	372
4.3. Discharges	373
4.3.1. Discharge per unit width	373
4.3.2. Total discharge	375
4.3.3. Tidal effects	377
4.4. Sediment transport	381
4.4.1. Introduction	381
4.4.2. Sampling methods	382
4.4.3. Elaboration of samples	383

4.5. Salinity	384
4.6. Grain-sizes	385
<b>REFERENCES</b>	<b>389</b>
<b>LIST OF MAIN SYMBOLS</b>	<b>395</b>

**PART I**

**GENERAL**

## CONTENTS

	Page
<b>PART I GENERAL</b>	
Chapter 1 THE INVESTIGATION (ADMINISTRATIVE)	1
1.1. The assignment and its antecedents	1
1.2. Terms of reference	2
1.3. The organization	3
1.4. Reports	7
Chapter 2 THE INVESTIGATION (TECHNICAL)	8
2.1. Defects of the river with relation to navigation	8
2.2. The execution of the project	11
Chapter 3 SUMMARY OF RECOMMENDATIONS	13

## Chapter 1

### THE INVESTIGATION (ADMINISTRATIVE)

#### 1.1. THE ASSIGNMENT AND ITS ANTECEDENTS

Early in 1969 the Colombian Departamento Nacional de Planeación (DNP) presented to the Royal Embassy of the Netherlands in Bogotá a list of projects to be considered for execution within the framework of the Agreement on Bilateral Technical Assistance between the Colombian and Netherlands Governments for the period 1969 through 1971. In this list of projects figured a project entitled "Mejoras al Río Magdalena y al Canal del Dique". The pertaining general conditions, scope of the study and provisional estimate of required financial contributions from Colombia and the Netherlands were worked out by officers of the Asociación Nacional de Navieros (ADENAVI) and the Colombian Ministerio de Obras Públicas (MOP), in concert with Mr. T. Douma, a Dutch expert from the Delft Hydraulics Laboratory (DHL), at that time attached to the Centro de Estudios Técnicos e Investigaciones Hidráulicas (CETIH) of the Universidad de los Andes in Bogotá.

In October, 1969, the Directorate for International Technical Assistance (DITH) of the Netherlands Ministry of Foreign Affairs sent Mr. D. Gersie of the DHL to Colombia on a fact-finding mission in order to discuss details of the project with representatives of DNP, MOP and ADENAVI. The terms of reference for this mission were drawn up by DITH in consultation with the Netherlands Engineering Consultants (NEDECO).

Mr. D. Gersie, Deputy Head of the Site Investigation Service of the DHL, sometimes accompanied by Mr. H.J. du Marchie Sarvaas, then First Secretary of the Royal Embassy of the Netherlands in Bogotá, had many discussions with DNP, MOP, and ADENAVI officers in Bogotá and Barranquilla and visited the Río Magdalena (from La Dorada to Puerto Wilches) and the Canal del Dique area.

In February, 1970, Mr. J.G.H.R. Diephuis, Head of the Site Investigation Service of the DHL, visited Colombia and had additional talks on the project with the Embassy and with the earlier-mentioned Colombian authorities, especially on the financial aspects of the study (which could be more accurately evaluated after Mr. Gersie's mission) and on the question whether the required survey vessel should preferably be built in Colombia or in the Netherlands.

Based on the information included in the project proposal of DNP and the additional information obtained from the missions of Mr. Gersie and Mr. Diephuis, DITH drew up a draft Administrative Agreement. This draft, together with the Schedule of Operations (being an integral part of the Agreement), was slightly amended and then approved by DNP and thereafter presented by the Netherlands Government to the Colombian Ministry of Public Works on September 17, 1970. On September 24, 1970, the Agreement was formally signed in Bogotá by His Excellency Dr Argelino Durán Quintero, Minister of Public Works, and Dr. Jorge Ruiz Lara, Head of DNP, for the Colombian Government, and

## I, 1.1

by Mr. H.J. du Marchie Sarvaas, at that time Chargé d'Affaires for the Government of the Netherlands.

Meanwhile, and in anticipation of the signing of the Agreement, several actions had already been undertaken to ensure that the activities could start according to the time schedule. These activities, in chronological order, were:

- As soon as it became clear that it would be preferable for the required survey vessel to be constructed in the Netherlands, the Naval Architects and Consulting Engineers Propulsion N.V. were commissioned on May 1, 1970, to design a survey vessel according to specifications provided by NEDECO. At a later stage Propulsion was also commissioned to ask for tenders and, thereafter, to supervise the building of the vessel by the selected shipyard.

- As team-leader, Mr. A. Prins, Scientific officer of the Delft University of Technology (THD) was appointed. He made a preliminary visit to Colombia from May 20 to June 11, 1970, to discuss with Mr. Federico Holguín, Director of ADENAVI, a detailed Schedule of Operations. During his visit to Colombia he was accompanied by Mr. W. Boiten, a Dutch expert still working with CETIH but already earmarked for early activities for the Río Magdalena study.

- Early in July, 1970, Mr. W. Boiten moved from Bogotá to Cartagena and started on preparatory work for the measurements to be done in the Canal del Dique area.

- Tenders for the construction of the survey vessel were received from four Dutch shipyards in August, 1970.

- Part of the hydrographic measuring equipment to be used in the Canal del Dique area was ordered in the Netherlands and shipped to Colombia.

On October 5, 1970, after the formal signing of the Administrative Agreement between both Governments, a contract to build the survey vessel was signed between the Netherlands Government and the Visscher Shipyard in Zwartsluis. Meanwhile, it had become clear that, due to the time necessary for the construction of the vessel and its transport to Colombia, actual measurements on the Río Magdalena could not start before mid-1971. The time schedule, outlined in the Administrative Agreement and Schedule of Operations, as signed in September, 1970, was adapted accordingly to allow for full activities as from about July, 1971. In compliance with the contents of the Schedule of Operations, the execution of the study was delegated by the Colombian and Netherlands Governments to ADENAVI and NEDECO respectively. The agreement between MOP and ADENAVI was signed in Bogotá on March 4, 1971, and that between the Government of the Netherlands and NEDECO was signed in The Hague on December 7, 1970, although NEDECO had already been involved in the earlier-mentioned activities since 1969.

The execution of the project is described in Para. 2.2 of Part I of this Report.

## 1.2. TERMS OF REFERENCE

Having decided to co-operate in a survey project in the field of river transport on the Río Magdalena and the Canal del Dique in Colombia, and having regard to the provisions of Article III of the Agreement concerning Technical Co-operation between the

Kingdom of the Netherlands and the Republic of Colombia, signed in Bogotá on July 19, 1966, both Governments agreed on the following terms of reference:

"The two parties shall co-operate in surveying certain stretches of the Río Magdalena and the Canal del Dique, within the framework of a project to be known as: The Río Magdalena and the Canal del Dique Survey Project" (Article I.1 of the Administrative Agreement).

"The purpose of the Project is to provide the Colombian Authorities recommendations for improving, such with relatively limited means and at short notice, the navigability of the Río Magdalena and the Canal del Dique, the protection of the inhabitants of the banks of the rivers mentioned and their properties, and the reliability of the harbour installations" (Article I.2).

"The project will be carried out on several stretches of the Río Magdalena between La Dorada and Gamarra and the Canal del Dique" (Article I.3); and

"The aforesaid co-operation between the two parties is planned to last approximately two years as of July 1st, 1971" (Article I.4).

In the Schedule of Operations, which formed an integral part of the Agreement, it is stated that the purpose of the Project will be carried out by:

- Evaluating existing data on the Río Magdalena and studying the hydrological, hydraulic and morphological aspects of the river based on those data;
- Observing on the spot those phenomena of which no data or insufficient data are available, but which have to be known for the proper execution of the Project;
- Planning and preliminary designing of such structures or dredging schemes as will be recommended to attain the required improvements;
- Designing and executing a limited number of test structures, or executing trial dredgings if these are recommended and fit into the frame of the present Agreement;
- Drawing up recommendations regarding the system of beacons and prediction of depths; and
- Training of counterpart staff, especially in relation to specific measurements.

A detailed description of the present bottlenecks in the river which had to be studied is given in Para. 2.1 of Part I of this Report.

At the end of 1972 the Colombian Government requested a prolongation of the Project, during which experimental dredging will take place and some visits of Dutch experts to Colombia will enable further advice to be given.

### 1.3. THE ORGANIZATION

For the execution of the study, as far as activities in Colombia are concerned, the Misión Técnica Colombo-Holandesa (MITCH) was established (see Figure 1.3.1).



MITCH team in front of Barrancabermeja office

As from June, 1971, the MITCH team was housed in an adequate office in Barrancabermeja, rented by ADENAVI. Previously from July, 1970, a small office in Cartagena was used as a base for the work to be done on the Canal del Dique, but was abandoned in April, 1972.

The activities of MITCH and its contact with the Colombian authorities were coordinated by a committee, the Comité Coordinador, which consisted of representatives of ADENAVI, DNP and MOP (see Figure 1.3.1). Co-directors of the study held meetings with the Comité Coordinador nearly every month. During some of these meetings a representative of the Royal Embassy of the Netherlands was also present and in some cases the Netherlands project coordinator when his visit to Colombia coincided with those meetings.

In the Netherlands the activities were in the hands of NEDECO, with Mr. C.R. Kras, Deputy Director of NEDECO, in charge of the general conduct. Mr. D. Gersie was appointed project coordinator and, as such, acted as a representative of NEDECO in all contacts between MITCH and the NEDECO home office, and as the daily supervisor of the project.

Mr. Gersie paid regular visits to Colombia for discussions with MITCH, the Comité Coordinador, and the Royal Embassy of the Netherlands in Bogotá to discuss technical and organizational features of the Project.



Part of MITCH team with crew of "Explorador"

In February and September, 1972, the Ambassador of the Netherlands, the project coordinator and the co-director, visited the Minister of Public Works, His Excellency Dr. Argelino Durán Quintero, to report verbally on the progress of the study and to discuss some pending problems.

Short visits to Colombia (in combination with visits to other countries) were made by Mr. J.G.H.R. Diephuis, as already mentioned in Para. 1.1, from February 13 to 16, 1970, and by Mr. C.R. Kras from March 30 to April 5, 1972. In August, 1972, Mr. M.M. van Poll, of DITH, visited Colombia and discussed in Barrancabermeja details of the project organization with both co-directors.

During the study two experts from the Netherlands visited Colombia to advise the team on specialized items:

- Mr. J. Glas, dredging expert, visited Barrancabermeja, the Canal del Dique area and Barranquilla from January 10 to February 22, 1972, in connection with a dredging experiment on the Río Magdalena near the Sogamoso Confluence and to make a general survey of dredging techniques at present applied on the river and the Canal del Dique; and

- Dr. M. de Vries, Coordinator of Scientific Research of the DHL, who stayed in Colombia from August 12 to August 23, 1972, to discuss with the team-members the elaboration of the measurements and especially the findings of the sediment-transport measurements.

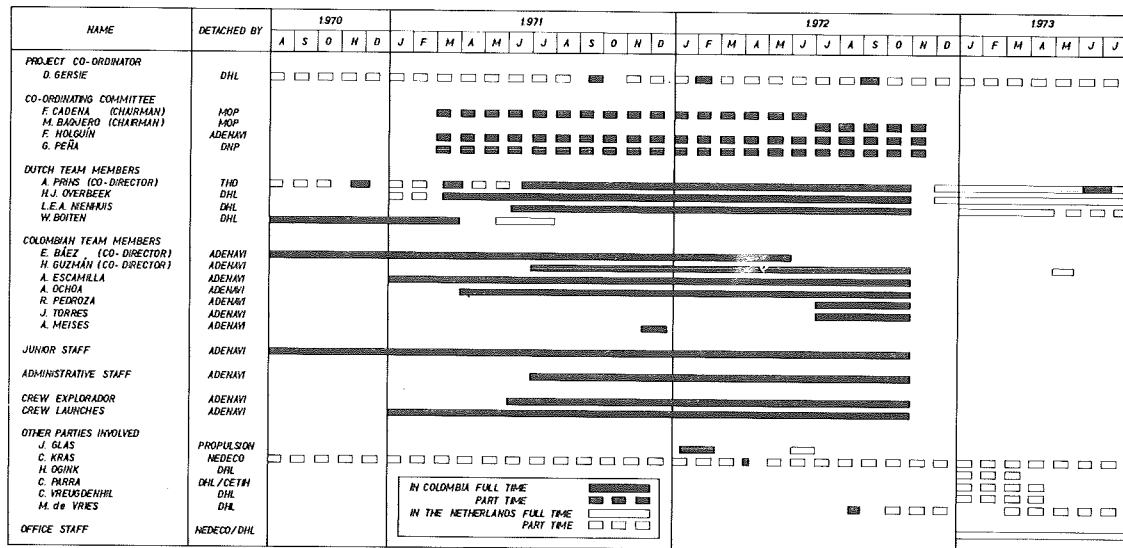


Figure 1.3.1 Bar chart of Persons involved in the Río Magdalena and Canal del Dique Survey Project

Capt. F.Ch. Hayes, Chief Hydrographer of the DHL, arranged for the purchase and forwarding of all hydrographic and hydrological measuring equipment sent to Colombia during the course of the study. He also maintained close contact with the Visscher Shipyard during the construction of the survey vessel. The survey vessel made its trial runs on March 30 and 31, 1971, and after being christened "Explorador" by Sra. Marieluz de Navás, the wife of Dr. Alberto Navas de Brigard, Chargé d'Affaires of Colombia in The Hague, was formally delivered to the Netherlands Government on April 2, 1971. The "Explorador" was shipped to Colombia on April 14, 1971, and arrived in Barranquilla on April 27. Trial runs with the vessel on the Río Magdalena took place in the second half of May, 1971. The ADENAVI crew was trained in handling the ship by Messrs. M. Visscher and J.A. Visscher of Visscher' Shipyard who, for this purpose, stayed in Barranquilla from May 23 to May 29, 1971. Representing the Netherlands Government His Excellency Dr. J. Varekamp, Ambassador in Colombia, transferred the "Explorador" in Barrancabermeja on July 3, 1971, by lowering the Dutch colours while the President of the Republic of Colombia, His Excellency Dr. Misael Pastrana Borrero, accepted the ship by hoisting the Colombian flag.

In the course of 1972 MOP installed a Dirección General de Navegación Fluvial y Puertos and MITCH (as from January 1, 1973, continuing under the name "Unidad de Estudios Fluviales") was administratively fitted into this new Direction. Dr. German Silva was appointed Director-General.

#### 1.4. REPORTS

Article X of the Administrative Agreement states that "The Netherlands Co-director of the Project shall report to the Colombian Executive Authority. He shall submit a Quarterly Report in the Spanish and English languages on the progress made with the execution of the Project to both Executive Authorities". In compliance with this article, NEDECO prepared and submitted seven Quarterly Reports (Spanish and English versions) to DITH, covering the period August 1970, to July 1972.

Article X also states: "At the termination of the Project the Netherlands Co-director shall submit the Final Report in the Spanish and English languages of all the aspects of the work done in connection with the Project to all parties involved". The present Report, however, only deals with that part of the study carried out according to the time schedule belonging to the original Administrative Agreement, that is to say, with the study carried out in the period 1970 through 1972. On all aspects and results of studies to be carried out in or after 1973, within the framework of the prolongation of the Project, as referred to in Para. 1.2, a separate Report will be drawn up at a later date.

## Chapter 2

### THE INVESTIGATION (TECHNICAL)

#### 2.1. DEFECTS OF THE RIVER WITH RELATION TO NAVIGATION

In the Colombian transport system the fluvial transport in the Río Magdalena basin plays a historic and important part together with the transport by rail, road or pipeline. As, in general, the transport by river of, especially, bulk goods is cheaper than by rail or by road, it is advisable from the national-economic point of view to make an optimum use of the river system of the country. However, the efficient use that can be made of a particular river as a means of transport is often hampered by a number of shortcomings, such as:

- Insufficient depths and channel widths (often very locally) during a part of the year;
- too sharp bends;
- a high sinuosity (so that distances to be covered by the ships compare unfavourably with distances as the crow flies);
- the instability of navigable channels;
- rapids and locally-encountered high velocities; and
- the presence of floating debris.

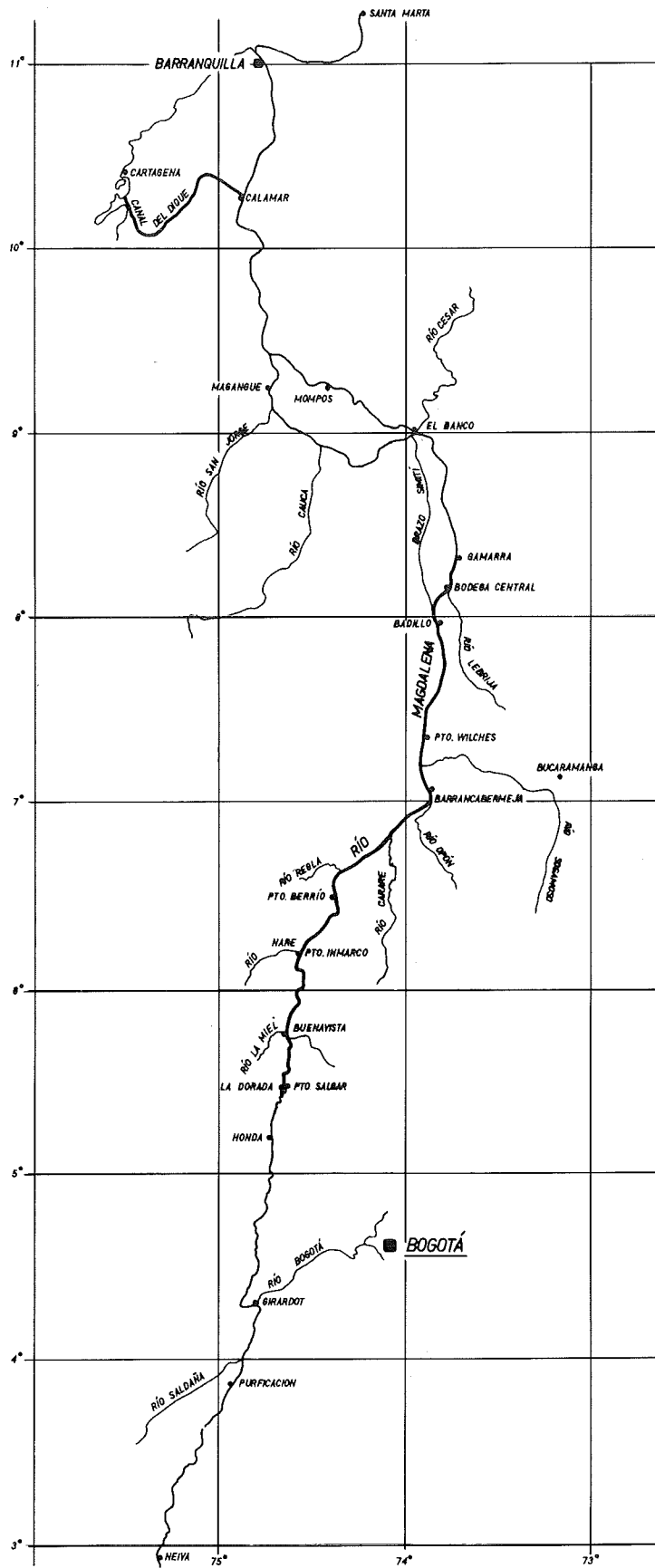
To improve a river transport system, different measures can be considered such as channel patrols, channel marking, river regulation by the construction of groynes and/or spur-dikes (or even dams and navigation locks), dredging in areas forming bottlenecks, or adapting the type of ships (as already done on the Río Magdalena) to the prevailing conditions. Justification of the (often very high) investments required to pursue a certain goal can only be obtained by carrying out a thorough economic feasibility study. Such a study should evaluate the economic value of the river system in relation to the integral transport system of the country. In most cases, however, the situation is more complicated, as not only must the technical requirements and economic value of fluvial transport be considered, but the river should also be studied from the point of view of, for instance, water management (drainage, flood protection), irrigation and generation of hydro-electricity.

In Colombia integral and sufficiently complete transport and river basin studies in the Río Magdalena basin had not yet been carried out when in 1969/1970 the terms of reference for the navigation study of the river had to be drawn up. This explains why in this study no reference is made to the economic feasibility of recommendations, and the scope of the study could only be limited. The purpose of the study was limited to "Providing recommendations for improving, such with relatively limited means and at short notice, the navigability of the Río Magdalena and the Canal del Dique, the protection of the banks of the mentioned rivers and their properties, and the reliability of the

harbour installations", and even then only to some parts of the river, namely "several stretches of the Río Magdalena between La Dorada and Gamarra and the Canal del Dique".

The problems to be studied were listed in the Schedule of Operations, as annexed to the Administrative Agreement, and can be summarized as follows (see also Figure 2.1.1):

- La Dorada/Pto. Salgar (km 889-886)  
The harbour facilities in La Dorada and Pto. Salgar are only accessible for navigation with difficulty because of sedimentation in the forelying river. The study will have to ascertain whether the situation can be improved by means of recurrent dredging, by some simple regulation works, or by a combination of both. Furthermore, shifting of the river-bed upstream of this area creates the risk that a meander will be cut short, thus attacking La Dorada town.
- The Río Magdalena and the Río Nare Confluence (km 774)  
The shallow crossing in the Río Magdalena just upstream of the Río Nare Confluence presents difficulties to navigation and needs improving as it is one of the shallowest crossings between La Dorada and Pto. Berrío.
- The Río Magdalena between km 720 and km 730  
This river-stretch is braided and contains several bad crossings. Temporary regulation works (closing of secondary branches) were carried out here in 1960 by ADENAVI.
- The Río Magdalena and the Río Regla Confluence (km 706)  
Near to this confluence the Río Magdalena is extremely wide and the channel is unstable. The study might indicate whether recurrent dredging could solve the problems. Downstream of the confluence strong current velocities present difficulties to navigation.
- The Río Magdalena and the Río Carare Confluence (km 675)  
The Río Magdalena near the Río Carare Confluence is braided, its channels are unstable and it needs improvement.
- Barrancabermeja (km 631)  
A stretch of about 15 km of the Río Magdalena just upstream of Barrancabermeja is unstable. The situation around Barrancabermeja itself is such that easy access to all quays is not always present, although this is very important in view of the importance of this river harbour.
- The Río Magdalena from km 610 to km 630  
Difficulties for navigation are encountered on this river-stretch where the river splits up into two branches of nearly equal capacity just downstream of Barrancabermeja.
- The Río Magdalena and the Río Sogamoso Confluence (km 612)  
Many secondary branches are formed near this confluence, where a high amount of sediment is brought down by the Río Sogamoso. Also here either dredging or regulation works are to be considered. ADENAVI has already closed some secondary branches here (1966).



RÍO MAGDALENA AND CANAL DEL DIQUE

SCALE 1:4,000,000

FIG. 2.1.1

- The Río Magdalena from Pto. Wilches to Gamarra (km 597 - km 472)  
This river-stretch is braided, contains several bad crossings, and needs improving. The capacity of the river is reduced by the branching off of two large channels (Brazo Simití and Brazo Morales).
- Calamar (Río Magdalena km 91; Canal del Dique km 0)  
A large amount of sediment, brought down by the Río Magdalena, enters the Canal del Dique. For this reason the first kilometers of the Canal silt up. The Centro de Estudios Técnicos e Investigaciones Hidráulicas (CETIH) of the Universidad de Los Andes, Bogotá, has been carrying out a model study on this problem and the Mission has to evaluate the solution as recommended by CETIH.
- The Canal del Dique (km 80 - km 120)  
Through the Caño Correa (near km 83) part of the river water flows to the Bahía de Barbacoas. Downstream of the confluence of the Canal del Dique and the Caño Correa sedimentation takes place. Still more downstream, especially near the outlet of the Canal into the Bahía de Cartagena, sedimentation also occurs, with salt water intrusion possibly playing a role. Improvement of this stretch of the Canal is required and possibilities thereto should be studied. ADENAVI mentioned the possibility of diverting a greater portion of the sediment, brought down by the Canal towards the Bahía de Barbacoas. Such a scheme should be evaluated by the Mission.

## 2.2. THE EXECUTION OF THE PROJECT

The hydraulic and morphological properties of the Río Magdalena and the Canal del Dique are different from those of many other large rivers, especially because of the large amount of sediment transported in suspension.

During the past two decades many new methods have been developed to predict the changes in a river due to human intervention. The execution of the study, therefore, aimed at:

- Checking the validity of the available methods for a river like the Río Magdalena (especially in regard to the sediment transport);
- determining the constants to be applied when suitable methods (formulae) are used; and
- determining the required data in those fields where theory does not yet give sufficient information.

Starting from the above-mentioned aims it appeared that relatively few measurements were sufficient. A summary of the measurements carried out in 1970, 1971 and 1972 is given in Figure 2.2.1.

In Part II a detailed motivation of the measurements is presented, as well as their results, further elaborations and a theoretical outline of suitable methods for the prediction of river changes. In Part III the technical applications for the Río Magdalena and the Canal del Dique are given. Part IV contains a description of the instruments used and of the measuring techniques applied.

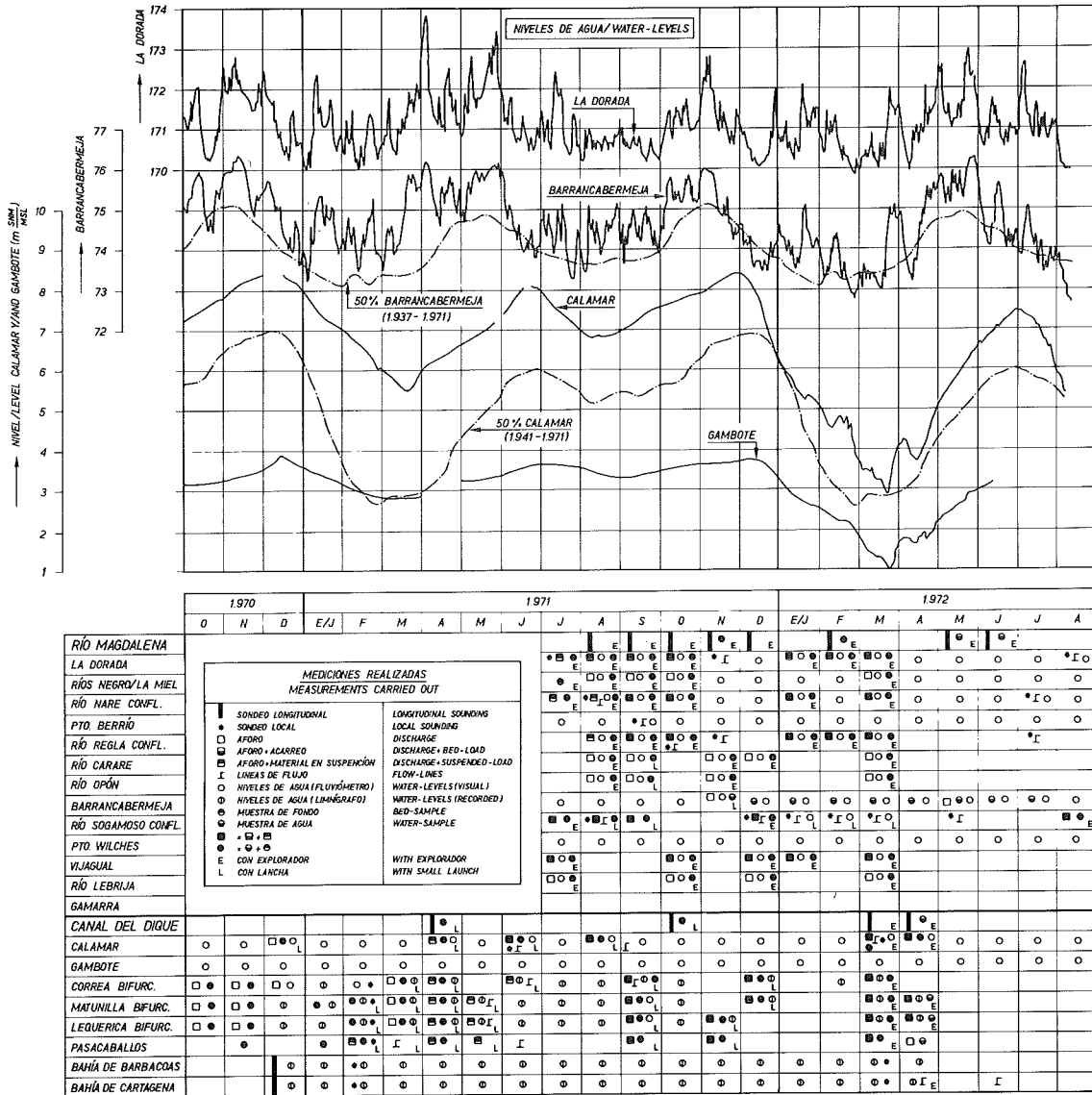


Figure 2.2.1 Summary of Measurements

In the Canal del Dique area measuring activities started in October 1970, using small launches and canoes. On the Río Magdalena the measurements started in July 1971 when the survey vessel "Explorador" became available. Part of the work on this river was also done using small launches. As during the period February-April 1971 the water-levels on the Canal del Dique were much higher than in a normal low water period, the measurements in this area were continued until April 1972. In this way the measurements on both the Río Magdalena and the Canal del Dique covered a more or less normal range of low water and high water stages, thus allowing a rather complete evaluation of the hydraulic and morphological characteristics of both river-stretches for a wide range of conditions.

## Chapter 3

### SUMMARY OF RECOMMENDATIONS

In this Report a great number of recommendations are made, the most important of which are summarized below.

#### RIVER STUDIES

##### Water-levels

It is recommended to maintain the following existing main gauge-stations: Arrancaplumas, Pto. Salgar, Pto. Berrío, Barrancabermeja, Pto. Wilches, Calamar and Gambote (Part II, Para. 2.3).

It is recommended to (re)install and maintain main gauge-stations at: Pto. Triunfo, Pto. Inmarco, Gamarra, El Banco, Bahía de Cartagena (Carare)(II, 2.3) and along the Río Cauca at Las Varas (II, 2.7).

It is recommended to reinstall and/or maintain the following secondary gauge-stations: Río Negro, Río La Miel, Río Nare, Correa, Matunilla, Lequerica and Bahía de Barbacoas (Piedracitas)(II, 2.3).

It is recommended that the zero-levels of the main and secondary gauges be connected to the IGAC network of benchmarks (II, 2.3).

It is recommended that for measurements in the Río Carare and the Río Opon the water-levels be measured in relation to nearby benchmarks.

It is recommended that for length soundings the water-levels at Sta. Lucía, Soplaviento and Mahates be related to nearby benchmarks.

It is recommended that as a reference-level for all navigation purposes Low River Level (L.R.L.) be adopted (II, 2.3.5).

It is recommended to compile and publish in a Year Book all water-levels of main and secondary gauge-stations. These Year Books should also contain duration-curves and L.R.L. values (longterm average, five-year average, etc.) frequency-curves, and frequencies of extreme levels (II, 2.3).

Remark: The shifts found in zero-levels of the past (II, 2.3.4) should be taken into account.

It is recommended that the L.R.L. value for Pto. Inmarco be more accurately determined as soon as additional water-level data have become available (II, 2.5).

It is recommended that in connection with the elaboration and publication of water-level data, an optimum co-ordination be established between all interested agencies (II, 2.3.1).

### Discharge and channel roughness

It is recommended to continue discharge measurements at all gauge-stations except in the Bahía de Cartagena (II, 3.3 and II, 3.5.3), and to adapt the stage-discharge relations every five years accordingly. The data of the main gauge-stations should be incorporated in the aforementioned Year Books.

It is recommended that in general all discharge measurements be carried out in combination with measuring of the local water-level gradient in order to determine the local channel roughness (II, 2.4.1).

It is recommended that future data regarding the channel roughness be elaborated according to the principles used in this Report. For the time being the Einstein/Barbarossa relation can be used, but in future an Alam/Kennedy type of relation appears more appropriate (II, 3.4.3).

It is recommended to carry out discharge measurements in the Brazo's Simití and Morales and to study the division of the discharge (III, 3.5).

### Sediment transport

It is recommended to continue sediment-transport measurements for the sections indicated in Figures II, 3.5.5 to 3.5.7 and 3.5.9 to 3.5.15 and to adapt the presented stage-transport relations accordingly. These relations should also be included in the aforementioned Year Books.

It is recommended to commence sediment-transport measurements at Pto. Triunfo.

It is recommended to study further the vertical suspended-load division per grain-size fraction. For this purpose measurements should be carried out for long durations (II, 3.3.4).

It is recommended that in future measurements the DF-sampler raising time be included in the total measuring time. The catches of the DF-sampler should also be corrected according to the DF-manual (II, 3.3.3).

It is recommended to determine fall velocities of grain-size fractions by means of a Visual Accumulation Tube (VAT) (II, 3.3.4).

It is recommended that wash-load concentrations be determined as function of time and place accordingly to (II, 3.3.5).

### MISCELLANEOUS MEASUREMENTS

It is recommended that longitudinal soundings be regularly made to be used as navigational aids and for dredging purposes (III, 2.2 and III, 5).

It is recommended that local soundings and flow-line measurements be carried out to be used for dredging purposes and for the design of permanent river-works (III, 5 and Part III in general).

It is recommended to take new aerial photographs about every 5 years and to change the river maps accordingly (III, 2.2).

It is recommended that velocity-verticals be measured over a number of crossings at different water stages for further testing of the sensitivity of the computations (II, 3.6 and II, 3.8).

It is recommended to study the influence of the daily variation in water temperature on bed roughness and rate of sediment-transport (II, 3.4 and II, 3.5).

It is recommended to study further the size of the salt water wedge entering the Canal del Dique channels (II, 3.3.9).

## RIVER CONSERVANCY

### Navigation

It is recommended to extend the channel patrol service. Daily channel information should be made available (III, 2.2); besides the usual beacons, buoys should be used in areas such as Sebastopol, Río Regla Confluence and Río Sogamoso Confluence. Buoys should also be placed at the mouth of the Canal del Dique near Pasacaballos. Kilometer boards should be placed at 5 km intervals along the Río Magdalena and be indicated on the river maps.

It is recommended that river maps be regularly made available to river operators.

It is recommended that the possibility of water-level predictions for the Lower Río Magdalena and the Canal del Dique for dredging and navigation purposes be further studied (II, 2.7).

It is recommended that taking-over manoeuvres of barge-trains be prohibited in the Canal del Dique (III, 4.2.2) and at some bad crossings in the Río Magdalena.

### Dredging

It is recommended that if the results of the test dredging are favourable, at short notice a dredger be procured according to the description given (III, 2.3.5). Possibly at a later date a second dredger will be required. For the time being this second dredger could be substituted by the converted DH6 (III, 5.2.2).

It is recommended to study how the dredging efficiency can be improved. Better preparation of dredging works is required; a special unit of pipe-layers laying a landline before the arrival of the dredger can reduce the waiting time of the dredger (III, 5).

It is recommended that better use be made of the available dredging equipment. The equipment should not be used for work for which it is not suitable and for small amounts of excavation along the river a number of draglines can better be purchased.

It is recommended that the minimum available water depth along the Río Magdalena be improved by recurrent dredging of crossings. Up to Barrancabermeja a minimum water depth of 7'6" should be maintained. Awaiting the results of the "Magdalena River Area Transport Study", a minimum water depth of 5' between Pto. Berrfo and Barrancabermeja is suggested, while a depth of 4'6" should be maintained between La Dorada and Pto. Berrfo (III, 3 and III,5).

It is recommended that a large sand-trap be dredged at Calamar (III, 4.3.3). If the results are favourable, this dredging should be carried out each year (a permanent land-line is then preferred).

It is recommended to carry out recurrent dredging in the Canal del Dique and to remove the sand sedimented along the Canal especially at the Correa, Matunilla and Lequerica Bifurcations (III, 4.4 and III, 5).

It is recommended to maintain the capacity of the Canos Correa, Matunilla and Lequerica by recurrent dredging near the bifurcation, as well as at the mouths of the Canos Matunilla and Lequerica. In future a new Caño near El Recreo may be required (III, 4.4 and III, 5).

It is recommended to carry out recurrent dredging at the mouth of the Canal del Dique near Pasacaballos (III, 4.4).

It is recommended to improve the Canal del Dique by dredging according to the design section (width 75 m at L.R.L., depth 2.60 m below L.R.L.) (III, 4.2.2). The advised width and depth should be maintained as closely as possible, while larger and smaller dimensions should be corrected.

It is recommended that the access to a number of ports be maintained by dredging in those years when the entrance tends to sediment (Pto. Salgar, Pto. Berrío, Barrancabermeja, Pto. Wilches and Gamarra) and when justified by the amounts of cargo to be transported (III, 3).

#### Permanent river improvements

It is recommended that river-works upstream of Barrancabermeja along the right bank of the Río Magdalena be designed and executed, to guarantee access to the quay wall (III, 3.5.3).

It is recommended to carry out a model study of the La Dorada - Pto. Salgar area to verify the alignment and extent of the river-works indicated in (III, 3.2.3 and III, 3.2.6).

It is recommended to investigate further the advantages and disadvantages of new port facilities at Pto. Triunfo to partly substitute the facilities of La Dorada and Pto. Salgar (III, 3.2.6).

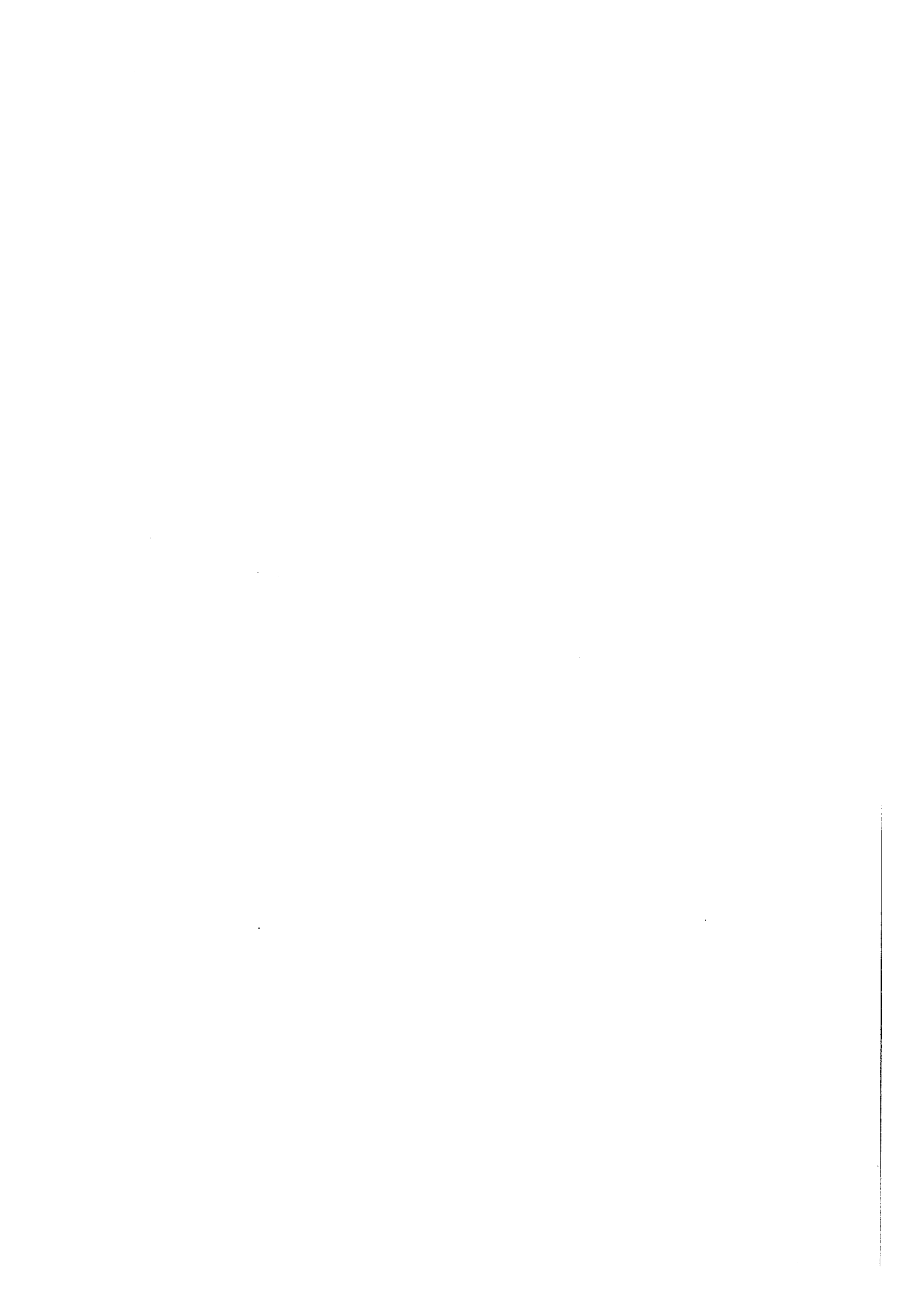
It is recommended to study further and prepare river-works at the Río Regla Confluence, to be carried out as soon as the amount of river traffic warrants such development (III, 3.4.4).

FINANCIAL CONSEQUENCES

A rough estimate of the financial consequences (in Col. pesos/year) of the above recommendations are tabulated below (based on 1972 prices).

"Unidad de Estudios Fluviales" including preparation of river-works	10,000,000
Channel Patrol Service	3,000,000
Dredging - 7'6" channel up to Barrancabermeja and back-log dredging of Canal del Dique, 3,000,000 m <sup>3</sup> /year	30,000,000
- depending increase of transport-flow upstream of Barrancabermeja (up to La Dorada), 1,000,000 m <sup>3</sup> /year	(10,000,000)
Model studies	2,000,000
Permanent river-works based on a yearly improvement of three kilometer river-bank	<u>70,000,000</u>
	Total: Col. \$ 115,000,000/year =====

The information required to draw up the above summary of cost can be found in Part III of this Report. The dredging works upstream of Barrancabermeja and the yearly execution of permanent river-works will of course strongly depend on the results of the "Magdalena River Area Transport Study". It should be remarked that permanent river-works (estimate of total annual cost: Col. \$ 70,000,000) often also need to be executed to serve other than navigational purposes.



## PART II

### HYDROLOGY AND MORPHOLOGY



## CONTENTS

PART II HYDROLOGY AND MORPHOLOGY.		Page
Chapter 1	GENERAL	19
Chapter 2	HYDROLOGY	20
2.1.	Introduction	20
2.2.	Rainfall and evaporation	20
2.3.	Water-levels	22
2.3.1.	Introduction	23
2.3.2.	The hydrographs	24
2.3.3.	Statistical elaboration of water-levels	25
2.3.4.	Presentation of water-level data	30
2.3.5.	Reduction-levels	46
2.4.	Discharges	49
2.4.1.	Introduction	49
2.4.2.	Statistical elaboration of discharges	53
2.5.	Relation-curves	54
2.6.	Computation of backwater-curves	59
2.7.	Prediction of water-levels	61
Chapter 3	MORPHOLOGY	64
3.1.	General	64
3.2.	Sediment transport	64
3.2.1.	Bed-load	64
3.2.2.	Suspended-load	65
3.2.3.	Wash-load	67
3.3.	Collected and elaborated data	67
3.3.1.	Introduction	67
3.3.2.	The discharge and sediment-transport measurements	68
3.3.3.	The suspended-load measurements	78
3.3.4.	The suspended-load concentrations	80
3.3.5.	The wash-load concentrations	84
3.3.6.	Bed-samples and grain-sizes	88
3.3.7.	Longitudinal profiles	93
3.3.8.	Local soundings	95
3.3.9.	Miscellaneous information	96

	Page
3.4. Bed-forms, channel roughness	100
3.4.1. General	100
3.4.2. Grain roughness	100
3.4.3. Bed-form roughness	101
3.5. Selection of the transport-equation	110
3.5.1. Introduction	110
3.5.2. Selection of the transport-equation	111
3.5.3. The discharge, channel roughness, water-level gradient and sediment-transport relations	118
3.5.4. Procedures when no measurements are available	125
3.6. One-dimensional morphological computations	129
3.6.1. General	129
3.6.2. Mathematical base	130
3.6.3. The influence of the suspended-load on the behaviour of the river-bed	136
3.6.4. Application	137
3.6.5. Selection of a dominant discharge	142
3.7. Three-dimensional phenomena	142
3.7.1. Introduction	142
3.7.2. Plan-form	143
3.7.3. Flow in river bends	149
3.8. Applications	152
3.8.1. Introduction	152
3.8.2. Test case of the Río Sogamoso Confluence and the Río Regla Confluence	152

## Chapter 1

### GENERAL

This Part of the Report contains the results of all measurements carried out by the Mission, and the elaborations of these results. Furthermore, it contains a description of the available methods to design river-works, and to predict changes which will occur locally as well as along the river's course as a result of river-works.

During the past twenty years many new developments and methods have become available, making the prediction of the influence of river-works more reliable. On the other hand, the Río Magdalena is a river with properties different from many other rivers studied in the past, due to the large amounts of sediment carried in suspension, and suspended-load is one of the subjects about which there is still a great lack of knowledge.

In view of the above, the following are the reasons why the measurements were carried out:

- To check the validity of the available methods for the Río Magdalena;
- to determine a number of "constants" required in using the described methods and not (yet) available from theory; and
- to determine in general all those features required for the design of river-works which are not yet sufficiently covered by theory.

To fulfil these conditions it was sometimes necessary to select measuring-sites which are less favourable from the point of view of the measuring technique (e.g., the measuring cross-section at Pto. Inmarco, Río Magdalena km 773). When the measurements are tuned to the conditions mentioned, relatively few measurements are sufficient.

Although great advances have been made during the past twenty years, there are still a large number of aspects in river morphology which are not yet sufficiently understood. These aspects are often related to three-dimensional phenomena. The methods used in this respect (which are of a more statistical kind) have also been mentioned in this Part of the Report.

It is of great importance that in the future Colombian river engineers continue to carry out measurements and amend the methods given in accordance with the data found later on. This is especially important during and after the execution of river-works; in this way, as experience is gained, the computations will not only become more accurate and reliable, but also more in accordance with the specific properties of the Río Magdalena.

## Chapter 2

### HYDROLOGY

#### 2.1. INTRODUCTION

This Chapter contains all the hydrological data which were gathered by the Mission.

Firstly, some general aspects of rainfall and evaporation are mentioned (Para. 2.2). It follows that these data mostly have a limited use only and because water-levels can be more easily and rapidly obtained, they are to be considered as the basic data for a river-study. The elaboration and presentation of the water-level data are given in Para. 2.3, together with a correlation between water-levels and rainfall data, which enables the accuracy of the available records of water-level data to be checked.

The background information regarding the discharge is presented in Para. 2.4. The inter-relationship between discharges and water-levels implies a similarity in the elaboration of these data. Para. 2.5 deals with relation-curves between gauge-stations, while the computation of backwater-curves is treated in Para. 2.6.

Finally, some remarks regarding the prediction of water-levels are made in Para. 2.7, and a concept for further study in this field is outlined.

#### 2.2. RAINFALL AND EVAPORATION

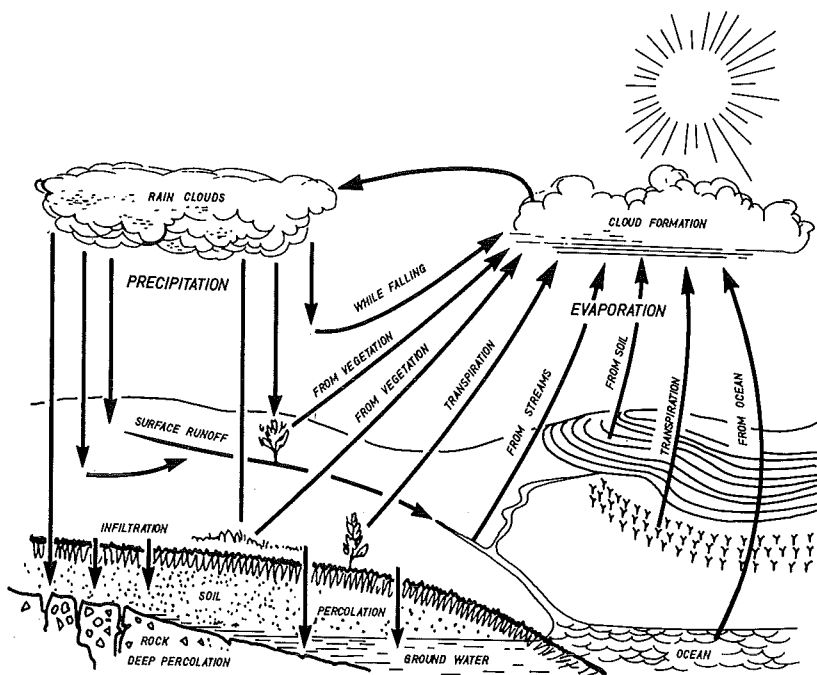


Figure 2.2.1 The Hydrological Cycle

The circulation of water from the oceans, through the atmosphere, over the land surface and back to the oceans is called the hydrological cycle. This cycle has been indicated schematically in Figure 2.2.1 [1]. Actually, the cycle movement is controlled by a very complex relationship between several transport phenomena with different storage elements.

The present study only deals with part of the total cycle, namely, the runoff process in the river basin. Simplified, this runoff process may be presented as shown in Figure 2.2.2.

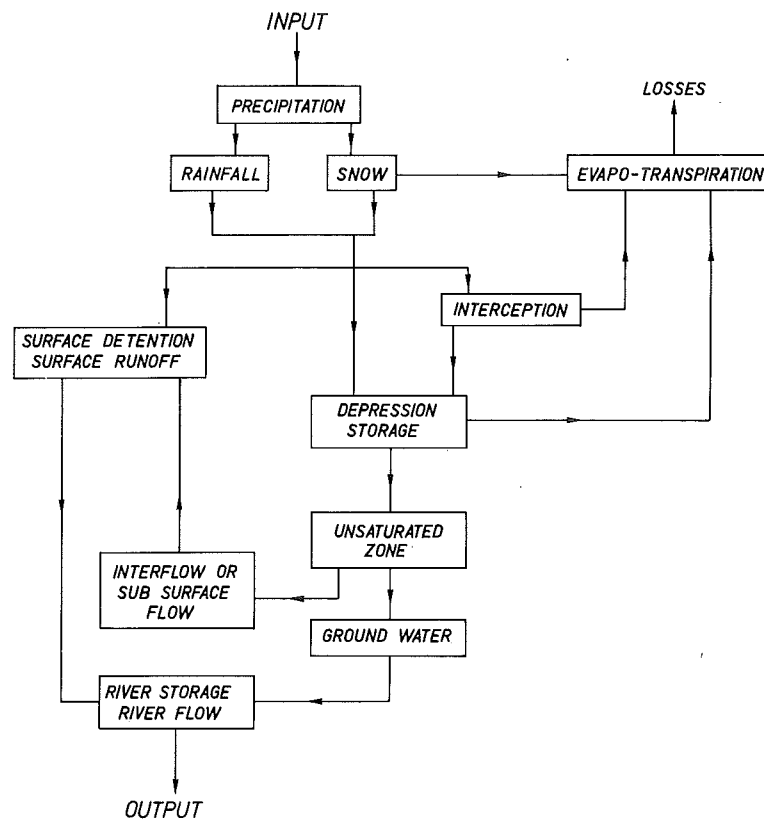


Figure 2.2.2 Simplified Presentation of Runoff Process in Catchment Area

One part of the rainfall infiltrates into the soil and moves down or percolates into the saturated groundwater zone after the moisture deficit in the unsaturated zone has filled up. This water flows very slowly through the aquifers to the river-branches. The water remaining on the surface coalesces into small rivulets, then into large channels and finally reaches the river-branch. In contrast to the groundwater flow, this surface runoff component is a rapid stream process. Together, these components feed the river, and ultimately the water is transported to the sea. During the whole process water is lost by evaporation from open water, bare soil, and by transpiration from the vegetation.

## II, 2.3

Summarizing this pattern, the runoff from a catchment area is affected by:

- The hydro-meteorological factors (rainfall, evaporation, etc.); and
- the physical characteristics of the catchment area.

From this it follows that the river runoff can be computed from rainfall and evaporation data if sufficient knowledge is available about the two major groups mentioned above. For this computation many methods have been proposed. However, because of strong non-uniform basin physiography an extensive study would be required. Therefore, it is preferable to determine the discharge by measuring the water-level, when the stage-discharge relationship is known. Although this method has been used, rainfall and evaporation data may nevertheless be used, e.g., for the explanation of and check on the measured hydrograph, or to fill up the gaps in such a graph.

For the determination of the rainfall use has been made of data from the rain-gauges installed by the Servicio Colombiano de Meteorología e Hidrología (SCMH), which had created a network of rain-gauges over the catchment area of the Río Magdalena. The considerable variability of the rainfall, caused by differences in origin and climate, as well as by orographical effects, has also been taken into account. The estimate of the amount of evaporation and transpiration (commonly linked together and referred to as evapo-transpiration) is more difficult. The evapo-transpiration depends on the net incoming radiation, wind, relative humidity of the air and the availability of water. Since only rough data for time intervals of one year are required, the mean temperature (as a measure for the available heat-energy) and the total rainfall can be used for the determination of the actual amount of evapo-transpiration.

### 2.3. WATER-LEVELS

#### 2.3.1. Introduction

As was explained in Para. 2.2, rainfall data have only a limited use, especially for large rivers. As many of the phenomena in rivers have a relationship with the water-level and, moreover, water-levels can easily and rapidly be obtained, it is logical to consider water-levels as the most basic data for a river study. Very often, as a first approximation, the relationship between water-levels and other phenomena may be considered simple and unique. If, for instance, the Chézy equation is considered as  $Q=C Bh^{3/2}I^{1/2}$  and C, B and I are assumed to be constants or in any case a function of h only, then  $Q=Q(h)$ . This means that a statistical elaboration of water-levels enables statistical information on the discharge to be obtained, although the discharge has not been measured daily like the water-levels.

As will be seen, statistical elaboration of water-levels is also necessary to judge the available depth for navigation and to decide about the extension of required river-works. Fortunately the importance of river-levels has also been realized in Colombia. Along the Río Magdalena and the Canal del Dique water-level data were available for at least some time at Arrancaplumas, Pto. Salgar, Pto. Berrío, Barrancabermeja, Pto. Wilches, Gamarra, Calamar and Gambote. In addition to these main gauges a number of temporary gauges were installed during the study: in the Río Magdalena at Pto. Inmarco and in affluents

of the Río Magdalena, namely, the Río Negro, the Río La Miel and the Río Nare. In the Canal del Dique a number of automatic gauges were temporarily installed at the bifurcations of the Caño Correa, the Caño Matunilla and the Caño Lequerica, as well as at Carare (Bahía de Cartagena) and Piedracitas (Bahía de Barbacoas).

The gauge in Gamarra was re-installed after having been abandoned in 1955. It is suggested that this gauge be again considered as a permanent main station. It is also advised to continue the readings on the gauge at Pto. Inmarco as a permanent main station (Table 2.3.1).

Location	Records Available to the Mission	Zero-level of Gauge in respect of M.S.L. (zero of IGAC)
<u>Main water-level stations along the Río Magdalena</u>		
Arrancaplumas (km 931)	1934-1972	195.18
Pto. Salgar (km 887)	1936-1972 (minus 1958 and 1960)	165.83
Pto. Inmarco (km 773)	from July 1971	122.50
Pto. Berrfo (km 730)	1936-1972 (minus 1957 and 1960)	106.44
Barrancabermeja (km 631)	1937-1972 (minus 1947, 1948, 1951, 1952, 1953 and 1960)	70.51
Pto. Wilches (km 597)	1934-1972 (minus 1955, 1957, 1958 and 1960)	61.12
Gamarra (km 473)	1934-1945, 1955-1956 (minus 1938)	(35.42)
Calamar (km 91)	1934-1972 (minus 1960)	- 0.35
<u>Temporary water-level station along the Río Magdalena</u>		
Pto. Triunfo (km 821)	September-October 1971 (data collected by INTEGRAL)	
		(Zero-level of Gauge below local B.M.)
<u>Water-level stations in affluents of the Río Magdalena</u>		
Río Negro	from August 1971	6.40 m below BH-MCH4
Río La Miel	from August 1971	6.93 m below BH-MCH3
Río Nare	from July 1971	7.30 m below BH-MCH1A
<u>Main water-level station along the Canal del Dique</u>		
Gambote (km 66)	1961-1972	- 0.62
<u>Temporary water-level stations in the Canal del Dique area</u>		
Correa (km 82.5)	September 1970-January 1972	- 0.62
Matunilla (km 100)	September 1970-January 1972	- 0.62
Lequerica (km 108)	September 1970-January 1972	- 0.62
Piedracitas (Bahía de Barbacoas)	January 1971-January 1972	- 0.62
Carare (Bahía de Cartagena)	September 1970-January 1972	- 0.62
Remark: The zero-levels of the stations in the Canal del Dique area are related to the net of Mantilla. Reference is made to Table 2.3.9.		

Table 2.3.1 Water-level Stations and Available Records

Although a lot of data are available, the elaboration has sometimes been difficult due to the fact that gauges are not always operated properly (zero-levels of the gauges have changed during repairs without a proper levelling afterwards, gauges or the gauge-readers are not checked, etc.). It seems important to make one agency in Colombia responsible for the gauge-readings in order to ensure that gauges are properly maintained and read. This agency should, of course, also have a special office where the data are elaborated and published annually.

## II, 2.3

For the gauges mentioned (Table 2.3.1), the data have, as far as available, been elaborated by the Mission. Not all the hydrographs can be given here, and it is therefore suggested that the agency responsible for the gauge network will make these hydrographs available in future in a printed form, together with the statistical elaboration of these data.

It has to be stressed that the results of the elaboration, as given in this Part (duration-curves, frequencies, etc.), should be amended at least every 5 years because such data are only valid for a limited time, especially when river-works are carried out.

### 2.3.2. The hydrographs

A first insight into the characteristics of the water-levels at a certain station can generally be obtained by plotting the daily levels against the time (the so-called hydrograph). In the course of the elaboration of water-levels the need is often felt to plot the daily readings per hydrological year instead of per calendar year. The hydrological year is the period of one year between the two points of minimum discharge. Depending on the onset of the rainy season there are, of course, variations from year to year of the moment at which discharges reach a minimum. For the Rfo Magdalena the hydrological year can, on the average, be taken from March 1 to February 28 (or 29). However, as it is a common practice in Colombia to plot the daily levels per calendar year, and the advantages of the introduction of the hydrological year were not too strongly felt by the Mission, all the water-levels and elaborations in this Report will be given per calendar year.

Some general information can already be obtained by studying the hydrographs; e.g., whether dealing with a pure rainy river in which the daily levels fluctuate rapidly with steep rises and falls, or with a monsoon river in which the consecutive seasons result in a difference between the average high and low levels. When compared with a frequency-curve obtained from a number of years (Figure 2.3.1) it can, moreover, be decided if the year under consideration could be characterized as a wet or a dry year. However, in general, the hydrograph must be considered to be the main base for further elaborations.

It is most important for an agency dealing with water-levels that the levels of the main gauge-stations are compiled daily in the office and plotted directly on time-charts. The levels of secondary and temporary gauge-stations can be compiled once every one week or, at the utmost, two weeks. Such an up-to-date record is required for the planning of other types of measurements, as well as for the execution of river-works.

If the range between the high and low levels at a certain gaugestation is very great, a number of gauges located at different heights can be used to collect the daily levels. If these gauges do not have the same zero-level, the daily levels should be immediately adjusted to the reference zero-level which is also mentioned on the hydrograph.

### 2.3.3. Statistical elaboration of water-levels

Given the records of daily gauge-readings, there are three ways in which the distribution of the levels can be considered. First, the distribution of the water-levels at a certain date can be calculated for the period of observation, and from this the frequency-curves for every day of the year result; an example is given in Figure 2.3.1.

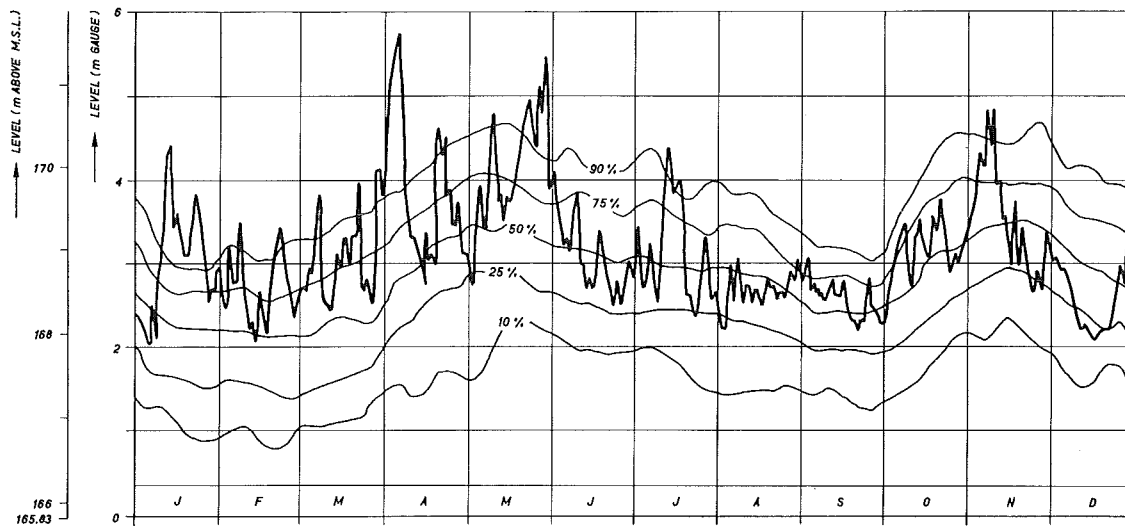


Figure 2.3.1 Frequency-curves of the Water-level at Pto. Salgar Compared with the Hydrograph of the Year 1971

A frequency-curve shows for every day of the year which water-level has not been reached during a certain part of the period over which the records are available. Generally, it is the practice to prepare the frequency-curves for 10%, 25%, 50%, 75% and 90% of the period. As can be seen from Figure 2.3.1, the water-level as read on the gauge at Pto. Salgar on May 1, was lower than 4.05 m during 75% of the period and lower than 2.90 m during 25% of the period. Consequently, during 50% of the period the water-level at Pto. Salgar on that date has been between 2.90 and 4.05 m. Comparing the annual hydrograph with the frequency-curves, an impression is obtained of how wet or dry the year under consideration actually has been and what will be the frequency of an even wetter or dryer year.

A second elaboration of the daily gaugereadings will be to consider the variation of the number of days per year during which the water-levels are lower (or higher) than a certain level. Such elaborations lead to the duration-curves, which can again be made for 10%, 25%, 50%, 75% and 90% of the number of years that the records are available.

From the duration-curves given in Figure 2.3.2 it can be read, for instance, that during 75% of the period the water-level at Pto. Salgar has been lower than 3.55 m during 200 days. On the other hand, this means also that during the same 75% of the year the water-level has been higher than 3.55 m during 165 (365-200) days. The area contained by

## II, 2.3

the duration-curve and the axes is an indication of the total discharge of the pertaining year(s). If it is intended to compare the annual runoff with the annual rainfall, it has advantages to determine the duration-curve for the hydrological year instead of the calendar year.

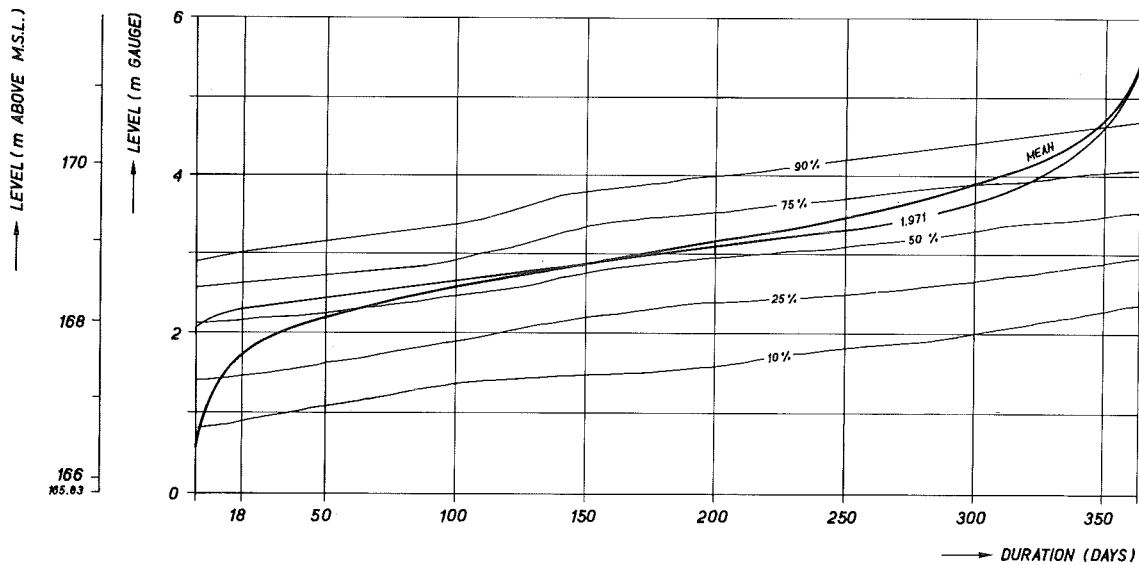


Figure 2.3.2 Duration-curves of the Water-level at Pto. Salgar

In Figure 2.3.2 also the average duration-curve of the water-level at Pto. Salgar is given. This average curve is not identical to the 50% curve. The former indicates the average number of days of the years considered during which the water-level is lower than a certain level. The 50%-curve indicates the number of days during 50% of the years of record during which the water-level has been lower than a certain level. The average duration-curve is especially important for the determination of the reduction-level at a certain station (more about this is said in Para. 2.3.5).

Lastly, the records of the daily gauge-readings can be elaborated to predict the frequency of extreme high and low water-levels. For the design of river-works the extreme levels are of great importance (e.g., the height of a bank protection, the crest of a dam, or to what height areas will have to be reclaimed to serve their purpose for the extension of a city or new port sites). The frequency of extreme levels is mostly expressed as a frequency of once in 10, 50 or 100 years in which a certain level is reached or exceeded by the highest yearly level, or the lowest yearly level is occurring or falling lower. A frequency of once in, e.g., 100 years must, however, definitely not be read as stating that the phenomenon will occur once in every 100 years. It only expresses the probability of the occurrence, but this might as well happen to-morrow or in two consecutive years.

The decision on which frequency of occurrence will be acceptable has to be made on the basis of an economic consideration: the extra investment required for the heightening of, say, an embankment has to be weighed against the savings in, say, the increased output of the yearly crops. Such a decision will be even more difficult if the safety of human life is also involved. In general practice an over-height is introduced for town extensions, etc., so as to be sure to be on the safe side. To discuss in greater detail the considerations which have to be taken into account is beyond the scope of this Report, so it must suffice to say that usually the "benefit-cost ratio" determines the height of the riverworks. In this Report only the method of estimating the frequency of a certain water-level (or discharge) is dealt with.

First of all, it has to be decided which water-level must be considered to be the highest yearly level. If the hydrograph of a certain station clearly shows one dry and one wet period, this will not cause difficulties; the highest level reached will have to be considered. However, often (as in Colombia), two rainy seasons can clearly be distinguished in the hydrograph. The first occurs in the months of April and May and the second in the months of October and November. In view of the fact that, generally, the recession of the water-levels in between the two high water peaks is not as low as in the dry season (January up to March) and because of the heavy rainfall in the second half of the year, the highest annual water-levels are reached in the months of October and November. If the highest water-levels recorded in the two high water peaks of the hydrograph are completely independent of each other, both should be used for the determination of the frequency of extreme levels. However, especially in the lower region of the Río Magdalena, this assumption is not true; the presence of the great storage areas along the Río Magdalena ("cienagas") results only in a slight recession of the water-level during the months of July and August, and the highest levels reached during the second half of the year are, at least, partly determined by the levels recorded in the first half of the year. It is therefore obvious that only one highest yearly level must be used instead of two.

From a record of extreme values it is possible to give any of these values its plotting position by arranging them in order of magnitude. The probability that each value from the record will not be exceeded (in case of extreme high water-levels) can be calculated according to several methods. The most simple one is the "California method" which defines the frequency of the  $i$ -value as:  $m_i/(n+1)$ , with  $m_i$  = the number of the value ( $i$ ) if arranged in order of magnitude, and  $n$  = the total number of values in the record.

The records of the extreme yearly levels will mostly comprise a period between 20 and 50 years, which means that an extrapolation of the available data will be required if a frequency has to be determined of, say, once in 100 years. It will be clear that such an extrapolation will only be possible to a limited degree of accuracy if the available data can be plotted on a straight line. Again, many methods exist to obtain such a straight line. Gumbel (1958) [2] indicated a method based on the statistics of extreme values; he also designed a special probability-paper, which has been used here. As an example, the extreme yearly levels of Calamar are given in Figure 2.3.3, plotted on Gumbel's probability-paper.

II, 2.3

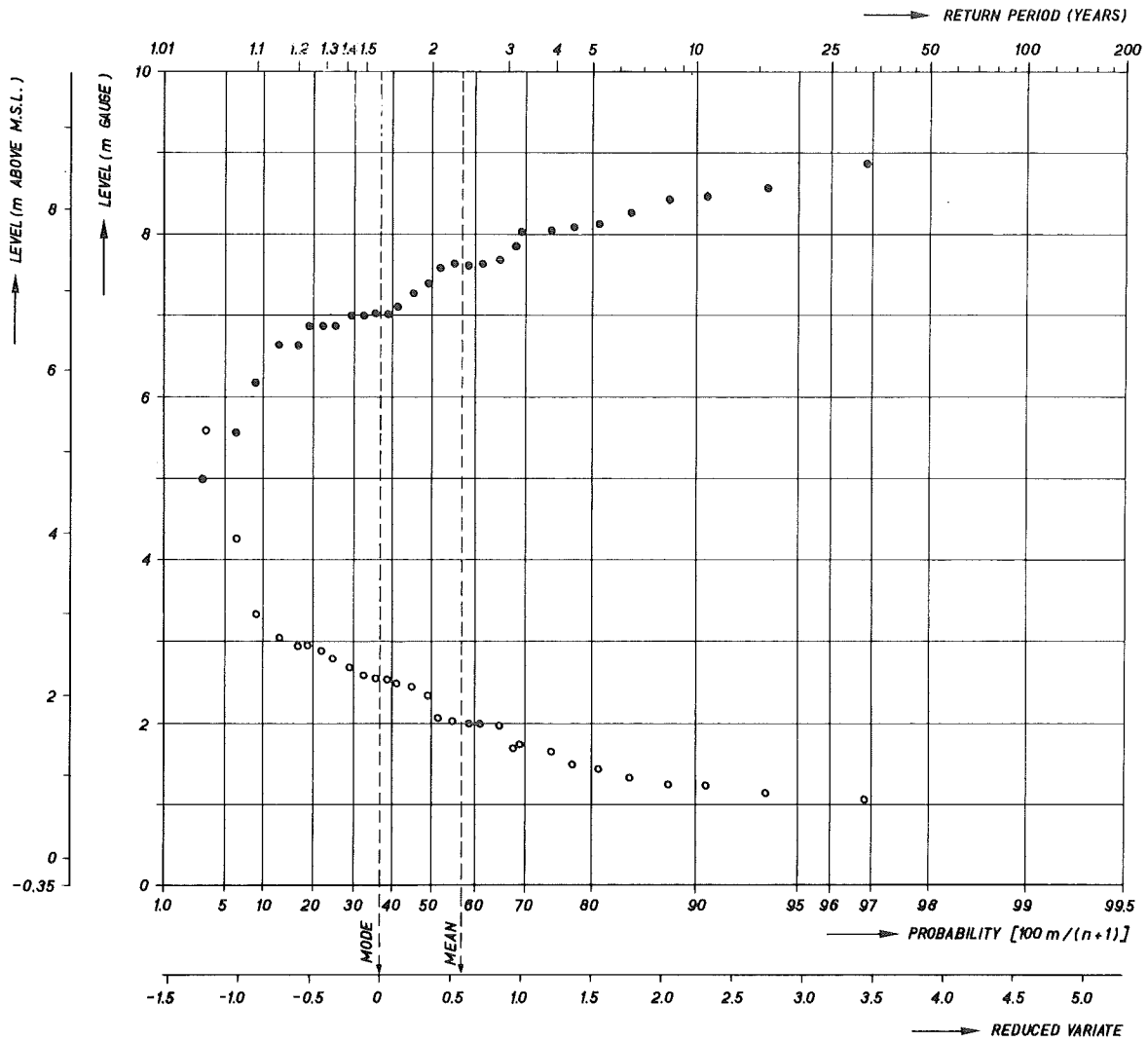


Figure 2.3.3 Extreme Yearly Levels at Calamar

A few remarks may be made here about the used probability-paper. The frequency of the phenomenon is plotted on the horizontal scale according to the function:

$$\phi(y) = e^{-e^{-y}}$$

while the vertical scale is a linear one. The relation between the probability  $P(x)$  and  $\phi(y)$  is:

$$P(x) = e^{-e^{-\alpha(x-\mu)}} \quad (2.3.1)$$

with  $y = \alpha(x-\mu)$ . The parameters  $\alpha$  and  $\mu$  of Eq.(2.3.1) must be estimated ( $\hat{\alpha}$  and  $\hat{\mu}$ ), which can, e.g., be done with the method of the least squares (see Figure 2.3.4).

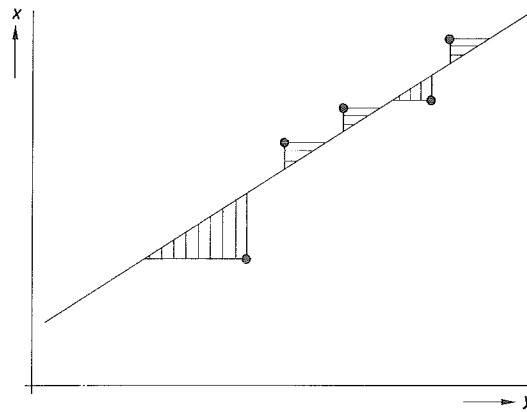


Figure 2.3.4 Determination of Straight Line with Least Squares Method

The straight line between the plotted points can best be drawn by observing that the sum of the areas of the rectangular triangles  $(x_i, y_i)$  will be a minimum. This method finally yields to:

$$\hat{\alpha} = \frac{\sigma_y}{s}$$

$$\hat{\mu} = \bar{x} - \frac{\bar{y}}{\hat{\alpha}}$$

$$s = \sqrt{\frac{\sum x_i^2 - \frac{(\sum x_i)^2}{n}}{n-1}} \quad (= \text{standard deviation}). \quad (2.3.2)$$

The rounded values of  $\bar{y}$  and  $\sigma_y$  (both functions of  $n$ ) have been calculated for different  $n$ -values, according to the function  $\phi(x_i) = i/(n+1)$  and are given in Table 2.3.2.

$n$	$\bar{y}$	$\sigma_y$	$n$	$\bar{y}$	$\sigma_y$
10	0.50	0.95	26	0.53	1.10
11	0.50	0.97	27	0.53	1.10
12	0.50	0.98	28	0.53	1.10
13	0.51	1.00	29	0.54	1.11
14	0.51	1.01	30	0.54	1.11
15	0.51	1.02	31	0.54	1.12
16	0.52	1.03	32	0.54	1.12
17	0.52	1.04	33	0.54	1.12
18	0.52	1.05	34	0.54	1.13
19	0.52	1.06	35	0.54	1.13
20	0.52	1.06	36	0.54	1.13
21	0.53	1.07	37	0.54	1.13
22	0.53	1.08	38	0.54	1.14
23	0.53	1.08	39	0.54	1.14
24	0.53	1.09	40	0.54	1.14
25	0.53	1.09	50	0.55	1.16

Table 2.3.2  $n$ -,  $\bar{y}$ - and  $\sigma_y$ - values

## II. 2.3

The probability-paper as given in Figure 2.3.3 has, apart from the probability scale, also a "return period" scale related to each other according to:

$$T = \frac{1}{1-P}$$

in which:

P= the frequency that a certain level will not be exceeded; and

T= the return period, indicating that once in T-years the considered level will be reached or exceeded.

The linear scale, the "reduced variate" (y), has to be used to determine which part of the levels of the considered record will be smaller than a certain value (x).

If the highest and lowest yearly levels have been elaborated in this manner, it is possible to extrapolate the available data to smaller frequencies. However, it must be stressed that such frequencies will still be only an estimate. The accuracy of such estimates can be obtained by the determination of the confidence limits of the straight line. However, such elaborations are considered to be beyond the scope of the present Report.

Apart from the determination of the frequency of extreme levels, it is often possible to distinguish also certain physical features of the river under consideration. In a river without a flood-plane, or, in other words, with infinitely high banks, the frequency-curve of the highest yearly level will show a straight line. If, however, one or more flood-planes on different heights exist along the river, the frequency-curve will consist of a broken line composed of straight stretches. The bends in the lines indicate the height of the storage areas which will be flooded at still higher water stages. Consequently, the gradient of the lines gradually decreases at higher stages.

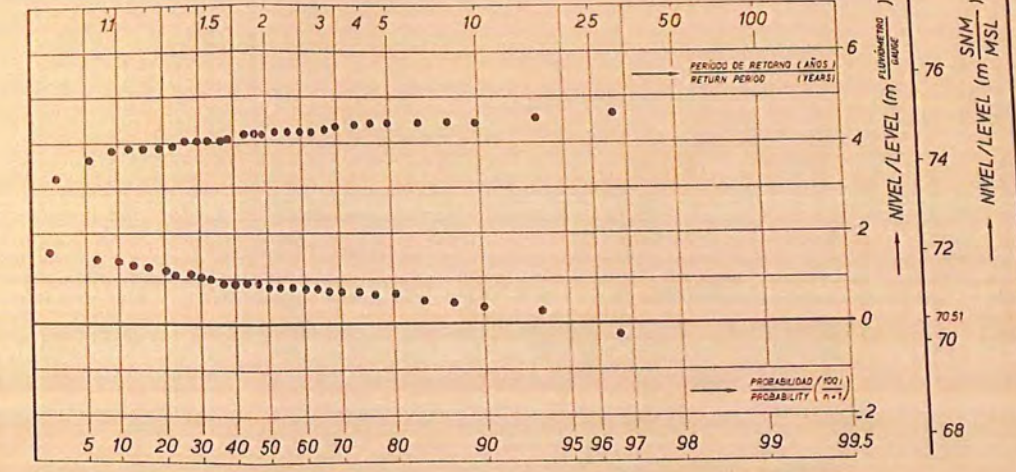
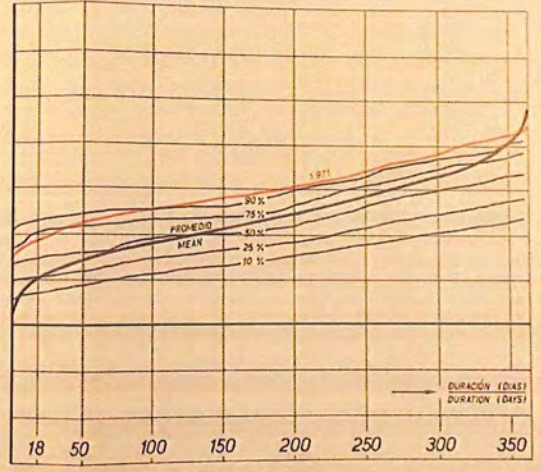
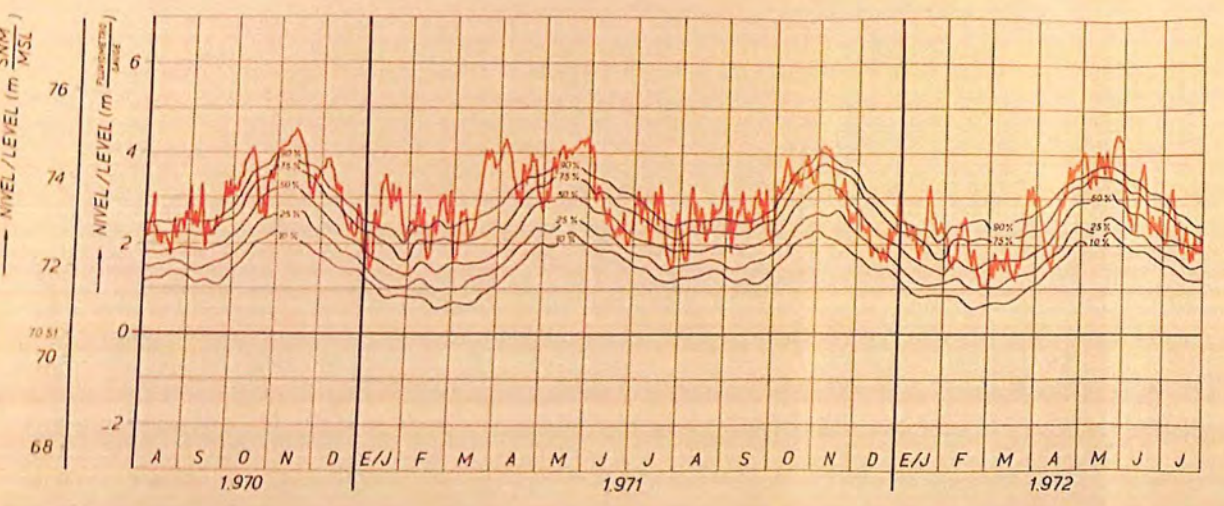
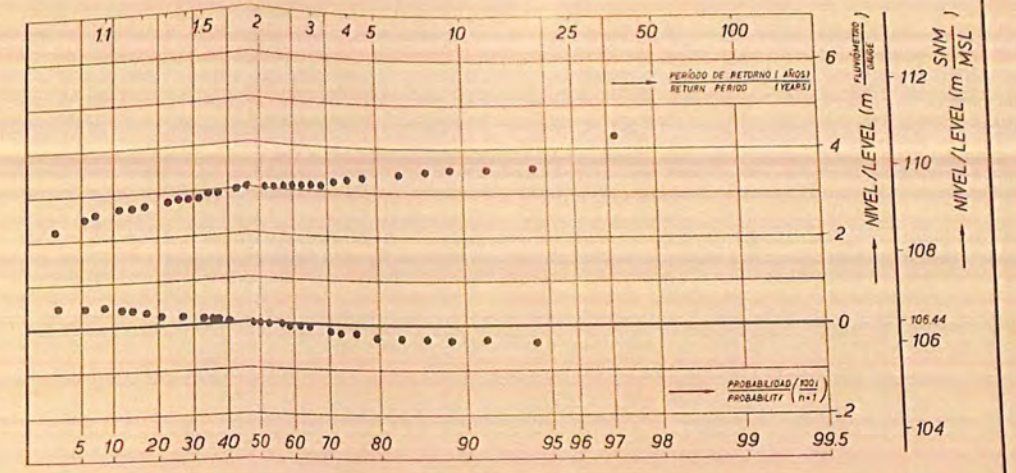
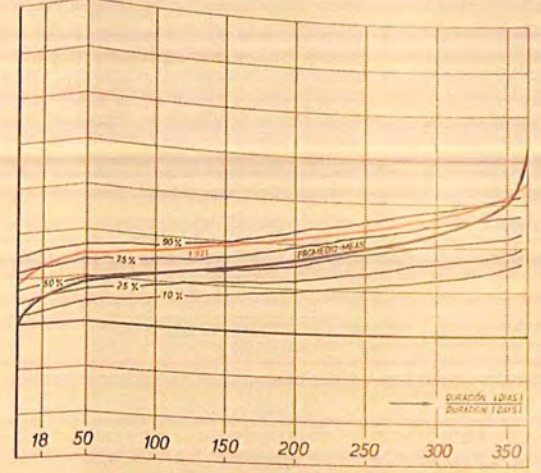
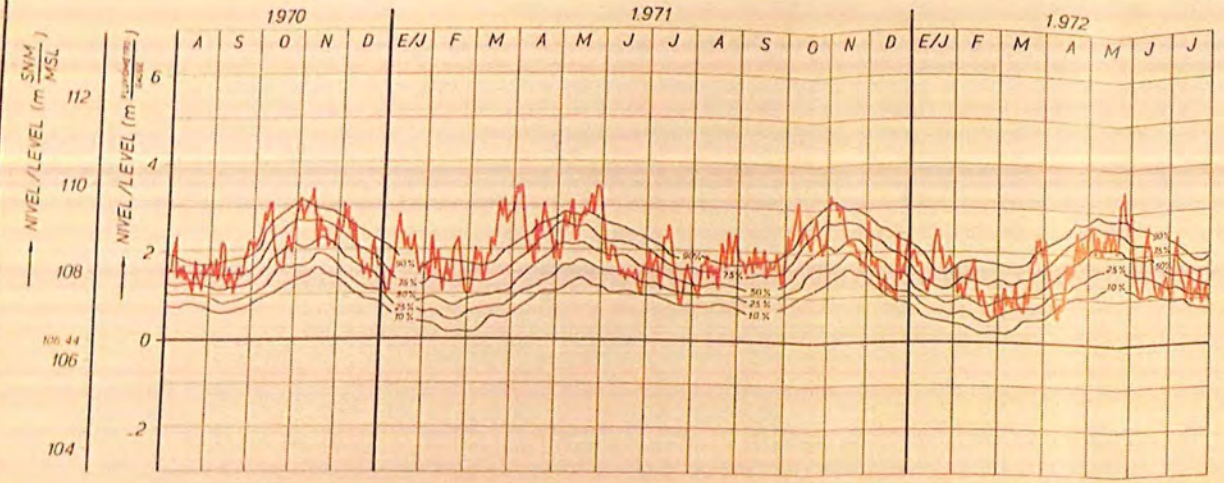
It is still possible to further elaborate the daily water-levels, by obtaining the relation-curves, celerities of flood-waves and predictions of water-levels. However, these phenomena cannot be studied without more insight into the inter-relationship between the water-levels and the discharges. Therefore, further information about this is given in Para. 2.5.

### 2.3.4. Presentation of water-level data

The water-level data as they were compiled and elaborated by the Mission are presented in this paragraph. The data of the main gauge-stations along the Rfo Magdalena, viz., Arrancaplumas, Pto. Salgar, Pto. Berrfo, Barrancabermeja, Pto. Wilches and Gamarra, are presented in Figures 2.3.5 to 2.3.7. It is advised to consider Gamarra in future also as a main gauge-station, although the available records of water-levels of this station are as yet insufficient to allow elaborations. The data of the main gauge-stations along the Rfo Magdalena and the Canal del Dique, viz., Calamar and Gambote, are presented in Figure 2.3.8.

PTO. BERRÍO

RÍO MAGDALENA km 730 1936-1971

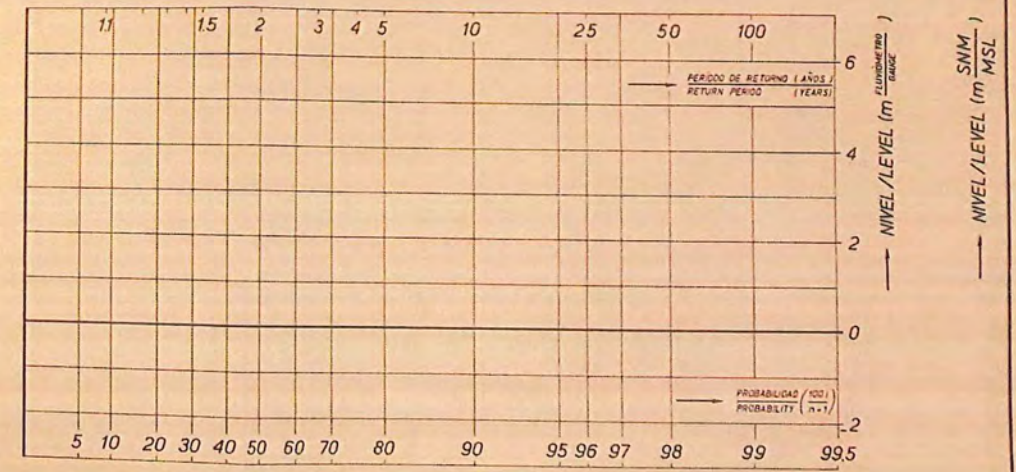
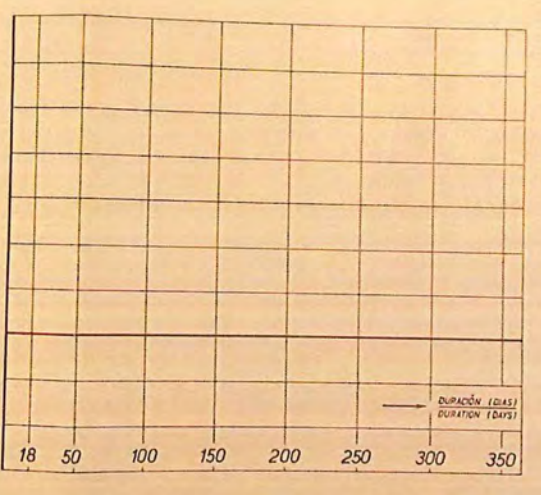
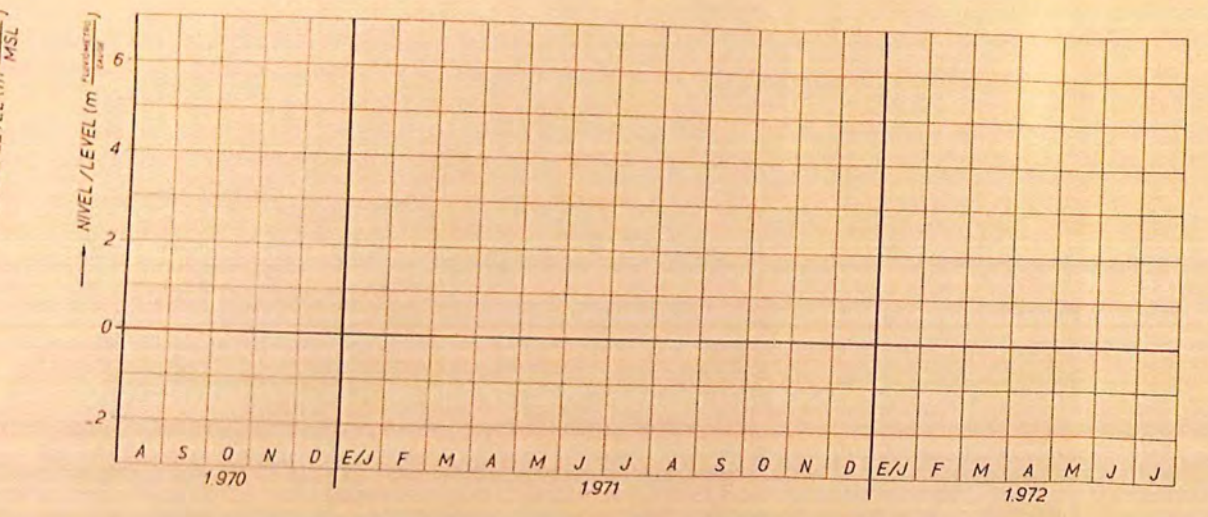
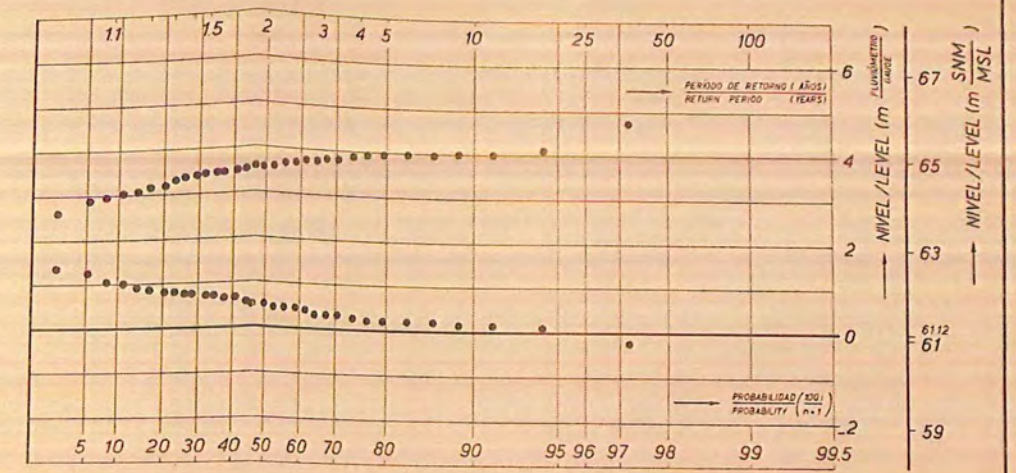
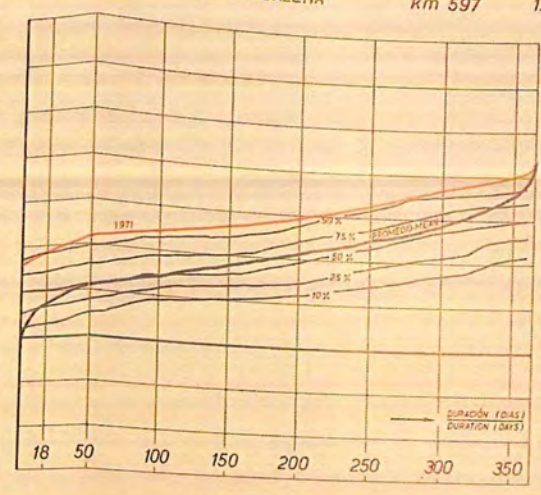
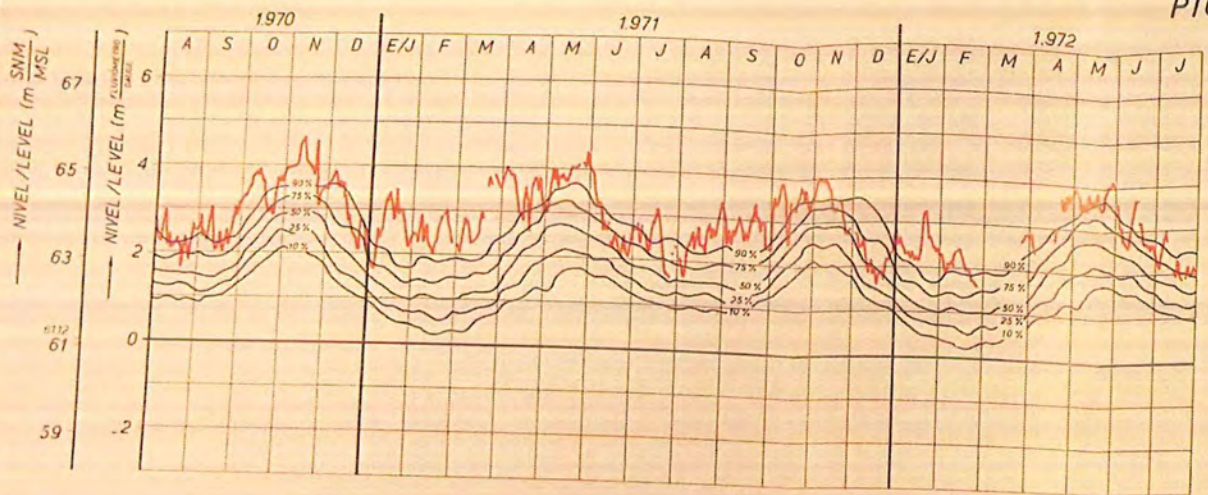


BARRANCABERMEJA RÍO MAGDALENA km 631 1937-1971

NIVELES DE AGUA / WATER LEVELS		PTO. BERRÍO - BARRANCABERMEJA	
NIVELES DE AGUA 1970-1972		DAILY LEVELS 1970-1972	
CURVAS DE FRECUENCIA - CURVAS DE DURACIÓN		FREQUENCY CURVES - DURATION CURVES	
FRECUENCIA DE NIVELES EXTREMOS		FREQUENCY OF EXTREME LEVELS	
NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT			FIG. 2.3.6

PTO. WILCHES

RÍO MAGDALENA km 597 1934-1971



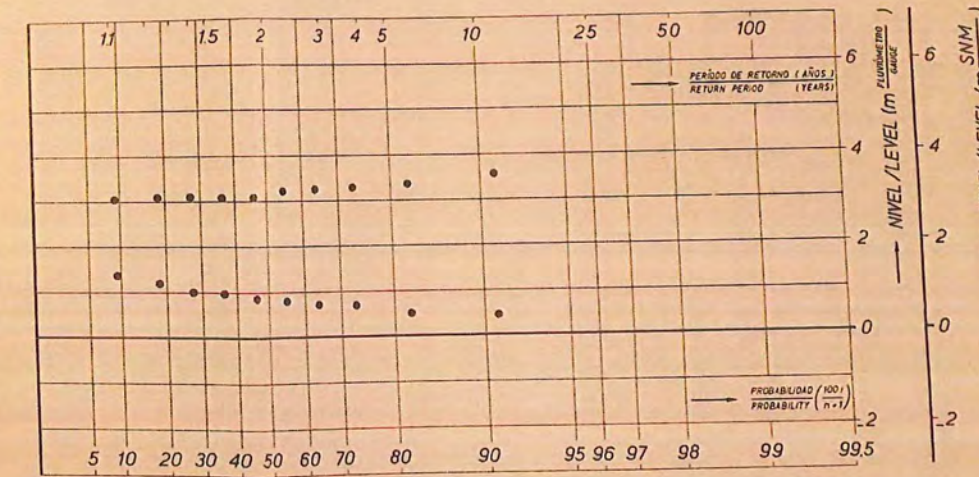
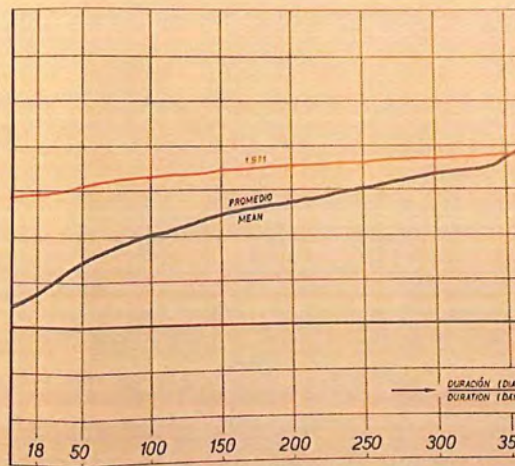
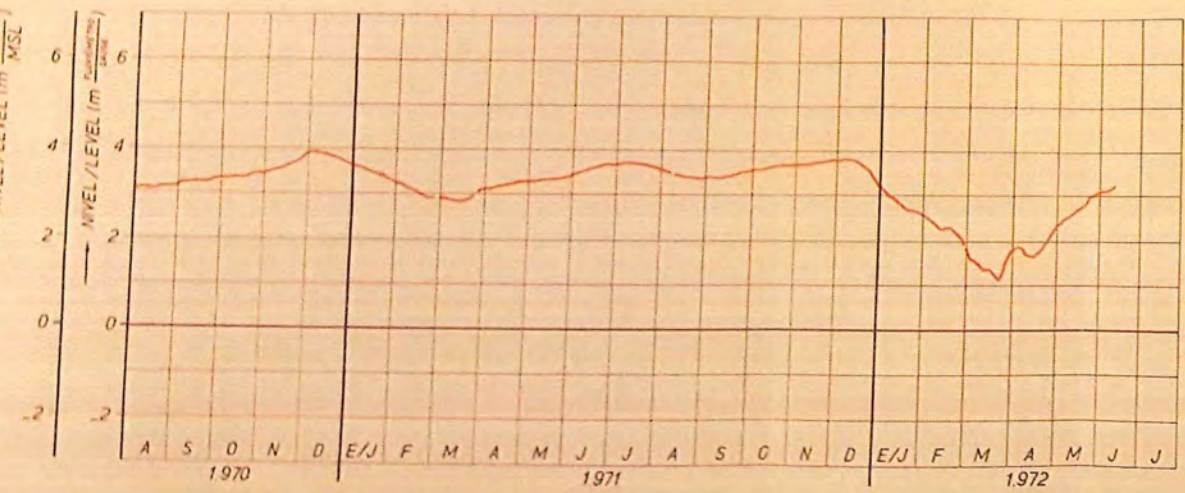
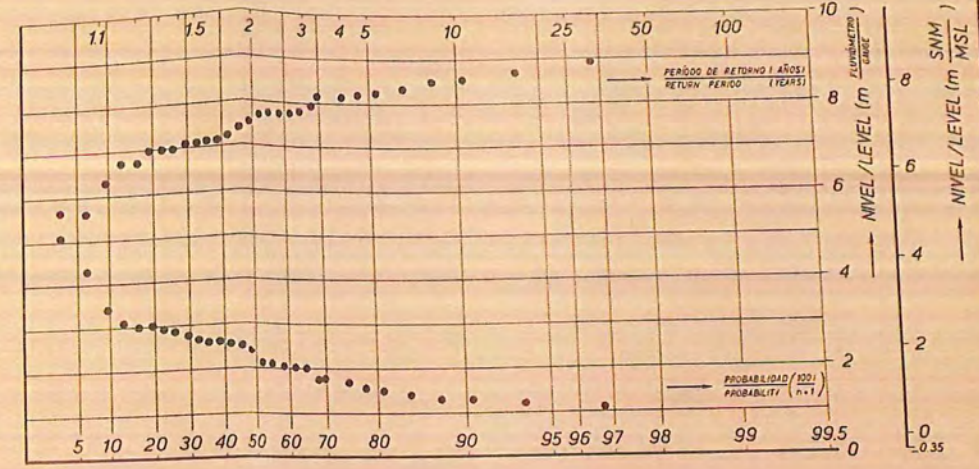
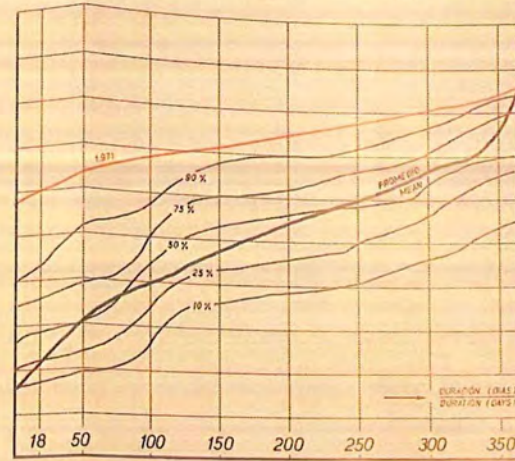
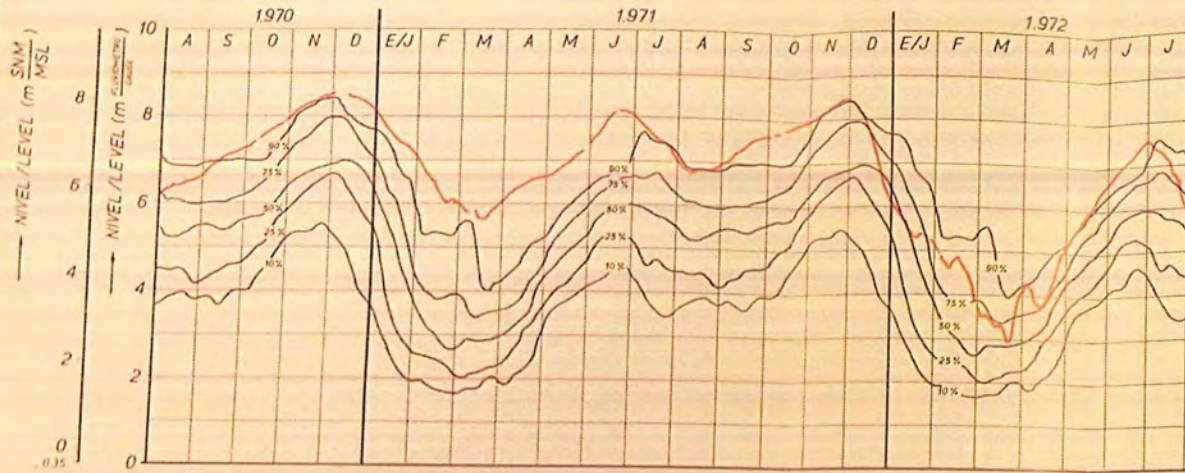
GAMARRA

RÍO MAGDALENA km 473 19 - 19

NIVELES DE AGUA WATER LEVELS		PTO. WILCHES - GAMARRA	
NIVELES DE AGUA 1970-1972 CURVAS DE FRECUENCIA - CURVAS DE DURACIÓN FRECUENCIA DE NIVELES EXTREMOS		DAILY LEVELS 1970-1972 FREQUENCY CURVES - DURATION CURVES FREQUENCY OF EXTREME LEVELS	
NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT		FIG. 2.3.7	

CALAMAR

RÍO MAGDALENA km 91 1941-1971



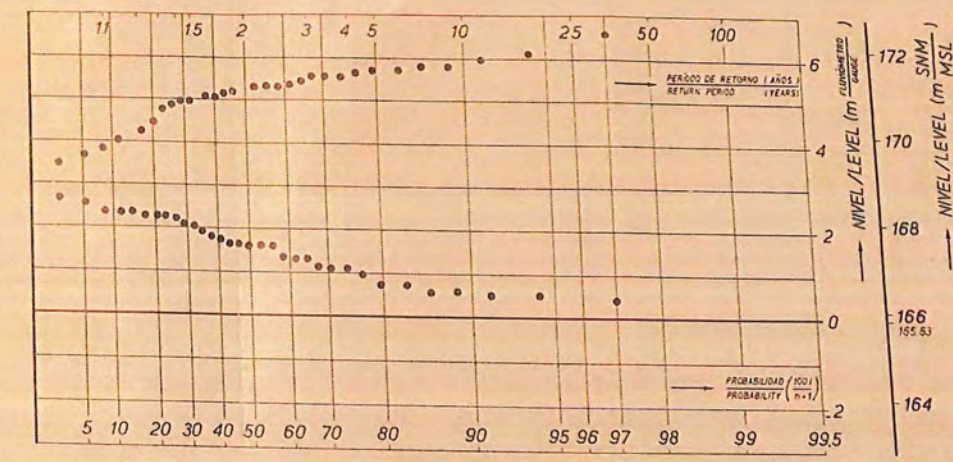
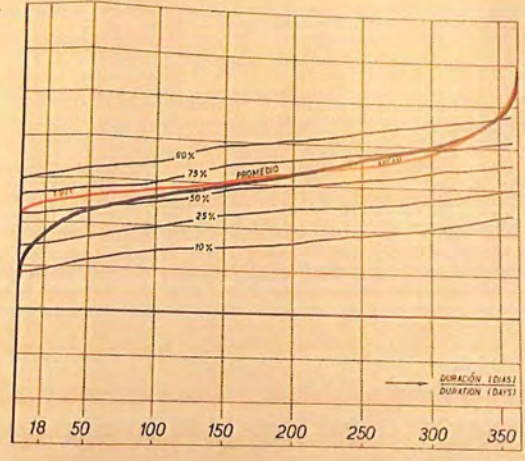
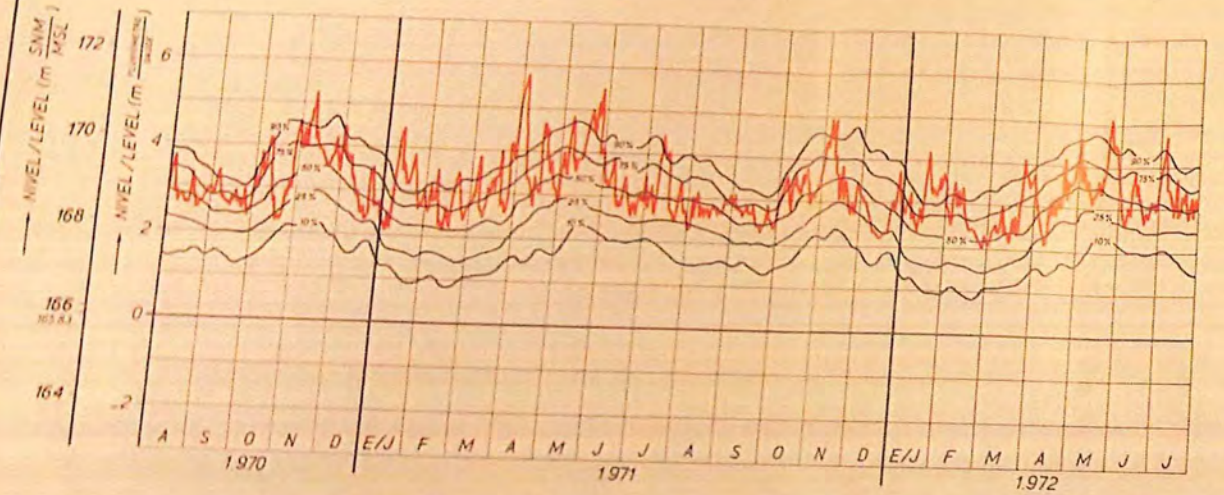
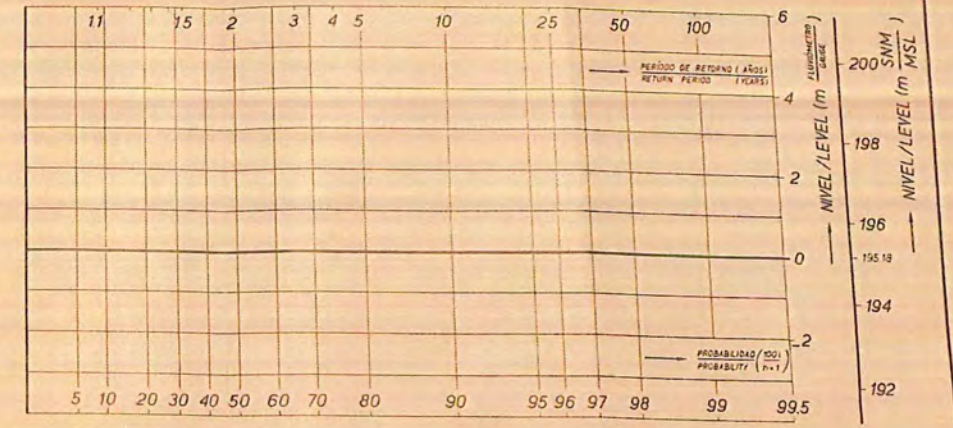
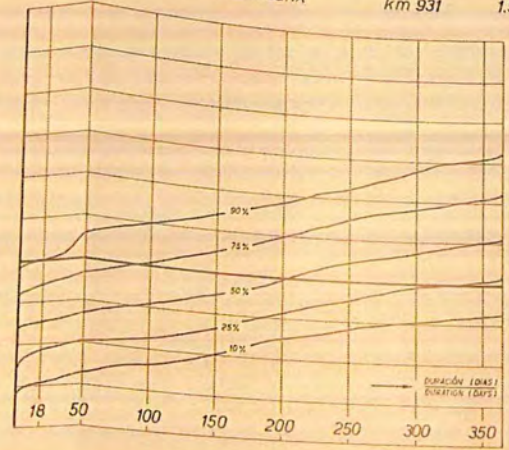
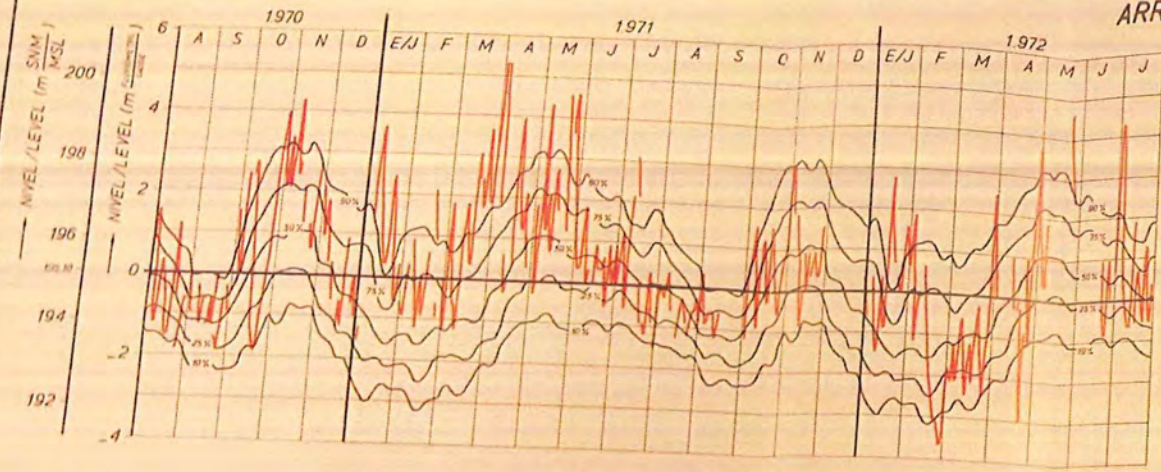
GAMBOTE

CANAL DEL DIQUE km 66 1960-1971

NIVELES DE AGUA WATER LEVELS		CALAMAR-GAMBOTE	
NIVELES DE AGUA 1970-1972		DAILY LEVELS 1970-1972	
CURVAS DE FRECUENCIA - CURVAS DE DURACIÓN		FREQUENCY CURVES - DURATION CURVES	
FRECUENCIA DE NIVELES EXTREMOS		FREQUENCY OF EXTREME LEVELS	
NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT			FIG. 2.3.8

ARRANCAPLUMAS

RÍO MAGDALENA km 931 1934 - 1971



PTO. SALGAR

RÍO MAGDALENA km 887 1936 - 1971

NIVELES DE AGUA / WATER LEVELS		ARRANCAPLUMAS - PTO. SALGAR	
NIVELES DE AGUA 1970-1972		DAILY LEVELS 1970-1972	
CURVAS DE FRECUENCIA - CURVAS DE DURACIÓN		FREQUENCY CURVES - DURATION CURVES	
FRECUENCIA DE NIVELES EXTREMOS		FREQUENCY OF EXTREME LEVELS	
NEDECO		RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT	FIG. 2.3.5

The water-level data of the gauge at Pto. Inmarco (also to be considered a main gauge-station in future) are given in Figure 2.3.9. As water-levels were only read on this gauge for a period of about one year, only the hydrograph has been presented. Downstream of Pto. Triunfo at the Hacienda San Fernando (Río Magdalena, km 822.5) high water-levels were read in the months of September and October 1971. These data were collected by INTEGRAL at the site where the future road-bridge Bogotá-Medellín is to be constructed. For the sake of completeness, these data are given in Figure 2.3.10.

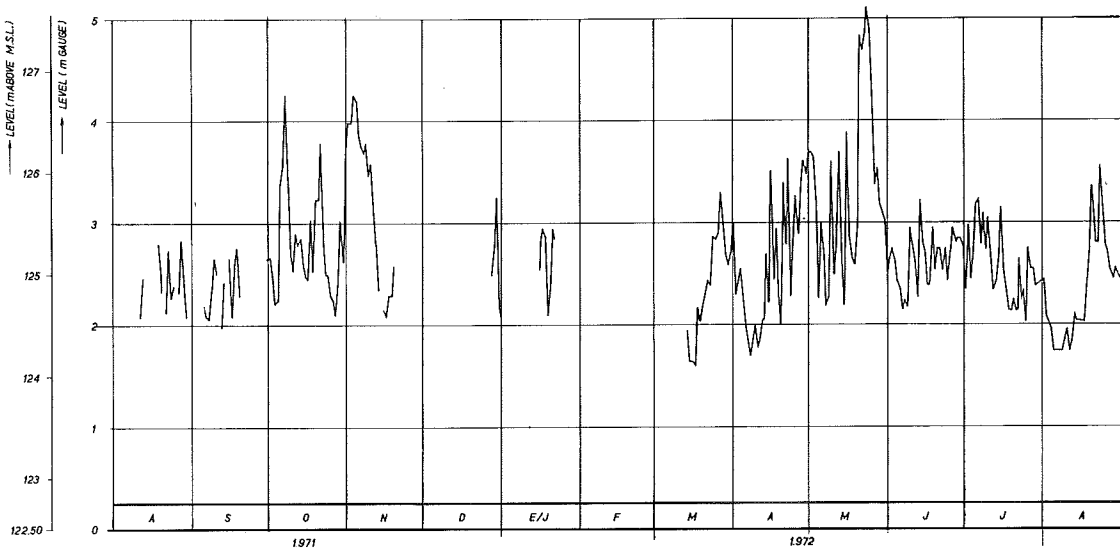


Figure 2.3.9 Water-levels at Pto. Inmarco

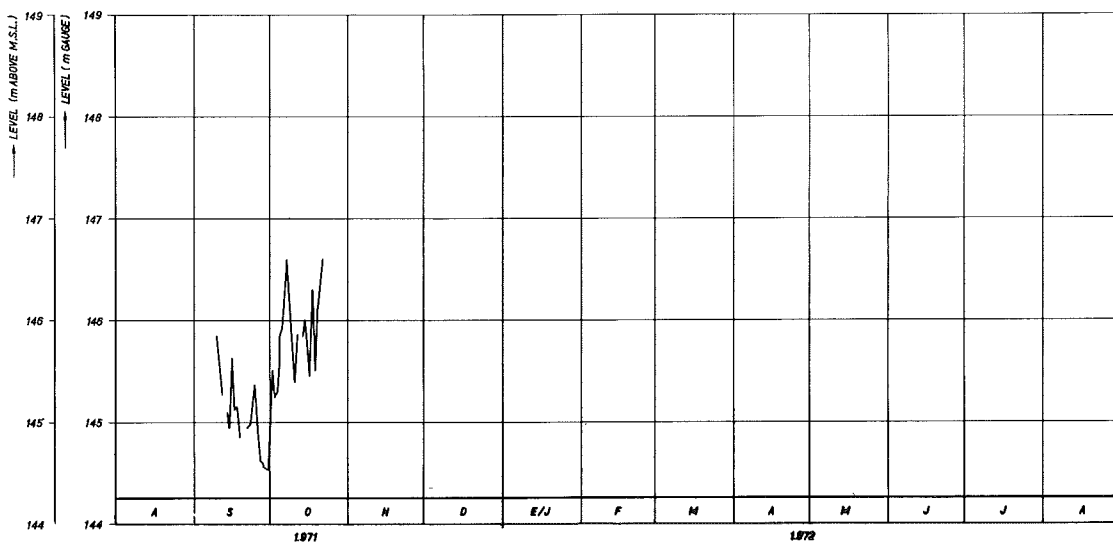


Figure 2.3.10 Water-levels at Pto. Triunfo

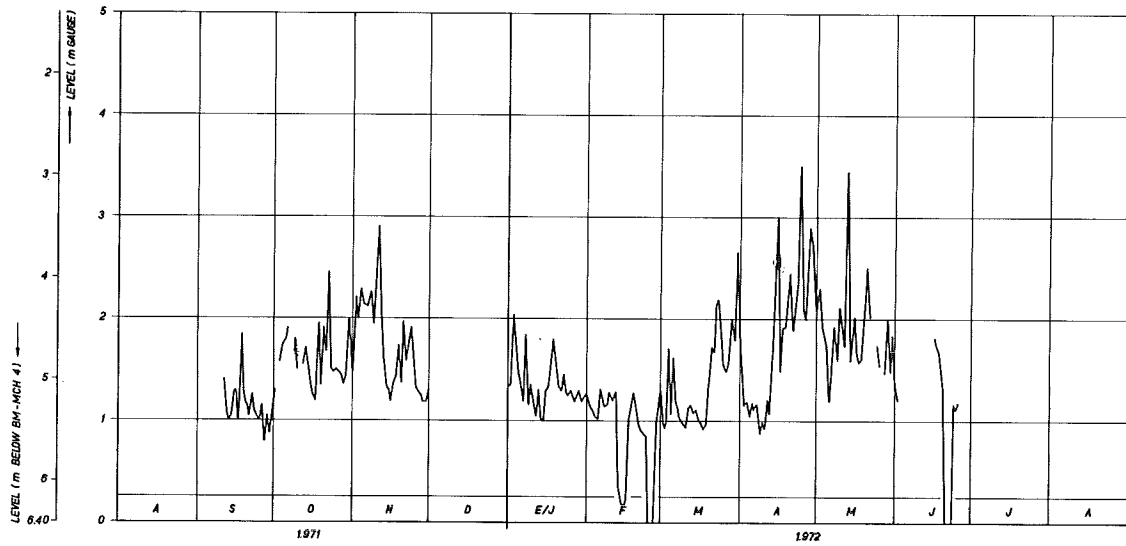


Figure 2.3.11 Water-levels in the Rfo Negro

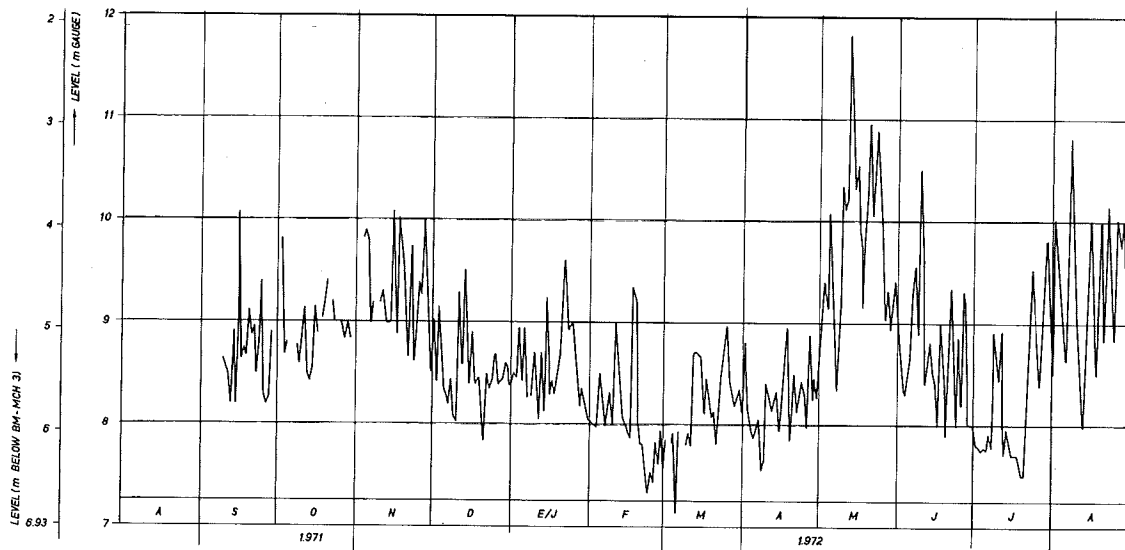


Figure 2.3.12 Water-levels in the Rfo La Miel

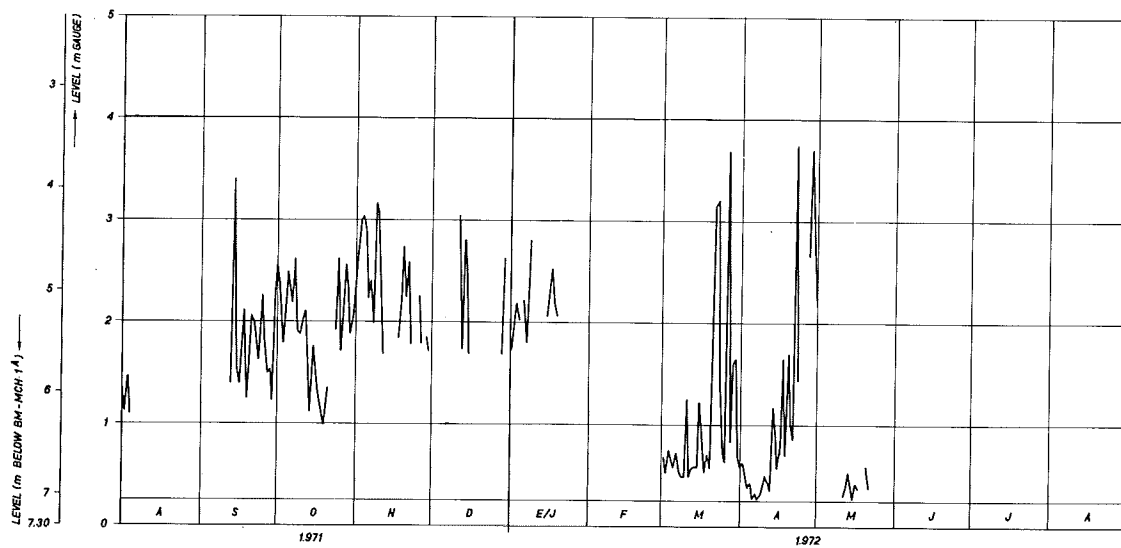


Figure 2.3.13 Water-levels in the Rfo Nare

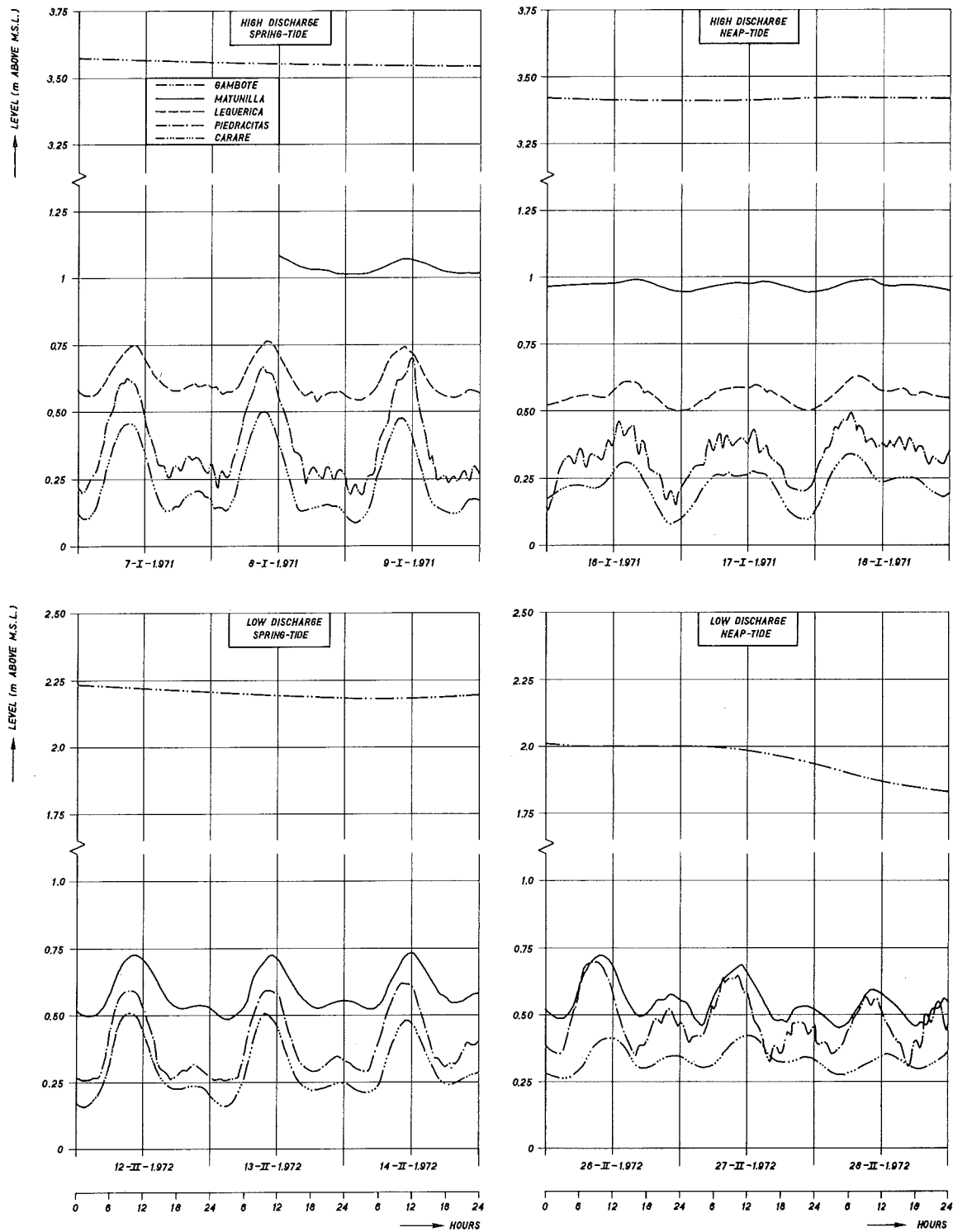


Figure 2.3.14 Water-levels along the Lower Region of the Canal del Dique

## II, 2.3

The water-level data of the stations which were installed in affluents of the Rfo Magdalena, viz., in the Rfos Negro, La Miel and Nare, are presented in Figures 2.3.11 to 2.3.13. Of these gauges again only the hydrographs have been given, as the available records cover only a period of about one year. The zero-levels of these gauges are as yet only connected to locally installed bench-marks, but in future connection should be made to the levelling net of the "Instituto Geográfico Agustín Codazzi" (IGAC) to relate these zero-levels also to M.S.L.

The available records of the automatic gauges in the Canal del Dique area are not all presented in this Report. An impression of the course of the water-levels along the lower region of this waterway can be obtained from Figure 2.3.14, where the water-levels are plotted for short periods only (3 days) at high and low discharges of the Canal del Dique, and during spring-tide and neap-tide in the Bahía de Cartagena and the Bahía de Barbacoas respectively.

All the water-level data and the elaborations presented in Figures 2.3.5 to 2.3.14 inclusive have been plotted in reference to the zero-levels of the gauges as presented in Table 2.3.1. However, the question arises whether the zero-levels of the main gauge-stations along the Rfo Magdalena, where waterlevels have been read for a period of more than 30 years, are valid for the whole period covered by the records. It is a known fact that discrepancies exist between the given zero-level of one and the same gauge by agencies such as ADENAVI (which copied the zero-level as given by the Julius Berger Konsortium), the SCMH or APRON Y DUQUE Ltda. It is also known that gauges were frequently shifted (e.g., the first gauge in La Dorada was installed by the Julius Berger Konsortium along the left bank; later on this gauge was shifted to the right bank), and it is doubtful whether a proper levelling to the reference benchmark was always carried out.

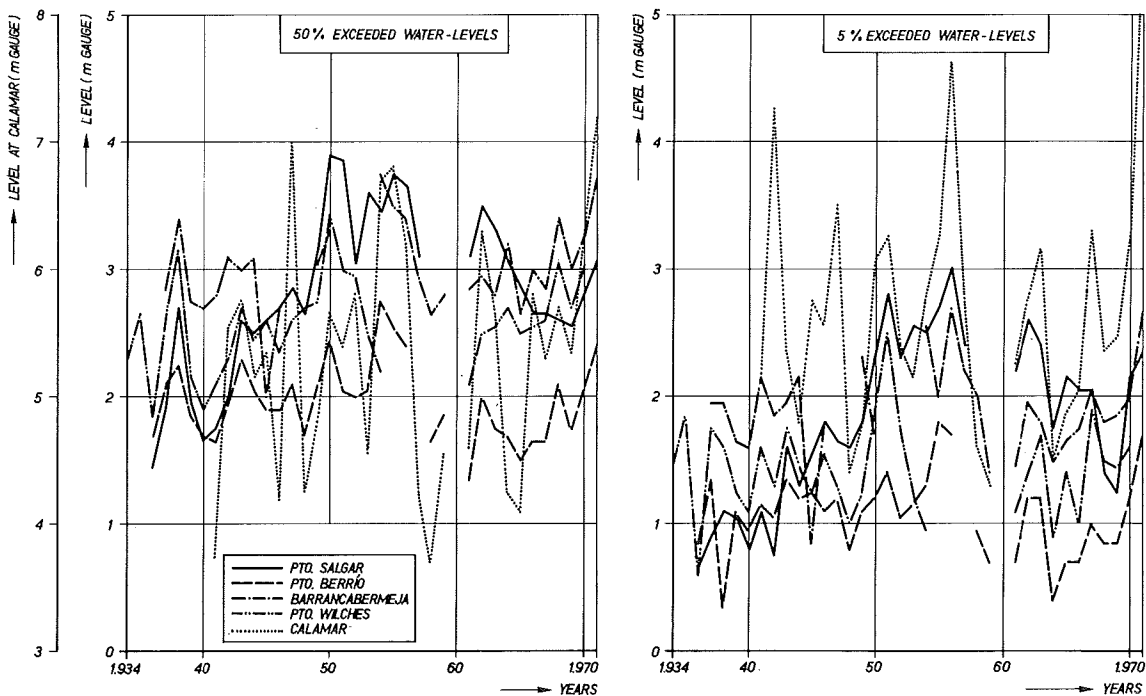


Figure 2.3.15 50% and 5% Exceeded Water-levels along the Rfo Magdalena

Whether the available records of water-level data need to be corrected with discrepancies in zero-levels was studied by the Mission. The 50% (182 days) and 5% (18 days) exceeded water-levels of the yearly duration-curves of Pto. Salgar, Pto. Berrfo, Barrancabermeja, Pto. Wilches and Calamar have been plotted in Figure 2.3.15. (The water-levels of the gauge at Arrancaplumas have not been included in this figure because the data were only elaborated in the final stage of the compilation of this Report and the gauge is situated far upstream).

The variations in these levels and the tendencies indicated by the curves are so random that discrepancies in the zero-levels of the gauges must not be taken into account. Consequently, the question arises whether these variations can be tied up with the yearly rainfall data. This question will be primarily answered by examining only the median water-level variations at Pto. Salgar (Figure 2.3.16) and the mean yearly runoff computed from the water-balance of the Río Magdalena basin, upstream of Pto. Salgar.

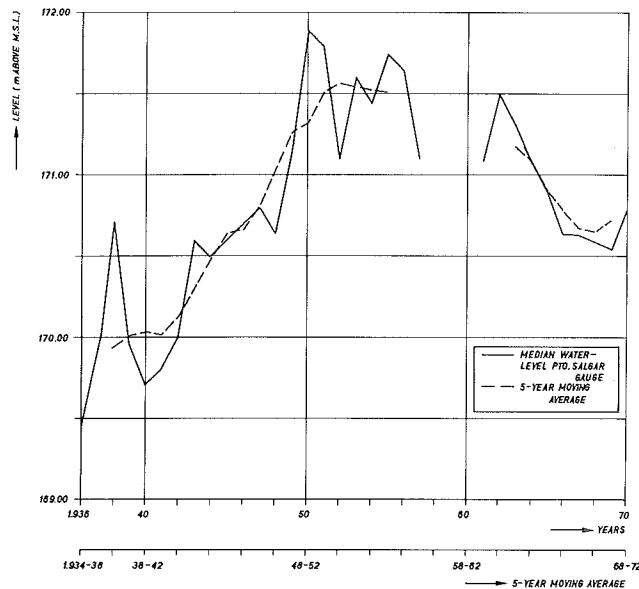


Fig. 2.3.16 Median Water-level at Pto. Salgar

For a time-period  $\Delta t$  the water-balance equation can be expressed as:

$$S(t) + \int_t^{t+\Delta t} (P-E-R)dt = S(t+\Delta t) \quad (2.3.3)$$

in which:

- P = precipitation (mm/year)
- E = evapo-transpiration (mm/year)
- R = runoff (mm/year)
- S = storage (mm)

II, 2.3

Setting up this storage-equation for a time-period of one year, the difference in storage at the beginning and the end of the considered year will be small compared with the other terms in Eq.(2.3.3). Consequently, Eq.(2.3.3) can be simplified to:

$$\int_t^{t+\Delta t} (P-E-R)dt = 0 \tag{2.3.4}$$

with  $\Delta t =$  one year.

The amount of the yearly rainfall has been determined by means of monthly rainfall-data (supplied by the SCMH) of 52 stations in the catchment area upstream of Pto. Salgar. The average rainfall depth over this area has been computed with the help of the Thiessen method [3]. (This method defines the zone of influence of each station by drawing lines between pairs of gauges, bisected with perpendiculars, and the assumption is made that the area enclosed by these intersecting perpendiculars has the same amount of rainfall as the enclosed gauge). In this way the amount of the yearly rainfall has been determined for a period of 26 years (1945-1971). However, in 34 of the 52 stations the rainfall data were only available for a period of 10 years (1961-1971), and the amount of rainfall for the remaining period (1945-1961) had to be determined on the basis of the data of stations with longer records (the latter are mainly concentrated in the Departamento de Cundinamarca, schematically indicated by area A in Figure 2.3.17).

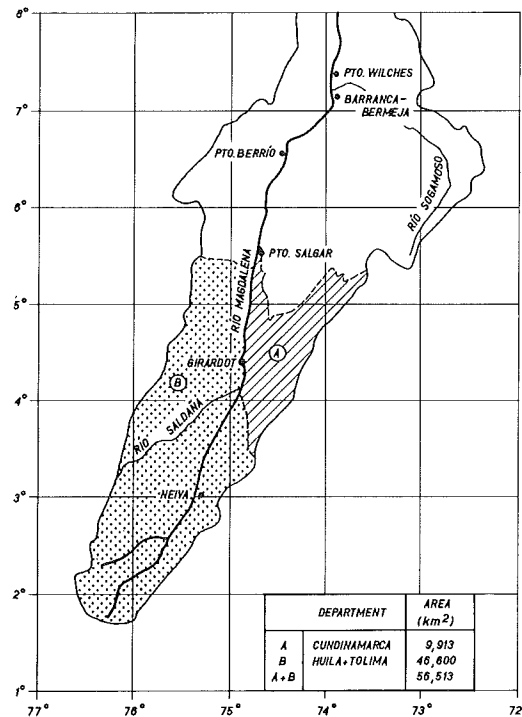


Figure 2.3.17 Catchment Area of the Rio Magdalena Upstream of Pto. Salgar

This has been done by the introduction of a correction factor ( $\alpha$ ) which is the statistical mean of the ratio between the rainfall depth of the total catchment area (A+B) and that of area A only over the period 1961-1971. The average correction factor is determined as  $\alpha=1.48$  (see Table 2.3.3).

Year	$\bar{P}$ (mm/year) area A (9,913 km <sup>2</sup> )	$\bar{P}$ (mm/year) area B (46,600 km <sup>2</sup> )	$\bar{P}$ (mm/year) total area A+B (56,513 km <sup>2</sup> )	$\alpha = \frac{\bar{P}_{(A+B)}}{\bar{P}_A}$
1970	1,190	2,020	1,870	1.57
69	1,130	1,890	1,753	1.55
68	1,220	1,915	1,789	1.47
67	1,150	1,685	1,586	1.38
66	1,100	1,750	1,627	1.48
65	1,090	1,735	1,613	1.48
64	915	1,760	1,603	1.76
63	1,300	1,830	1,730	1.33
62	1,230	1,730	1,638	1.33
1961	1,130	1,770	1,652	1.46
				$\bar{\alpha} = 1.48$

Table 2.3.3 Correction-factor ( $\alpha$ ) for Yearly Rainfall Upstream of Pto. Salgar

The yearly rainfall has been summarized in Table 2.3.4 and Figure 2.3.18 (anticipating the final analysis Figure 2.3.18 shows the same trend as the median water-level plotted in Figure 2.3.16).

Year	$\bar{P}_{A+B}$ (mm/year)	$E_T$ (mm/year) (T=20.5°C)	P-E (mm/year)	$h_{50\%}$ (m)	$R_{50\%}$ (m <sup>3</sup> /s)	$R_{50\%}$ total area (mm/year)
1970	1,870	980	890	2.80	1,160	648
69	1,753	965	788	2.55	970	547
68	1,789	969	820	2.60	1,020	569
67	1,586	939	647	2.65	1,060	592
66	1,627	945	682	2.65	1,060	592
65	1,613	943	670	2.90	1,240	692
64	1,603	942	661	3.10	1,360	759
63	1,730	961	769	3.30	1,520	848
62	1,638	947	691	3.50	1,670	933
61	1,652	950	702	3.10	1,360	759
60	1,696	952	744	-	-	-
59	1,868	982	886	-	-	-
58	1,233	853	380	-	-	-
57	1,587	934	653	3.10	1,360	759
56	1,855	980	875	3.65	1,800	1,005
55	2,270	1,021	1,249	3.75	1,880	1,050
54	1,873	981	892	3.45	1,650	921
53	1,978	994	984	3.60	1,740	972
52	1,706	957	749	3.10	1,360	759
51	2,201	1,015	1,186	3.80	1,910	1,067
50	1,949	989	960	3.90	2,000	1,118
49	1,572	937	635	3.15	1,400	781
48	1,301	874	427	2.65	1,060	592
47	1,656	950	706	2.80	1,160	648
46	1,264	860	404	2.70	1,080	603
1945	1,650	948	702	2.60	1,020	569

Table 2.3.4 Rainfall, Evapo-transpiration and Runoff Upstream of Pto. Salgar

II, 2.3

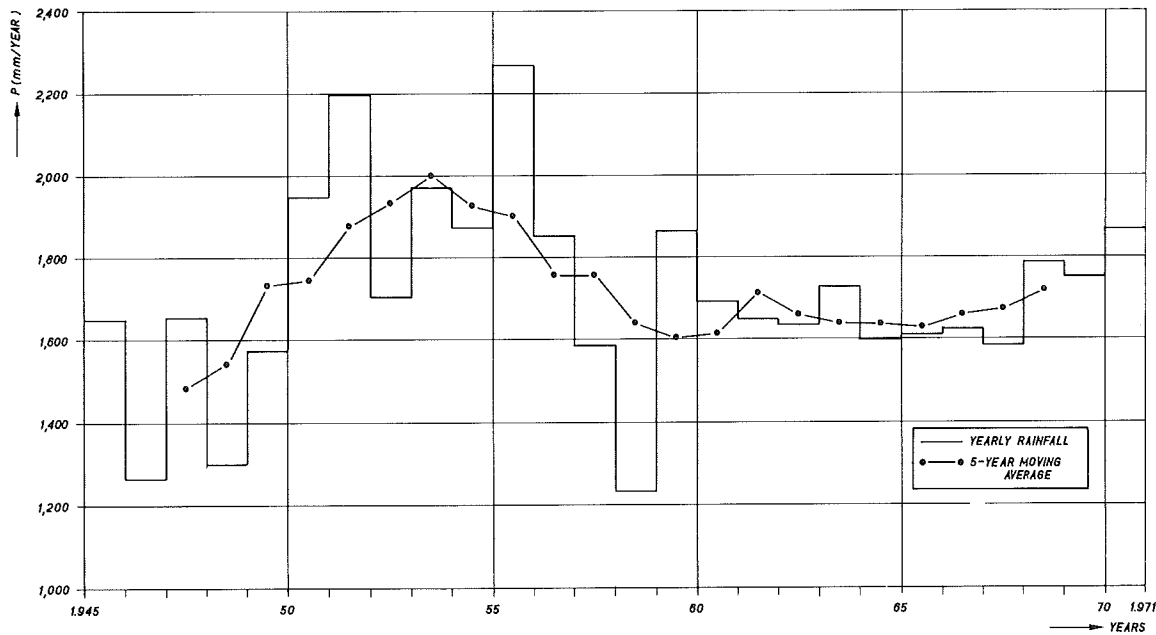


Figure 2.3.18 Yearly Rainfall Upstream of Pto. Salgar

The actual yearly evapo-transpiration has been calculated with Turc's formula [ 4 ] .

$$\bar{E}_a = \frac{\bar{P}}{\sqrt{0.9 + \frac{(\bar{P})^2}{(L(T))^2}}} \quad \text{if} \quad \frac{(\bar{P})^2}{(L(T))^2} \geq 0.1 \quad (2.3.5a)$$

and:

$$\bar{E}_a = \bar{P} \quad \text{if} \quad \frac{(\bar{P})^2}{(L(T))^2} \leq 0.1 \quad (2.3.5b)$$

The terms in the Eq. (2.3.5) represent:

$\bar{E}_a$  = mean actual evapo-transpiration (mm/year)

$\bar{P}$  = mean yearly rainfall (mm/year)

$L(T) = 325 + 21T + 0.9T^2$

T = mean yearly temperature in °C

The term  $L(T)$  can be seen as the potential evapo-transpiration  $\bar{E}_p$  (the evapo-transpiration under optimal availability of water), because if  $\bar{P} \rightarrow \infty$ ,  $\bar{E}_a$  approaches  $L(T)$ . The function  $L(T)$ , presented above, has been proposed by Langbein as a correction on Turc's formula for higher temperatures [ 5 ].

In the absence of better data a constant mean temperature of 20.5°C (determined from the isothermal map given in the "Atlas de Colombia", 2nd edition, 1969) has been assumed. Brief data available elsewhere in the Río Magdalena basin indicated a possible variation in the mean yearly temperature of about 3°C. This means that, apart from the possible error by the selection of the Turc/Langbein formula (Eq. 2.3.5), a variation of about 100 mm/year in the evapo-transpiration can be expected, due to the variation in temperature (see Figure 2.3.19). The calculated yearly amount of evapo-transpiration for the period 1945-1971 has also been given in Table 2.3.4.

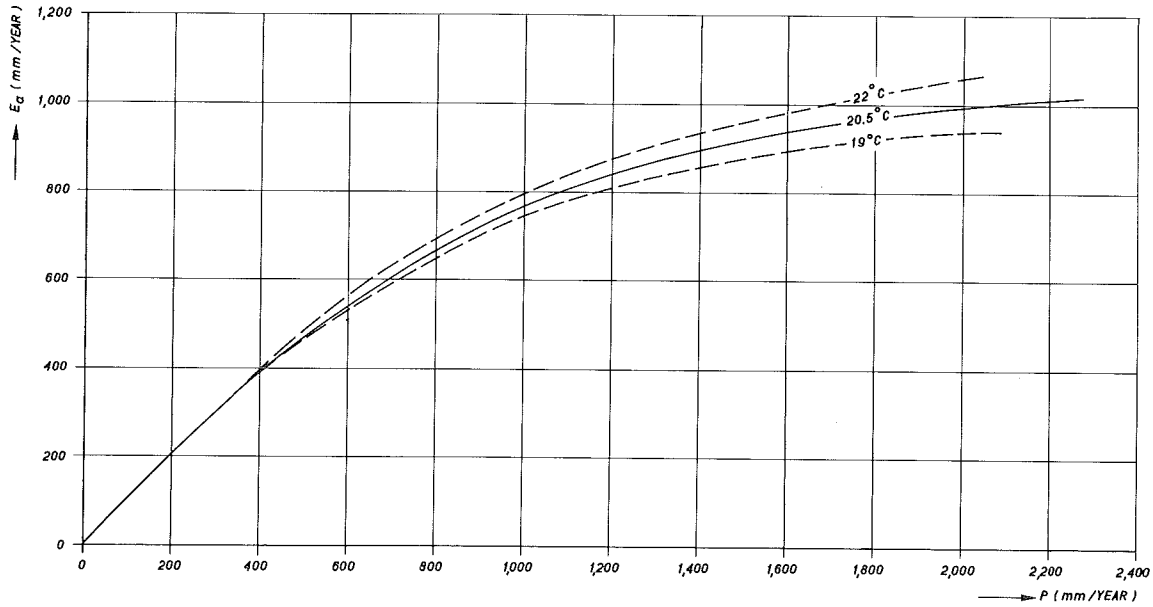


Figure 2.3.19 Evapo-transpiration According to the Turc/Langbein Formula

According to Eq.(2.3.4) the subtraction of  $\bar{E}_p$  from  $\bar{P}$  yields the yearly runoff. The median runoff has also been determined from the median water-level by means of the stage-discharge relation established for the Pto. Salgar gauge (see Table 2.3.4 and Figure 2.3.20). From these values the 5-year moving averages have been computed (Table 2.3.5).

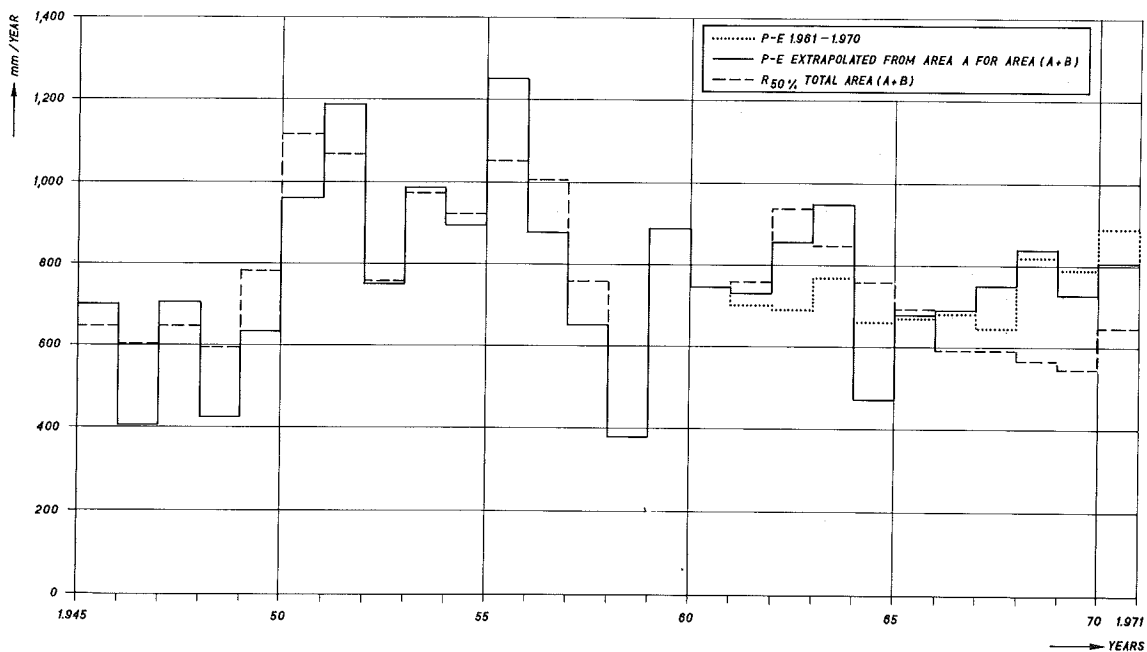


Figure 2.3.20 Comparison between "Measured" and Computed Runoff

II, 2.3

5-year moving average	$n=1-4$ $\sum_{n=1} P_{(A+B)}/5$	$n=1-4$ $\sum_{n=1} (P-E)_{A+B}/5$ ( $T=20.5^{\circ}C$ )	$n=1-4$ $\sum_{n=1} R_{50\%(A+B)}/5$
1970-66	1,725	765	590
69-65	1,674	721	598
68-64	1,644	696	641
67-63	1,632	686	697
66-62	1,642	695	765
65-61	1,647	699	798
64-60	1,664	713	-
63-59	1,717	758	-
62-58	1,617	681	-
61-57	1,607	673	-
60-56	1,648	708	-
59-55	1,763	809	-
58-54	1,764	810	-
57-53	1,913	931	941
56-52	1,936	950	941
55-51	2,006	1,012	954
54-50	1,941	954	967
53-49	1,881	903	939
52-48	1,746	791	863
51-47	1,736	783	841
50-46	1,548	626	748
1949-45	1,489	575	654

Table 2.3.5 Five-year Moving Average of Rainfall, Effective Rainfall, and Runoff Upstream of Pto. Salgar

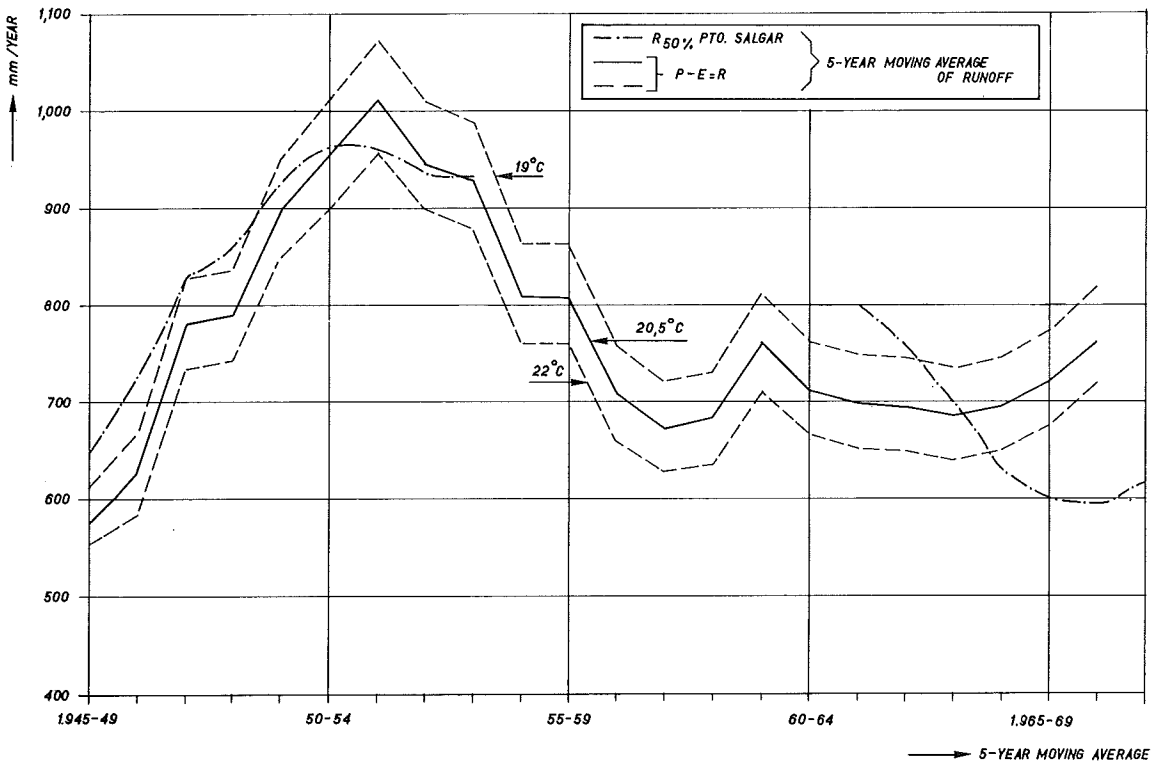


Figure 2.3.21 Comparison between Measured and Computed Runoff at Pto. Salgar

As can be seen from Figure 2.3.21, the computed runoff shows the same tendency as the measured one. From this analysis the conclusion can be drawn that the increase of the median water-levels over the period 1948-1955 can be explained by an increase of the total amount of rainfall. Between 1965 and 1972 the agreement is less than in the foregoing period, and although an explanation of this is difficult to give, the most obvious explanation would be that during the period 1965-1971 either:

- The climate has changed, causing an increase in temperature and consequently higher evapo-transpiration; or
- the observed water-levels at the Pto. Salgar gauge are less reliable.

After these comments on the reliability of the zero-level of the gauge at Pto. Salgar and the relation between the water-level variations and the rainfall data in the Rfo Magdalena basin upstream of Pto. Salgar, the water-level data of the other main gauge-stations along the Rfo Magdalena (Pto. Berrfo, Barrancabermeja and Pto. Wilches) will now be considered. The water-level data of the Calamar gauge cannot be taken into account because of the great distance from the nearest gauge (Pto. Wilches), the vast storage areas along the Rfo Magdalena and the influence of the Rfo Cauca and (although less important) the Rfo César.

To start with, the hydrographs of the four main gaugestations (Pto. Salgar included) have been checked on the zero-level of the gauge mentioned on the hydrographs (given by ADENAVI), the completeness of the hydrographs, and the linking of the hydrographs in the consecutive years (the difference in water-level recorded on December 31 in the foregoing year and January 1 in the following year). The latter information must not be regarded as a difference in zero-level of the gauge in the consecutive years as allowance must also be made for a water-level variation occurring between the gauge reading on December 31 and that on January 1. All this information is presented in Table 2.3.6.

Puerto Salgar				Puerto Berrfo				Barrancabermeja				Puerto Wilches			
Year	Zero-level	Linking	Missing Data	Year	Zero-level	Linking	Missing Data	Year	Zero-level	Linking	Missing Data	Year	Zero-level	Linking	Missing Data
1971	168.00			1971	108.39			1971	71.80			1971	62.55		
70	168.00	+ 0.15		70	108.39	+ 0.05		70	71.80	0.00		70	62.55	0.00	some days in March, June and July
69	168.00	+ 0.50	some days in January	69	108.39	+ 0.20		69	71.80	+ 0.55		69	62.55	+ 0.15	July
68	168.00	- 0.15	some days in December	68	108.39	+ 0.14		68	71.80	- 0.35		68	62.55	+ 0.10	
67	168.00	+ 0.30		67	108.39	+ 1.00		67	71.80	- 0.30		67	62.55	+ 0.05	
66	168.00	+ 0.45		66	108.39	+ 0.06		66	71.80	+ 0.05		66	62.55	0.00	
65	168.00	- 0.90		65	108.39	+ 0.14		65	71.80	+ 0.40		65	62.50	- 0.38	
64	168.00	+ 0.10		64	108.39	0.00	some days in February	64	71.80	+ 0.40		64	62.50	+ 0.10	
63	168.00	- 1.08		63	108.39	- 0.60		63	71.80	+ 0.56		63	62.50	- 0.15	
62	168.00	+ 0.14		62	108.39	0.00		62	71.80	0.00		62	62.50	+ 0.10	
61	168.00	- 0.64		61	108.39	0.00		61	71.80	+ 0.52		61	62.50		last 10 days of Dec.
60	-		whole year	60	-		whole year	60	-		whole year	60	-		whole year
59	168.00			59	108.39			59	71.80			59	62.55		
58	-		whole year	58	108.39	- 0.35		58	71.80	0.00		58	-		whole year
57	168.00			57	-		whole year	57	71.80	+ 0.60		57	-		whole year
56	168.00	- 0.05		56	108.39			56	71.80	- 1.30		56	62.55		
55	168.00	- 0.35		55	108.39	0.00		55	71.80	- 0.30		55	-		whole year
54	168.00	+ 1.00		54	108.39	- 0.20		54	71.85	+ 0.10	January to March	54	62.55		
53	168.00	0.00		53	108.39	+ 0.10		53	-		whole year	53	62.55	+ 0.15	
52	168.00	+ 0.30		52	108.39	+ 0.15		52	-		whole year	52	62.55	+ 0.30	
51	168.00	+ 0.45		51	108.39	+ 0.05		51	-		whole year	51	62.55	0.00	
50	168.00	+ 0.15		50	108.39	+ 0.10		50	71.80		November	50	62.55	+ 0.45	
49	168.00	0.00		49	108.39	+ 0.05		49	71.80	+ 0.40	January to March 15	49	62.55	0.00	
48	168.00	- 0.30		48	108.39	- 0.05		48	-		whole year	48	62.55	- 0.05	
47	168.00	+ 0.05	April to July	47	108.39	+ 0.10		47	-		whole year	47	62.55	+ 0.10	
46	168.00	- 0.25		46	108.39	+ 0.05		46	71.80	- 1.00	June and August	46	62.55	- 0.10	January
45	168.00	+ 0.45		45	108.39	0.00		45	71.80	+ 1.30		45	62.55		December
44	168.00	+ 0.45		44	108.39	+ 0.05		44	71.80	- 0.45		44	62.20		
43	167.67	- 0.25		43	108.39	0.00		43	71.80	+ 0.25		43	62.20	+ 0.30	
42	167.67	0.00		42	108.39	+ 0.30		42	71.80	- 0.35		42	62.20	+ 0.45	
41	167.67	+ 0.25		41	108.39	+ 0.25		41	71.80	+ 0.15		41	62.20	+ 0.05	
39	167.67	0.00		39	108.39	- 0.40		39	71.80	- 0.10		40	62.20	+ 0.10	April
38	167.73	+ 0.35		38	108.39	+ 0.20	July	38	71.80	+ 0.45		39	62.00	+ 0.15	May
37	167.73	+ 0.10		37	108.39	+ 0.20		37	71.80	0.00		38	62.20	+ 0.45	
1936	168.00	+ 0.40		1936	108.39	- 0.10		1937	71.80			37	62.20	+ 0.45	
												36	62.00	- 0.50	
												35	62.00	+ 0.15	
												1934	62.00	- 0.10	

Table 2.3.6 Check on Hydrographs of Pto. Salgar, Pto, Berrfo, Barrancabermeja and Pto. Wilches

II, 2.3

The median yearly discharges of these four main gauge-stations were then computed from the median water-level by means of the stage-discharge relations (these relations are given in Para. 3.5.3). The median water-levels of the main gaugestations are compiled in Table 2.3.7.

Year	Median Water-level at Gauge-station				Year	Median Water-level at Gauge-station			
	Pto. Salgar	Pto. Berrfo	Barranca-bermeja	Pto. Wilches		Pto. Salgar	Pto. Berrfo	Barranca-bermeja	Pto. Wilches
1971		1.95	2.90		1952	3.10	1.55	-	2.40
70	2.80	1.60	2.45	2.45	51	3.80	1.65	-	2.50
69	2.55	1.30	2.25	2.15 *	50	3.90	2.00	2.50 *	2.90
68	2.60	1.65	2.60	2.50	49	3.15	1.65	2.25 *	2.20
67	2.65	1.20	2.05	2.05	48	2.65	1.25	-	2.15
66	2.65	1.20	2.20	2.00	47	2.80 *	1.65	-	2.00
65	2.90	1.05	1.85	1.95	46	2.70	1.50	1.90 *	1.80 *
64	3.10	1.25	2.40	2.15	45	2.60	1.45	1.20/2.20 **	1.95 *
63	3.30	1.30	2.00	2.00	44	2.50	1.60	2.30	1.90
62	3.50	1.55	2.15	1.95	43	2.60	1.75	2.20	2.15
61	3.10	1.40	2.10	1.79 *	42	2.00	1.55	2.35	1.90
60	-	-	-	-	41	1.75	1.20	1.90	1.55
59	3.20	1.40	2.00	1.85	40	1.65	1.25	1.80	1.35 *
58	-	1.25	1.90	-	39	1.95	1.45	1.90	1.60 *
57	3.10	-	2.15	-	38	2.70	1.80 *	2.60	2.50
56	3.65	1.95	2.60	1.60	37	1.95	1.65	2.05	2.05
55	3.75	2.10	2.70	-	36	1.45	1.25	-	1.30
54	3.45	2.30	2.95 *	1.65	35	-	-	-	2.10
1953	3.60	1.65	-	1.95	1934	-	-	-	1.75

\* Approximated median water-level, because of incomplete hydrograph  
 \*\* Level shift of 1.00 m (in agreement with Tables 2.3.6 and 2.3.8)

Table 2.3.7 Median Water-levels at Main Gauge-stations

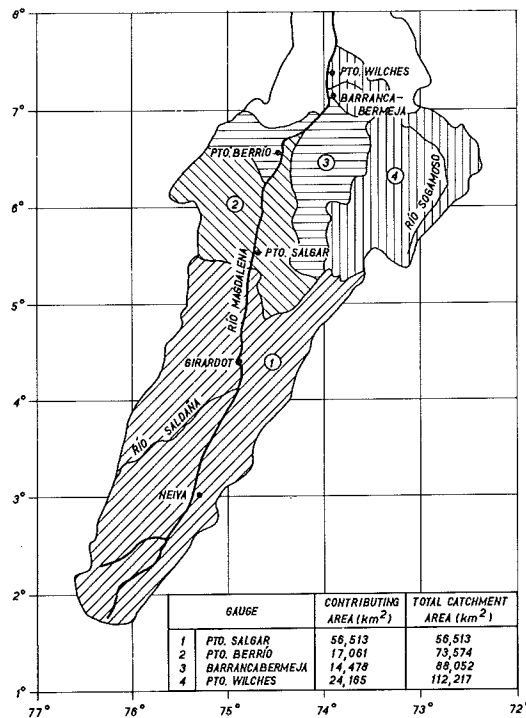


Figure 2.3.22 Schematized Catchment Areas in the Río Magdalena Basin

According to the law of continuity, the mean yearly runoff must increase in a downstream direction as a result of the contributions to the river runoff by the intervening catchment areas. These contributing areas have been presented schematically in Figure 2.3.22. The check on the law of continuity is given in Table 2.3.8.

Mean Yearly Runoff at Main Gauge-stations						
Year	Pto. Salgar	Pto. Berrío	Barrancabermeja	Pto. Wilches	R(t) Barrancabermeja + R(t) Río Sogamoso	Remarks
1971		2,970	3,600			
70	1,160	2,370	2,980	3,670	3,428	
69	970	1,940	2,720	3,180*)	-	
68	1,020	2,450	3,160	3,750	3,707	
67	1,060	1,790	2,460	3,025	2,911	
66	1,060	1,790	2,670	2,940	-	
65	1,240	1,575	2,230	2,880	2,722	
64	1,360	1,860	2,920	3,180	3,371	
63	1,520	1,940	2,390	2,940	2,964	
62	1,670	2,295	2,600	2,800	3,085	
61	1,360	2,010	2,520	2,650*)	2,907	
60	-	-	-	-	-	
59	1,450	2,080	2,390	2,710	-	
58	-	1,860	2,270	-	-	
57	1,360	-	2,600	-	-	
56	1,800	2,970	3,160	2,310	-	no continuity
55	1,880	3,140	3,290	-	-	
54	1,650	3,440	3,660*)	2,390	-	no continuity
53	1,740	2,450	-	2,880	-	
52	1,360	2,295	-	3,590	-	
51	1,910	2,450	-	3,750	-	
50	2,000	2,990	3,040*)	4,380	-	
49	1,400	2,450	2,720*)	3,260	-	
48	1,060	1,860	-	3,180	-	
47	1,160*)	2,450	-	2,940	-	
46	1,080	2,225	2,270*)	2,660*)	-	
45	1,020	2,150	1,580/2,670**)	2,880*)	-	continuity if level shift is applied
44	940	2,370	2,780	2,800	-	
43	1,020	2,600	2,670	3,180	-	
42	630	2,295	2,840	2,800	-	no continuity
41	500	1,790	2,270	2,220	-	no continuity
40	460	1,860	2,180	1,950*)	-	no continuity
39	600	2,150	2,270	2,310*)	-	
38	1,080	2,675*)	3,160	3,750	-	
37	600	2,450	2,460	3,025	-	
36	370	1,860	-	1,900	-	
35	-	-	-	3,100	-	
1934	-	-	-	2,550	-	

\* Deduced from approximated median water-level  
 \*\* Level shift of 1.00 m

Table 2.3.8 Mean Yearly Runoff at Main Gauge-stations

A comparison between the data of Pto. Salgar and Pto. Berrío shows that the contribution of the catchment area between these two gauges (indicated in Figure 2.3.22 with 2) varies roughly between 400m<sup>3</sup>/s and 1,500 m<sup>3</sup>/s. This means that the yearly rainfall in this area must range from 1,100 - 4,100 mm/year. The available discharge data of the three main affluents in this area (the Ríos Negro, La Miel and Nare) as well as the data of the isohyetal map presented in the "Atlas de Colombia" support a high amount of rainfall in this area.

## II, 2.3

A comparison between the data of Pto. Berrío and Barrancabermeja shows that the contribution of the catchment area between these two gauges is rather low. It does not seem correct to attribute this only to a decrease of the rainfall in this area. Other explanations which need to be considered are: a less accurate stage-discharge relationship and/or a less accurate median water-level. In 1945 the gauge at Barrancabermeja must have been shifted, because otherwise the continuity principle does not hold (see Table 2.3.8).

For the period 1961-1971 the median yearly discharge at Pto. Wilches (fifth column of Table 2.3.8) has been compared with the sum of the discharge at Barrancabermeja and the runoff of the Río Sogamoso (sixth column). These figures agree reasonably well. The zero-levels of the gauge at Pto. Wilches, as mentioned on the hydrographs, must be wrong in the period 1954-1956, while the values in the period 1940-1942 must be treated with suspicion.

Summarizing the foregoing analysis, it can be concluded that no systematic shifts of the gauges (resulting in other zero-levels of the gauges than those mentioned on the hydrographs) can be pointed out. Nevertheless, incidental shifts of the gauges, as far as they could be demonstrated from the available information, have taken place. It is recommended that in future the agency responsible for the gauge network uses the zero-levels as given in Table 2.3.8. For the present, it suffices to conclude that along the Río Magdalena a reasonable agreement between the recorded water-levels and the rainfall could be found, and that the incidental shifts of the gauges will hardly influence the elaborations of the total records of water-level data (1936-1971) as carried out by the Mission.

The data of the gauge-stations along the Canal del Dique will now be considered. The location of the gauges and the period covered by the available records have already been given in Table 2.3.1. In addition to these gauges a net of benchmarks along the Canal del Dique is available which was installed by Mantilla (DICON/Junta del Canal del Dique) [6,7] and which has been used as a reference for all the water-level data gathered at an earlier date. To be able to make a comparison with the data gathered by the Mission, the latter have also been related to the net of Mantilla. One remark must be made here. In Colombia it is common practice to express the level of M.S.L. along the Canal del Dique as 100, instead of zero as used along the Río Magdalena. To prevent confusion, and in accordance with the data of the Río Magdalena, all the levels given in this Report (as far as they are related to M.S.L.) refer to a M.S.L. of zero.

From the available data the conclusion can be drawn that M.S.L. in the Bahía de Cartagena is + 0.33m (or 100.33 m if the zero of the net of the Junta del Canal del Dique/DICON is referred to as 100).

Levellings carried out by the Mission showed some discrepancies in the net of DICON. Conclusions are therefore difficult to draw, because the datums should be very accurate in view of the small water-level gradients which, generally, occur along the Canal del Dique. Nevertheless, the datums which were derived from the available information and used by the Mission are given in Table 2.3.9.

Description/Location	Junta del Canal del Dique/ Mantilla (DICON)	Related to H.S.L. (= 0 m)	IGAC
BH DICON No. 1	102.113	1.783	
BH DICON No. 045	130.700	30.370	
BH DICON No. 059	114.923	14.593	
BH A. CODAZZI No. 131			11.747
BH A. CODAZZI No. 179			6.097
BM JUNTA No. 11 (Gambote)	104.819	4.489	4.202
Carare, base of automatic gauge	101.282	0.952	
Piedracitas, base of automatic gauge	101.526	1.196	
BH Church Pasacaballos	104.136	3.806	3.508
Pasacaballos, zero of gauge	99.976	-0.354	
BH Lequerica	101.334	1.004	
Lequerica, base of automatic gauge	101.715	1.385	
Recreo, monument	103.555	3.225	
Recreo, zero of gauge	100.455	0.125	
BH Matunilla	101.443	1.113	
Matunilla, base of automatic gauge	101.849	1.519	
Zero of gauges along the Caño Matunilla, F2	100.063	-0.267	
F3	100.045	-0.285	
F4	99.898	-0.432	
F5	99.935	-0.395	
F6	99.859	-0.471	
Correa, base of automatic gauge	102.536	2.206	

Table 2.3.9 Datums of Benchmarks and Zero-levels of Gauges along the Canal del Dique

For all the levels of those benchmarks which are not included in Table 2.3.9, the values as used by Mantilla [6,7] should be kept. However, it is important to further check the levels of all the benchmarks along the Canal del Dique. In the course of the study it was suggested that the IGAC be asked to carry out a new levelling in the lower region of the Canal del Dique, but these data are not yet available. As soon as they are, it is recommended that all the benchmarks of the net of the Junta del Canal del Dique (DICON) are levelled and related to the new net of IGAC. The levelling of the gauges at Carare (Bahía de Cartagena), Piedracitas (Bahía de Barbacoas), Lequerica, Matunilla and Correa should then be repeated too.

From several levellings as carried out by the Mission it follows that the zero of the net of DICON lies 0.62 m below the zero as applied by IGAC. Mantilla mentioned in his Reports that he used the same base as established in the past by the Julius Berger Konsortium which used M.S.L. at Bocas de Ceniza as a reference. This appears strange, because a difference of 0.62 m was also found in the comparison between Mantilla and IGAC, while the latter uses M.S.L. as a reference too (obtained from records at several places along the Caribbean and Pacific Coast).

The gauge in Calamar was levelled by the Mission a number of times to the benchmark (No. 26N7, 1970) of IGAC, located close to the church. It appeared that the zero of this gauge lies 0.35 m below M.S.L.

2.3.5. Reduction-levels

For the comparison of hydrographic measurements, e.g., longitudinal soundings of a river or detailed soundings of a certain river section, the assumption of a reduction-level is required. In Colombia it is the practice to relate all water-levels to M.S.L. However, especially for navigation purposes, such a reduction-level is not very suitable. A river operator needs to know the Least Available Depth (L.A.D.) in a certain river stretch in relation to the pertaining water stages and, therefore, the reduction-level should have a relation to the daily gauge readings. (In canals with a more or less constant bed-level the water-levels can be related to the bed-level, but in free flowing rivers the changes in bed-level are so great and occur so rapidly, that this procedure cannot be followed).

It will be clear that for navigation purposes the reduction-level should be related to low water-levels (low discharge) and, moreover, that anywhere along the river the probability of the occurrence of a lower water-level for a certain duration is equal; in other words, that on an average the reduction-level at each place along the river is exceeded during the same number of days per year.

The reduction-level, the so-called Low River Level (L.R.L.), which has been adopted by the Mission is defined as the level with an exceedance of 95% of the year. This means that on an average only during 18 days of a year will the actual water-level be lower than the L.R.L.

A study of the definition of the L.R.L. makes it clear that the reduction-level must be determined from the average duration-curves and not from the 50% duration, because the actual water-level must on an average be lower only during 18 days of a year. The line connecting the reduction-levels of the successive stations is a curve which more or less follows the average longitudinal profile of the river. However, it cannot be considered as an actual occurring water stage. The L.R.L. values (as read on the gauge) as they were adopted by the Mission along the Río Magdalena and the Canal del Dique are given in Table 2.3.10.

Rfo Magdalena			Canal del Dique		
Station		L.R.L. (m)	Station		L.R.L. (m)
Pto. Salgar	(km 887)	1.68	Calamar	(km 0)	2.13
Pto. Inmarco	(km 773)	0.20	Sta. Lucfa	(km 10)	1.95
Pto. Berrfo	(km 730)	0.60	Soplaviento	(km 33)	1.44
Barrancabermeja	(km 631)	0.99	Gambote	(km 66)	0.70
Pto. Wilches	(km 597)	0.75	Correa	(km 82.5)	0.51
Calamar	(km 91)	2.13	Matunilla	(km 100)	0.30
			Lequerica	(km 108)	0.21
Gamarra	(km 473)	36.70 (above M.S.L.)	Bahfa de Barbaosas		0.12 (L.L.W.S.)
			Bahfa de Cartagena		0.12 (L.L.W.S.)

Table 2.3.10 L.R.L. Values as Read on the Gauge

Some remarks are necessary about some of the Low River Levels (as presented in Table 2.3.10). For those gauge-stations with a long record of water-level data, the L.R.L. values have been determined from the average duration-curves, while for those gauge-stations with only a short record of water-level data, the L.R.L. values have been determined from other sources. Along the Río Magdalena the L.R.L. at Pto. Inmarco (0.20 m) could be found by means of the "line of equal discharge" for the measuring-stations of

Pto. Inmarco and Pto. Berrfo (this is further treated in Para. 2.5). The L.R.L. at the Gamarra station was found from the available record of water-level data (which covered a period of about 10 years) and could only be determined in relation to M.S.L., to wit, 36.70 m above M.S.L. (this value corresponded to a level of 1.28 m on the gauge which was installed by the Mission, but washed away by the current in 1972).

The path of the L.R.L. along the Río Magdalena is presented in Figure 2.3.23. For comparison, the water-levels recorded at high and low river stages have also been given.

At the gauge-station Calamar the L.R.L. could also be found from an average duration-curve. By means of relation-curves (Para. 2.5) the L.R.L. could be extended along the Canal del Dique up to Sta. Lucía and Gambote. For Soplaviento the reduction-level was adopted as 1.44 m found from the interpolation between the L.R.L. values at Sta. Lucía and Gambote.

Before dealing with the L.R.L.'s in the Lower Canal del Dique area, some remarks must be made regarding the tidal influence on the definition of the L.R.L. In fact, another reduction-level has to be determined in coastal regions, in view of the tide penetrating inland (another notation is therefore often used: Chart Datum or Standard Low Water). Nevertheless, at sea the reduction-level must coincide with the reference level in use. Different methods are used to define the reduction-level at sea, e.g., Mean Lower Low-water Spring (M.L.L.S.), Mean Low-water Spring (M.L.S.), Mean Low Water (M.L.W.), Mean Sea Level (M.S.L.), etc. The prevailing tidal constituents mainly determine which definition can best be adopted. Studying the tide in the Bahía de Cartagena (gauge-station Carare), the following remarks can be made (Figure 2.3.24).

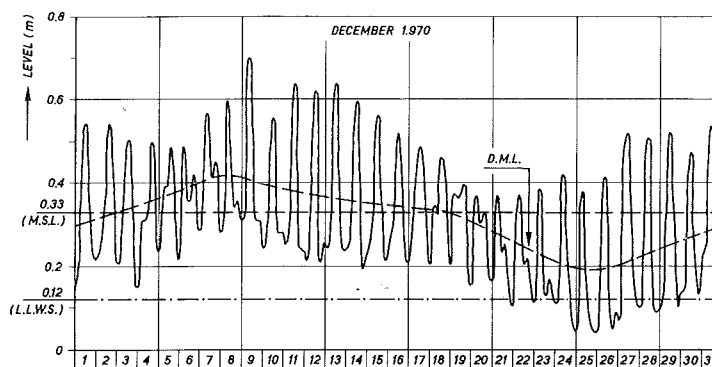


Figure 2.3.24 Tide (schematically) in the Bahía de Cartagena (Carare)

The tide in the Bahía de Cartagena (as elsewhere along the Caribbean Coast) can be characterized as a mixed tide. The diurnal tidal constituents  $O_1$ ,  $K_1$  and  $P_1$  predominate during springtides, apparently transforming the tide to a diurnal one, while especially during neap-tides the influence of the semi-diurnal constituents  $M_2$ ,  $K_2$  and  $S_2$  increases. Mixed tides are characterized by a great diurnal inequality, which means that in the tidal curve successively a high high-water, a high low-water, a low high-water and a low low-water can be clearly distinguished.

II, 2.3

A harmonic analysis of the tide is possible from a month's series of hourly readings. It is not advisable to take a longer series as the upland discharge (with its influence on the tide) can hardly be considered to remain constant during a month throughout the year. Moreover, the seasonal variation at sea can no longer be neglected if longer periods are considered (although this influence is generally smaller than that of the upland discharge). The harmonic analysis can be carried out according to, e.g., the method as published in the Admiralty Manual of Tides, which should be referred to for an explanation of the principles of this method [8]. Let it suffice to say here that not only can the principal astronomic constituents  $O_1$ ,  $K_1$  and  $F_1$  (diurnal),  $M_2$ ,  $K_2$  and  $S_2$  (semi-diurnal) be computed from a month series, but the shallow water constituents  $2MS_2$  (semi-diurnal),  $M_4$  and  $MS_4$  (quarter-diurnal) and the less-known  $MS_f$  (fortnightly) as well. Each constituent is defined by its modulus  $H$  (in ft) and its phase-lag  $g$  (in arc-degrees). The modulus is defined as half the range of the corresponding sinusoidal wave, and  $g$  indicates the lag between the astronomically determined phase and the actual phase of the constituent. The astronomically determined phase is usually indicated by  $(v+u)$  (or  $E+u$ ) and can be found in the Admiralty Tide Tables. The analysis also gives the mean level during the analysed month. When its height is expressed in relation to Chart Datum (the reference-level used on hydrographic charts), it is mostly indicated by  $Z_0$ . The Chart Datum defined in the Bahía de Cartagena corresponds to Low Low-Water Spring (L.L.W.S.) which has a value of 0.12 m.

In Figure 2.3.24 the Daily Mean Levels (D.M.L.; the arithmetical mean value of the daily hourly readings) has been plotted as well. The variation of the D.M.L. is accounted for by the  $MS_f$ -constituent, which period of  $14\frac{1}{2}$  days corresponds to the variation of the spring-tides and neap-tides.

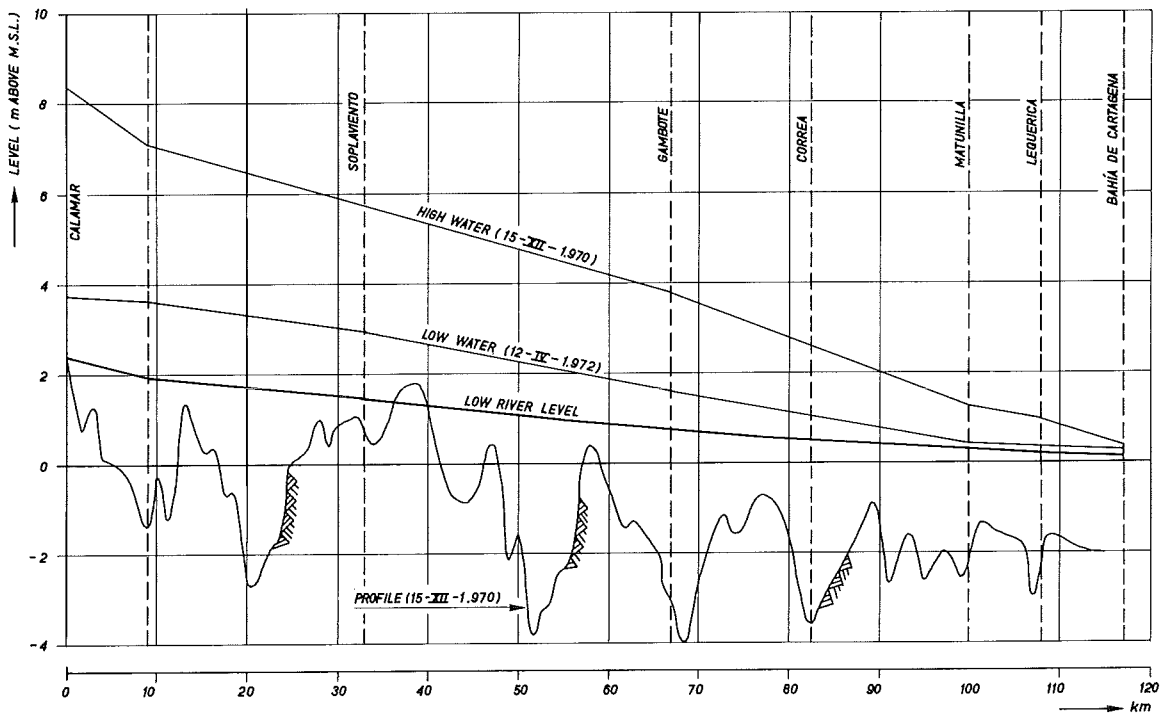


Figure 2.3.25 Reduction-level along the Canal del Dique

After these remarks about the tidal influence, it follows that in coastal regions the Low River Level which is exceeded 95% of the time should be corrected with a tidal reduction. However, in view of the facts that the reduction-level should coincide with L.R.L. at Gambote (0.70 m) and L.L.W.S. in the Bahía de Cartagena (0.12 m), and the small difference between these two levels, the reduction-level in between Gambote and the Bahía de Cartagena was adopted as a straight line (the error made in the reduction of soundings as a result of this procedure will only be in the order of a few centimeters and can well be neglected). Consequently, for the gaugestations Correa, Matunilla and Lequerica, the reduction-levels were established as given in Table 2.3.10.

The path of the L.R.L. along the Canal del Dique is presented in Figure 2.3.25. For comparison, the water-levels recorded at high and low river-stages have also been given.

## 2.4. DISCHARGES

### 2.4.1. Introduction

In Para. 2.2 it has been mentioned that a relation exists between the water stages in a river and the discharges and, generally, that it is known that higher stages correspond to higher discharges. Expressing the discharge of a river as  $Q = v.A$  and inserting the equation of Chézy ( $v=C\sqrt{R.I}$ ), the discharge-formula is derived (see Figure 2.4.1) thus:

$$Q = \int_0^B C \sqrt{R I} h_y dB \quad (2.4.1)$$

in which:

$Q$  = discharge;

$C$  = Chézy coefficient for the total bed-roughness;

$I$  = gradient of the energy-level along the river; in practice approximated by the water-level gradient;

$R$  = hydraulic radius, defined as the wetted area of the cross-section, divided by the wetted perimeter;

$h_y$  = depth of the river at a location  $y$ ; and

$B$  = width of the river.

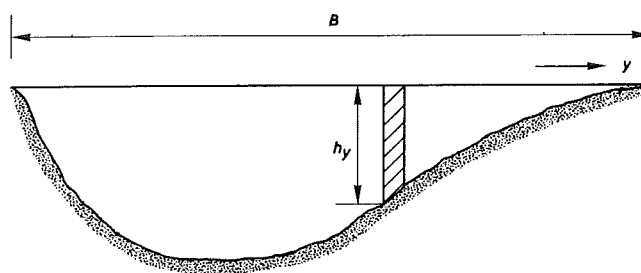


Figure 2.4.1 Discharge Cross-section

## II, 2.4

As for most rivers the width is very large compared to the depth, the hydraulic radius can often be replaced by the depth and Eq. (2.4.1) yields:

$$Q = \int_0^B C \sqrt{I} h_y^{3/2} dB \quad (2.4.2)$$

In hydrographic surveys it is common knowledge that discharges have to be measured quite extensively. However, studying Eq.(2.4.2) it appears that with the known characteristics of a certain cross-section (the relation between the depth  $h_y$  and the width) and an estimate of the bed-roughness ( $C$ ), only measuring the water-level gradient ( $I$ ) will be sufficient to determine the total discharge of the cross-section. Indeed, this procedure may well be adopted for fixed-wall channels with known roughness parameters. For different water-stages the variation in the bed-roughness factor can be expressed as:

$$C = 18 \log \frac{6h}{a+\delta/7} \quad (2.4.3)$$

in which:

$a$  = roughness length ( $a = 1/2k$ );

$k$  = the Nikuradse roughness parameter, also expressed as the diameter of homogeneous round sandgrains glued on the bed;

$\delta$  = thickness of the laminar boundary layer ( $\delta \approx 12 \nu/v_*$ );

$\nu$  = kinematic viscosity; and

$v_*$  = shear velocity ( $v_* = \sqrt{ghI}$ ).

Eq.(2.4.3) can be used universally to express the roughness of hydraulically smooth ( $a \ll \delta$ ) and hydraulically rough ( $\delta \ll a$ ) walls. As the latter condition is prevailing in rivers, Eq.(2.4.3) can be simplified to:

$$C = 18 \log \frac{12h}{k} \quad (2.4.4)$$

Remark: In Anglo-Saxon literature the roughness of channels is often expressed according to Manning in a formula which is most suitable for application in small channels. The relation between the roughness parameter of Chézy ( $C$ ) and Manning ( $n$ ) is given by:

$$C = 1.49 R^{1/6} / n$$

For different boundary materials and types of vegetation the coefficient  $n$  can be found in the literature [9].

In alluvial channels with continuously changing bed-forms it is quite impossible to estimate accurately the bed-roughness coefficient  $C$  (see Para. 3.4). A change in the water-level will also result in a change in the bed-level, but the latter may locally differ considerably from the former (e.g., due to retarded scour during the fall of the water-level). Consequently, only by measuring the total discharge of a cross-section and the local water-level gradient will it be possible to compute the bed-roughness coefficient. As this coefficient must be used again in the sediment-transport formulae, the necessity for extensive discharge measurements in alluvial channels is clearly demonstrated (further reference is made to Chapter 3).

The influence of the shape of the cross-section on the discharge is expressed in Eq. (2.4.2) by the term  $\int h_y^{3/2} dB$  (or, written in a simplified notation:  $\int Bh^{3/2}$ ); the so-called conveyance of the cross-section. For a fixed-wall cross-section the contribution of the conveyance to the discharge can be computed by means of the unique relation between the water-level and the conveyance of the cross-section. However, the profile of the river-bed is, generally, not stable and hence the contribution of the shape of the cross-section to the discharge will vary too.

From Eq. (2.4.2) it follows that a change in the water-level gradient will also result in a change of the discharge. Generally, the water-level gradient becomes steeper when the discharge increases, because a greater (permanent) discharge implies a greater water-depth. The reverse occurs with decreasing discharges. Hence the passing of a flood-wave, resulting in a water-level rise and fall, leads to different discharges in the same stage, i.e., greater discharges in the rising stage and lower discharges in the falling stage (Figure 2.4.2).

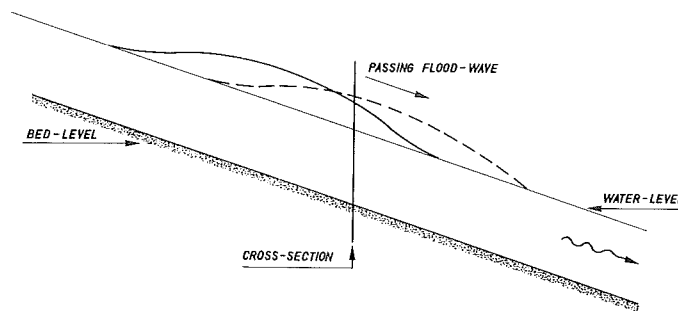


Figure 2.4.2 Flood-wave Travelling down a River (schematically)

When plotting the relationship between the water-level gradient and the stage of the river, the so-called stage-slope curve, it will be clear that such a curve will show a loop. When the water-level gradient is calculated from the difference in head divided by the distance between two gaugestations, the loop in the stage-slope curve becomes more pronounced because the travelling time of the flood-wave is also included. The loop becomes still more marked when between the two gauge stations storage areas are filled and emptied by the passing wave. In view of this, it will be clear that along the Río Magdalena and the Canal del Dique the water-level gradient has to be measured near the discharge measuring cross-section, because of the great distance between the gaugestations on the one hand and the presence of vast storage areas alongside the river on the other.

According to the shape of the stage-slope curve, rivers can be classified as (Figure 2.4.3):

- Main rivers, namely, upper course (e.g., Río Magdalena, Girardot)  
middle course (e.g., Río Magdalena, Pto. Berrío)  
lower course (e.g., Río Magdalena, Calamar);
- tributaries (e.g., Río Nare); and
- distributaries (e.g., Caño Correa).

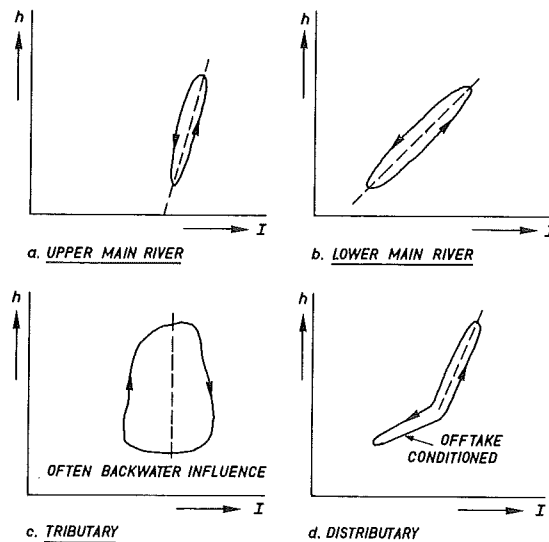


Figure 2.4.3 Schematic Stage-slope Curves for Different Types of Rivers

The loop in the stage-slope curve also implies a loop in the relationship between the stage of the river and the discharge: the so-called stage-discharge curve or rating curve. In view of the fact that the water-level gradient ( $\partial h/\partial x$ ) already decreases when the stage of the river still increases (Figure 2.4.2), the maximum discharge will occur before the maximum stage of the river is reached (Figure 2.4.4).

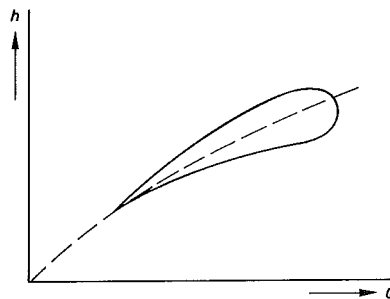


Figure 2.4.4 Schematic Stage-discharge (or rating) Curve

Assuming the roughness coefficient ( $C$ ) and the width ( $B$ ) of the river to be constant, the variation in the discharge because of a variation in the water-level gradient can be derived from Eq. (2.4.2):

$$\frac{Q_i}{Q} = \frac{I_i + (\partial h/\partial x)}{I} = \frac{I_i + (\partial h/\partial t) \cdot (1/c)}{I} \quad (2.4.5)$$

with  $c$  = the celerity of the flood-wave.

In upper main rivers with, generally, steep slopes, the variation in the discharge will mostly be small. However, in the lower regions the water-level gradient generally decreases and the term  $(\partial h/\partial t) \cdot (1/c)$  will cause a greater loop in the stage-discharge curve.

### 2.4.2. Statistical elaboration of discharges

When a number of discharges in one cross-section have been measured, it is possible to determine the relationship between the discharge of the section and the corresponding water-level: the stage-discharge curve. The most accurate curve is obtained when the measurements are evenly spread over the complete range of the occurring water-levels. However, generally the observed discharges will not be situated on a smooth curve. In Para. 2.4.1 it has already been explained that the stage-discharge curves generally show a loop because the discharge is different in rising and falling water stages. The variation in the discharge can be approximated with Eq. (2.4.5). For example, when in the Río Magdalena a water-level gradient of  $I = 40 \times 10^{-5}$ , a water-level rise of 1.5 m/day and a celerity of the flood wave of 150 km/day is assumed, Eq. (2.4.5) yields a variation in the discharge of about 1.5%. This variation can well be neglected in respect of the total error made in the measurements and the elaborations (about 10%). However, in the lower region of the Río Magdalena (Calamar) or in the Canal del Dique, the variation in the discharge as a result of variation in the water-level gradient can amount up to about 10%, because of the smaller water-level gradients.

Regarding the shape of the stage-discharge curve, the following remark must still be made. When the measurements are plotted on logarithmic paper, the stage-discharge relation for a river with a more or less uniform longitudinal profile will be a straight line for the lowest water stages. In higher water stages the relation will often be smoothly curved because of the flooding of storage areas. In such cases the measuring cross-section can best be divided into different sections. The sum of the discharge of the separate sections then yields the total discharge (see Figure 2.4.5;

$$Q_{\text{total}} = Q_I + Q_{II} + Q_{III}.$$

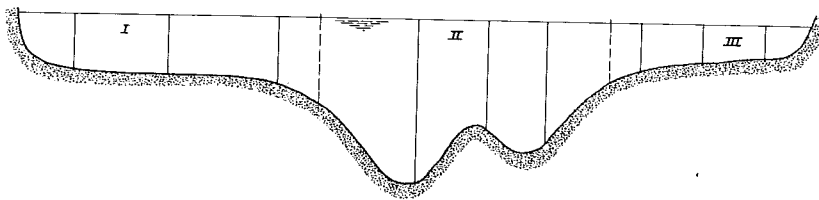


Figure 2.4.5 Discharge of Cross-section with Flood-planes

- A correction of the discharge measurements will often be necessary in view of:
- The inaccuracy of the measurements and the elaborations;
  - the local differences of the river-bed (such as ripples, vegetation, etc.) and the more gradual changes of the bed by erosion and scour (therefore, the results are not only plotted for a single year, but for a certain period of years; e.g., 5 years); and
  - at confluences and bifurcations the law of continuity must hold for the discharges of the separate branches.

Generally it appears that the discharges were not measured at the moment of occurrence of the extreme water stages, so some extrapolation of the stage-discharge curve will then be required. Although care should be taken, a more or less straight stage-discharge curve (on logarithmic paper) will not be too difficult to extrapolate.

Similar to a hydrograph of the water-levels (Para. 2.3.2), the discharges can also be plotted daily by means of the stage-discharge curve. In specific cases (e.g., storage reservoirs, or hydro-electricity) it can be advantageous to use monthly or yearly discharges: to be found by the integration of the daily discharges. Also duration-curves and frequency-curves can be plotted in the same way as described for the water-levels (Para. 2.3.3). Especially are the duration-curves of discharges often used in the mathematical computations, because one of the boundary conditions in such computations will mostly be represented by a discharge or a regime. As the shape of a duration-curve of discharges varies less from year to year than the water-levels, the former are often inserted in the computations.

2.5. RELATION-CURVES

When the stage-discharge curves for consecutive gaugestations along the same river have been drawn, relation-curves between the water-levels at these stations can be established. However, the assumption must be made that the discharge in between two stations is not modified by, e.g., tributaries or "cienagas". A schematic example is given in Figure 2.5.1.

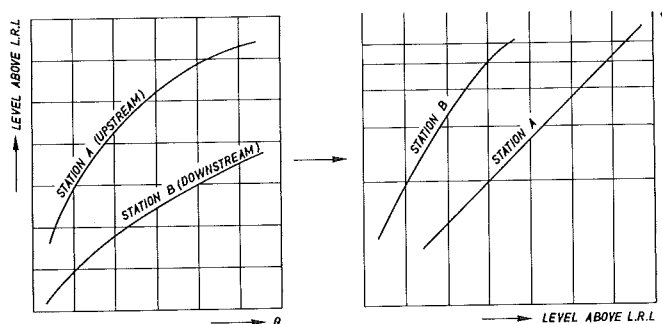


Figure 2.5.1 Theoretical Relation-curve

However, it has already been mentioned in Para. 2.4.1 that stage-discharge curves are not unique relations due to the scatter of the measured discharges around the average. To prevent this scatter, permanency of the discharge should be assumed. The assumption made (no modification of the discharge between two stations occurs) will, generally, never hold in practice. Nevertheless, water-level relations can sometimes still be established. The water-levels actually observed should then be smoothed due to the disturbances by the non-permanency of the discharge, and the travelling time of the flood-wave required to reach the downstream station (the so-called time-lag) should not be taken into account. A schematic example is given in Figure 2.5.2:

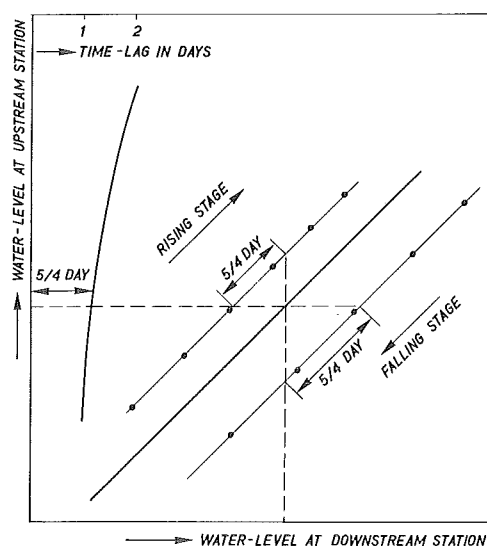


Figure 2.5.2 Graphic Determination of Relation-curve

The procedure to establish the relation-curve is as follows: Simultaneous water-levels for rising stages and falling stages are plotted. The relation-curve connects those points where the time-interval required for the propagation of the flood-wave (the time-lag) is equal along the relation for rising stages and that of the falling stages. If no lateral inflow between the two stations occurs or the discharge decreases by the flooding of storage areas, the relation-curve should coincide with the "line of equal discharge" for both the stations.

The relation-curves along a river can be used to fill up gaps in the water-level record of a certain gaugestation, to transfer a certain level (e.g., L.R.L.) from one station (where L.R.L. could be determined from the average duration-curve) to another station (where a record of water-levels has only existed for a short period), to check whether discrepancies in the readings of a station occur, or for the prediction of water-levels (see Para. 2.7). However, for the gaugestations along the Río Magdalena no relation-curves could be established because of the vast storage areas alongside the river. The influence of the lateral inflow can clearly be seen from the examples plotted in Figure 2.5.3.

In Para. 2.3.5 the remark was made that the L.R.L. at the gaugestation Pto. Inmarco could not be established from an average relation-curve as the water-level data of this station cover only a period of about one year. In view of the fact that the distance between the gaugestations of Pto. Inmarco and Pto. Berrío is only 43 kilometers and in between these two stations there are no great storage areas, it has been tried to determine the L.R.L. at Pto. Inmarco by means of a relation-curve of the water-levels at Pto. Inmarco and Pto. Berrío. The water-levels at both stations for the period of March 13, 1972, up to August 31, 1972, have been plotted. In view of the great scatter of the plots (in the order of 1.5 m), an average relation-curve covering this period has been given in Figure 2.5.4.

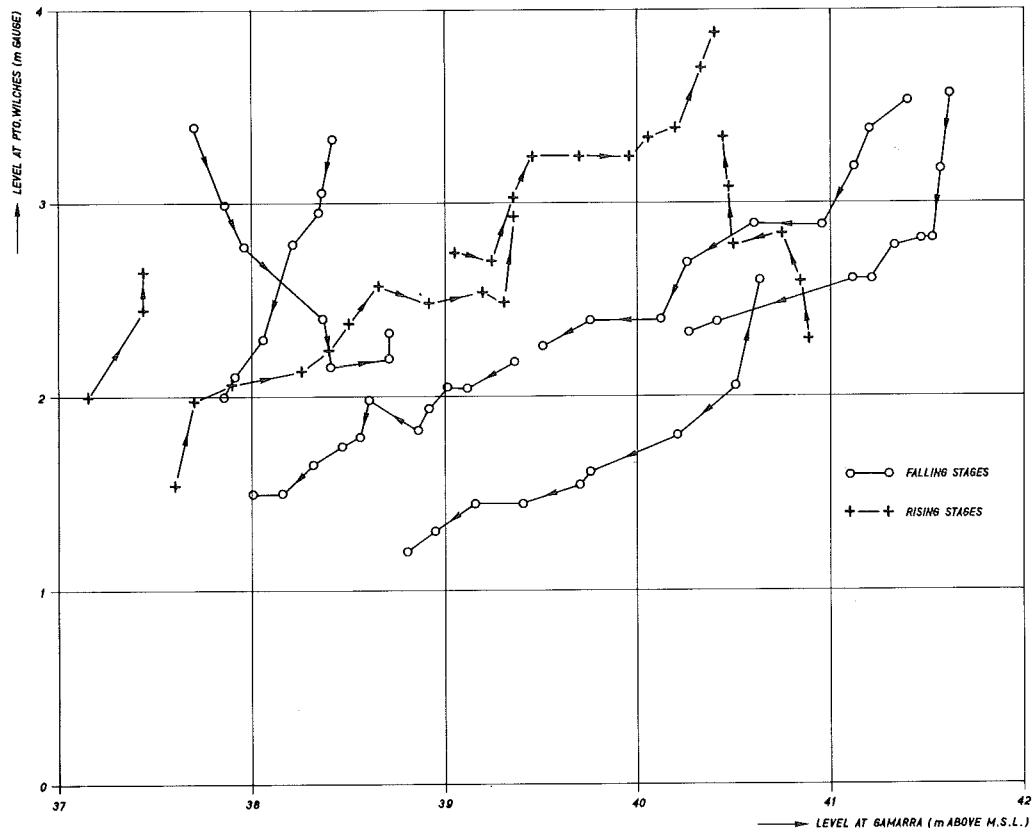


Figure 2.5.3 Example of Relation-curve along the Río Magdalena

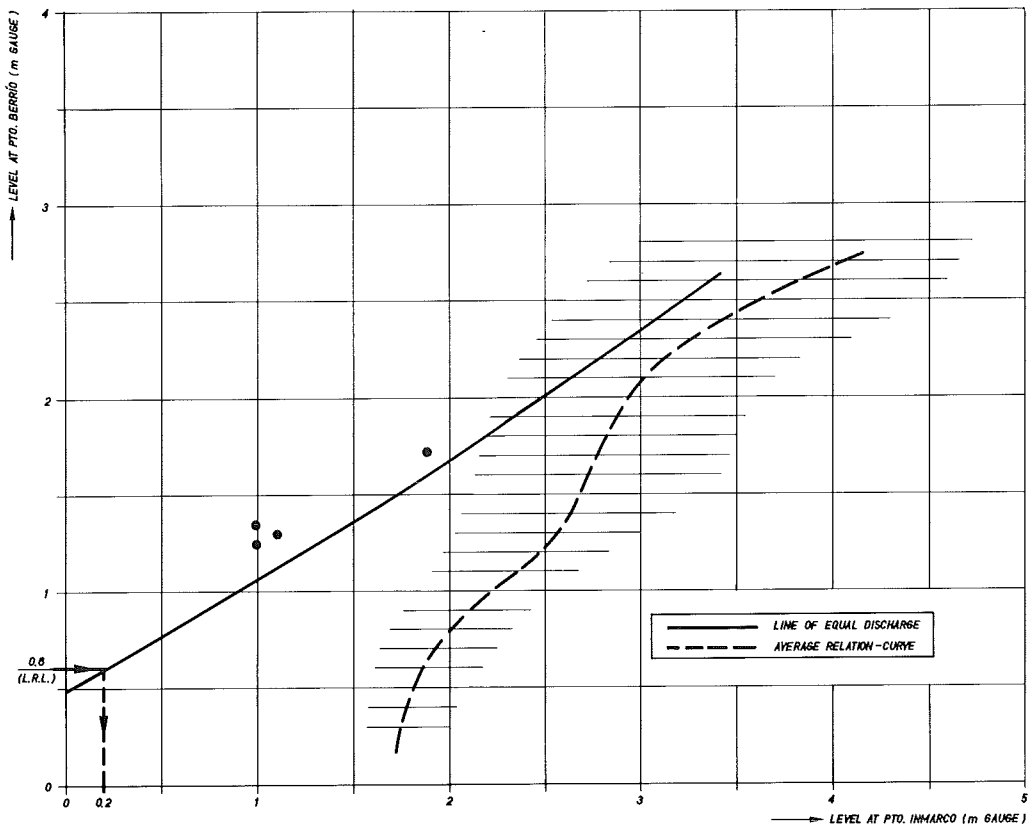


Figure 2.5.4 Relation-curve of the Gaugestations Pto. Inmarco and Pto. Berrío

In addition to the average relation-curve a few single plots of the water-levels at both stations of an earlier date are also given in Figure 2.5.4. The line of equal discharge has been presented too. It can be seen that this line deviates considerably from the relation-curve, especially for the low water stages. In view of the great scatter of the water-level data, the line of equal discharge has been used to determine the L.R.L. at Pto. Inmarco (a value of 0.20 m can be read from Figure 2.5.4).

Further reference is made to the remarks regarding the Pto. Inmarco gaugestation in Para. 3.5.4.

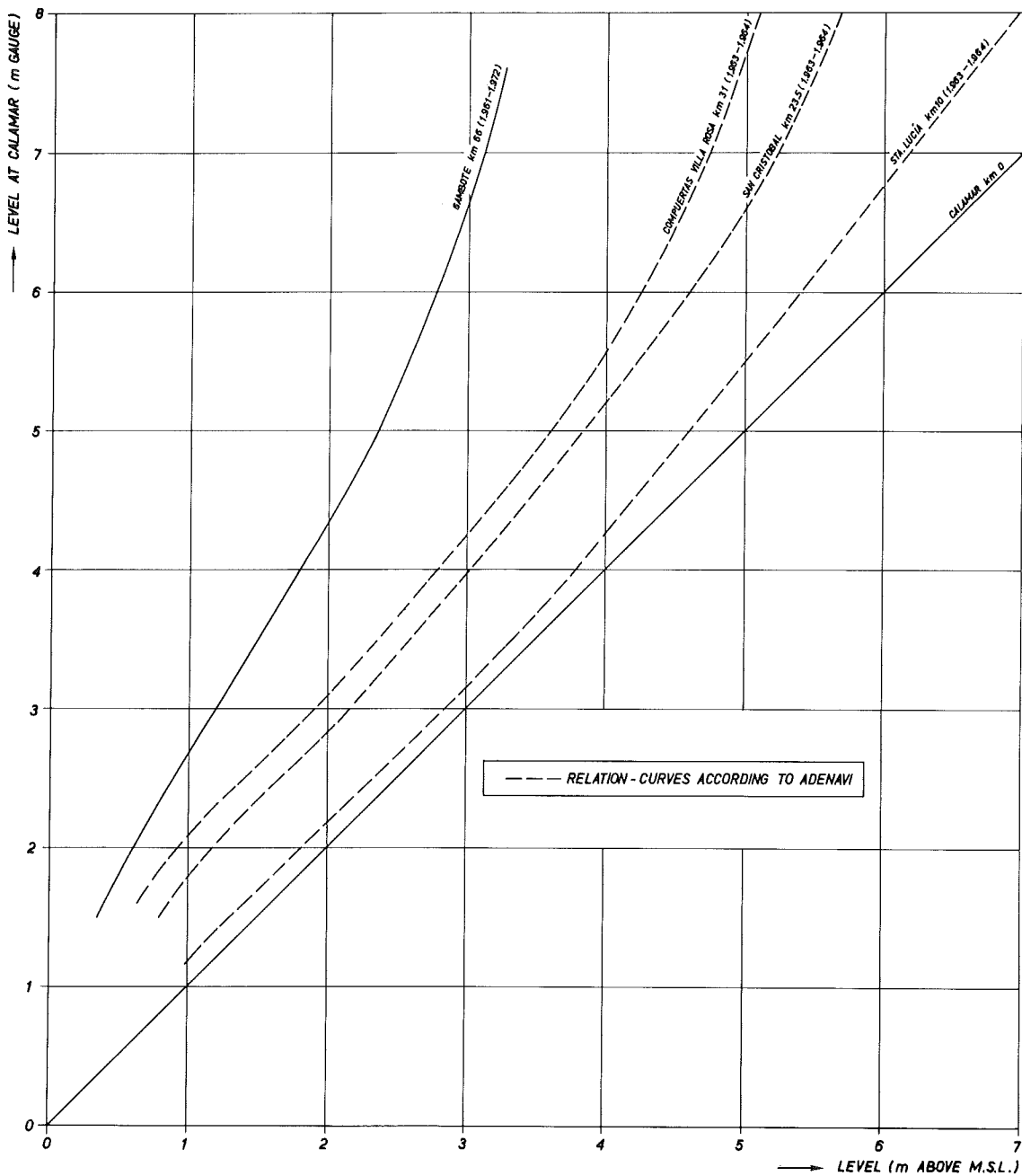


Figure 2.5.5 Relation-curves along the Canal del Dique (upper region)

II, 2.5

For the water-level data of the gaugestations along the Canal del Dique it has also been tried to draw relation-curves. For the gaugestations in the upper region of the Canal del Dique, namely, Calamar, Compuertas Villa Rosa, San Cristobal, Sta. Lucía, and Gambote, the relation-curves are presented in Figure 2.5.5 (only the last one has been determined by the Mission, the others are copied from data supplied by ADENAVI).

For the gaugestations in the lower region of the Canal del Dique, namely, Gambote, Correa, Matunilla and Lequerica, the relation-curves are presented in Figure 2.5.6.

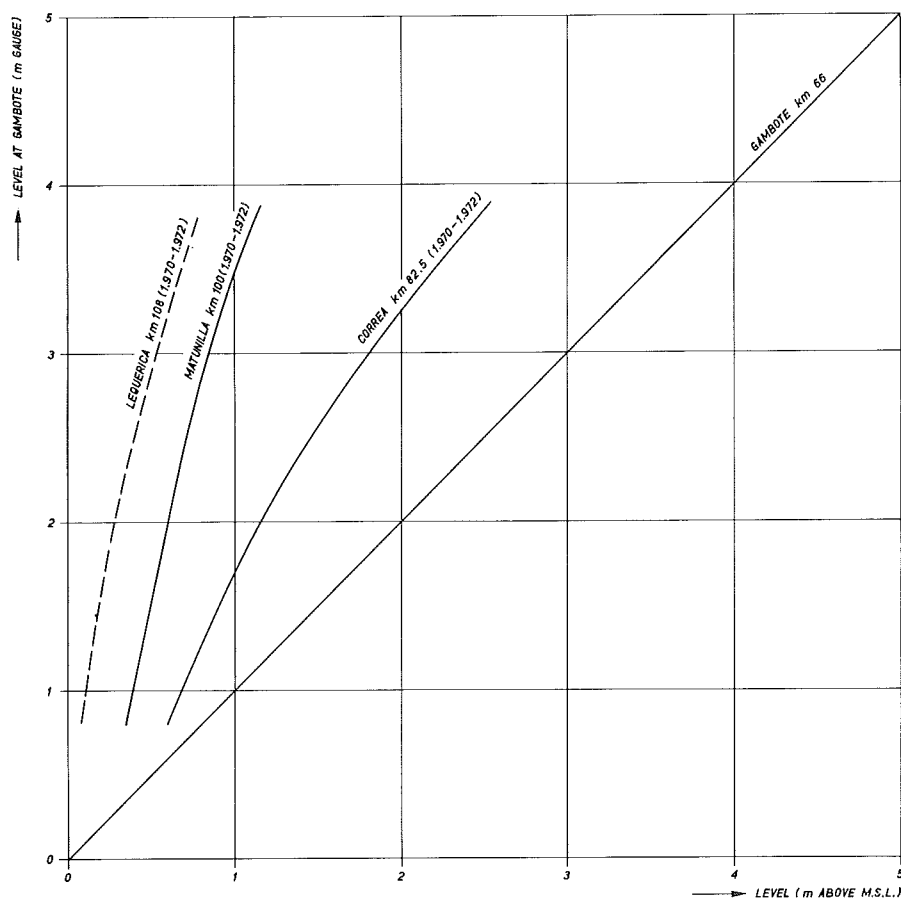


Figure 2.5.6 Relation-curves along the Canal del Dique (lower region)

For a short period water-levels were also read at a number of gauges along the Caño Matunilla. This information is partly presented in Para. 3.3.9, Figure 3.3.43.

2.6. COMPUTATION OF BACKWATER-CURVES

In hydrographic practice the computation of a backwater-curve is often required, because, firstly, the water-level will mostly not be parallel to the bed-level as a result of the variations in the discharge and the changes in the river-bed (either in the width, or in the depth), and secondly, when the execution of riverworks is considered the consequences will have to be computed beforehand.

For one-dimensional steady uniform flow the shear stress along the bed can be expressed as  $\tau = \rho g R I$ , or  $\tau = \rho g h I$  for rivers with  $b \gg h$ . The friction per unit of mass can then be written as:  $w = \tau / \rho h = g I$ . Inserting the equation of Chézy, this can also be expressed as:

$$w = g I = g v^2 / C^2 h \quad (2.6.1)$$

If only friction is considered, the equation of motion can be written as:

$$\frac{d}{dx} \left( \frac{v^2}{2g} \right) + \frac{dh}{dx} + I_{\text{bed}} = - \frac{v^2}{C^2 h} \quad (2.6.2)$$

The terms of Eq.(2.6.2) can be explained by writing this equation as:

$$\begin{aligned} - \frac{v^2}{C^2 h} - \frac{d}{dx} \left( \frac{v^2}{2g} \right) - \frac{dh}{dx} &= I_{\text{bed}} = \text{the bed-level gradient; or} \\ - \frac{v^2}{C^2 h} - \frac{d}{dx} \left( \frac{v^2}{2g} \right) &= I_{\text{bed}} + \frac{dh}{dx} = \text{the water-level gradient; and} \\ - \frac{v^2}{C^2 h} &= I_{\text{bed}} + \frac{d}{dx} \left( h + \frac{v^2}{2g} \right) = \text{the gradient of the energy-head.} \end{aligned}$$

In case of uniform flow ( $d/dx=0$ ), these equations yield the well-known expression:

$$I_{\text{bed}} = I_{\text{water-level}} = I_{\text{energy-head}}$$

The differential equation of the backwater-curve was often solved in the past by means of the Bresse-function. This type of solution may still be considered in the case of a long irrigation canal with a more or less constant shape of the cross-section along the canal. For rivers, however, this method is not very appropriate, because of the change in width and depth of the consecutive cross-sections in a river stretch. Moreover, the equation of the backwater-curve can easily be solved numerically (by hand, or by means of a computer).

Generally, in rivers the term  $(d/dx(v^2/2g))$  in Eq.(2.6.2) will be small in respect to  $(dh/dx)$  and can then be neglected. At the downstream end of the river section the water-level must be known. Because of the fact that the conditions upstream are governed by those downstream, the computation of the backwater-curve should always be carried out in an upstream direction. Eq.(2.6.2) shows that when in the first step of the computation too great a value of  $(dh/dx)$  will be found, the greater depth and smaller velocity ( $Q=\text{constant}$ ) lead to a too small value of  $(dh/dx)$  in the second step; in other words, the computation converges (possibly, the computation should be repeated another time with the average value of  $(dh/dx)$ ). If, however, the computation of the backwater-

II, 2.6

curve is started from the upstream end, Eq.(2.6.2) diverges. (Only in sub-critical flow should the computation be started from the upstream end, otherwise the computation no longer converges).

A schematic example of the computation of backwater-curves is given in Figure 2.6.1. For a river section, a constant discharge of  $Q=2,000 \text{ m}^3/\text{s}$ , width of  $B=800 \text{ m}$  and roughness coefficient  $C=40 \text{ m}^{1/2}/\text{s}$  has been assumed. Over a length of  $1,000 \text{ m}$ , the width will be reduced to  $400 \text{ m}$  (in the example the bed-level is also considered to be constant, while in reality a reduction of the width will result in an increase of the depth; however, this has been neglected).

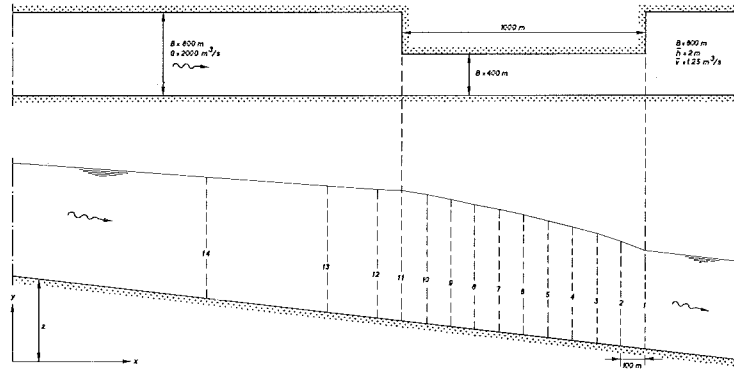


Figure 2.6.1 Schematic Example of Backwater-curves

The gradient of the energy-head can be computed for each step ( $\Delta x$ ) according to:

$$I_{(1-2)} = \frac{Q^2}{B^2 C^2 H^3 (1-2)} \quad \text{with, } H_2 = H_1 - (I_{\text{bed}} - I_{(1-2)}) \cdot \Delta x \quad (2.6.3)$$

For each step, first the energy-head ( $H$ ) is computed, after which the water-level ( $h$ ) is to be found by subtracting  $(v^2/2g)$ . (If the term  $(d/dx(v^2/2g))$  in Eq.(2.6.2) is so small that it can be neglected, Eq.(2.6.3) represents the water-level gradient). The computation has been compiled in Table 2.6.1. (In view of the small values of the term  $(v^2/2g)$ , only the water-level has been drawn in Figure 2.6.1).

Width 400 m					Width 800 m				
Number of step	$\Delta x$ (m)	$H_i$ (m)	$v_i$ (m/s)	$\frac{v_i^2}{2g}$ (m)	Number of step	$\Delta x$ (m)	$H_i$ (m)	$v_i$ (m/s)	$\frac{v_i^2}{2g}$ (m)
1		2.00	2.50	0.32	12	100	2.63	0.95	0.05
2	100	2.13	2.35	0.28	13	200	2.58	0.97	0.05
3	100	2.23	2.24	0.26	14	500	2.46	1.02	0.05
4	100	2.31	2.16	0.24	15	1,000	2.27	1.10	0.06
5	100	2.38	2.10	0.22	16	1,000	2.15	1.16	0.07
6	100	2.44	2.05	0.21	17	1,000	2.08	1.20	0.07
7	100	2.49	2.00	0.20	18	1,000	2.05	1.22	0.08
8	100	2.54	1.97	0.20	19	1,000	2.02	1.23	0.08
9	100	2.58	1.94	0.19	20	1,000	2.01	1.24	0.08
10	100	2.62	1.91	0.18	21	3,000	2.00	1.25	0.08
11	100	2.66	1.88	0.18					

Table 2.6.1 Computation of Backwater-curves

In the morphological computations (Para.3.6), the computation of the backwater-curve follows the procedure just outlined.

## 2.7. PREDICTION OF WATER-LEVELS

River operators often want to predict water-levels in the dry season a few days beforehand so as to be able to load the cargo fleet up to the maximum permissible draught. For example, when cargo has to be transported from Barrancabermeja to Pto. Salgar and the Least Available Depth (L.A.D.) on this section of the Rfo Magdalena is known, the possibility to predict the water-levels at Pto. Berrfo (1 day) and Pto. Salgar (3 days) in advance would enable the operator to load his barges up to such a draught that no unwanted delay at any crossing to await higher water stages has to be considered. However, the impossibility of establishing with a certain degree of accuracy relation-curves for the Rfo Magdalena has been outlined in Para. 2.5. It will, therefore, be clear that the prediction of water-levels is also impossible. When a denser network of gauges has been established along the Rfo Magdalena for a number of years, and the readings of gauges in the upper region of the Rfo Magdalena are also taken into account, the possibility of the prediction of water-levels should be studied in greater detail. With the presently (1973) available information, prediction of water-levels must be considered impossible.

Along the Canal del Dique the need to predict water-levels is hardly felt. The water-level at Calamar changes so gradually that a reasonable estimate can always be made, while in the lower region of the Canal del Dique the tidal influence always provides for higher water-levels during a number of hours each day. An exception, however, must be made regarding the yearly maintenance dredging which has to be carried out near the mouth of the Canal del Dique at Calamar. When in future the recurrent dredging along the Rfo Magdalena and the Canal del Dique has to be carried out by a small number of dredgers in a short period, it will be advantageous to be able to predict not only the yearly lowest level at Calamar but also the time of occurrence.

A first attempt can be made by studying the fall of the water-level at Calamar, given in the hydrograph. From these curves the depletion-curve can be determined which represents the lowest part of the hydrograph as a result of the emptying of the storage areas upstream of Calamar. Plotted on logarithmic paper, the depletion-curve is given by a straight line (Figure 2.7.1) and can be expressed in formula as:

$$h_t = h_{t_0} \exp [ - 0.033 (t-t_0) ] \quad (2.7.1)$$

with

$h_t$  = depletion-curve

$h_{t_0}$  = the water-level where the hydrograph deflects from the depletion-curve; and

$t$  = time in days

However, this depletion-curve gives only an impression of the lowest yearly level at Calamar, but this method neither answers what actually will be the lowest level which can be expected, nor does it give the time of occurrence of this level. Rainfall in the first months of the year will affect the water-level at Calamar, but its influence is not included in this method.

II, 2.7

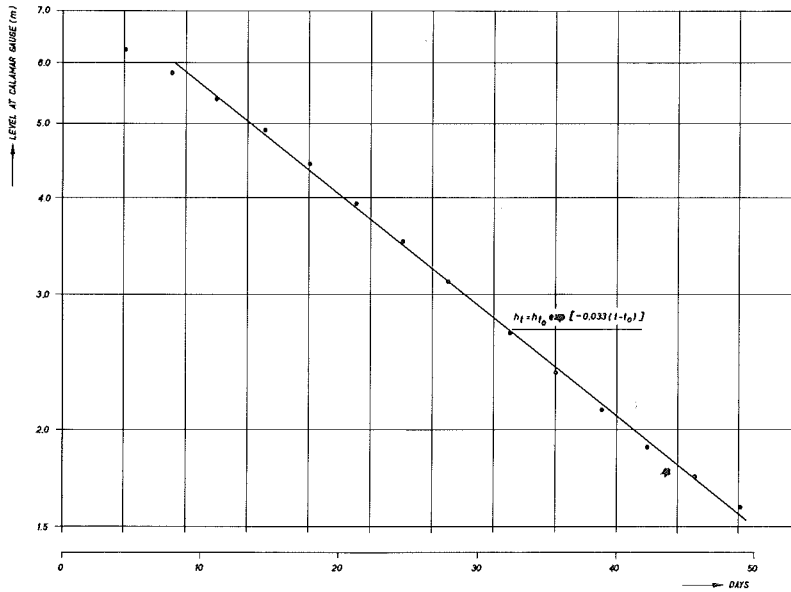


Figure 2.7.1 Depletion-curve at Calamar

A more accurate result may be obtained if the rainfall in the first months of the year is also considered. If the distribution of the average monthly rainfall in Colombia is studied (see Figure 2.7.2, copied from the "Atlas de Colombia"), it becomes clear that on the average the rain in the first months of the year mainly falls in the upper region of the Río Magdalena basin, upstream of the 5th degree of latitude. Consequently, this rainfall will also affect the hydrographs of the gaugestations at Gamarra (Río Magdalena) and Las Varas (Río Cauca). If a procedure could, therefore, be developed to predict the lowest yearly level at Calamar from the hydrographs of the stations at Gamarra and Las Varas, a more accurate result would be obtained.

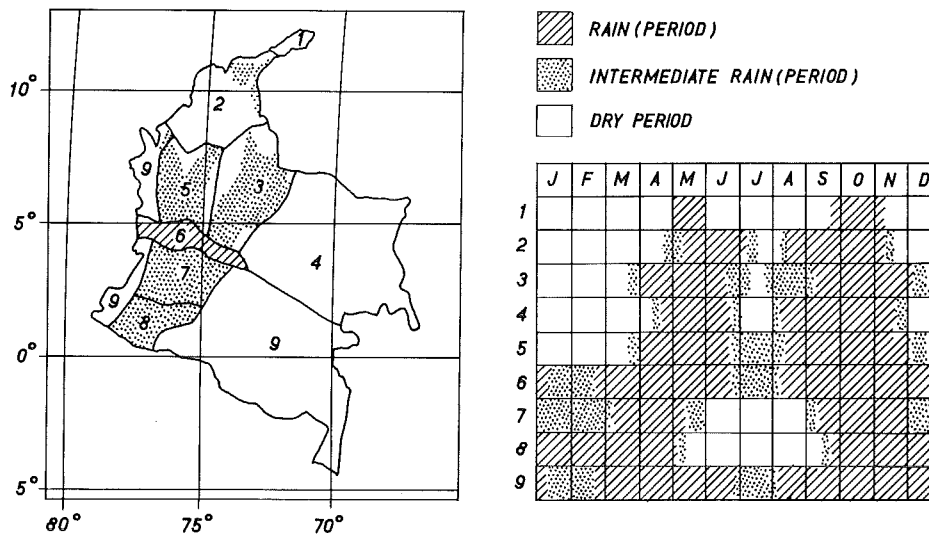


Figure 2.7.2 Average Monthly Rainfall in Colombia

The amount of effective rainfall at Gamarra and Las Varas must then be computed as the disturbance of  $R_t$ , relative to the respective depletion-curves for  $R_t$  (called  $R_t^*$ ). These relative runoffs ( $R_t^*$ ) of the Río Magdalena and the Río Cauca may give, after an as yet unknown transformation (i.e., translation and distortion), the runoff of the Río Magdalena at Calamar, again relative to the depletion-curve for  $R_t$  at Calamar. This can be expressed in formula as follows:

$$R_{C_t} = R_{C_{to}} \cdot \exp(-\alpha t) + \beta \cdot R_{LV}^*(t-\Delta t_1) + \gamma R_G^*(t-\Delta t_2) \quad (2.7.2)$$

with,

$R_{C_t}$  = discharge of the Río Magdalena at Calamar;

$R_{LV}^*$  = relative discharge of the Río Cauca at Las Varas;

$R_G^*$  = relative discharge of the Río Magdalena at Gamarra;

$\alpha, \beta, \gamma$  = coefficients to be determined from respective depletion-curves ( $\alpha$  being 0.033 as found from Figure 2.7.1); and

$\Delta t_1, \Delta t_2$  = time-lags due to the translation.

By means of Eq.(2.7.2) the discharge at Calamar can be computed and be transformed to the water-level at Calamar by means of the stage-discharge curve. However, the as yet unknown parameters ( $\beta, \gamma, \Delta t_1$  and  $\Delta t_2$ ) in Eq.(2.7.2) can only be solved when the hydrograph and the stage-discharge curves for the Río Magdalena at Gamarra and the Río Cauca at Las Varas have become available for a number of years. To be able to predict in future the lowest yearly level at Calamar and the time of occurrence, it is therefore recommended to carry out the relevant measurements in Gamarra and Las Varas.

## Chapter 3

### MORPHOLOGY

#### 3.1. GENERAL

River-works are often carried out to obtain changes in the morphological situation of a river (e.g., the cutting of a meander bend). All river-works in alluvial rivers have morphological consequences. Prediction of morphological changes, locally and along the river, are, therefore, required when river-works are designed; in fact, the design should be based on such predictions. The prediction of morphological changes has to be based on the mechanism of movement of water and sediment, or, if knowledge is insufficient, on statistical considerations about the phenomena.

The first paragraphs of this Chapter deal with these mechanisms. Para. 3.2 gives some general information on the mechanism of sediment transport; Para. 3.3 gives the collected data, while Para. 3.4 treats the resistance to the water-flow (the channel roughness). In Para. 3.5 the relation between the water-flow and the sediment transport is treated, resulting in a sediment-transport equation to be used in further computations.

To carry out computations it is always necessary to make a certain schematization. So in Para. 3.6 the river is schematized by a one-dimensional model, while the schematizing of a regime into one discharge (dominant discharge) is also dealt with. In Para. 3.7 the shortcomings in the one-dimensional model (as far as the three-dimensional aspects are concerned) are considered, while Para. 3.8 treats the possible application in general (for the Río Sogamoso Confluence). The actual applications to the Río Magdalena will be given in Part III of this Report.

#### 3.2. SEDIMENT TRANSPORT

The transport of sediments is often divided into bed-load, suspended-load and wash-load.

##### 3.2.1. Bed-load

Bed-load is the transport of sediment particles sliding, rolling or jumping over and near the river-bed, generally in the form of moving bed forms such as dunes and ripples. Many formulae have been developed to describe the mechanism of the bed-load, some being completely experimentally found, while others are based on a model of the transport mechanism. Most of these equations, however, have in common that they contain a number of "constants" which have to be modified according to the field data collected for a certain river. In fact, all the deviations in bed-load from the theoretical results are counter-acted by selecting the right "constants".

Most of the available bed-load functions can be written as a relation between the transport parameter

$$X = T/\sqrt{\Delta g D^3}$$

and the flow parameter

$$Y = \Delta D/\mu h I$$

where:  $T$  = transport in solid volume per unit width (often also use will be made of the transport including voids ( $S$ ); then  $S(1-\epsilon) = T$ , in which  $\epsilon$  is the porosity)

$h$  = depth of flow

$D$  = grain diameter

$\Delta$  = relative density =  $(\rho_s - \rho_w)/\rho_w$

$I$  = slope

$\mu$  = so-called ripple factor, in reality a factor of ignorance, used to obtain agreement between measured and computed values of  $T$ .

As an example of such an  $X$  versus  $Y$  relation the well known Meyer-Peter/Müller bed-load function may be given;

$$X = A(Y - 0.047)^{3/2} \quad (3.2.1)$$

with  $A=8$ .

This is an example of an experimentally developed equation. It is, therefore, necessary that the parameters are used in exactly the same way as during the experiments. This is especially important with regards to the ripple factor  $\mu$  and the selection of the significant grain diameter  $D$ . This consideration is in fact valid for all bed-load equations, as all of them are, at least partly, experimentally developed. However, when the results of the equation are not in accordance with the field data, modification may be carried out by the selection of another method to determine  $D$  and  $\mu$ . For example, for the Niger River it was found that the Meyer-Peter/Müller formula fitted the data when for  $A=6.5$  was introduced instead of 8.

Typical bed-load equations like the Meyer-Peter/Müller equation do not include suspended-load. This is different for the total-load equation given in Para. 3.2.2, although the construction of these equations will be seen to be similar.

Remark: When bed-load measurements are carried out, it is important to realise that this transport takes place as the propagation of bed forms; the transport intensity on the top of the dunes is large and in the troughs small or nil. Measurements should therefore cover at least the time required for several dunes to pass through the measuring section.

### 3.2.2. Suspended-load

Suspended-load is the transport of bed particles when the gravity force is counterbalanced by upward forces due to the turbulence of the flowing water. This means that the particles make larger or smaller jumps, but return eventually to the river-bed. By that time, however, other particles from the bed will be in suspension and, consequently, the concentration of particles transported as suspended-load does not change rapidly in the various layers.

## II, 3.2

A strict division between bed-load and suspended-load is not possible; in fact, the mechanisms are related. It is therefore not surprising that the so-called total-load equations have a similar construction as the bed-load equations. Bed-load and suspended-load together are often called bed-material load.

The distinction between the different modes of transport is in fact more important in relation to measuring techniques. This means that in practice bed-load is the sediment transport measured by a bed-load sampler, while suspended-load is the transport measured by a suspended-load sampler (see also Part IV). The definitions are used for this Report in this way.

In Para. 3.6.3 it will be seen that to compute scour and sedimentation an equation of continuity is selected which requires the use of a total-load equation. This is generally necessary in rivers with relatively large quantities of suspended-load.

Some total-load equations contain the temperature as parameter (Colby, Toffaleti). In order to avoid complicating matters, for the Río Magdalena only equations have been tried which do not contain the temperature as parameter; namely, the equation of Einstein and Brown (1950) [10] and the equation of Engelund and Hansen (1967) [11] (see Para. 3.5.2).

The equation of Engelund and Hansen may be written as:

$$X = 0.05 Y^{-5/2} \quad (3.2.2)$$

with  $\mu = \left(\frac{C^2}{g}\right)^{2/5}$ ,  $C$  = roughness coefficient of Chézy and  $D = D_{50}$ .

The equation of Einstein and Brown may be written as:

$$\frac{X}{F} = 40Y^{-3} \quad (3.2.3)$$

with  $F = \sqrt{\frac{2}{3} + \frac{36v^2}{\Delta g D^3}} - \sqrt{\frac{36v^2}{\Delta g D^3}}$  and  $D = D_{50}$

Equations 3.2.2 and 3.2.3 do not give information on the distribution of the concentration of particles in the vertical. The value of the concentration ( $C$ ) is often determined theoretically, based on the reasoning that the vertical downward transport of particles due to gravity for continuity reasons should be equal to the upward transport due to turbulence, or;

$$C \cdot w + \epsilon_s \frac{dC}{dy} = 0 \quad (3.2.4)$$

with  $C$  = concentration,  $w$  = fall velocity and  $\epsilon_s$  = diffusion factor for suspended-load material. This equation may be solved if  $\epsilon_s$  is assumed to be equal to  $\epsilon$ , the diffusion factor for momentum transferred by means of turbulence. In that case the following equation was derived by Rouse (1937) [12] by taking  $\epsilon = x \cdot v_y (1 - y/h)$ :

$$C_y = C_a \left( \frac{a}{h-a} \cdot \frac{h-y}{y} \right)^z \quad (3.2.5)$$

with  $z = \frac{w}{K \cdot v_*}$

This equation gives the concentration ( $C_y$ ) in relation to a reference concentration ( $C_a$ ) at height  $a$  above the bed. (see Para. 3.3.4).

Several attempts have been made to determine the reference concentration theoretically and thus arrive at a transport equation by taking

$$s_s = \int_0^h v \cdot C_y dy \quad (3.2.6)$$

with  $s_s$  = suspended-load.

The main problems are:

- Equation 3.2.6 is not valid for high concentrations near the river-bed;
- the fall velocity is influenced by the concentration; and
- the assumption  $\epsilon = \epsilon_s$  is not valid close to the river-bed.

Suspended-load measurements carried out by the Mission contain measuring points at various heights in each vertical in order to compare z-values from Equation 3.2.5 with those found from the measurements (see also Para. 3.3.4).

### 3.2.3. Wash-load

Wash-load is the transport of small particles finer than the bulk of the bed material and rarely found in the bed. Transport quantities found from bed-load, suspended-load and total-load formulae do not include wash-load quantities.

Whereas for a certain cross-section quantities of suspended-load and bed-load can be calculated with the use of the locally valid hydraulic conditions, this is not the case for wash-load. The rate of wash-load is mainly determined by climatological characteristics and the erosion features of the whole catchment area.

As there is normally no interchange with bed particles, wash-load is not important for local scour. Due to the very low fall velocity of the wash-load particles, wash-load only contributes to sedimentation in areas with low current velocities (harbours, reservoirs, dead river branches, "cienagas", sea). Due to the small fall velocity, in turbulent water the concentration of the particles over a vertical (generally expressed in parts per million, p.p.m.) is rather uniform, so that even with one water sample a fairly good impression can be obtained. The wash-load concentration over the width, however, may vary considerably (see Para. 3.3.5).

## 3.3. COLLECTED AND ELABORATED DATA

### 3.3.1. Introduction

In this paragraph all the measurements in the Río Magdalena and the Canal del Dique, which were carried out in the course of the project, are given. As far as possible the data collected by other agencies have been included, of which especially the discharge measurements carried out by the Julius Berger Konsortium, Apron y Duque Ltda. and the SCMH are mentioned. Para. 3.3.2 deals with the discharge and sediment transport measurements and although the data of these measurements is further used in Chapters 3.4 and 3.5, some explanations regarding the data and the measuring cross-sections are given in this Chapter. During the elaboration of the transport measurements it appeared that due to the method of measuring used, the measured data give higher values than the true transport. Therefore, the

## II, 3.3

suspended-load measurements are considered separately in Para. 3.3.3. The suspended-load concentrations are dealt with in Para. 3.3.4, and the wash-load concentrations in Para. 3.3.5. The data of the grain-sizes of the bed material are presented in Para. 3.3.6; the longitudinal profiles of the Río Magdalena and the Canal del Dique are considered in Para. 3.3.7; and the local soundings of shallow areas are given in a table in Para. 3.3.8, and, these maps are also partly presented in Part III of this Report, dealing with river improvements. In the Canal del Dique some other data were still gathered which have been presented as miscellaneous information in Para. 3.3.9.

### 3.3.2. The discharge and sediment transport measurements

All the data of the discharge and sediment transport measurements carried out in the course of the project are presented in Tables 3.3.1 and 3.3.2. Apart from the various data measured in the field, also some elaborated data are inserted which are being used in Chapters 3.4 and 3.5, for the determination of the channel roughness and the selection of the transport equation.

Anticipating the following chapters, however, in relation to some of these data a few remarks will be made here.

- The water-levels mentioned are the levels as read directly on the nearby main gauge-station. For measurements made in the affluents the levels as read on the local gauge or with reference to a bench-mark are inserted as well. In the lower (tidal) region of the Canal del Dique the levels of Gambote are given as a reference (the zero-levels of the gauges have already been mentioned in Table 2.3.1).
- The water-level gradient has been measured near the cross-sections, mostly over a distance of about 500 m, but it must be stressed that these values often differ considerably from the values determined as the average gradient between two main gauge-stations.

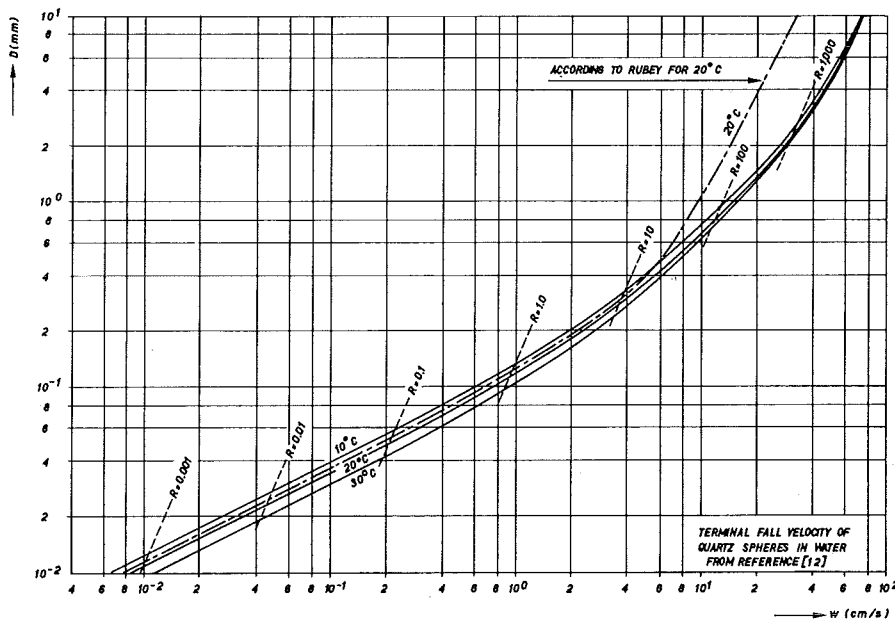


Figure 3.3.1 Fall Velocity of Quartz Spheres in Water.

- The grain-sizes mentioned have been determined as mean values of all the bed-samples which were taken in or very close to the cross-sections (see also Para. 3.3.6). The standard deviation (S) of the  $\bar{D}_{50}$  has also been given. The fall velocities correspond to the values of  $\bar{D}_{50}$  for a water temperature of 30°C (see Figure 3.3.1).
- The roughness from the grains of the bed material alone ( $C'$ ) has been determined according to the formula:

$$C' = 18 \log 12h/k_s \quad (3.4.5)$$

and is given in Figure 3.3.2. The graph has been used by inserting  $k_s = 2 \bar{D}_{65}$  (see Para. 3.4.2).

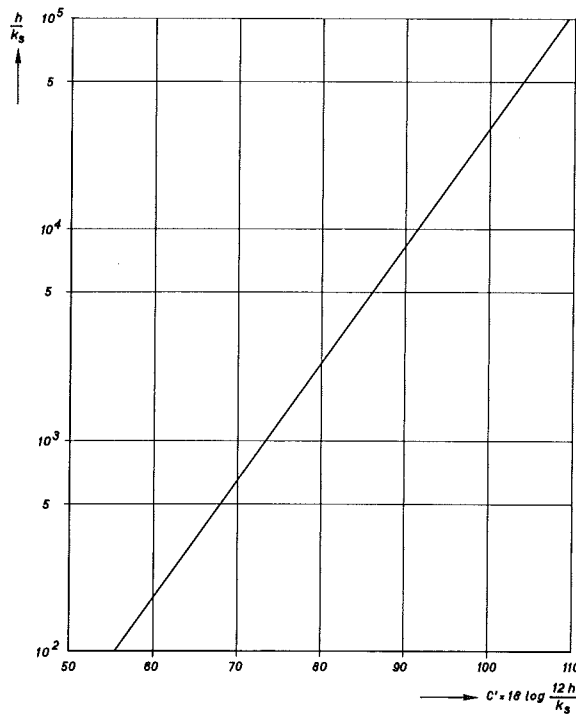


Figure 3.3.2 Chézy Coefficient Related to the Grains

The roughness of the bed-form ( $C''$ ) has been calculated from:

$$1/C^2 = 1/(C')^2 + 1/(C'')^2 \quad (3.4.10)$$

while the Darcy/Weisbach friction factor ( $f$ ) follows from:

$$f = 8g/C^2 \quad (3.4.4)$$

- The kinematic viscosity ( $\nu$ ) is dependent on the temperature. Although no water temperatures were measured in the various cross-sections during the discharge measurements, temperatures were measured daily in Barrancabermeja during a few months (see Figure 3.3.11). An average water temperature of 30°C has been used to determine the value of  $\nu = 0.8 \times 10^{-6} \text{ m}^2/\text{s}$ .
- The sediment transport given in the tables is the measured transport. The method of measuring the suspended-load and the resulting error in the measurements is treated in Para. 3.3.3, while the comparison between the measured transport and the computed transport according to the Einstein/Brown and Englund/Hansen equations is given in Para. 3.5.

## II, 3.3

### A. The Río Magdalena (Tables 3.3.1 and Figure 3.3.3)

The location of the cross-sections used for the discharge and sediment transport measurements carried out by the Mission in the Río Magdalena between La Dorada and Gamarra, at Calamar, and in the affluents are given in Figure 3.3.3. All the data of these measurements are presented in Tables 3.3.1A and B. About some of these cross-sections a few remarks must be made.

#### The Río Nare Confluence (Figure 3.3.3c)

The cross-section at Pto. Inmarco just downstream of the Río Nare Confluence is situated in front of the cement factory. Because of the wide and shallow river stretch upstream and the strong restriction of the width downstream, this cross-section could not be located perpendicular to the flow-lines. However, during the measurements the angles between the flow-lines and the cross-section were also measured and only the component of the flow perpendicular to the cross-section has been used to compute the total discharge. Assuming that both the bed-load and the suspended-load are transported parallel to the flow-lines, the same principle has been used to calculate the total sediment transport.

During the measurements in July and August 1971, the water-level gradient in the Río Nare cross-section was not measured. Proportional to the height of the water-level during these measurements, the values have been estimated in accordance with the water-level gradient measured at a later date.

#### The Río Regla Confluence (Figure 3.3.3d)

The cross-section at Ballena, located upstream of the Río Regla, at present (1973) consists of 3 branches. The main branch along the right bank is separated by islands from the two secondary branches along the left bank. Of these minor secondary branches, only the discharge and no sediment transport was measured. The results of these discharge measurements are listed in Table 3.3.1 separately. The increase in the width of the main branch which was found at the time of the last measurements (March 1972) was the result of the erosion occurring along the left bank.

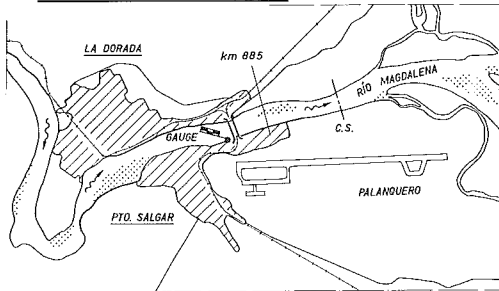
Just downstream of the Ballena section the Río Regla enters the Río Magdalena along the left bank, and downstream of a very wide and partly shallow stretch of about 4 km, the Río Viejo branches off along the right bank. The Río Viejo is a narrow and shallow channel and probably the remainder of a former course of the Río Magdalena. The Río Viejo joins the main course of the Río Magdalena again at km 696. At present (1973), the main course of the Río Magdalena downstream of the Río Viejo Bifurcation is called the Río Nuevo and consists of a narrow and deep channel.

The data of the 4 cross-sections of this Confluence, viz., the Ballena, the Río Regla, the Río Viejo and the Río Nuevo sections, presented in Table 3.3.1 have not been adjusted. This means that the inflow is not necessarily equal to the outflow. This can partly be caused by a difference in water-level on the consecutive days of the measurements, and partly by an angle between the flow-lines and the measuring cross-section in the wide and shallow branch at Ballena.

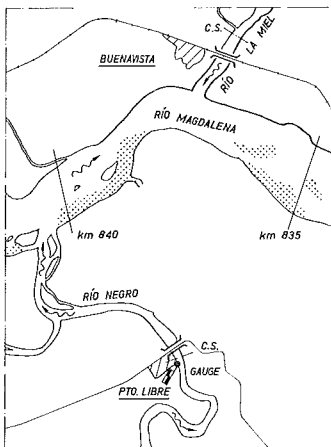
APPROXIMATE SCALE OF SITUATIONS 1:120,000  
(EXCEPT CALAMAR)

f. RÍO OPÓN CONFLUENCE AND BARRANCABERMEJA

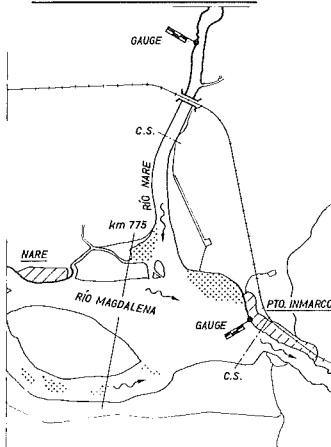
a. LA DORADA - PTO. SALGAR



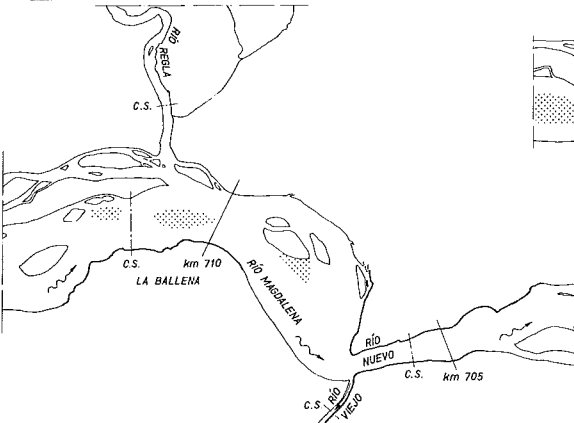
b. RÍO NEGRO AND RÍO LA MIEL CONFLUENCES



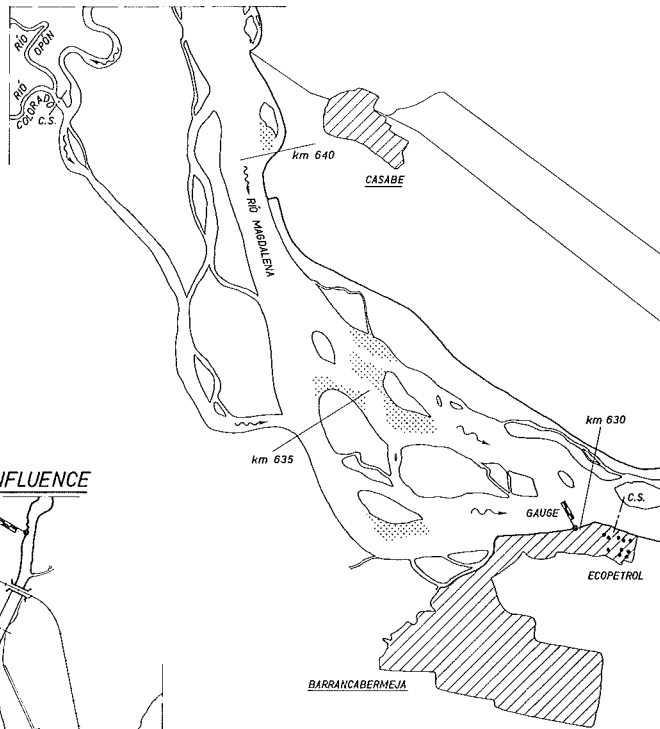
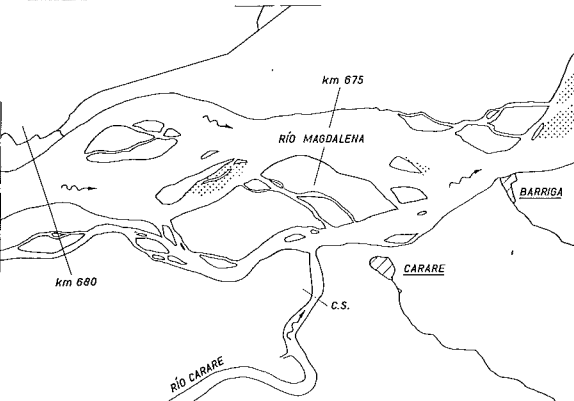
c. RÍO NARE CONFLUENCE



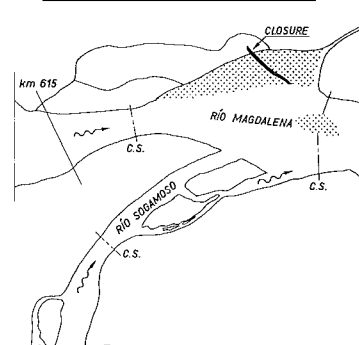
d. RÍO REGLA CONFLUENCE



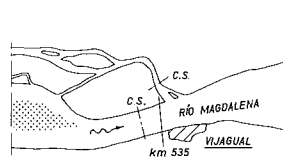
e. RÍO CARARE CONFLUENCE



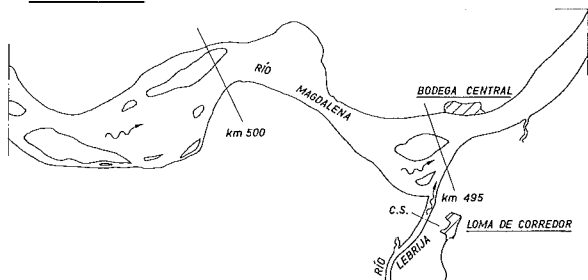
g. RÍO SOGAMOSO CONFLUENCE



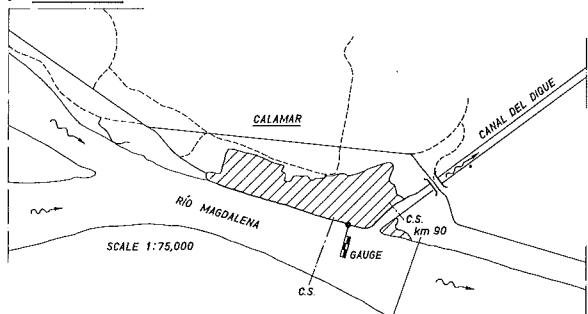
h. VJAGUAL



i. RÍO LEBRIJA



j. CALAMAR



Date	Level	B	A	$\bar{h}$	$Bh^{3/2}$	$(h^{5/2})^{2/5}$	Q	$\bar{v}$	$I \times 10^{-5}$	C	C'	C''	f	f'	f''	$\frac{(h^{5/2})^{2/5} I}{h \bar{v}_{50}}$	$\frac{\bar{v}^2}{(C')^2 \bar{v}_{50}^2}$	$\frac{(C')^2 \bar{v}_{50}^2}{\bar{v}^2}$	$\frac{\bar{v}}{g \bar{v}_{50}}$	$\frac{\bar{v} \cdot \bar{h}}{v^2}$	$\frac{W}{V_R}$	$\bar{v}^5$	Remarks	
	(m)	(m)	(m <sup>2</sup> )	(m)	(m <sup>5/2</sup> )	(m)	(m <sup>3</sup> /s)	(m/s)		(n <sup>1/2</sup> /s)	(n <sup>1/2</sup> /s)	(n <sup>1/2</sup> /s)	(x10 <sup>-2</sup> )	(x10 <sup>-2</sup> )	(x10 <sup>-2</sup> )		(C') <sup>2</sup> $\bar{v}_{50}^2$			(x10 <sup>5</sup> )	(m <sup>5</sup> /day)			
Location: La Dorada, Rfo Magdalena km 884. $\bar{D}_{35}=340\mu m$ ; $\bar{D}_{50}=500\mu m$ (S=145 $\mu m$ ); $2\bar{D}_{65}=2,780\mu m$ ; $\Delta=1.68$ ; $v=8 \times 10^{-2}$ (m/s); $w\bar{D}_{50}/v=50$																								
9-VIII-71	3.80	295	986	3.34		2,316	2.35										0.42	1.64	19.56	4.76	1.06	34,900		
8-VIII-71	2.75	300	835	2.78	1,380	2.87	1,140	1.37	21	57	73	91	2.42	1.46	0.96	0.72	0.42	1.64	19.56	4.76	1.06	34,900		
9-IX-71	2.80	295	890	3.01	1,550	3.10	1,310	1.47	23	56	74	85	2.53	1.43	1.09	0.85	0.47	1.45	20.99	5.53	0.97	50,000		
19-X-71	3.60	295	1,088	3.69	2,170	3.92	1,785	1.65	24	53	76	75	2.78	1.37	1.41	1.12	0.57	1.20	23.56	7.61	0.86	56,700		
9-I-72	2.40	290	810	2.79	1,360	2.90	793	0.98	22	39	73	47	5.08	1.46	3.62	0.76	0.21	3.20	13.99	3.42	1.02	20,970		
15-II-72	2.58	285	849	2.98	1,498	3.10	979	1.15	17	50	74	68	3.12	1.44	1.69	0.63	0.29	2.36	16.42	4.28	1.11	24,400		
8-III-72	2.25	284	592	2.08	874	2.17	604	1.02	23	46	71	59	3.78	1.55	2.23	0.59	0.24	2.78	14.56	2.65	1.14	16,130		
Location: Rfo Negro. Levels given at local gaugestation and Pto. Salgar gauge respectively																								
9-VIII-71	0.80	2.90	91	93	1.02		69	0.74																
11-IX-71	1.47	2.75	97	175	1.80		175	1.00																
21-X-71	2.49	2.75	134	283	2.11		347	1.23																
9-III-72	0.99	2.55	94	90	0.96		86	0.95																
Location: Rfo La Miel. Levels given at local gaugestation and Pto. Salgar gauge respectively																								
9-VIII-71	0.82	2.90	142	174	1.22		130	0.75																
10-IX-71	1.66	2.60	158	336	2.13		303	0.90																
20-X-71	2.35	3.35	158	430	2.72		370	0.86																
9-III-72	0.88	2.55	144	212	1.47		187	0.88																
Location: Rfo Nare. Levels given at local gaugestation and Pto. Inmarco gauge respectively																								
$\bar{D}_{35}=670\mu m$ ; $\bar{D}_{50}=1,080\mu m$ (S=604 $\mu m$ ); $2\bar{D}_{65}=5,090\mu m$ ; $\Delta=1.68$ ; $v=17 \times 10^{-2}$ (m/s); $w\bar{D}_{50}/v=230$																								
11-VII-71	1.18	198	310	1.57	393	1.59	261	0.84	36	35	64	42	6.41	1.90	4.50	0.32	0.09	6.57	8.16	1.65	2.28	690		
13-VIII-71	1.74	1.64	195	425	2.18	627	2.18	475	1.12	46	35	67	42	6.29	1.76	4.53	0.55	0.16	4.00	10.88	3.05	1.71	4,580	
13-IX-71	2.71	2.41	194	625	3.22	1,118	3.25	886	1.42	57	33	70	38	7.12	1.61	5.51	1.02	0.23	2.72	13.80	5.72	1.27	20,150	
23-X-71	2.10	2.57	190	467	2.46	747	2.52	504	1.08	41	33	68	38	7.07	1.71	5.36	0.57	0.14	4.42	10.49	3.32	1.71	5,350	
10-III-72	1.03	1.55	197	259	1.32	300	1.33	232	0.90	35	41	63	55	4.59	1.99	2.61	0.26	0.11	5.49	8.74	1.49	2.52	1,240	
Location: Puerto Inmarco, Rfo Magdalena km 773. $\bar{D}_{35}=575\mu m$ ; $\bar{D}_{50}=1,050\mu m$ (S=1,340 $\mu m$ ); $2\bar{D}_{65}=3,530\mu m$ ; $\Delta=1.68$ ; $v=16 \times 10^{-2}$ (m/s); $w\bar{D}_{50}/v=210$																								
12-VIII-71	2.19	610	2,400	3.94	4,900	4.15	2,710	1.13	48	25	74	27	12.32	1.42	10.89	1.13	0.13	4.17	11.13	5.57	1.14	56,690		
12-IX-71	1.89	620	2,120	3.42	3,970	3.60	2,270	1.07	54	25	73	26	12.96	1.47	11.49	1.10	0.12	4.51	10.54	4.57	1.19	46,190		
22-X-71	3.09	620	3,220	5.19	7,480	5.54	3,720	1.16	41	25	76	26	13.01	1.35	11.66	1.29	0.13	4.19	11.43	7.53	1.08	62,520		
11-I-72	0.99	605	1,380	2.28	2,475	3.08	1,900	1.38	36	40	70	50	4.79	1.60	3.19	0.63	0.23	2.48	13.60	3.93	1.78	63,480		
11-III-72	1.10	622	1,262	2.03	2,335	3.20	1,400	1.11	49	27	69	29	10.70	1.65	9.05	0.89	0.15	3.74	10.94	2.82	1.62	38,900		
Location: Ballena, Rfo Magdalena km 711.5. Levels given below local BM at Ballena and at Pto. Berrfo gauge respectively																								
$\bar{D}_{35}=330\mu m$ ; $\bar{D}_{50}=405\mu m$ (S=37 $\mu m$ ); $2\bar{D}_{65}=1,160\mu m$ ; $\Delta=1.68$ ; $v=6.8 \times 10^{-2}$ (m/s); $w\bar{D}_{50}/v=34$																								
28-VIII-71	5.30	2.40	785	2,200	2.80	4,065	3.39	2,630	1.20	36	34	80	38	6.75	1.22	5.53	1.79	0.33	2.48	19.04	4.20	0.68	84,940	$Q_s=385$ (m <sup>3</sup> /s)
21-IX-71	6.03	1.65	785	1,570	2.00	2,615	2.60	1,940	1.24	48	34	78	38	6.84	1.30	5.54	1.83	0.39	2.17	19.67	3.10	0.70	33,120	$Q_s=250$ (m <sup>3</sup> /s)
25-X-71	5.26	2.20	798	2,105	2.64	4,140	3.72	3,090	1.46	62	30	80	32	6.73	1.23	7.50	3.39	0.49	1.66	23.16	4.82	0.54	85,800	$Q_s=410$ (m <sup>3</sup> /s)
12-III-72	6.74	0.80	845	1,290	1.53	1,930	2.16	1,260	0.98	40	33	76	36	7.37	1.38	5.99	1.27	0.25	3.29	15.55	1.87	0.88	19,830	$Q_s=150$ (m <sup>3</sup> /s)
Location: Rfo Regla. Levels given at Pto. Berrfo gauge																								
29-VIII-71	2.00	140	100	0.71			75	0.75																
20-IX-71	3.25	140	140	1.00			104	0.74																
24-X-71	2.40	149	170	1.14			80	0.47																
Location: Rfo Viejo. Levels given at Pto. Berrfo gauge																								
29-VIII-71	2.00	48	60	1.25			35	0.58																
22-IX-71	1.65	43	58	1.35			25	0.43																
24-X-71	2.40	48	84	1.75			65	0.77																
Location: Rfo Nuevo, Rfo Magdalena km 705.5. Levels given below local BM in Rfo Nuevo and at Pto. Berrfo gauge respectively																								
$\bar{D}_{35}=700\mu m$ ; $\bar{D}_{50}=920\mu m$ (S=673 $\mu m$ ); $2\bar{D}_{65}=2,590\mu m$ ; $\Delta=1.68$ ; $v=15 \times 10^{-2}$ (m/s); $w\bar{D}_{50}/v=172$																								
29-VIII-71	4.67	2.00	400	1,850	4.62	4,175	4.99	2,720	1.47	13	57	78	84	2.40	1.39	1.11	0.42	0.23	3.30	15.47	8.49	1.95	103,100	
22-IX-71	4.99	1.65	415	1,720	4.15	3,810	4.70	2,480	1.44	17	50	77	66	3.15	1.32	1.83	0.52	0.23	3.37	15.16	7.47	1.80	51,400	
24-X-71	4.43	2.40	415	1,945	4.69	4,575	5.26	3,085	1.58	22	45	78	56	3.80	1.29	2.51	0.75	0.27	2.87	16.63	9.26	1.49	84,100	
12-I-72	5.84	1.10	400	1,350	3.38	2,535	3.56	1,615	1.20	17	49	75	64	3.29	1.38	1.91	0.39	0.16	4.65	12.63	5.07	2.00	33,030	
18-II-72	5.70	1.15	400	1,220	3.05	2,330	3.41	1,665	1.36	17	55	75	81	2.61	1.41	1.20	0.38	0.21	3.54	14.31	5.18	2.10	38,900	
13-III-72	6.07	0.80	395	988	2.50	1,810	3.09	1,250	1.26	38	35	73	41	6.25	1.47	4.78	0.76	0.19	3.96	13.26	3.94	1.55	20,450	
Location: Rfo Carare. Levels given below local BM in the Rfo Carare and at Pto. Berrfo gauge respectively																								
30-VIII-71	2.03	2.00	160	457	2.86		235	0.51																
15-IX-71	1.84	1.70	160	480	3.00		269	0.56																
29-XI-71	2.05	2.50	160	487	3.04		321	0.66																
30-XII-71	1.45	2.25	160	573	3.58		346	0.60																
13-III-72	3.08	0.80	150	278	1.85		165	0.59																

Remark: Values underlined indicate estimated values;  $Q_s$  indicates discharge in secondary parallel branch(es)

Table 3.3.1 A Discharge and Sediment Transport Data of the Rfo Magdalena

Date	Level	B	A	$\bar{h}$	$B\bar{h}^{3/2}$	$(\bar{h}^{5/2})^{2/5}$	Q	$\bar{v}$	$I \times 10^{-5}$	C	C'	C''	f	f'	f''	$\frac{(\bar{h}^{5/2})^{2/5} \cdot I}{\Delta \bar{D}_{50}}$	$\frac{\bar{v}^2}{(C')^2 \Delta \bar{D}_{50}}$	$\frac{(C')^2 \Delta \bar{D}_{50}}{\bar{v}^2}$	$\frac{\bar{v}}{g \bar{D}_{50}}$	$\frac{\bar{v} \cdot \bar{h}}{v \cdot (x 10^6)}$	$\frac{W}{V_A}$	S	Remarks		
	(m)	(m)	(m <sup>2</sup> )	(m)	(m <sup>3/2</sup> )	(m)	(m <sup>3</sup> /s)	(m/s)		(m <sup>1/2</sup> /s)	(m <sup>1/2</sup> /s)	(m <sup>1/2</sup> /s)	(x10 <sup>-2</sup> )	(x10 <sup>-2</sup> )	(x10 <sup>-2</sup> )					(x10 <sup>6</sup> )		(m <sup>3</sup> /day)			
Location: Rfo Opón. Levels given below local BH in the Rfo Opón and at Barrancabermeja gauge respectively																									
30-VIII-'71	1.30	3.35	95	366	3.85		198	0.54																	
15-IX-'71	2.27	2.40	91	275	3.02		174	0.63																	
30-XI-'71	2.62	2.80	98	285	2.91		108	0.38																	
14-III-'72	3.97	1.50	91	167	1.84		54	0.32																	
Location: Barrancabermeja, Rfo Magdalena km 629. Levels given at Barrancabermeja and Pto. Wilches gauge respectively																									
12-XI-'71	3.70	3.80	920	3,725	4.05	8,380	5,080	1.36	27	37															
			242	1,004	4.15	2,150	1,396	1.39	27	39														secondary branch	
18-XI-'71	3.30	3.40	613	2,750	4.49	6,720	3,940	1.43	27	35														Q <sub>s</sub> =150(m <sup>3</sup> /s)	
-3-III-'72	1.25	1.30	452	1,312	2.90	2,450	1,520	1.15	27	38															
23-V-'72	4.40	4.05	450	3,538	7.86	10,280	6,450	1.82	28	37															
Location: Upstream Rfo Sogamoso, Rfo Magdalena km 614. Levels given at Barrancabermeja and Pto. Wilches gauge respectively																									
							$\bar{D}_{35}=325\mu\text{m}; \bar{D}_{50}=375\mu\text{m} (S=110\mu\text{m}); 2\bar{D}_{65}=902\mu\text{m}; \Delta=1.68; w=6.1 \times 10^{-2}(\text{m/s}); w_{\bar{D}_{50}}/v=29$																		
27-VII-'71	2.30	2.35	446	2,080	4.66	5,250	3,080	1.48	20	41	86	47	4.56	1.06	3.50	1.90	0.47	1.85	24.40	8.62	0.64		82,565		
1-IX-'71	2.30	2.50	450	2,000	4.44	5,300	2,680	1.34	20	36	86	39	6.14	1.07	5.07	1.53	0.39	2.24	22.09	7.44	0.65		104,140		
21-IX-'71	2.80	3.13	454	2,400	5.29	6,800	3,080	1.28	20	32	87	34	7.65	1.03	6.62	1.77	0.34	2.53	21.10	8.46	0.60				
14-XII-'71	1.90	1.90	434	1,685	3.88	3,890	4.92	1.95	15	42	85	48	4.48	1.09	3.38	1.71	0.31	2.82	19.46	5.72	0.81		16,310		
1-III-'72	1.60		451	1,593	3.53	3,300	4.01	1.95	15	48	84	59	3.36	1.11	2.25	0.95	0.34	2.55	20.28	5.43	0.85		34,565		
16-III-'72	1.30	1.0	437	1,350	3.09	2,505	3.31	1.49	13	52	83	67	2.87	1.14	1.73	0.68	0.28	3.05	18.30	4.29	0.97		11,780		
9-VIII-'72	0.85		174	1,145	6.58	3,130	7.15	1.370	10	44	89	50	4.10	0.99	3.10	1.13	0.39	3.00	19.78	9.87	0.77		14,725		
Location: Rfo Sogamoso. Levels given at Barrancabermeja and Pto. Wilches gauge respectively																									
							$\bar{D}_{35}=230\mu\text{m}; \bar{D}_{50}=265\mu\text{m} (S=30\mu\text{m}); 2\bar{D}_{65}=618\mu\text{m}; \Delta=1.68; w=4 \times 10^{-2}(\text{m/s}); w_{\bar{D}_{50}}/v=13$																		
29-VII-'71	2.40	2.35	270	364	1.39	460	1.50	284	0.78	45	29	80	31	9.37	1.24	8.03	1.52	0.22	4.04	15.30	1.36	0.51		6,280	
1-IX-'71	3.30	2.50	290	628	2.17	956	2.16	611	0.97	45	30	83	32	8.65	1.13	7.51	2.18	0.31	2.84	19.02	2.63	0.41		9,855	
15-XI-'71	3.15	3.30	280	805	2.88	1,420	2.96	652	0.81	45	22	85	22	16.75	1.08	15.68	2.99	0.21	4.30	15.89	2.92	0.35			
14-XII-'71	1.90	1.90	93	294	3.16	537	3.22	256	0.87	45	22	86	23	15.54	1.06	14.48	3.26	0.23	3.79	17.06	3.44	0.34			
16-III-'72	1.30	1.0	115	199	1.73	271	1.83	156	0.78	45	27	81	29	10.66	1.18	9.47	1.85	0.21	4.21	15.30	1.69	0.46			
10-VIII-'72	0.90		76	188	2.47	325	3.17	237	1.26	45	34	84	38	6.64	1.11	5.53	3.21	0.52	1.73	24.71	3.89	0.38			
Location: Downstream Rfo Sogamoso, Rfo Magdalena km 610. Levels given at Barrancabermeja and Pto. Wilches gauge respectively																									
							$\bar{D}_{35}=280\mu\text{m}; \bar{D}_{50}=320\mu\text{m} (S=130\mu\text{m}); 2\bar{D}_{65}=755\mu\text{m}; \Delta=1.68; w=5.1 \times 10^{-2}(\text{m/s}); w_{\bar{D}_{50}}/v=20$																		
28/29-																									
VII-'71	2.40	2.35	460	2,200	4.78	5,300	5.35	3,050	1.39	15	47	88	56	3.55	1.02	2.54	1.49	0.47	1.88	24.81	8.31	0.61		72,970	
15-XII-'71	1.90	2.10	605	1,865	3.08	3,605	3.40	2,085	1.12	11	55	84	73	2.59	1.10	1.49	0.70	0.33	2.07	19.99	4.31	0.88		right branch	
			394	1,010	2.56	1,680	2.68	837	0.83	11	48	83	58	3.48	1.14	2.34	0.55	0.19	4.69	14.81	2.66	0.97		left branch	
10-VIII-'72	0.90		295	1,432	4.85	3,340	5.10	1,560	1.09	5	66	88	100	1.80	1.02	0.78	0.47	0.29	3.06	19.45	6.61	1.08			
Location: Vijagual, Rfo Magdalena km 535. Levels given at Pto. Wilches gauge																									
							$\bar{D}_{35}=270\mu\text{m}; \bar{D}_{50}=310\mu\text{m} (S=58\mu\text{m}); 2\bar{D}_{65}=705\mu\text{m}; \Delta=1.68; w=5 \times 10^{-2}(\text{m/s}); w_{\bar{D}_{50}}/v=19$																		
23-VII-'71	1.80		410	2,600	6.35	6,640	6.37	2,820	1.08	16	34	91	36	6.96	0.96	6.00	1.96	0.27	3.19	19.58	8.57	0.49		35,000	
30-IX-'71	2.40		405	2,808	6.94	7,470	6.94	3,850	1.37	16	41	91	46	4.73	0.94	3.78	2.13	0.43	2.01	24.84	11.88	0.48		38,975	
2-XII-'71	2.80		410	3,125	7.62	8,770	7.81	3,940	1.26	19	33	92	35	7.39	0.93	6.46	2.85	0.36	2.42	22.85	12.00	0.42		65,215	
14-I-'72	2.30		410	2,695	6.57	7,050	6.92	2,858	1.06	16	32	91	34	7.64	0.95	6.69	2.13	0.26	3.33	19.22	8.71	0.49		29,155	
17-III-'72	1.0		415	1,684	4.06	3,230	4.24	1,529	0.86	29	28	87	29	10.16	1.04	9.12	2.36	0.19	4.65	15.59	4.36	0.47		12,260	
Location: Rfo Lebríja. Levels given below local BH in the Rfo Lebríja and at Pto. Wilches gauge respectively																									
24-VII-'71	1.70		50	326	6.52		333	1.04																	
1-X-'71	1.79		3.00	100	864	8.64		905	1.05																
3-XII-'71	1.26		2.75	100	830	8.30		950	1.14																
18-III-'72	2.91		98	536	5.47			86	0.16																
Location: Calamar, Rfo Magdalena km 91. $\bar{D}_{35}=195\mu\text{m}; \bar{D}_{50}=210\mu\text{m} (S=48\mu\text{m}); 2\bar{D}_{65}=460\mu\text{m}; \Delta=1.68; w=3 \times 10^{-2}(\text{m/s}); w_{\bar{D}_{50}}/v=8$																									
22-XII-'70	8.29		578	7,550	13.06	28,000	13.78	14,260	1.89	7.4	59	100	74	2.24	0.79	1.45	2.89	1.02	0.91	41.64	30.85	0.31			
13-IV-'71	6.34																							76,450	
1/2-IX-'71	6.91		582	7,730	13.28	29,700	14.23	10,200	1.32	6.2	43	100	48	4.24	0.79	3.41	2.50	0.50	1.87	29.08	21.91	0.33		180,765	
21-III-'72	2.86		570	5,025	8.82	16,150	10.20	3,650	0.73	2.6	44	96	50	3.99	0.84	3.15	0.75	0.16	5.72	16.08	8.05	0.63		3,170	
18-IV-'72	3.96		565	5,247	9.29	16,775	10.58	4,800	0.91	3.5	48	97	56	3.35	0.84	2.52	1.05	0.25	3.71	20.05	10.57	0.53		23,540	

Remarks: Values underlined indicate estimated values; Q<sub>s</sub> indicates discharge in secondary parallel branch(es)

Table 3.3.1 B Discharge and Sediment Transport Data of the Rfo Magdalena

Barrancabermeja (Figure 3.3.3f)

Only the discharges which have been measured in front of the Ecopetrol refinery are listed in Table 3.3.1 (for the suspended-load concentrations reference is made to Para. 3.3.4). The cross-section used for the discharge measurements was not located at one fixed spot. For the computation of the Chézy-values an average water-level gradient between Barrancabermeja and Pto. Wilches has been used, which turns out to be a more or less constant value of  $27 \times 10^{-5}$ . This average value differs greatly from the locally-measured gradients near the confluence of the Río Magdalena and the Río Sogamoso.

The Río Sogamoso Confluence (Figure 3.3.3g)

The measurements were carried out in the Río Magdalena both upstream and downstream of the Río Sogamoso Confluence and in the Río Sogamoso itself. The cross-section downstream of the Río Sogamoso is divided into two branches, separated by islands.

In view of the short distance (about 30 km) between the gauges of Barrancabermeja and Pto. Wilches it had originally been thought that the average water-level gradient between these two gauges could be used for the computation of the roughness in the Sogamoso area. However, during the last measurements (August 1972) it appeared that the local gradients differed considerably from this average value. Checks made at a later date at different water-level stages confirmed these findings and the water-level gradients of the earlier measurements have been estimated accordingly.

Vijagual (Figure 3.3.3h)

Again the discharge of the secondary minor branch is listed separately.

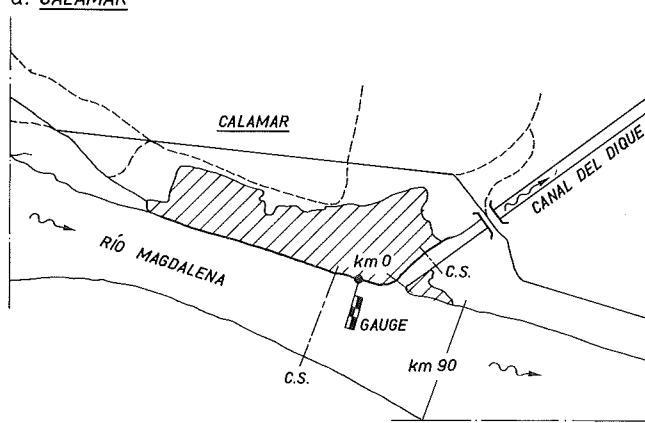
Calamar (Figure 3.3.3j)

The cross-section in the Río Magdalena at Calamar is situated in front of the village, about 400 m upstream of the bifurcation of the Río Magdalena and the Canal del Dique. Only during the measurements in March and April 1972, was the local water-level gradient measured. In accordance with checks carried out at a later date at higher water-level stages, the local gradient can be estimated dividing the actual gauge-readings at Calamar (zero-level of the gauge is - 0.35 m in respect of M.S.L.) by the distance between Calamar and Bocas de Ceniza (112 km). This procedure was followed for the measurements in December 1970 and September 1971 respectively.

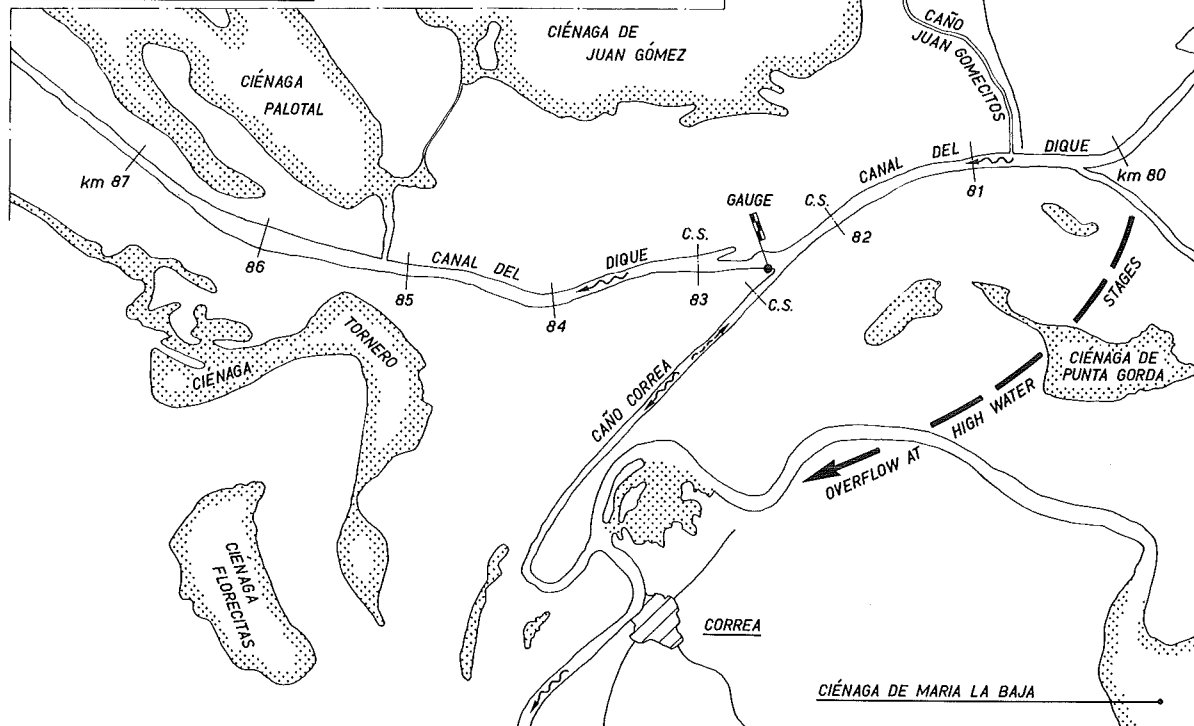
B. The Canal del Dique (Table 3.3.2 and Figure 3.3.4)

The locations of the cross-sections used for the discharge and sediment transport measurements in the Canal del Dique between Calamar and the Bahfa de Cartagena and in the distributing branches are given in Figure 3.3.4. All the data resulting from these measurements are presented in Table 3.3.2. A few remarks regarding the cross-sections again seem necessary.

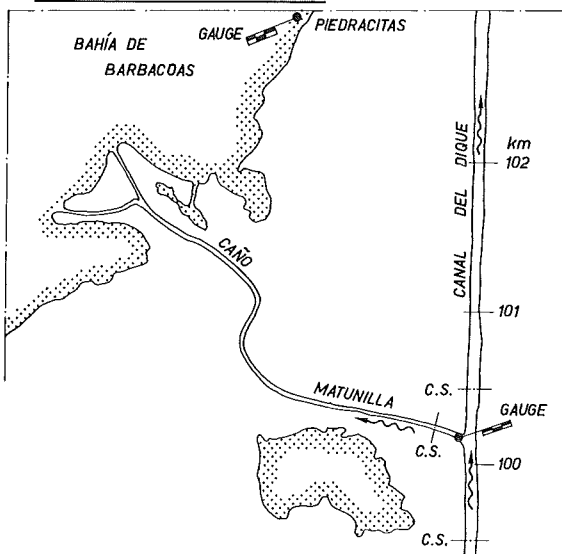
a. CALAMAR



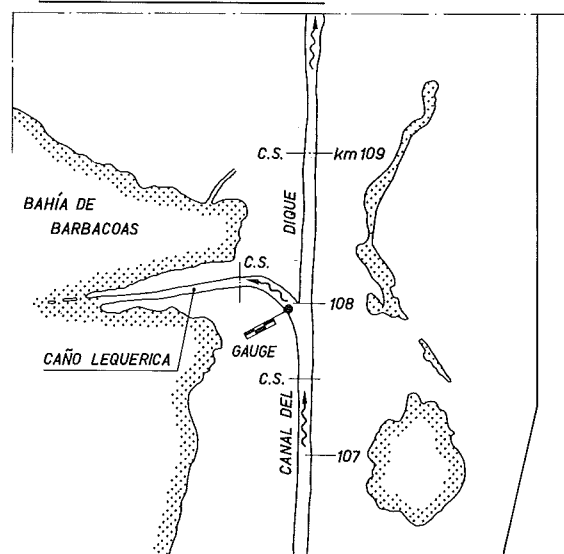
b. CAÑO CORREA BIFURCATION



c. CAÑO MATUNILLA BIFURCATION



d. CAÑO LEQUERICA BIFURCATION





Calamar (Figure 3.3.4a)

The cross-section in the Canal del Dique at Calamar is situated about 200 m downstream of the bifurcation. The water-level gradient has not been measured and had to be estimated. The total length of the Canal del Dique is about equal to the distance along the Río Magdalena from Calamar to Bocas de Ceniza, and the difference in head between the water-level at Calamar and M.S.L. at Bocas de Ceniza and in the Bahía de Cartagena is also about the same. Therefore, the assumption has been made that the water-level gradient in the Canal del Dique close to the bifurcation of the Río Magdalena would be equal to the water-level gradient in the Río Magdalena at Calamar.

The Caño Correa Bifurcation (Figure 3.3.4b)

The Caño Correa leaves the Canal del Dique at km 82.5 and debouches with many outlets into the Bahía de Barbacoas. Locally water-level gradients have not been measured, but determined as average gradients. For the section upstream of the Caño Correa the gradient has been found from the water-levels at Gambote and at the bifurcation of the Caño Correa; for the downstream section the water-levels at Correa and Matunilla have been used. Because no gauge readings are available near the outlet of the Caño Correa, the records of the gauge at Piedracitas in the Bahía de Barbacoas have been used. The water-level gradient in the Caño has been determined as an average gradient between this gauge and the one at the bifurcation. At high water stages such an average value of the water-level gradient cannot be correct because downstream of Gambote an overflow of the low-lying left bank of the Canal del Dique occurs. This overflowing water partly enters the Caño Correa some distance downstream of the bifurcation with the Canal del Dique, resulting in a local rise of the water-level (see Figure 3.3.4b). Consequently, the local water-level gradient in the Caño Correa close to the bifurcation decreases, and occasionally it may even be noticed that the Caño Correa discharges into the Canal del Dique.

The Canal del Dique and the Caño Matunilla Bifurcation (Figure 3.3.4c)

The Caño Matunilla leaves the Canal del Dique at about km 100 and discharges into the Bahía de Barbacoas. The water-level gradients have been determined as average gradients between the gauges of Correa, Matunilla, Lequerica and Piedracitas (Bahía de Barbacoas) respectively. When the readings of the last gauge were not available, the readings of the gauge at Carare (Bahía de Cartagena) were used.

The Canal del Dique and the Caño Lequerica Bifurcation (Figure 3.3.4d)

The Caño Lequerica leaves the Canal del Dique at about km 108 and discharges also in the Bahía de Barbacoas. The water-level gradients have been determined as average gradients between the gauges of Matunilla, Lequerica, Carare (Bahía de Cartagena) and Piedracitas (Bahía de Barbacoas) respectively. When the readings of one of these gauges were not available, those of a nearby gauge were used. Occasionally, the tide predictions at Cartagena were used if no readings of the Carare gauge were available.

3.3.3. The suspended-load measurements

For the measuring technique of the suspended-load sampler, reference is made to Part IV of this Report and the operation manual of the instrument. However, some remarks regarding the measurements must be made here because the method of measuring, as used by the Mission results in too high a suspended-load. For small and moderate concentrations of suspended-load, the time interval for measuring with the Delft Bottle (DF) in a certain point in the cross-section is large compared with the time needed to lower the sampler to the required depth, and to raise the instrument after the measurement has been completed. For instance, for the Rhine branches (the Netherlands) the measuring time is 20 min., whereas in the Rfo Magdalena the measuring time had to be restricted to 3 min., because otherwise too much sediment was caught in the sampler, thus possibly reducing the efficiency of the instrument. Therefore, the majority of the measurements in the Rfo Magdalena have been carried out with a constant measuring time of 3 min. when the instrument was positioned at the required depth in the vertical. In view of the high concentrations of suspended-load, these catches have had to be corrected for the surplus caught during the time interval necessary to lower and raise the instrument. Some information about this reduction factor is given below.

Generally, the time interval necessary to lower the instrument to the required depth is small compared with the time interval needed for the measurement itself and to raise the instrument afterwards. Hence, this time interval is neglected.

In the first instance, a logarithmic distribution of the sediment concentration in the vertical will be considered in the following Figure 3.3.5.

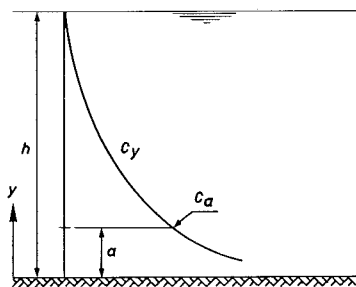


Figure 3.3.5 Logarithmic Distribution of the Sediment Concentration

The sampler is positioned at distance  $a$  above the river-bed. At this point a sediment transport  $s^1(a)$  is present:

$$s^1_0(a) = \overline{c(a) \cdot v(a)} \approx \overline{c(a)} \cdot \overline{v(a)} \tag{3.3.1}$$

This transport is measured during the time interval  $\theta_0$ . The time interval  $\theta_1$  is required to raise the instrument after the measurement has been completed. Hence, instead of the true transport  $s^1_0(a)$  a value  $s^1_1(a)$  is measured:

$$s^1_1(a) = s^1_0(a) + \frac{1}{\theta_0} \int_0^{\theta_1} c(y) \cdot v(y) dt \tag{3.3.2}$$

Assuming that the instrument is raised with a constant speed, the following expression holds:  $(h-a)dt = \theta_1 dy$ ; Eq.(3.3.2) therefore becomes:

$$s^1_1(a) = s^1_0(a) + \frac{\theta_1}{\theta_0} \int_a^h c(y) \cdot v(y) \frac{dy}{(h-a)}$$

This can also be written as:

$$s'_1(a) = \frac{1}{\sigma} s'_0(a)$$

In which:

$$\frac{1}{\sigma} = 1 + \frac{\theta_1}{\theta_0} \int_a^h \frac{C(y) \cdot v(y)}{C(a) \cdot v(a)} \cdot \frac{dy}{(h-a)} \quad (3.3.3)$$

As an approximation a uniform velocity distribution, ( $v(y) = v(a)$ ) is assumed (the influence of the true velocity distribution is of secondary importance only). Adopting, moreover, the theoretical distribution of the concentration:

$$\frac{C_y}{C_a} = \left[ \frac{a}{(h-a)} \cdot \frac{(h-y)}{y} \right]^z \quad \text{with } z = \frac{w}{K \cdot v_*} \quad (3.2.5)$$

the reduction factor  $\sigma$  can be expressed, after some derivation, in the parameters  $\theta_1$ ,  $\theta_0$ ,  $a$ ,  $h$  and  $z$ :

$$\sigma = \left[ 1 + \frac{\theta_1}{\theta_0} \cdot \frac{(a/h)^z}{(1-a/h)^{z+1}} \cdot \int_{a/h}^1 \left( \frac{1-\eta}{\eta} \right)^z d\eta \right]^{-1} \quad (3.3.4)$$

In which:  $\eta = \frac{y}{h}$

Equation 3.3.4 can also be written as:

$$\sigma = \frac{1}{1 + \alpha \theta_1 / \theta_0} \quad (3.3.5)$$

when introducing

$$\alpha = \frac{(a/h)^z}{(1-a/h)^{z+1}} \int_{a/h}^1 \left( \frac{1-\eta}{\eta} \right)^z d\eta = \alpha \left( \frac{a}{h}, z \right) \quad (3.3.6)$$

The reduction factor ( $\sigma$ ) depends on 3 parameters ( $\theta_1/\theta_0$ ,  $a/h$  and  $z$ ) and the graphical determination can best be served by using two graphs (Figure 3.3.6). Figure 3.3.6a gives the parameter  $\alpha$  (Eq.3.3.6) as a function of  $a/h$  and  $z$ ; Figure 3.3.6b relates the reduction factor  $\sigma$  (Eq.3.3.5) to  $\theta_1/\theta_0$  and  $\alpha$ .

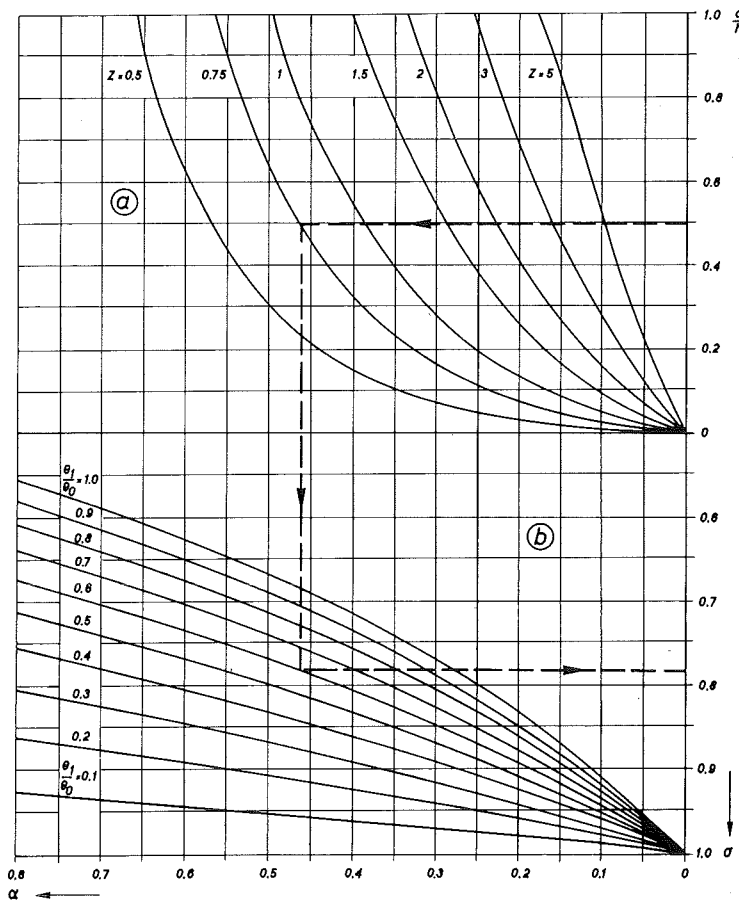


Figure 3.3.6 Reduction Factor for Suspended-load Measurements

Figure 3.3.6 can be used universally for all values of  $z$ . In Para. 3.3.4 it is shown that the observed values of the exponent  $z$  in Eq.(3.2.5) ( $z_1$ ) along the Río Magdalena generally fluctuates between 0 and 1 (see Figure 3.3.8). Assuming that the instrument is raised at a (constant) speed of 0.20 m/s, the maximum time interval required to raise the instrument in a water-depth of, e.g., 4 m becomes 20 sec. (or  $\theta_1 = \frac{1}{9} \theta_0$ ). For the given range of  $z_1$ -values the error in the measured catches of the DF-sampler is smaller than 11%. For all the measuring points in the vertical and an average  $z_1$ -value ( $z_1 = 0.5$ ), the error will be in the order of 5%. The error determined in this way implies that the efficiency of the sampler does not change when raising the instrument. This, in fact, will not be true, and it seems logical that the efficiency will sharply decrease due to the angle of attack between the current and the axis of the sampler when the instrument is raised. Consequently, the average error in the measured transport vertical will even be less than 5%, and it can be concluded that the influence of this reduction factor can well be neglected. (An exception must perhaps be made for the measurements in the Río Magdalena at Calamar where the river is very deep; however, until more is known about the efficiency of the sampler when it is raised, it is suggested that this reduction factor be neglected.

Another correction factor which has to be considered depends on the efficiency of the DF-sampler itself (reference is made to the operation manual of the instrument and also Part IV of this Report). The principle of the instrument is that the water-sediment mixture flows through a bottle-shaped sampler, the shape of which induces a low pressure at the back in such a way that the mixture enters the nozzle of the sampler with almost the same velocity as in the undisturbed flow. The decrease of the velocity in the sampling compartments causes a settlement of the sediment. The correction factor which has to be applied on the measured catches is the ratio of the loss-coefficient and the hydraulic-coefficient. The loss-coefficient, being the ratio of the total sediment volume which enters through the nozzle and the actual catch, increases with increasing velocities and decreasing grain sizes. The hydraulic-coefficient is the ratio of the discharge through the nozzle and the discharge through the same imaginary orifice if the instrument would not have been present. The correction factors are given in tables in the manual of the instrument, and for the pertaining conditions on the Río Magdalena show a value of about 0.9. This means that the catches of the sampler are 10% higher than the actual transport of sediment. Although this correction factor has not been applied in the data under consideration in this Report, it is suggested that this will be done with the data of future measurements.

#### 3.3.4. The suspended-load concentrations

During the sediment transport measurements carried out in the Río Magdalena it appeared that the total amount of sediment transported by the river is extremely high and also that the percentage of suspended load is, generally, 90% or more of the bed-material load. When an insight into the distribution of the suspended-load in the vertical can be obtained, the computation of the sediment transport is also possible by means of Eq.(3.2.6). Moreover, the laborious and time-consuming transport measurements can then be reduced to a single measuring point in each vertical.

To enable concentration verticals to be compared, they are generally plotted in a dimensionless form, dividing the concentrations in the consecutive measuring points of a vertical by one and the same reference concentration ( $C_a$ ), this being the concentration found at a small distance ( $a$ ) above the bed. Experimental investigations, both in the laboratory and in the field, have confirmed that the concentration distribution in a vertical can best be expressed as (see Figure 3.3.5) [13]:

$$C_y = C_a \left( \frac{a}{h-a} \cdot \frac{h-y}{y} \right)^z \tag{3.2.5}$$

in which:

- $C_a$  = reference concentration at height  $a$  above channel bed
- $C_y$  = concentration at height  $y$  above channel bed
- $h$  = total depth at measuring site
- $z = w/(\kappa \cdot v_*^*)$
- $\kappa$  = von Kármán constant (= 0.4 for clean water)
- $w$  = fall velocity
- $v_*^*$  = shear velocity.

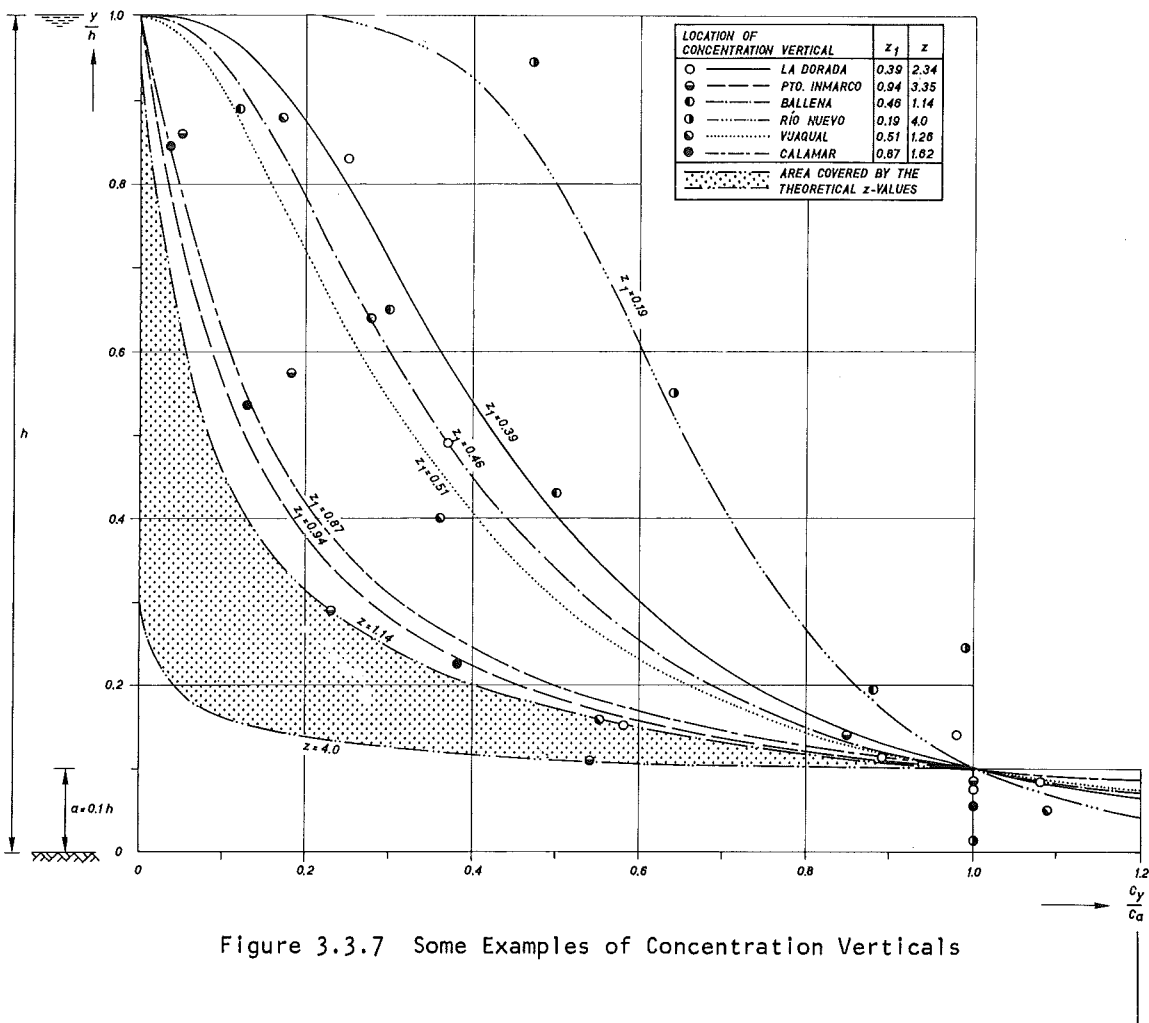


Figure 3.3.7 Some Examples of Concentration Verticals

Comparison of the measured suspended-load verticals with the theoretical concentration curves (that is to say, comparison of theoretical  $z$ -values with  $z$ -values as resulting from the Río Magdalena measurements) was done as follows:

- A dimensionless graph was made plotting  $C_y/C_a$  versus  $(h-y)/y$ ;
- the measured suspended-load catches were transformed into concentrations (concentration being the catch divided by the pertaining flow-velocity); and

II, 3.3

- the reference concentration ( $C_a$ ) was determined at distance  $a=0.1h$  above the bed. As the Delft Bottle used for the suspended-load measurements measures the sediment transport at fixed distances above the bed-level, the reference concentration at  $0.1 h$  above the bed was found by interpolation of the concentrations measured at  $0.1 m$ ,  $0.2 m$ ,  $0.3 m$ ,  $0.4 m$  and  $0.5 m$  above the bed. The  $z_1$ -values were then calculated from

$$z_1 = \frac{\log C_y/C_a}{\log \left( \frac{h-y}{y} \cdot \frac{0.1 h}{0.9 h} \right)}$$

This equation is based on the assumption that Eq.(3.2.5) holds, which means that on logarithmic paper the plots can be schematized by a straight line.

Some results are given in Figure 3.3.7. For convenience sake, the examples have been plotted here on a linear scale. To enable a comparison to be made between verticals of measuring cross-sections with different depths,  $y/h$  has been used as ordinate instead of  $(h-y)/y$ . In Figure 3.3.7 the area covered by the theoretical  $z$ -values ( $z = w/\kappa v_*$ ) is also indicated.

In Figure 3.3.8 a comparison is given between the theoretical  $z$ -values ( $z$ ) and the observed  $z_1$ -values. A significant difference can be noted and needs explanation.

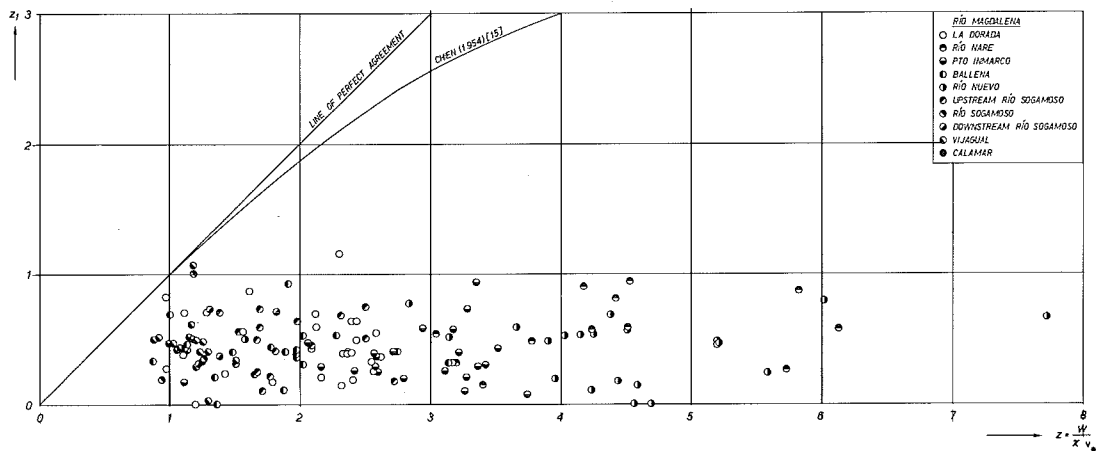


Figure 3.3.8 Suspended-load Concentrations in the Rio Magdalena

Some possible reasons have already been mentioned in Para. 3.2.2, but also the following aspects need to be mentioned:

- In the calculations  $\kappa = 0.4$  has been used. However, in doing so the influence of the presence of suspended material on the  $\kappa$ -value (the turbulence) has been neglected. Einstein et al. (1954) [14], discussing this effect, also found  $z_1 < z$ . This was confirmed by experiments by Chien (1954) [15] of which the results are also given in Figure 3.3.8. Flume experiments by Vanoni (1941, 1946) [16 and 17] showed a tendency of decreasing  $\kappa$ -values ( $\kappa < 0.4$ ) for increasing

- suspended-load concentrations. This was explained by suggesting that a reduction of  $\kappa$  means that mixing is less effective and, apparently, the presence of sediment particles in suspension suppresses or damps the turbulence. However, for values of  $\kappa$  inferior to 0.4, the theoretical  $z$ -values even increase and the difference from the measured values becomes still greater.
- When discussing the derivation of Eq.(3.2.5) it was mentioned that the diffusivity of solid particles ( $\epsilon_s$ ) was assumed to be equal to the diffusivity for momentum by means of turbulence ( $\epsilon$ ). This means that  $\beta=1$  was used in Eq.(3.2.4). Apparently the relation given by  $\epsilon_s = \beta \epsilon$  (Eq.(3.3.7)) needs a closer examination. Carstens (1952) [18] expressed the turbulence as linear fluctuations in the velocity and after presenting a mathematical expression for  $\beta$ , he concluded that  $\beta$  never exceeds unity. This is supported by carefully conducted experiments by Brush et al. (1962) [19], Matyukhin et al. (1966) [20] and Majumdar et al. (1967) [21]. However, Singamsetti (1966) [22] expressed turbulence as eddies in which the solid particles tend to fly off due to centrifugal forces, increasing the diffusion and meaning that  $\beta > 1$  and because  $z_1 = z/\beta$ ,  $z_1 < z$ .
  - The theoretical  $z$ -value can only be calculated for a known value of the fall velocity ( $w$ ). Einstein (1950) [23] and others compute the fall velocity for the different fractions of which the suspended material is composed and totals the concentrations of all fractions. However, to arrive at the results as given in Figure 3.3.8 the fall velocity corresponding to the mean diameter ( $D_{50}$ ) of the bed material was used, because not all catches of the Rio Magdalena suspended-load measurements could be analysed. As the bed material is generally coarser than the suspended particles, this will have resulted in too high  $z$ -values. In future it may be useful to measure the fall velocity for the fractions of a number of samples, for which purpose a Visual Accumulation Tube (VAT) can be recommended (see Part IV of this Report).
  - To have some insight into the correct fall velocity, near Barrancabermeja a number of suspended-load measurements were carried out over such a long period (10 min.) that the catches were large enough to permit analyzing the samples (contrary to the other suspended-load data, the data of these measurements were corrected with the correction factor of the DF-sampler). With these results, for those groups of grain-sizes and the corresponding fall velocities a further comparison was made between the theoretical and observed  $z$ -values. The results are presented in Figure 3.3.9, from which it can be seen that the correlation is still very bad. It should be noted that even negative  $z_1$ -values are found. This means that in a vertical the concentration near the water surface is greater than near the river-bed. This may be possible under special circumstances (helicoidal flow or lateral supply of suspended material at a high level), but does not look very probable for the measuring site near Barrancabermeja (straight river stretch with parallel flow-lines).

## II, 3.3

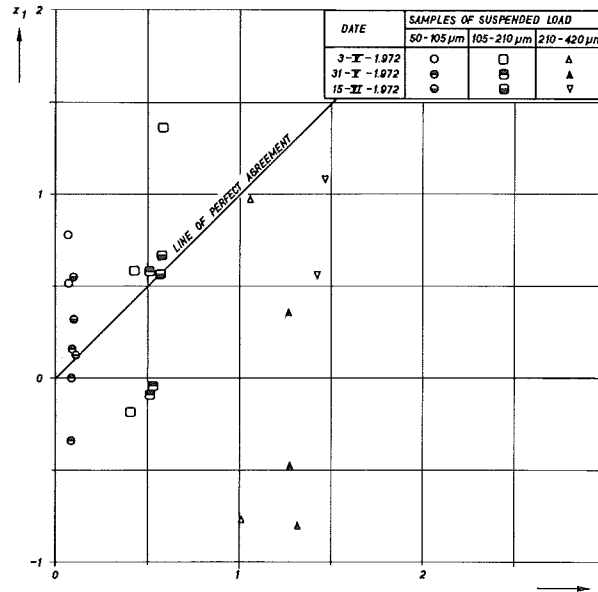


Figure 3.3.9 Suspended-load Concentrations near Barrancabermeja

Summarizing, it can be concluded that for the Río Magdalena the distribution in a vertical of the suspended-load concentrations does not correspond with theoretical distributions and that no satisfactory explanation of the observed discrepancies can be given. It is recommended that this subject be studied further in Colombia after the results of additional measurements have become available. (For more background information on this subject reference is made to, amongst others, Graf (1971) [24].

Discussions on the influence of the suspended-load on the (changes of the) bed-level, are to be found in Para. 3.6.3.

### 3.3.5. The wash-load concentrations

In Para. 3.2.3 the wash-load has been defined as the transport of particles finer than the bulk of the bed material and therefore rarely found in the bed. The amount of wash-load depends on climate, erosion in the catchment area, bank caving, etc., and not on the characteristic flow parameters as is the case with bed-load and suspended-load. Consequently, it cannot be computed from such parameters but has to be measured. Owing to its small size fractions, it moves readily in suspension and is merely washed through the river sections. As such, it is of no importance for the computations of local scour and sedimentation but settles only in areas with very low velocities or in stagnant water (for example, in "cienagas", harbours, or outlets into the sea). Along the Río Magdalena and the Canal del Dique various cienagas can be distinguished (e.g., from an aerial view) which show completely clear water in which apparently settlement has already occurred, while others still show silty water. The presence of cienagas in the river valley may cause fluctuations in the wash-load concentrations in the river over, relatively, short distances. Generally, the filling of the cienagas occurs by overflow of the river bank and the emptying, at low water stages in the river, via small drainage channels.

Downstream of such a cienaga the wash-load concentration in the river can be lower (e.g., near Gambote in the Canal del Dique; Figure 3.3.12), if the outflow of the cienaga no longer contains silt. However, also higher concentrations may be found if none, or only part, of the silt has settled in the cienaga and if during the emptying of the cienaga the finer particles of the bed material of the drainage channels are also picked up and transported as wash-load.

Due to the small fall velocity of the single silt particles the wash-load can be assumed to be evenly distributed over the vertical. However, over the cross-section the concentration needs not be constant. Such an example can be found downstream of the confluences of the Rfo Magdalena and the Rfo Negro (km 841) and the Rfo La Miel (km 837). The Rfo Negro discharges chalky particles along the right bank of the Rfo Magdalena while the generally clear water of the Rfo La Miel enters the Rfo Magdalena a few kilometers downstream along the left bank. The aerial photographs of this section of the Rfo Magdalena show a clear division between the discharge of both these rivers, and at Pto. Boyacá (km 805) it cannot be assumed that the river water is again homogeneously mixed.

As the behaviour of wash-load is largely controlled by electrochemical processes, the presence of salt in tidal regions or in the sea can cause a coagulation of the single silt particles to greater flocs. Consequently, the fall velocity increases and even with moderate flow-velocities sedimentation can already occur.

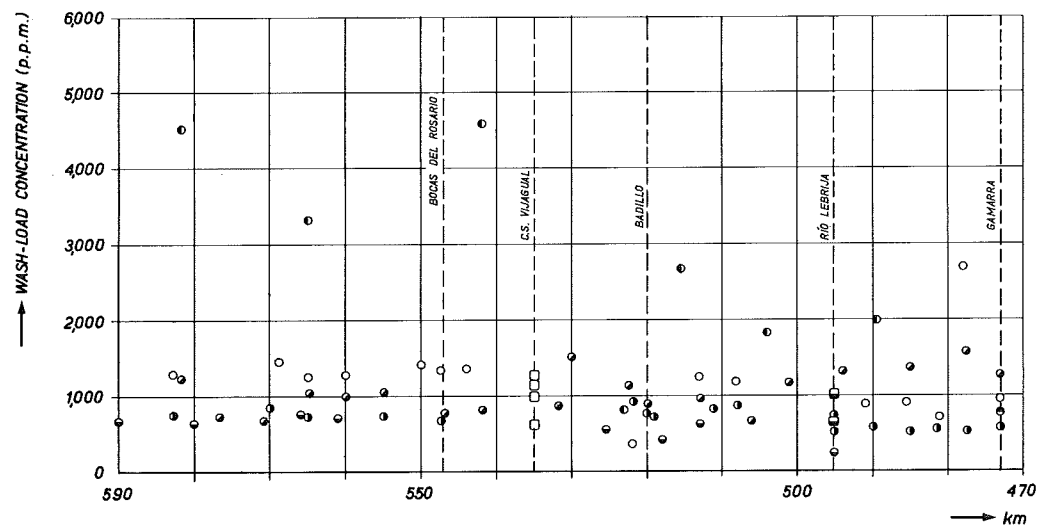
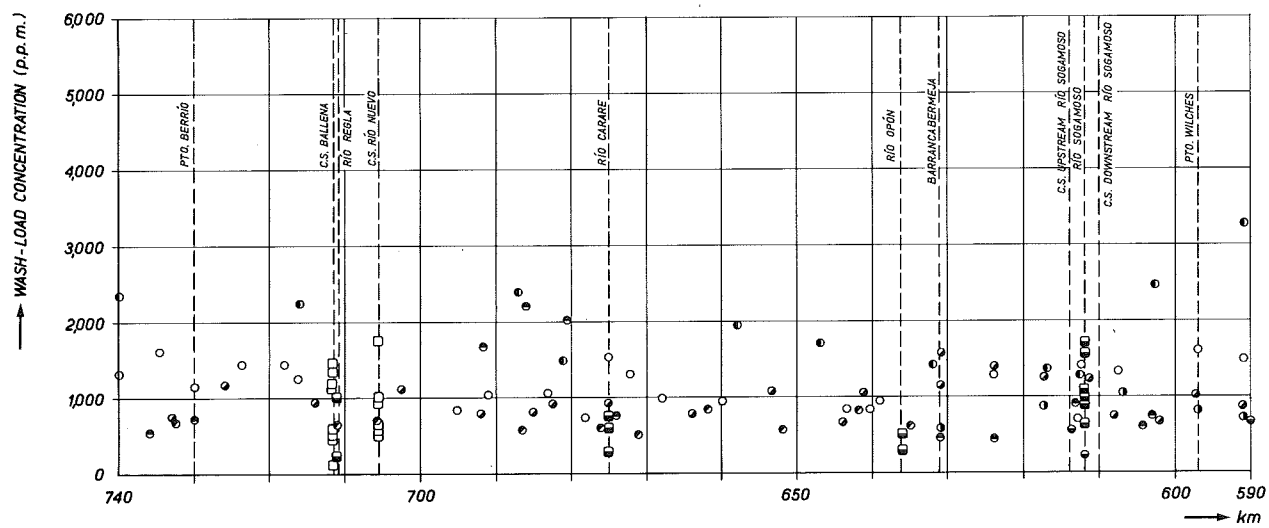
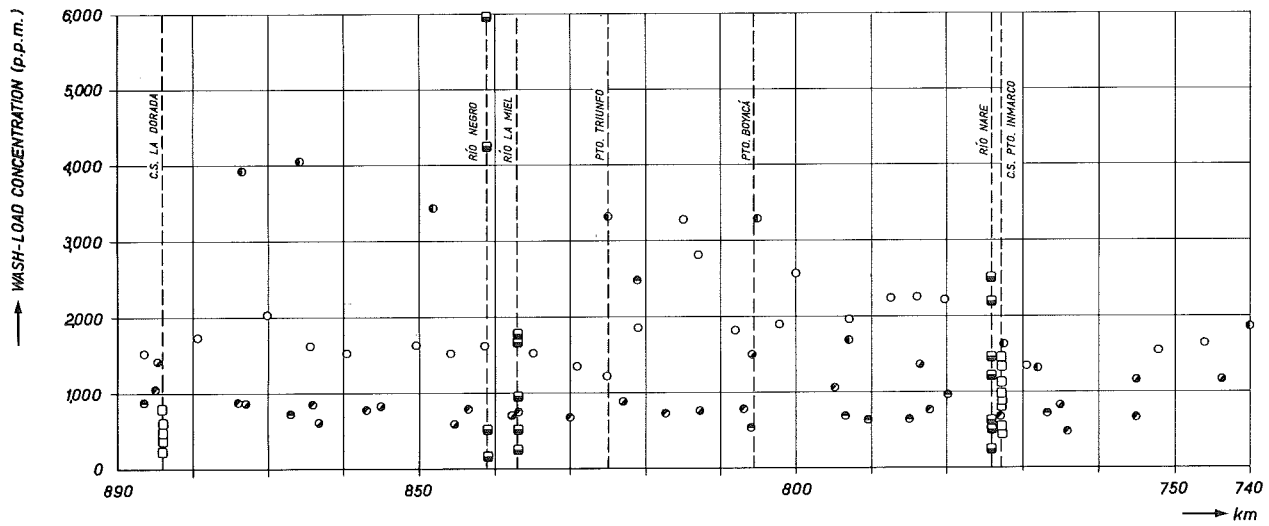
During the study a large number of water-samples were taken. The concentration was determined by filtration, drying and weighing of the samples and is expressed in parts per million (p.p.m.).

#### A. The Rfo Magdalena

Water-samples were taken during the discharge and sediment transport measurements along the Rfo Magdalena. In addition, samples were also taken during the longitudinal soundings at regular distance intervals. The data have been compiled in Figure 3.3.10.

Attention has already been drawn to the fact that the wash-load concentration may vary considerably over the width of the river. This is the reason for the fluctuations in the concentration found in the Rfo Magdalena downstream of the confluence with the Rfo Negro. When the navigation channel in the Rfo Magdalena followed the right bank (discharge of the Rfo Negro) high concentrations were found, while along the opposite bank (discharge of the Rfo La Miel) the concentrations were considerably lower.

To get an insight into the fluctuation of the wash-load concentration, daily water-samples were taken in Barrancabermeja. This information can be found in Figure 3.3.11. For reference purposes, the daily water-levels, as well as the water-temperature during the months of July, August and September 1972, have also been plotted.



LEGEND	
○	NOVEMBER 1971
●	DECEMBER 1971
⊙	MARCH 1972
⊕	MAY 1972
⊖	JUNE 1972
⊗	AUGUST 1972
⊘	SEPTEMBER 1972
□	MEASURING C.S. IN THE RÍO MAGDALENA
▣	AFFLUENTS

WASH-LOAD CONCENTRATIONS ALONG THE RÍO MAGDALENA AND IN AFFLUENTS

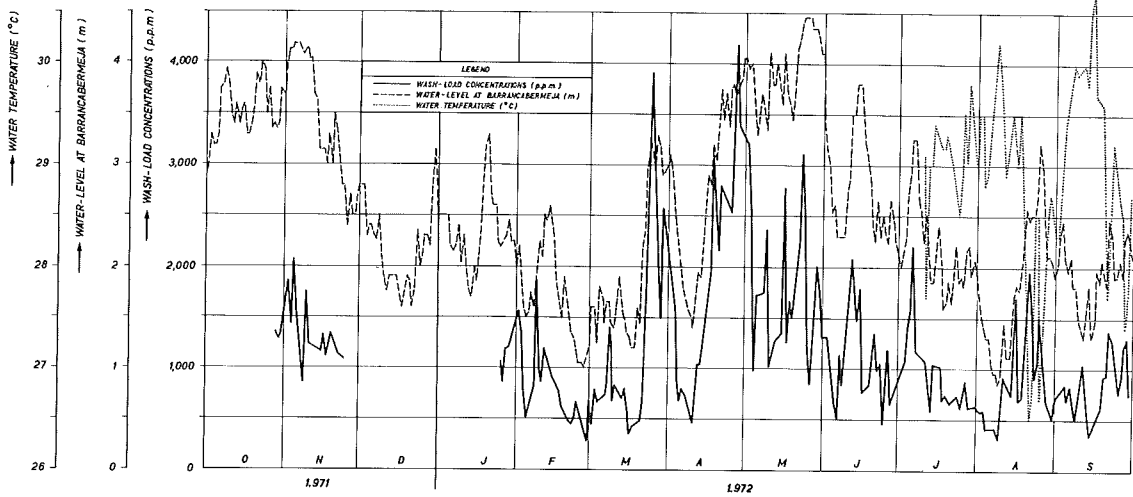


Figure 3.3.11 Wash-load Concentrations in Barrancabermeja

B. The Canal del Dique

Water-samples were taken during the discharge and sediment transport measurements and, moreover, at regular distance intervals during the longitudinal soundings. The data have been compiled in Figure 3.3.12, which also includes information gathered by the Laboratoire Central d'Hydraulique de France in 1964.

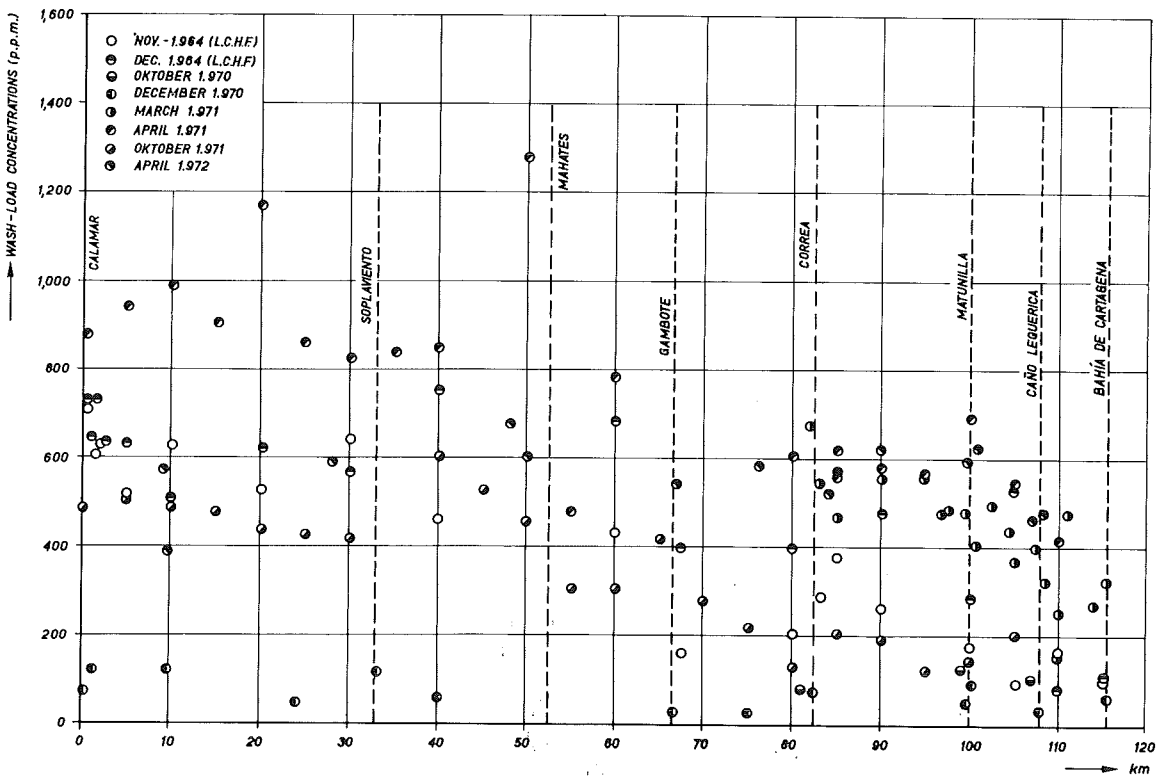


Figure 3.3.12 Wash-load Concentrations along the Canal del Dique

II, 3.3

The daily fluctuation of the wash-load concentration in Gambote is presented in Figure 3.3.13. This information was gathered in the period of 1965 to 1969 by DICON, for the use of the Waterworks Department in Cartagena. The average daily concentrations are presented, together with the average daily water-levels in Gambote during the period under consideration.

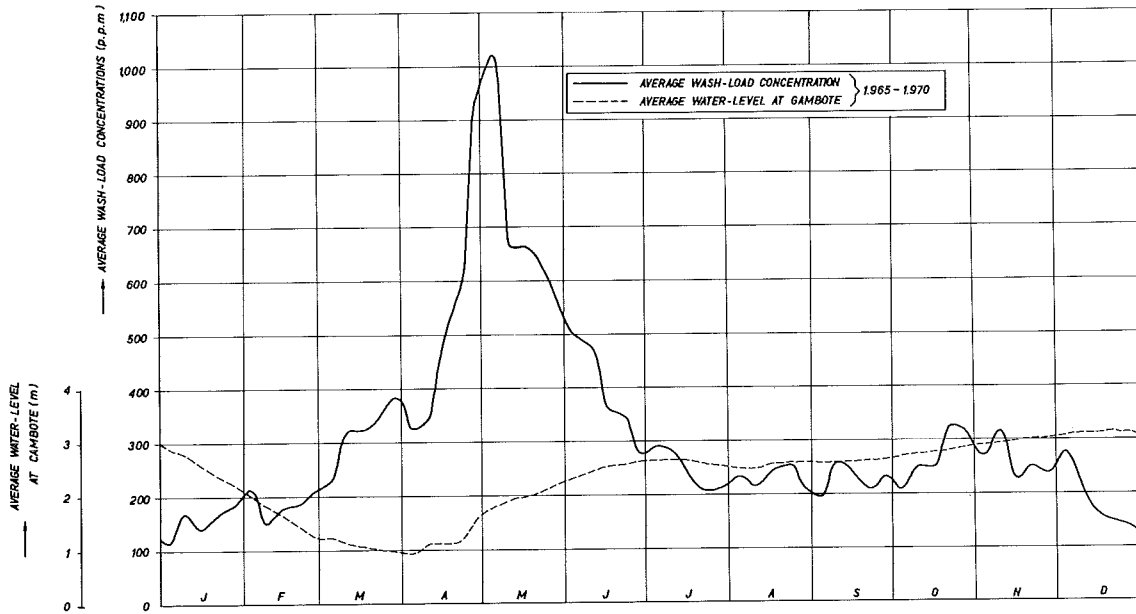


Figure 3.3.13 Wash-load Concentrations in Gambote

3.3.6. Bed-samples and grain-sizes

Together with the water-samples bed-samples were also taken along the Río Magdalena and the Canal del Dique. After sieving the samples, grain-size distribution-curves of the bed material were made, and from these curves the values of  $D_{35}$ ,  $D_{50}$  and  $D_{65}$  respectively could be read. A few of these curves are presented in Figure 3.3.14.

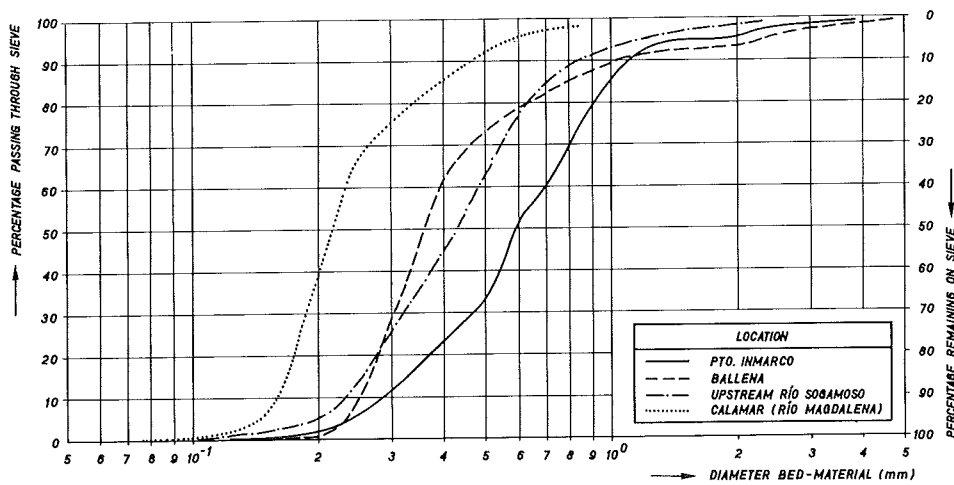


Figure 3.3.14 Some Examples of Grain-size Distribution-curves

From a number of these samples the porosity ( $\epsilon$ ) has been determined. Most of the transport equations express the transported amount of sediment in solid volume or in weight, while both the measured volumes of sediment transport and the local deformations of the river-bed arising from changes in the transport capacity include the voids between the grains as well. For a comparison of the computed transport and the field data and to predict changes in the bed-level, it will, therefore, be necessary to insert a porosity-factor in the transport equation. Obviously, a proper determination of the porosity can only be made by means of undisturbed samples of the upper-layer of the river-bed. As it is impossible to take such samples, the porosity has therefore been determined from a number of the samples taken by means of a bottom-grab. The values of  $\epsilon$  were found to vary between 0.3 and 0.5 and an average value of  $\epsilon = 0.4$  has been used.

It is often noted that along the river the particle-size of the bed material gradually decreases, and various investigators have tried to correlate this phenomenon to hydraulic parameters. The first (incorrect) attempt tried to establish a correlation between the size of the soil particles carried by the flow on the one hand, and the water-level gradient on the other hand. This seems logical, because the water-level gradient is also found to diminish gradually from the source of the river to its final outlet into the sea. Thus it was suggested, since the steeper gradients are symptomatic of higher velocities, that the large particles of the bed material are capable of being transported by the flow only in the upper reaches of the river. As the water-level gradient and, therefore, also the velocity, diminishes downstream, the coarser particles are gradually left behind and replaced by the finer grains, characteristic of the lower stretches of the river [25].

A second attempt correlated the gradual reduction in the size of the grains to the effect of abrasion consequent on the friction between the particles, as they are pushed and rolled down the river by the flow. This attempt led to the formula of Sternberg (1875)[26]:

$$W = W_0 \cdot e^{-\alpha L} \quad (3.3.8)$$

in which:

$W$  = weight, or size, of a particle, being  $W_0$  in the origin of  $L$ ; and

$\alpha$  = coefficient depending on the nature of the grain and the characteristics of the river (to be determined from measurements).

A third attempt was made, amongst others, by Lokhtine (1909) [27] who accepted the correlation between the water-level gradient and the average grain-size as a physical fact. Lokhtine concluded that all natural streams can be divided into two fundamental classes, described respectively as stable and unstable channels. A correlation exists between the "stability" of a channel on the one hand and a "coefficient of fixation" on the other. This coefficient is equal to  $\bar{D}/I$  because the resistance of a particle to being moved is proportional to its weight ( $D^3$ ), while the force which tends to propel the particle downstream is proportional to the exposed area ( $D^2$ ) of the particle on the one hand and the square of the velocity (or, according to Chézy, to the water-level gradient) on the other hand.

## II, 3.3

It has not been tried to establish a correlation between the average grain-sizes of the bed-samples taken along the Río Magdalena and the Canal del Dique and the water-level gradient. First of all, because of the great number of affluents which debouch material of different sizes and origin into the main river (e.g., the Río Nare of which the bed material is very coarse indeed), and secondly, because of the very great number of samples required to establish such a correlation.

The average sieving-curve of the samples taken at one place has been used for the calculation of the bed-roughness (Tables 3.3.1 and 3.3.2) and to determine the theoretical z-value of the concentration vertical according to Eq.(3.2.5) (Figure 3.3.8). According to the derivation of the different transport equations (Para.3.2) a representative diameter of the bed material has to be chosen (e.g.,  $D_{50}$ ). This implies that the rate of transport at a certain place depends on parameters of the river stretch upstream where the motion of the particles originated. However, this "history" of the sediment transport is at present always neglected because the assumption is made that the adaption of the sediment transport to the flow-parameter is instantaneous.

### A. The Río Magdalena

The  $D_{50}$  values of the bed-samples taken along the Río Magdalena and in the affluents close to the main river are presented in Figure 3.3.15. A few remarks must be made about some of the measuring cross-sections.

The river-bed near the cross-section in La Dorada, downstream of the railway bridge, consists mostly of sand, but also small stones and boulders, originating from the diluvial settlements more upstream, are found. It is noteworthy that while the bed samples consisted predominantly of sand, the samples of the bed-load sampler (B.T.M.A.) mostly consisted of the stones rolling and sliding along the bed. It has already been mentioned that the bed-load is only a negligible part of the bed-material load and, therefore, it has been assumed that the diameters of the bed-samples are representative of the material in suspension. If this assumption is true, it implies that, gradually, armouring of the river-bed occurs, viz., the finer particles are transported downward while the stones and boulders, although not at rest, stay behind.

The cross-section at Pto. Inmarco is situated just downstream of the confluence of the Río Magdalena and the Río Nare. It has already been mentioned that the bed material of the Río Nare is very coarse ( $\bar{D}_{50}=1,080\mu\text{m}$ ). The majority of the bed-samples taken at Pto. Inmarco are fine ( $\bar{D}_{50}=350\mu\text{m}$ ), but on a few ridges in the river bed material was encountered which might well originate from the Río Nare. This is the reason why the average diameters mentioned in Table 3.3.1 are so high ( $\bar{D}_{50}=1,050\mu\text{m}$ ).

At the cross-section in the Río Nuevo the same phenomenon may occur as mentioned for the La Dorada section. The banks and, most likely, part of the river bed too, consist of diluvial deposits and, consequently, the average diameters mentioned in Table 3.3.1 are rather high ( $\bar{D}_{50}=920\mu\text{m}$ ). Near the Ballena section, about 5 km upstream of the Río Nuevo section where the river is wide and shallow, the bed material appears to be much finer ( $\bar{D}_{50}=405\mu\text{m}$ ). It is shown in Para. 3.5.3 that the bed-material load transported through the

II, 3.3

Ballena section is more or less transported through the Rfo Nuevo section too. However, if the sediment transport is computed by means of the equation of Engelund and Hansen using the average grain diameters mentioned in Table 3.3.1, it follows that the computed transport in the Ballena section is about four times higher than the transport through the Rfo Nuevo section. It is therefore likely that the average grain diameters of the bed material given in Table 3.3.1 for the Ballena and Rfo Nuevo sections are not representative of the total sediment transport. (Moreover, it must be considered that the sediment transported by the Rfo Regla may be of a different composition from the sediment of the Rfo Magdalena). Further reference is made to Para. 3.5.3.

B. The Canal del Dique

The  $D_{50}$  values of the bed-samples taken in the Canal del Dique are presented in Figure 3.3.16. The samples which were taken in the measuring cross-sections in the distributing branches have also been inserted.

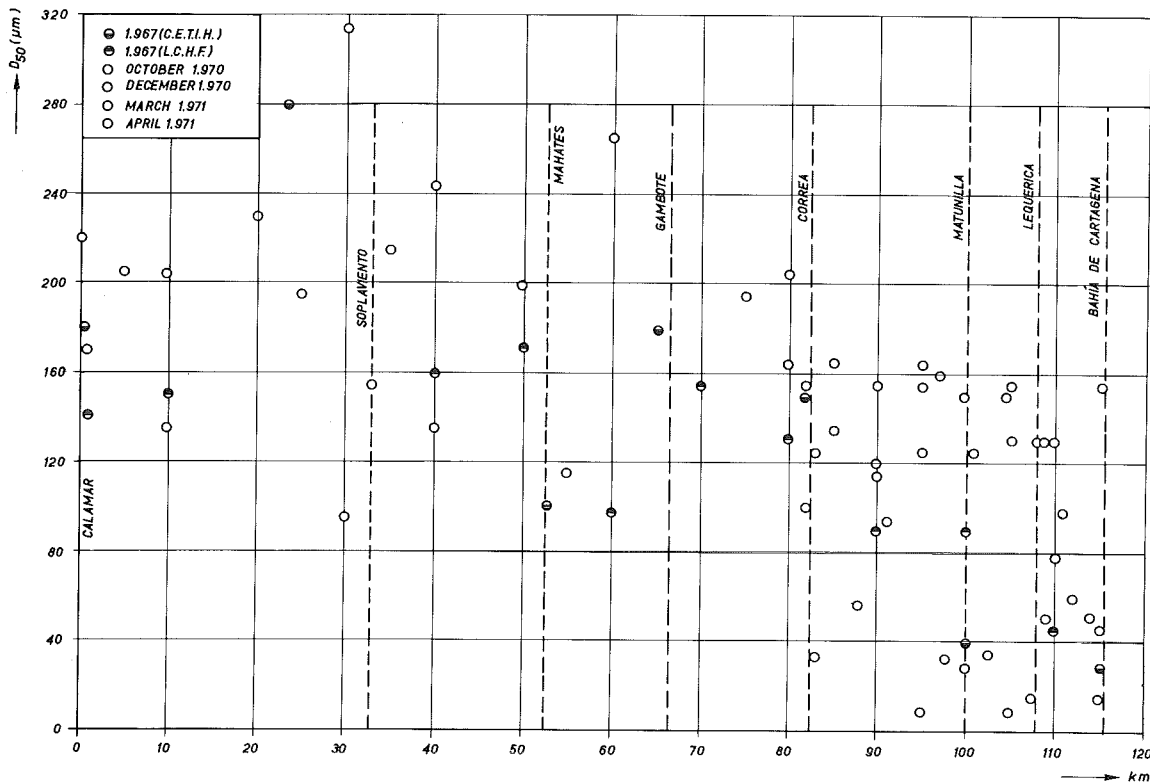
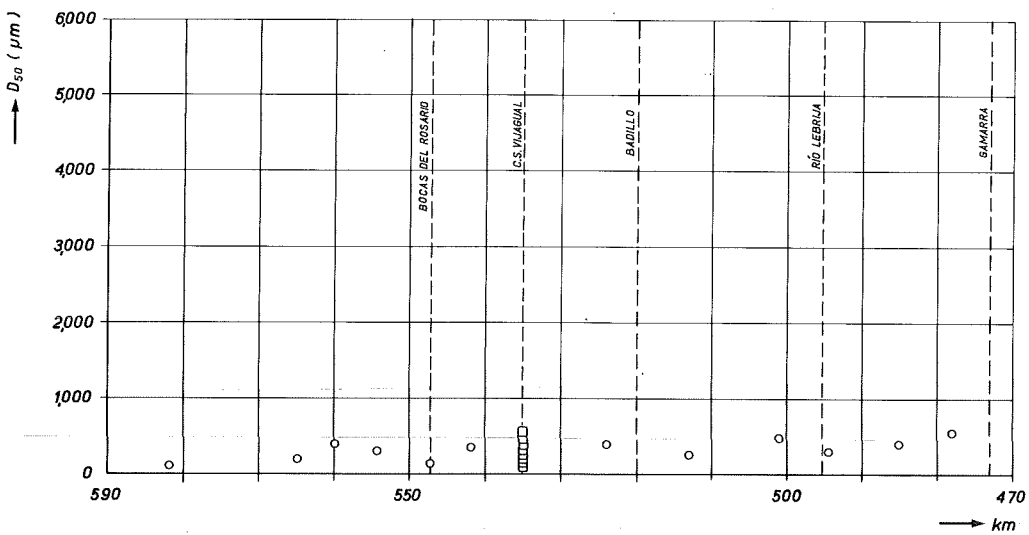
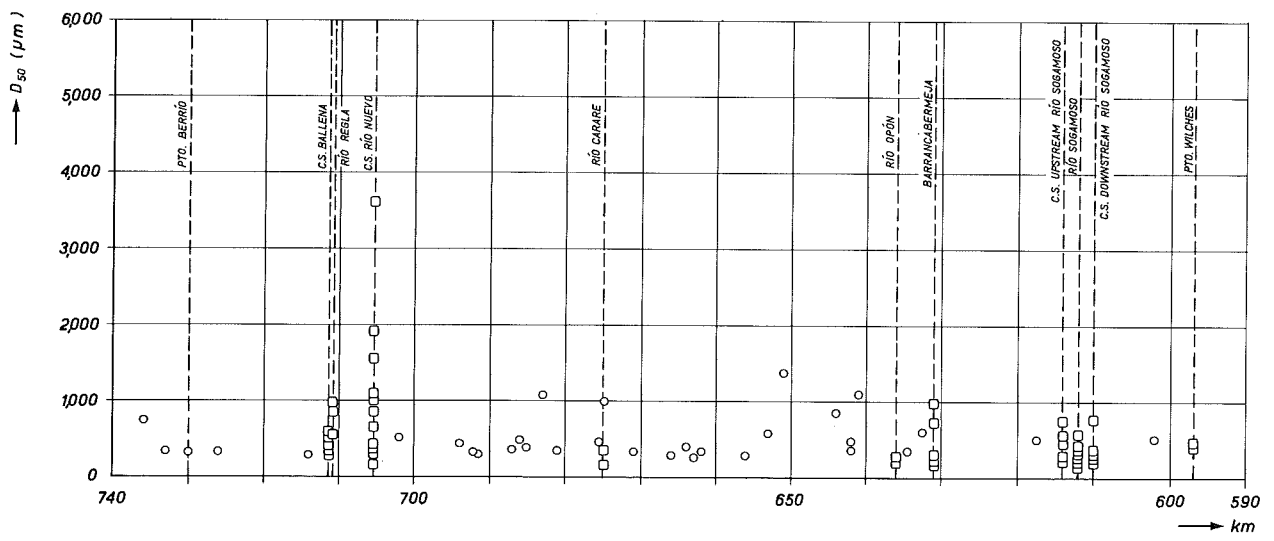
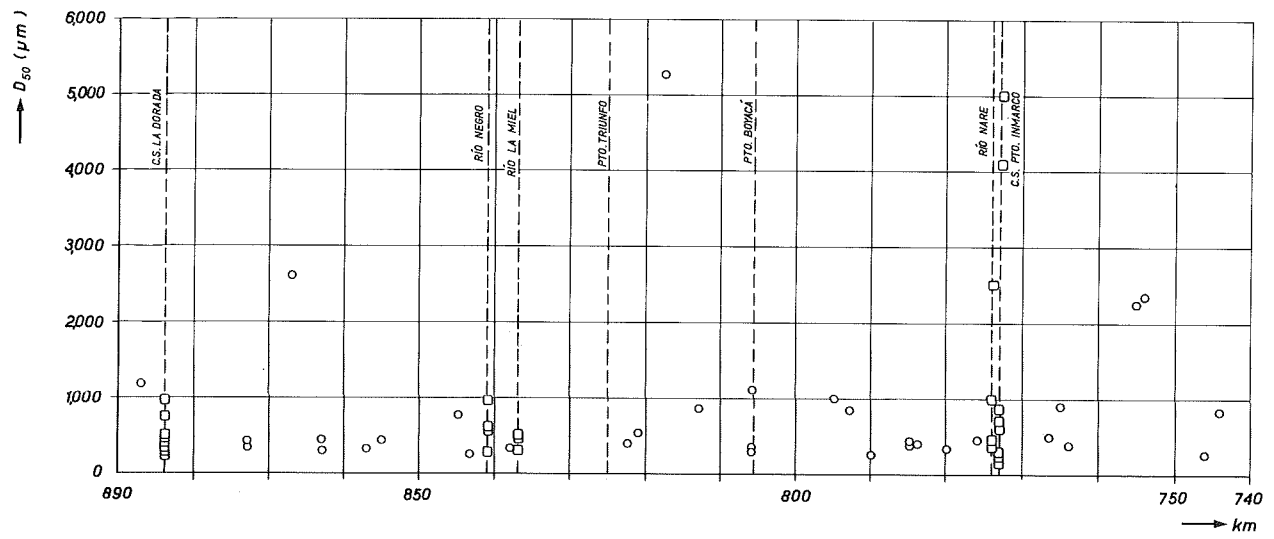


Figure 3.3.16  $D_{50}$  of Bed Material along the Canal del Dique



LEGEND  
 ○ ALONG RÍO MAGDALENA  
 □ MEASURING C.S. IN THE RÍO MAGDALENA OR AFFLUENTS

D<sub>50</sub> OF BED MATERIAL ALONG THE RÍO MAGDALENA AND IN AFFLUENTS

FIG. 3.3.15

3.3.7. Longitudinal profilesA. The Rfo Magdalena

The longitudinal soundings which have been made in the course of the study are presented in Figures 3.3.17 to 3.3.38 inclusive. In the plan form of the river the sailing routes have also been indicated. It must be stressed that the soundings do not represent everywhere the so-called "talweg", or the line connecting the deepest point in each consecutive cross-section. However, particularly in shallow areas, the deepest crossing from one channel to the other was looked for. It must also be considered that the route taken by barge-trains sometimes has to deviate from the talweg because such convoys will not always be able to follow exactly the often large sinuosity of the talweg. Therefore, it can be concluded that the soundings represent the maximum water depth available to navigation. The soundings are reduced to the L.R.L. (for the values of L.R.L. at the different gauge-stations, reference is made to Table 2.3.10).

The kilometer net indicated in the plan-form is based on the data as found during the survey of the Julius Berger Konsortium. The base of this net was established at the outlet of the Rfo Magdalena in the Caribbean at Bocas de Ceniza. The base was later moved by ADENAVI to the Barranquilla Terminal by subtraction of 16.8 kilometers from the Julius Berger data. Comparing the aerial photographs of 1972 with the maps of Julius Berger, a number of locations can be considered to be unchangeable (such as villages and old ranches). These locations have been given the kilometers as found by Julius Berger, although in compliance with ADENAVI the kilometer of the Barranquilla Terminal was considered as zero (see Table 3.3.3).

Location	Kilometer	Location	Kilometer
Pto. Salgar Bridge	886.5	Barriga	671.7
Hda. San Cayetano	866.7	Chucurí	659.6
Pto. Triunfo	824.9	Barrancabermeja	631.2
Pto. Niño	812.8	Rfo Sogamoso Confluence	612.2
Hda. El Rebozo	804.7	Pto. Wilches	597.1
Hda. Caimital	799.7	San Pablo	582.0
Hda. La Plata	792.5	Paturia	564.5
Pto. Inmarco	772.7	Bocas del Rosario	547.5
Pto. Berrfo Bridge	730.3	Vijagual	534.6
Murillo	716.2	Badillo	520.0
Rfo Nuevo	706.6	Bodega Central	494.5
Hda. Mosquitera	693.4	Gamarra	472.7
Presidio	680.7		

Table 3.3.3 Kilometers Copied from the Julius Berger Survey

In between the locations given in Table 3.3.3 the kilometers were linearly interpolated along the plan-form of 1972. This means that in river stretches which straightened since the Julius Berger survey, one kilometer is shorter than 1,000 m (e.g., between km 887 and km 866.7, 1 km = 650 m, see Figures 3.3.17 and 3.3.18), while at other stretches a stronger meandering of the river has led to one kilometer being longer than 1,000 m (e.g., between km 736 and km 730, 1 km = 1,200 m, see Figures 3.3.24 and 3.3.25). This information has also been indicated on the route maps.

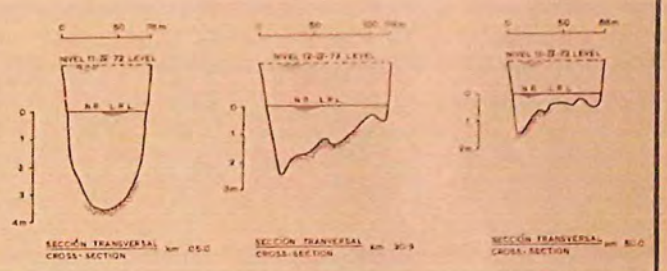
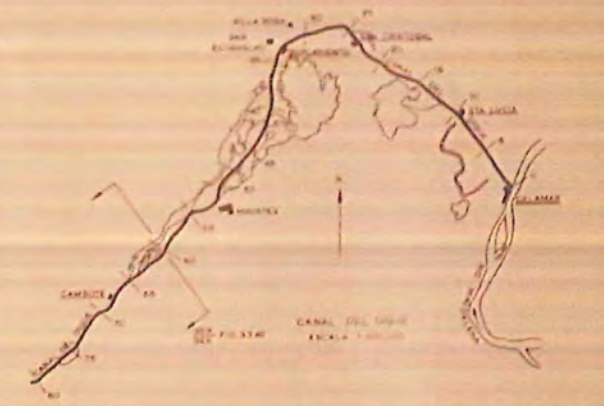
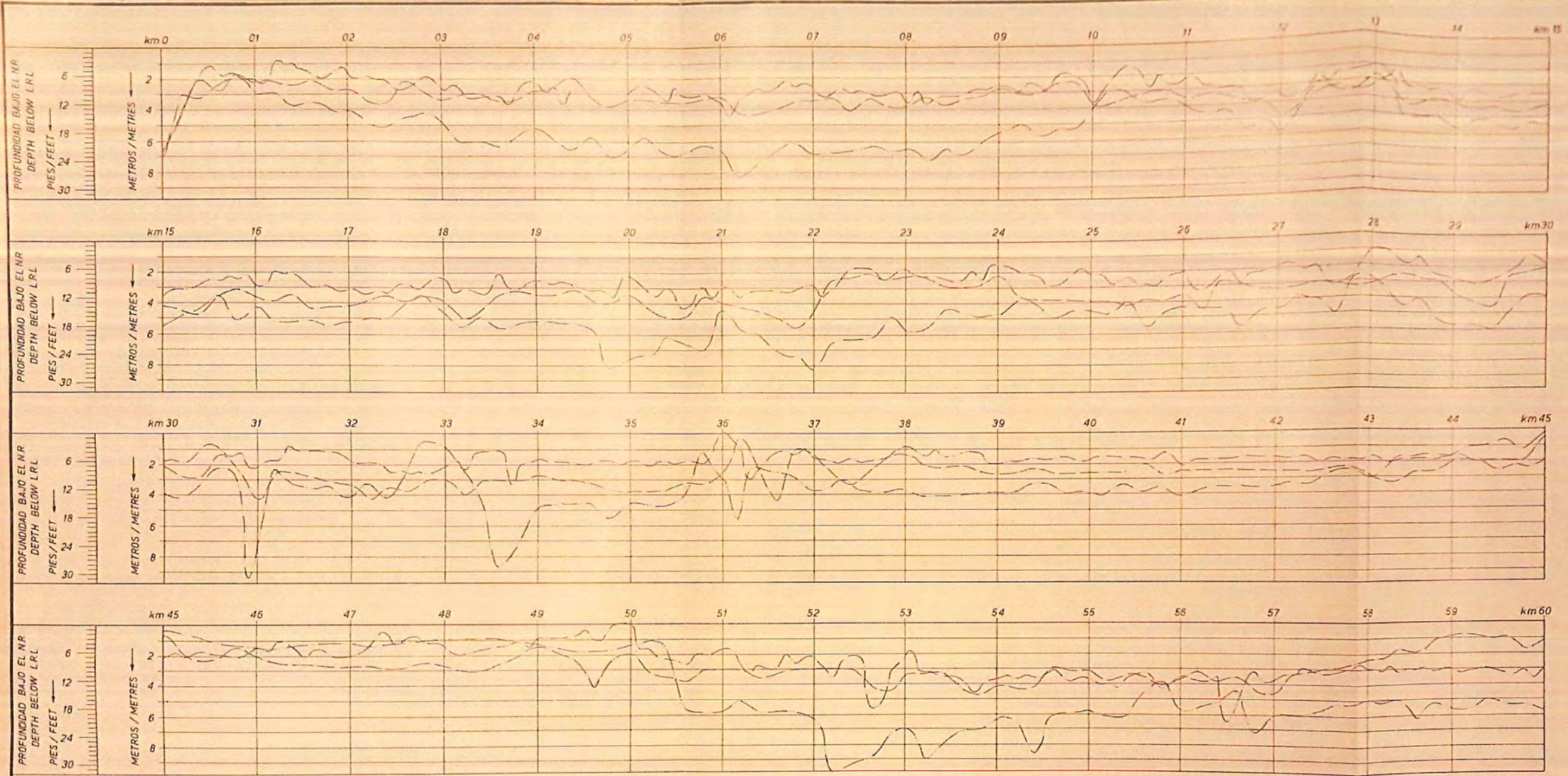
## II, 3.3

It is advised that for future soundings the same kilometer net is adopted as used by the Mission. This will not only facilitate the comparison of consecutive soundings, but also enable the exact location of "troublesome" spots to be pin-pointed.

To be able to schematize certain river stretches, cross-sections have been taken at regular intervals. Such schematizations are required in the morphological computations which will be treated in Part III of this Report. The cross-sections are also inserted in the pertaining maps.

### B. The Canal del Dique

The longitudinal soundings of the Canal del Dique are given in Figures 3.3.39 and 3.3.40. The sailing routes have been omitted in view of the limited width of the Canal. Some cross-sections are also given.



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	
ANCHO WIDTH	
CURVA CURVE	
OBSERVACIONES REMARKS	N B km 0 - 60 : 1 km = 1,000 m

LEYENDA LEGEND	FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL (OR REDUCTION LEVEL)							
		CALAMAR	STA. LUCÍA	SOPLAVIENTO	GAMBOTE	CORREA	MATUNILLA	LEQUERICA	BAHÍA DE CARTAGENA
---	13,25-I - 1.973	. m	. m	. m	. m	. m	. m	. m	. m
---	11-13-IV - 1.972	1.65 m	1.71 m	1.53 m	0.89 m	. m	0.08 m	0.20 m	0.18 m
---	17,20-I - 1.972	3.12 m	2.85 m	. m	1.91 m	. m	0.53 m	0.26 m	. m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

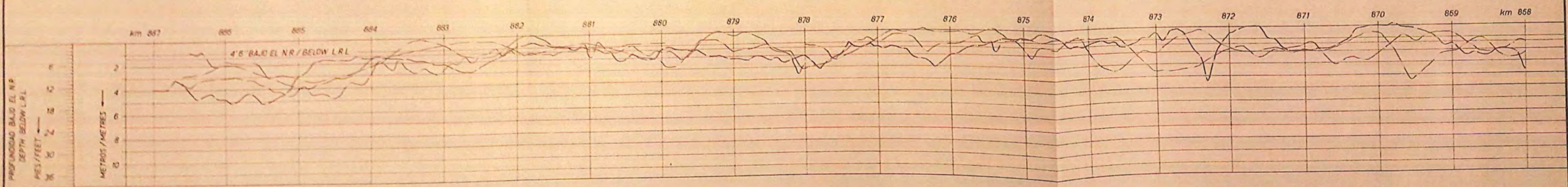
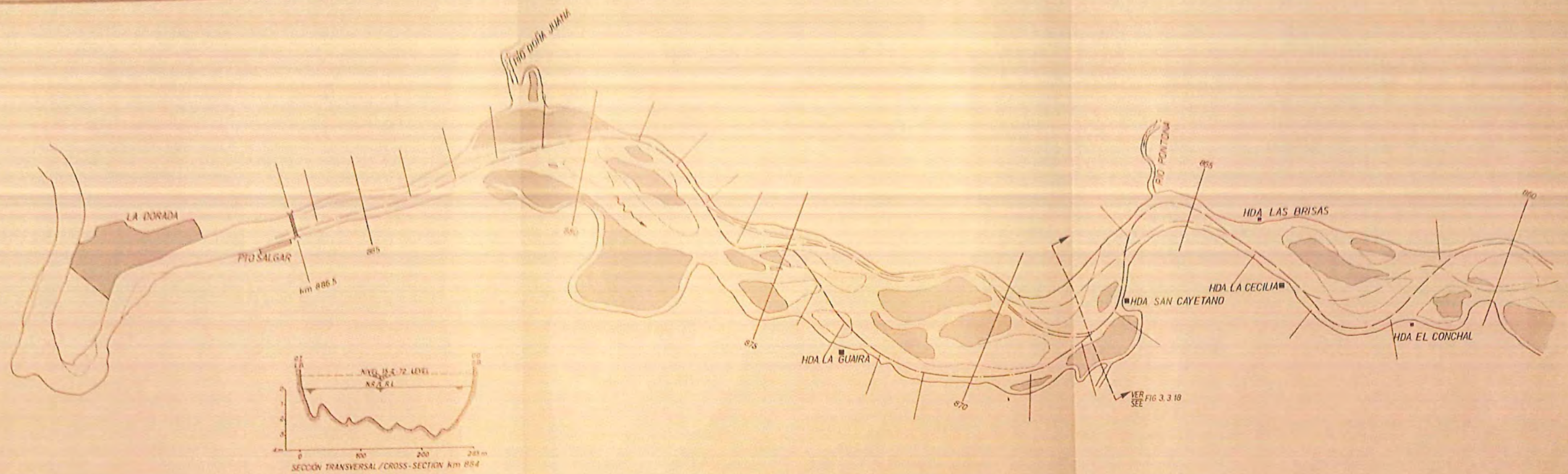
### CANAL DEL DIQUE km 0 - km 60

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.) OR REDUCTION LEVEL

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DE ADENAVI  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF ADENAVI

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.3.39



DIMENSIONES LIMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	15-II-1972 0.30 m <u>BAJO EL N.R.</u> BELOW L.R.L. km 877-878
ANCHO WIDTH	
CURVA CURVE	
OBSERVACIONES REMARKS	N 8 km 887 - km 887 1 km + 650 m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
} EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÓ	BARRANCABERMEJA	PTO WILCHES	GAMARRA
1-IX-1972	1.15 m	3.25 m	m	m	m	m
21-X-1972	2.92 m	4.35 m	2.56 m	m	m	m
15-II-1972	0.86 m	0.55 m	0.60 m	1.26 m	m	m
19-XI-1971	1.64 m	2.15 m	1.60 m	2.01 m	2.70 m	m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 887 - km 868

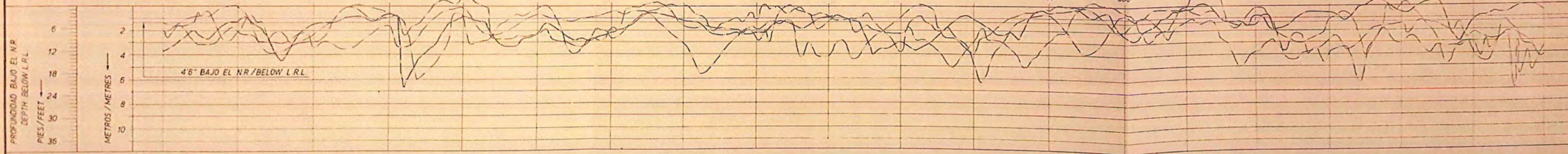
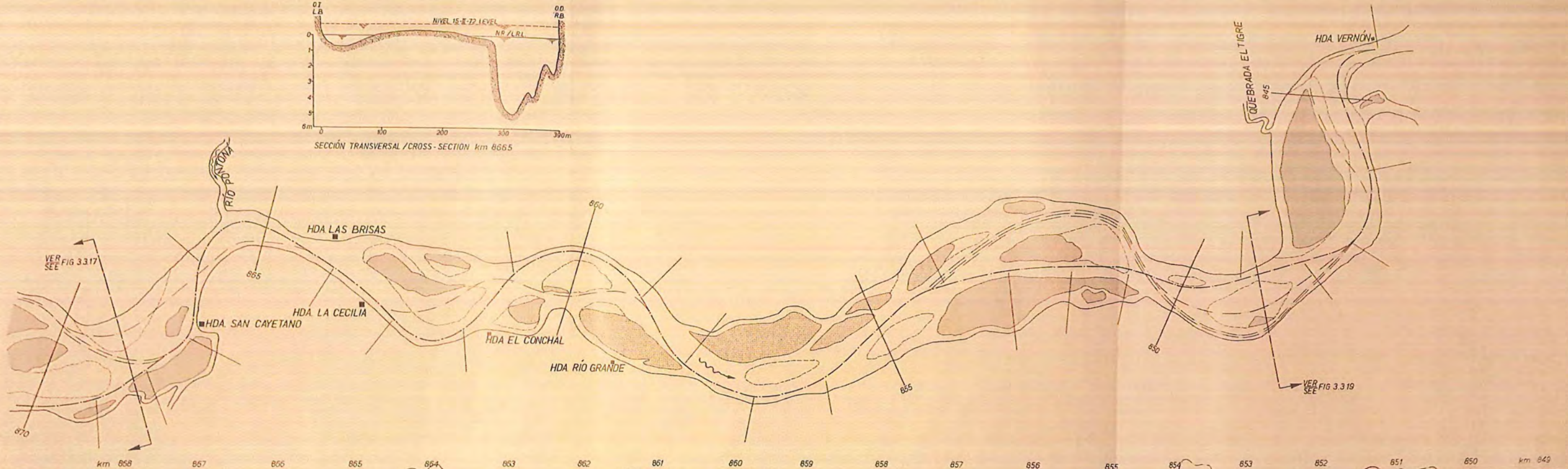
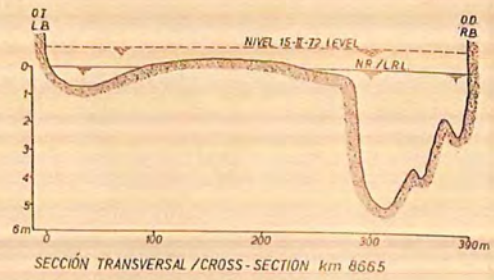
ESCALA / SCALE 1:40 000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.3.17



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	0.10 m <sup>SOBRE EL NR</sup> ABOVE L.R.L. km 859.5
ANCHO WIDTH	
CURVA CURVE	
OBSERVACIONES REMARKS	NB km 868 - km 867 1km-700m km 867 - km 849 1km-1000m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	— — — — —
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	- - - - -
} EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES	GAMARRA
1 - IX - 1972	1.15 m	3.25 m	m	m	m	m
21 - X - 1972	2.92 m	4.35 m	2.56 m	m	m	m
15 - II - 1972	0.86 m	0.55 m	0.60 m	1.26 m	m	m
19 - XI - 1971	1.64 m	2.15 m	1.60 m	2.01 m	2.70 m	m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

**RÍO MAGDALENA km 868 - km 849**

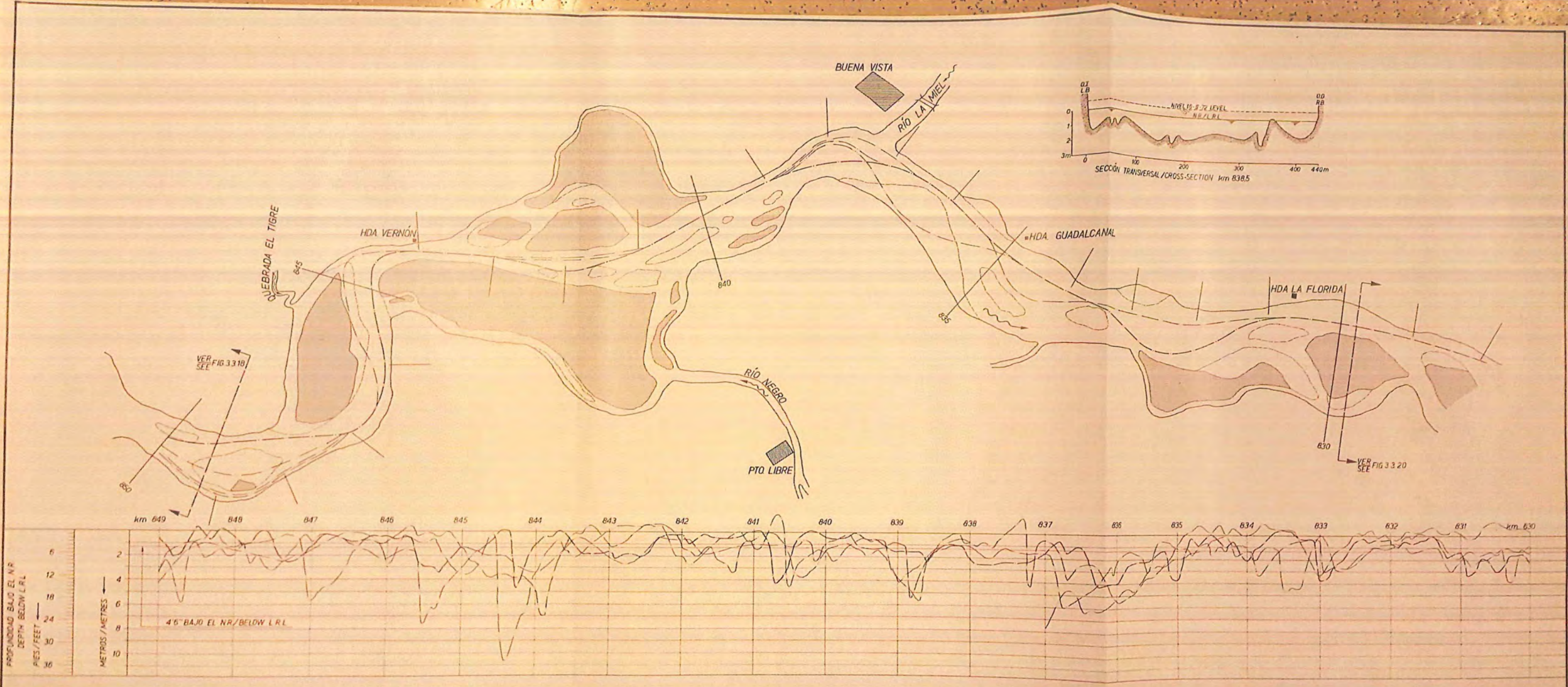
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (NR)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3.318



PROFUNDIDAD	DIMENSIONES LÍMITES
DEPTH	LIMITING DIMENSIONS
ANCHO	0.20 m BAJO EL NR
WIDTH	BELOW L.R.L.
CURVA	km 839.5
CURVE	
OBSERVACIONES	
REMARKS	N.B. km 849 - km 830 1km-1000m

LEYENDA	
LEGEND	
RUTA / SONDEO / PRINCIPAL	-----
MAIN / COURSE / SOUNDING	
RUTA / SONDEO / INICIAL	-----
INITIAL / COURSE / SOUNDING	
} EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL	
} IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN					
	WATER-LEVEL ABOVE LOW RIVER LEVEL					
DATE	PTO SALGAR	PTO INMARCO	PTO BERRÓ	BARRANCABERMEJA	PTO WILCHES	GAMARRA
1-IX-1972	1.15 m	3.25 m	m	m	m	m
21-V-1972	2.92 m	4.35 m	2.56 m	m	m	m
15-II-1972	0.80 m	0.55 m	0.50 m	1.06 m	m	m
13-II-1971	1.56 m	2.06 m	1.62 m	2.26 m	2.75 m	m

SONDEO LONGITUDINAL **RÍO MAGDALENA** km 849-km 830  
 LONGITUDINAL SOUNDING

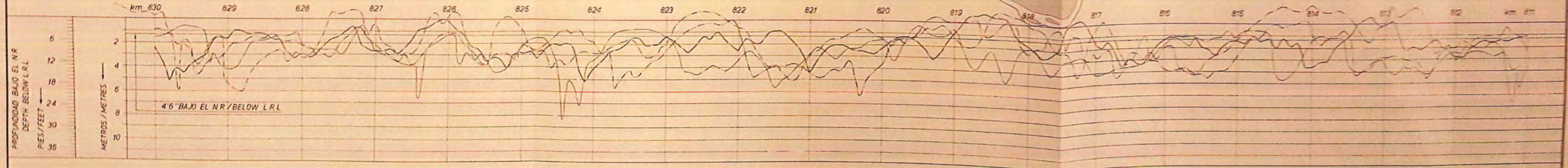
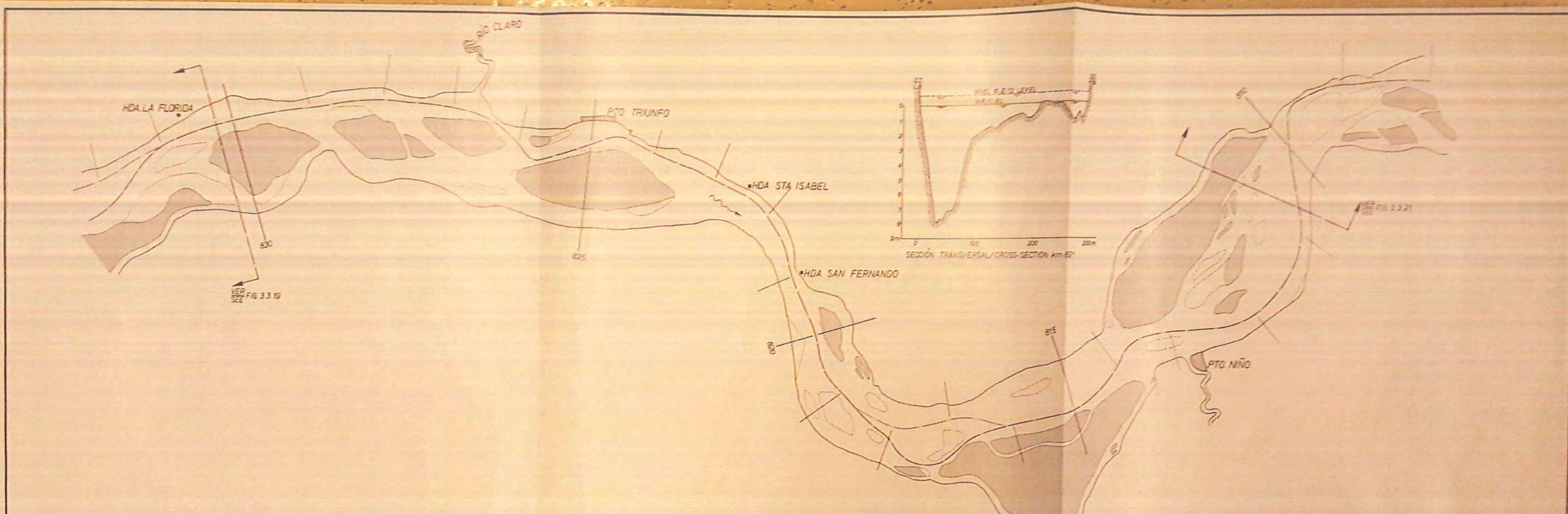
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
 DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
 KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3.3.19



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	0.35 m BAJO EL NR BELOW L.R.L.
ANCHO WIDTH	km 819.5
CURVA CURVE	
OBSERVACIONES REMARKS	N.B. km 830 - km 825 1 km = 1000 m km 825 - km 811 1 km = 800 m km 811 - km 811 1 km = 1200 m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÓ	BARRANCABERMEJA	PTO WILCHES	GAMARRA
1-II-1972	1.15 m	3.25 m	m	m	m	m
21-V-1972	2.92 m	4.35 m	2.56 m	m	m	m
16-II-1972	0.77 m	0.55 m	0.40 m	0.85 m	m	m
20-XI-1971	1.48 m	1.98 m	1.65 m	2.51 m	2.80 m	m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 830-km 811

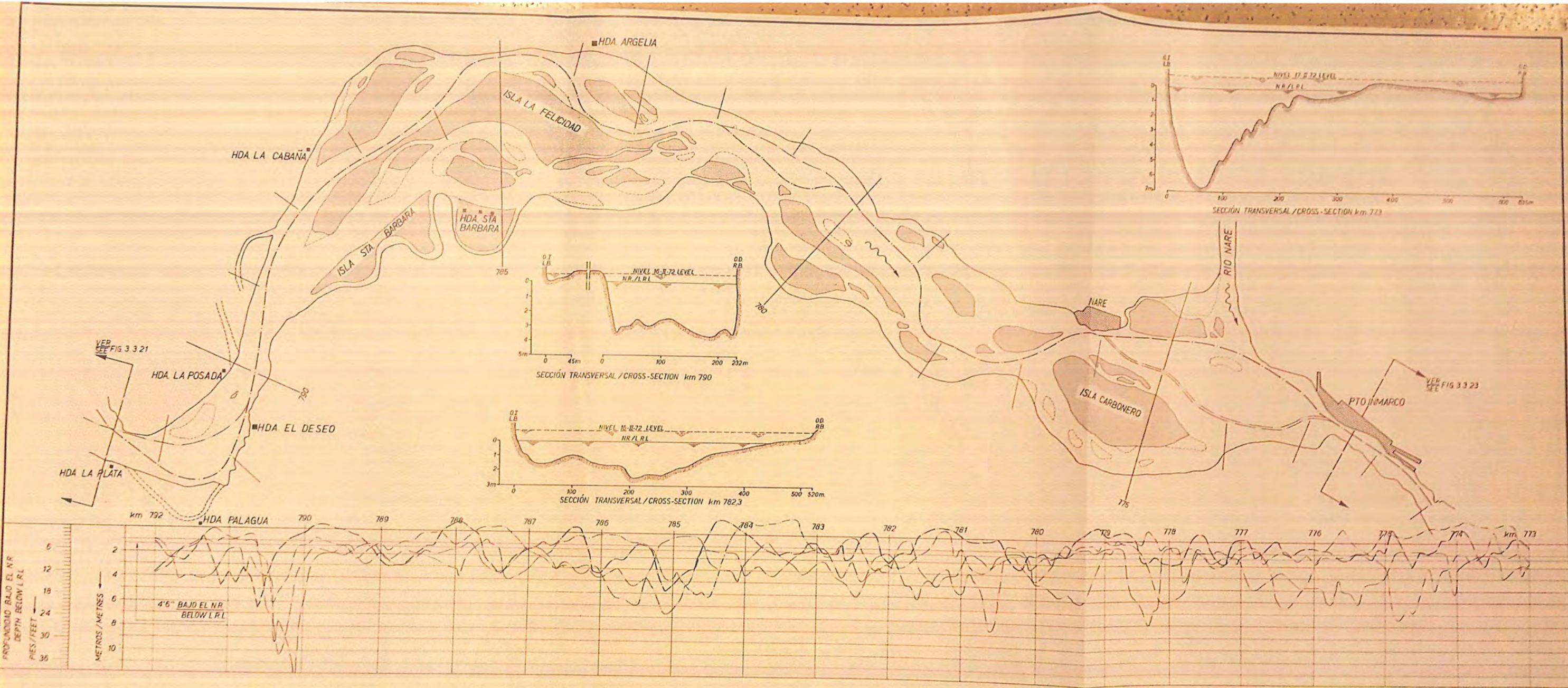
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJA-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO (BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM)  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3.3.20



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	15-II-1972 0.50 m BAJO EL N.R. BELOW L.R.L. km 779
ANCHO WIDTH	
CURVA CURVE	
OBSERVACIONES REMARKS	N.B. km 792-km 773 1 km = 1160 m

LEYENDA LEGEND	
PIUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	— — — — —
PIUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	- - - - -
EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL				
	PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES
2-IX-1972	1.15 m	3.25 m	0.95 m	m	m
22-X-1972	2.92 m	4.50 m	2.53 m	3.25 m	m
17-II-1972	0.72 m	0.70 m	0.32 m	0.78 m	m
21-XI-1971	1.48 m	1.98 m	1.65 m	2.51 m	2.80 m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 792 - km 773

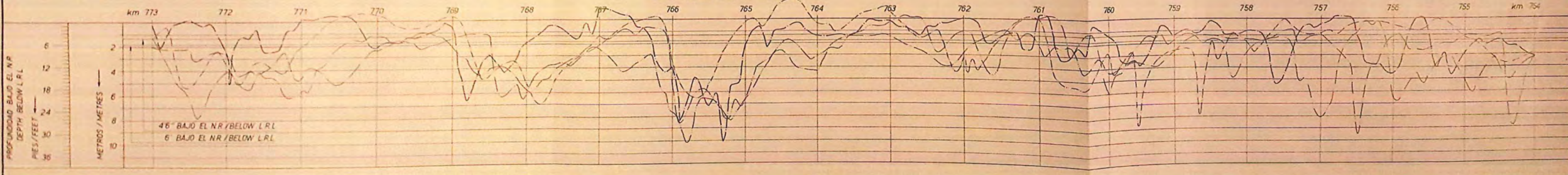
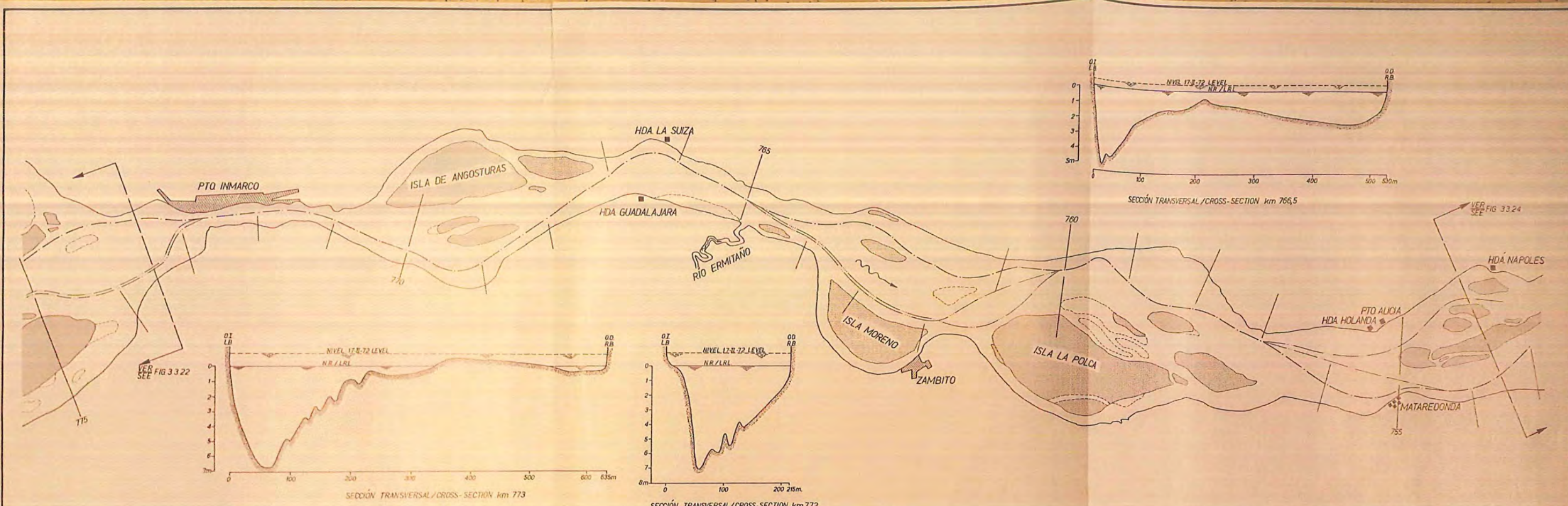
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3 3 22



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	0.90 m BAJO EL NR BELOW LRL km 7615, km 7575
ANCHO WIDTH	
CURVA CURVE	
OBSERVACIONES REMARKS	N.B. km 773 - km 754 1 km = 1040 m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER - LEVEL ABOVE LOW RIVER LEVEL					
	PTO. SALGAR	PTO. INMARCO	PTO. BERRÍO	BARRANCABERMEJA	PTO. WILCHES	GAMARRA
2-IX-1972	1.15 m	3.25 m	0.95 m	. m	. m	. m
22-V-1972	2.92 m	4.65 m	2.50 m	3.25 m	. m	. m
17-II-1972	0.67 m	0.85 m	0.25 m	0.71 m	. m	. m
23-X-1971	1.27 m	2.25 m	1.80 m	2.96 m	3.00 m	. m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 773-km 754

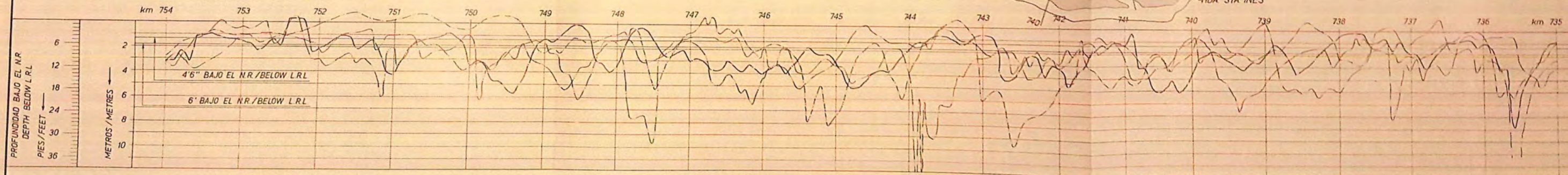
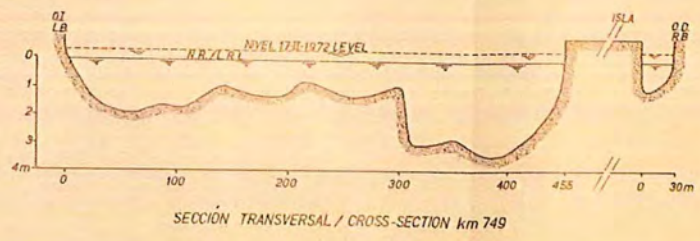
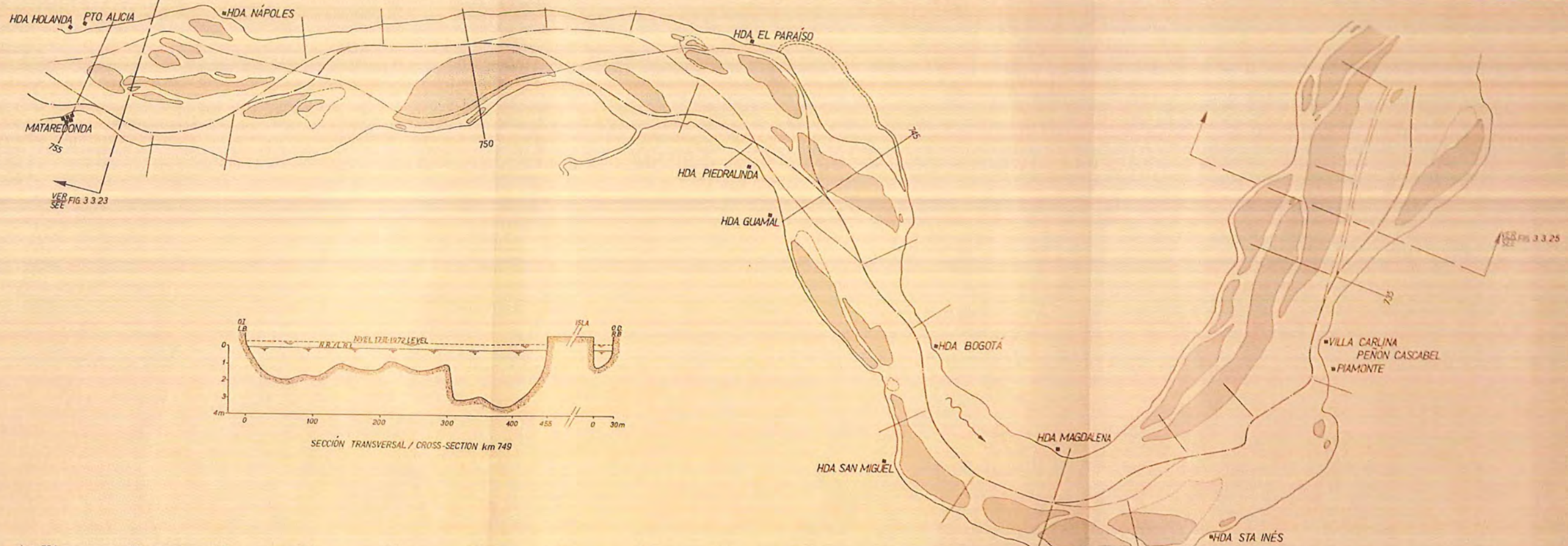
ESCALA / SCALE 1:40 000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (NR)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (LRL)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3.3.23



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	17-II-1972. 0.25 m BAJO EL N.R. BELOW L.R.L. km 736
ANCHO WIDTH	
CURVA CURVE	
OBSERVACIONES REMARKS	NB km 754- km 736. 1km • 1040 m km 736- km 735. 1km • 1200 m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
} EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES	GAMARRA
2-IX-1972	1.15 m	3.25 m	0.95 m	m	m	m
22-V-1972	2.92 m	4.65 m	2.50 m	3.25 m	m	m
17-II-1972	0.67 m	0.85 m	0.25 m	0.71 m	m	m
23-X-1972	1.27 m	2.25 m	1.80 m	2.96 m	3.00 m	m

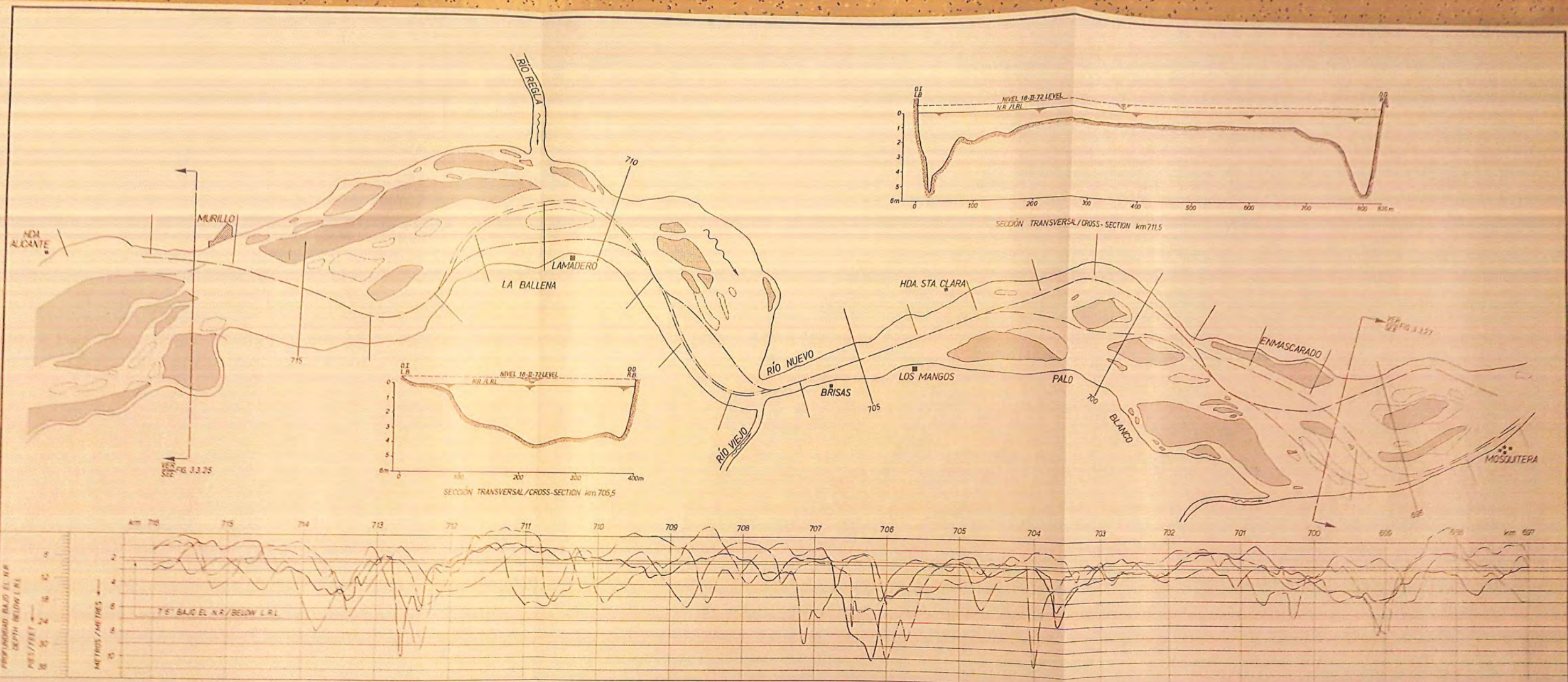
SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING **RÍO MAGDALENA km 754-km 735**

ESCALA / SCALE 1:40 000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT **FIG.3.3.24**



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	16'-II-1972 0.45 m BAJO EL N.R. BELOW L.R.L.
ANCHO WIDTH	km 709
CURVA CURVE	
DESIGNACIONES MARKERS	N.R. km 716-720 3km+1000 m km 708-697 3km+300 m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
	EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES	GAMARRA
3-II-1972	m	m	0.70 m	1.46 m	m	m
22-I-1972	2.92 m	4.65 m	2.50 m	3.25 m	m	m
19-II-1972	0.65 m	m	0.47 m	0.71 m	m	m
29-II-1971	1.77 m	m	1.50 m	1.71 m	2.05 m	m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDINGS

**RÍO MAGDALENA** km 716 - km 697

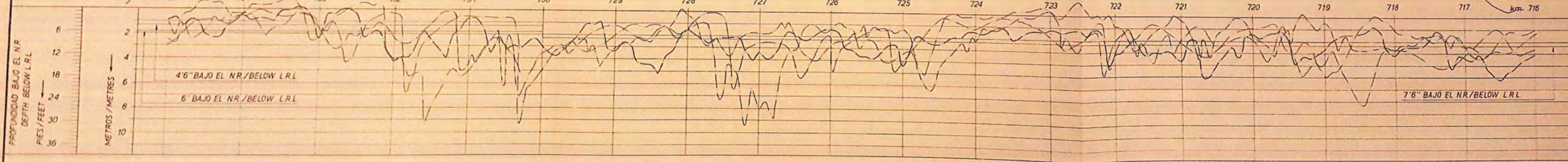
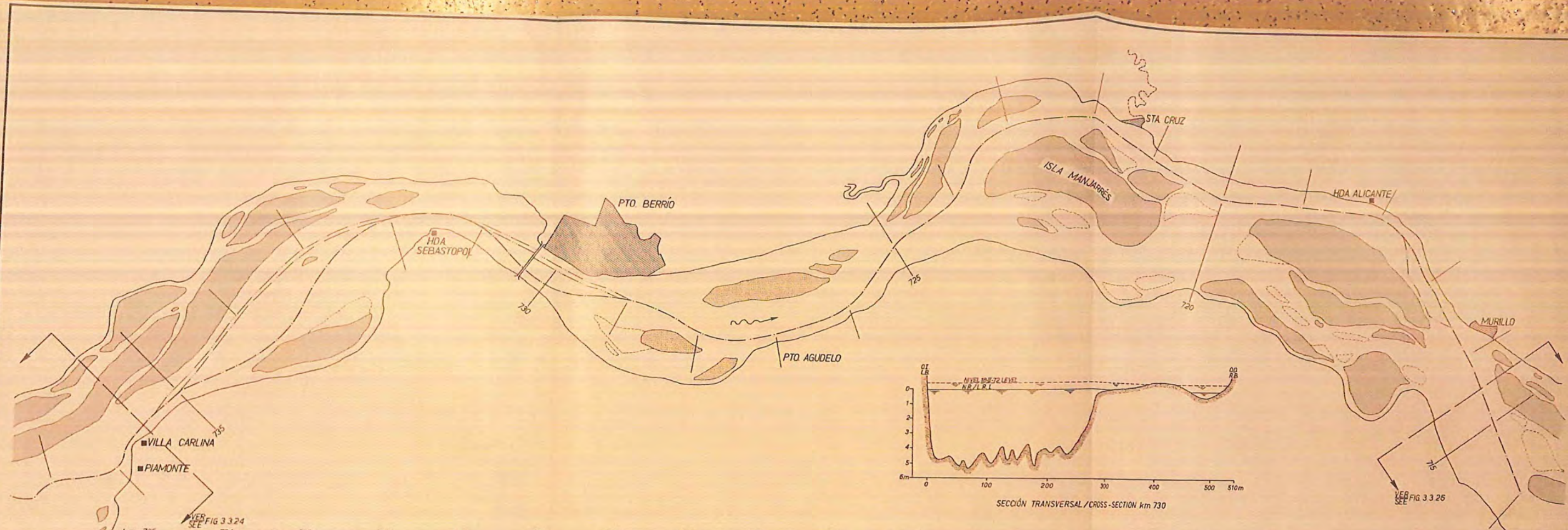
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3.3.26



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	17-II-1972. 0.10 m BAJO EL N.R. BELOW L.R.L. km 733.8
ANCHO WIDTH	
CURVA CURVE	
OBSERVACIONES REMARKS	N.B. km 735 - km 730. 1 km = 1200 m km 730 - km 716. 1 km = 1100 m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES	GAMARRA
2/3-IX-1972	1.15 m	3.25 m	0.77 m	1.46 m	m	m
22-V-1972	2.92 m	4.65 m	2.50 m	3.25 m	m	m
17/18-II-1972	0.60 m	0.85 m	0.40 m	0.60 m	m	m
21/22-IX-1971	1.50 m	2.25 m	1.65 m	2.30 m	2.50 m	m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 735 - km 716

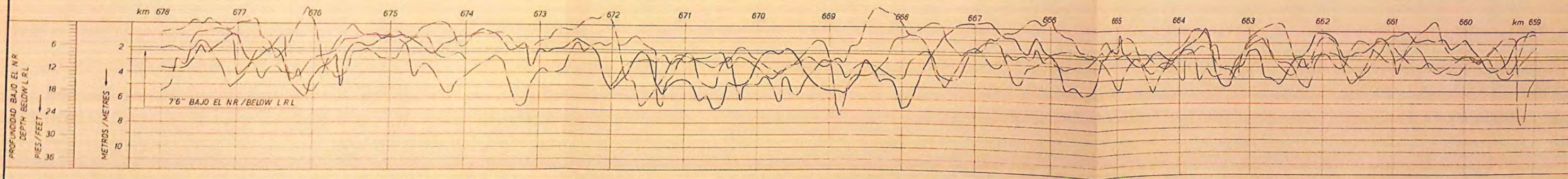
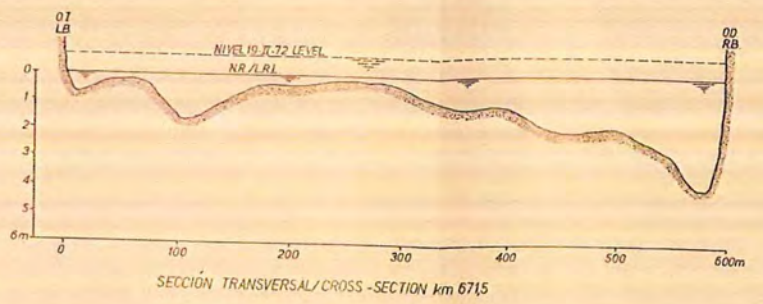
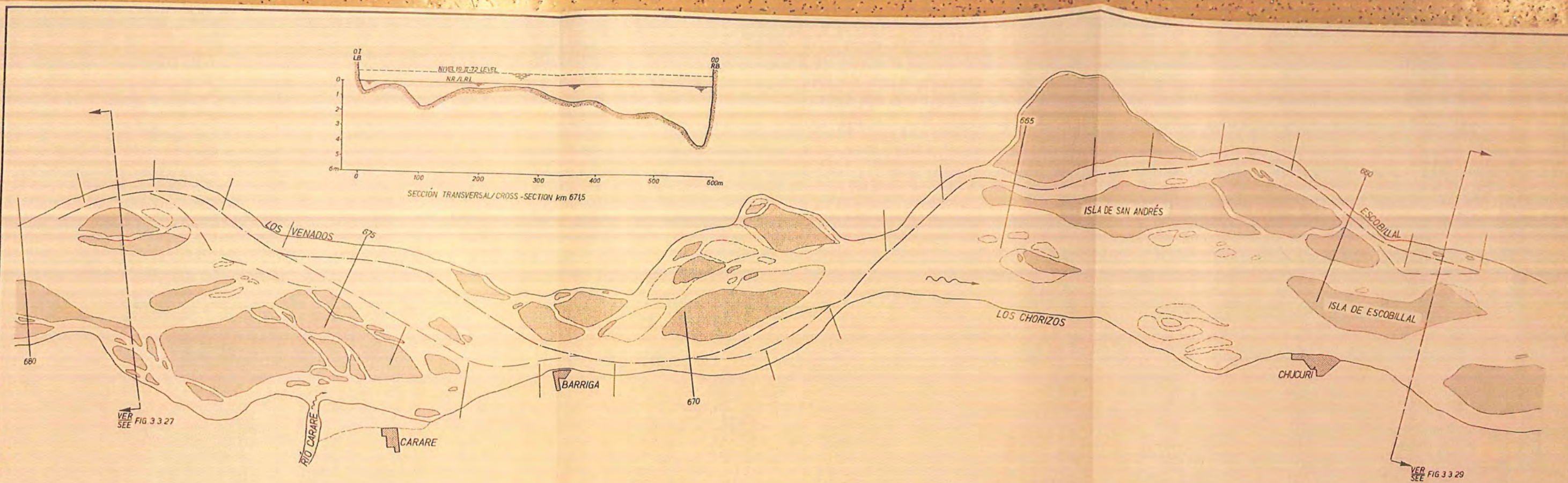
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 33.25



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	19-II-1972 0.40 m BAJO EL N.R. BELOW L.R.L.
ANCHO WIDTH	km 677.2
CURVA CURVE	
OBSERVACIONES REMARKS	N B km 678 - km 659 1km=1000m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES	GAMARRA
3-IX-1972	m	m	0.70 m	1.46 m	m	m
22-V-1972	2.92 m	4.65 m	2.50 m	3.25 m	m	m
19-II-1972	0.87 m	m	0.40 m	0.91 m	m	m
29-II/ 30-XII-1971	1.67 m	m	1.52 m	1.93 m	2.05 m	m

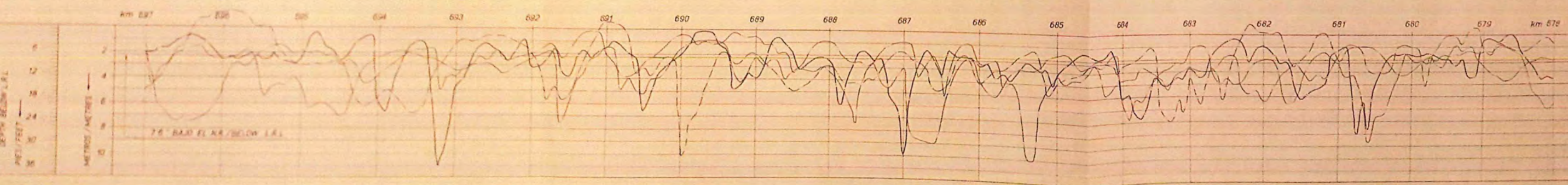
SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING **RÍO MAGDALENA km 678-km 659**

ESCALA / SCALE 1:40,000

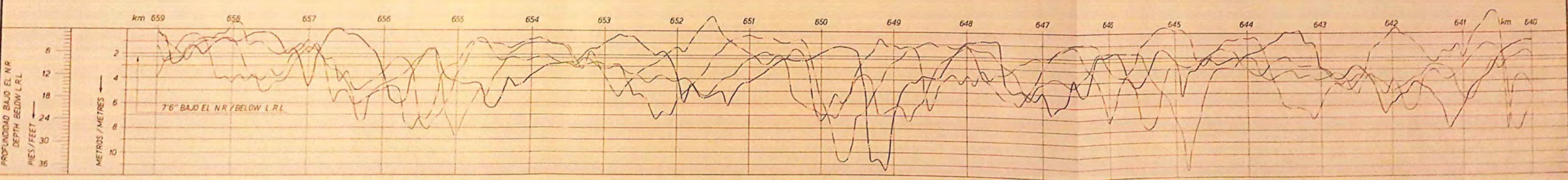
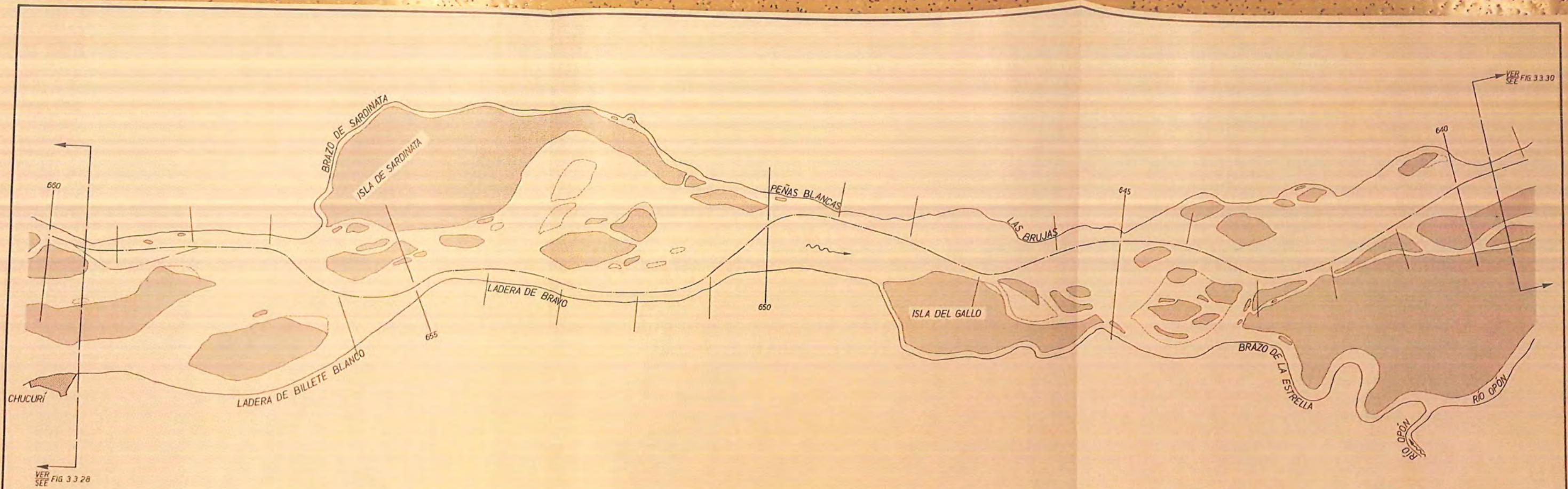
PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT **FIG 3.3.28**



<b>DIMENSIONES LÍMITES</b> LIMITING DIMENSIONS PROFUNDIDAD / DEPTH 19-IX-1972 0.80 m / 2.63 ft. (MÁS BAJO / BELOW L.R.L.) ANCHO / WIDTH km 690, km 687 CURVA / CURVE OBSERVACIONES / REMARKS N.B. km 687, km 683, km 680, km 678 km 682, km 681, km 680, km 679, km 678		<b>LEYENDA</b> LEGEND RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING RUTA / SONDEO / SECUNDARIA INITIAL / COURSE / SOUNDING EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING		<b>FECHA</b> DATE 3-IX-1972 22-X-1972 19-II-1972 29-XI-1971		<b>NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN</b> WATER-LEVEL ABOVE LOW RIVER LEVEL PTO SALGAR PTO INMARCO PTO BERRÓ BARRANCABERMEJA PTO WILCHES GAMARRA m m 0.70 m 1.46 m m m 2.92 m 4.65 m 2.50 m 3.25 m m m 0.87 m m 0.40 m 0.91 m m m 1.77 m m 1.50 m 1.71 m 2.05 m m						<b>SONDEO LONGITUDINAL</b> LONGITUDINAL SOUNDING <b>RÍO MAGDALENA km 697-km 678</b> ESCALA / SCALE 1:40,000 PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.) DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.) KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM	
---	--	---	--	--	--	--	--	--	--	--	--	---	--



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	19-II-1972 0.30 m BAJO EL N.R. BELOW L.R.L.
ANCHO WIDTH	km 658.5
CURVA CURVE	
OBSERVACIONES REMARKS	N.B. km 659-km 640 1km-1000m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÓ	BARRANCABERMEJA	PTO WILCHES	GAMARRA
3-IX-1972	m	m	0.70 m	1.46 m	m	m
22-V-1972	2.92 m	4.65 m	2.50 m	3.25 m	m	m
19-II-1972	0.87 m	m	0.40 m	0.91 m	m	m
30-XII-197	1.57 m	m	1.55 m	2.15 m	2.05 m	m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 659-km 640

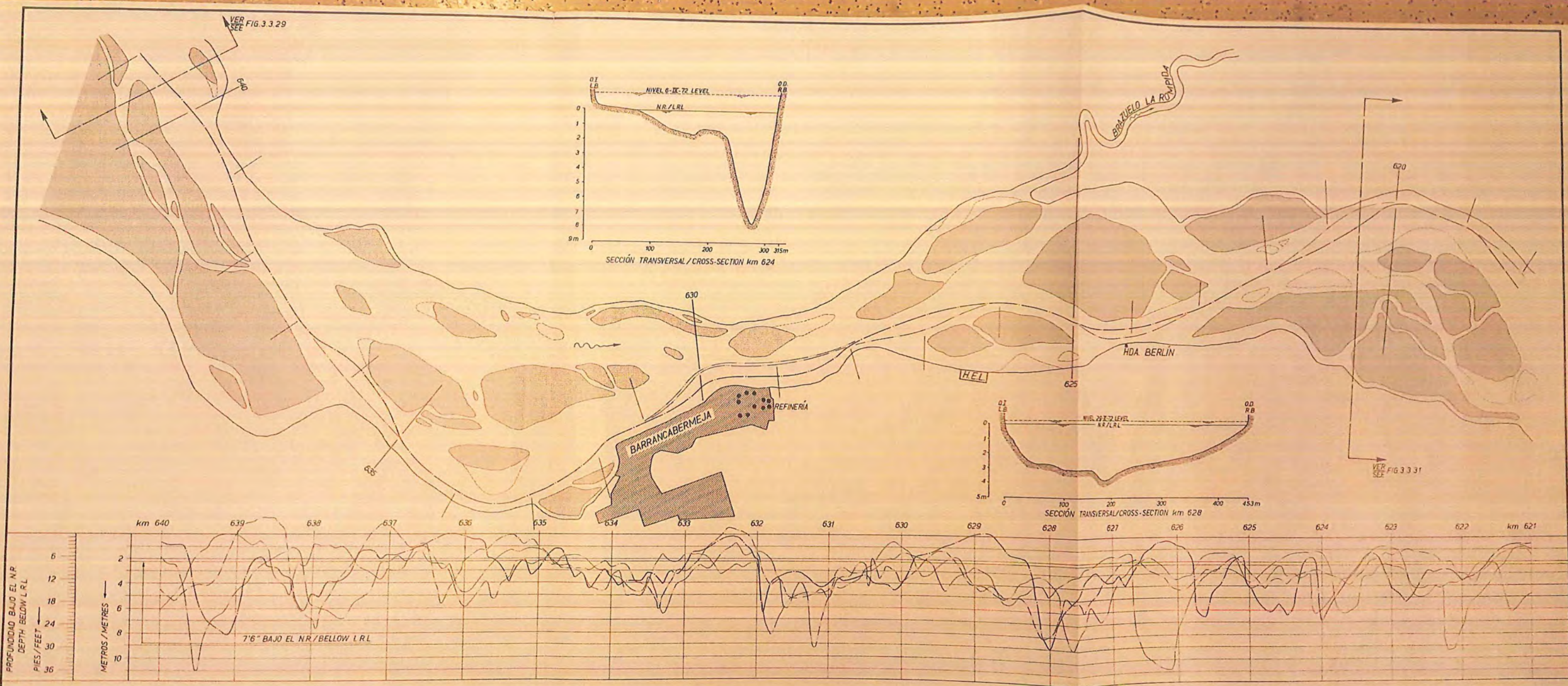
ESCALA / SCALE 1:40000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3.3.29



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	19-II-1972 070m BAJO EL NR BELOW LRL
ANCHO WIDTH	km 632.2
CURVA CURVE	
OBSERVACIONES REMARKS	N B km 640-km 621 1km=1000m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	—————
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
} EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER - LEVEL ABOVE LOW RIVER LEVEL					
	PTO. SALGAR	PTO. INMARCO	PTO. BERRÍO	BARRANCABERMEJA	PTO. WILCHES	GAMARRA
3-IX/ 6-IX-1972	m	m	0.70 m	1.26 m	1.43 m	m
22-X/ 7-XI-1972	1.90 m	m	1.46 m	2.25 m	2.00 m	4.28 m
19-IX/ 29-II-1972	0.57 m	m	0.43 m	0.57 m	0.40 m	m
30-XII/ 1-XI-1971	1.52 m	m	1.53 m	1.98 m	2.00 m	2.53 m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 640-km 621

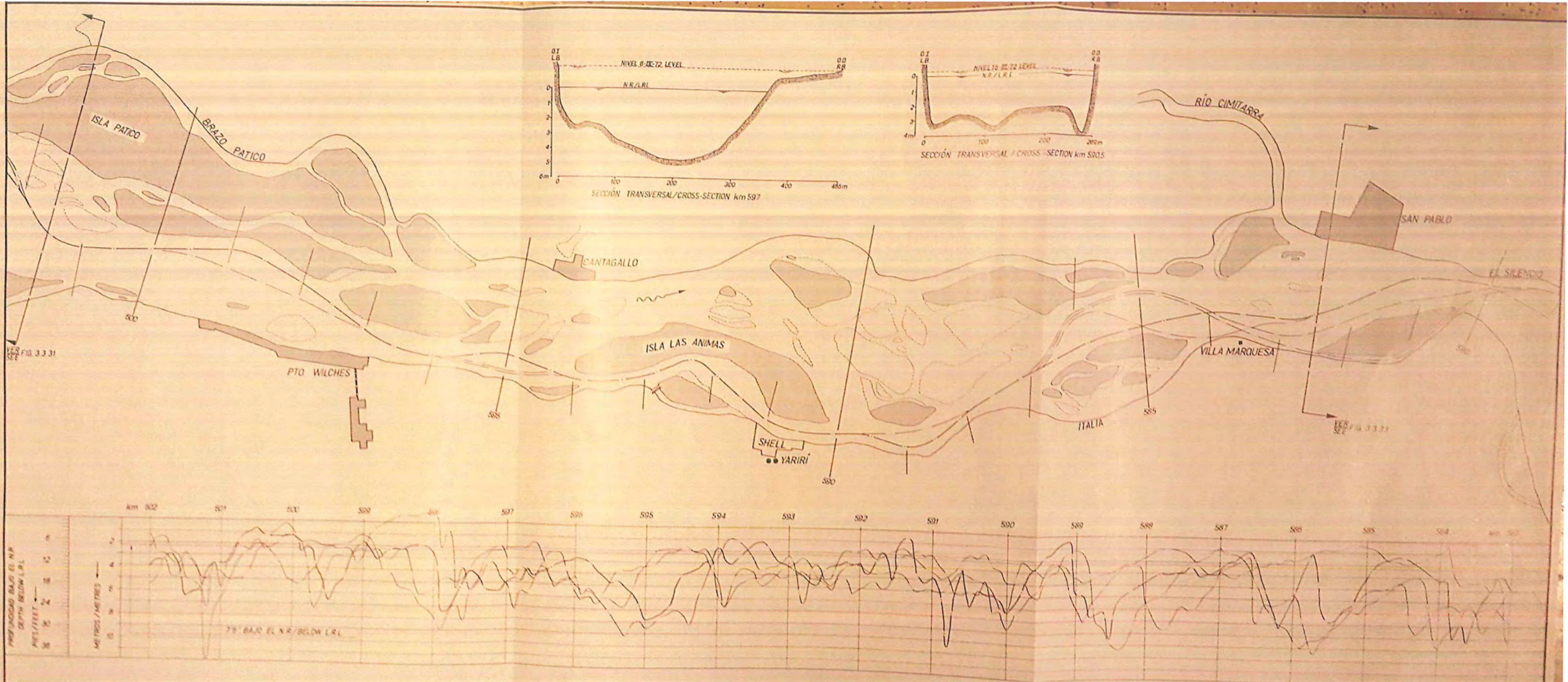
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.3.30



DIMENSIONES LÍMITES	
PROFUNDIDAD	1.20 m
ANCHO	km 602
LIMITS DIMENSIONS	
DEPTH	1.20 m
WIDTH	km 602
OBSERVACIONES	
REMARKS	
N.B. km 602 - km 583 1:40,000	

LEYENDA	
LEGEND	
RUTA / SONDEO / PRINCIPAL	MAN / COURSE / SOUNDING
RUTA / SONDEO / SECUNDARIA	MAN / COURSE / SOUNDING
EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL	
IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN					
	WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO. SALGAR	PTO. INMARCO	PTO. BERRÓ	BARRANCABERMEJA	PTO. WILCHES	GAMARRA
6-IX-1972	m	m	m	1.07 m	1.43 m	m
7-XI-1972	m	m	0.42 m	1.31 m	2.00 m	4.28 m
16-III-1972	m	m	0.24 m	0.29 m	0.32 m	0.56 m
1-XI-1971	m	m	1.51 m	1.81 m	2.00 m	2.53 m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 602-km 583

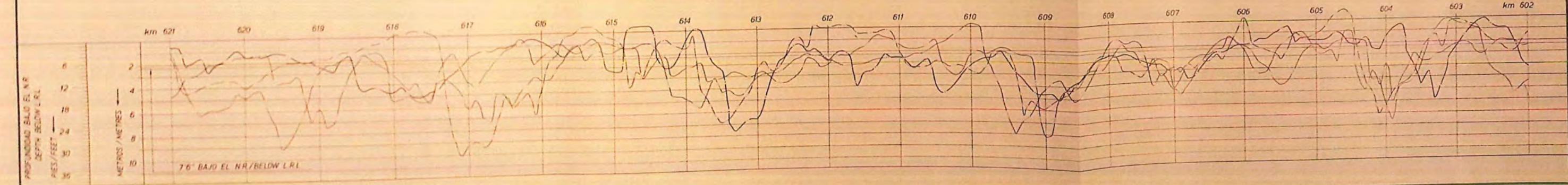
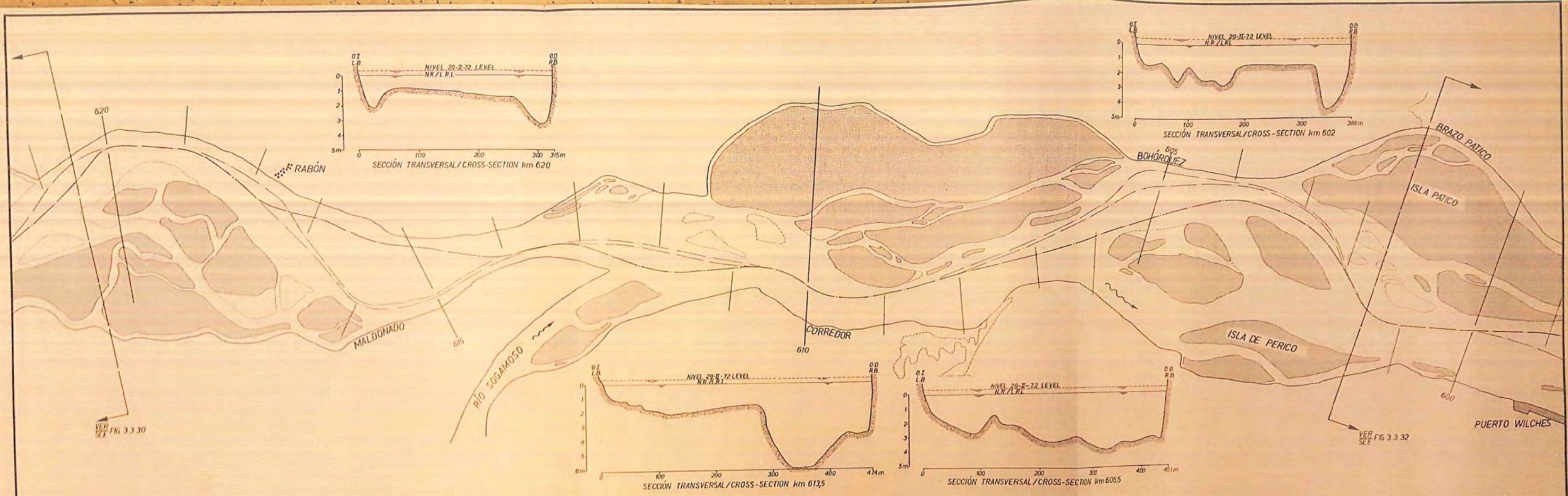
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3332



DIMENSIONES LÍMITES LIMITING DIMENSIONS	BAJO EL NR BELOW LRL
PROFUNDIDAD DEPTH	29-II-1972 0.60 m
ANCHO WIDTH	km 611.3
CURVA CURVE	
OBSERVACIONES REMARKS	N B km 621 - km 602 1km x 1040 m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	-----
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	-----
EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÓ	BARRANCABERMEJA	PTO WILCHES	GAMARRA
6-IX-1972	m	m	m	1.07 m	1.43 m	m
7-VI-1972	m	m	0.42 m	1.31 m	2.00 m	4.28 m
29-II-1972	m	m	0.46 m	0.24 m	0.40 m	m
1-XII-1971	m	m	1.51 m	1.61 m	2.00 m	2.53 m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

**RÍO MAGDALENA km 621 - km 602**

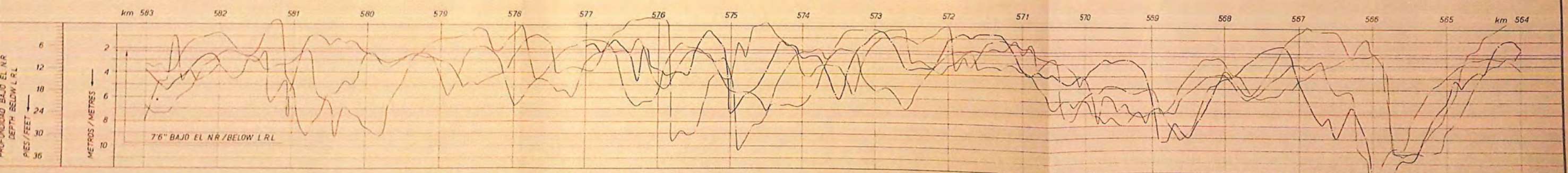
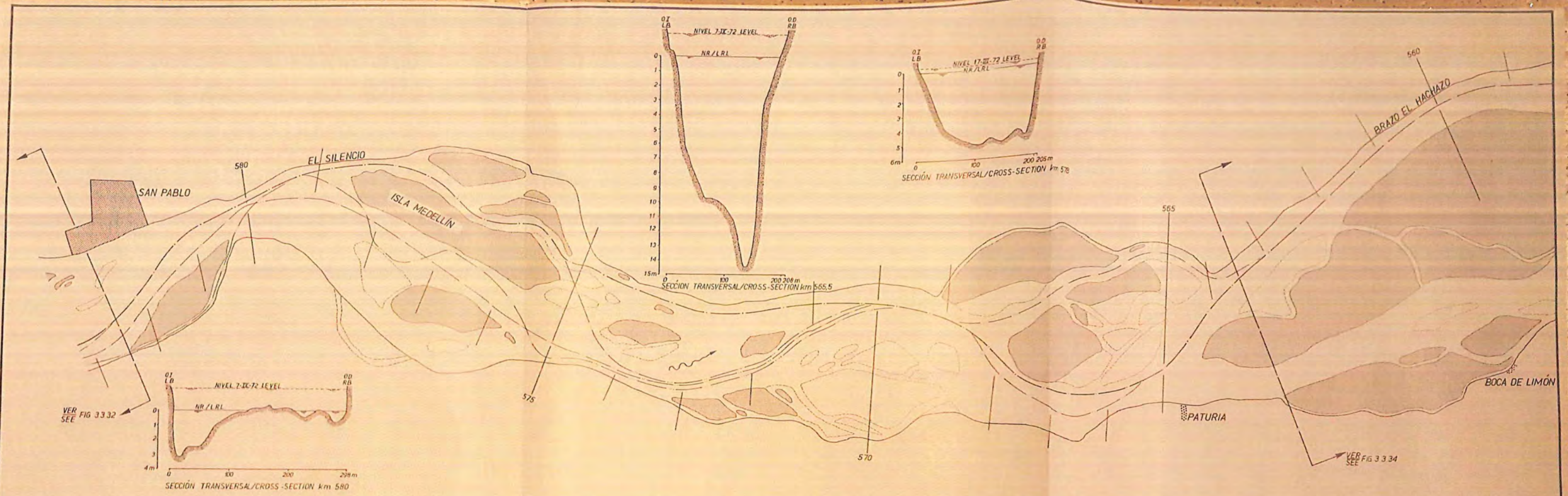
ESCALA / SCALE 1:40 000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (NR)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (LRL)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.3.31



DIMENSIONES LÍMITES	
LIMITING DIMENSIONS	
PROFUNDIDAD	BAJO EL N.R.
DEPTH	BELOW L.R.L.
ANCHO	km 527, km 571
WIDTH	
CURVA	
CURVE	
OBSERVACIONES	N B km 583 - km 546 1km=1000 m
REMARKS	

LEYENDA	
LEGEND	
RUTA / SONDEO / PRINCIPAL	-----
MAIN / COURSE / SOUNDING	
RUTA / SONDEO / INICIAL	-----
INITIAL / COURSE / SOUNDING	
EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL	
IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA		NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN					
DATE		WATER - LEVEL ABOVE LOW RIVER LEVEL					
		PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES	GAMARRA
7-IX-1972		m	m	m	m	1.43 m	190 m
7-XI-1972		m	m	0.42 m	1.31 m	2.00 m	4.28 m
17-III-1972		m	m	0.00 m	0.21 m	0.25 m	0.56 m
1-XII-1971		m	m	1.51 m	1.81 m	2.00 m	2.53 m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

**RÍO MAGDALENA** km 583 - km 564

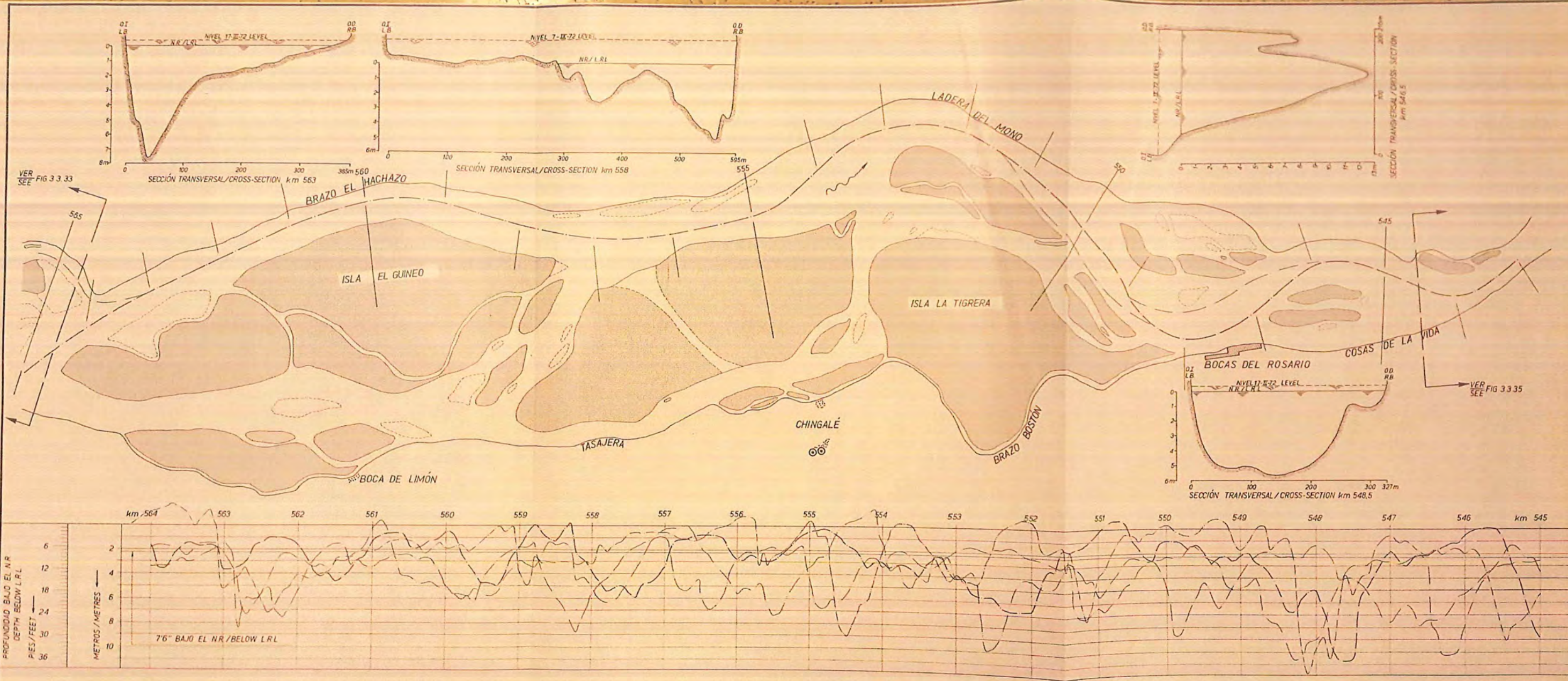
ESCALA / SCALE 1:40000

PROFUNDIDAD BAJO - NIVEL DE AGUA SOBRE - EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW - WATER - LEVEL ABOVE - LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT

FIG 3.3.33



DIMENSIONES LÍMITES LIMITING DIMENSIONS	0.90 m BAJO EL NR BELOW LRL
PROFUNDIDAD DEPTH	17-III-1972
ANCHO WIDTH	km 557
CURVA CURVE	
OBSERVACIONES REMARKS	N.B. km 564-km 548 1km=1050m km 548-km 545 1km=1000m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	---
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	---
EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
7-IX-1972	PTO SALGAR	PTO INMARCO	PTO BERRÓ	BARRANCABERMEJA	PTO WILCHES	GAMARRA
7-XI-1972	m	m	0.42 m	1.31 m	2.00 m	1.90 m
17-III-1972	m	m	0.00 m	0.21 m	0.25 m	4.28 m
1-XII-1971	m	m	1.51 m	1.81 m	2.00 m	0.56 m
						2.53 m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 564-km 545

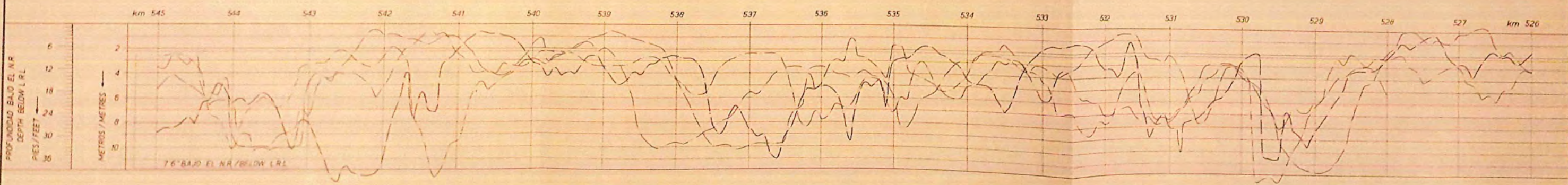
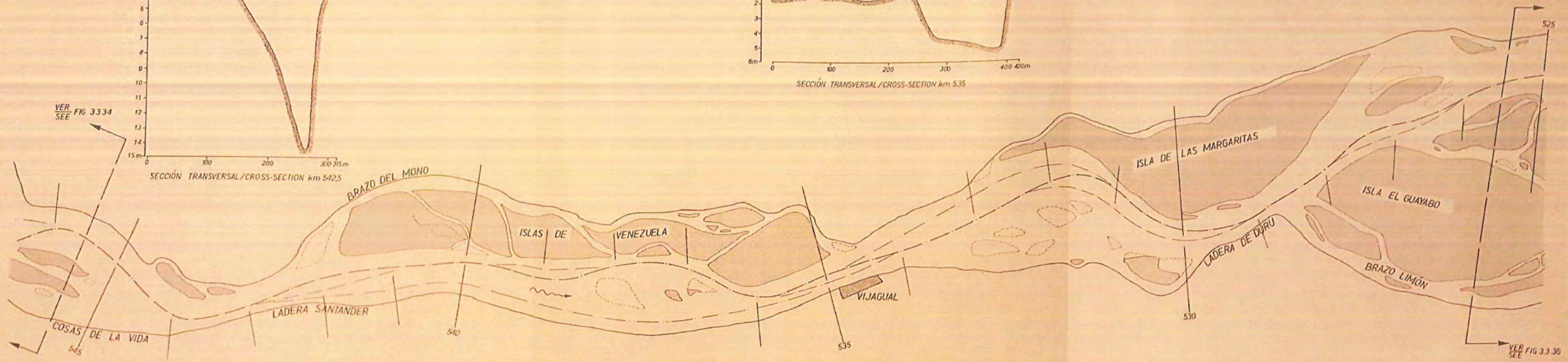
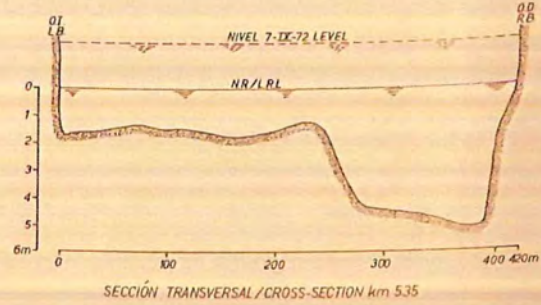
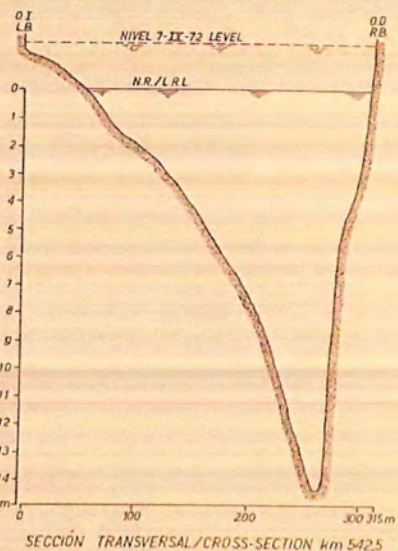
ESCALA / SCALE 1:40000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (NR)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (LRL)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.3.34



	DIMENSIONES LIMITES	LIMITING DIMENSIONS
PROFUNDIDAD	17-III-1972	1.70 m
ANCHO		km 542
CURVA		
OBSERVACIONES	N.B. km 545 - km 535 1km-1000m	
REMARKS	km 535 - km 526 1km-1080m	

	LEYENDA	LEGEND
RUTA / SONDEO / PRINCIPAL	---	MAIN / COURSE / SOUNDING
RUTA / SONDEO / INICIAL	---	INITIAL / COURSE / SOUNDING
	EN CASO NO EXISTENTES VER RUTA - SONDEO - PRINCIPAL	
	IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING	

	FECHA	DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN					
			WATER-LEVEL ABOVE LOW RIVER LEVEL					
	7-IX-1972		PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES	GAMARRA
	7-XI-1972		m	m	0.42 m	1.31 m	200 m	1.43 m
	17-III-1972		m	m	0.00 m	0.21 m	0.25 m	4.28 m
	1/2-XI-1971		m	m	1.35 m	1.81 m	1.87 m	0.56 m
								251 m

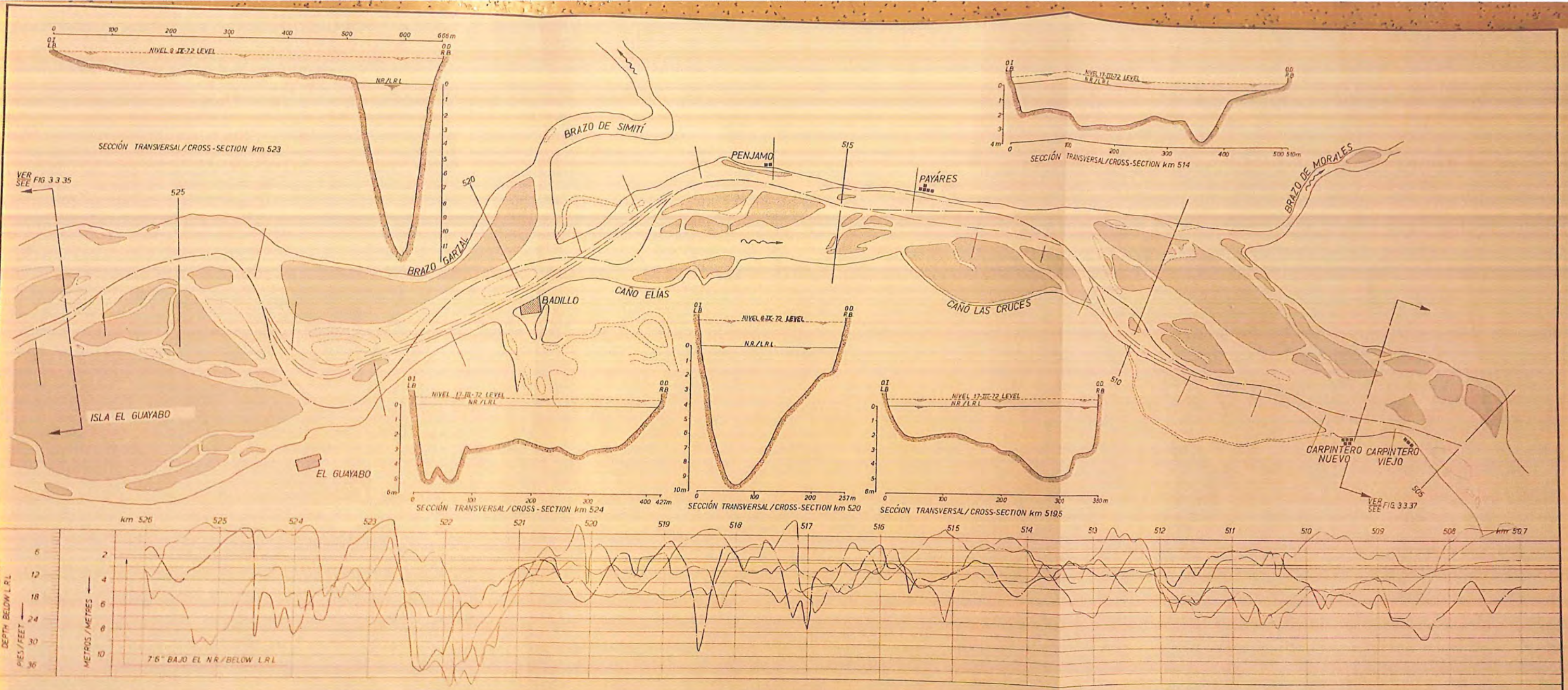
SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING **RÍO MAGDALENA km 545-km 526**

ESCALA/SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (NR)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT **FIG 3335**



DIMENSIONES LÍMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	17-III-1972 150 m BAJO EL N.R. BELOW L.R.L.
ANCHO WIDTH	km 507, 513, 519
CURVA CURVE	
OBSERVACIONES REMARKS	N.B. km 526 - km 520 1 km - 1080 m km 520 - km 507 1 km - 1000 m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	—
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	---
EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRÓ	BARRANCABERMEJA	PTO WILCHES	GAMARRA
7-IX-1972	m	m	m	m	1.43 m	1.90 m
7-XI-1972	m	m	0.42 m	1.31 m	2.00 m	4.28 m
17-III-1972	m	m	0.00 m	0.21 m	0.25 m	0.56 m
2-XII-1971	m	m	1.20 m	1.81 m	1.75 m	2.51 m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 526-km 507

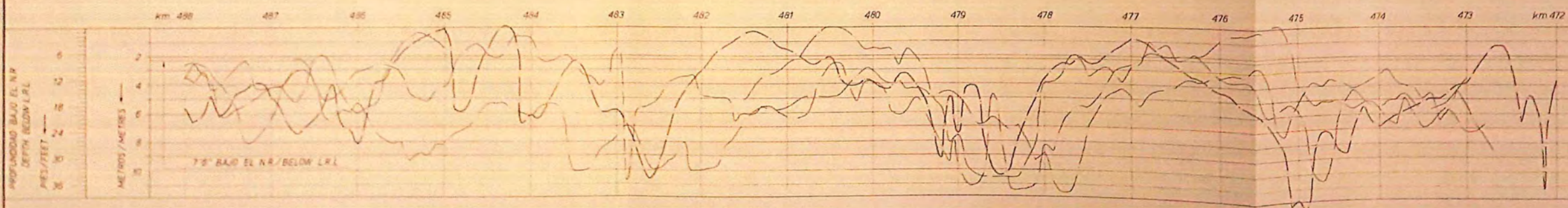
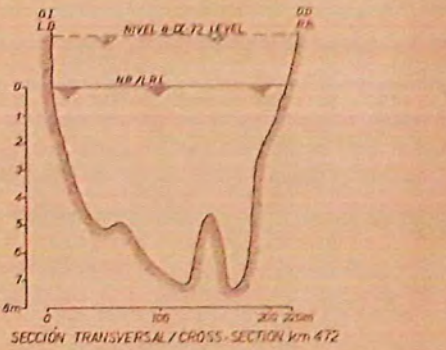
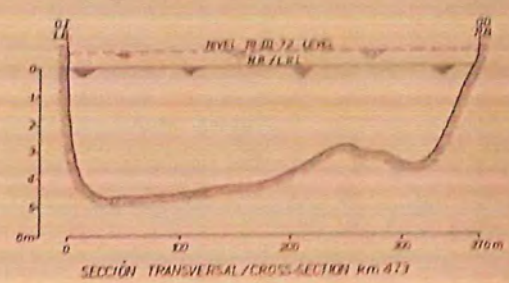
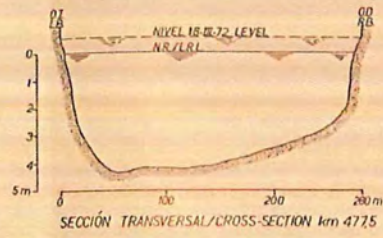
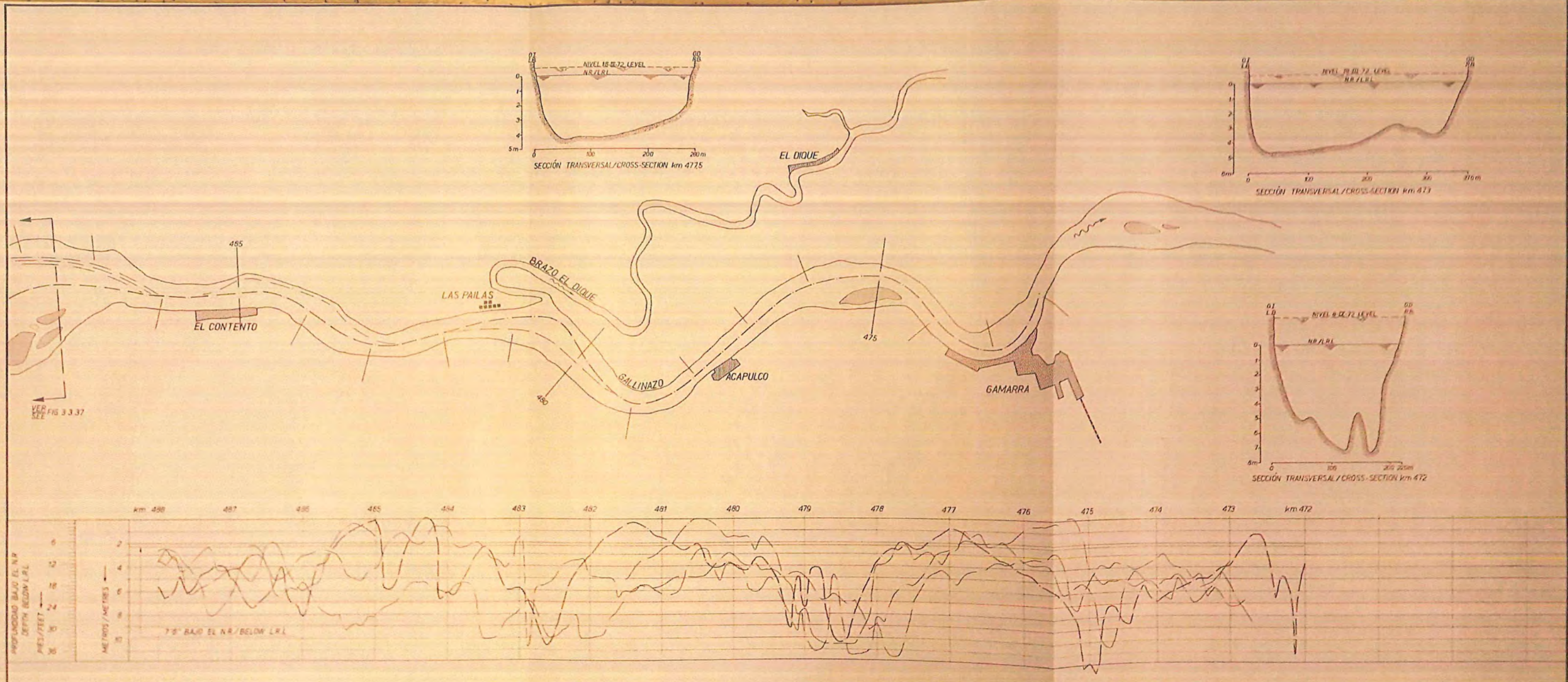
ESCALA / SCALE 1:40000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.3.36



PROFUNDIDAD / DEPTH	18-III-1972	0.80 m	BAJO EL RL / BELOW LRL
ANCHO / WIDTH		km 484	
CURVA / CURVE			
OBSERVACIONES / REMARKS	N a km 488 - km 472 1 km - 1000 m		

RUTA / SONDEO / PRINCIPAL	---	EN CASO NO EXISTENTES VER RUTA / SONDEO - PRINCIPAL
MAN / COURSE / SOUNDING	---	IF NOT GIVEN REFER TO MAIN COURSE - SOUNDING
RUTA / SONDEO / INICIAL	---	
INITIAL / COURSE / SOUNDING	---	

FECHA / DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN / WATER-LEVEL ABOVE LOW RIVER LEVEL					
	PTO SALGAR	PTO INMARCO	PTO BERRIO	BARRANCABERMEJA	PTO WILCHE'S	GAMARRA
8-IX-1972	m	m	m	m	1.43 m	1.90 m
7-II-1972	m	m	0.42 m	1.31 m	2.00 m	4.28 m
18-II-1972	m	m	0.46 m	0.20 m	0.25 m	0.56 m
3-III-1971	m	m	1.25 m	1.31 m	1.80 m	2.50 m

SONDEO LONGITUDINAL / LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 488 - km 472

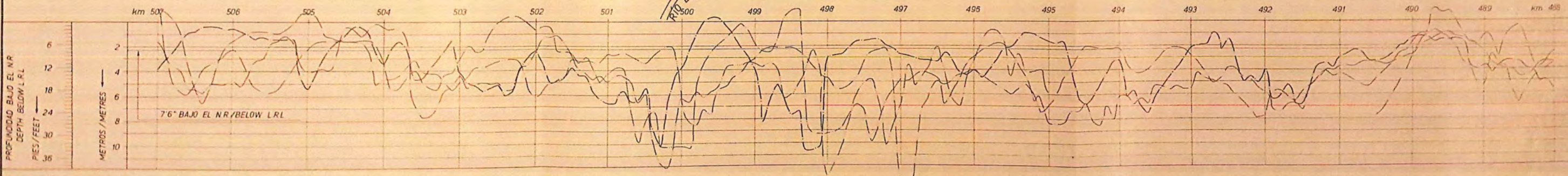
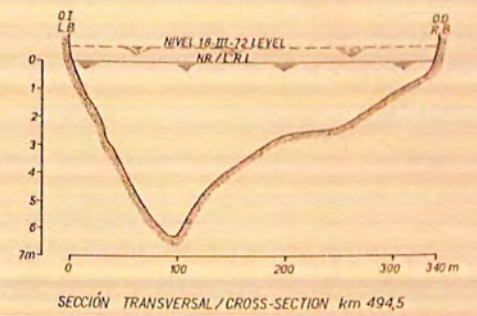
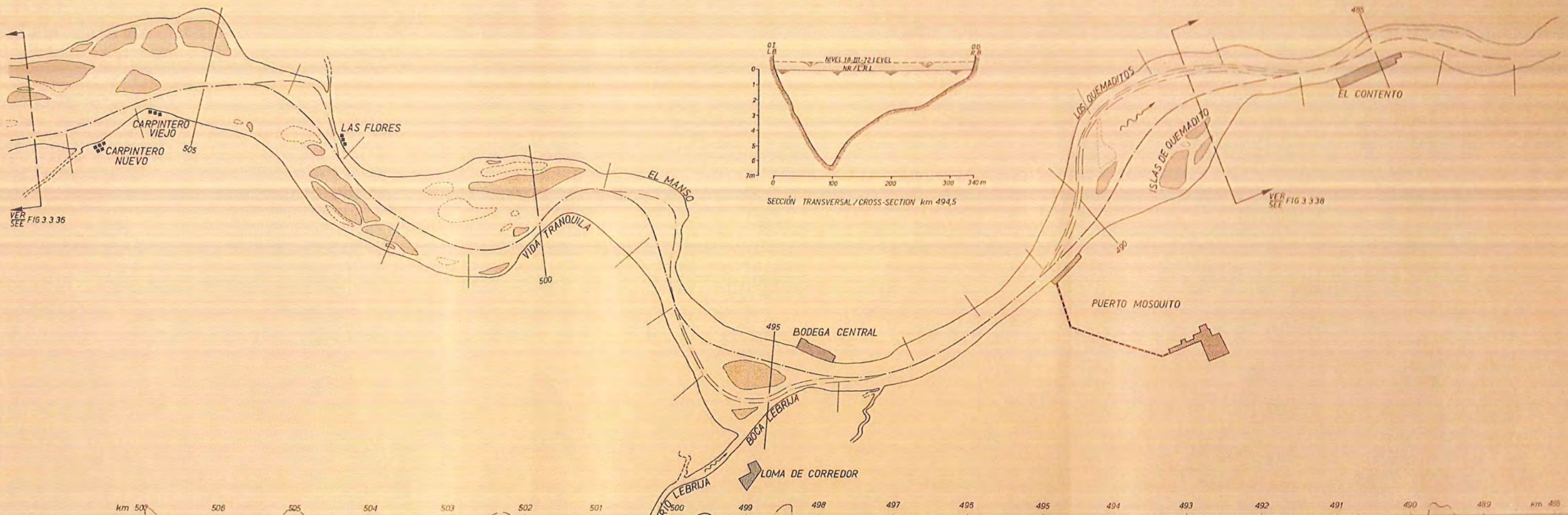
ESCALA / SCALE 1:40,000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.) / DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM / KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT

FIG 3.3.38



DIMENSIONES LIMITES LIMITING DIMENSIONS	
PROFUNDIDAD DEPTH	18-III-1972 0.90 m BAJO EL NR BELOW LRL
ANCHO WIDTH	km 489.5
CURVA CURVE	
OBSERVACIONES REMARKS	N.B. km 507 - km 488 1km = 1000m

LEYENDA LEGEND	
RUTA / SONDEO / PRINCIPAL MAIN / COURSE / SOUNDING	—
RUTA / SONDEO / INICIAL INITIAL / COURSE / SOUNDING	- - -
EN CASO NO EXISTENTES VER RUTA-SONDEO-PRINCIPAL IF NOT GIVEN REFER TO MAIN COURSE-SOUNDING	

FECHA DATE	NIVEL DE AGUA SOBRE EL NIVEL DE REDUCCIÓN WATER-LEVEL ABOVE LOW RIVER LEVEL				
	PTO SALGAR	PTO INMARCO	PTO BERRÍO	BARRANCABERMEJA	PTO WILCHES
8-IX-1972	m	m	m	m	1.43 m
7-XI-1972	m	m	0.42 m	1.31 m	2.00 m
17/18-III-1972	m	m	0.23 m	0.21 m	0.25 m
2/3-XII-1971	m	m	1.22 m	1.56 m	1.77 m
					GAMARRA 1.90 m

SONDEO LONGITUDINAL  
LONGITUDINAL SOUNDING

### RÍO MAGDALENA km 507-km 488

ESCALA / SCALE 1:40000

PROFUNDIDAD BAJO-NIVEL DE AGUA SOBRE-EL NIVEL DE REDUCCIÓN (N.R.)  
DEPTH BELOW-WATER-LEVEL ABOVE-LOW RIVER LEVEL (L.R.L.)

KILOMETRAJE A LO LARGO DEL RÍO BASADO SOBRE LOS DATOS DEL JULIUS BERGER KONSORTIUM  
KILOMETRES ALONG THE RIVER BASED ON THE DATA OF THE JULIUS BERGER KONSORTIUM

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG 3.3.37

3.3.8. Local soundings

Locally, soundings have been made of shallow areas where the available depth for navigation is smaller than the required depth. All the soundings which have been made by the Mission are listed in Table 3.3.4, but the maps will not be given here. In Part III of this Report, where the actual river improvement is treated, some of the soundings will be given in the chapters dealing with these areas. Together with the local soundings, flow-lines were often measured, and these are also indicated in Table 3.3.4.

	Location	Kilometer (approximate)	Date of Survey	Flow-lines	Presentation in Part III of this Report
Rto Magdalena	La Dorada - Pto. Salgar	Km 887	9-10 July '71	-	Para. 3.2.3
		895 - 888	8 Sept. '71	+	
		888 - 885	17 Nov. '71	+	
		888 - 885	31 Aug. '72	+	
		888 - 886.5	12 Oct. '72	+	
	Rto Nare Confluence	Km 778 - 776	19-21 Aug. '71	+	Para. 3.2.9
		776 - 773	19-21 Aug. '71	+	
	Rto Regla Confluence	Km 730 - 723	2 Nov. '71		Paras. 3.4.3 and 3.4.4
		723 - 717	28-31 Nov. '71		
		717 - 711	28-31 Nov. '71		
		711 - 706	2 Nov. '71	+	
		711 - 706	11,12 July '72	+	
	Rto Sogamoso Confluence	Km 614 - 609	4-10 Aug. '71	+	Para. 3.5.5
		616 - 614	15 Dec. '71	-	
		616 - 614	10 Jan. '72	+	
614 - 609		11 Jan. '72	+		
616 - 614		2 Febr. '72	-		
616 - 614		15 Febr. '72	+		
614 - 609		29 Febr. '72	+		
616 - 614		1- 3 March '72	+		
616 - 614		3- 8 May '72	+		
Calamar		17 June '71	+	Para. 4.3.3	
		3 Sept. '71	+		
		21-23 March '72	+		
Canal del Dique	Caño Correa Bifurcation		16 Febr. '71	+	
	Caño Matunilla Bifurcation		21 Jan. '71	+	
	Caño Lequerica Bifurcation		28 Jan. '71	+	
	Bahfa de Cartagena	Near Pasa- caballos	11 Febr. '71	+	Para. 4.4.3
			18 March '72		
Bahfa de Barbacoas	Outlet of Caño Matunilla	2- 3 Febr. '71		Para. 4.4.3	
		5- 6 April '72			
	Outlet of Caño Lequerica	4 Febr. '71		Para. 4.4.3	
		4 April '72			

Table 3.3.4 Local Soundings and Flow-lines

3.3.9. Miscellaneous information

Apart from the information presented in the foregoing Paragraphs some miscellaneous data were also gathered in the Canal del Dique, which will now be discussed separately.

- Sediment-transport measurements near Calamar

It has already been mentioned in Para 3.5.3 regarding the Calamar section in the Canal del Dique that the (measured) supply of sediment from the Rfo Magdalena to the Canal del Dique exceeds the (computed) transport capacity of the Canal and that, hence, sedimentation of the suspended particles must occur. To study whether the suspended-load actually decreases, measurements were carried out along the first 10 kilometers of the Canal del Dique on December 14-16, 1971, and the information will be presented in Part III, Figure 4.3.2, in which a slowly diminishing transport between km 0 and km 10 can be noticed.

Regarding the sediment-transport measurements along the Canal del Dique at Calamar in the Rfo Magdalena, it must be mentioned that only suspended-load data are considered. The Bed-load Transport Meter "Arnhem" (B.T.M.A.) was used in an effort to measure the bed-load, but this instrument can only measure bed-load coarser than 300  $\mu\text{m}$  and as the pertaining grain-size of the bed material is smaller, the pores of the meshed sampling basket of the B.T.M.A. were blocked and no bed-load could be measured.

- Velocity measurements along the Canal del Dique

Velocity verticals were measured along the Canal del Dique on April 11-13, 1972, and the information gathered, along with the wetted area of the cross-sections is presented in Figure 3.3.41.

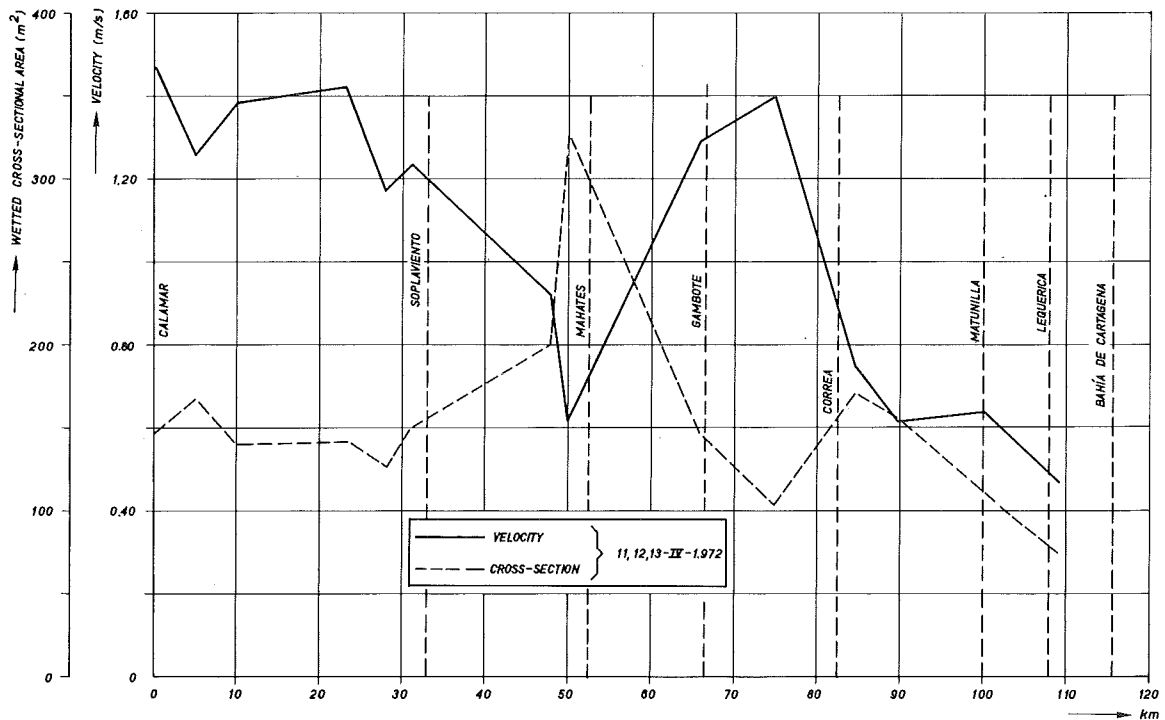
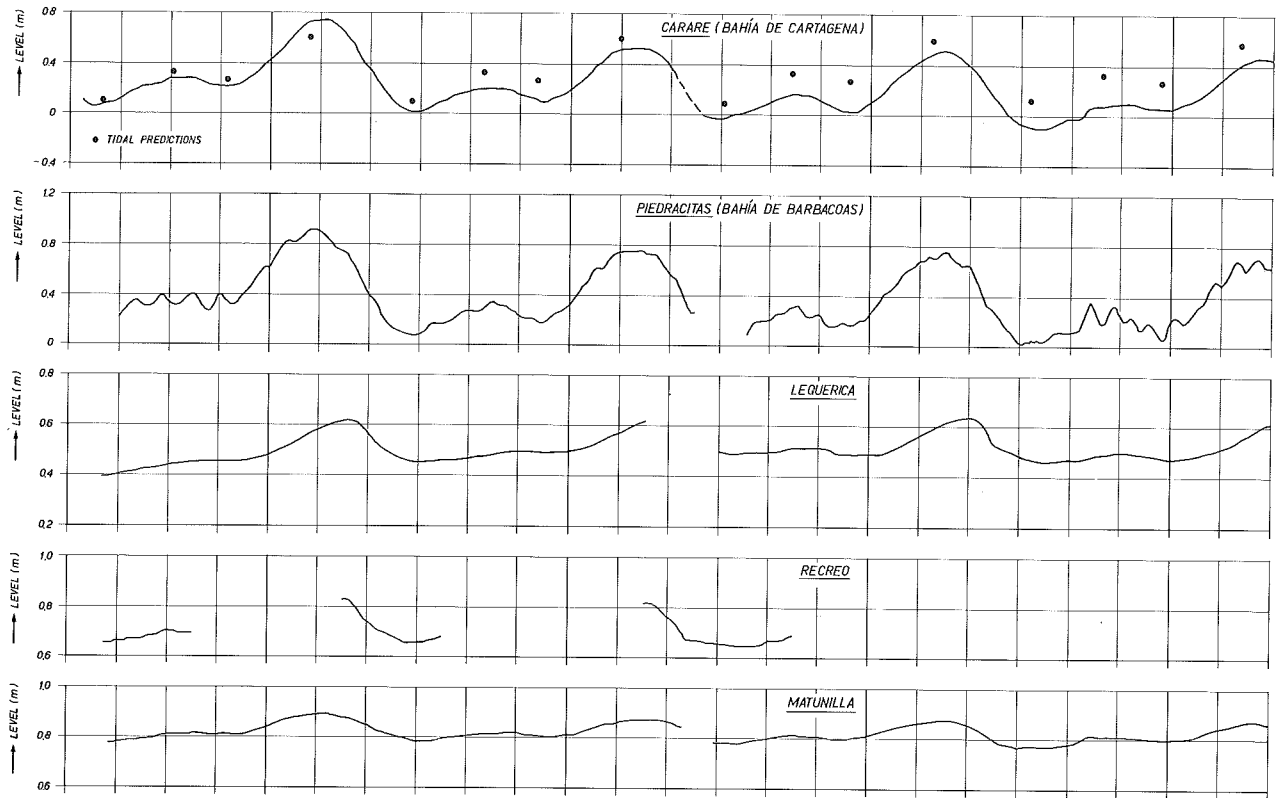
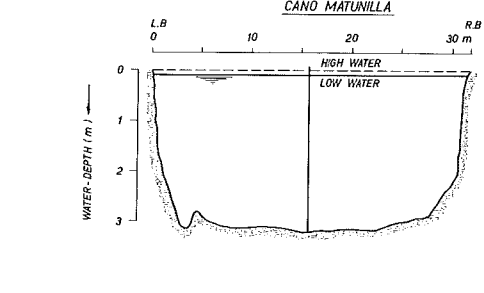
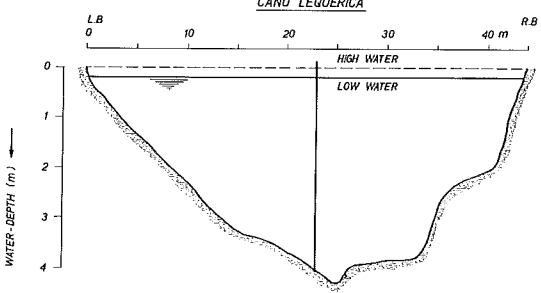
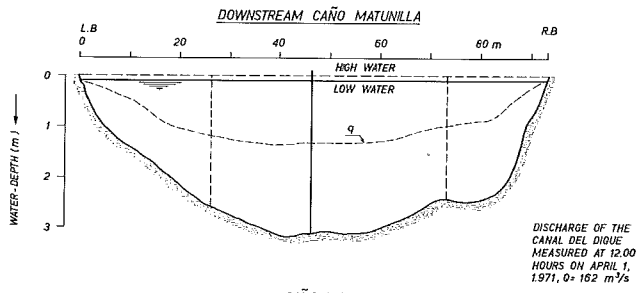
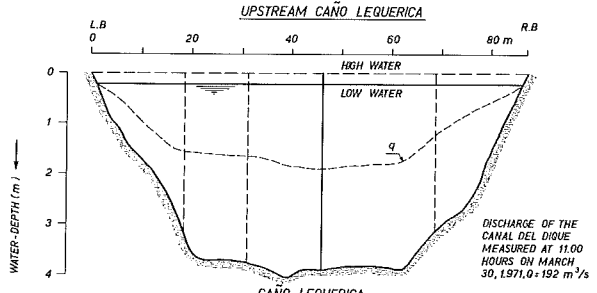
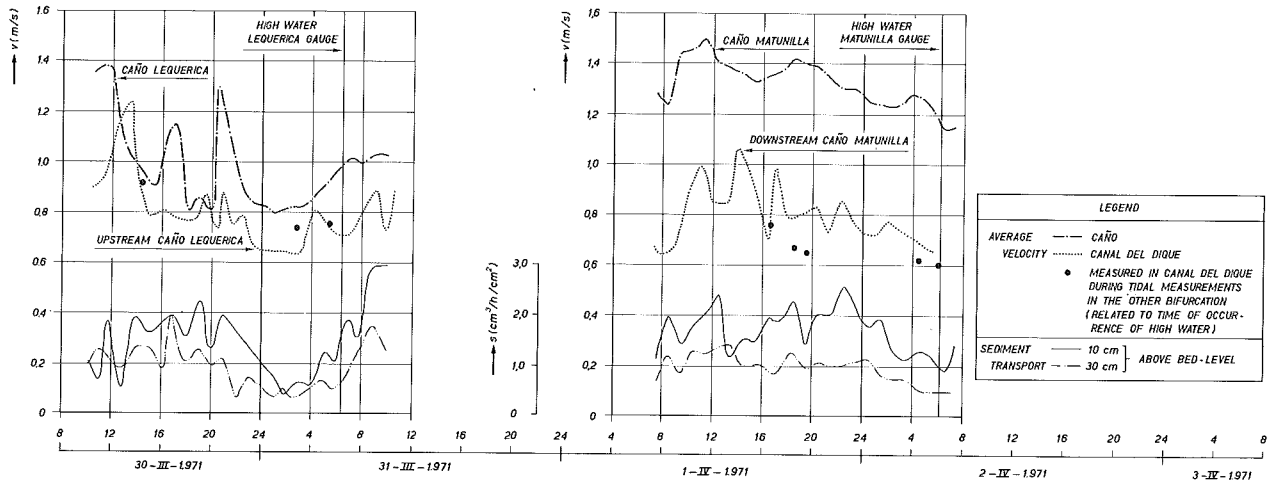


Figure 3.3.41 Average Velocity and Cross-sectional Area along the Canal del Dique



CORREA (WATER-LEVEL VARIATION BETWEEN 1.61 AND 163 m)



TIDAL MEASUREMENTS CAÑOS MATUNILLA AND LEQUERICA BIFURCATIONS

II, 3.3

- Tidal influence in the Lower Canal del Dique

Tidal measurements covering a complete tidal cycle (24 hours) were carried out in the Lower Canal del Dique region near the bifurcations of the Caño Matunilla and Caño Lequerica. The measurements were made at about the time of occurrence of spring tide in the Bahias de Cartagena and Barbacoas. The results of these measurements are given in Figure 3.3.42.

During these measurements in the low water period of 1971 the discharge of the Canal del Dique was still relatively high. For that reason the tidal measurements were repeated during spring tide in the low water period of 1972 (April 15-16, 1972). Although at that time the discharge of the Canal del Dique was considerably smaller ( $Q \approx 30 \text{ m}^3/\text{s}$ ) than in 1971, the velocities slackened also, but a reverse of the current direction was not found. In the Caño Matunilla the measured velocity ranged between  $v=0.35 - 0.70 \text{ m/s}$ ; downstream of the Caño Matunilla  $v=0.35 - 0.50 \text{ m/s}$ ; in the Caño Lequerica  $v=0.10 - 0.60 \text{ m/s}$ ; and downstream of the Caño Lequerica  $v=0.10 - 0.35 \text{ m/s}$ . Further data of these measurements are not presented in this Report.

During a short period water-levels were read on a number of gauges along the Caño Matunilla. The location of the gauges and the recorded water-levels at certain dates are presented in Figure 3.3.43.

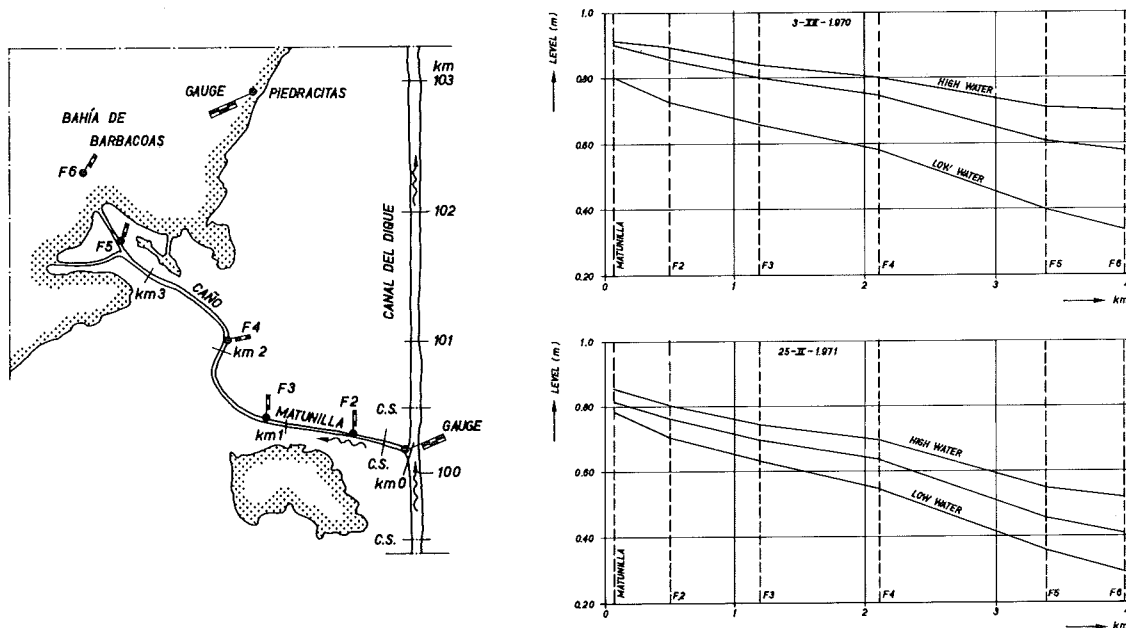


Figure 3.3.43 Water-levels along the Caño Matunilla

The data presented concern a mean tide in the Bahía de Barbacoas, while the discharge of the Caño Matunilla is estimated at about  $Q \approx 60 \text{ m}^3/\text{s}$ .

### - Salt intrusion

In the outlets of the Canal del Dique salt measurements were carried out a number of times at different discharges of the Canal and during different tides in the Bahías de Cartagena and Barbacoas. The salt water wedge measured  $\frac{1}{2}$  hour before high water near Pasacaballos in the Canal del Dique during spring tide of April 28, 1971, is presented in Figure 3.3.44.

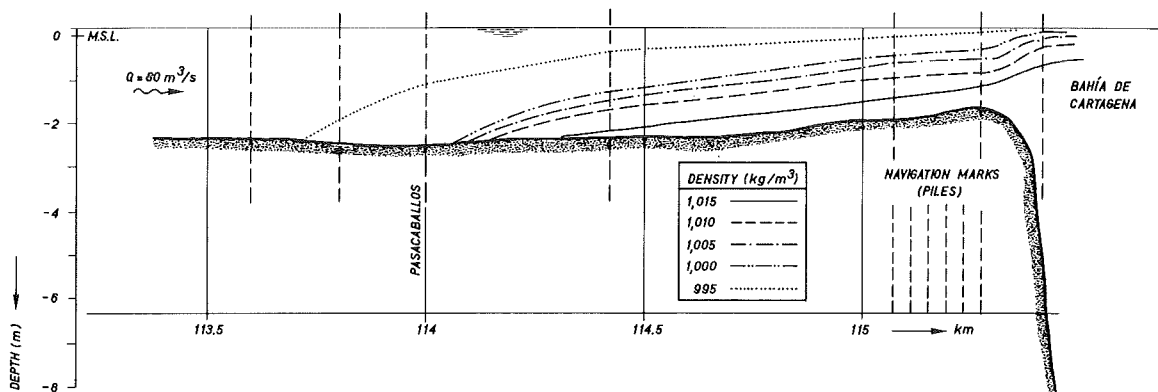


Figure 3.3.44 Salt Water Wedge near Pasacaballos

At still lower discharges of the Canal del Dique the salt water wedge will penetrate further inland. However, because of the small tidal amplitude, the small depth of the Canal del Dique and the relatively great depth in the Bahía de Cartagena, it is not likely that the salt water will penetrate beyond about km 113.

During the tidal measurements in April 1972 salt water was encountered near high water in the measuring cross-section in the Caño Lequerica close to the river-bed (about 0.20 m). Under extreme conditions (low discharge and spring tide) a thin layer of salt water may penetrate in the Caño Lequerica up to the bifurcation and be discharged again at Pasacaballos.

In the Caño Matunilla no salt intrusion was measured beyond the delta in the Bahía de Barbacoas, as the steeper water-level gradients in the Caño prevent further penetration.

3.4. BED-FORMS, CHANNEL ROUGHNESS3.4.1. General

In Para. 3.6, dealing with one-dimensional morphological computations, it will be seen that the equations of motion, for water as well as for sediment, contain a resistance parameter (channel roughness). Consequently for computations of river-works it is necessary to determine the channel roughness from the local geometry and flow parameters. The actual resistance to flow may be divided into a resistance due to the plan-form of a river such as meanders, a resistance due to energy losses behind bed-forms such as dunes and ripples, and a resistance due to the roughness of the grains. Generally, the plan-form resistance is neglected. Although the plan-form losses will probably be small in relation to bed-form roughness and grain roughness, the main reason for this neglect is a lack of knowledge in this field.

In this Report the total resistance will also be considered to consist of grain roughness and bed-form roughness only. This may be expressed as follows:

$$\tau = \tau' + \tau'' \quad (3.4.1)$$

where  $\tau'$  = the shear stress due to the grains and

$\tau''$  = the shear stress due to the bed-forms.

( $\tau''$  is not a true shear stress but an energy loss due to eddies, expressed as shear stress).

Eq. (3.4.1) may also be written as:

$$hI = (hI)' + (hI)'' \quad (3.4.2)$$

or as :

$$\frac{v^2}{C^2} = \frac{v^2}{(C')^2} + \frac{v^2}{(C'')^2} \quad (3.4.3)$$

Introducing  $C = \sqrt{\frac{8g}{f}}$  this can be written as:

$$f = f' + f'' \quad (3.4.4)$$

where  $f$  is called the Darcy/Weisbach friction factor.

It seemed useful to give all these different forms as it facilitates the explanation of the different methods given further on.

3.4.2. Grain roughness

Many investigators use for the grain roughness of a river with sediment transport the roughness of a river without sediment transport and a flat bed. In reality the two values differ. Noteworthy are the data gathered by Lovera and Kennedy (1969) [28] which are given in Figure 3.4.1. Comparing the results with the Nikuradse pipe-friction data, they found for the same values of  $f'$  and  $vR/v$  that  $R/k_s < R/D_{50}$  (in which  $k_s$  is the representative diameter of the grains). This means that for  $k_s$  a larger value than  $D_{50}$  has to be used, which is in accordance with results obtained by others:

- Einstein and Barbarossa (1952):  $k_s = D_{65}$  [29]
- Engelund and Hansen (1967) :  $k_s = 2D_{65}$  [11]
- Simons and Richardson (1966) :  $k_s = D_{85}$  [30]

For the Río Magdalena the Reynolds number ( $Re = vR/\nu$ ) varies between about  $2.5 \times 10^6$  and  $15 \times 10^6$ . Only a small part of this range is included in the Lovera/Kennedy graph (Figure 3.4.1).

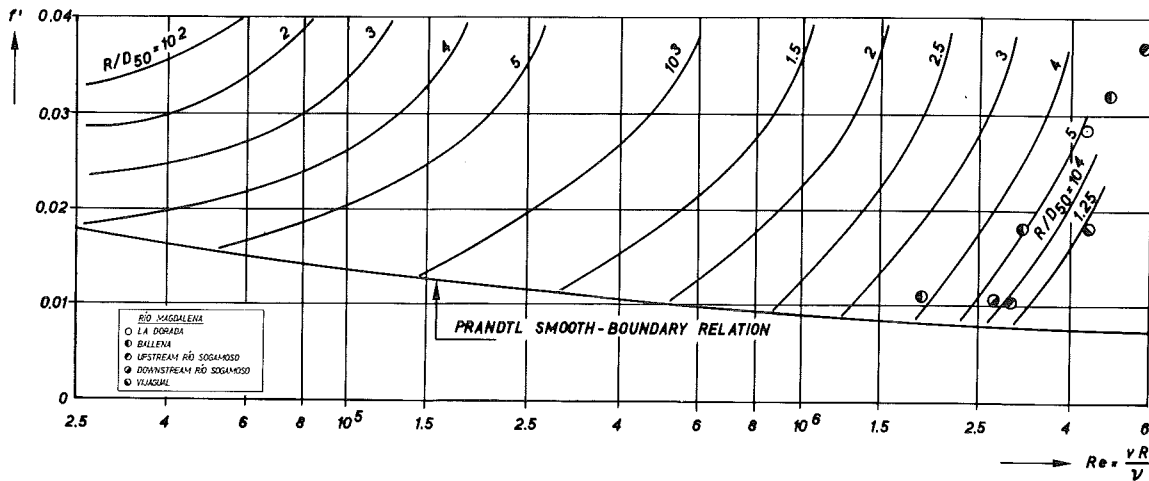


Figure 3.4.1 Lovera and Kennedy's Flat-Bed Friction Factor

For that part of the above-mentioned range of the Reynolds number which is included in the Lovera/Kennedy graph, a strongly diminishing influence of the Reynolds number on the  $f'$ -value can be noticed. This means that the grain roughness in that range mainly depends on  $h/D_{50}$ . Extrapolating the  $h/D_{50}$ -lines in the pertinent range of the Reynolds number would result in extremely high  $f'$ -values (in the order of 0.08), much higher than found on the Río Magdalena ( $f' \approx 0.01$ ). For the Río Magdalena  $f'$  has therefore been computed from:

$$C' = 18 \log \frac{12 h}{2D_{65}} \text{ and } f' = \frac{8g}{(C')^2} \tag{3.4.5}$$

Remark: The method used for the Río Magdalena is (contrary to the Lovera/Kennedy method) not dependent on temperature.

### 3.4.3. Bed-form roughness

The bed-form resistance is due to eddy losses in the expansion area behind the bed-forms. Bed-load may be considered as the propagation of bed-forms. In the case of two-dimensional bed-forms (dunes) this may be expressed as:

$$s = \alpha c \delta \tag{3.4.6}$$

where:  $\delta$  is the dune height,  $c$  the propagation velocity and  $\alpha$  a factor indicating the cross-sectional form of the dunes.

## II, 3.4

From Equation (3.4.6) a close relationship between the dimension of bed-forms and the rate of transport is suggested. In Para 3.2.1 it has been explained that the transport parameter ( $s/\sqrt{g\Delta D^3}$ ) is a function of the flow-parameter ( $\Delta D/\mu hI$ ). As a relation between bed-form roughness and the dimension of bed-forms also seems plausible, many authors studied bed-form roughness as a function of the flow parameter  $\Delta D/\mu hI$ .

In this paragraph the methods used by several authors will be indicated and compared, but before doing this, however, a few things should be mentioned for better understanding.

- Although suspended-load principally is not much different from bed-load, it does, of course, not take place as a bed-form propagation and Eq. (3.4.6) is not valid for suspended-load. As the percentage of the bed-material load transported as suspended-load does not depend on the flow parameter only, also the bed-form roughness depends on other parameters besides the flow parameter  $\Delta D/\mu hI$ . For practical purposes a  $C''$  versus  $\Delta D/\mu hI$  relation, as found from observations under conditions with heavy suspended-load, should be preferred for use on the Río Magdalena.
- Vanoni and Brooks (1957) [31] found that suspended-load reduces the turbulence, which results in an apparently smoother channel. This may be expressed in a graph as given by Engelund and Hansen [11] (Figure 3.4.2). It has been found that with sheetflow conditions the shear stress may fall below the value found from grain roughness experiments. Other investigators, however, sometimes found different results.

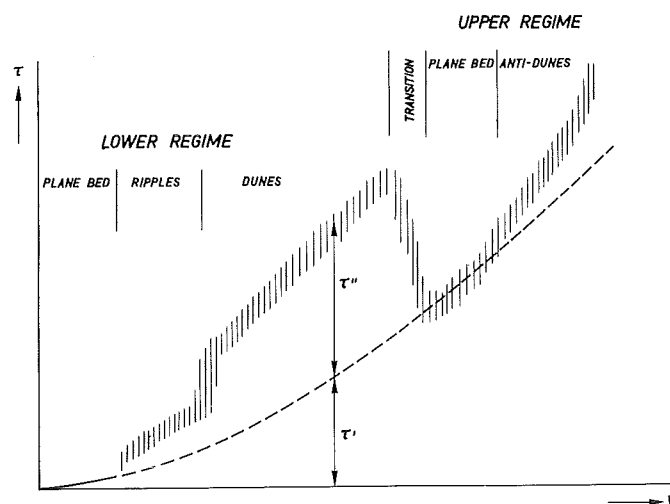


Figure 3.4.2 Schematic Relation between  $\tau$  and  $v$

- Some of the measurements were carried out after a relatively steep rise or fall in the water-level, the roughness found did therefore belong to hydraulic conditions different from those prevailing during the measurements.
- It should be remarked that recent investigations give reason to suspect a relatively large temperature influence on sediment transport, bed-forms and therefore bed-form roughness. In most methods this does not express itself.

Einstein and Barbarossa method [29]

Einstein and Barbarossa were the first (1952) to divide the total flow resistance into resistance due to the grains and that due to the bed-forms. This was done by a division of the cross-sectional area and the hydraulic radius into one part contributing only to grain resistance and another part contributing only to bed-form resistance. Thus in fact Eq. (3.4.2) was written as:

$$hI = I(h' + h'') \tag{3.4.7}$$

which may also be written as:

$$\left\{ \frac{V_*'}{V} \right\}^2 = \left\{ \frac{V_*''}{V} \right\}^2 + \left\{ \frac{V_*'''}{V} \right\}^2 \tag{3.4.8}$$

This method is derived from that used by Einstein (1939) to distinguish between bed resistance and side-wall resistance. Although the method is interesting, it does not seem in accordance with the actual phenomena. A more logical solution (introduced by Meyer-Peter and Müller (1948) [32] ) seems to write Eq.(3.4.2) as:

$$hI = h (I' + I'') \tag{3.4.9}$$

thus attributing part of the energy loss to the grain resistance and partly to the bed-form resistance.

This results in:

$$\frac{1}{C^2} = \frac{1}{(C')^2} + \frac{1}{(C'')^2} \tag{3.4.10}$$

which is in fact the method of separation used further on for the Río Magdalena.

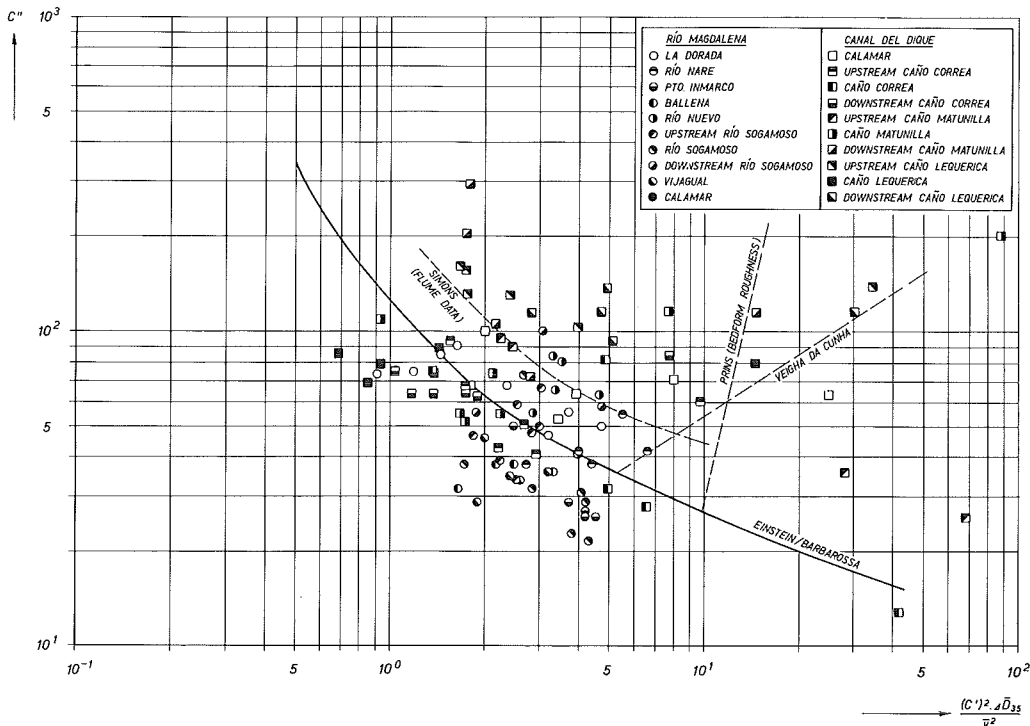


Figure 3.4.3 Einstein and Barbarossa's Graph for Flow Resistance due to Bed-Forms

II, 3.4

In Figure 3.4.3 the relation  $C''$  versus  $Y'$  ( $=\Delta D_{35}/h'I = (C')^2 \Delta D_{35}/v^2$ ) as found by Einstein and Barbarossa is given. In fact, Einstein and Barbarossa show the relation  $\bar{v}/\bar{v}''_*$  versus  $Y'$ . The ordinate  $\bar{v}/\bar{v}''_*$  differs a factor  $\sqrt{g}$  from  $C''$ .

Many of the data used by Einstein and Barbarossa are for rivers with large amounts of suspended-load and a good agreement with the Río Magdalena data was therefore expected. Although the Río Magdalena data are not really contrary to the Einstein/Barbarossa relation, the scatter is large and conclusions are difficult.

Some other curves have been found by others, also indicated in Figure 3.4.3.

As the use of the Einstein/Barbarossa method is rather complicated due to the use of  $h'$  and  $h''$  for the computation of  $C'$  and  $C''$ , for the Río Magdalena  $Y' = (C')^2 \Delta D_{35}/\bar{v}^2$  has been used with  $C' = 18 \log 12h/2\bar{D}_{65}$  (instead of  $C' = 18 \log 12 h'/D_{65}$  as used by Einstein and Barbarossa). Only slight changes may result from this, while the general tendency will be the same.

Shen method (1962) [33]

The Shen method is an extension of the Einstein/Barbarossa method. He found  $v''_*/\bar{v}$  ( $=\sqrt{f''/8}$ ) to be depending on  $wD_{50}/v$  besides the dependency on  $Y'$ . He plotted therefore  $Y'/\lambda$  against  $v''_*/\bar{v}$  with  $\lambda = F(wD_{50}/v)$ .

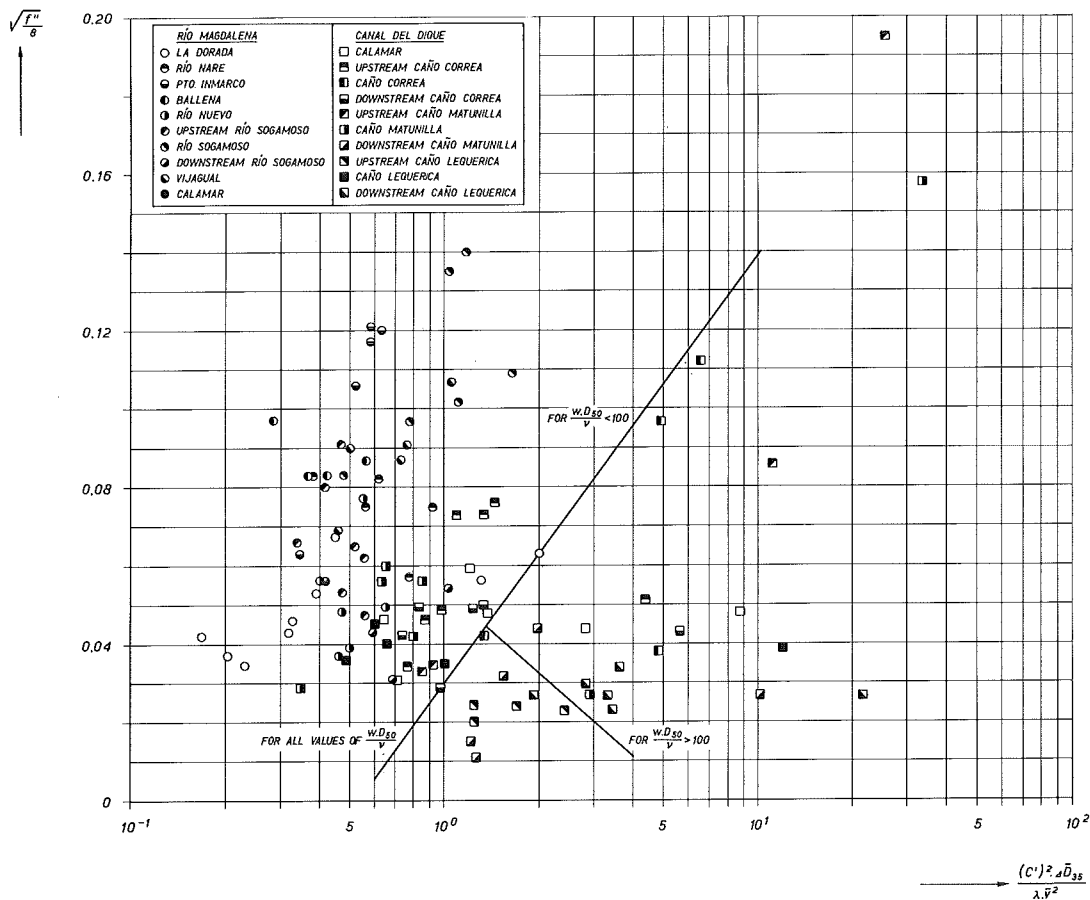


Figure 3.4.4 Shen's Graph for Flow Resistance due to Bed-Forms

For values of  $wD_{50}/\nu > 100$  the Einstein/Barbarossa curve can be used, but for values of  $wD_{50}/\nu < 100$  the value of  $\lambda$  has its influence. He also found a second branch to the Einstein/Barbarossa curve (see Figure 3.4.4).

For practical engineering purposes the method is nearly impossible to use. For separation of the bed-form roughness from the total roughness, the  $D_{65}$  has to be used according to Einstein. In  $Y'$  the  $D_{35}$  is used according to Einstein and Barbarossa, while for the correction factor ( $\lambda$ ) the value of  $D_{50}$  has to be used according to Shen.

The values of the Río Magdalena are quite distinct from those found according to Shen (Figure 3.4.4). Similar to the Einstein/Barbarossa plot, in  $Y'/\lambda$  the value of  $C'$  has been taken as  $C' = 18 \log 12 h/2\bar{D}_{65}$  for the Río Magdalena data.

The Shen method is temperature-dependent by means of  $\nu$ .

Engelund and Hansen method [11]

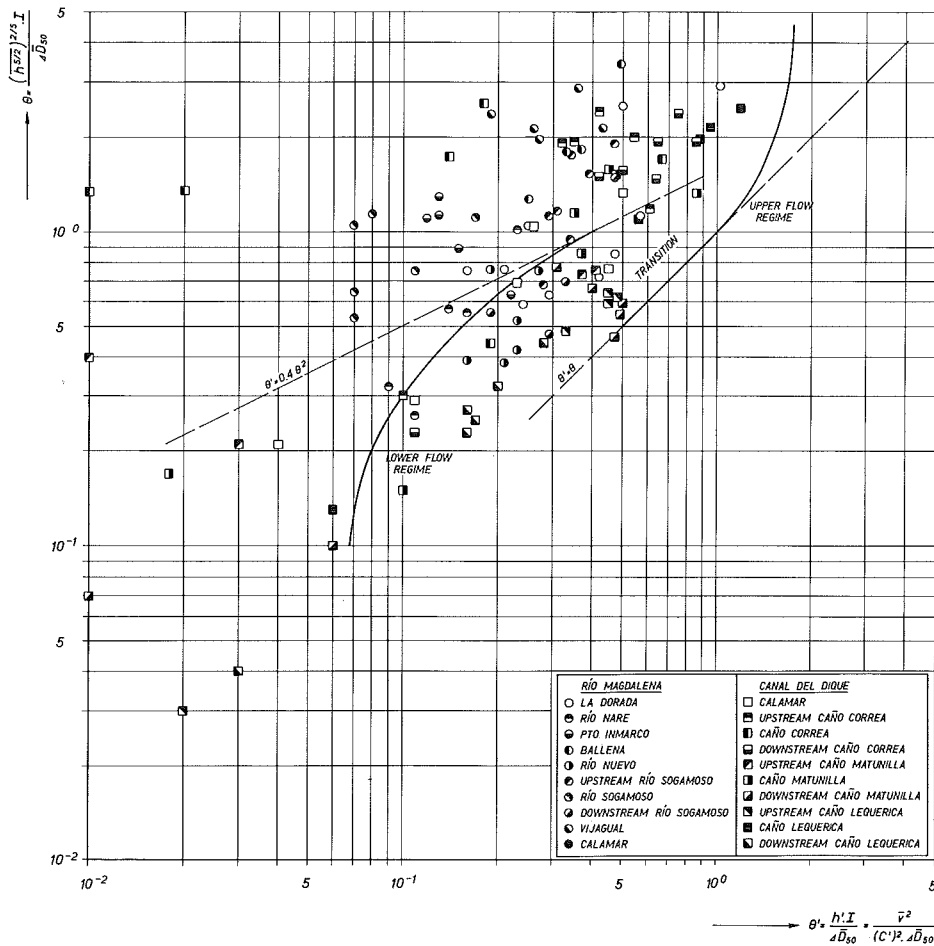


Figure 3.4.5 Engelund and Hansen's Relation between Normalized Grain Roughness Shear Stress and Total Shear Stress

## II, 3.4

Engelund and Hansen also separate bed-form roughness and grain roughness. They use the method indicated by Eq.(3.4.9). From their similarity hypothesis, it follows that:

$$\frac{hI}{\Delta D_{50}} = F \left\{ \frac{(hI)'}{\Delta D_{50}} \right\} \quad (3.4.11)$$

which is then writes as:

$$\frac{hI}{\Delta D_{50}} = F \left\{ \frac{h'I}{\Delta D_{50}} \right\} \quad (3.4.12)$$

If however, Eq.(3.4.12) is written as  $\bar{v}^2/c^2\Delta D_{50} = F(\bar{v}^2/(c')^2\Delta D_{50})$  it may be seen that, in fact, there is a relation in the form of:

$$c = F \left\{ \frac{\bar{v}^2}{(c')^2\Delta D_{50}} \right\} \quad (3.4.13)$$

Engelund and Hansen verified Eq.(3.4.13) by means of flume tests, of which the results are laid down in Figure 3.4.5.

Although the Engelund/Hansen method is based partly on theoretical considerations, for the Río Magdalena the method did not give better results than could be found by other methods.

### Alam and Kennedy method (1969) [34]

As indicated in Para. 3.4.2, Lovera and Kennedy used a different method to divide the total resistance into resistance caused by the grains and resistance caused by bed-forms. Alam and Kennedy also used this method to obtain  $f''$  ( $f'' = f - f'$ ;  $f'$  as given in Figure 3.4.1) and further studied a relation

$$f'' = F(v/\sqrt{gD_{50}}, h/D_{50}) \quad (3.4.14)$$

This relation was derived by means of dimensional analysis. They compiled their friction-factor chart from data gathered by several investigators (Figure 3.4.6).

For the Río Magdalena the value of  $h/D_{50}$  is generally larger than 2,500 and the value of  $\bar{v}/\sqrt{gD_{50}}$  is in the order of 25. From the graph it may be seen that in this range the influence of  $h/D_{50}$  is rather small, which means that the relation reduces to one between  $f''$  and  $\bar{v}/\sqrt{gD_{50}}$ . However, the results found on the Río Magdalena indicate a much stronger dependency on  $h/D_{50}$ . This is demonstrated in Figure 3.4.6, where the Río Magdalena values have been plotted in the Alam/Kennedy graph. In view of the remark made in Para. 3.4.2 regarding the high  $f'$ -values found from Figure 3.4.1 for the pertaining conditions on the Río Magdalena, the values presented in Figure 3.4.6 have either been plotted against the  $f''$ -value (for small values of  $f'$ , with  $f'' = f - f'$ ) or against the  $f$ -value (for high values of  $f'$ , with  $f'' = f - f' \approx 0$  and  $f \approx f'$ ).

The  $f''$  versus  $\bar{v}/\sqrt{gD_{50}}$  relation of Alam and Kennedy is only valid for sand. For other materials a relation  $f''$  versus  $\bar{v}/\sqrt{g\Delta D_{50}}$  would have to be used.

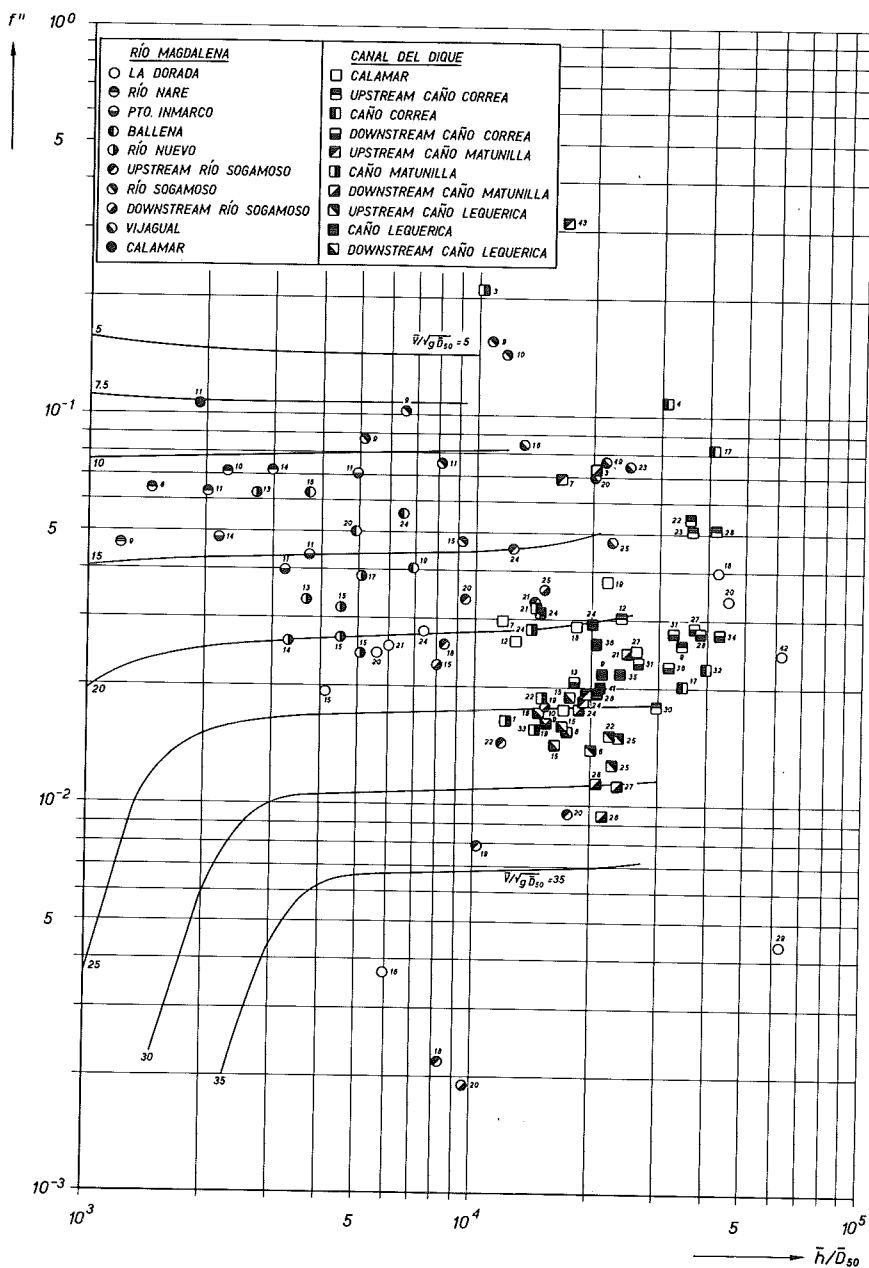


Figure 3.4.6 Alam and Kennedy's Graphical Expression of  $f''$  as Function of Froude Number and  $h/D_{50}$

Although a few other methods to compute the value of  $f''$  are still available, principally these are not very different from these mentioned here. However, there is one other method which must be referred to.

Regime theory

The Regime theory is a method based on statistical reasoning. It is therefore, thought to be a method which contains a risk for rivers such as the Río Magdalena, because this river has properties which are quite different from those on which the Regime theory is based.

The method used for the Río Magdalena

The methods which have been dealt with in this paragraph are compiled in Table 3.4.1.

Authors	Formulae	Remarks
Einstein and Barbarossa	$C'' = F \left\{ \frac{\Delta \bar{D}_{35}}{h'I} \right\} = F \left\{ \frac{\Delta \bar{D}_{35} \cdot (C')^2}{\bar{v}^2} \right\}$	For the computation of $h'$ , $D_{65}$ has to be used Not temperature-dependent
Shen	$C'' = F_1 \left\{ \frac{\Delta \bar{D}_{35}}{h'I}, \lambda \right\}$ $\lambda = F_2 \left\{ \frac{w \bar{D}_{50}}{v} \right\}$	For computation of $h'$ , $D_{65}$ has to be used The equation further contains $D_{35}$ and $D_{50}$ The method is temperature-dependent through $w/v$
Engelund and Hansen	$\frac{h'I}{\Delta \bar{D}_{50}} = F \left\{ \frac{h'I}{\Delta \bar{D}_{50}} \right\}$ may also be written as: $\frac{\bar{v}^2}{C^2 \Delta \bar{D}_{50}} = F \left\{ \frac{\bar{v}^2}{(C')^2 \Delta \bar{D}_{50}} \right\}$	For computation of $h'$ , $2D_{65}$ has been used Not temperature-dependent
Alam and Kennedy	$C'' = F(\bar{v}/\sqrt{g \bar{D}_{50}}, h/\bar{D}_{50})$	Slightly temperature-dependent, as the method is used in combination with the Lovera/Kennedy method to determine $C'$ ; the latter method contains the Reynolds number as parameter

Table 3.4.1 Methods Studied for Prediction of Bed-Form Roughness

From Table 3.4.1 it may be derived that the equations used can be written as:

$$C'' = F(h'I/\Delta D, C') \quad \text{(Einstein and Barbarossa),}$$

$$C'' = F(h'I/\Delta D, h/D) \quad \text{(Alam and Kennedy), and}$$

$$C = F(h'I/\Delta D, C') \quad \text{(Engelund and Hansen),}$$

while the Shen equation is similar to that of Einstein and Barbarossa with an additional correction factor ( $\lambda$ ):

As  $C'$  can be considered as a function of  $h/D$  (Eq.(3.4.5)) and  $h'I/\Delta D$  also contains  $h/D$ , it seems logical to try a relation  $C''$  versus  $h'I/\Delta D$  for the Río Magdalena. This has been indicated in Figure 3.4.7.

It may be seen that here also a considerable scatter is present. Part of this scatter may be attributed to the fact that measurements were carried out after a steep rise or fall of the water-level and, consequently, the roughness values belong to different hydraulic conditions. It can be seen that the scatter is not smaller than by using the Einstein/Barbarossa relation (Figure 3.4.3), which relation is based on many more data. For the computations in those sections of the Río Magdalena where no measurements were available, the Einstein/Barbarossa relation has been used, while for sections where measurements have been carried out, the local data have been used.

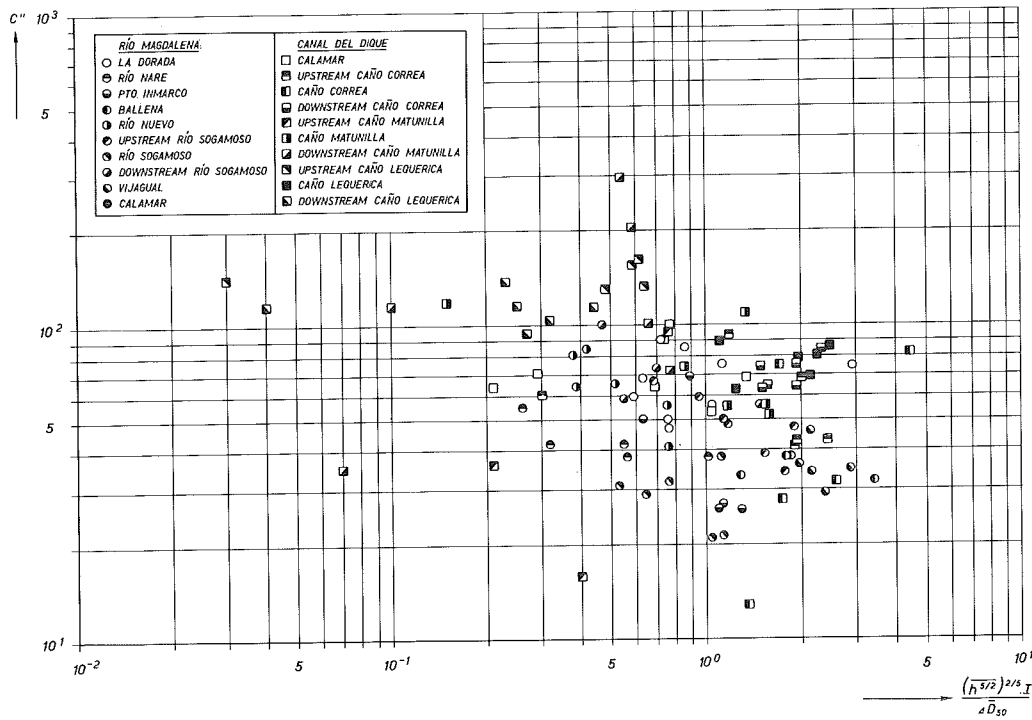


Figure 3.4.7 Relation between  $C''$  and  $hI/\Delta D$

Further study concerning bed roughness is required and possibly when more data become available an Alam Kennedy type of solution will appear more promising for the Río Magdalena.

Remarks:

In the foregoing part of this paragraph the time dependency of the alluvial roughness has not been taken into consideration, but in reality the channel roughness always lags behind the prevailing hydraulic conditions. Not much is known yet in this field. Generally, this aspect is not very important, although in some specific cases the change in roughness is large and can lag far behind. On the Río Magdalena this is sometimes the case on a crossing just after the high water period, when the bed-forms due to the high water conditions are still present. When required, an experimental exponential function may be used for the adaptation process as indicated in Figure 3.4.8 (the value of  $t_0$  has to be found from measurements).

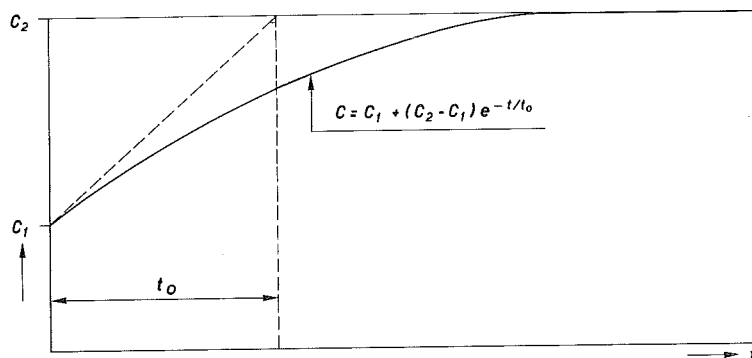


Figure 3.4.8 Roughness Adaptation as Function of Time

## II, 3.4

In this paragraph it has been assumed that the river-bed is alluvial, but there are places where it is not alluvial or only partly alluvial (Honda). In those cases the roughness is built up by the alluvial part (bed-forms) and the non-alluvial part (rocks). When the sediment is transported over the rocks, the voids between the rocks are partly filled and the non-alluvial part of the roughness is influenced by the sediment transport rate. This has been indicated in Figure 3.4.9 ( $\bar{s}$  represents the sediment transport and  $\bar{s}_0$  the sediment transport capacity; see van der Zwaard (1973) [35]).

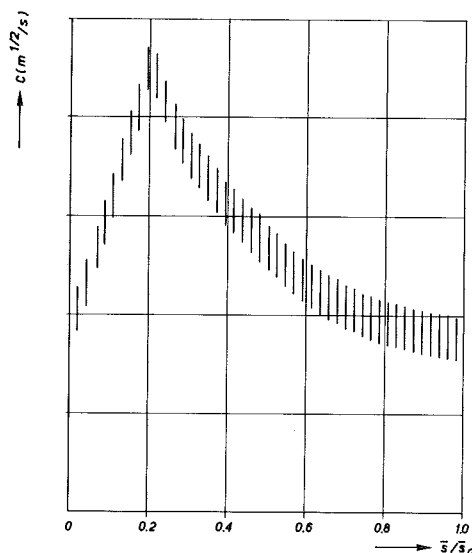


Figure 3.4.9 Roughness for Sediment Transport over a Fixed Bed

## 3.5. SELECTION OF THE TRANSPORT-EQUATION

### 3.5.1. Introduction

Reconsidering the foregoing paragraphs, the following questions still need to be answered:

- Which transport-equation best answers the prevailing conditions on the Río Magdalena?
- What is the relation between the water-levels and, respectively, the discharge, the roughness, the water-level gradient and the sediment transport in the various measuring cross-sections in the Río Magdalena and the Canal del Dique?

Moreover, anticipating the next Paragraph (3.6), a third question will arise:

- Which procedure can best be followed in those cases when no measurements are available? It will be seen in Para. 3.6 that one of the boundary conditions for the morphological computations is the stage-discharge relation. If such a relation is available at a nearby main gauge-station, the question will arise whether such a relation can be transferred to an area where no measurements are yet available. Also the channel roughness in the area under consideration needs then to be estimated.

Before answering these questions, however, a short recapitulation of the general introduction to this Part of the Report seems to be appropriate.

It was mentioned that various investigators have defined transport equations which established a relation between the transport parameter ( $s/\sqrt{\Delta g D^3}$ ) and the flow parameter  $\Delta D/\mu h I$ . In this relation the constants were chosen in such a way that the results agreed with measurements made in the field or in the laboratory. One of the main reasons of, especially, the transport measurements carried out by the Mission was to select that equation which best corresponded to the conditions on the Río Magdalena and the Canal del Dique. It would have been possible also to establish a new transport equation, according to the measurements, which might fit these data better than one of the earlier established equations. However, such a new equation has only a limited use, and extrapolation from a few measuring sites to the whole length of the river can lead to faulty results. By contrast, the known transport equations were, generally, tested on a great range of conditions, and if such an equation appears to agree reasonably well with the measured conditions, extrapolation to sites where no measurements are as yet available will give more reliable results.

For the same reason it has extensively been tried to correlate the channel roughness to the known theories. Another approach could have been simply to accept the relation between the roughness and the water-level found at the various measuring cross-sections and to estimate the roughness at those places where no data are known. However, again more reliable results are to be expected if a known theory is tested first at a number of places to check its applicability, and only thereafter extrapolated to sites where no data are available. Unfortunately, the data found on the Río Magdalena and the Canal del Dique did not appear to be universally in agreement with one of the theories on this subject. For some cross-sections the relation as presented by Einstein and Barbarossa showed good results, while at other places the Alam/Kennedy relation could better be used (see Para. 3.4). Therefore, it was finally decided to establish the relation between the water-level and, respectively, the discharge, the water-level gradient, the channel roughness and the sediment transport according to the measurements only (see Para. 3.5.3). At those places where no data are as yet available the Einstein/Barbarossa relation (see Figure 3.4.3) can best be used. This implies, however, that the sensitivity of the morphological computations needs to be checked on both the roughness relation according to the field data and the relation presented by Einstein and Barbarossa. This comparison is given in Chapter 8, in which the morphological computations of the Río Sogamoso Confluence are dealt with.

### 3.5.2. Selection of the transport-equation

It has been mentioned in Para. 3.3 that the sediment transport measurements carried out in the Río Magdalena revealed that the bed-load is, generally, only a small percentage (< 10%) of the bed-material load. It is, therefore, obvious to test the data of the Río Magdalena not on the so-called bed-load equations (e.g., Meyer-Peter/Müller) but to look for an equation which describes the total-load. The available formulae which appear to be suitable are:

## II, 3.5

- Einstein and Brown (1950) [10];
- Colby (1964) [36];
- Engelund and Hansen (1967) [11]; and
- Toffaleti (1969) [37].

The Colby and Toffaleti formulae are both temperature-dependent. Moreover, for both methods the bed material has to be divided into "standard" size fractions (as done by Einstein), and the total sediment transport is taken as the sum of the transport of each of the fractions. In both cases the total-load can be computed by means of graphs. The Task Committee for Preparation of Sediment Manual of the American Society of Civil Engineers [38] made this comment in its review on the available formulae of sediment-transport:

"The Colby, Toffaleti and Engelund/Hansen relations give consistently better agreement with the available data from field and laboratory measurements than the others. Because the Colby and Toffaleti relations have been shown to agree reasonably well with a large body of data from streams and flumes, the tendency is to rely more heavily on them than on the others. Also because the Engelund/Hansen equation gives results that agree with those from the Colby and Toffaleti relations, it seems that reasonable confidence can be placed in it".

As only in some cases were water-temperatures measured by the Mission, the testing of the transport equation has to be restricted to the Einstein/Brown and Engelund/Hansen formulae. (For these comparisons the C values used were derived from the discharge measurements.) Although it will appear that the Engelund/Hansen formula is in good agreement with the data measured on the Río Magdalena and the Canal del Dique, it is suggested that, especially, the water temperatures will also be measured in future. At a later date this will then also enable a comparison with the Colby and Toffaleti equations.

Before actually comparing the measured and computed volumes of sediment transport, it is useful to remember the manner in which a transport equation is derived (which implies that testing of the equation to measured data has to be carried out in the same way), and how the transport equation is used to compute the morphological changes of the river as a result of an implemented river-work.

Most of the transport-equations are derived according to flume data, which implies that they are related to the pertaining parameters per unit width. Multiplication by the width yields the total transport. However, in the morphological computations (see flow-diagrams of computations in Para. 3.6) only average parameters of the whole cross-section are introduced and computed. This is why the comparison of the measured transports to the computed ones according to the Einstein/Brown and Engelund/Hansen formulae has been carried out in three ways, namely:

- Comparison per vertical (Figure 3.5.1 and 3.5.3)

The data of each measured velocity and sediment-transport vertical have been used. It has been assumed that the locally measured water-level gradient and the computed roughness are the same for all the verticals in one cross-section. If a bed-sample had been taken in the vertical, the grain-diameter of this sample was used; if not, the average value of all the bed-samples taken in the cross-section has been used.

- Comparison per cross-section (Figure 3.5.2 and 3.5.4)

The sediment transport has been computed using the value of  $(h^{5/2})^{2/5}$  of the cross-section and the average grain-diameter of all the bed-samples.

- Comparison per schematized cross-section (see Figures in Para. 3.5.3)

The measured sediment transport can also be compared with the computed stage-transport relation for the cross-section under consideration. This stage-transport relation is computed using the parameters of the schematized cross-section and the relations between the water-level and, respectively, the channel roughness and the water-level gradient (see Para. 3.5.3).

The equation of Einstein and Brown [10] can be written as:

$$\frac{s}{\sqrt{\Delta g \bar{D}_{50}^3}} = \frac{40 F}{1-\epsilon} \left( \frac{hI}{\Delta \bar{D}_{50}} \right)^3 \quad (3.2.3)$$

in which:

$\epsilon$  = porosity ( $\epsilon = 0.4$ , see Para. 3.3.6)

$$F = \sqrt{\frac{2}{3} + \frac{36\nu^2}{\Delta g \bar{D}_{50}^3}} - \sqrt{\frac{36\nu^2}{\Delta g \bar{D}_{50}^3}}$$

$\nu$  = kinematic viscosity (the introduction of the kinematic viscosity makes this equation also temperature-dependent; however, in this Report  $\nu = 0.8 \times 10^{-6} \text{ m}^2/\text{s}$  has been used, see Para. 3.3.2).

It is somewhat uncertain which representative grain-size has to be used in the formula because the original literature does not give a decisive answer. However, in harmony with the afore-mentioned Task Committee of the ASCE the  $D_{50}$ -value has been used.

The comparison between the measured and computed sediment transport with the equation of Einstein and Brown is presented in Figures 3.5.1 and 3.5.2. The computations were carried out according to the division outlined above (Equation (3.2.3) expresses the sediment transport in  $\text{m}^3/\text{m}^1/\text{s}$ ; in the figures the transports are presented in  $\text{m}^3/\text{m}^1/\text{day}$ , respectively  $\text{m}^3/\text{day}$ .)

The equation of Engelund and Hansen [11] can be written as:

$$\frac{s}{\sqrt{\Delta g \bar{D}_{50}^3}} = \frac{0.05}{1-\epsilon} \cdot \frac{C^2}{g} \cdot \left( \frac{hI}{\Delta \bar{D}_{50}} \right)^{5/2} \quad (3.2.2)$$

in which:

$g$  = acceleration of gravity

$C$  = total bed roughness.

The comparison between the measured and computed sediment transport with the equation of Engelund and Hansen is presented in the Figures 3.5.3 and 3.5.4.

II, 3.5

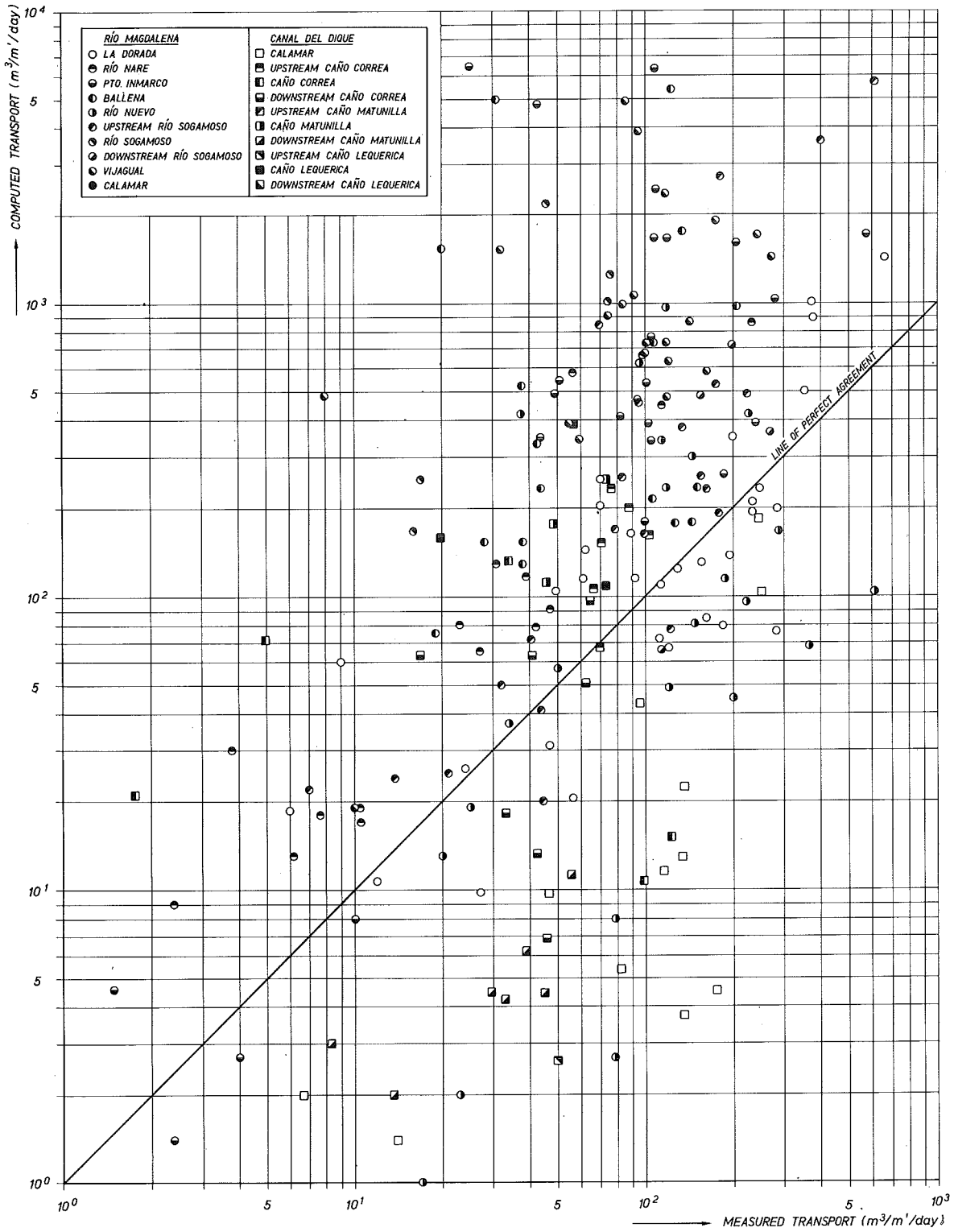


Figure 3.5.1 Comparison per Vertical between Measured and Computed Sediment Transport with Einstein/Brown Formula

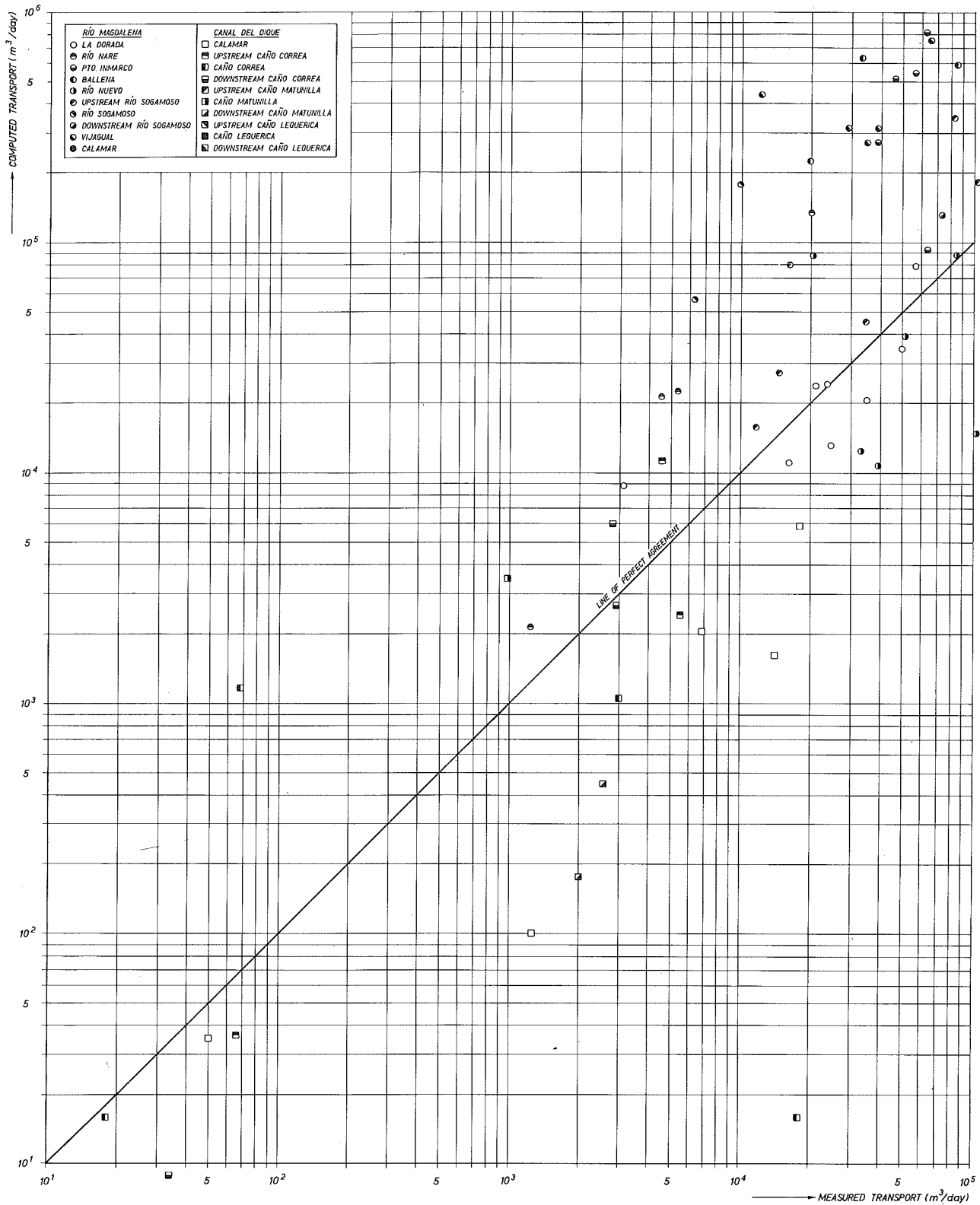


Figure 3.5.2 Comparison per Cross-section between Measured and Computed Sediment Transport with Einstein/Brown Formula

II, 3.5

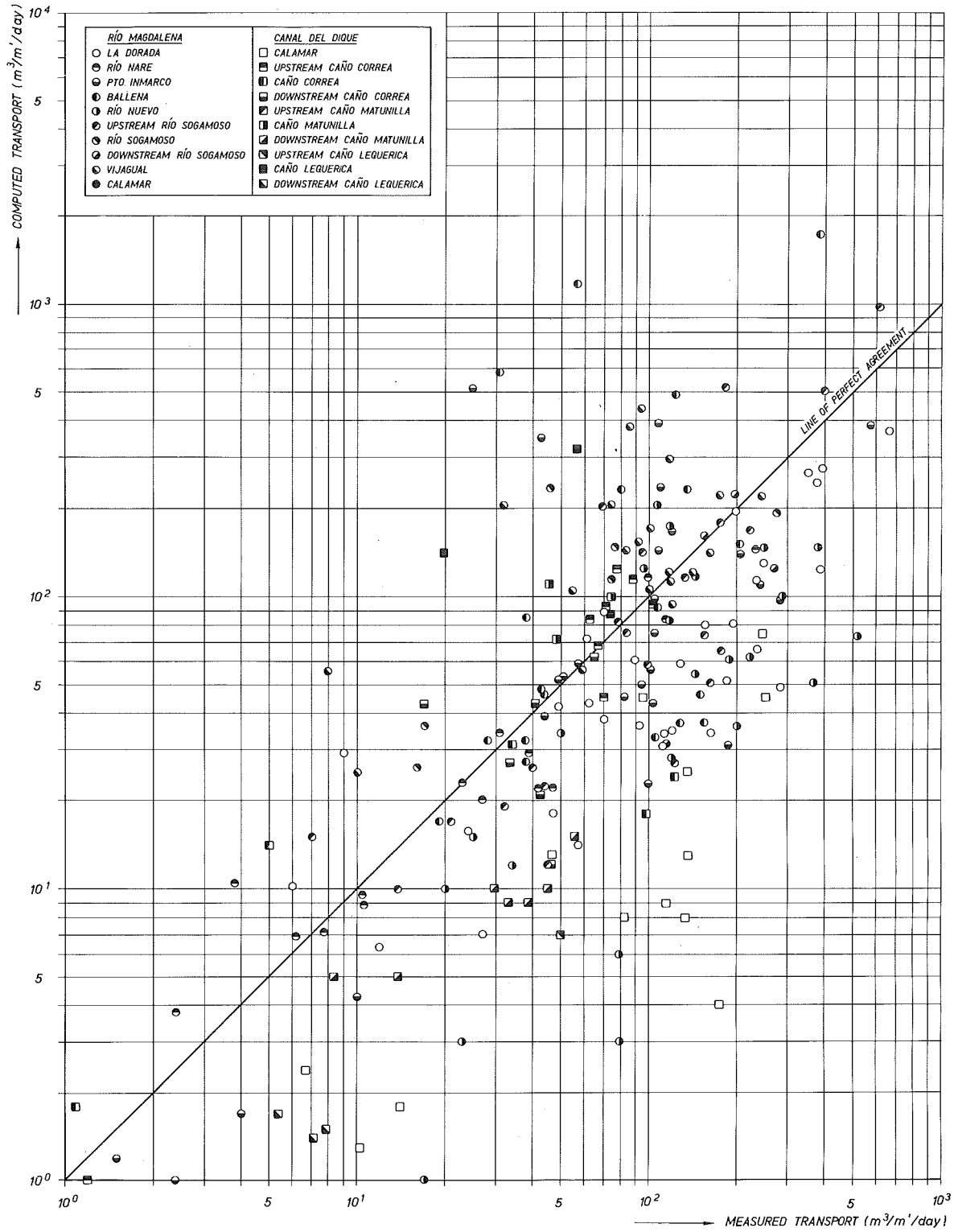


Figure 3.5.3 Comparison per Vertical between Measured and Computed Sediment Transport with Engelund/Hansen Formula

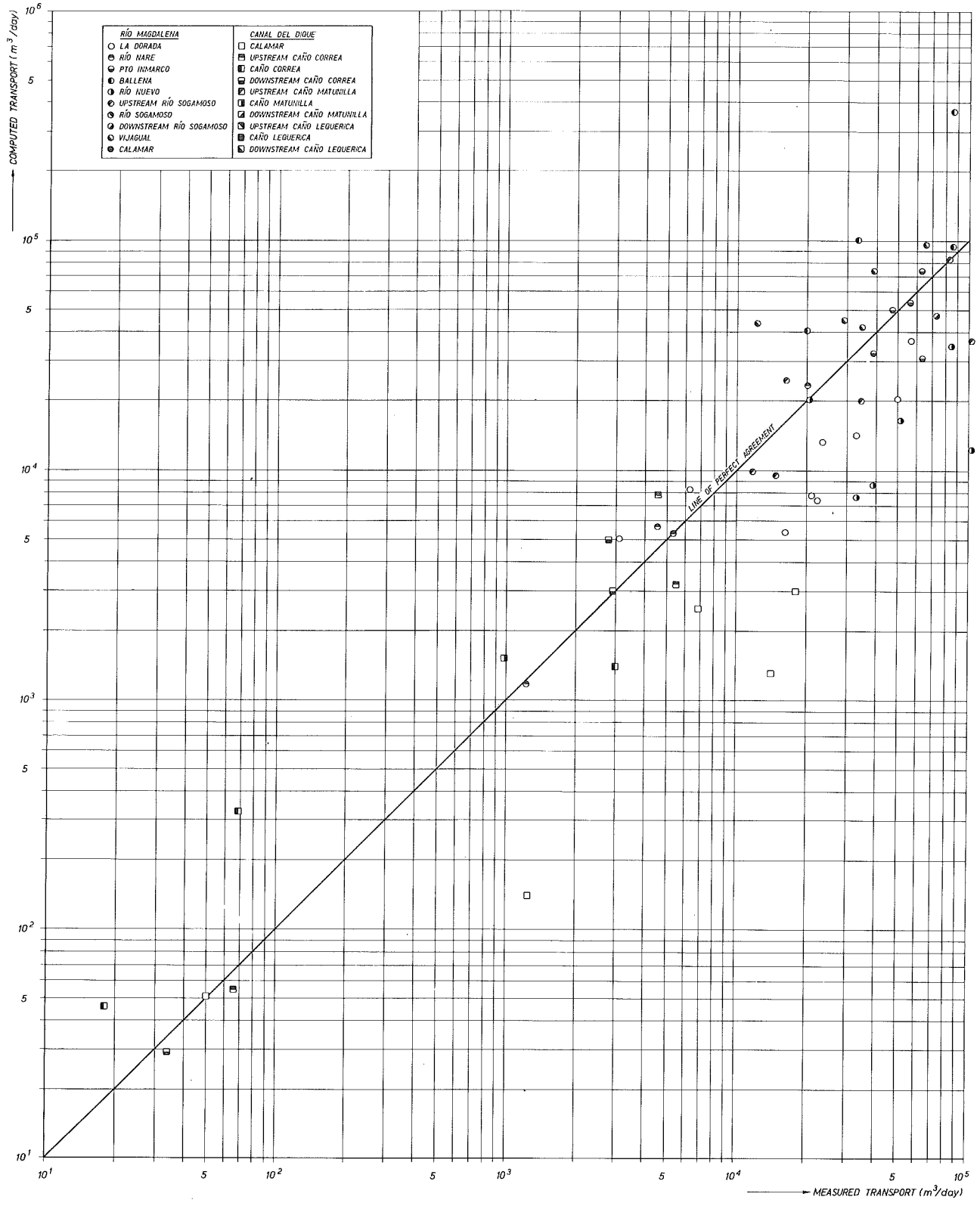


Figure 3.5.4 Comparison per Cross-section between Measured and Computed Sediment Transport with Engelund/Hansen Formula

## II, 3.5

Although a considerable scatter is still present, it can be concluded that the measured sediment transport agrees far better with the computed transport according to the Engelund/Hansen equation than with the Einstein/Brown equation. The morphological computations in this Report have, therefore, been carried out with the equation of Engelund and Hansen.

It must now be considered what the influence of the distribution of the suspended particles in the vertical (Para. 3.3.4) will be on the behaviour of the river-bed if morphological computations are carried out in transient flow. In Para. 3.6.3 it is shown that the change in sediment transport from one section to another results, also as far as the suspended-load is concerned, only in changes of the bed-level and hardly in a change of the concentration of the suspended particles. However, this is only valid if a total-load formula is used to describe the sediment transport. Because the equation of Engelund and Hansen is in good agreement with the measured data of the Rfo Magdalena and the Canal del Dique, and this equation was derived as a total-load formula, it can be concluded that if this equation is used in the morphological computations, the changes in concentration of the suspended particles can be neglected (Para. 3.6.3).

### 3.5.3. The discharge, channel roughness, water-level gradient and sediment-transport relations

In Para. 3.4 an attempt was made to correlate the channel roughness to the flow parameter. However, no proper relation could be found which was valid for all the measurements carried out on the Rfo Magdalena and the Canal del Dique. Therefore, the relationship between the water-level and, respectively, the channel roughness and the water-level gradient has to be found from the measurements themselves. From Chézy's law:

$$Q = Bh^{3/2} \cdot CI^{1/2} \quad (2.5.1)$$

it will be clear that, first of all, the cross-section must be schematized to find the contribution of the conveyance ( $Bh^{3/2}$ ). This has been done by using the average of all the cross-sections measured at one location. For convenience sake, the cross-sections have been plotted as mass curves. From the average schematized cross-section the relationship between the water-level and, respectively, the average water-depth ( $\bar{h}$ ), the value of  $(\bar{h}^{3/2})^{2/3}$  and of  $(\bar{h}^{5/2})^{2/5}$  has been determined. According to Eq. (2.5.1) it will be clear that in the morphological computations the value of  $h^{3/2}$  has to be an average value. For this reason the line of  $(\bar{h}^{3/2})^{2/3}$  has been determined. Similarly, for the sediment transport the line of  $(\bar{h}^{5/2})^{2/5}$  has to be used, because the power law of the equation of Engelund and Hansen is 5/2. Given the stage-discharge curve of the location, the Chézy coefficient or the water-level gradient can be estimated in such a way that, on the one hand, Eq. (2.5.1) holds and, on the other hand, the computed water-level gradients or Chézy coefficients correspond as best as possible to the measured data (for the measured data, reference is made to Tables 3.3.1 and 3.3.2, given in Para. 3.3). Moreover, with the selected transport equation of Engelund and Hansen, the stage-transport relation can be computed.

These computed relations will now be presented, while for convenience sake, the figures are given at the end of each of the two sub-sections of this paragraph.

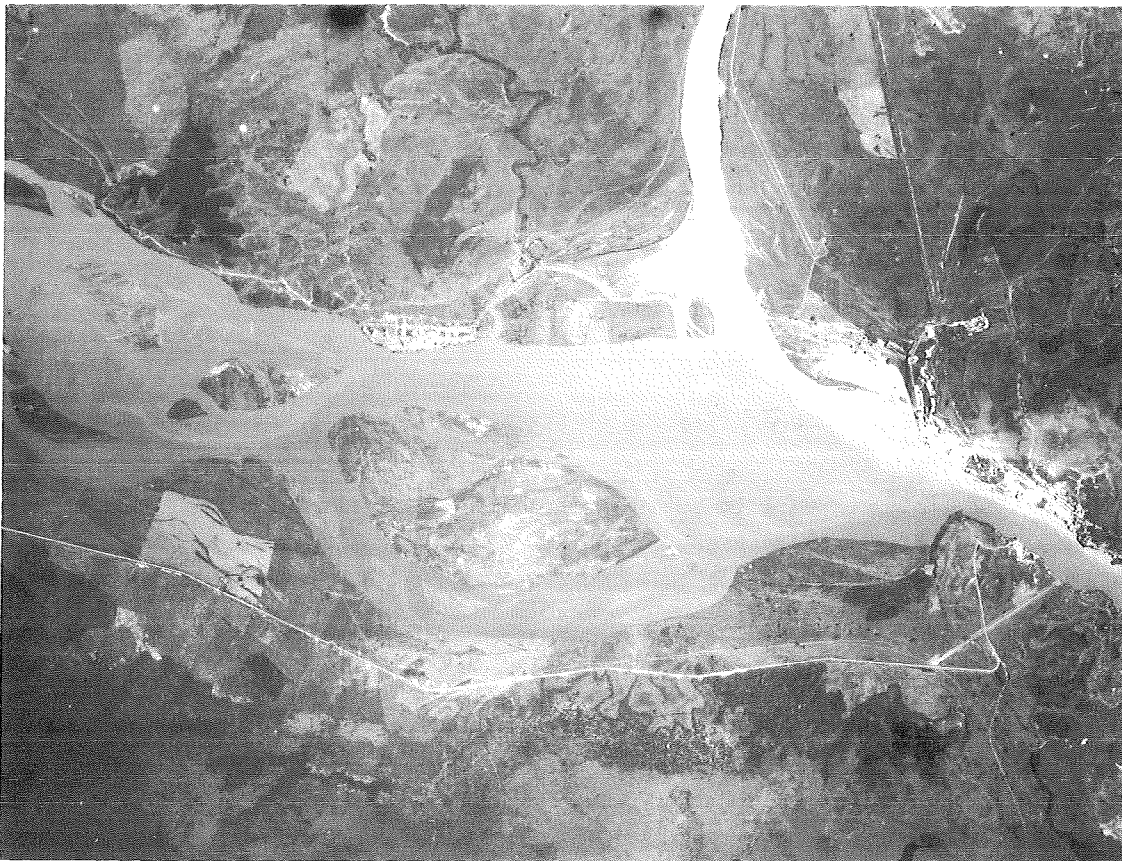
A The Rfo Magdalenaa) The La Dorada Section (Rfo Magdalena, km 884)

The stage-discharge curve of La Dorada (Pto. Salgar) has been drawn according to the measurements carried out by the Julius Berger Konsortium, Apron y Duque Ltda., the SCMH and the Mission. The data of the Julius Berger Konsortium have been corrected because of a difference in zero-level between the old and the present gauge. In the Julius Berger Report the zero-level of the gauge, situated in La Dorada, is given at 168.90 m above M.S.L. This gauge was later shifted to the other side of the river at Pto. Salgar where ADENAVI used a zero-level of 168.00 m above M.S.L. According to a levelling to the bench-mark of the IGAC, the zero-level of the gauge at Pto. Salgar appeared to be 165.83 m above M.S.L. The water-levels during the measurements of the Julius Berger Konsortium have been corrected to this latter zero-level, starting from the former zero-level of 168.00 m above M.S.L. If the true zero-level of the Berger gauge is used (168.90 m), the discharges are far too small.

The relations are presented in Figure 3.5.5.

b) The Rfo Nare Section (just downstream of the railway bridge)

The relations of this cross-section are given in Figure 3.5.6. It can be seen that the water-level is only related to the height of the benchmark situated near the gauge, but not yet to M.S.L. This should still be done by levelling to the IGAC benchmark established near the railway bridge.

c) The Pto. Inmarco Section (Rfo Magdalena, km 773)

## II, 3.5

The relations of this section are presented in Figure 3.5.7. Some remarks have to be made regarding the stage-slope relation (see Figure 2.4.3). The plots of the water-level gradient versus the water-level show two branches, one for the rising stages and one for the falling stages. In accordance with the example given in Figure 2.4.3b, the plots turn counter-clockwise as is the case for a stage-slope relation of a gauge in the main river. But for such a station a definite stage-slope relation would be expected with only a small scatter of the plots. The considerable scatter of the plots given in Figure 3.5.7 must, therefore, be attributed to the discharge of the R $\bar{f}$ o Nare. This becomes clear when studying the aerial photograph of this river-stretch.

In the photograph the distinction between the discharge of the R $\bar{f}$ o Magdalena and that of the R $\bar{f}$ o Nare is clearly marked. It can also be seen that the gauge near the cement factory in Pto. Inmarco is not located in the main river (the R $\bar{f}$ o Magdalena) but, in fact, in the tributary (the R $\bar{f}$ o Nare) upstream of the confluence with the main river. Consequently, it must be considered why the stage-slope relation at Pto. Inmarco has, on the one hand, the character of such a relation of a main river (turning counter-clockwise) and, on the other hand, the character of a tributary (considerable scatter). If the assumption is made that the rainfall in the R $\bar{f}$ o Nare valley and the R $\bar{f}$ o Magdalena valley occurs, generally, at the same time but that the run-off (discharge) in the R $\bar{f}$ o Nare increases and decreases earlier than in the R $\bar{f}$ o Magdalena, the following explanation can be given (Figure 3.5.8):

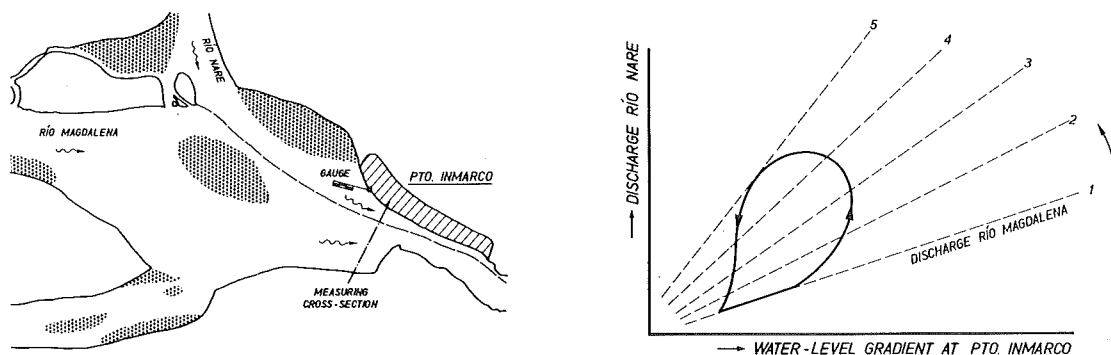


Figure 3.5.8 Schematization of Stage-Slope Relation at Pto. Inmarco

The lines marked 1 to 5 in Figure 3.5.8 indicate stage-slope relations (Figure 2.4.3b) for increasing discharges of the R $\bar{f}$ o Magdalena. If the gauge at Pto. Inmarco depends on the discharge of both the R $\bar{f}$ o Magdalena and the R $\bar{f}$ o Nare, the stage-slope relation at Pto. Inmarco will indeed turn counter-clockwise (as in the main river) but consist of two branches (as in the tributary).

It can be concluded that the gauge at Pto. Inmarco may best be shifted downstream to the narrow section of the R $\bar{f}$ o Magdalena, out of the influence of the R $\bar{f}$ o Nare.

For the computations of the relation between the roughness and the water-level, the average stage-slope curve has been used, as indicated in Figure 3.5.7.

d) The Ballena Section (Río Magdalena, km 712)

The relations of this section are given in Figure 3.5.9. Again a remark must be made regarding the stage-slope relation. It appears that this relation shows the character of a tributary (Figure 2.4.3c); namely, turning clockwise and consisting of two branches (although the branches are less pronounced than for the Pto. Inmarco Section). Similarly to the explanation given for the Pto. Inmarco Section, the stage-slope relation for a gauge in the Río Magdalena upstream of the confluence should indeed turn clockwise (although in Figure 3.5.9 the water-level gradients are plotted versus the water-levels at Pto. Berrío, the character of this relation will not change if the slopes are plotted against the local water-levels at Ballena; see Table 3.3.1). However, this implies that the discharge of the Río Regla must, at least, be a considerable percentage of the discharge of the Río Magdalena. This does not correspond to the available data of the Río Regla, which is thought to be only a minor tributary, although unfortunately no data are available of the situation just before and after a considerable rainy period. Consequently, no proper explanation can yet be given. The average stage-slope curve as presented in Figure 3.5.9 has again been used for the roughness computations.

The stage-transport relations show that the computed transports are about twice as large as the measured transports. Something more about this will be said later when the Río Nuevo Section is discussed.

c) The Río Nuevo Section (Río Magdalena, km. 705.5)

The relations are presented in Figure 3.5.10. The stage-transport relation of this section shows that the computed transports are at least half the size of the measured transports. The question arises whether one has to rely for both the Ballena and the Río Nuevo Section more heavily on the measured transports than on the computed transports. If only the measured transports of these sections are considered, it appears that for both sections more or less the same stage-transport relation exists. It seems logical that the amount of sediment transported through the Ballena Section is also transported through the Río Nuevo Section. On the other hand, if the computed stage-transport relations hold, a considerable sedimentation in the river-stretch between the two sections would occur. This is definitely not true and, therefore, it is better to rely more heavily on the measured stage-transport relation.

Consequently, it must be considered why the computed stage-transport relation can deviate so considerably from the measured relation. The first explanation may be found in the analysis of Einstein and Barbarossa (1952) [29] who have pointed out that an average water-level gradient and average cross-section dimensions obtained along a river stretch should be used instead of the water-level gradient and cross-section observed at one location. The irregularities and non-uniformities which, invariably, characterize river channels will then be less pronounced. The average water-level gradient between the main gauge-stations Pto. Berrío and Barrancabermeja will be in the order of  $35 \times 10^{-5}$ . This value differs considerably from the locally-measured water-level gradients in the Río Nuevo Section (15 to  $30 \times 10^{-5}$ ) and in the Ballena section (40 to  $60 \times 10^{-5}$ ). Using the average water-level gradient

## II, 3.5

would therefore lead to an increase of the computed sediment transport in the Rfo Nuevo Section and to a decrease of the transport in the Ballena Section.

Also the selected diameter of the bed material will have its influence on the computed stage-transport relations. It has been mentioned already in Para. 3.3.6 that it was difficult to establish the average grain-size diameter in the Rfo Nuevo Section. Table 3.3.1 shows that the used  $\bar{D}_{50}$  for the Rfo Nuevo Section is at least twice as large as that of the Ballena Section. Studying the transport equation of Engelund and Hansen it follows that the computed sediment transport is inversely proportional to the chosen  $\bar{D}_{50}$ . Consequently, if the average grain-size diameter of the bed material in the river-stretch between the Ballena and the Rfo Nuevo Sections is assumed to be more or less the average of the selected diameters for the two sections, the computed sediment transport in the Ballena Section would decrease and that in the Rfo Nuevo Section increase.

Both considerations tend in the same direction, and it is therefore logical to use in the morphological computations for both sections the same stage-transport relation, based on the measurements only.

### f) The Rfo Sogamoso Confluence

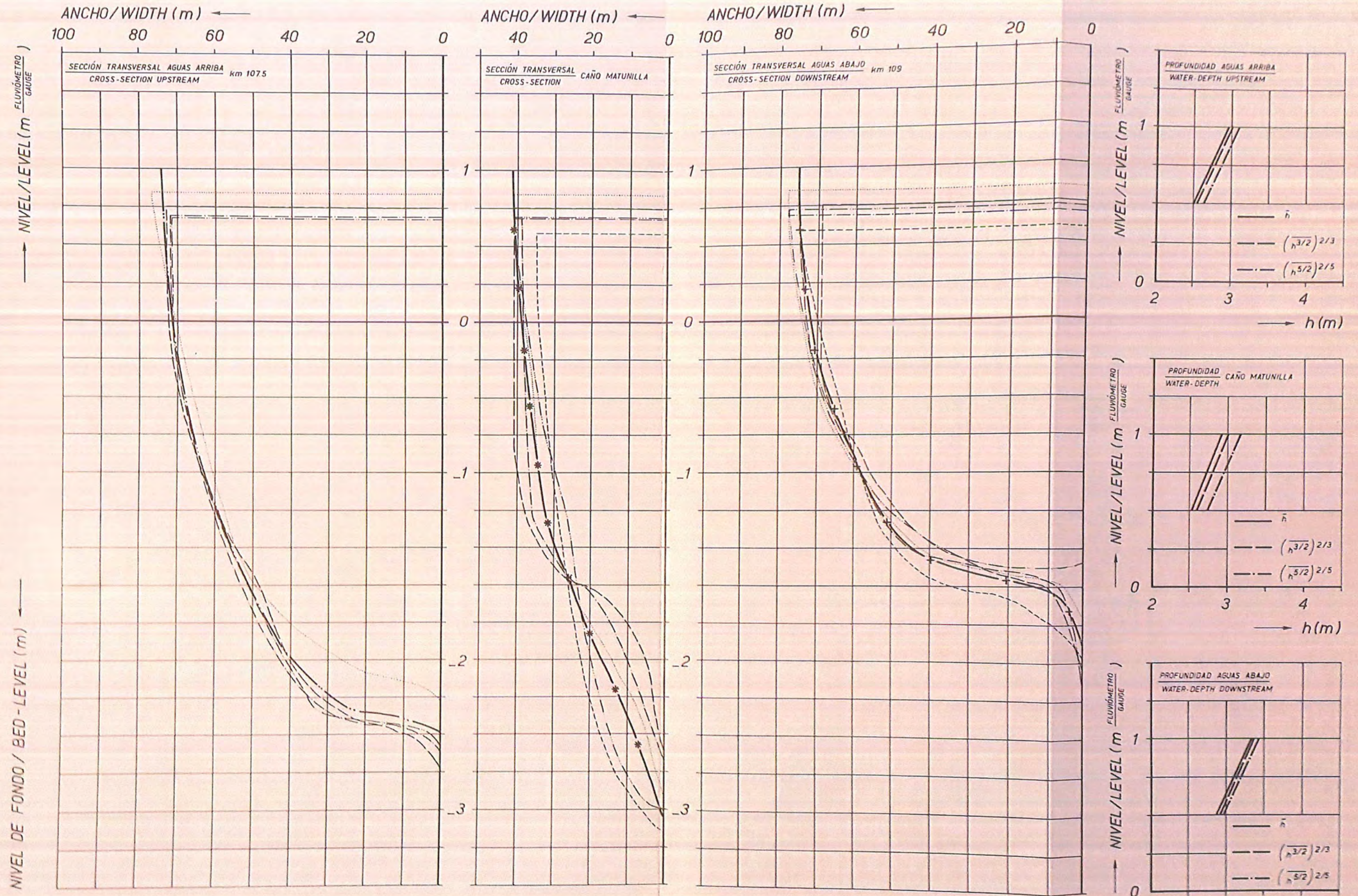
The relations presented in Figure 3.5.11 only concern the measuring station in the Rfo Magdalena (km 614), upstream of the confluence with the Rfo Sogamoso. Although measurements have also been made in the Rfo Sogamoso itself and in the Rfo Magdalena downstream of the confluence, no relations are given for these two sections. The measuring cross-section in the Rfo Sogamoso was not a fixed section because of the shallowness of the Rfo Sogamoso. Therefore, the cross-sections used for the consecutive measurements differ considerably in width, and hence in depth too. Moreover, the sediment transport could only be measured at high water stages. Nor is the measuring cross-section in the Rfo Magdalena downstream of the confluence with the Rfo Sogamoso at a fixed location. The river consists of a number of branches, separated by islands. The sediment transport has only been measured once, at high water stages in the main branch along the right bank.

### g) The Vijagual Section (Rfo Magdalena, km 535)

The relations of this section are given in Figure 3.5.12. In the plotted discharges, measured by the Mission, the discharge of the minor secondary branch along the left bank is also included. The sediment transport measurements only refer to the main branch along the right bank.

### h) The Calamar Section (Rfo Magdalena, km 91)

The relations of this section are given in Figure 3.5.13.



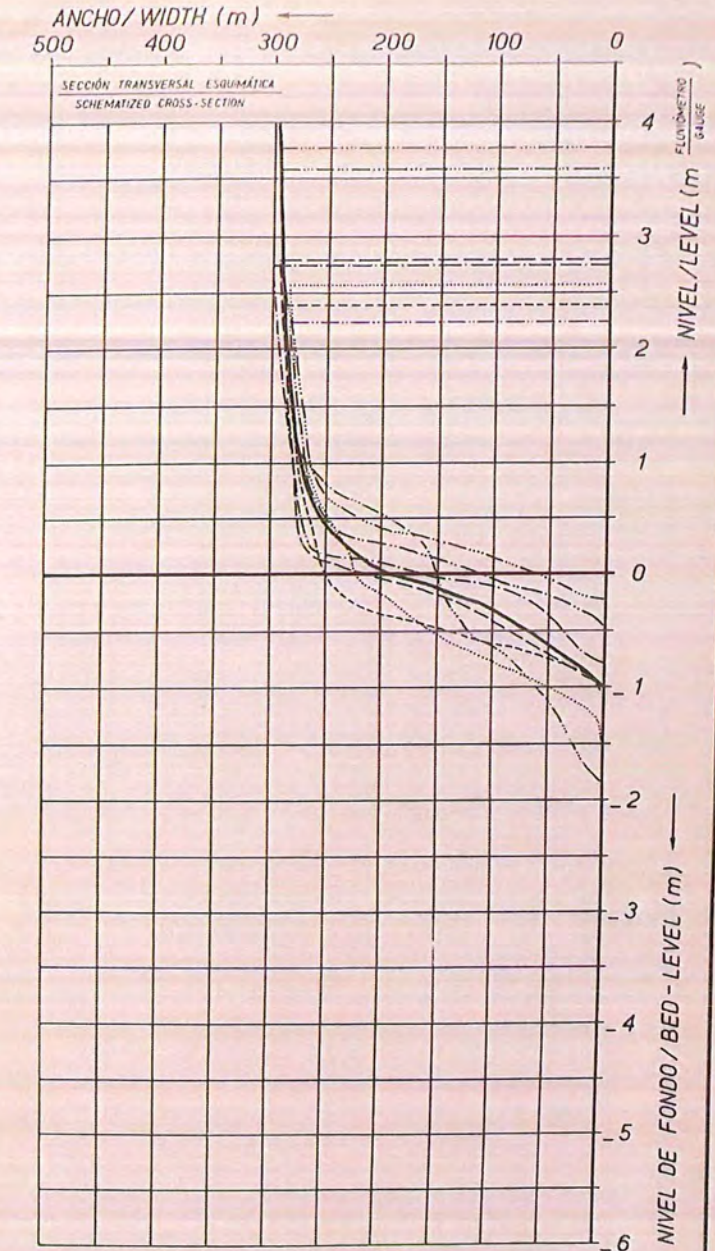
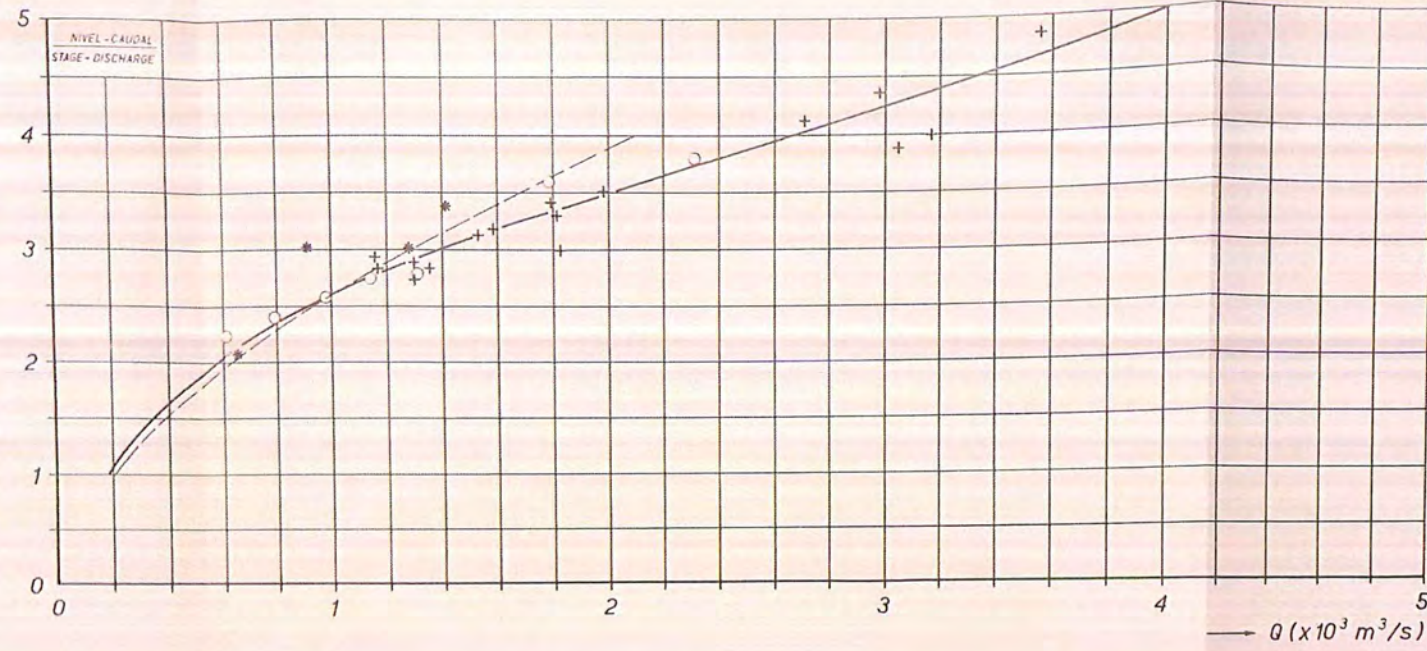
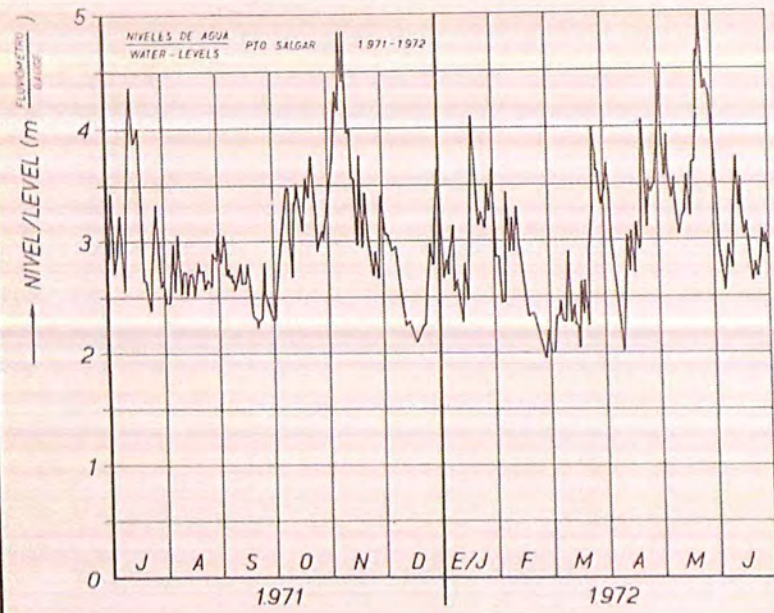
BIFURCACIÓN / BIFURCATION **CAÑO LEQUERICA** CANAL DEL DIQUE km 108

DATOS DE LAS SECCIONES TRANSVERSALES ESQUIMÁTICAS / DATA OF SCHEMATIZED CROSS-SECTIONS

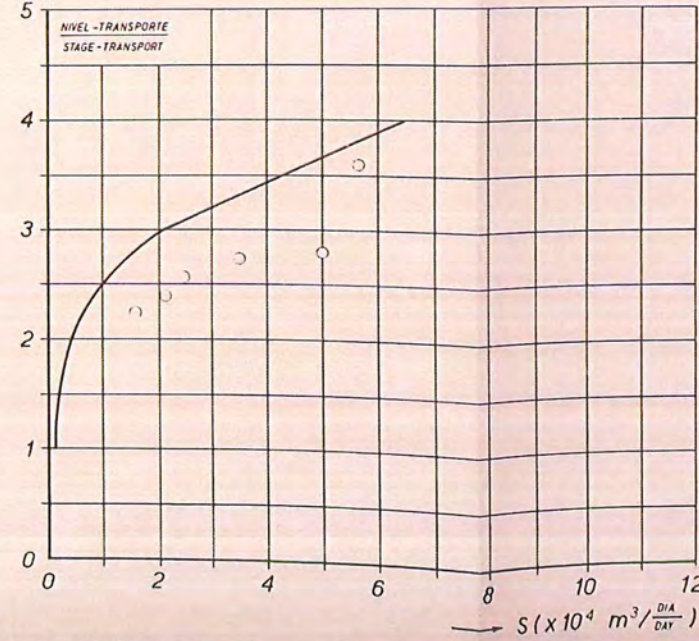
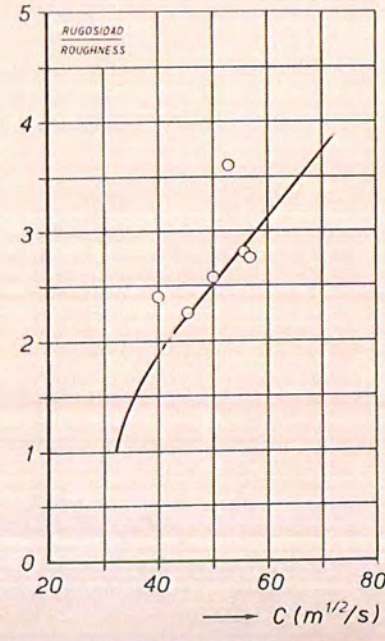
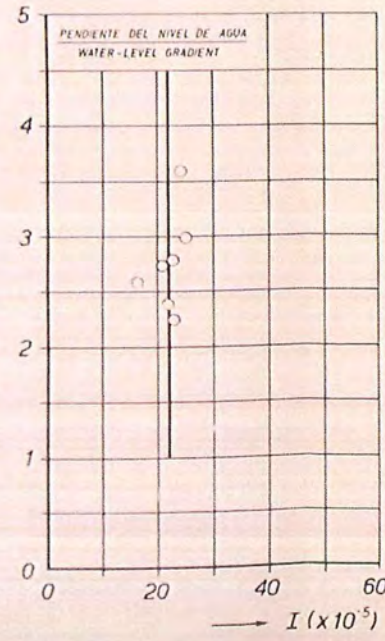
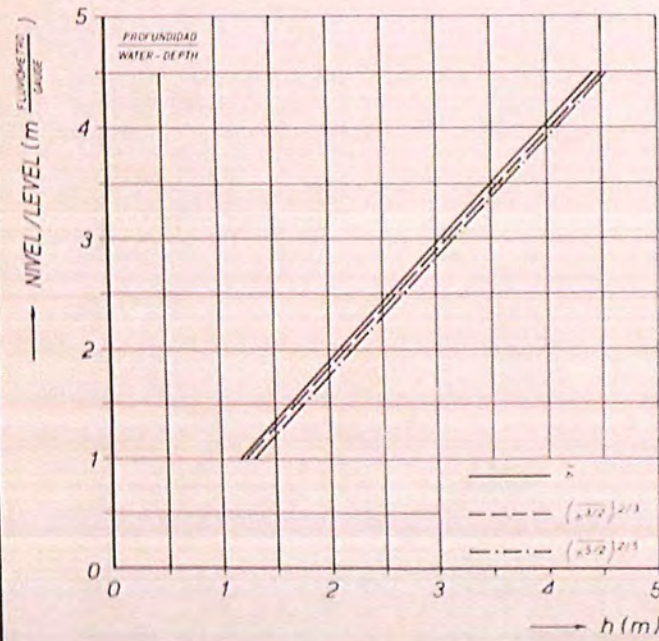
NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT **FIG.3.5.19**

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA

--- 26 - I - 1970	--- 26 - III - 1972
--- 3 - II - 1970	
--- 15 - II - 1971	
--- 30 - II - 1971	— PROMEDIO / MEAN



CERO FLUVIOMÉTRICO / ZERO GAUGE PTO. SALGAR 165.83 m. SMM MSL



DATOS AFOROS / DISCHARGE DATA	
•	JULIUS BERGER 1923-1924
—	CS 7M 1970
+	SMM 1970-1972
○	MATCH 1971-1972

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA			
---	9-VII-1971	---	8-I-1972
---	8-VIII-1971	---	8-II-1972
---	9-IX-1971	---	8-III-1972
---	19-X-1971	---	PROMEDIO / MEAN

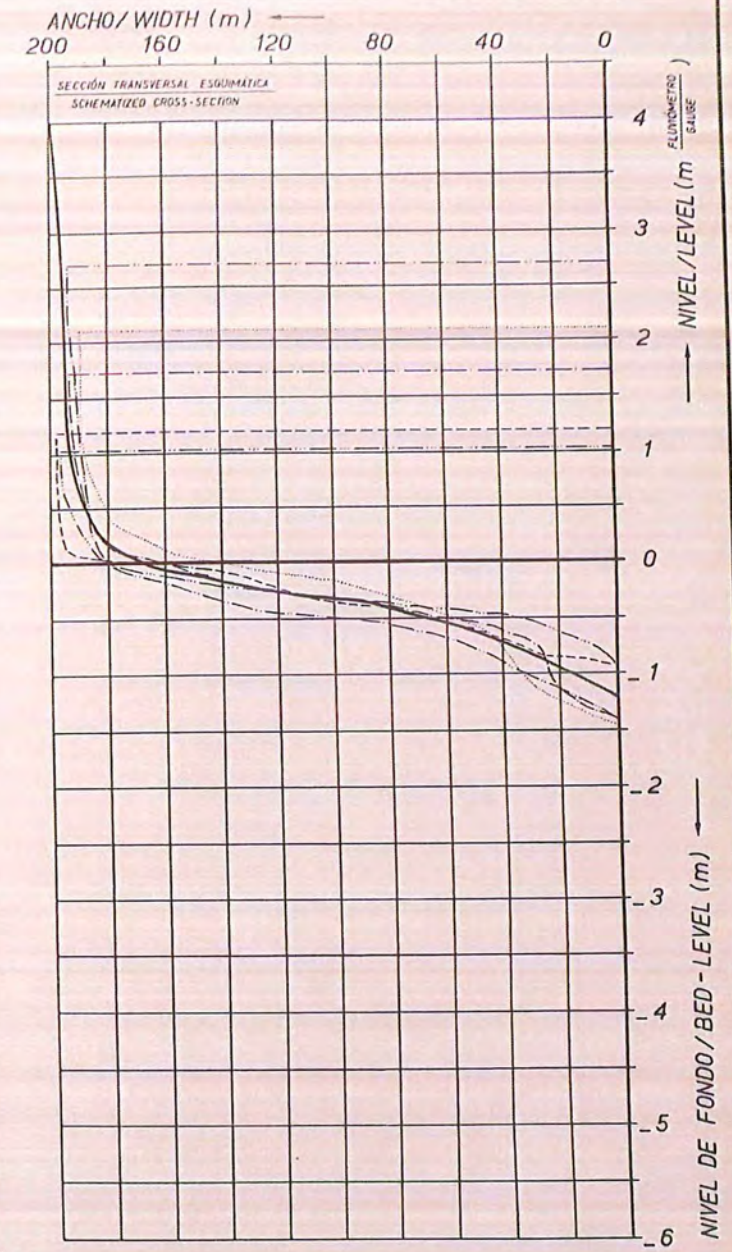
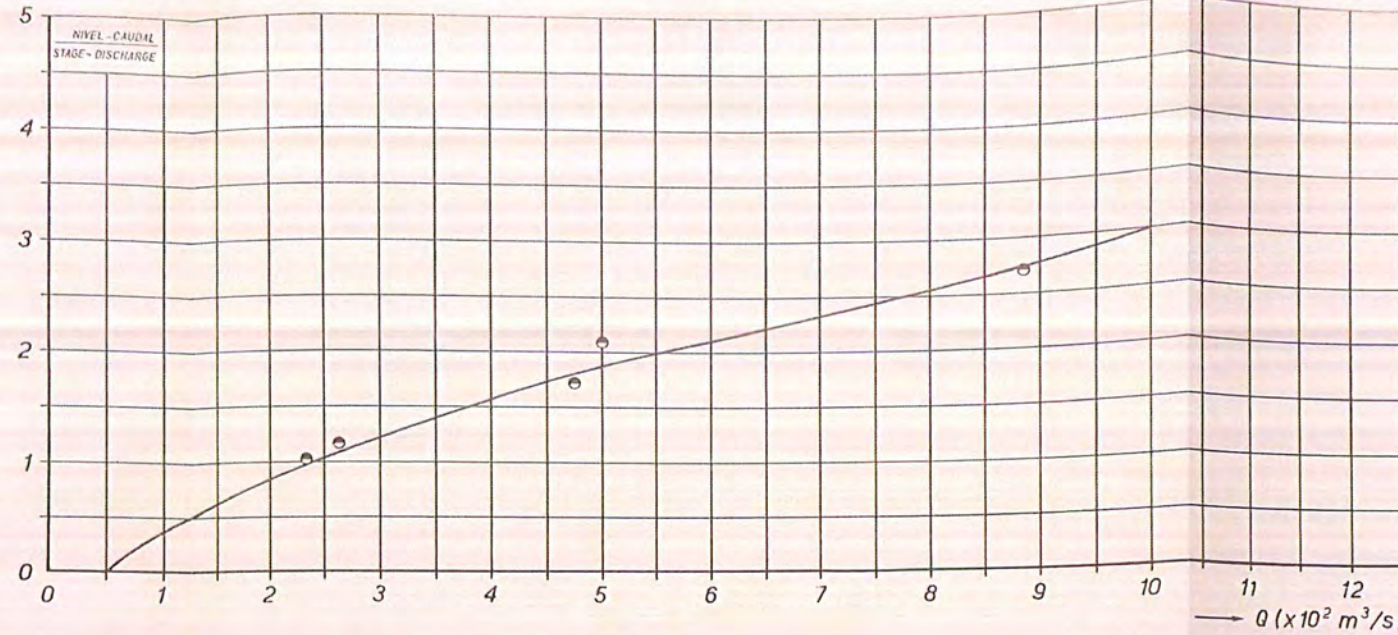
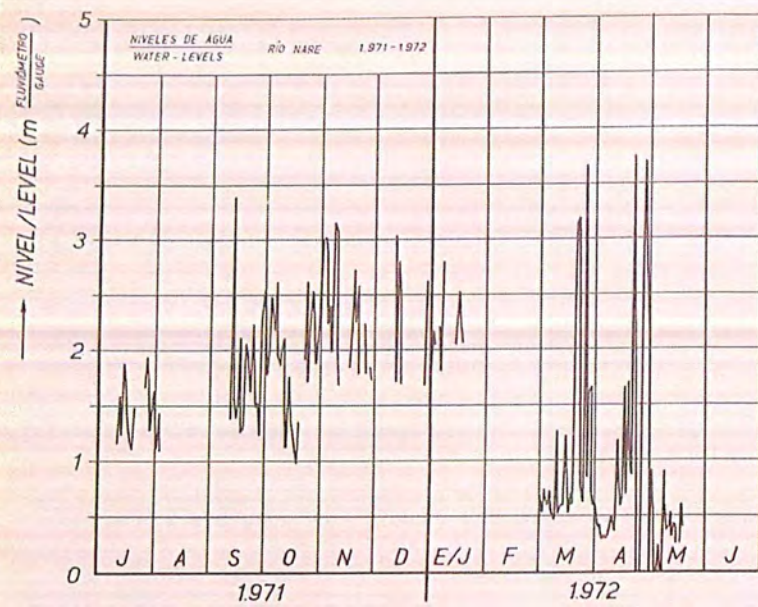
DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA					
$\bar{D}_{35}$	340 $\mu m$	$\bar{D}_{50}$	500 $\mu m$	$\bar{D}_{85}$	1390 $\mu m$

**LA DORADA RÍO MAGDALENA km 884**

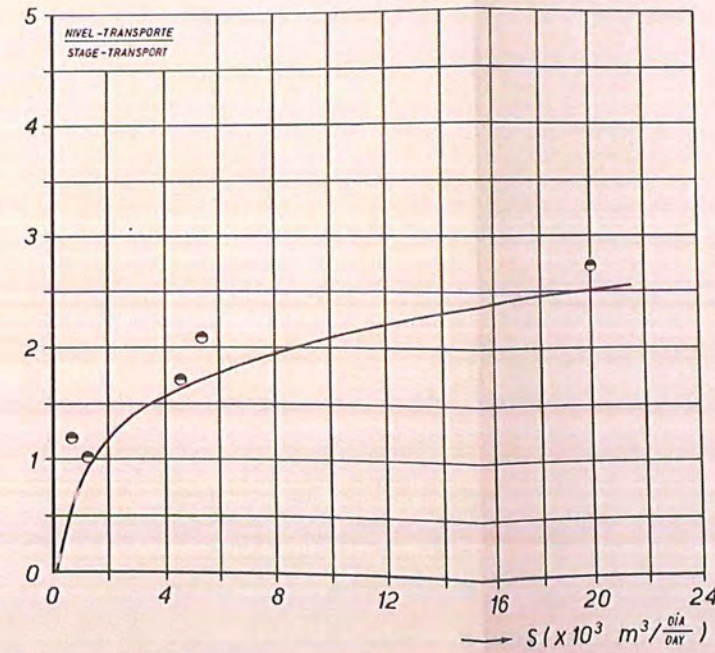
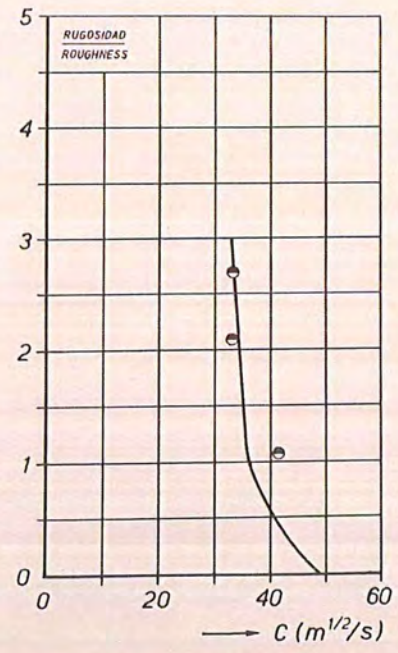
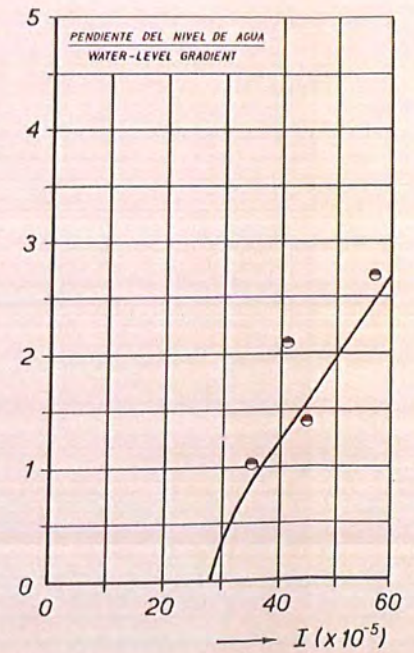
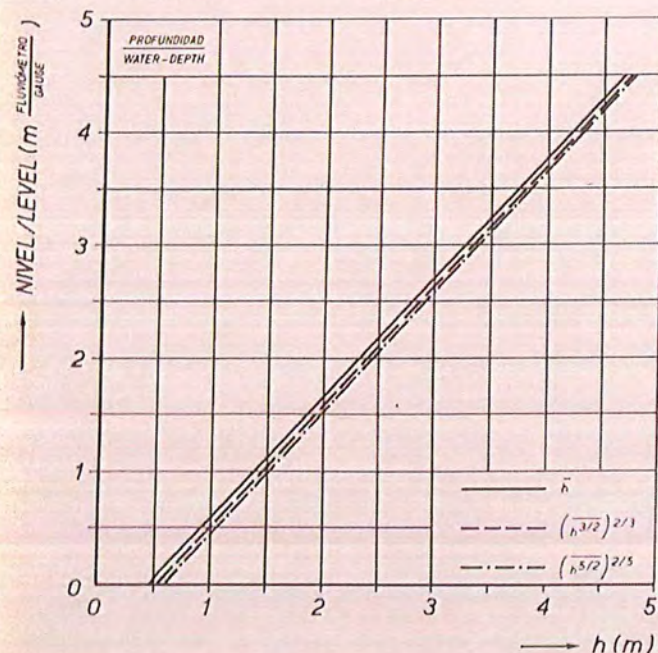
DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.5.5



CERO FLUVIÓMETRO RÍO NARE 7.30m BAJO BM MCH-1ª  
 ZERO GAUGE RÍO NARE 7.30m BELOW BM

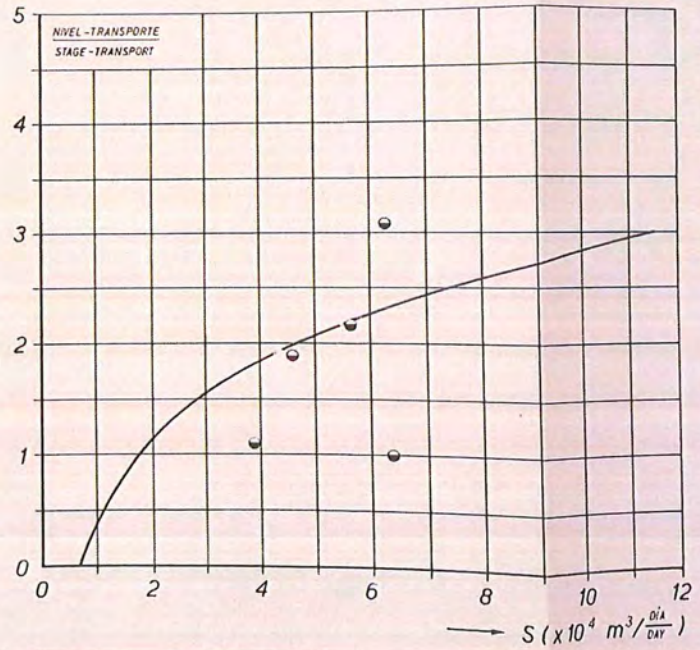
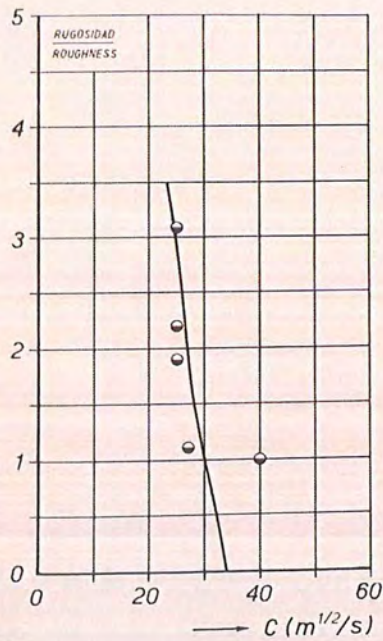
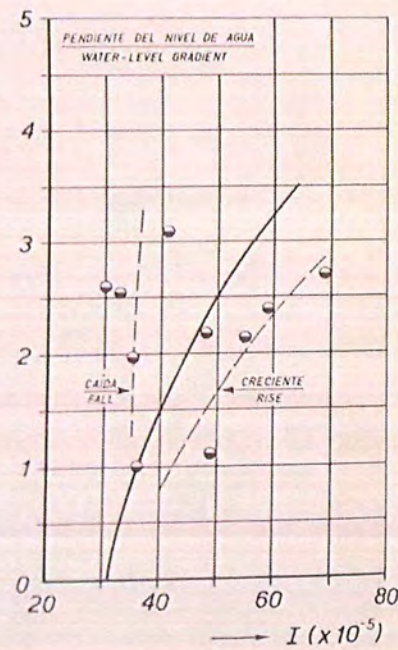
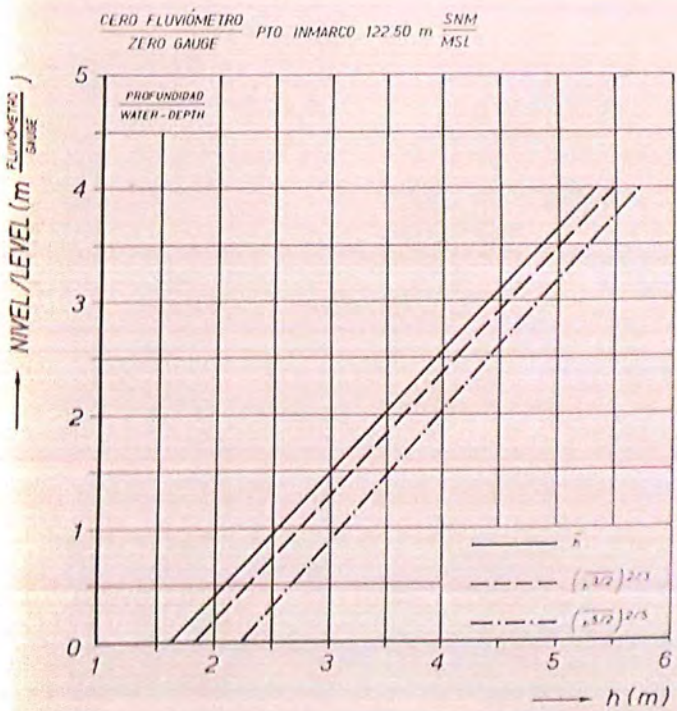
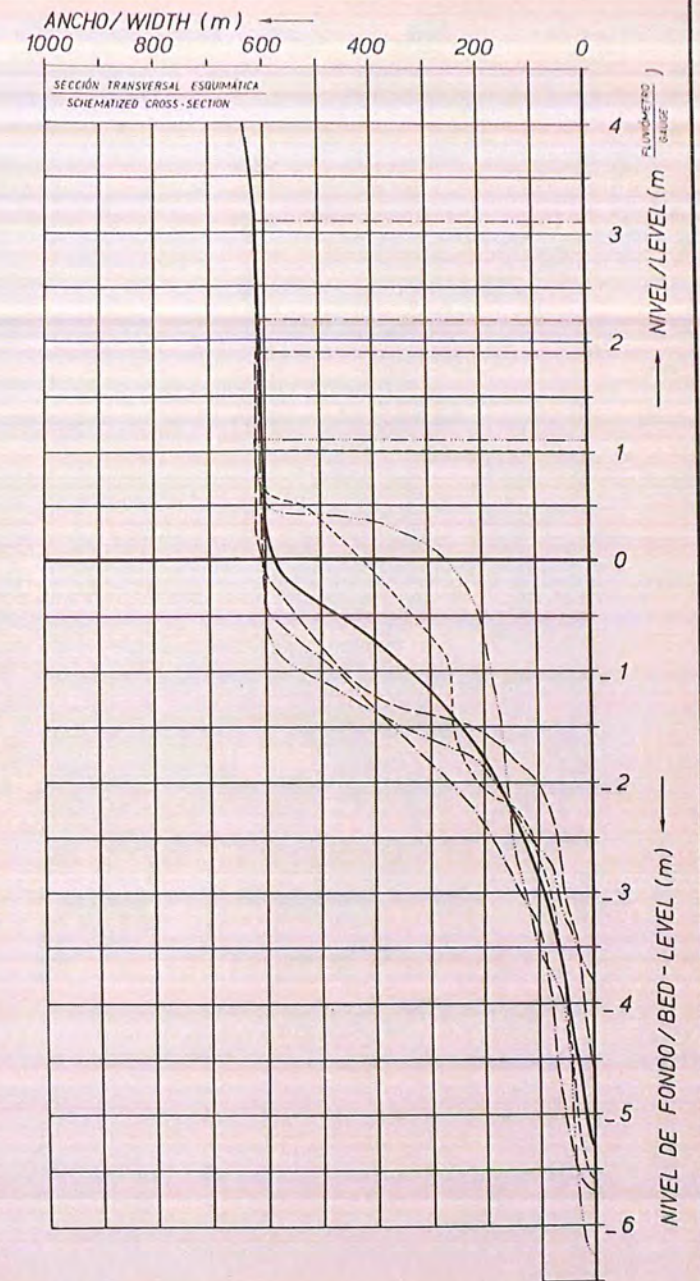
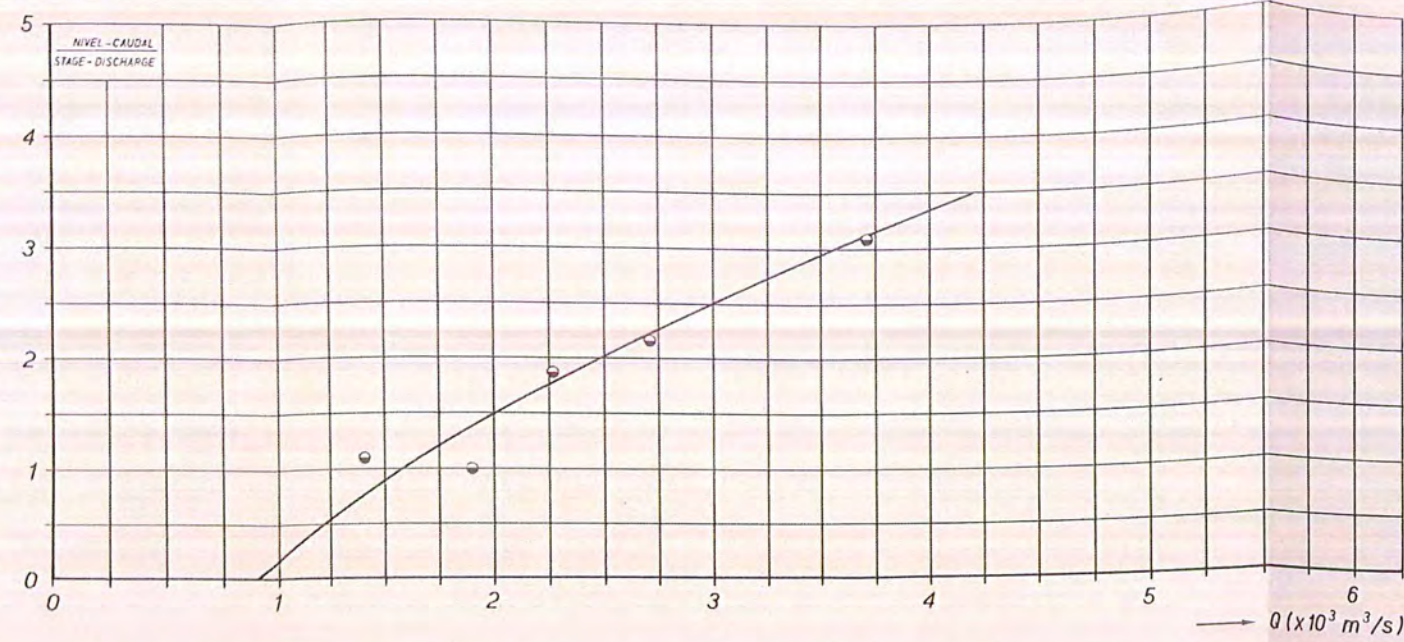
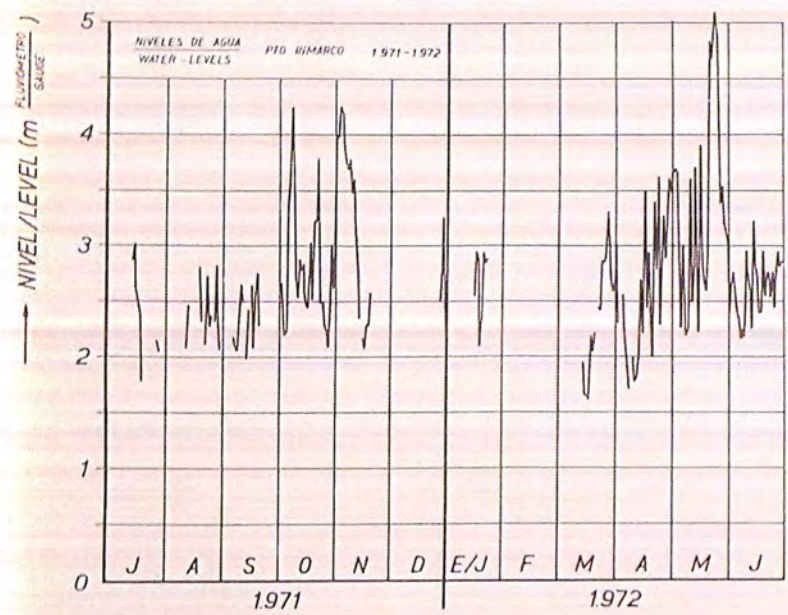


DATOS AFOROS / DISCHARGE DATA  
 ● MITCH 1971-1972

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA  
 --- 11-VII-1971 --- 3-III-1972  
 --- 13-VIII-1971  
 --- 13-X-1971  
 --- 23-XI-1971 ——— PROMEDIO / MEAN

DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA  
 $\bar{D}_{35} = 670 \mu\text{m}$ ;  $\bar{D}_{50} = 1080 \mu\text{m}$ ;  $\bar{D}_{85} = 2545 \mu\text{m}$

**RÍO NARE**  
 DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA  
 NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT FIG. 3.5.6



DATOS AFOROS / DISCHARGE DATA

● MICH 1971-1972

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA

--- 12-VIII-1971 --- 11-III-1972  
 --- 12-IX-1971  
 --- 22-X-1971  
 --- 11-III-1972  
 ——— PROMEDIO / MEAN

DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA

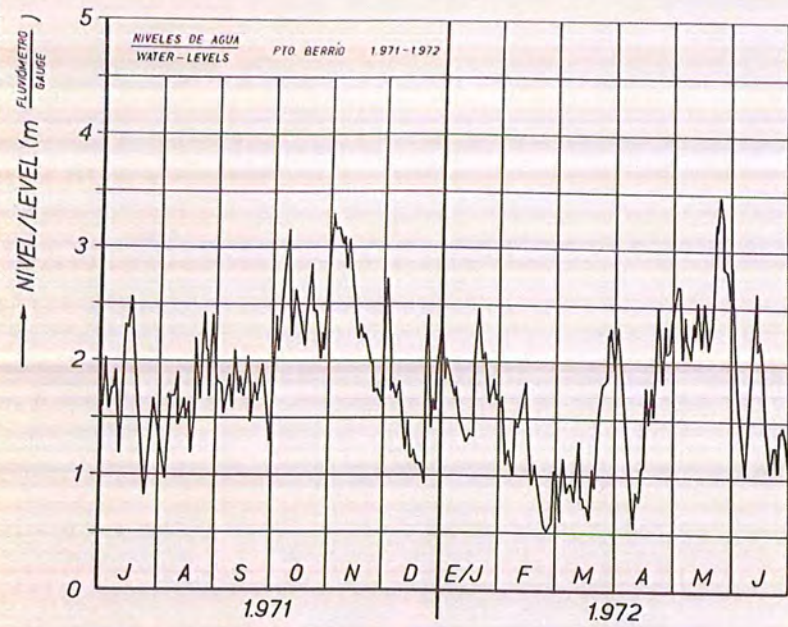
$\bar{D}_{35} = 575 \mu m$ ,  $\bar{D}_{50} = 1050 \mu m$ ,  $\bar{D}_{65} = 1765 \mu m$

**PUERTO INMARCO** RÍO MAGDALENA km 773

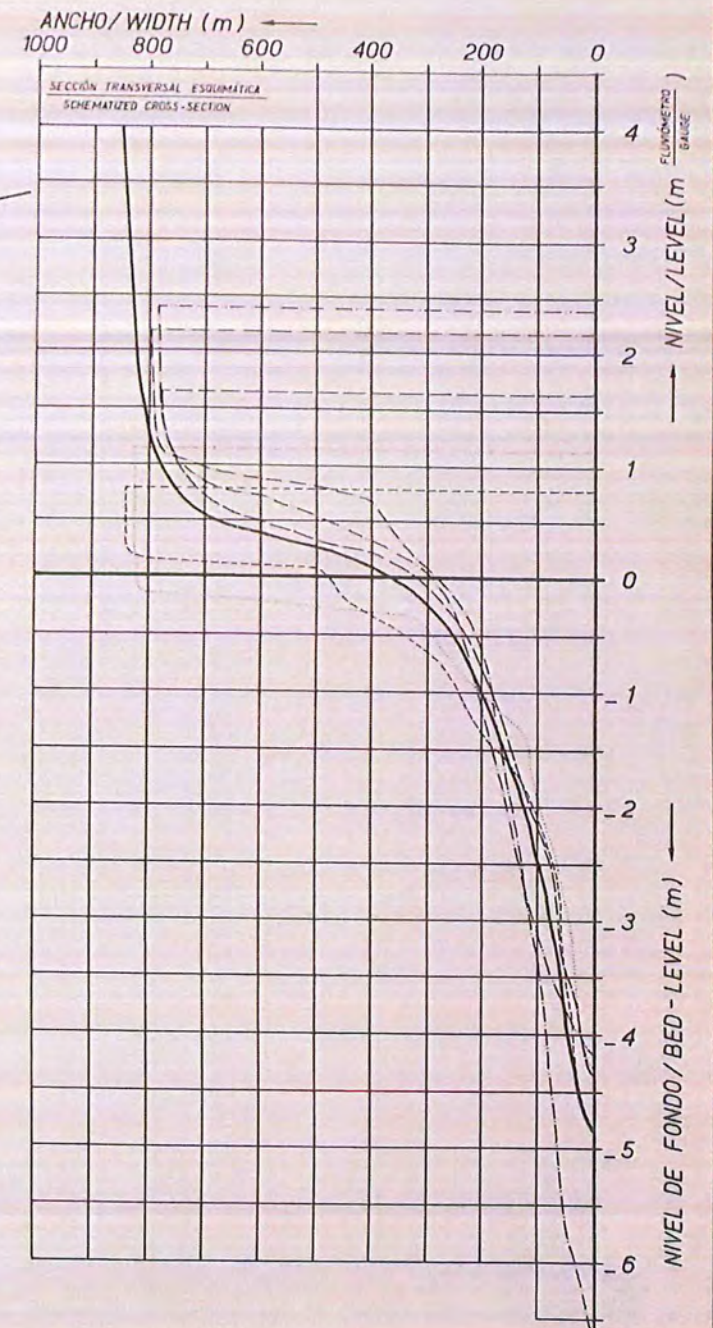
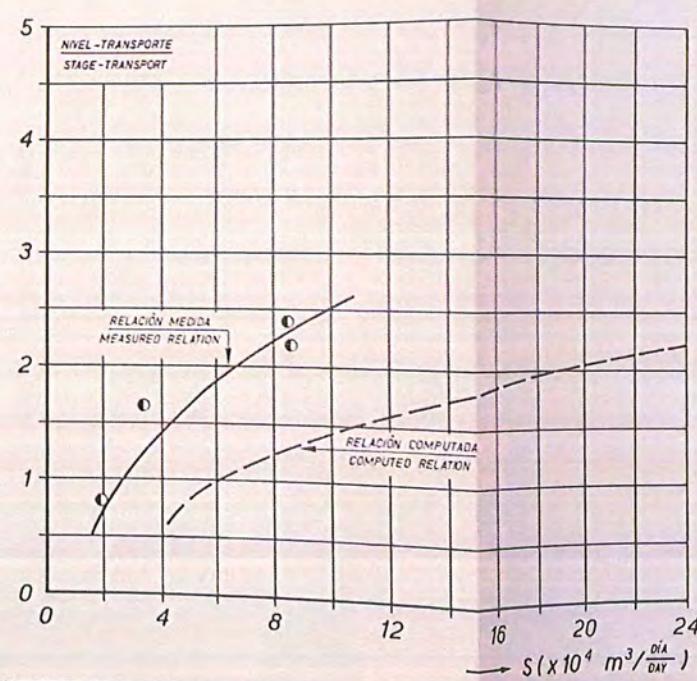
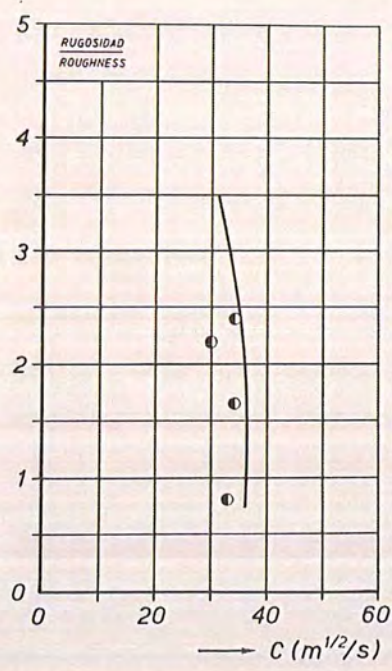
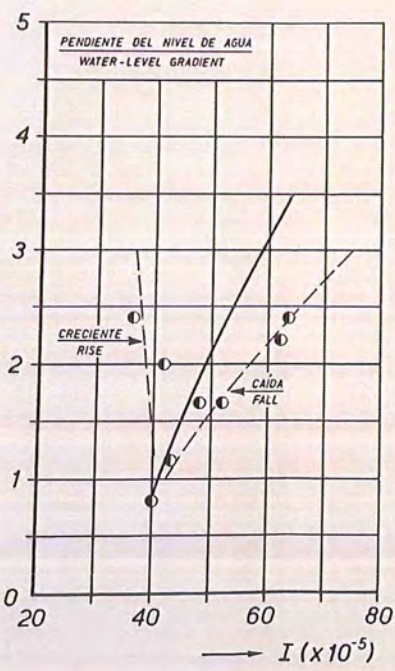
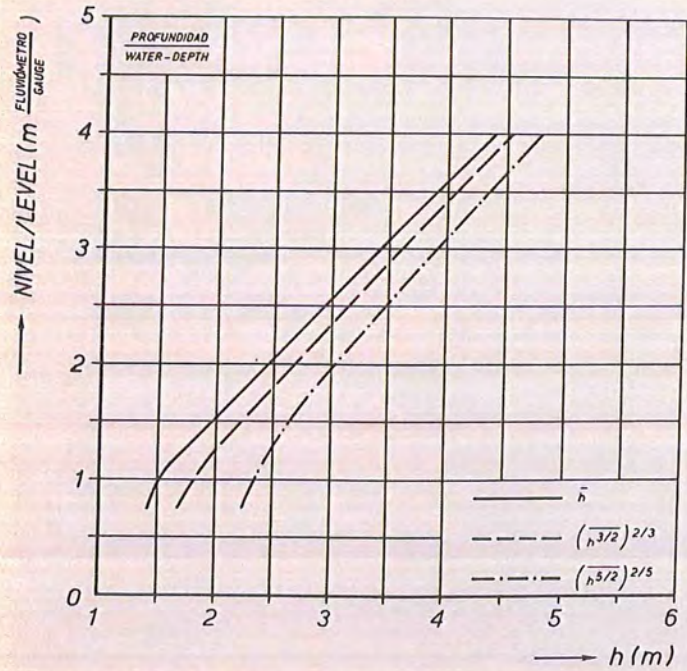
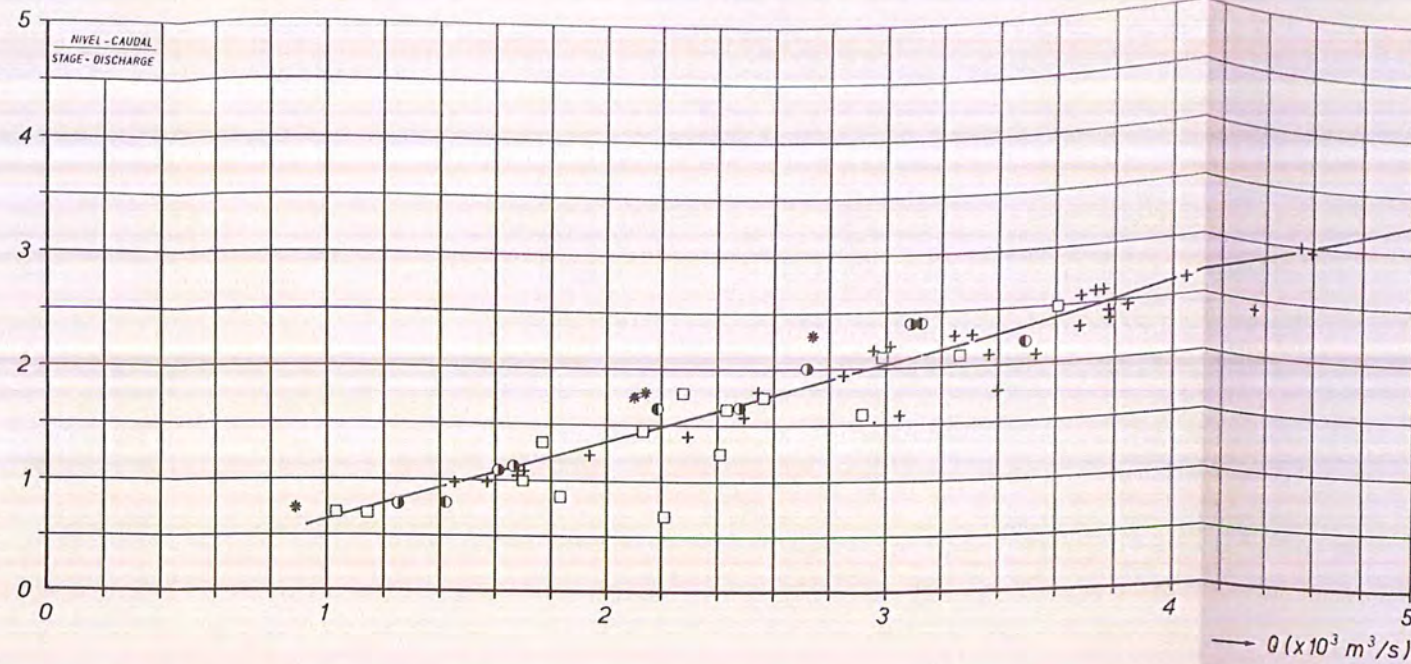
DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.5.7



CERO FLUJIMETRO PTO BERRÍO 106.44 SNM  
ZERO GAUGE MSL

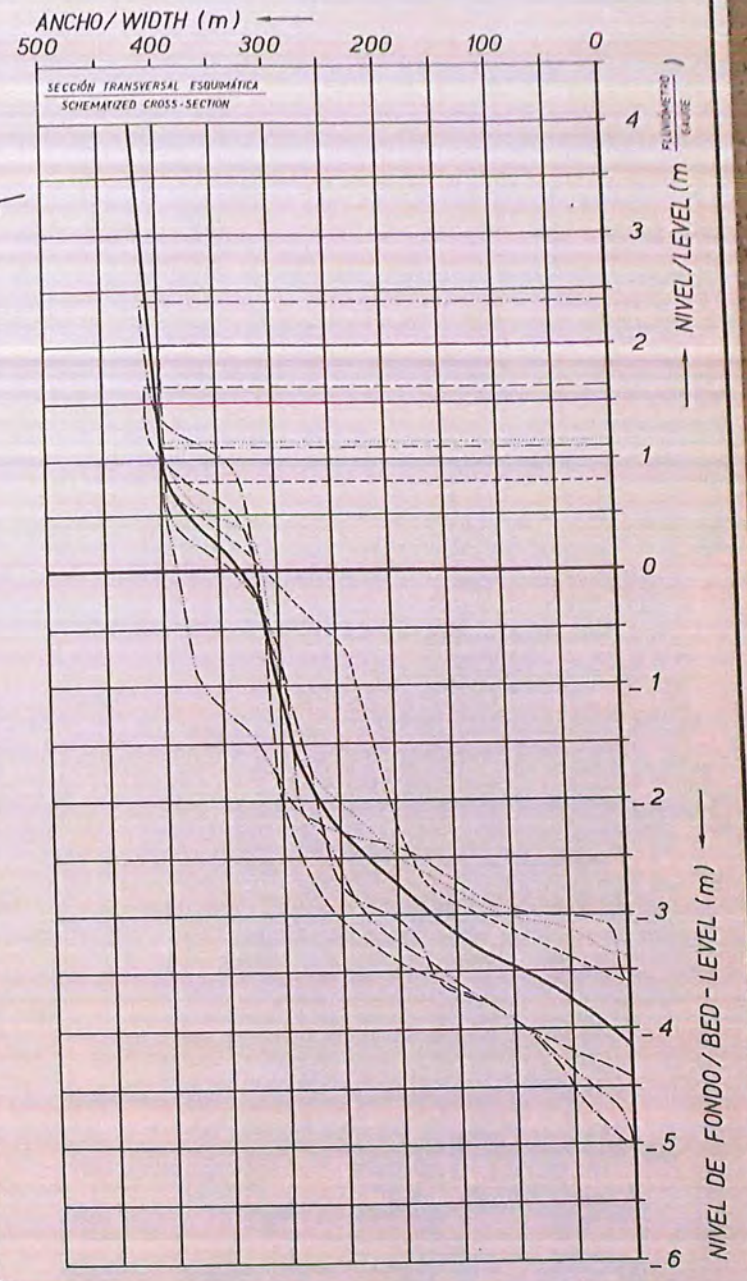
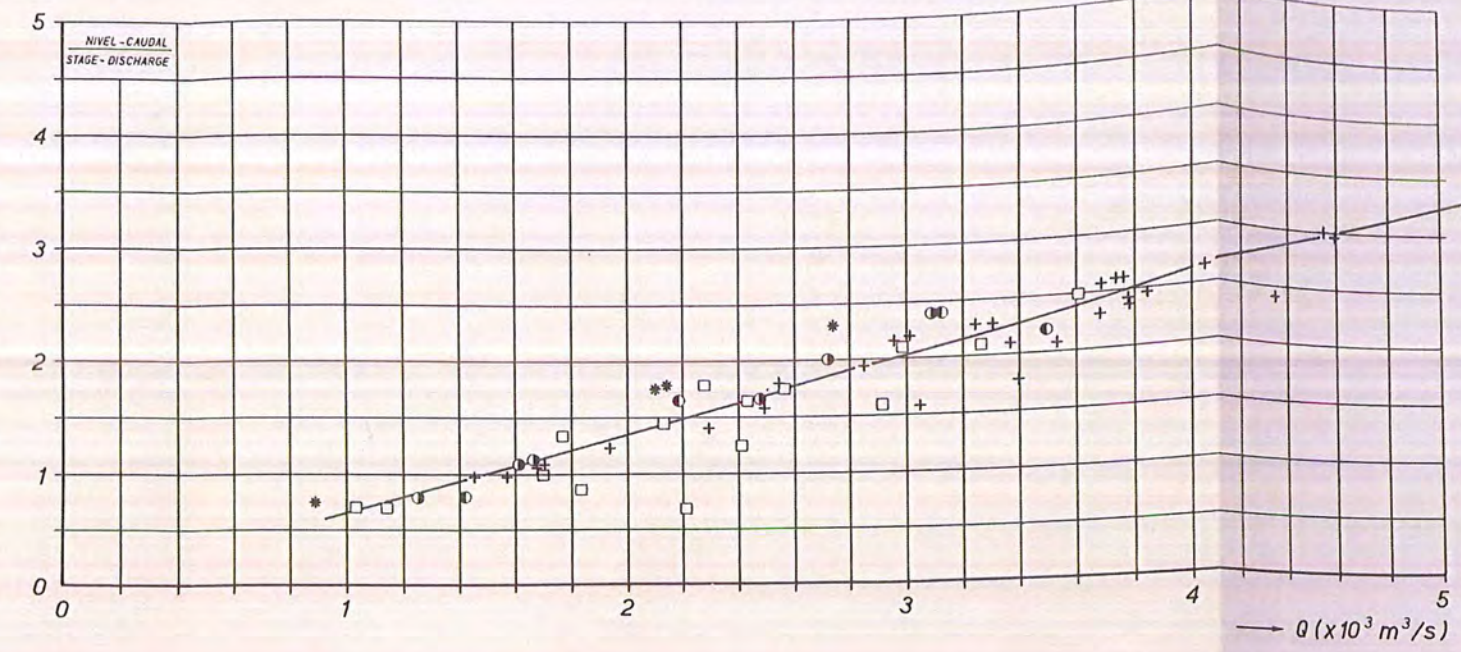
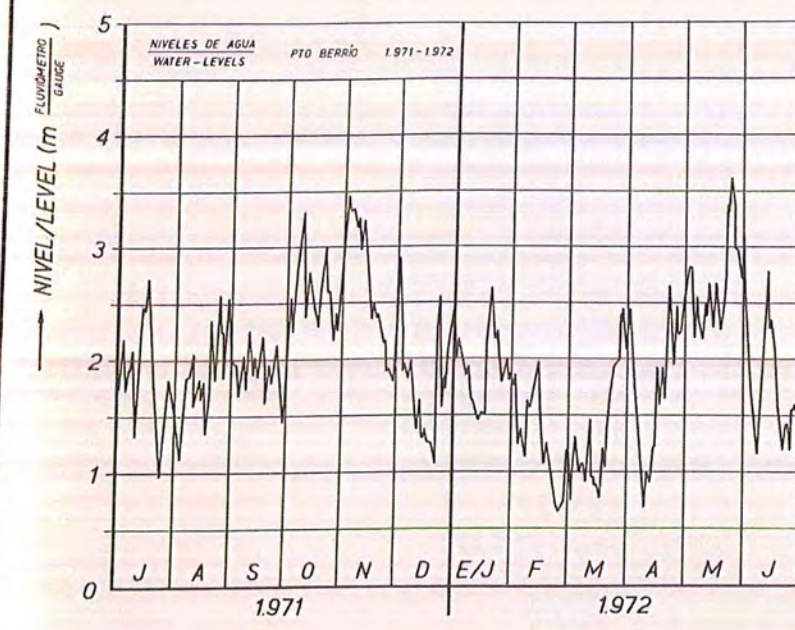


DATOS AFOROS / DISCHARGE DATA  
 \* JULIUS BERGER KONSORTIUM 1.923-1.924  
 □ APRON Y DUQUE LTDA. 1.966  
 + SCMH 1.969-1.972  
 ● MITCH 1.971-1.972

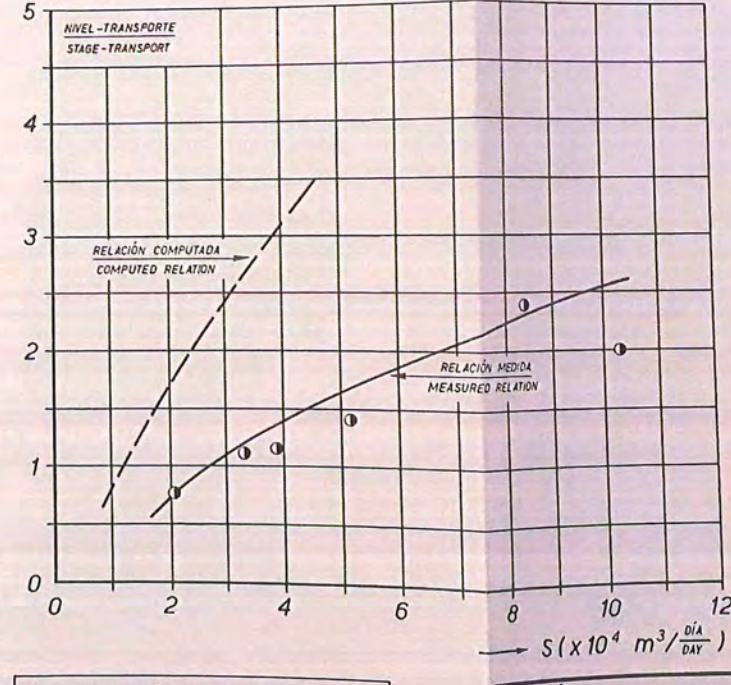
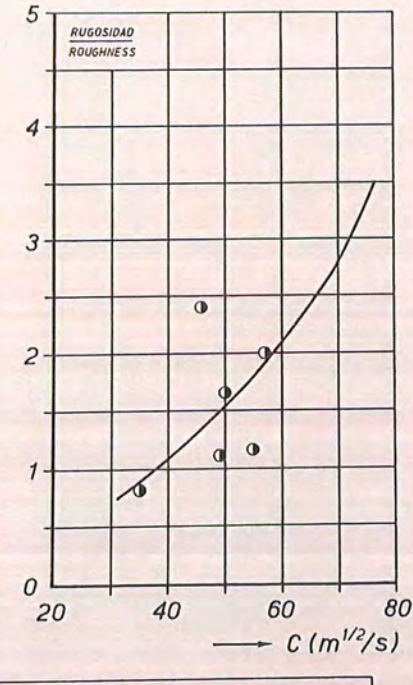
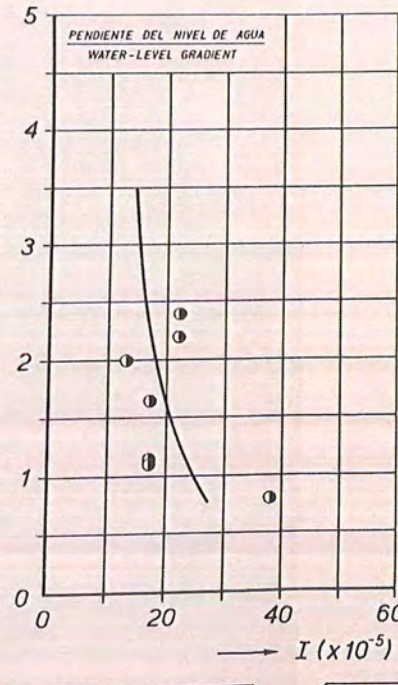
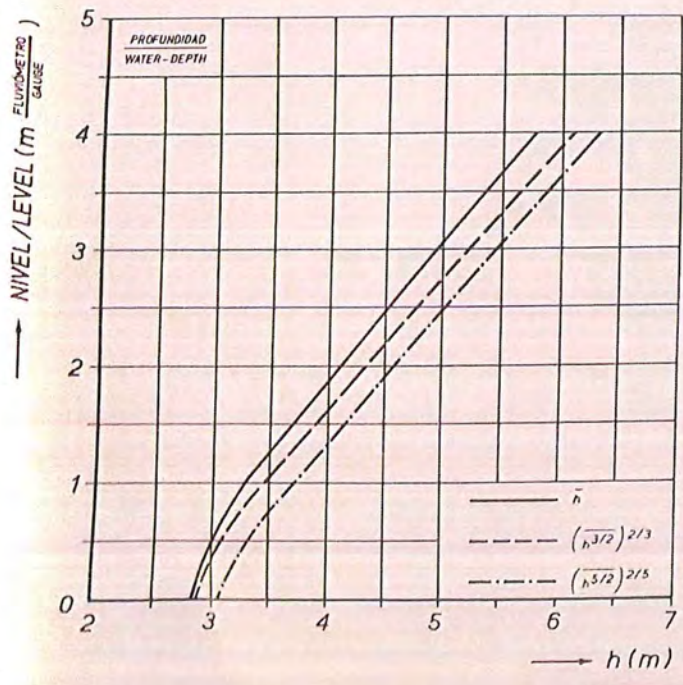
DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA  
 - - - - - 28-VIII-1971 - - - - - 12-III-1972  
 - - - - - 21-IX-1971  
 - - - - - 25-X-1971  
 - - - - - 18-II-1972 - - - - - PROMEDIO / MEAN

DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA  
 $\bar{D}_{35} = 330 \mu\text{m}$ ;  $\bar{D}_{50} = 405 \mu\text{m}$ ;  $\bar{D}_{85} = 580 \mu\text{m}$

**BALLENA** RÍO MAGDALENA km 711.5  
 DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA  
 NEDCO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT FIG. 3.5.9



CERO FLUJIMETRO PTO BERRIO 106.44 m SNM  
ZERO GAUGE MSL



**DATOS AFOROS / DISCHARGE DATA**

- \* JULIUS BERGER KONSORTIUM 1923-1924
- APRON Y DUQUE LTDA 1966
- + SCMH 1969-1972
- MITCH 1971-1972

**DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA**

- 29-III-1971
- 22-IX-1971
- 24-X-1971
- 12-I-1972
- ..... 18-II-1972
- ..... 13-III-1972
- PROMEDIO / MEAN

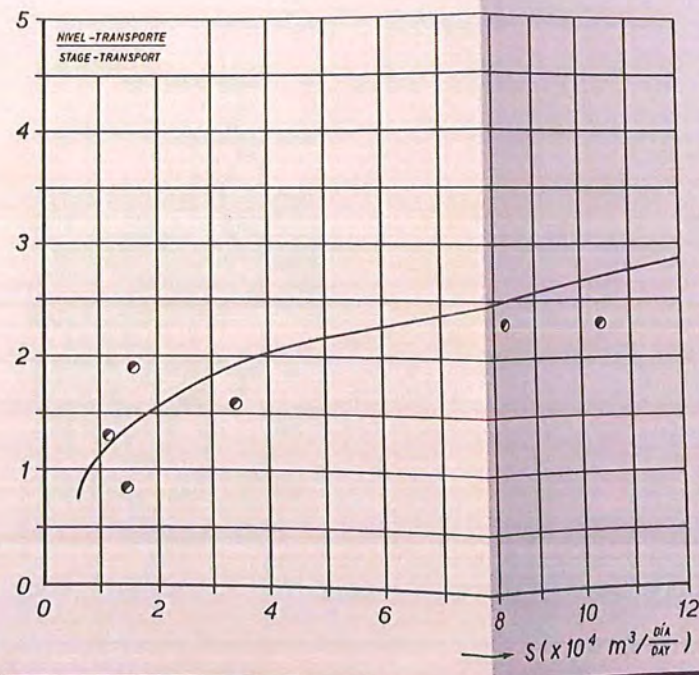
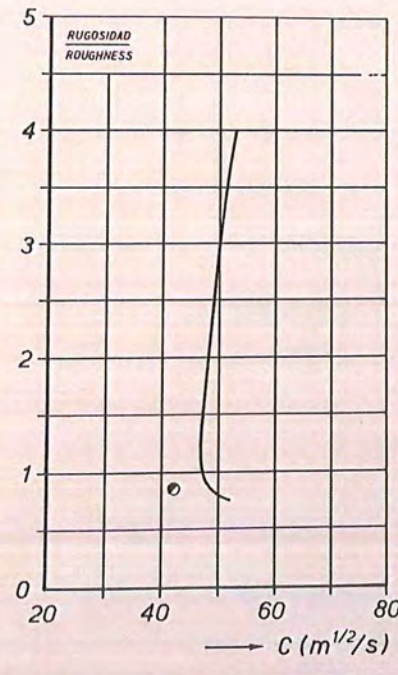
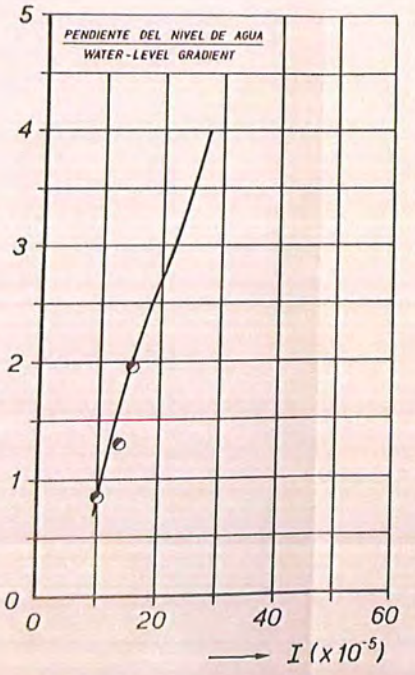
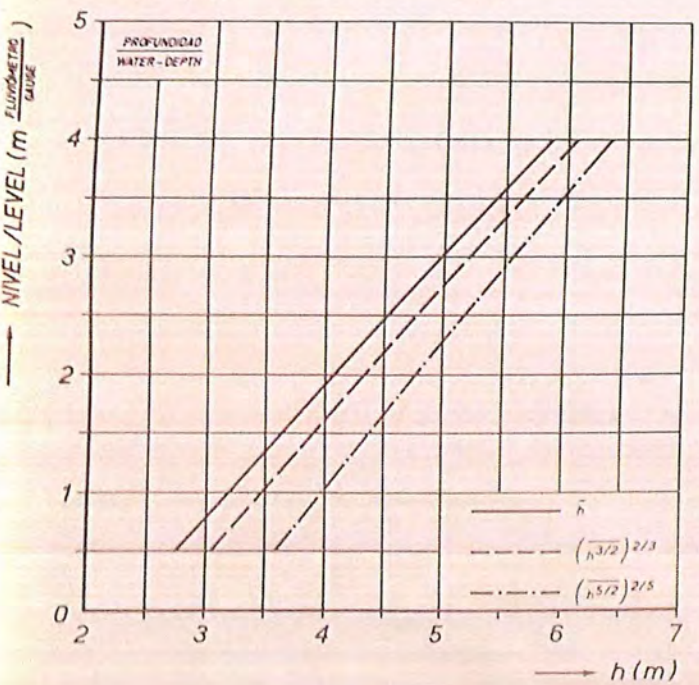
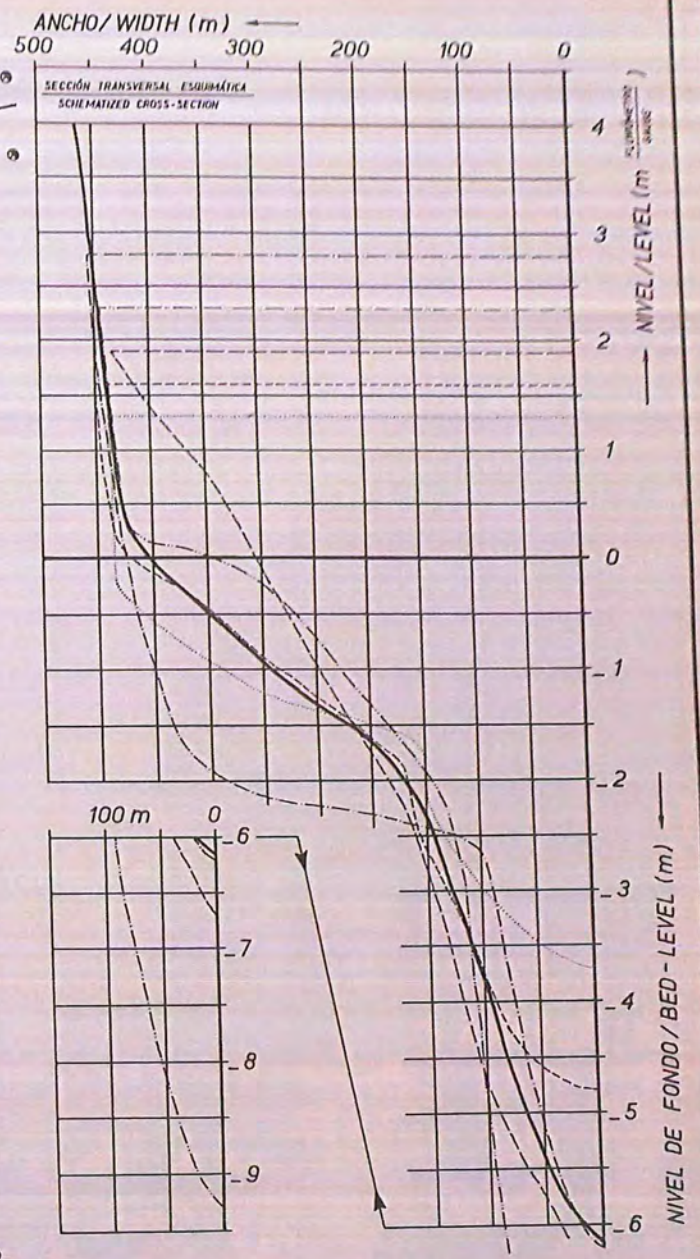
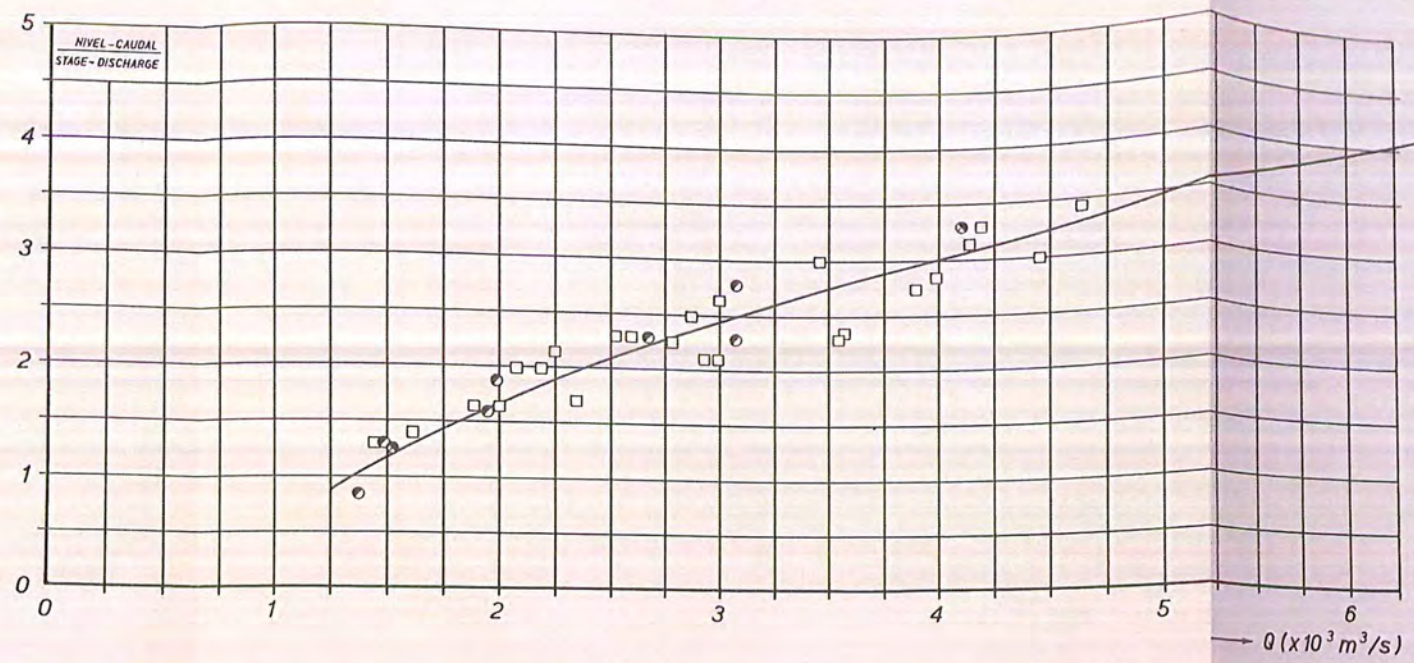
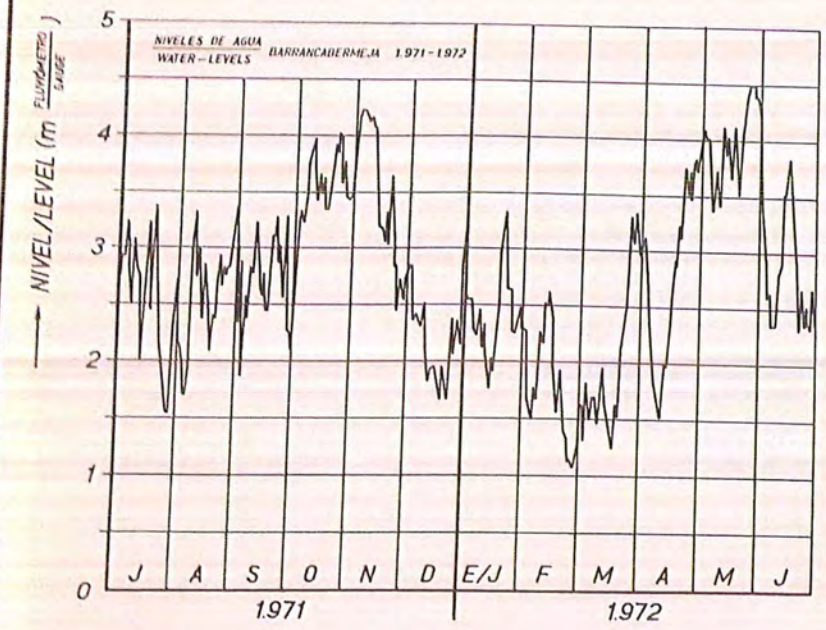
**DATOS GRANULOMETRICOS / BED-MATERIAL DATA**

$\bar{D}_{35} = 700 \mu m$ ;  $\bar{D}_{50} = 920 \mu m$ ;  $\bar{D}_{65} = 1295 \mu m$

**RÍO NUEVO RÍO MAGDALENA km 705.5**

DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT



DATOS AFOROS / DISCHARGE DATA

□	APRON Y DUQUE LTDA 1966
●	MITCH 1971-1972

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA

---	27-VII-1971	-----	16-III-1972
---	21-IX-1971	---	4-XII-1971
---	16-III-1972	---	PROMEDIO / MEAN

DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA

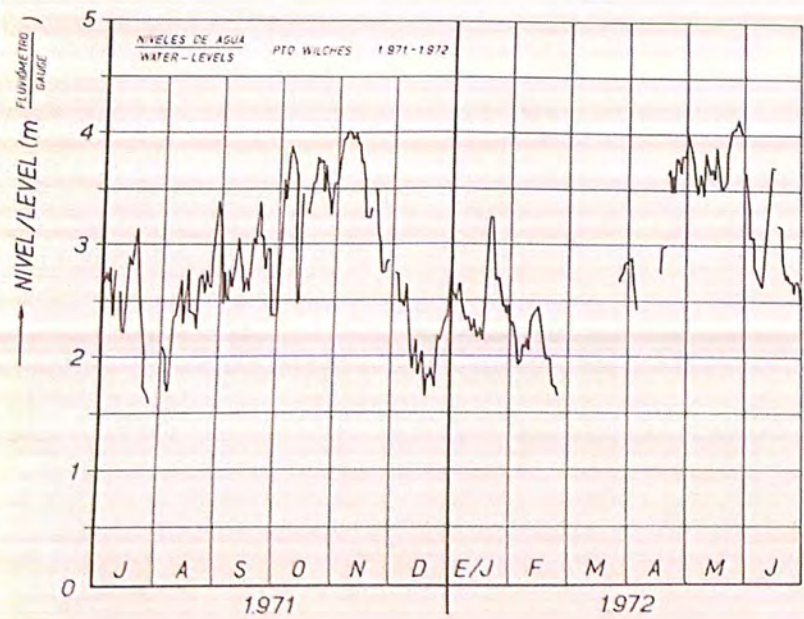
$\bar{D}_{35} = 325 \mu m$ ;  $\bar{D}_{50} = 375 \mu m$ ;  $\bar{D}_{85} = 450 \mu m$

AGUAS ARRIBA / UPSTREAM **RÍO SOGAMOSO** RÍO MAGDALENA km 614

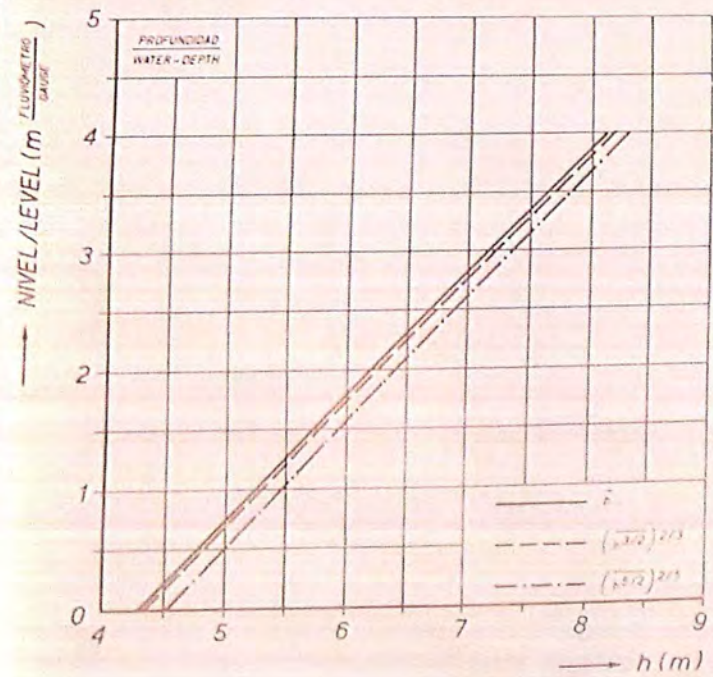
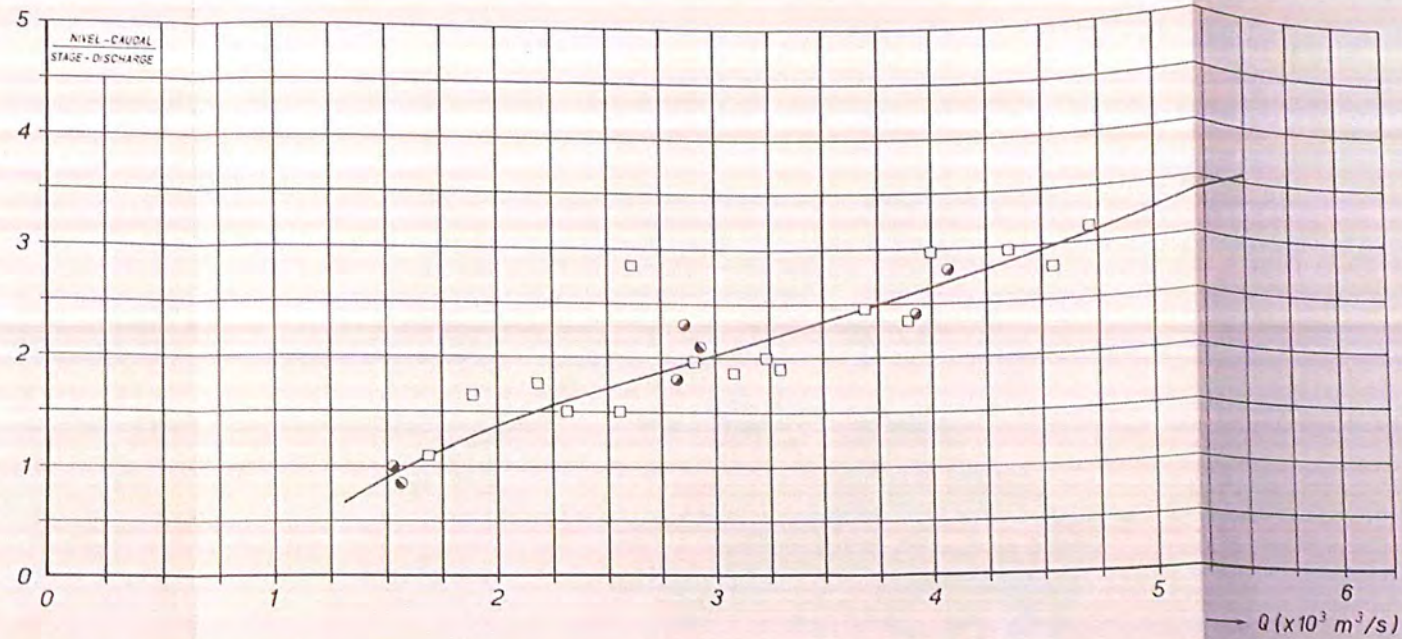
DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

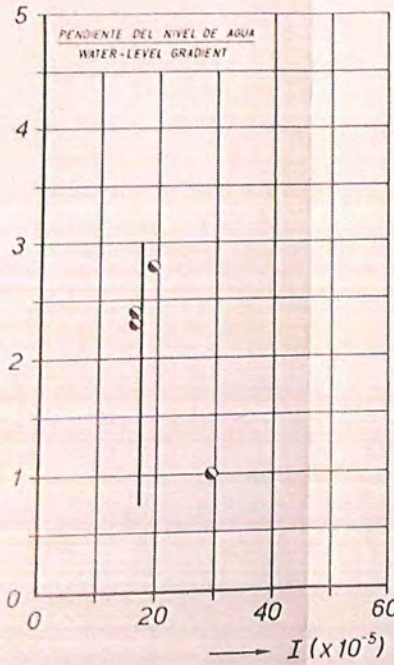
FIG. 3.5.11



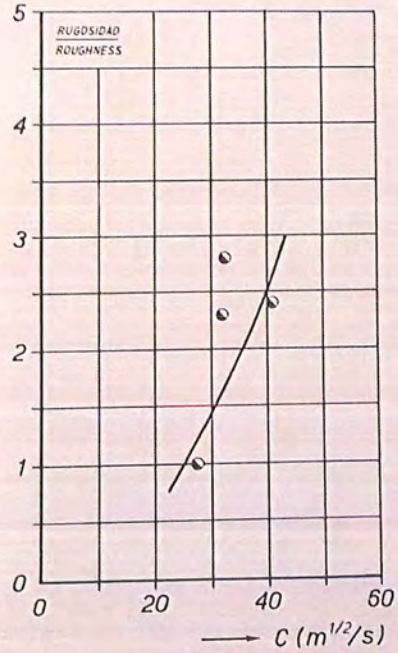
CERO FLUVIOMETRO PTD WILCHES 61,12 m SNM  
ZERO GAUGE



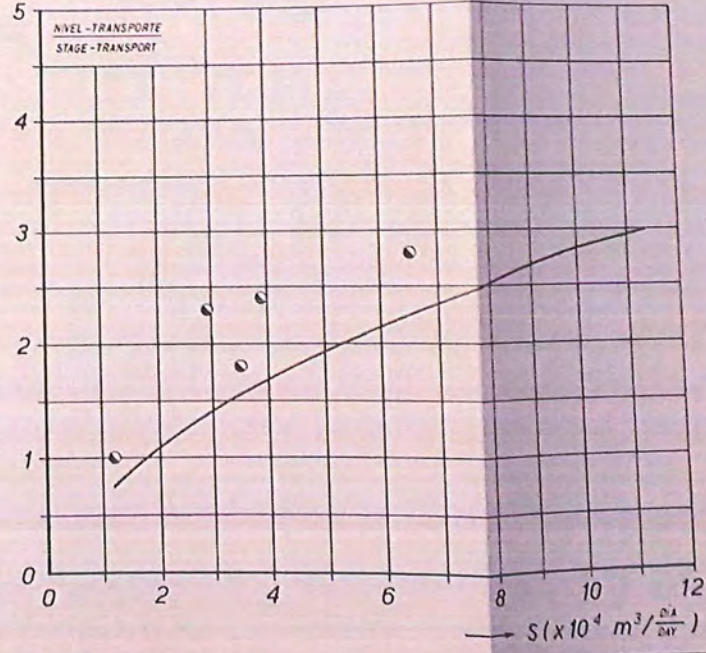
DATOS AFOROS / DISCHARGE DATA  
 □ APRON Y DIQUE LTDA 1966  
 ● MICH 1971-1972



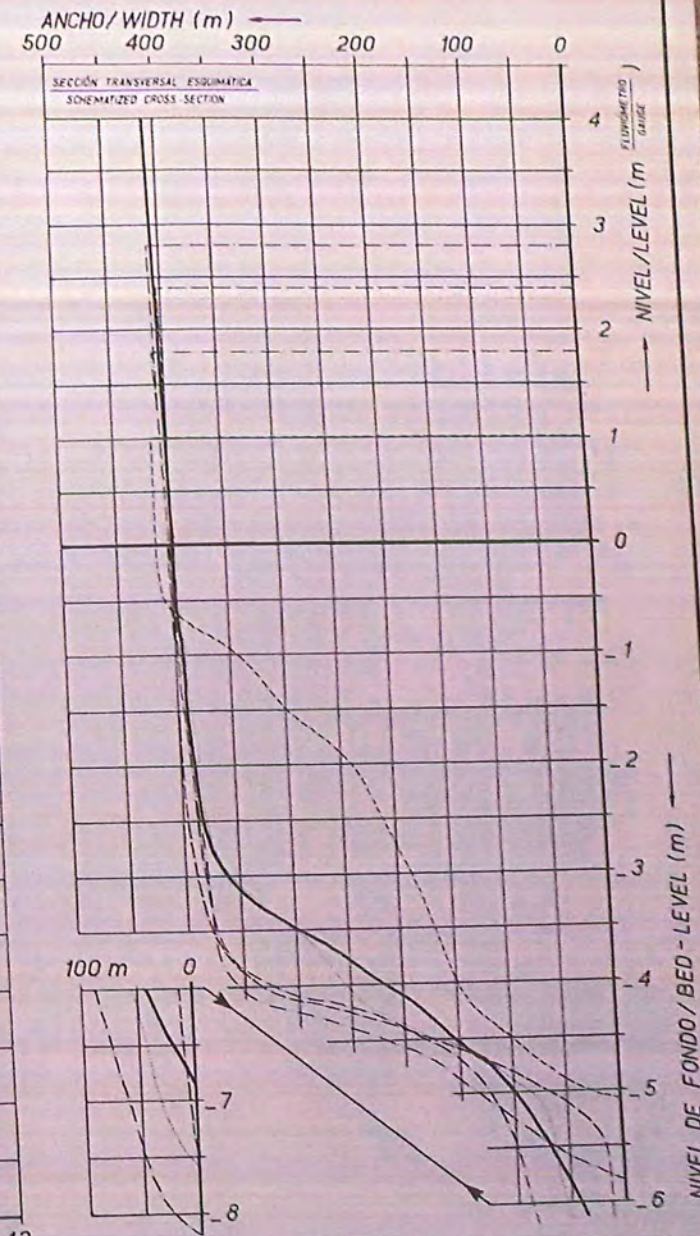
DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA  
 --- 23-VII-1971 --- 17-III-1972  
 - - - 30-IX-1971 - - -  
 - - - 2-XII-1971 - - -  
 - - - 14-I-1972 - - - PROMEDIO / MEAN

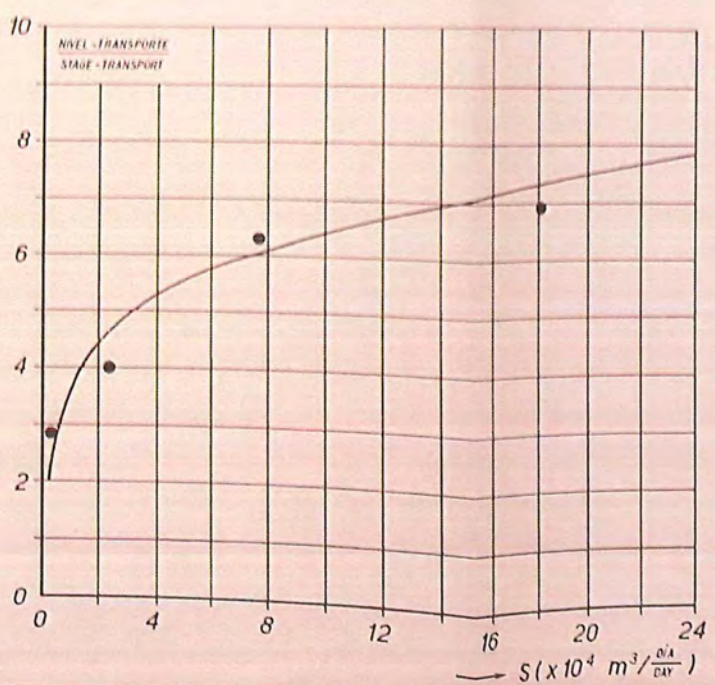
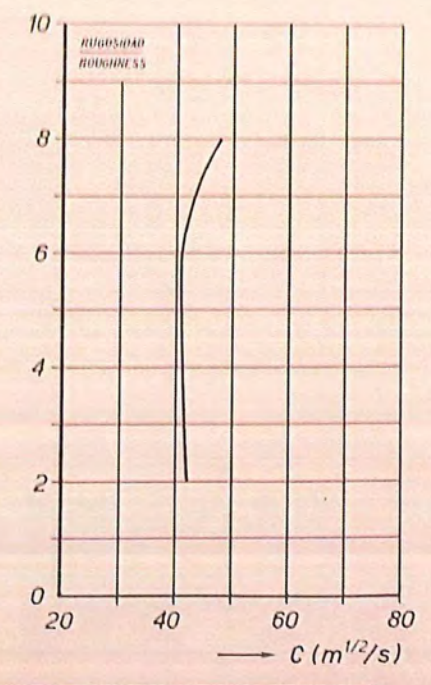
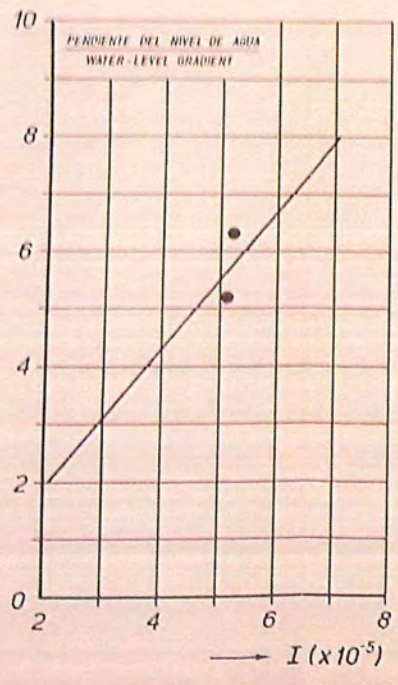
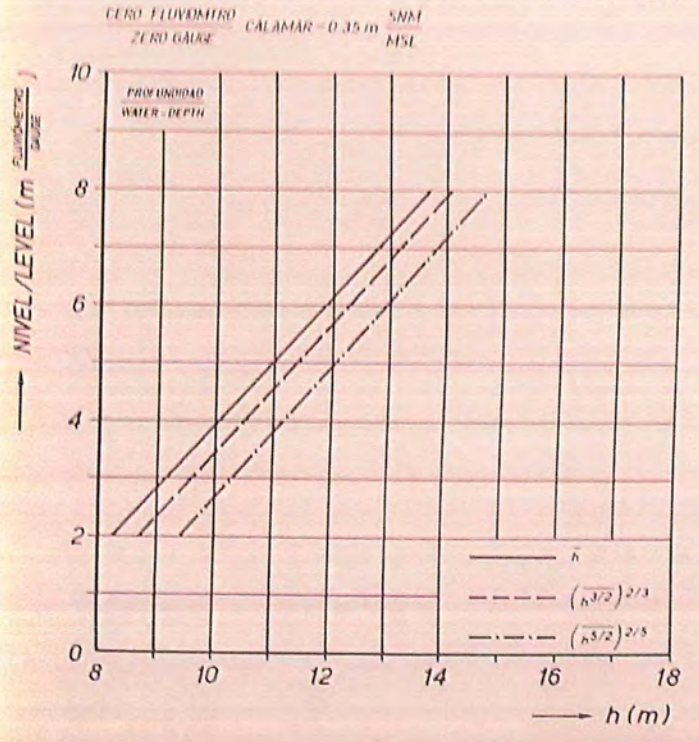
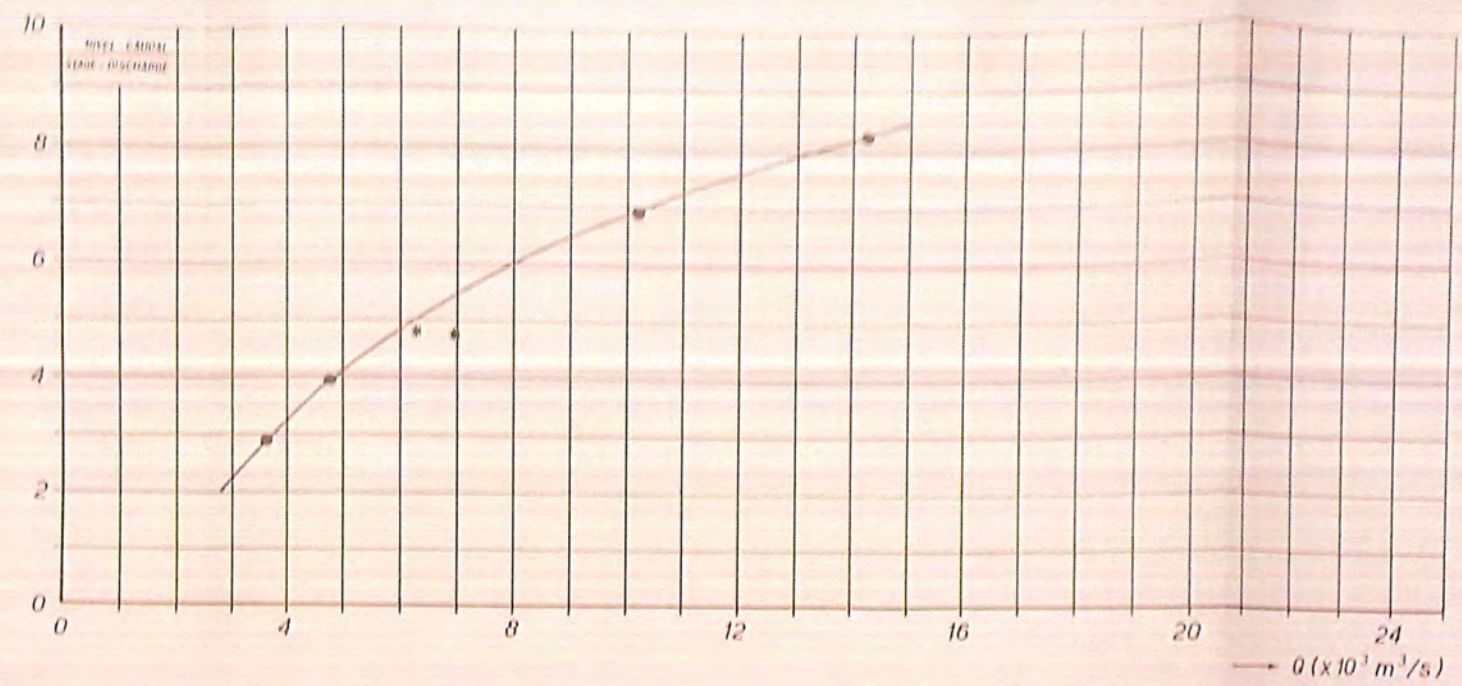
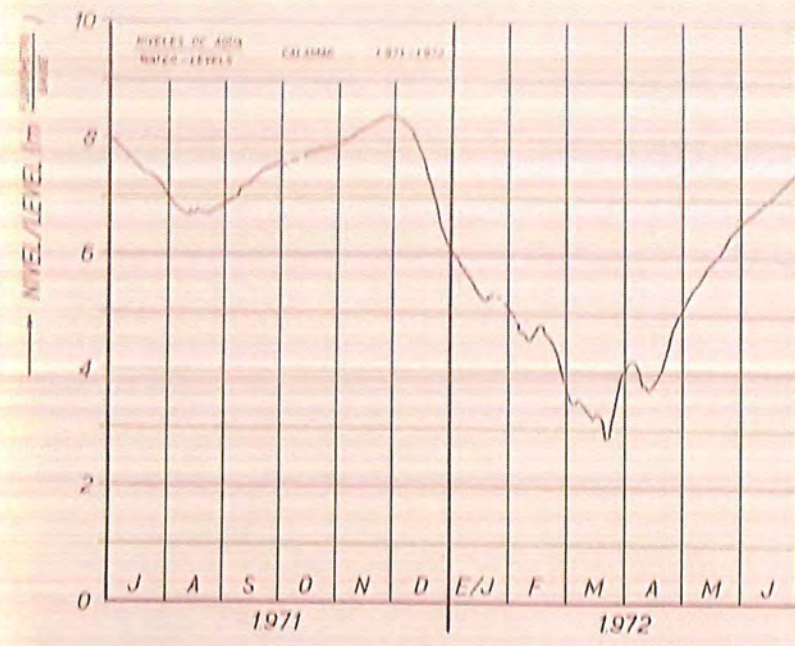


DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA  
 $\bar{D}_{35} = 270 \mu m$ ,  $\bar{D}_{50} = 310 \mu m$ ,  $\bar{D}_{85} = 353 \mu m$



VIJAGUAL RÍO MAGDALENA km 535  
 DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA  
 NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT



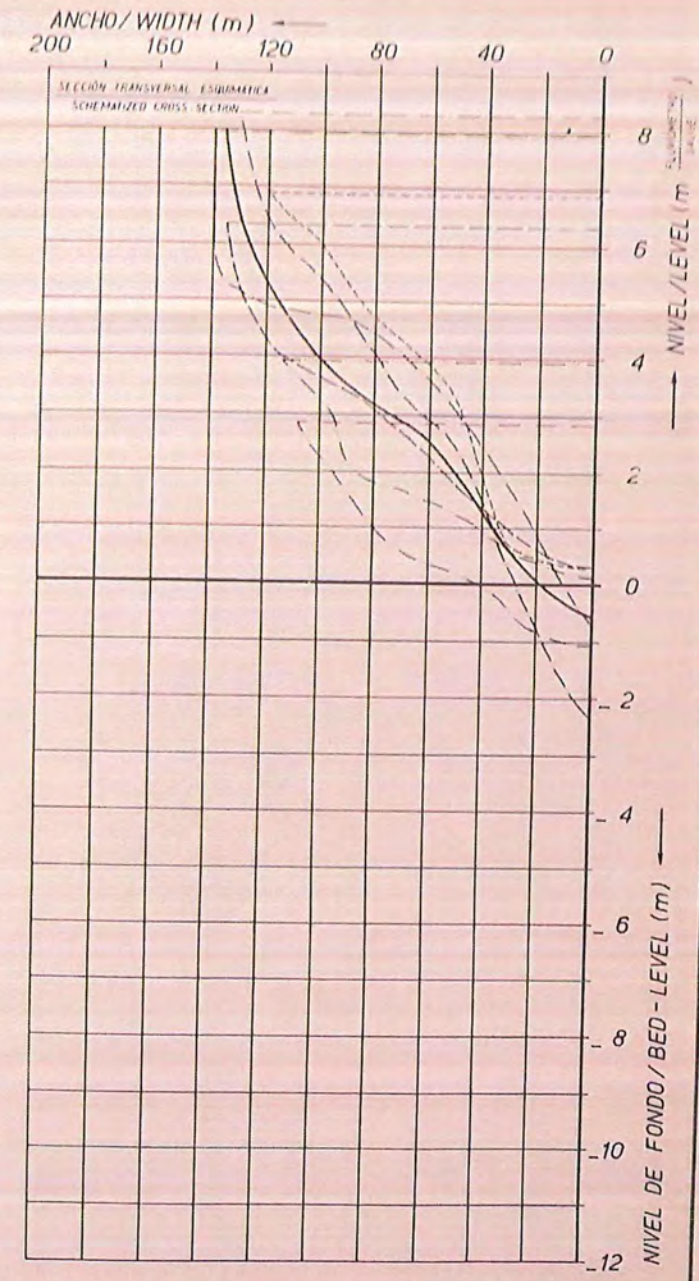
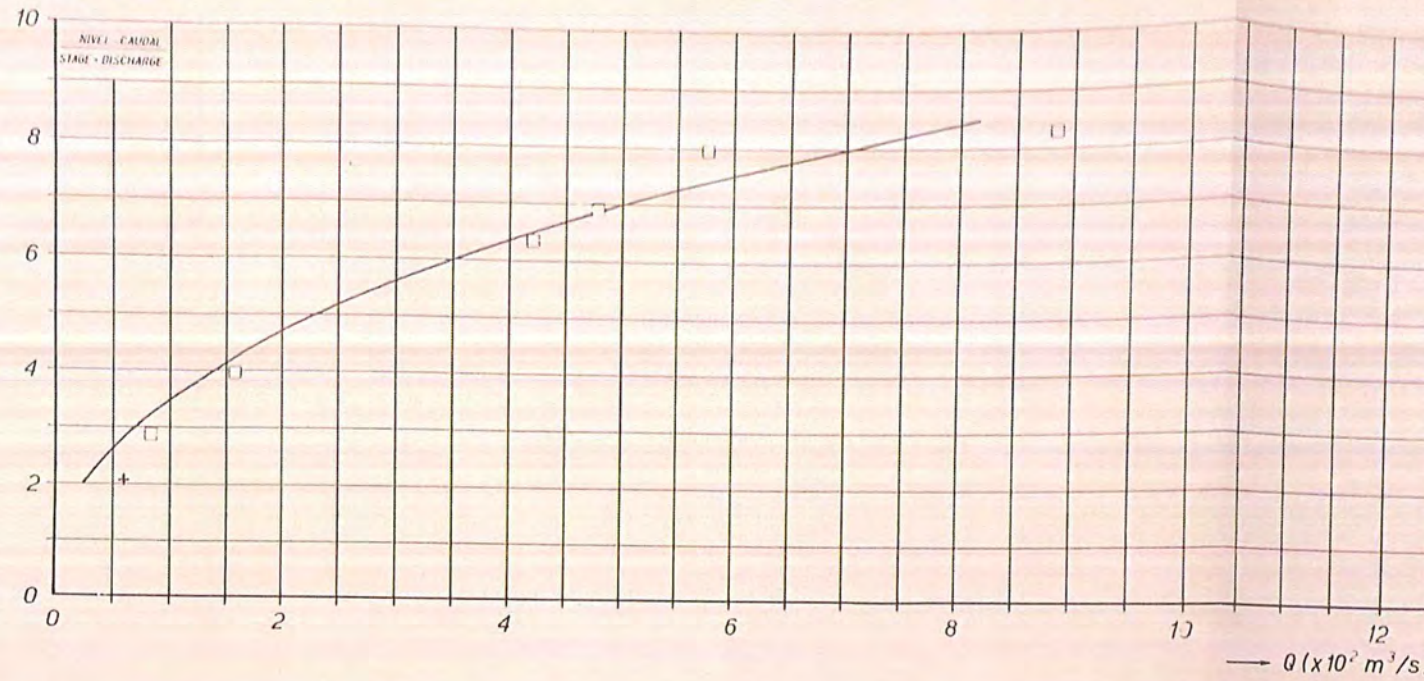
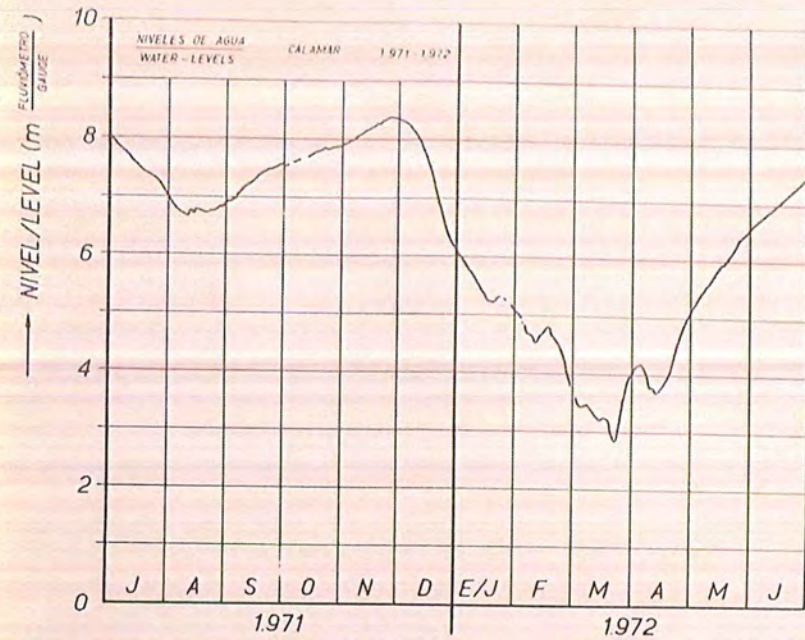


DATOS AFOROS / DISCHARGE DATA  
 \* JULIUS BERGER 1923  
 ● MITCH 1970-1972

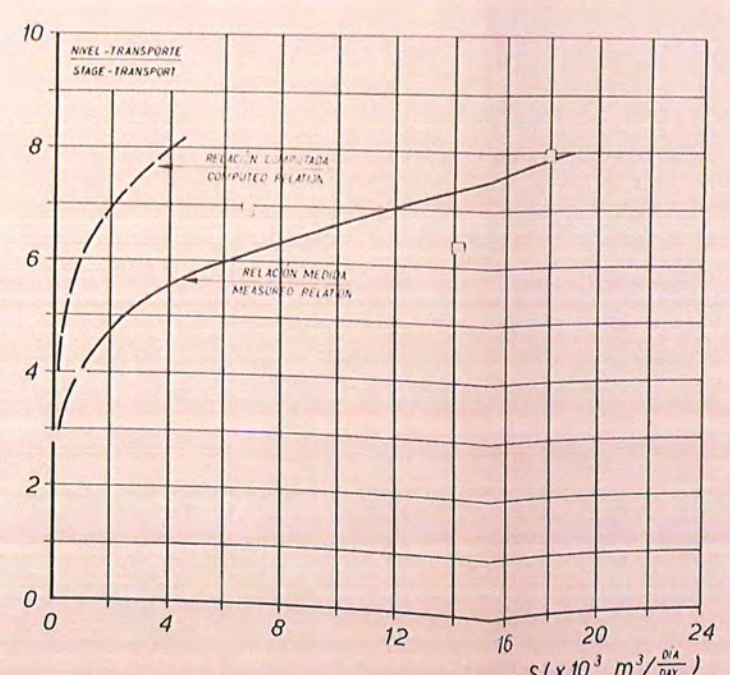
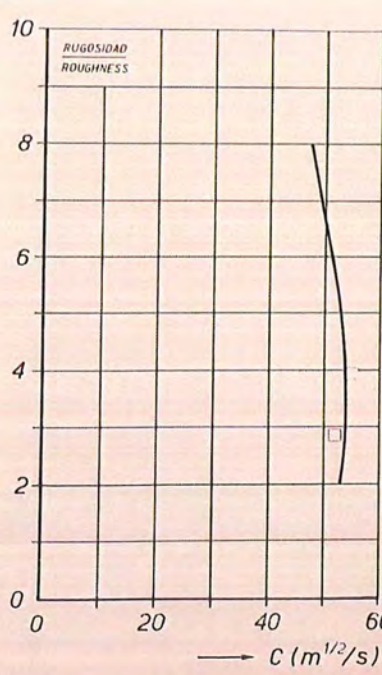
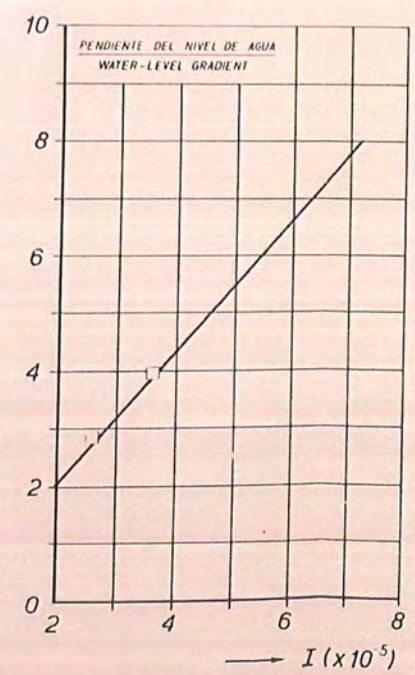
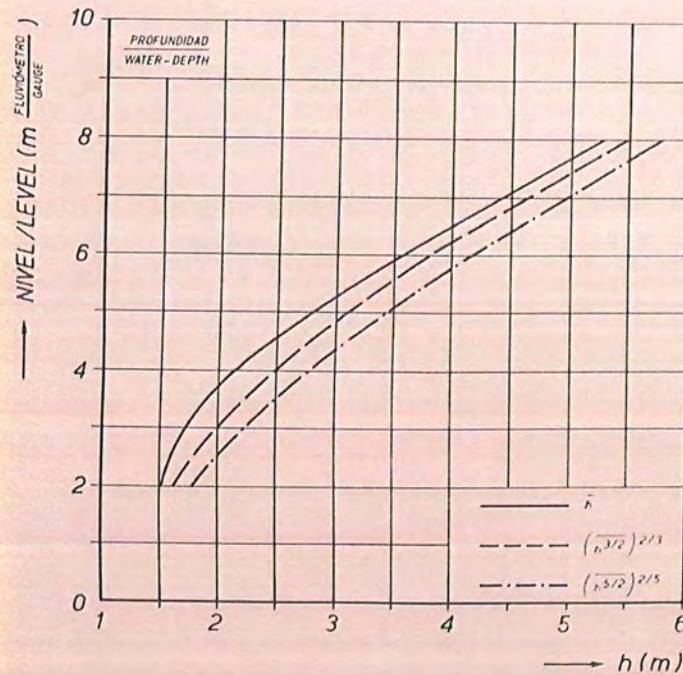
DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA  
 --- 22-XII-1970  
 - - - 1-IX-1971  
 - · - · 21-III-1972  
 · · · · · 18-IV-1972  
 ——— PROMEDIO / MEAN

DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA  
 $\bar{D}_{35} = 195 \mu m$ ,  $\bar{D}_{50} = 210 \mu m$ ,  $\bar{D}_{85} = 230 \mu m$

**CALAMAR RÍO MAGDALENA km 91**  
 DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA  
 NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT **FIG. 3.5.13**



CF-RO FLUVIOMETRO CALAMAR - 0.35 m SNM  
ZERO GAUGE CALAMAR - 0.35 m MSL

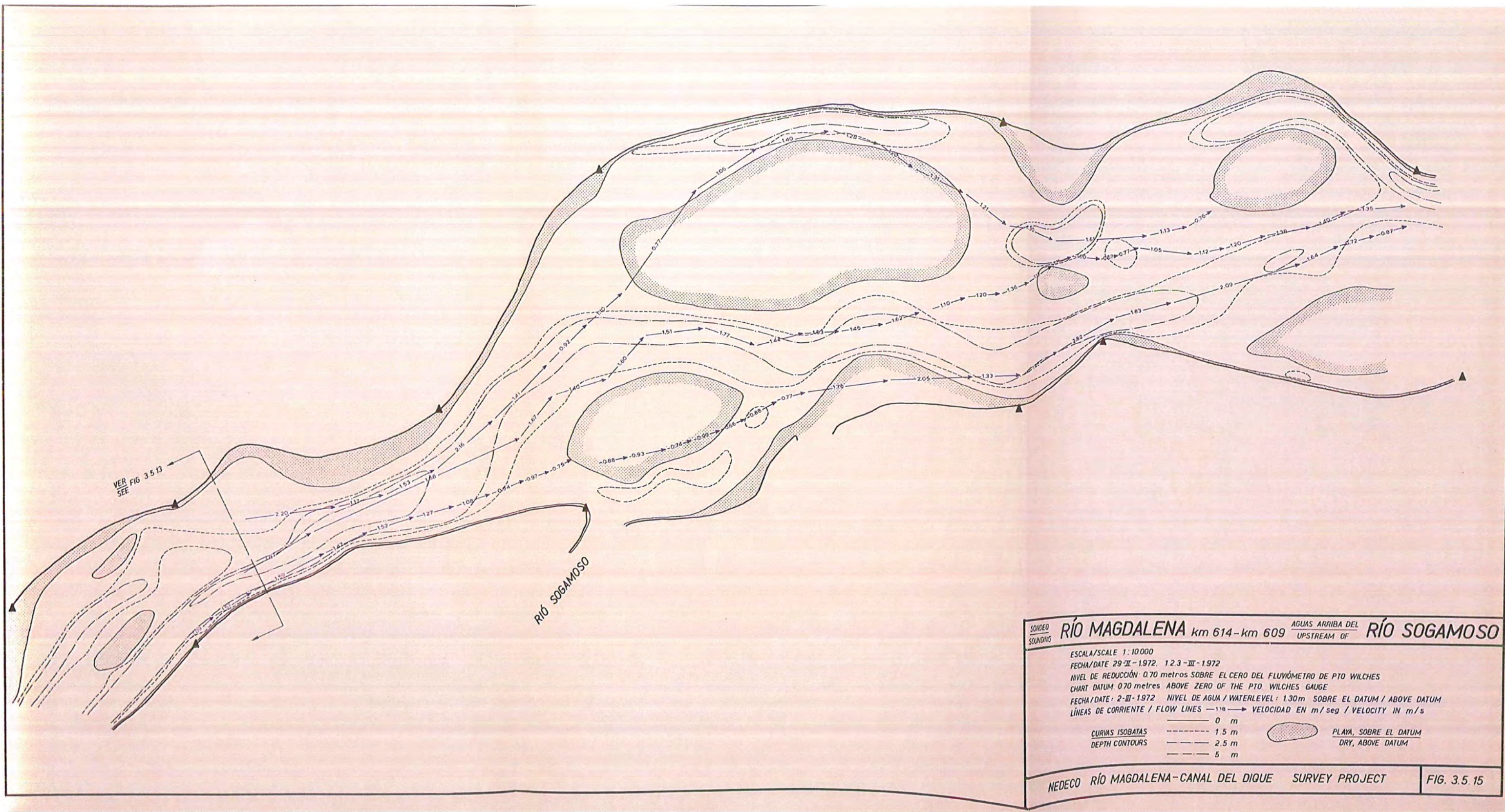


DATOS AFOROS / DISCHARGE DATA	
+	ADENAVI (1957-1958)
□	MITCH (1971-1972)

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA			
---	22-XII-1971	---	22-III-1972
---	14-IV-1971	---	18-IV-1972
---	5-VI-1971	---	
---	1-IX-1971	---	PROMEDIO / MEAN

DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA	
$\bar{D}_{35}$	190 $\mu\text{m}$ , $\bar{D}_{50}$ 210 $\mu\text{m}$ , $\bar{D}_{65}$ 235 $\mu\text{m}$

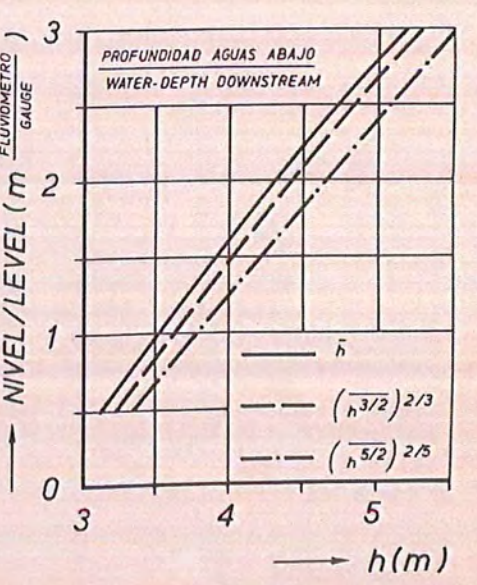
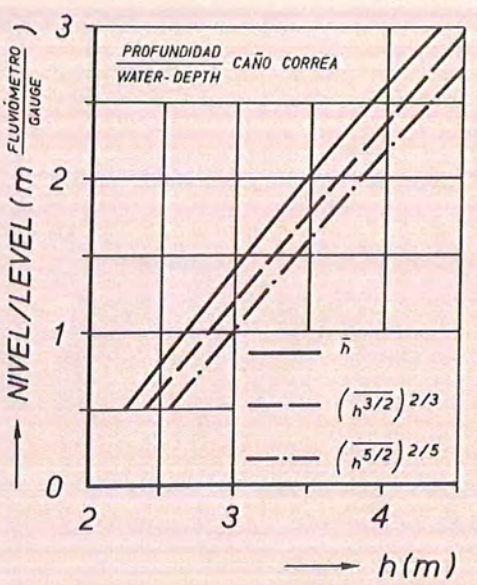
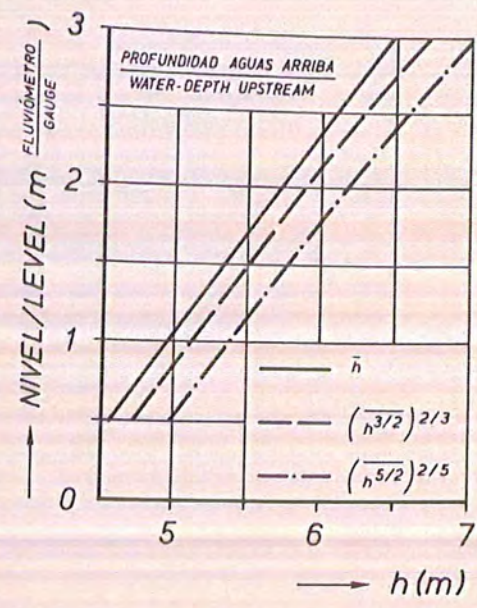
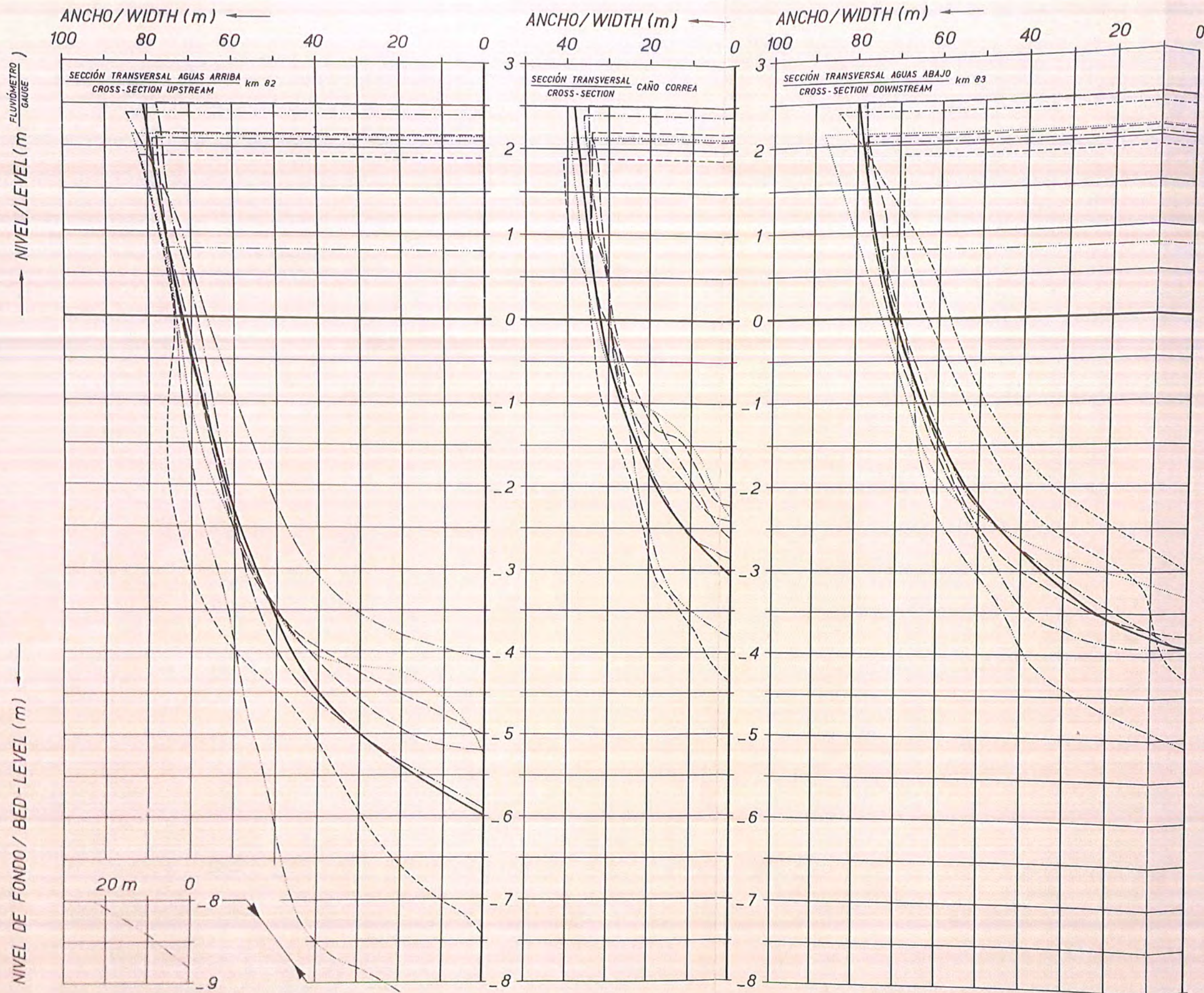
**CALAMAR CANAL DEL DIQUE km 0.2**  
 DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA  
 NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT FIG. 3.5.14



VER FIG 3.5 13  
SEE

RÍO SOGAMOSO

SONDEO / SOUNDINGS		<b>RÍO MAGDALENA</b> km 614 - km 609		AGUAS ARRIBA DEL / UPSTREAM OF <b>RÍO SOGAMOSO</b>	
ESCALA/SCALE 1:10.000					
FECHA/DATE 29-III-1972. 1.2.3-III-1972					
NIVEL DE REDUCCIÓN: 0.70 metros SOBRE EL CERO DEL FLUJÓMETRO DE PTO WILCHES					
CHART DATUM: 0.70 metres ABOVE ZERO OF THE PTO WILCHES GAUGE					
FECHA/DATE: 2-III-1972 NIVEL DE AGUA / WATERLEVEL: 1.30m SOBRE EL DATUM / ABOVE DATUM					
LÍNEAS DE CORRIENTE / FLOW LINES		VELOCIDAD EN m/seg / VELOCITY IN m/s			
CURVAS ISOBATAS / DEPTH CONTOURS		0 m		PLAYA, SOBRE EL DATUM / DRY, ABOVE DATUM	
		1.5 m			
		2.5 m			
		5 m			
NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT			FIG. 3.5 15		



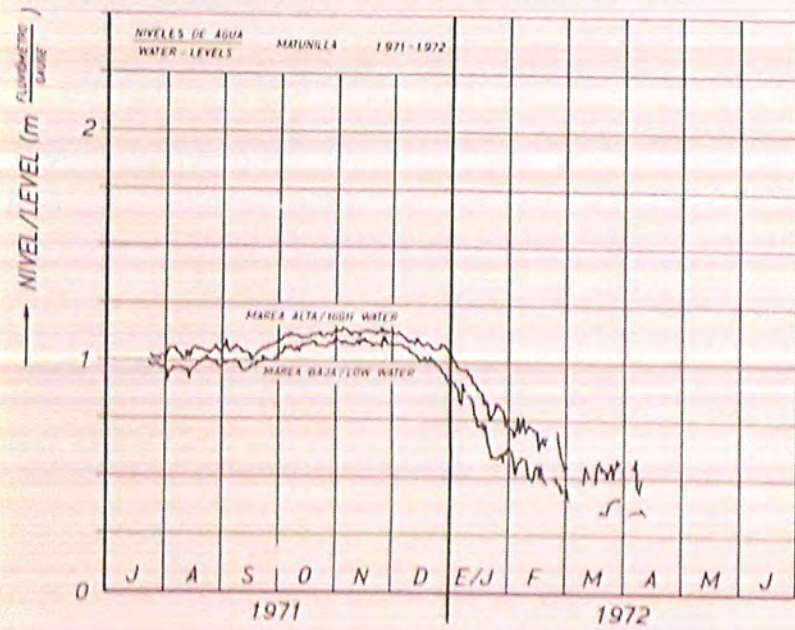
BIFURCACIÓN / BIFURCATION **CAÑO CORREA** CANAL DEL DIQUE km 82.5

DATOS DE LAS SECCIONES TRANSVERSALES ESQUIMÁTICAS / DATA OF SCHEMATIZED CROSS-SECTIONS

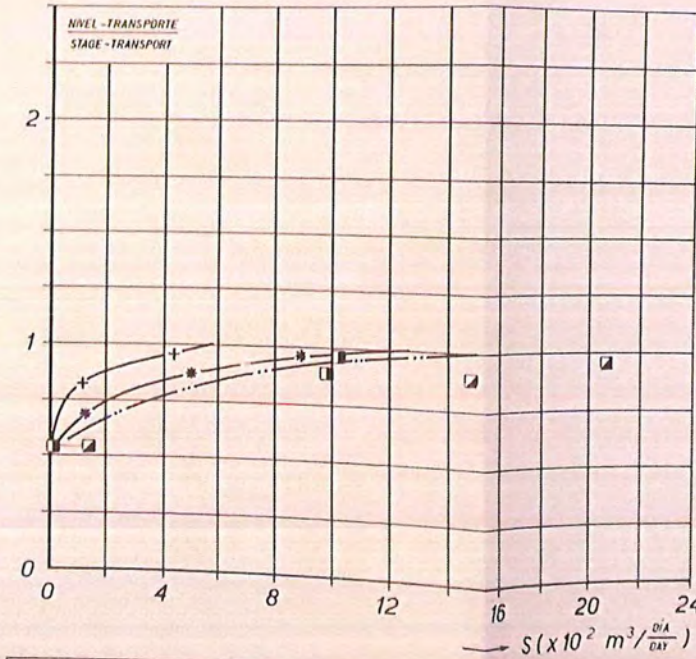
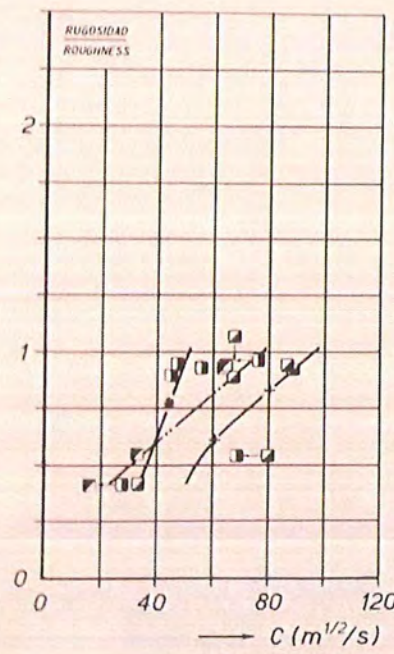
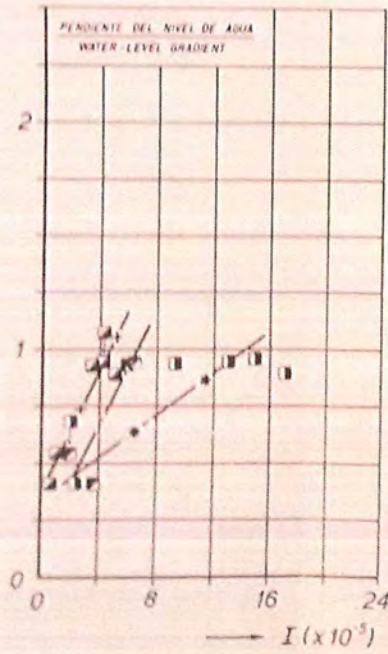
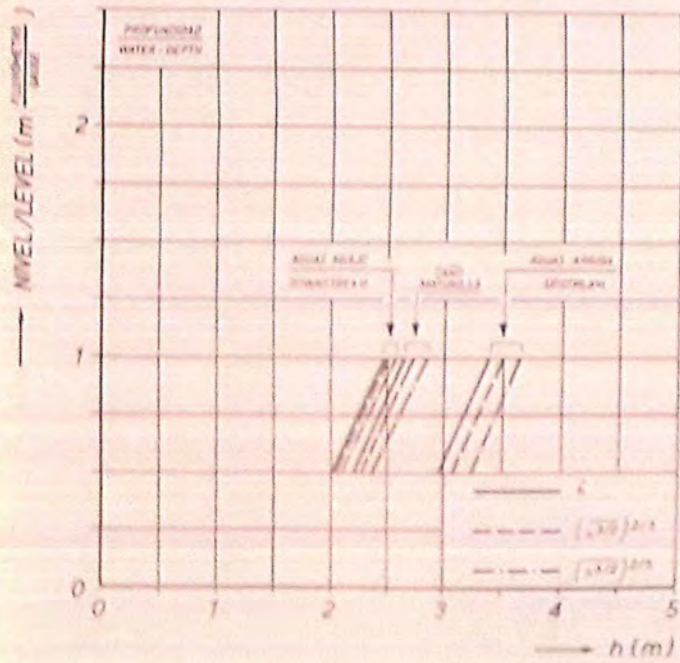
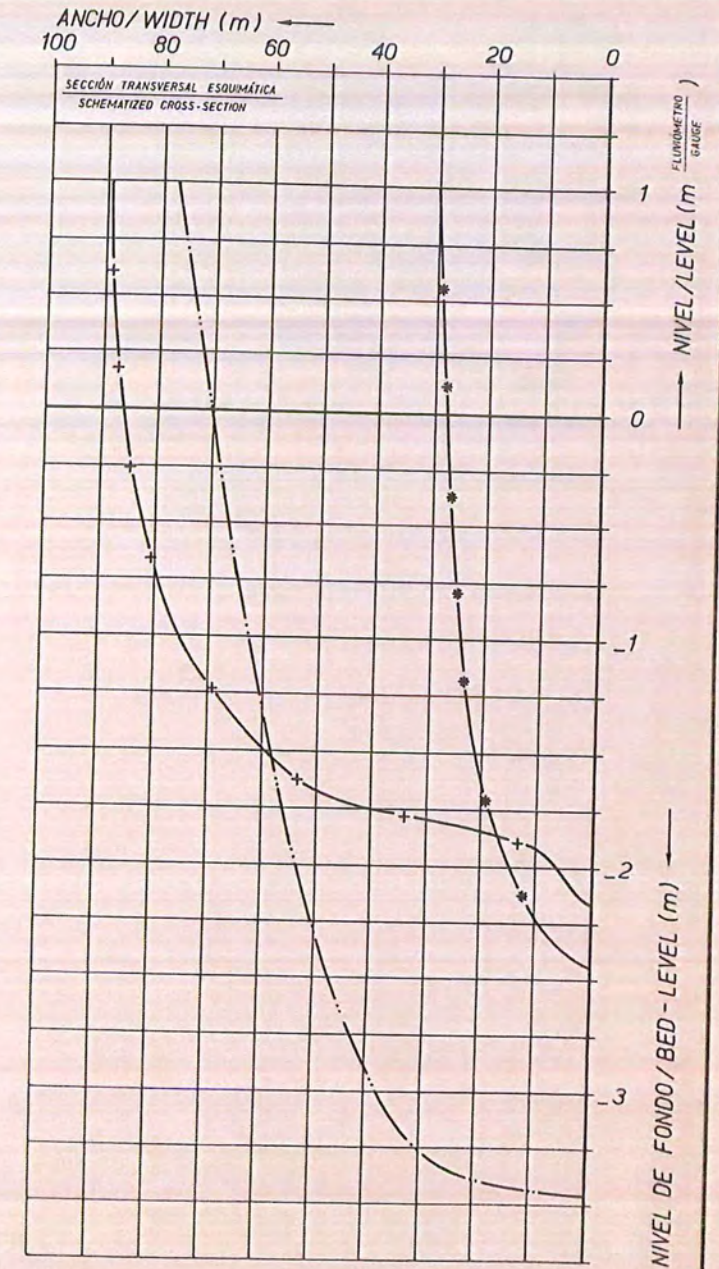
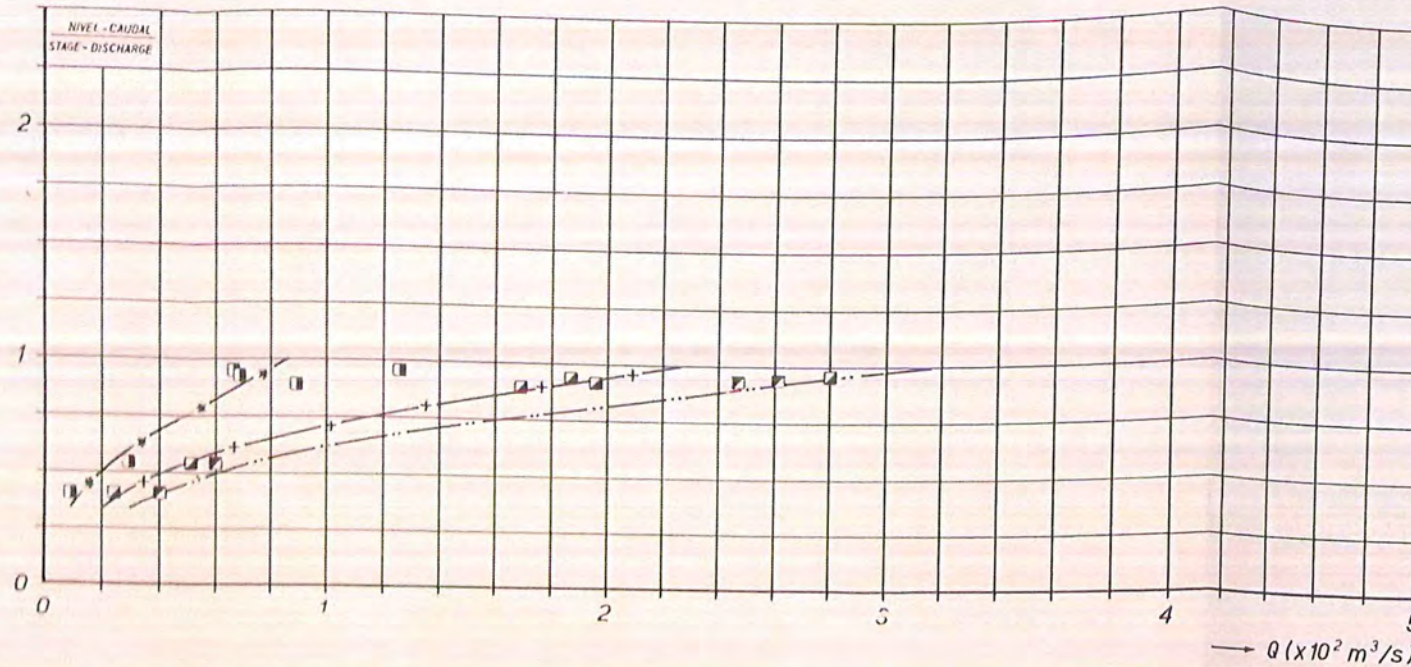
NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT **FIG. 3.5.16**

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA

----- 29-X-1970	----- 10-IX-1971
----- 4-XI-1970	----- 2-XII-1971
----- 2-XII-1970	----- 24-III-1972
----- 2-VI-1971	----- PROMEDIO / MEAN
----- 9-IX-1971	



CERO FLUVIDIMETRO / ZERO GAUGE  
CAÑO MATUNILLA - 0.62 m AMSL  
SNM



**DATOS GEOMÉTRICOS / BED - MATERIAL DATA**

ARREBA / UPSTREAM	$D_{25} = 185 \mu m$	$D_{50} = 190 \mu m$	$D_{85} = 200 \mu m$
CAÑO MATUNILLA	$D_{25} = 145 \mu m$	$D_{50} = 195 \mu m$	$D_{85} = 245 \mu m$
ABAJO / DOWNSTREAM	$D_{25} = 155 \mu m$	$D_{50} = 170 \mu m$	$D_{85} = 135 \mu m$

**DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA**

PROMEDIO / MEAN	AGUAS ARRIBA / UPSTREAM km 99.5
	CAÑO MATUNILLA
	AGUAS ABAJO / DOWNSTREAM km 100.5

**DATOS AFOROS / DISCHARGE DATA**

MITCH 1971-1972

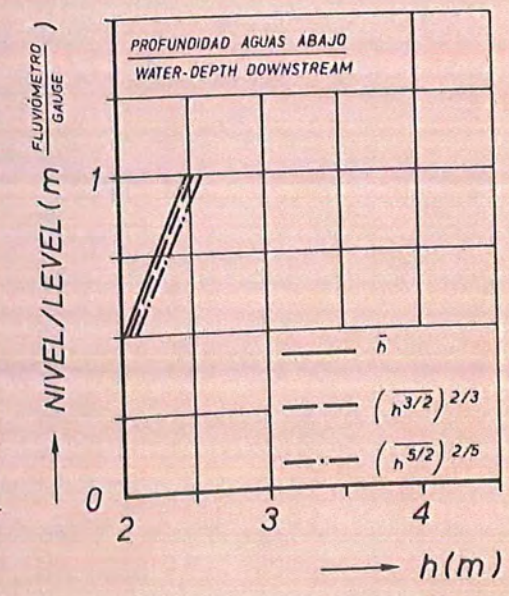
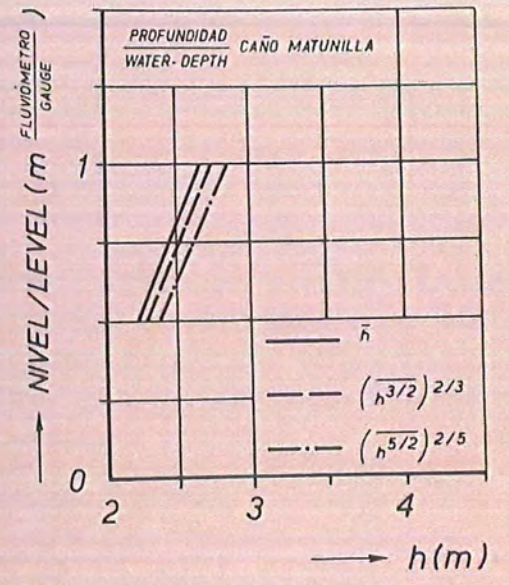
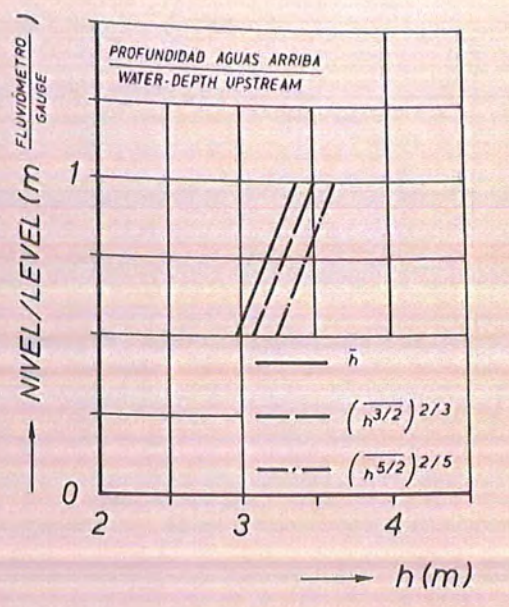
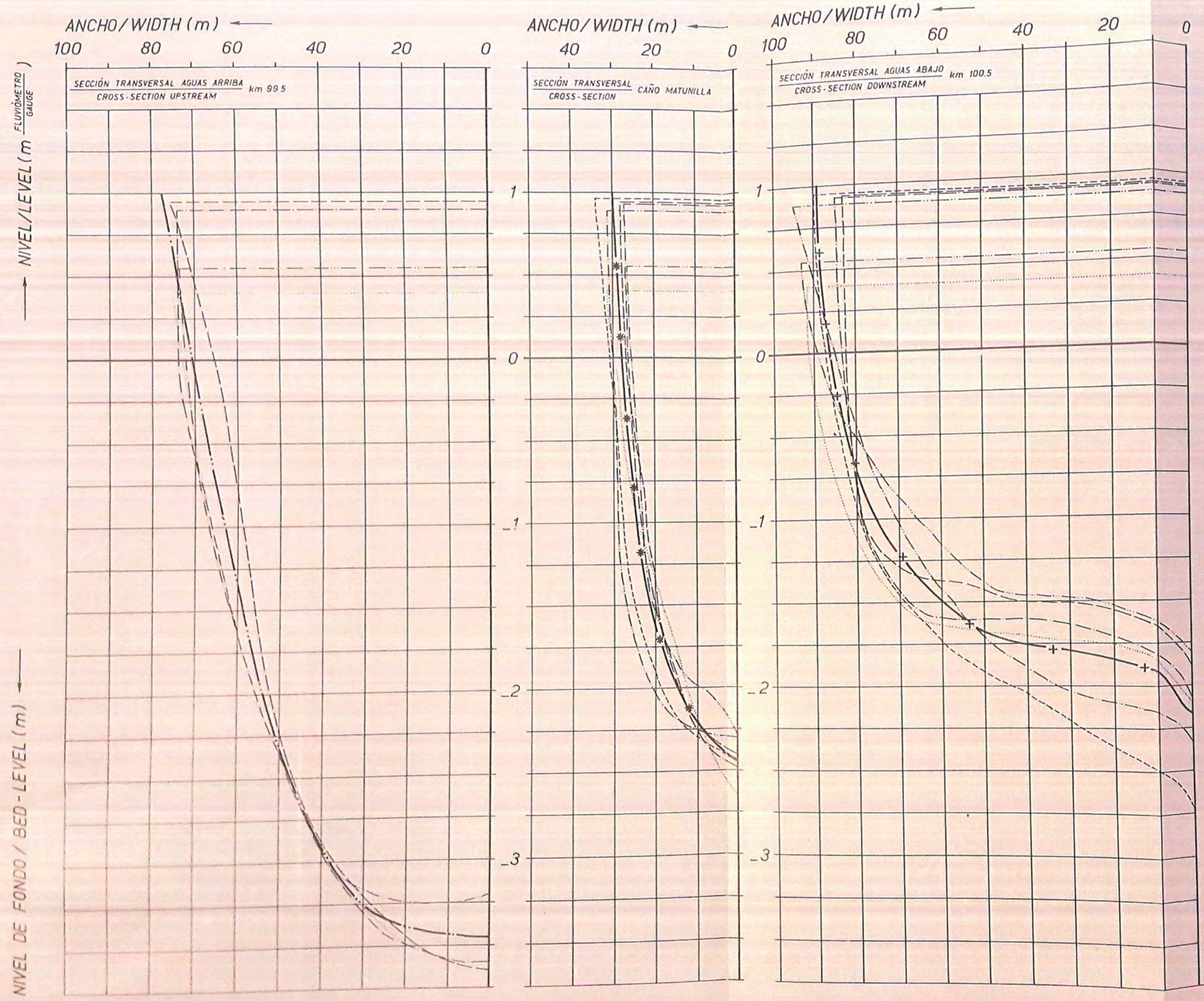
AGUAS ARRIBA / UPSTREAM	CAÑO MATUNILLA
AGUAS ABAJO / DOWNSTREAM	CAÑO MATUNILLA

BRUNCIÓN / BRUNCIÓN  
**CAÑO MATUNILLA CANAL DEL DIQUE km 100**

DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT - TRANSPORT DATA

NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT

FIG 3.5.17



BIFURCACION / BIFURCATION **CAÑO MATUNILLA** CANAL DEL DIQUE km 100

DATOS DE LAS SECCIONES TRANSVERSALES ESQUEMATIZADAS / DATA OF SCHEMATIZED CROSS-SECTIONS

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA			
— · — · —	28 - X - 1970	— · — · —	25 - III - 1972
— · — · —	3 - XI - 1970	— · — · —	26 - IV - 1972
— · — · —	6 - V - 1971	— · — · —	PROMEDIO / MEAN
— · — · —	15 - IX - 1971		

B The Canal del Diquei) The Calamar Section (Canal del Dique, km 0.2)

The relations for this section are presented in Figure 3.5.14. Worthy of note is the difference between the computed sediment transport and the measured data. The computed sediment transport represents the transport-capacity of the Canal del Dique, while the measured data near Calamar indicate the actual supply of sediment to the Canal del Dique by the Río Magdalena. As the measured supply is 5 to 10 times greater than the transport-capacity of the Canal del Dique, sedimentation of the suspended-load particles is occurring. This, in fact, is observed in reality in the first kilometers of the Canal del Dique. More about this follows in Chapter 4 of Part III of this Report.

j) The Caño Correa Bifurcation (km 82.5)

The relations for the three measuring cross-sections of this bifurcation (respectively in the Canal del Dique upstream and downstream of the Caño Correa, and in the Caño Correa itself) are presented in Figure 3.5.15, and are compiled in one graph. The data of the three measuring cross-sections are given separately in Figure 3.5.16. For the computations (presented in Chapter 4 of Part III) regarding the distribution of the discharge, sediment and wash-load over the different branches in the lower region of the Canal del Dique, use will be made of the stage-discharge relations of all these branches plotted against the water-level at the Gambote gauge (Figure III, 4.4.1). All the stage-discharge curves presented in Figure III, 4.4.1 are adjusted, so that the law of continuity holds. The stage-discharge curves presented in Figure 3.5.15 (plotted against the local water-levels at the Correa gauge) are the transferred stage-discharge curves related to the water-levels at Gambote by means of the relation curve of the water-levels at these two stations (see Figure 2.5.6). Here also the check was made that for the three branches of this bifurcation the law of continuity again holds.

The remark made in Para. 3.3.2B regarding the decrease of the water-level gradient in the Caño Correa at high water stages is clearly illustrated in Figure 3.5.15. For more detailed information, reference is made to Chapter 4 of Part III.

k) The Caño Matunilla Bifurcation (km 100)

The relations for the three measuring cross-sections of this bifurcation (respectively in the Canal del Dique upstream and downstream of the Caño Matunilla, and in the Caño Matunilla itself) are presented in Figure 3.5.17. The data of the three measuring cross-sections are given separately in Figure 3.5.18. For the determination of the stage-discharge curves (plotted against the local water-levels at the Matunilla gauge) the same principle has been used as outlined above for the Caño Correa bifurcation (however, here the relation curve of the water-levels at Gambote and Matunilla was used; see Figure 2.5.6). For the stage-discharge curves presented in Figure 3.5.17 again the law of continuity holds (for further reference see Chapter 4 of Part III).

## II, 3.5

### 1) The Caño Lequerica Bifurcation (km 108)

For the three branches of this bifurcation (the Canal del Dique upstream and downstream of the Caño Lequerica, and in the Caño Lequerica itself) only the data of the schematized cross-sections are presented in Figure 3.5.19. Although the data of the discharge measurements of this bifurcation could be used to establish the discharge distribution over the three branches versus the water-levels at Gambote (see Figure III, 4.4.1), the data are insufficient to determine such relations versus the local water-level at the Lequerica gauge. For the relations of the water-levels at Lequerica versus the discharge, the channel roughness, the water-level gradient and the sediment transport, the tide which penetrates inland from the Bahía de Cartagena and the Bahía de Barbacoas respectively can not be left out of consideration. For such relations, the measurements should be continued over a full tidal cycle to eliminate the tidal influence. As the majority of the data were gathered in any phase of the tide, such relations could not be drawn. For the data of the measurements which were carried out over a full tidal cycle reference is made to Figure 3.3.42. The complete lower region of the Canal del Dique is further discussed in Chapter 4 of Part III.

#### 3.5.4. Procedures when no measurements are available

In Para. 3.6 it is shown that one of the boundary conditions for morphological computations to design river-works will be the stage-discharge relation. The stage-discharge relations for the main gauge-stations (Pto. Salgar, Pto. Inmarco, Pto. Berrío, Barrancabermeja and Pto. Wilches) have been given in Para. 3.5.3. However, if in the computations a river stretch is to be considered in between these main gauge-stations, the question arises whether the stage-discharge relation established at a main station, can be transferred to the area under consideration, or if it will be necessary first to carry out a number of measurements. For example, can the stage-discharge relation at Pto. Berrío be used for the computations near the confluence of the Río Magdalena and the Río Regla (Part III, Chapter 3.4)? This question can also be put as: Is the discharge at one station at a certain level (related to L.R.L.) equal to the discharge at the same level above L.R.L. at another station if the discharge of the affluents between the two stations is considered, but the influence of storage neglected? To answer this question, two stretches of the Río Magdalena have been considered:

- between Pto. Berrío and Barrancabermeja; and
- between La Dorada/Pto. Salgar and Pto. Inmarco (and Pto. Berrío).

#### The Río Magdalena between Pto. Berrío and Barrancabermeja

The stage-discharge relations at the upstream end (Pto. Berrío, km 730), and downstream end (Barrancabermeja, km 631) of this stretch have been presented in Figures 3.5.10 and 3.5.11. The affluents of the Río Magdalena in this stretch are the Río Regla (km 711), the Río Viejo (km 707), the Río Carare (km 673) and the Río Opón (km 636). The discharge of the Río Regla is already partly included in the stage-discharge relation of Pto. Berrío (Figure 3.5.10), because the discharges of the Río Nuevo (downstream of the Río Regla Confluence) measured by Apron y Duque Ltda. and MITCH have also been used to establish this relation. Therefore the discharge of the Río Regla has not again been taken into account. As the Río Viejo is in fact a minor branch of the Río Magdalena which joins the main river again at km 697, it has also been left out of consideration. The question remains whether the sum of the stage-discharge relations of the Río Magdalena at Pto. Berrío, the Río Carare and the Río Opón is equal to the stage-discharge relation of the Río Magdalena at Barrancabermeja.

The stage-discharge relations of the Río Carare and Río Opón are presented in Figure 3.5.20. The discharge data of the Río Carare are plotted against the water-levels at the Pto. Berrío gauge. The discharges show a considerable scatter, due to the fact that the water-levels at Pto. Berrío have been used (instead of the local water-levels) and the discharge of the Río Carare is influenced by backwater effect of the Río Magdalena. The discharge data of the Río Opón are first plotted against the water-levels at Barrancabermeja, and then this stage-discharge relation is transferred to Pto. Berrío, considering that at the same water-level (related to the L.R.L. in Barrancabermeja and Pto. Berrío respectively), the discharge is equal too. Similarly, the stage-discharge relation of the Río Magdalena at Barrancabermeja has been transferred to such a relation but then plotted against the water-

II, 3.5

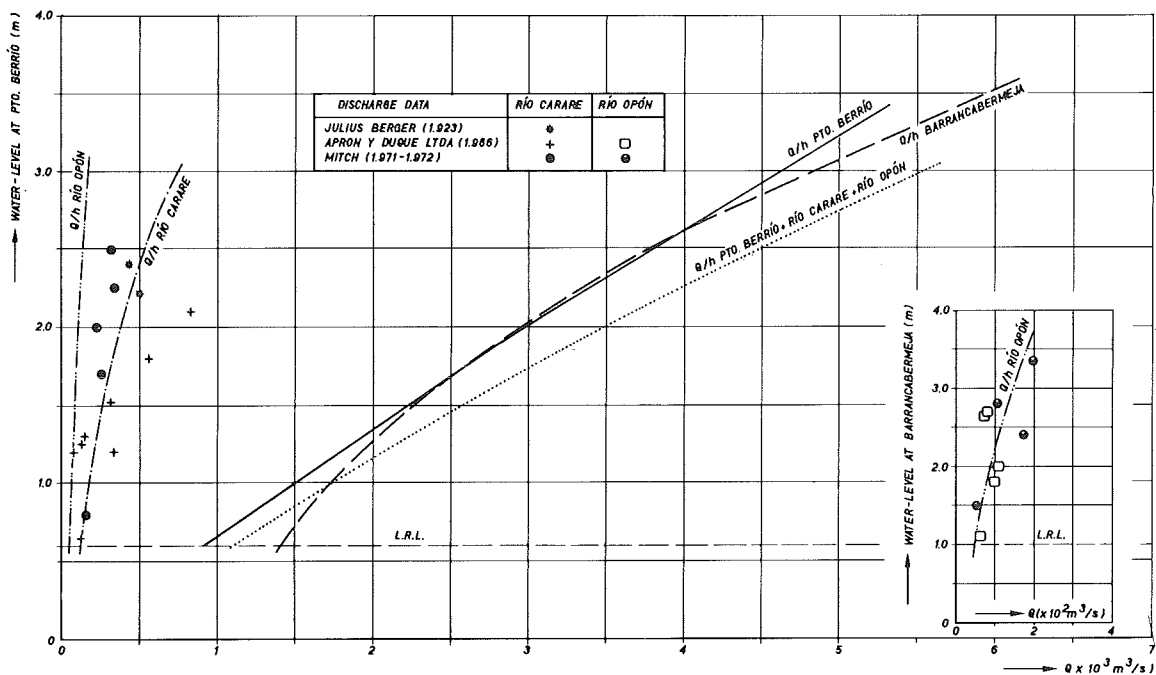


Figure 3.5.20 Stage-Discharge Relations between Pto. Berrío and Barrancabermeja

levels at Pto. Berrío. Thereafter, the sum of the stage-discharge relations of the Río Magdalena at Pto. Berrío, the Río Carare and the Río Opón can be compared with the stage-discharge relation of the Río Magdalena at Barrancabermeja. Both the curves agree reasonably well, especially if the scatter of the original stage-discharge relations of the Río Magdalena (Figures 3.5.10 and 3.5.11) are taken into account. Moreover, the stage-discharge relations of the two tributaries should, in fact, have been plotted against the local water-levels. As these water-levels, however, cannot be related to the L.R.L. in the Río Magdalena, both have been plotted directly against the water-levels of the main gauge-stations. It can be concluded that:

- For computations in the river-stretch between Pto. Berrío and the confluence of the Río Magdalena and the Río Carare, the stage-discharge relation of Pto. Berrío (Figure 3.5.10) can be used; and
- for computations in the river-stretch downstream of the confluence of the Río Magdalena and the Río Carare, the stage-discharge relation of Barrancabermeja (Figure 3.5.11) has to be taken.

The Río Magdalena between La Dorada/Pto. Salgar and Pto. Inmarco (and Pto. Berrío)

The stage-discharge relations at the upstream end (La Dorada, km 887) and downstream end (Pto. Inmarco, km 773) of this stretch have been presented in Figures 3.5.5 and 3.5.7. The major tributaries of the Río Magdalena in this stretch are the Río Negro (km 841), the Río La Miel (km 837) and the Río Nare (km 774). The stage-discharge relations of the Río Negro and the Río La Miel have been plotted directly against the water-levels

at the Pto. Salgar gauge (Figure 3.5.21). Both curves are questionable, not only because no local water-levels were used, but also in view of the small range of water-levels covered by the measured data.

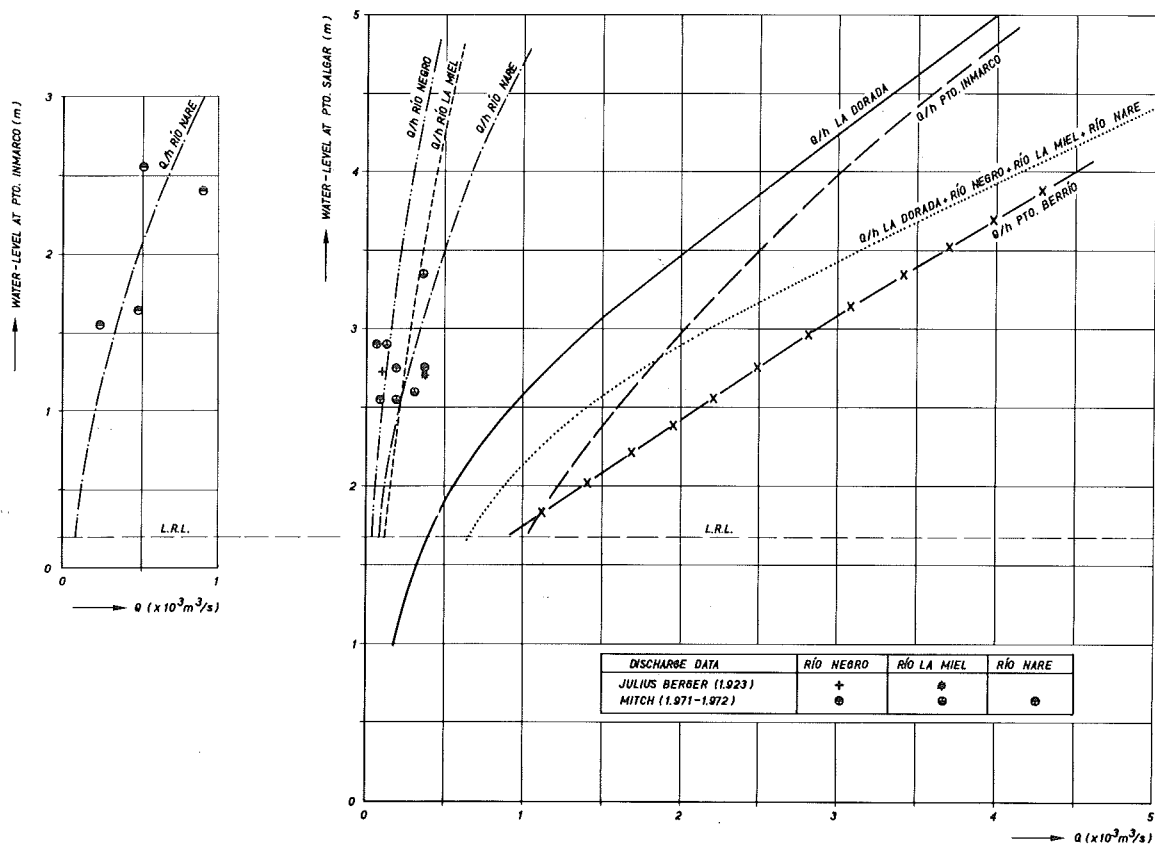


Figure 3.5.21 Stage-Discharge Relations between Pto. Salgar and Pto. Inmarco (and Pto. Berrfo)

The stage-discharge relation of the Río Nare was already presented (Figure 3.5.6), but as this curve cannot be related to the L.R.L. in the Río Magdalena, another relation is presented of the discharge of the Río Nare plotted against the water-levels at the Pto. Inmarco gauge (Figure 3.5.21). All the stage-discharge relations have been transferred to the Pto. Salgar gauge and the sum of the stage-discharge relations of the Río Magdalena at La Dorada and the Rios Negro, La Miel and Nare can be compared with the stage-discharge relation of the Río Magdalena at Pto. Inmarco.

These two curves do not agree very well. This can be explained if the stage-discharge relation of the Río Magdalena at Pto. Berrfo is also considered. In view of the fact that between Pto. Inmarco and Pto. Berrfo no other tributaries enter the Río Magdalena, the sum of the stage-discharge relation of the Río Magdalena at La Dorada and the Rios Negro, La Miel and Nare can also be compared with the stage-discharge relation of the Río Magdalena at Pto. Berrfo (Figure 3.5.21). Contrary to the comparison for Pto. Inmarco, these two curves appear to agree reasonably well.

II, 3.5

Moreover, Figure 3.5.21 shows that for small discharges ( $Q < \text{about } 1,500 \text{ m}^3/\text{s}$ ) the same discharge corresponds to a lower water-level at Pto. Inmarco than at Pto. Salgar and Pto. Berrío, while for high discharges ( $Q > \text{about } 1,500 \text{ m}^3/\text{s}$ ) a higher water-level at Pto. Inmarco is found. In other words, the water-levels covered by the stage-discharge curves show a more or less equal range at both Pto. Salgar and Pto. Berrío (from about 1.5 m to 4 m) but a definitely greater range at Pto. Inmarco (from about 1 m to 5.5 m). It is recalled (Para. 2.3.5) that the L.R.L. at Pto. Inmarco could not be established by means of an average duration curve because the available record of water-level data at Pto. Inmarco was insufficient. As the relation curve between the water-levels at Pto. Inmarco and Pto. Berrío showed a considerable scatter (Figure 2.5.4), the L.R.L. at Pto. Inmarco could only be established by means of the "line of equal discharge". Consequently, due to the greater range of water-levels at Pto. Inmarco, it follows that the L.R.L. at Pto. Inmarco must fall below the (straight) line connecting the L.R.L. at Pto. Salgar and Pto. Berrío. However, this is only a local phenomenon, determined by the great reduction of the width of the Río Magdalena just downstream of Pto. Inmarco and the location of the gauge at the confluence of the Río Magdalena and the Río Nare. Therefore, the L.R.L. at Pto. Inmarco should only be used locally, while, e.g., longitudinal soundings along the Río Magdalena should be reduced to the (straight) line connecting the values of L.R.L. at the Pto. Salgar and Pto. Berrío gauges (except very near to Pto. Inmarco, see Figure 3.5.22).

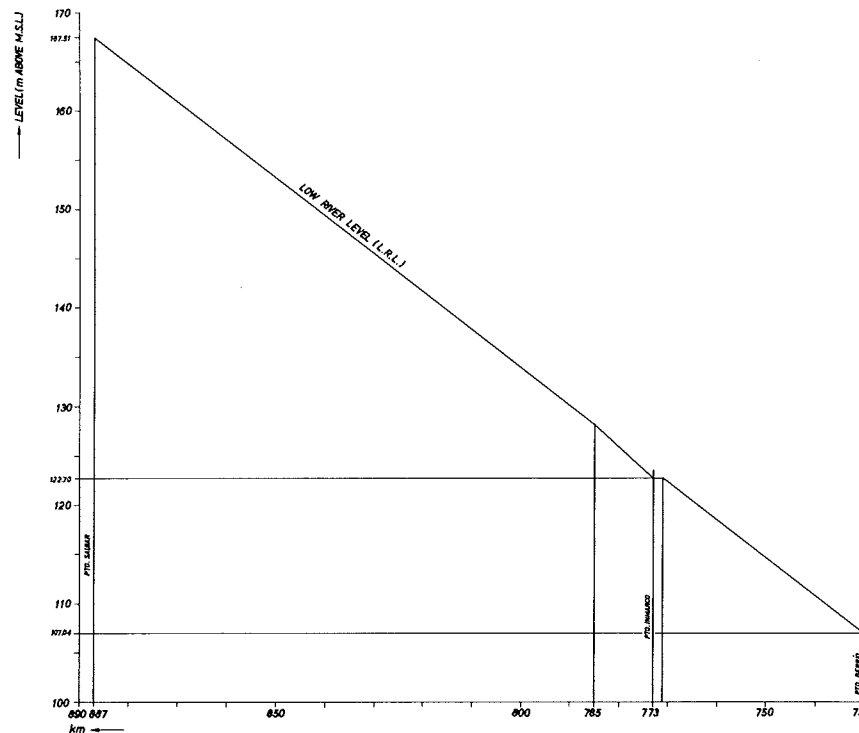


Figure 3.5.22 L.R.L. near Pto. Inmarco

In the light of these considerations, the following conclusions can be drawn:

- For computations in the river stretch between La Dorada and the confluence of the Rfo Magdalena and the Rfo Negro, the stage-discharge relation of La Dorada (Figure 3.5.5) can be used.
- For computations in the river stretch downstream of the confluence of the Rfo Magdalena and the Rfo La Miel, the sum of the stage-discharge relations of the Rfo Magdalena at La Dorada and the Rfos Negro and La Miel (Figure 3.5.21) may be used. As the determination of the stage-discharge relations of the two tributaries, however, is rather arbitrary, it is advised to establish a new gauge-station and measuring cross-section in this river stretch, e.g., near Pto. Triunfo.
- For computations near the confluence of the Rfo Magdalena and the Rfo Nare (upstream of the narrow river stretch just downstream of Pto. Inmarco) the stage-discharge relation of Pto. Inmarco (Figure 3.5.7) must be used.
- For computations downstream of the narrow river stretch at Pto. Inmarco, the stage-discharge relation of Pto. Berrfo (Figure 3.5.9) has to be taken.

A final remark must be made regarding the channel roughness which has to be used in the morphological computations. Near the main gauge-stations the relationship between the water-level and the roughness as presented in the figures of Para. 3.5.3 can be used. Where no data on the roughness are available, it is concluded in Para. 3.4.3 that the bed roughness relation as presented by Einstein and Barbarossa (Figure 3.4.3) can best be used.

### 3.6. ONE-DIMENSIONAL MORPHOLOGICAL COMPUTATIONS

#### 3.6.1. General

The purpose of morphological computations is to predict the consequences of changes made in a river by means of river-works, or to determine the extent to which river-works have to be carried out to produce a required result.

Morphological computations are rather complicated because the value of the parameters to be predicted are time-dependent in two ways:

- The equilibrium situation is not reached immediately after completion of river-works. The time required to attain equilibrium may be important; if the time to reach a required situation is excessive, it may be necessary to look for another solution or to find means to accelerate the process.
- The discharge of a river changes with time, and therefore also other parameters (velocity, depth, bed-level, etc.) will change with time.

In view of the time-dependent character of a river, different approaches for computations are possible:

#### Computation a

This computation is carried out for steady conditions, and makes it possible to judge the change which will have taken place after a final equilibrium situation has been reached. Such a computation will generally be carried out (manually) to judge the feasibility of a

## II, 3.6

project. It will be necessary to schematize the (discharge) regime of the river into one single discharge, the dominant discharge. In para. 3.6.5 more is said about the selection of the dominant discharge.

### Computation b

This computation may serve to estimate the time required before a new equilibrium situation is reached, not only for the section where river-works are carried out, but especially how long it will take before the influence of the river-works will have reached a place at a certain distance from those works. This computation can also be carried out with a dominant discharge.

### Computation c

This computation can be carried out for regime conditions. Besides giving information about the regime influence, a more detailed impression is obtained about the time required before a new equilibrium is attained (especially locally).

For the design of river-works generally all three types of computation are required. In Para. 3.6.2 the mathematical base is given; then the different computations are applied to a schematized example; and finally the selection of a dominant discharge is dealt with.

### 3.6.2. Mathematical base

For the changes in the river-bed due to natural changes in plan-form or the construction of river-works, the equations of motion and continuity for water and sediment will have to be used. These equations may be written thus (per unit width):

For water:

$$\frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + g \frac{\partial h}{\partial x} + g \frac{\partial z}{\partial x} = -g \frac{v|v|}{C^2 h} \quad (3.6.1)$$

$$\frac{\partial h}{\partial t} + v \frac{\partial h}{\partial x} + h \frac{\partial h}{\partial x} = 0 \quad (3.6.2)$$

For sediment: The Engelund/Hansen equation as found in Para. 3.5.2 which may be written as:

$$\frac{s}{\sqrt{\Delta g \bar{D}_{50}^3}} = \frac{0.05}{1-\epsilon} \frac{C^2}{g} \left( \frac{v^2}{C^2 \bar{D}_{50}} \right)^{5/2} \quad (3.2.2)$$

$$\frac{\partial z}{\partial t} + \frac{\partial s}{\partial x} = 0 \quad (3.6.3)$$

In fact, also the width (B) should be taken as parameter, but as the computations are one-dimensional, no actual difference is introduced as long as it is realized that:  $v = \bar{v} = Q/B\bar{h}$ ;  $s = \bar{s} = S/B$ ;  $h = \bar{h} = F/B$  and  $q = \bar{q} = Q/B$ . Of course, also in the computation B may change along the channel axis ( $dB/dx \neq 0$ ).

For the three types of computations (a, b and c) different terms may be neglected and the use of the equations may differ. This is dealt with separately for each type of computation.

Computation a

As this computation considers an equilibrium situation, the computation is carried out for steady conditions and all time derivatives finish. Eq. (3.6.1) reduces to:

$$\bar{v} \frac{d\bar{v}}{dx} + g \frac{d\bar{h}}{dx} + g \frac{dz}{dx} = -g \frac{|\bar{v}|}{c^2 \bar{h}}$$

which may also be written as:

$$\frac{d}{dx} \left( \frac{\bar{v}^2}{2g} \right) + \frac{d\bar{h}}{dx} + I_b = - \frac{\bar{v}^2}{c^2 \bar{h}} \quad (3.6.4)$$

Taking into account the remark about the width Eq.(3.6.2) becomes:

$$\bar{v} \frac{d\bar{h}}{dx} + \bar{h} \frac{d\bar{v}}{dx} = 0$$

which may be written as:

$$\frac{dQ}{dx} = 0 \quad (3.6.5)$$

Eq.(3.2.2) does not contain time derivatives and therefore does not change.

Eq.(3.6.3) becomes:

$$\frac{ds}{dx} = 0 \quad (3.6.6)$$

Eqs.(3.2.2) and (3.6.4) contain the roughness parameter  $C$  which has to be found from measurements or, if no measurements are available, from the method given in Para. 3.4.

To solve the equations, the following boundary conditions are required (see Figure 3.6.1):

- The water-level at some place  $x = L$ ;
- the discharge  $Q_0 (=Q_x)$  (dominant discharge); and
- the amount of sediment transport  $S_0 (=S_x)$ , which possibly has to be found from computations, in which case the water-level gradient has to be measured or estimated.

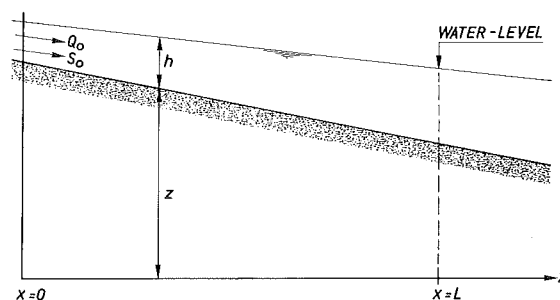
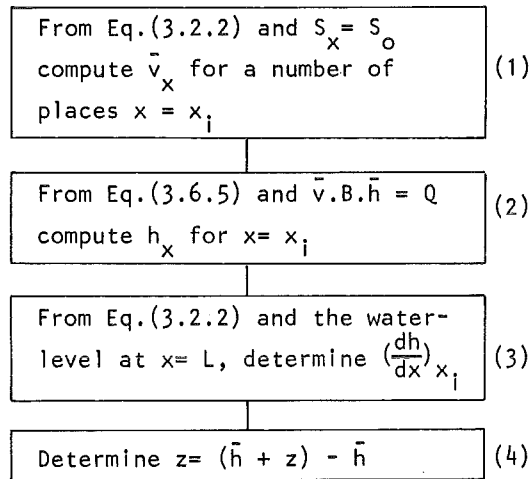


Figure 3.6.1 Boundary Conditions

With these data a computation can be carried out. As an example, a flow diagram for such a computation is given below, although often the required computation can be simpler than that given in the flow diagram.



In Para. 3.6.4 an example of the application of this type of computation is given.

Computation b (constant discharge)

It can be derived that for low Froude numbers the propagation of disturbances in the river-bed is small in relation to the propagation of disturbances in the water-level (see de Vries (1969) [39] ).

This means that when scour and sedimentation of the bed are considered, the propagation of water-level disturbances may be considered as infinitely large and  $\frac{\partial v}{\partial t}$  and  $\frac{\partial h}{\partial t}$  in Eqs. (3.6.1) and (3.6.2) may be neglected in respect of the other terms. Application of this method for higher Froude numbers should be done with caution. The influence of the higher Froude numbers may be estimated from Figure 3.6.2.

The equations to be used may now be written as:

$$\frac{d}{dx} \left( \frac{\bar{v}^2}{2g} \right) + \frac{d\bar{h}}{dx} + I_b = - \frac{\bar{v}^2}{c^2 \bar{h}} \tag{3.6.4}$$

$$\frac{dQ}{dx} = 0 \tag{3.6.5}$$

$$\frac{s}{\sqrt{\Delta g D_{50}^3}} = \frac{0.05}{1-\epsilon} \frac{c^2}{g} \left( \frac{\bar{v}^2}{c^2 \Delta D_{50}} \right)^{5/2} \tag{3.2.2}$$

$$\frac{\partial z}{\partial t} + \frac{\partial s}{\partial x} = 0 \tag{3.6.3}$$

These equations can only be solved numerically. This, however, would make Computation b exactly the same as Computation c, albeit for a constant discharge. It would then be better to skip Computation b altogether and carry the computation out for regime conditions (as Computation c). This, in fact, is very often done.

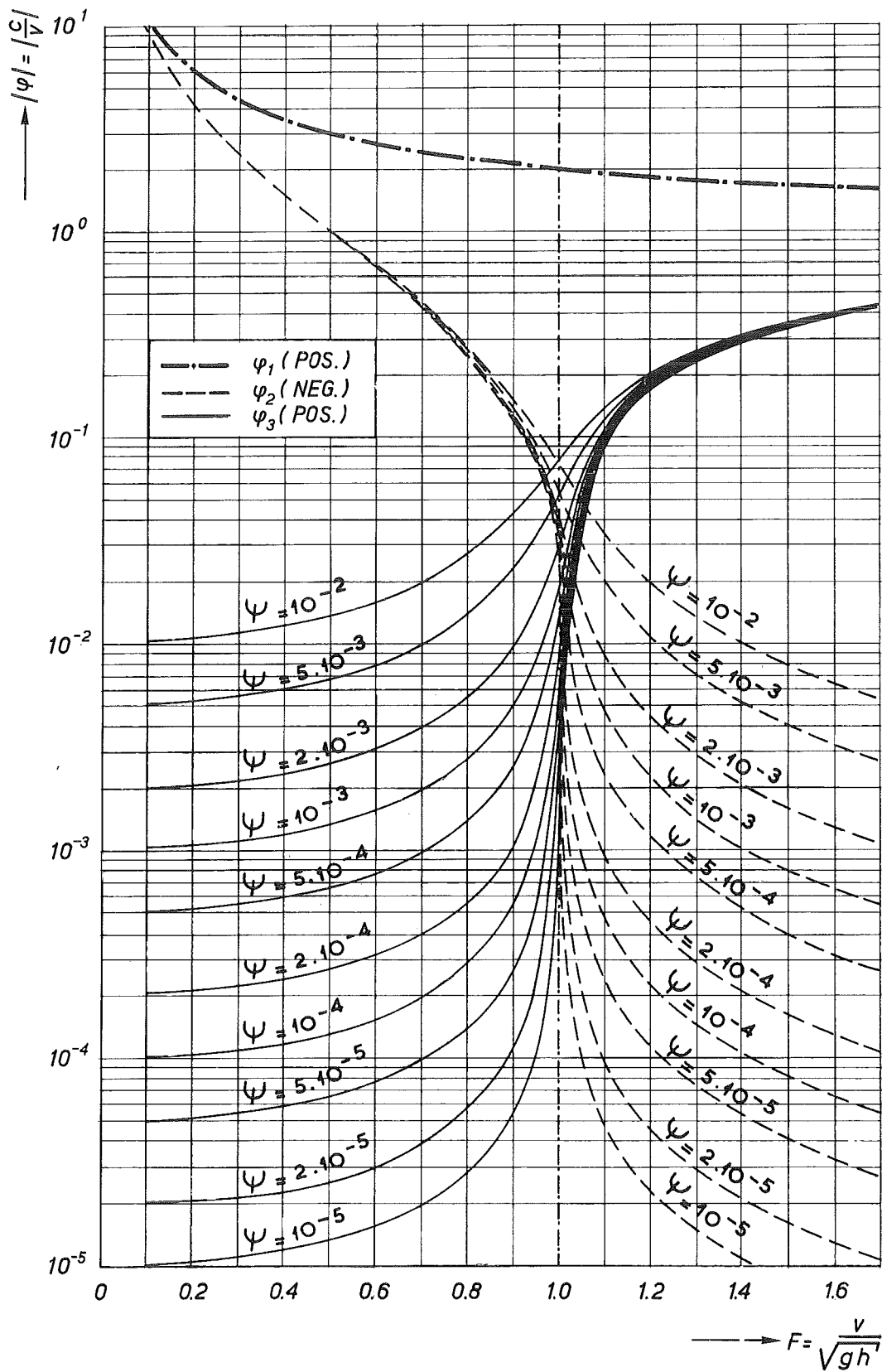


Figure 3.6.2 Relative Celerities

Lately, however, some analytical solutions have been derived by introducing a few simplifications into the above equations. Depending on the simplifications, the above set of equations reduces to a parabolic or a hyperbolic equation. For the derivations reference should be made to de Vries [40, 41] and Vreugdenhil and de Vries (1973) [42]. Here it is sufficient to indicate the derived parabolic equation:

$$\frac{\partial z}{\partial t} - K \frac{\partial^2 z}{\partial x^2} = 0 \quad (3.6.7)$$

with

$$K = \frac{1}{3} \frac{C^2 q}{v^2} \frac{ds}{dv} (= \frac{1}{3} \frac{bs}{I}) \quad (3.6.8)$$

The main assumption made for the derivation of these equations is the assumption of uniform flow. If further as sediment-transport equation  $s = a.v^b$  is used, the value for  $K$  may be approximated (by linearization of Eq.3.6.8) by  $K \approx 1/3 bs/I$ . The parabolic equation may now be solved analytically if  $K$  can be approximated by a constant. In other cases it is better to use the hyperbolic model, which, however, is outside the scope of this Report. An application of the parabolic solution will be given in Para. 3.6.4. In view of the assumption of uniform flow, the solution may only be used for  $x \gg K/c$ , which can be shown to be equivalent to  $x \gg 1/3 (1-F^2) h/I$ .

For the parabolic equation also a varying discharge can give an analytical solution. The time-variation is reflected in the differential equation by the fact that now  $K=K(t)$ . For the boundary conditions used, the solution then reads:

$$z(T) = z_0 \operatorname{erfc} \left( \frac{x}{2 \sqrt{\int_0^T K(t) dt}} \right)$$

This gives the possibility in principle to make estimates on some information of  $Q(t)$ , and a complete information on the hydrograph (or  $h(t)$ ). (The hyperbolic equation does not permit an analytical solution for this case.)

The boundary conditions for solving the parabolic equation are:

- The water-level at some places  $x = L$  (as a function of time).
- The discharge at the upper boundary  $Q(o, t)$ .
- The amount of bed-material load at the upper boundary  $S(o, t)$ .
- The initial bed-level  $z(x, o)$ .

#### Computation c

For Computation c the same basic equations are used as for Computation b (Equations (3.6.4), (3.6.5), (3.2.2) and (3.6.3)). A straightforward difference-scheme to solve these equations appears to be unstable under certain conditions. The following difference-scheme may, therefore, be used (see also de Vries [39] and Figure 3.6.3).

$$\frac{z_4 B_4 - \left\{ \frac{1}{2} \alpha z_1 B_1 + (1-\alpha) z_2 B_2 + \frac{1}{2} \alpha z_3 B_3 \right\}}{\Delta t} + \frac{S_3 - S_1}{z \Delta x} = 0 \quad (3.6.9)$$

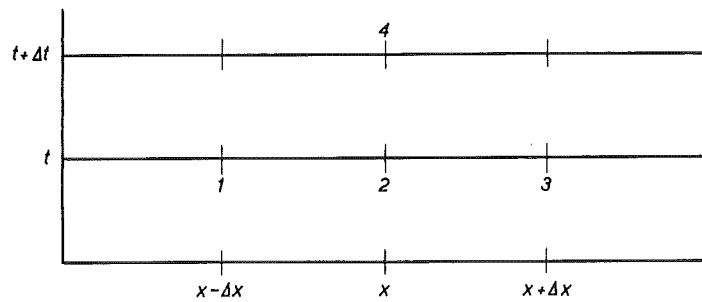
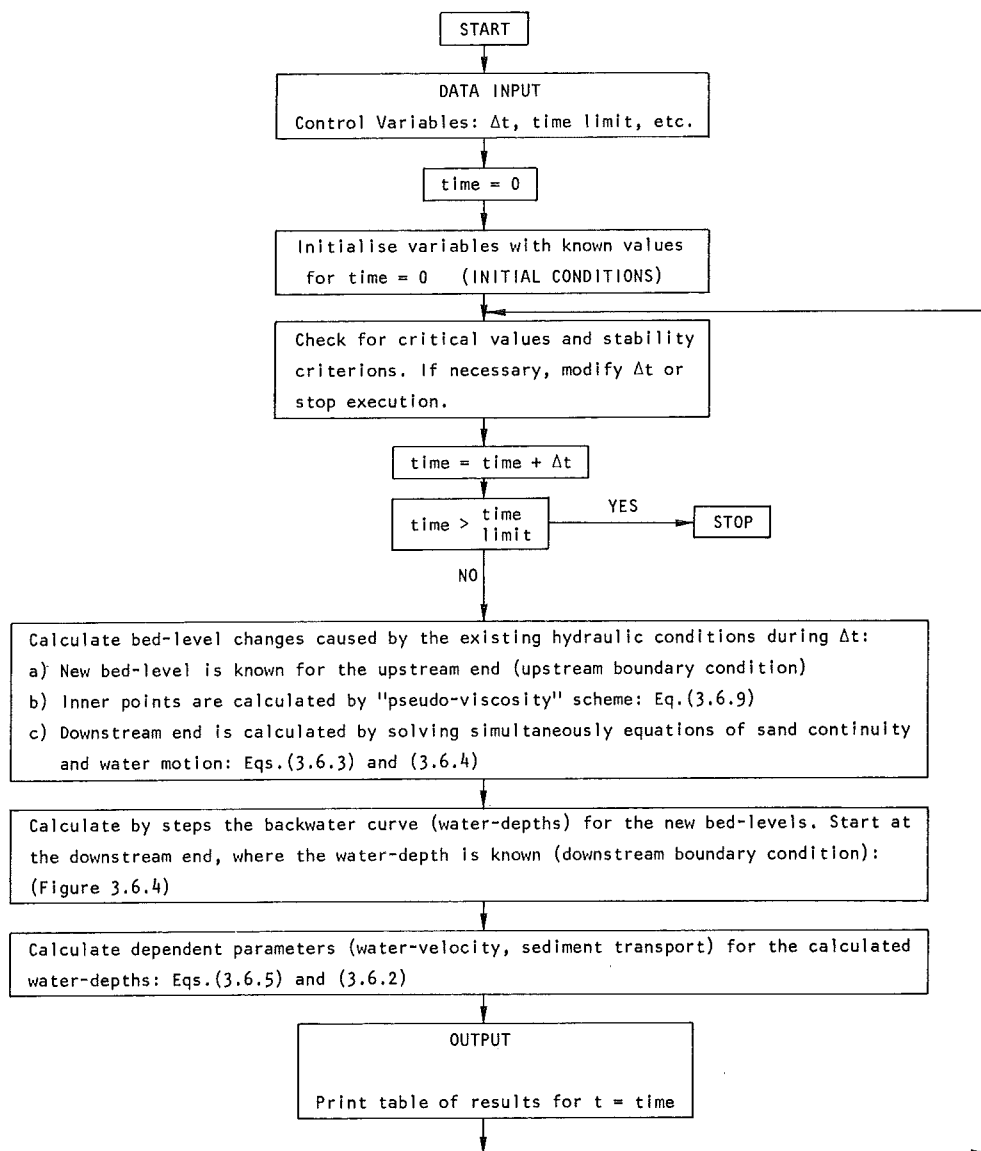


Figure 3.6.3 Difference-scheme

The boundary conditions are the same as given for Computation b. A flow diagram of the computation is given below. Also this method is used in Para. 3.6.4.



Before giving an example of the application of the three methods, further comment must be made about the equations used in relation to the Río Magdalena.

3.6.3. The influence of the suspended-load on the behaviour of the river-bed

In Para. 3.5.2 the conclusion was drawn that the equation of Engelund and Hansen is in good agreement with the sediment transport measurements. However, in Para. 3.3.4 an attempt was made to bring the distribution of the suspended particles as found on the Rfo Magdalena into agreement with the available theories, but a proper solution could not be reached. Consideration must, therefore, be given to what the influence of the distribution of the suspended particles on the behaviour of the river-bed will be if morphological computations are carried out in transient flow.

The equation of continuity for the bed-load can be written as:

$$\frac{\partial z}{\partial t} + \frac{\partial s_b}{\partial x} - D + E = 0 \quad (3.6.10)$$

in which  $z$  = bed-level;

$s_b$  = bed-load per unit width;

$D$  = deposition; and

$E$  = scour of the river-bed (see Figure 3.6.4).

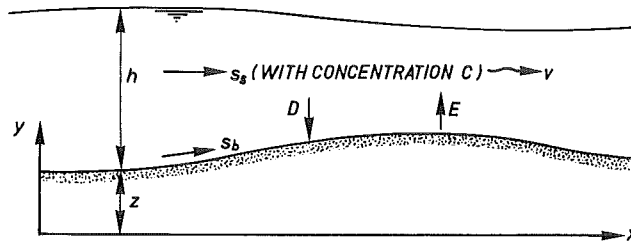


Figure 3.6.4 Continuity of Bed-material Load

The equation of continuity for the suspended-load is:

$$\frac{\partial}{\partial t} (Ch) + \frac{\partial s_s}{\partial x} + D - E = 0 \quad (3.6.11)$$

in which  $C$  = average concentration of suspended particles in the vertical with height  $h$ ; and  $s_s$  = suspended-load per unit width.

Combination of Equations (3.6.10) and (3.6.11) yields to the overall equation of continuity:

$$\frac{\partial z}{\partial t} + \frac{\partial}{\partial t} (Ch) + \frac{\partial s_t}{\partial x} = 0 \quad (3.6.12)$$

in which  $s_t = s_b + s_s$  = total-load per unit width.

If the bed-load is only a negligible percentage of the total-load (as indeed is the case on the Rfo Magdalena), the total-load can be expressed as:

$$s_t \approx s_s = \alpha C q \quad (3.6.13)$$

in which:

$\alpha$  = coefficient in view of the non-uniform distribution of the velocity and the concentration in the vertical.

Moreover, the total-load formula of Engelund and Hansen can be simplified to:

$$s_t = \alpha v^b \quad (3.6.14)$$

The equation of motion of the water is:

$$v \frac{\partial v}{\partial x} + g \frac{\partial h}{\partial x} + g \frac{\partial z}{\partial x} + \frac{g v |v|}{c^2 h} = 0 \quad (3.6.4)$$

Inserting  $q = vh$  in Eq.(3.6.4) yields:

$$(v - \frac{g \cdot g}{v^2}) \frac{\partial v}{\partial x} + g \frac{\partial z}{\partial x} + \frac{g v |v|}{c^2 h} = 0 \quad (3.6.15)$$

(In Equations(3.6.4) and (3.6.15)  $C$  represents the Chézy roughness. Implicitly, it has been assumed that the presence of the suspended particles does not have any bearing on the equation of motion.)

Combining Equations (3.6.13) and (3.6.14) with (3.6.12), the latter yields:

$$\frac{\partial z}{\partial t} + \frac{\partial}{\partial t} \left( \frac{\partial}{\alpha} v^{b-1} \right) + \frac{\partial}{\partial x} (av^b) = 0$$

or, written in a simplified form:

$$\frac{\partial z}{\partial t} + \beta \frac{\partial v}{\partial t} + \gamma \frac{\partial v}{\partial x} = 0 \tag{3.6.16}$$

In which:

$$\beta = \frac{b-1}{\alpha} \frac{s}{v^2}$$

$$\gamma = b \frac{s}{v}$$

Equations(3.6.15) and (3.6.16) can be solved by means of the theory of characteristics. It can be shown that three characteristic celerities (c) are present. The first two represent the celerities of small disturbances at the water-surface travelling upstream or downstream; de Vries (1969) [39]. The third root (c<sub>3</sub>) expresses the celerity of disturbance of the bed and is determined by the following relation:

$$\begin{vmatrix} -c_3 & \gamma - \beta \cdot c_3 \\ g & v - \frac{g \cdot q}{v^2} \end{vmatrix} = 0$$

or,

$$c_3 = \frac{\gamma q}{\beta g + \frac{g \cdot q}{v^2} - v} \tag{3.6.17}$$

If in Equation (3.6.17)  $\beta = 0$  is inserted, the expression is found when only bed-load is considered, Equation(3.6.17) then yields:

$$\frac{c_3}{v} = \frac{\Psi}{1 - Fr^2} \tag{3.6.18}$$

with  $\Psi$  = transport parameter.

For small Froude numbers the term (-v) in Equation(3.6.17) may be neglected in relation to the other terms. The influence of the presence of the suspended-load is expressed by the value of the term ( $\beta g$ ) in relation to the term ( $gq/v^2$ ), as

$$\frac{\beta g v^2}{gq} = \frac{b-1}{\alpha} \cdot \frac{s}{q} = (b-1)C \ll 1 \tag{3.6.19}$$

This may be seen when remembering that for the Engelund/Hansen equation  $b = 5$  while for the Rfo Hagdalena  $C \leq 0.01$ . The second term of Equation (3.6.16) can thus be neglected, and this equation can be simplified to the well-known expression:

$$\frac{\partial z}{\partial t} + \frac{\partial s}{\partial x} = 0 \tag{3.6.3}$$

However, it must be stressed that only a transport equation which describes the total-load (as, for example, the Engelund/Hansen equation) can be used. It can be concluded that the change in the sediment transport from one section to another results also, as far as the suspended-load is concerned, only in changes of the river-bed level, while the changes in the concentration of the suspended particles are of hardly any influence.

As it is theoretically impossible to distinguish between bed-load and suspended-load, an exact definition of the bed-level is also impossible. A more strict derivation is possible when no distinction is made between bed-load and suspended-load but bed-material load is used. This latter derivation is more complicated, but finally leads to the same results and has therefore not been given here.

### 3.6.4. Application

#### Introduction

In Figure 3.6.5 the strongly schematized situation near Pto. Berrfo is given.

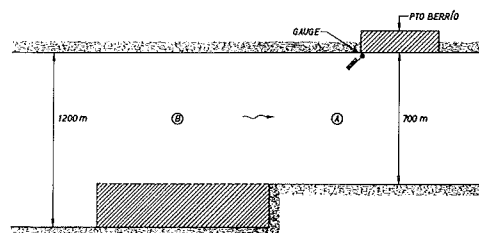


Figure 3.6.5 Schematized Situation (Pto. Berrfo)

## II, 3.6

The navigation conditions in the narrow part (A) are good, but in the wider part (B) the river is too shallow in the low water season. An improvement might be obtained by extending the narrow part by means of river-works, but then the following questions have to be answered:

- 1) Does the section with a width of 700 m indeed offer sufficient depth when the water-level is at L.R.L. (equilibrium conditions, Computation a)?
- 2) What are the hydraulic changes upstream of the river-works after the execution of these works? (As will be shown, the water-level gradient in the wider part is different from that in the narrow part. This means that the river-works will cause a change in the level of the bed and the water-surface in B which will also result in bed-level and water-level changes upstream of B).
- 3) How long will it take for the changes in bed-level and water-level to reach a certain place along the river upstream of B?
- 4) How long will it take for a new equilibrium situation to be reached in B?
- 5) What is the influence of the regime on the bed-level, or, more specifically, is sufficient depth at L.R.L. also available after a rapid drop of the water-level?

Answers to Questions 1 and 2 are required to judge the feasibility of the river-works. These answers can be found from a relatively simple computation (type a).

Answers to Questions 3, 4 and 5, which contain the time as parameter, can be given by Computation c. Question 3 can, however, also be answered by way of Computation b.

The available data are:

- Water-level data on the Pto. Berrío gauge.
- A stage-discharge curve at Pto. Berrío.
- Water-level gradient in the narrow section ( $I = 25 \cdot 10^{-5}$ ).
- $D_{35} = 350 \cdot 10^{-6} \text{m}$ ,  $D_{50} = 400 \cdot 10^{-6} \text{m}$  and  $2D_{65} = 1200 \cdot 10^{-6} \text{m}$ .

### Computation a (Questions 1 and 2)

To obtain the depth below L.R.L. in the narrow part (Question 1) it is not necessary to follow the flow diagram given in Para. 3.6.2 because it is not the sediment transport which is given as boundary condition, but the water-level gradient. The depth will be calculated for dominant discharge by applying Equation (3.6.4). To obtain the depth below L.R.L., it will be assumed (for a first estimate) that the bed-level does not change when the water-level drops to L.R.L.

The dominant discharge is estimated as 2,500 m<sup>3</sup>/s from Figures 2.3.6 and 3.5.9. (More about the selection of the dominant discharge is given in Para. 3.6.5).

Equation (3.6.4) was given as:

$$\frac{d}{dx} \left( \frac{\bar{v}^2}{2g} \right) + \frac{d\bar{h}}{dx} + I_b = - \frac{v^2}{C^2 \bar{h}}$$

The terms  $d/dx(v^2/2g) + d\bar{h}/dx + I_b$  represent the slope of the energy-level which can be taken as  $25 \cdot 10^{-5}$  because the term  $d/dx(v^2/2g)$  can be neglected for this section.

As further  $\bar{v} = Q/B \bar{h}$ , Equation(3.6.4) reduces to the following expression:  $25.10^{-5} = Q^2/C^2 B \bar{h}^3$  where C should be found according to the method given in Para. 3.4. This has to be solved by iteration as follows:

Estimate  $C = 45 \text{ m}^{1/2}/\text{s}$ , which results in  $h = 2.93 \text{ m}$ ,  $v = 1.22 \text{ m/s}$ ,  $C' (= 18 \log \frac{12h}{2D_{65}}) = 80 \text{ m}^{1/2}/\text{s}$ . The value of  $C''$  is found from Figure 3.4.3 as  $C'' = 54 \text{ m}^{1/2}/\text{s}$ . From Equation(3.4.10) the value of C is found as  $C = 45 \text{ m}^{1/2}/\text{s}$ . If this value would have been different from the estimated value further iteration is required.

This means that the available depth with dominant discharge (level at the Pto. Berrfo gauge 1.65 m) is 2.93 m. As the L.R.L. at Pto. Berrfo is 0.60 m, the available depth below L.R.L. is  $2.93 - (1.65 - 0.60) = 1.88 \text{ m}$  which is sufficient for the river stretch upstream of Pto. Berrfo.

The second question dealt with the hydraulic changes upstream. These changes concern mainly a change in slope over the section in which river-works are carried out. In equilibrium conditions this slope will become  $25.10^{-5}$ ; the present slope is not known but can be computed when the sediment transport for the present narrow section is taken as a boundary condition. First, this transport will therefore be computed, after which a computation according to the flow diagram can be carried out. This flow diagram can be simplified because of the constant width, which means that the computation only has to be carried out for one cross-section. As the sediment upstream has the same diameter and density, the dimensionless sediment transport  $s/\sqrt{\Delta g D_{50}^3}$  may be used.

$$s/\sqrt{\Delta g D_{50}^3} = \frac{0.05}{0.6} \times \frac{45^2}{9.8} \times \left( \frac{1.22^2}{45^2 \times 1.68 \times 400.10^{-6}} \right)^{5/2} = 21.54$$

In the wider part this will be :  $21.54 \times \frac{700}{1,200} = 12.57$

For the narrow part estimate  $C = 40 \text{ m}^{1/2}/\text{s}$ , then:

$$12.57 = \frac{0.05}{0.6} \times \frac{40^2}{9.8} \left( \frac{v^2}{40^2 \times 1.68 \times 400.10^{-6}} \right)^{5/2} \text{ which gives } v = 1.02 \text{ m/s.}$$

Then  $h = 2.04 \text{ m}$ ;  $C' = 78 \text{ m}^{1/2}/\text{s}$ ;  $(C')^2 \Delta D_{35}/\bar{v}^2 = 3.4$ ;  $C'' = 44 \text{ m}^{1/2}/\text{s}$  and  $C = 38 \text{ m}^{1/2}/\text{s}$ .

The computation will be repeated for  $C = 38 \text{ m}^{1/2}/\text{s}$ . The results are:

$v = 0.99 \text{ m/s}$ ;  $h = 2.1 \text{ m}$ ;  $C' = 78 \text{ m}^{1/2}/\text{s}$ ;  $C'' = 43\frac{1}{2} \text{ m}^{1/2}/\text{s}$ ; and  $C = 38 \text{ m}^{1/2}/\text{s}$  (the depth below L.R.L. becomes 1.05 m, which is insufficient). The gradient I may be taken as  $v^2/C^2 h = 32.3 \times 10^{-5}$ . The future gradient in this section will be  $25 \times 10^{-5}$ , which means that the gradient will be reduced by  $7 \times 10^{-5}$ . If the river-works are extended over a length of 3 km, the drop in level upstream of these works will be 0.21 m (see Figure 3.6.6). If several such works are carried out, the drop in water-level and bed-level may be appreciable, and as, moreover, large quantities of sand have to be removed by the river, not too many of these constructions should be carried out at the same time.

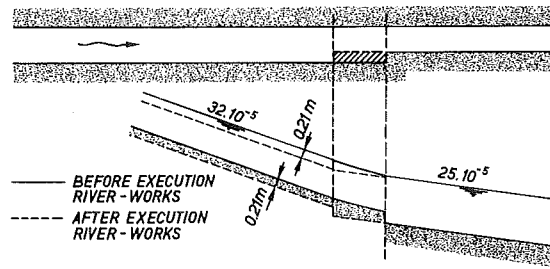


Figure 3.6.6 Changes Expected due to River-works

Computation b (Question 4)

From the Computation a it followed that the river-works will cause a change in the gradient of the bed-level and the water-level, which will have its effect upstream of the river-works. By means of Computation b it will be possible to obtain an insight into the time required for such an influence to be noticed at a place far upstream (Pto. Inmarco, La Dorada) without making use of the complete numerical Computation c. The parabolic solution may only be used for  $x \gg 1/3 (1-Fr^2) \cdot h/I$  or  $x \gg 5$  km for the Río Magdalena near Pto. Berrío.

For example, the question may arise how much time is required before half of the final lowering in levels can be expected at Pto. Inmarco (42 km upstream). The parabolic equation can be solved when it is assumed that the drop in level at Pto. Berrío is instantaneous; this is allowed as can be concluded from Computation c given later. In that case  $\Delta z = \Delta z_0 \operatorname{erfc} (x/2\sqrt{Kt})$ , or for half the final drop in levels:

$$\operatorname{erfc} \left( \frac{x}{2\sqrt{Kt}} \right) = \frac{1}{2}$$

With the help of a table for error functions, it can be found that  $x = 0.96\sqrt{Kt}$ , in which  $x = 42 \cdot 10^3$  m;  $K = 1/3 bs/I \text{ m}^2/\text{s}$  and  $s/\sqrt{\Delta g D_{50}^3} = 21$  (determined with Computation a and therefore  $s = 6.8 \times 10^{-4}$ ). The value of  $K$  can now be found by introducing this value for  $s, b = 5$  and  $I = 40 \times 10^{-5}$ . It follows that  $K = 2.84 \text{ m}^2/\text{s}$  and the time  $t$  required is found to be  $674 \times 10^6 \text{ s} \approx 21$  years.

Computation C (Questions 3 and 5)

With this computation the regime influence as well as the time required to obtain equilibrium will be found (Questions 3 and 5).

The regime has been schematized as indicated in Figure 3.6.7, which has been based on the 50%-frequency curve of Pto. Berrío, with a steep fall of the water-level to L.R.L. at the end.

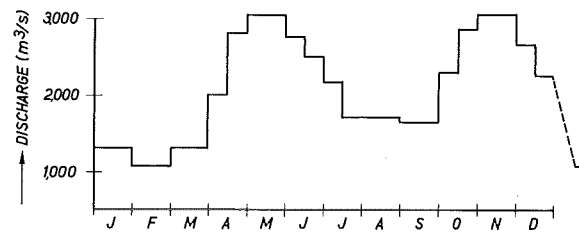


Figure 3.6.7 Regime Used in the Computation

The results of the computation itself are given in Figure 3.6.8. In Figure 3.6.8a the path of the bed-level and water-level is given as a function of place; in Figure 3.6.8b the minimum depth is given as a function of the time.

Before commenting further on these results, something more about the boundary conditions for the computation must be said. As mentioned earlier, the boundary condition  $S(o,t)$  often presents problems. To obtain the correct value for  $S(o,t)$  from a sediment-transport equation, a fixed cross-section should be assumed at  $x = 0$ , or, in other words, a fixed upper boundary. This, in fact, has been done for the computation of Pto. Berrfo given here. To prevent the (incorrect) influence of a fixed upper boundary, this boundary should be taken far away, with the distance depending on the duration over which computations are extended and on the propagation of a disturbance in the bed. For the computation given here (which is only an example) the upper boundary is in fact too close. The minimum depth and the time required before an equilibrium is reached locally will be practically correct, but the actual bed-level and the propagation of the disturbance upstream will be incorrect.

As taking the upper boundary far upstream makes the computations costly, another boundary condition may be tried upstream which does not fix the bed-level and does not require a boundary so far upstream. For the computation of the Rfo Regla Confluence and the crossings presented in Part III of this Report, this has been done by making the energy gradient at the upper boundary the normal energy gradient (parallel flow at the upper boundary), which has a similar effect as taking the upper boundary far upstream. From Figure 3.6.8a it may also be seen that this condition is very close to the real situation.

As far as the computation for Pto. Berrfo is concerned, it can be seen that after 43 days an equilibrium has not quite been reached, but that after 86 days there is an equilibrium situation locally. This means that no dredging would be required. It may also be concluded that the assumption of an instantaneous change as used for Computation b is reasonable. From Figure 3.6.8b it may be concluded that the minimum available depth in the section will be 1.76 m below L.R.L. (this value should be compared with the value of 1.88 m as found with Computation a).

### 3.6.5. Selection of a dominant discharge

For the application of the Computations a and b, a dominant discharge has to be determined which sufficiently represents the regime of the river to obtain reliable values for the parameters to be computed. It can be demonstrated theoretically that no single discharge can be determined which represents correctly different parameters, or even one parameter at different places. Moreover, the discharge found corresponds to present conditions and not to future conditions (see Prins and de Vries (1971) [43]).

Nevertheless, in literature several methods are given, based on intuitive considerations. Several of those methods can, with some simplifying assumptions, be written as:

$$(h_o)^n = \frac{1}{T} \int_0^T h^n dt \quad (3.6.20)$$

in which T is the duration of a sufficiently long period,  $h_o$  the dominant depth and  $Q_o$  the dominant discharge corresponding with  $h_o$  in the stage-discharge curve.

For the methods available the value of n varies between 1 and  $1\frac{1}{2}$ . When Q is simplified as  $Q = Q(h^{3/2})$  this means that the dominant discharge ( $Q_o$ ) varies between a value corresponding with the average depth ( $\bar{h}$ ) and a value corresponding with the average  $h^{3/2}$  (which is  $\bar{Q}$ ).

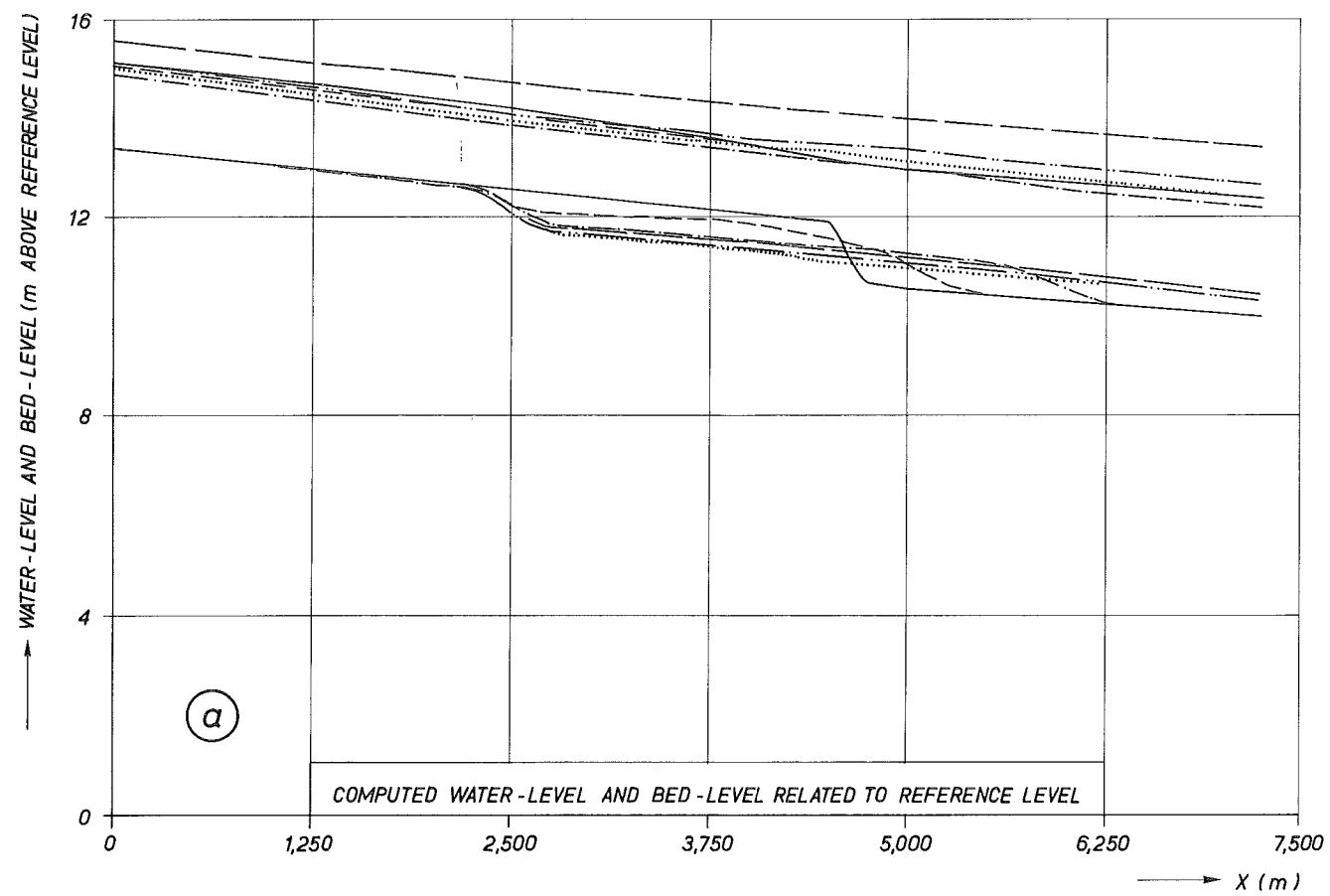
For a first insight it is not important which discharge is taken, and the average discharge is generally sufficient. For the final design, a computation of type c is always recommended.

In Para. 3.7 it will be seen that the use of an hydraulic model may also be of great help; but a dominant discharge selected in a more reliable way may then be required [43].

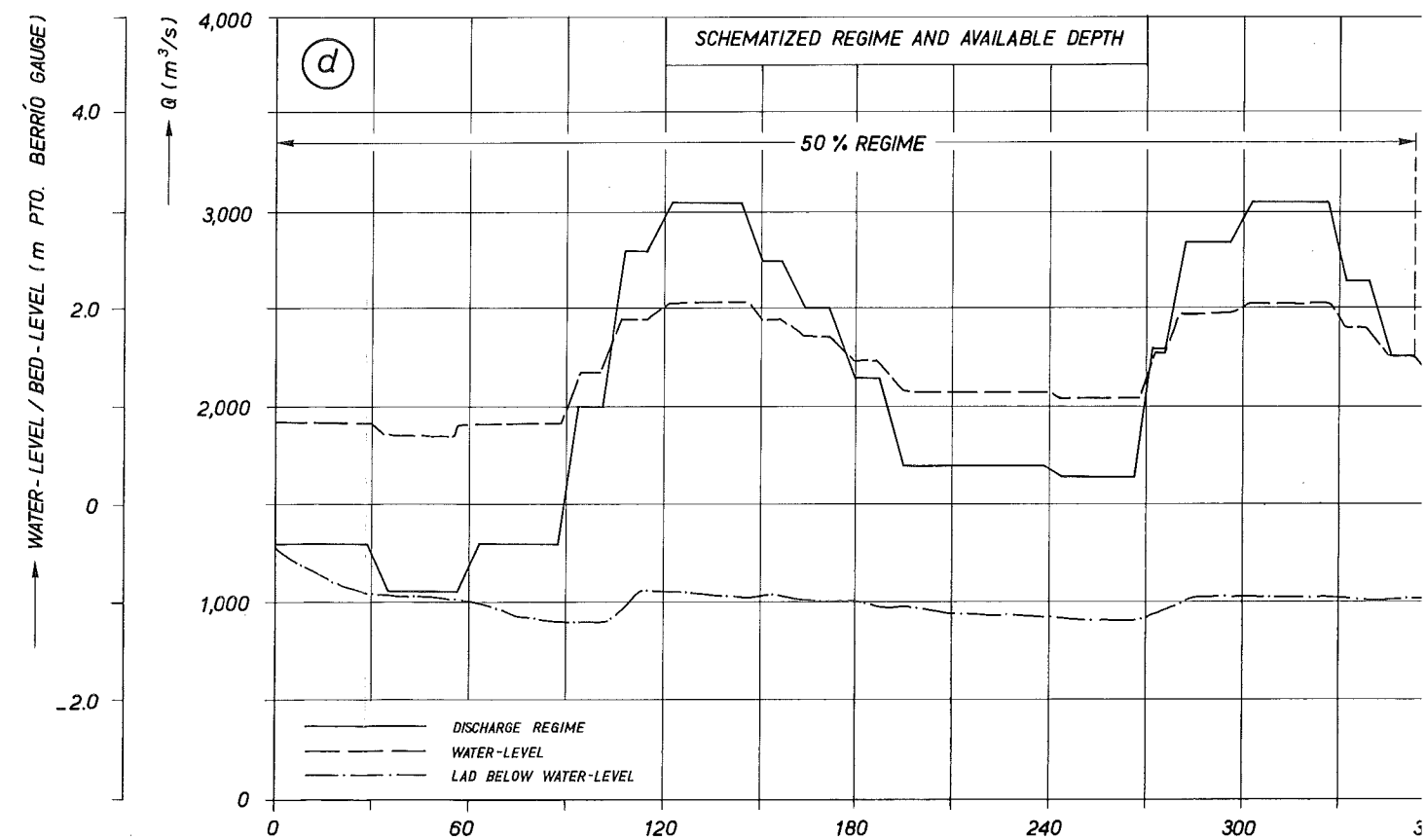
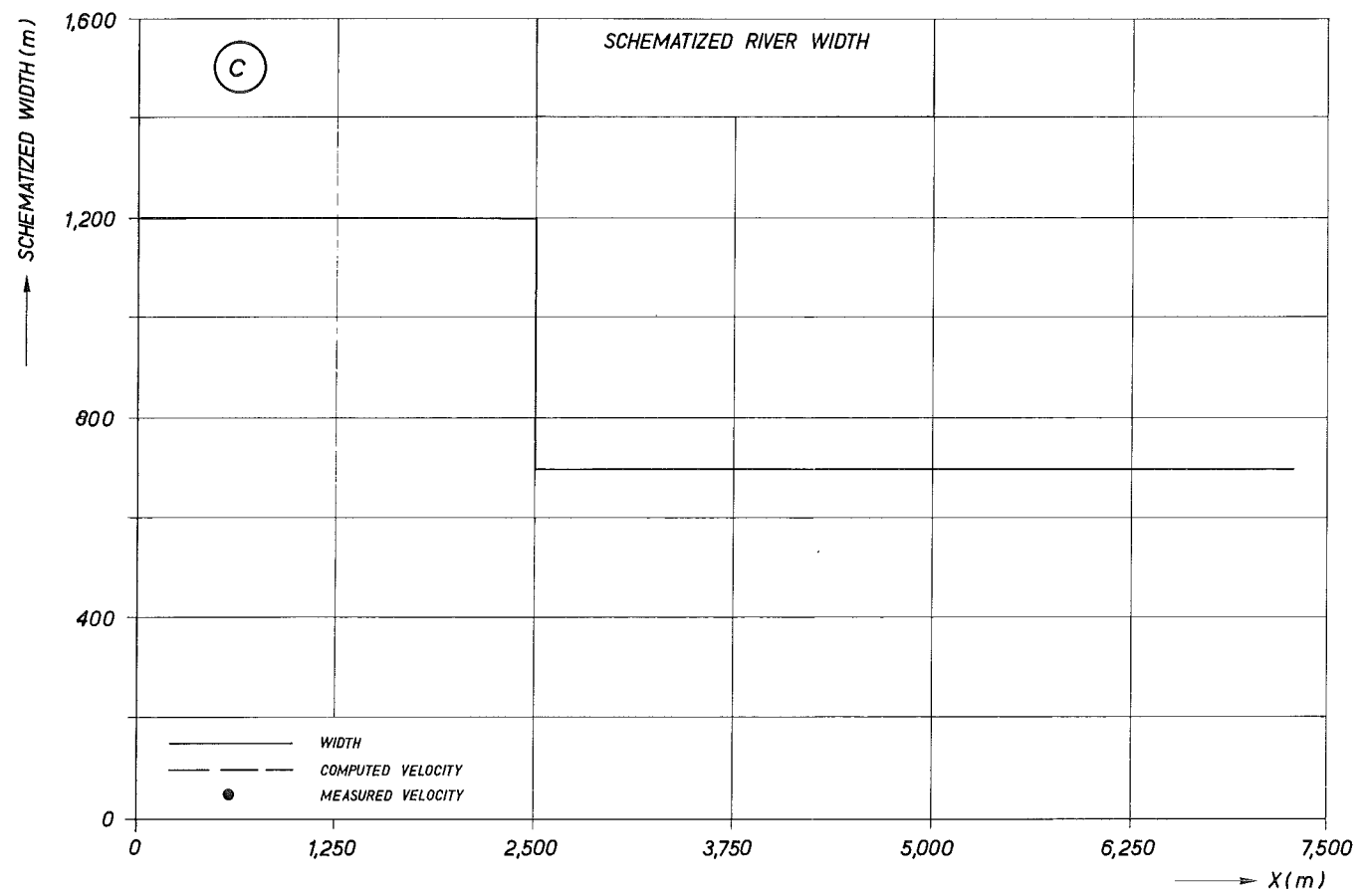
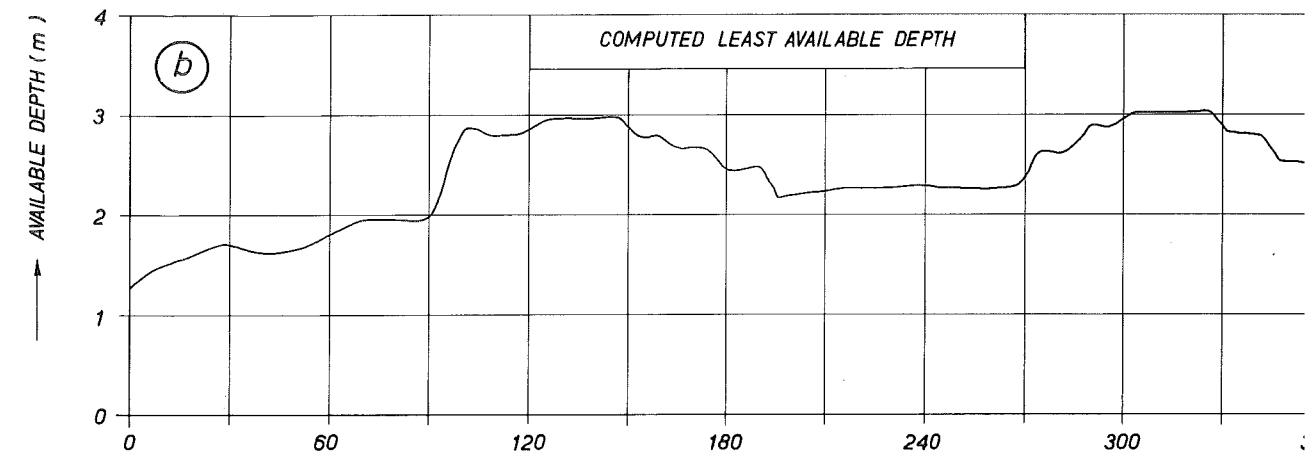
## 3.7. THREE-DIMENSIONAL PHENOMENA

### 3.7.1. Introduction

So far in this Chapter the river has only been dealt with in a one-dimensional sense. In reality a river is three-dimensional, and the configuration in a horizontal plane is called the plan-form. In the foregoing part of this Chapter the "tools" have been given to predict the changes in a river bed which occur as a result of a change in plan-form or in the hydraulic conditions. In this Paragraph on the one hand schematization of the plan-form into a one-dimensional model will be dealt with (Para. 3.7.2), and on the other hand something will be said about the actual three-dimensional phenomena (Para. 3.7.3). In Para. 3.7.2 something will also be said about the plan-form of natural rivers, as this information is required for the actual design of river-works and as boundary condition for the computation of three-dimensional effects.



WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—	0	0	1,300
- - -	14	6	1,300
· · ·	50	6.5	1,070
—	108	5	2,800
- - -	260	3.5	1,650
· · ·	368	7	1,300



RÍO MAGDALENA NEAR km 730 (PTO. BERRÍO)

RESULTS OF MORPHOLOGICAL COMPUTATIONS

### 3.7.2. Plan-form

The following plan-forms may be distinguished in natural rivers:

- Meandering,
- braiding, and
- straight.

Meandering, called after the Meander River (Büyük Menderes) in Turkey, means that the stream-bed consists of consecutive bends (see Figure 3.7.1).



Figure 3.7.1 Meanders Upstream of La Dorada

When the stream-bed consists of a number of parallel channels, separated by flats and islands, the river is called braiding (Figure 3.7.2). Straight rivers are almost nonexistent. Although short reaches of the channel may be straight, it can be stated as a generalization that reaches which are straight for distances exceeding ten times the channel width are rare (Figure 3.7.3).

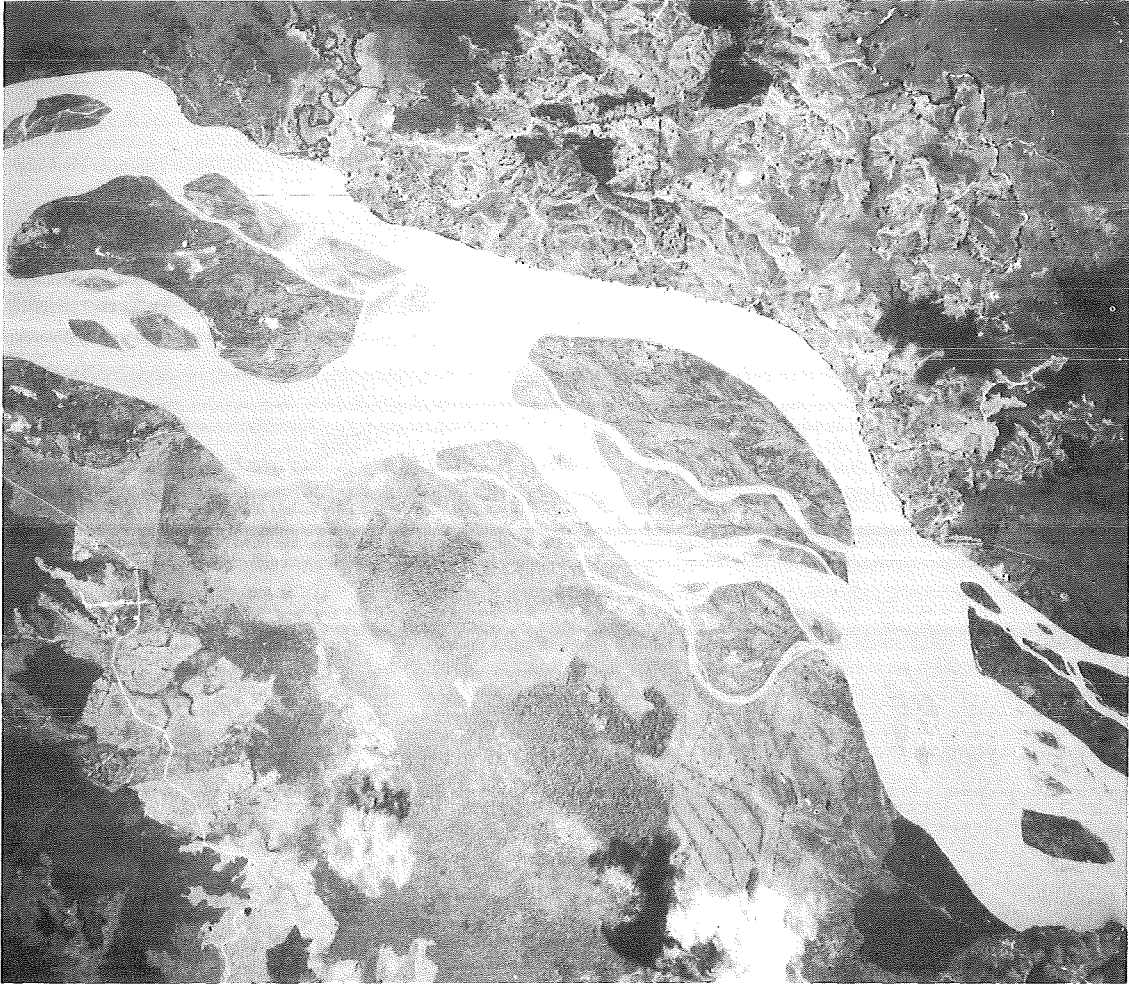


Figure 3.7.2 Braided River (Rfo Magdalena, downstream Pto. Berrfo)

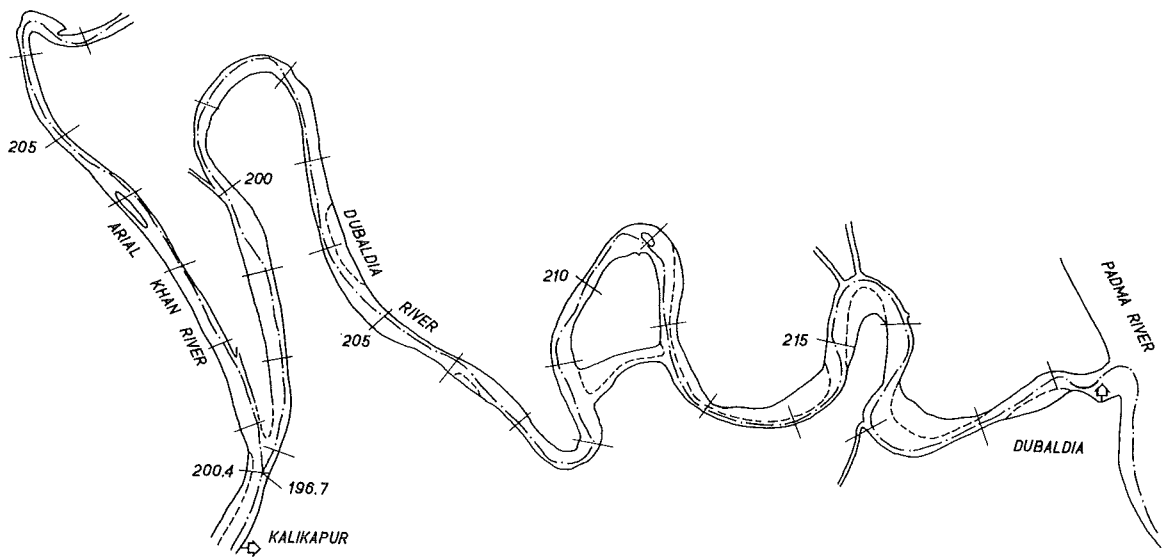


Figure 3.7.3 Straight Reaches in the Dubaldia River, Bangladesh

A meandering river stretch generally gives better navigation conditions than a braided stretch. In braided rivers often none of the branches is predominant. At one time one branch draws more water, at another time another branch, resulting in shallow channels in either branch. River-works will, therefore, often aim at reducing the braiding by the closure of one or more branches.

Present knowledge cannot give an answer to the question why a river meanders, but statistically some idea can be obtained. In Figure 3.7.4 a graph is given (Leopold et al. (1964)[44]) from which the tendency for braiding or meandering can be predicted.

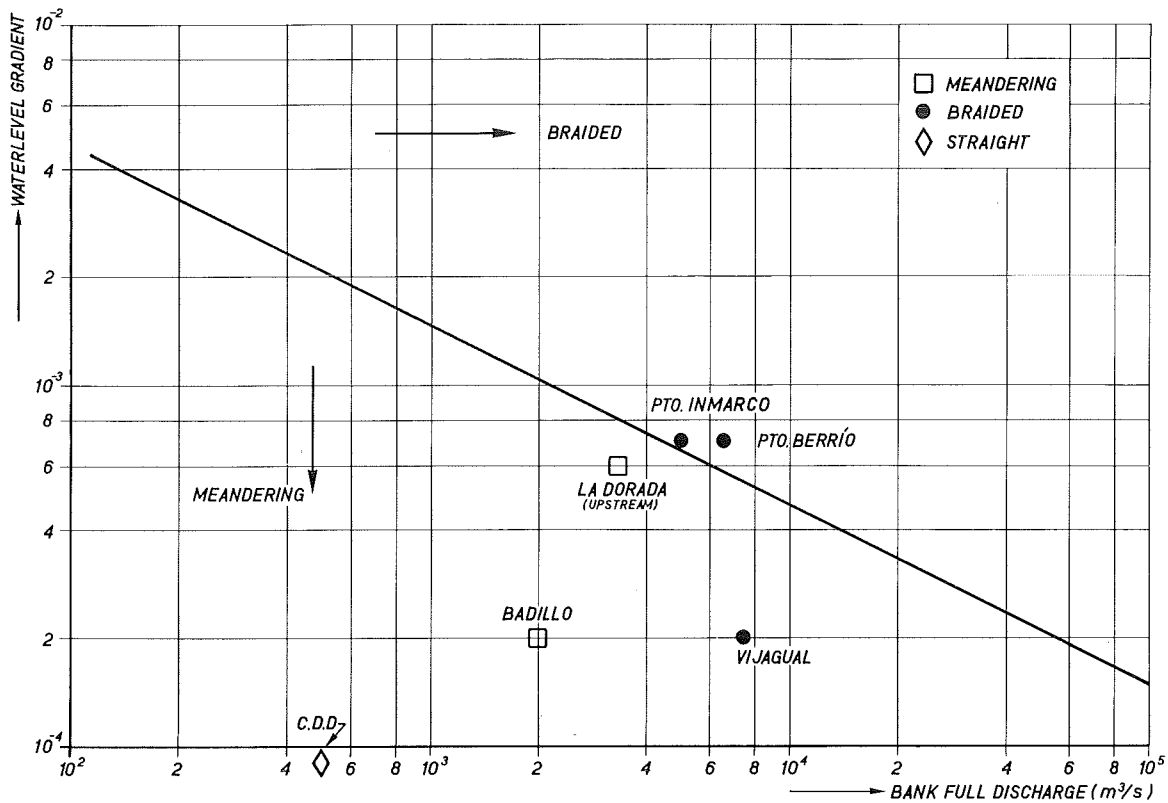


Figure 3.7.4 Prediction of Braiding or Meandering

It should be kept in mind that this graph is mainly based on U.S. rivers and extrapolation to other rivers has to be done with caution. A few plots have been made for the Río Magdalena system and, in general, the results fit the data of Leopold and Wolman (Río Magdalena braiding, Canal del Dique straight with a tendency to meandering). The plot of La Dorada should be on the line, as near La Dorada the character of the river changes from meandering to braiding. For the Río Magdalena the discharge for bankfull stage is difficult to determine in regions of "cienagas", if the bankfull discharge is taken from a stage-discharge curve (the change in  $dQ/dh$  indicates the "feeding" of the cienagas).

II, 3.7

A schematization of the plan-form is required to make computations (one-dimensional) possible. In Figure 3.6.5 the situation at Pto. Berrfo was schematized in a very simple way, namely in a straight river section. Although in practice the schematization would be more detailed, no great difficulties would be encountered (see also Part III, the Río Regla Confluence). Much more difficult is the schematization of the plan-form for crossings and braided river sections.

In Figure 3.7.5 an aerial photograph is given of the situation of the crossing at km 841. In Figure 3.7.6 a schematization of the plan-form is given together with two longitudinal profiles (one of which is the computed longitudinal profile).

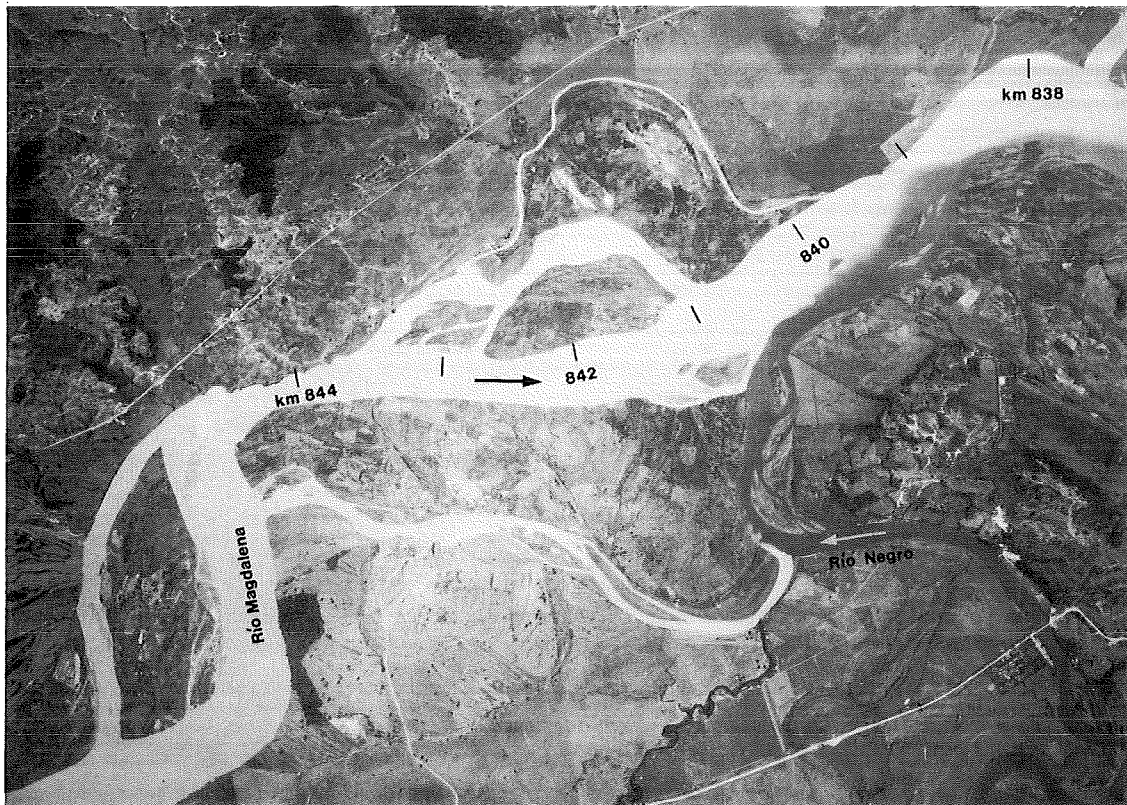


Figure 3.7.5 Crossing at km 841

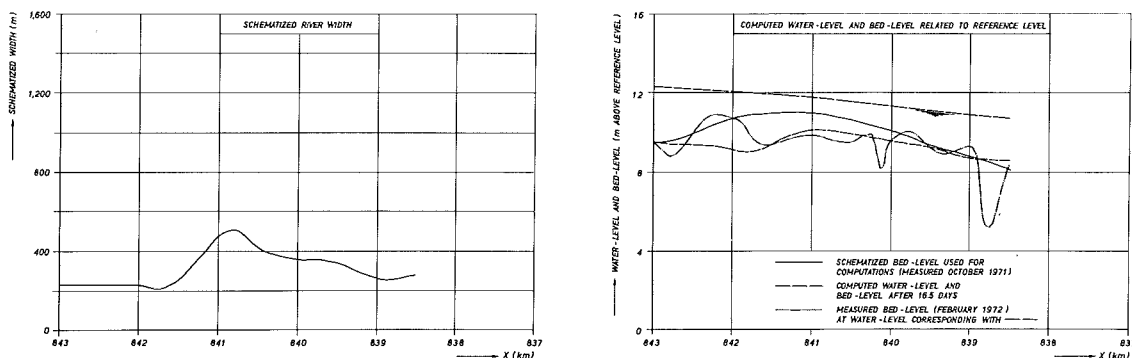


Figure 3.7.6 Schematization of the Crossing at km 841

For this schematization, use could be made of recent aerial photographs and the route map (Figure 3.3.19). An impression of the flow-lines could be obtained from the photographs, as well as reasonable insight into the underwater topography. It will be clear that after some time the path of the flow-lines and the underwater topography change so that schematization is only possible with the help of measured flow-lines and the underwater topography as obtained from route maps and local soundings. Route maps should, therefore, be done frequently and include a number of cross-sections over the crossings. New photographs will be required about every five years.

The given schematization is made by taking the total width of the two main channels and assuming that no water flows over the sand-bank. Also the influence of the Rio Negro is neglected. Taking the two channels together means that the following assumptions are made: The water-level gradient, the depth and the channel roughness in both branches are equal.

In some cases these assumptions may be so far from the real situation that actually two or more channels have to be considered in the computation, which is then carried out for a network. As a consequence, the sediment transport division over the branches has to be estimated (see Para. 3.8). As an intermediate form, a schematization may be made with local (lateral) entrance or subtraction of discharge and sediment. These amounts are usually not known from measurements and have to be estimated.

All these schematizations are very subjective and depend very much on the skill and experience of the engineer.

Schematization of braided river sections, as required for the computation of the effect of the closure of secondary branches, is done in a similar way but is generally somewhat easier. The most difficult part is estimating the transport division.

For the computation of the three dimensional effects, it is necessary to know the plan-form or, more specifically, the bend-radius corresponding to a certain channel width. At present only statistical information on this subject is available, and this is given in Table 3.7.1.

Author(s)	Formulae
Inglis (1947) [45]	$ML :: Q^{0.5}$
Charlton and Benson (1966) [46]	$ML :: Q^{0.555} \times D^{-0.285}$
Anderson (1967) [47]	$ML :: Q^{0.5} \times h^{-0.5}$ and $ML :: Q^{0.39}$
Hansen (1967) [48]	$ML :: Q^{0.525} \times D^{-0.316}$
Ackers and Charlton (1970) [49]	$ML :: Q^{0.467 \pm 0.024}$

Table 3.7.1 Meander Length (ML) versus Discharge (Q) and Diameter of Grain-size (D)

Also the graph given by Zeller [50] may give useful information; see Figure 3.7.7.

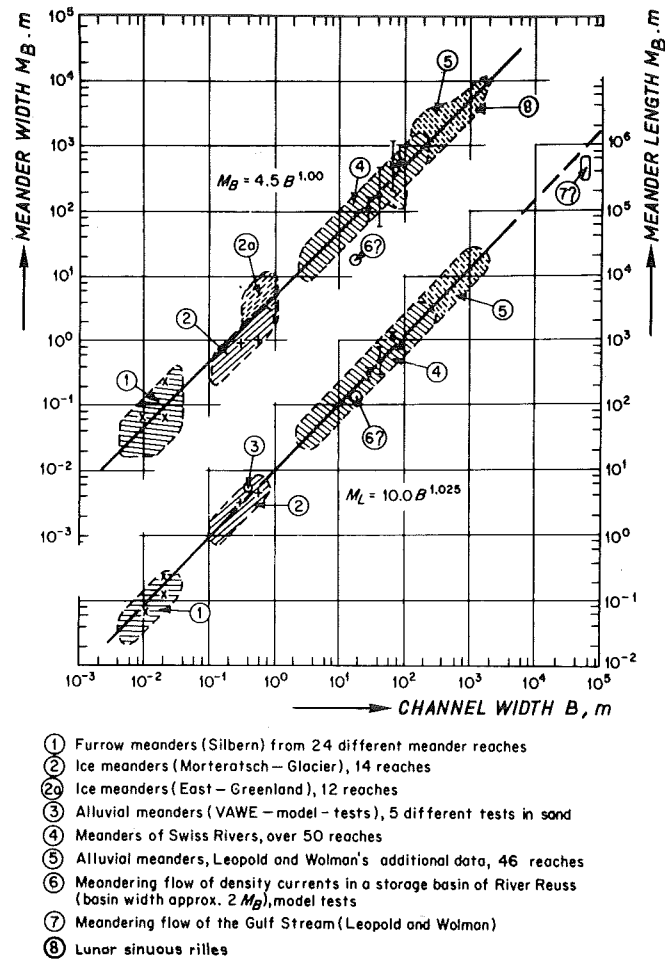


Figure 3.7.7 Empirical Relation for Geometry of Meanders

For the design of river-works the most interesting information would be, as mentioned, the relation between channel width and curvature or bend-radius. It is strange that only a few investigators have explicitly studied this relation. Generally, a relation between meander length and width of the river is given. Leopold and Wolman [51] give:

$$ML = 10.9 \times B^{1.01} \tag{3.7.1}$$

and

$$ML = 4.7 \times R^{0.98} \tag{3.7.2}$$

(R = meander radius).

Combining these equations results in:

$$R = 2.36 \times B^{1.03} \tag{3.7.3}$$

These equations are based on data from U.S. rivers and flume investigations. The data given by Zeller (Figure 3.7.7) support the information of Leopold and Wolman. Zeller gives:

$$ML = 10 B^{1.075} \tag{3.7.4}$$

found under a large range of conditions, even including lunar rilles. Bagnold [52] theoretically arrives at  $R = 2B$  to  $3B$ .

It is possible that with controlled width (generally narrower than the natural width) larger values of  $R$  have to be used in relation to the controlled width ( $B$ ). (On the Missouri it was found that  $R = 10B$  to  $20B$  resulted in a minimum of maintenance cost on the bank protection).

### 3.7.3. Flow in river bends

A complete mathematical description of flow in bends is not possible with the present knowledge of hydraulics. As it is not possible for the fixed boundary case, the loose boundary case is even more tricky. Nevertheless, for the determination of the depth in front of bank protections in bends, some insight into this phenomenon is required. In view of these difficulties, the computations can best be carried out for a constant (dominant) discharge as a secondary approach after one-dimensional computations have been made.

A solution may then be obtained by making the following assumptions:

- Axially symmetric flow. This means that in a vertical the net radial flow is zero (see Figure 3.7.8).
- The internal inertia terms and the acceleration due to the shear-force are neglected.

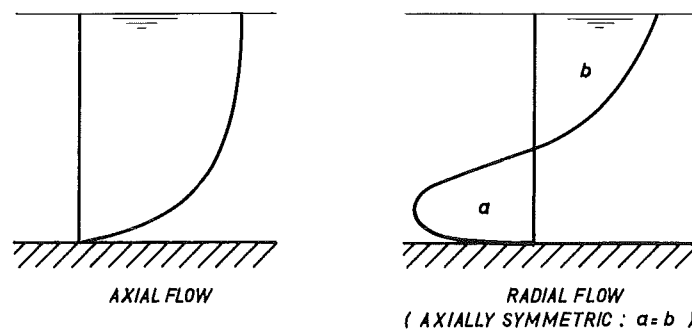


Figure 3.7.8 Axial and Radial Flow

The following equations may now be deduced when a logarithmic velocity distribution is assumed (Bouwmeester (1972) [53]):

The radial slope  $I_R$ :

$$I_R = \alpha \frac{\bar{v}^2}{gR} \quad (3.7.5)$$

with

$$\alpha = 1 + 3 \left( \frac{\sqrt{g}}{KC} \right)^2 - 2 \left( \frac{\sqrt{g}}{KC} \right)^3$$

This function is given in Figure 3.7.9.

II, 3.7

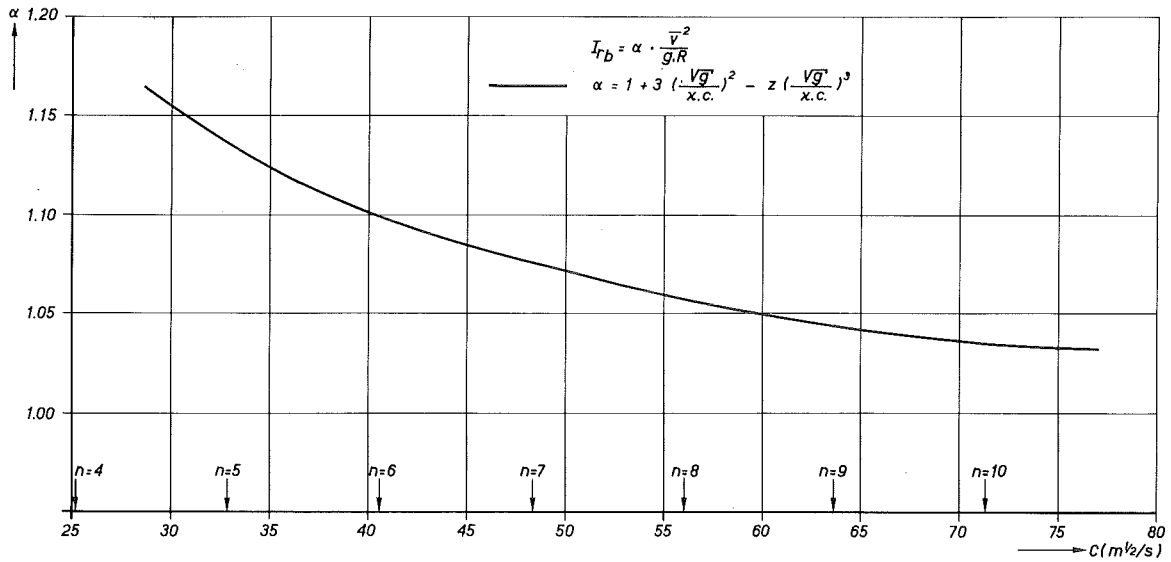


Figure 3.7.9 Radial Bed Slope  $I_{R_b}$

The radial shear stress component along the bed

$$\tau_{R_b} = \gamma_1 \frac{\rho h \bar{v}^2}{R} \tag{3.7.6}$$

with  $\gamma_1 = 2 \left(\frac{\sqrt{g}}{Kc}\right)^2 - 2 \left(\frac{\sqrt{g}}{Kc}\right)^3$  represented in figure 3.7.10

With these two functions it is possible to obtain information on the radial slope of the river-bed.

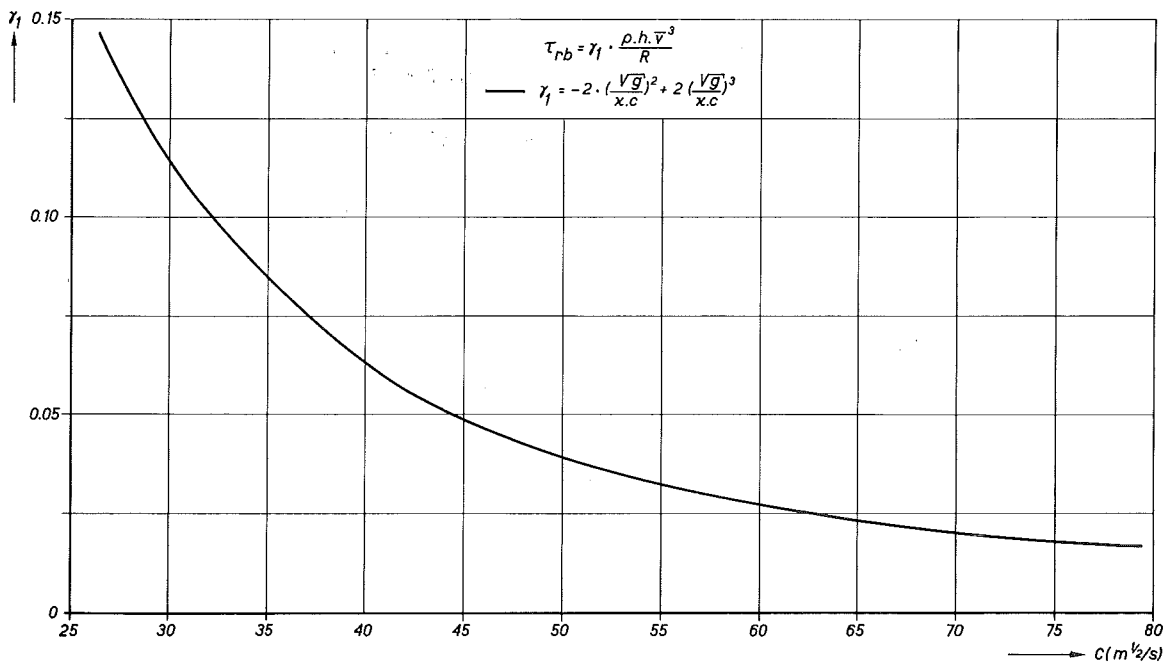


Figure 3.7.10 Radial Shear Stress  $\tau_{R_b}$

It may be assumed that for equilibrium  $G \sin \alpha = F_R$  has to hold, where  $F_R$  is the radial shear-force component. For the forces on a grain this may be expressed as

$$\beta \tau_{R_b} \cdot D^2 = (\rho_s - \rho_w) g D^3 \sin \alpha \quad (3.7.7)$$

in which  $\beta$  depends on the form of the grains and their relative positions. Introducing Equation (3.7.6), (3.7.7) may also be written as:

$$\sin \alpha = \frac{dh}{dR} = \frac{\beta \gamma_1 \cdot h \bar{v}^2}{\Delta g D R} \quad (3.7.8)$$

where  $\gamma_1$  can be found from Figure 3.7.10.

Introducing  $v^2 = C^2 h I$ , Eq.(3.7.8) becomes

$$\sin \alpha = \frac{dh}{dR} = \frac{\beta \cdot \gamma_1 \cdot C^2}{g} \frac{h^2 I}{\Delta D R} = A \frac{h^2 I}{\Delta D R} \quad (3.7.9)$$

Van Bendegom (1947) [54], who arrives at a similar equation, finds the value of

$$A \approx 10 \text{ à } 15.$$

If for  $\beta$  unity is assumed, the value of  $A$  would be about 10 for a wide range of  $C$  and  $\gamma_1$  (see Figure 3.7.11). A value  $A = 10$  may also be used for the Rfo Magdalena (see Figure 3.7.12).

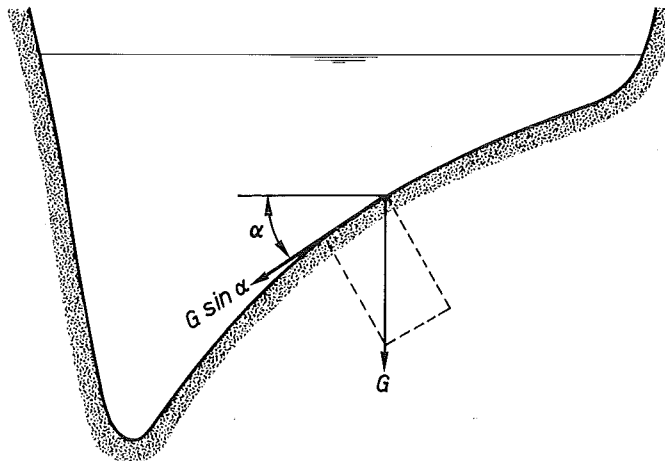


Figure 3.7.11 Forces on Radial Bed Slope

Van Bendegom [54] arrives at a simplified form of Eq.(3.7.9):

$$\frac{dh}{dR} = A \frac{I_o \cdot R_o}{\Delta D} \cdot \frac{h^2}{R^2} \text{ with } A \approx 10$$

by assuming  $I = I_o \cdot \frac{R_o}{R}$  with  $I_o$  and  $R_o$  being the values for  $I$  and  $R$  in the outer bend.

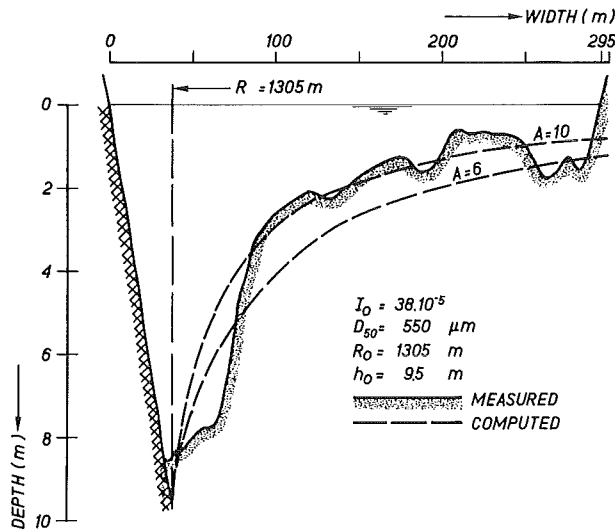


Figure 3.7.12 Computed and Measured Bend Cross-section (Downstream Pto. Triunfo)

The assumptions made at the beginning of this Paragraph should be kept in mind, but it is reasonable to expect similar results without the omissions mentioned; however, with somewhat different values of  $A$ . Combining the theory given in Para. 3.6 and in this Paragraph, it should be possible to compute the bed Configuration (semi three-dimensional). Up till now, however, this has not yet been done but research on this subject is still being carried out.

### 3.8. APPLICATIONS

#### 3.8.1. Introduction

The theory and methods given in the foregoing chapters of Part II make it possible to carry out computations and thus give reasonably reliable predictions about the effects of permanent and temporary river-works. In Part III these methods will be applied mainly to predict the effects of river-works carried out near the Río Regla Confluence and to obtain values for the related scour at a number of crossings. In this Chapter the problems encountered and the interpretation of the results will be given particular attention.

#### 3.8.2. Test case of the Río Sogamoso Confluence and the Río Regla Confluence

As no measurements can generally be carried out at a water-level near L.R.L. and during the time of the study, all measurements were made at higher levels, the depth that would be available at a crossing at a water-level near L.R.L. can only be found by means of computations. In Figure 3.8.1 some measurements are given of the available depth at a crossing at higher levels and, as can be seen from these measurements, an estimate of the available depth when the water-level drops to L.R.L. is impossible (in the same figure the result of a computation is plotted).

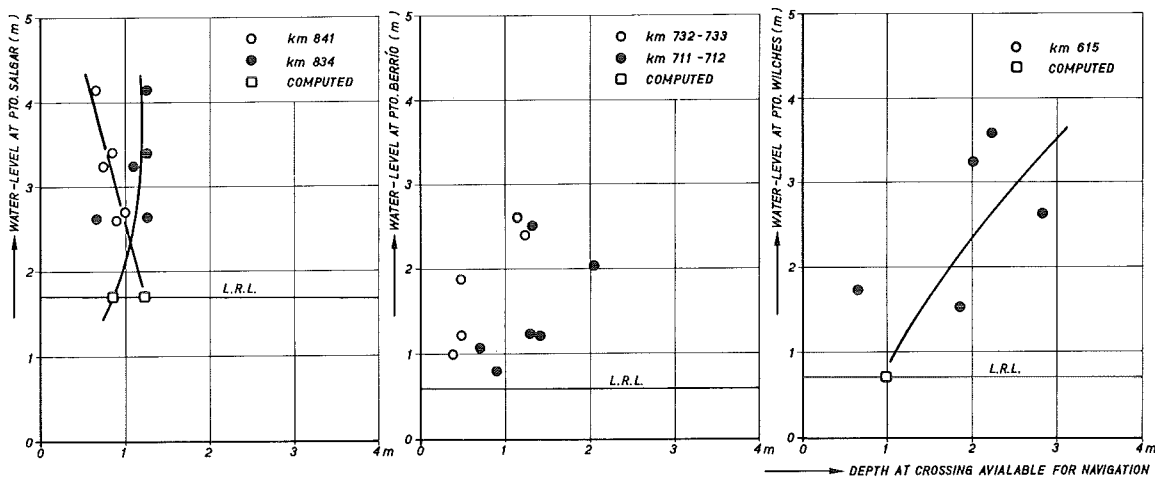


Figure 3.8.1 Depth at Crossings Available for Navigation

To be reasonably sure that the methods given are applicable, the computations carried out for the Rfo Sogamoso Confluence were compared with results from soundings. It must be realised that the results depend very much on the schematization of the plan-form that was used, and it must be expected that when a large number of crossings are computed, a number of these computations may give results that differ from the value in nature. However, the general impression obtained from a number of crossings is correct.

Schematization of the planform

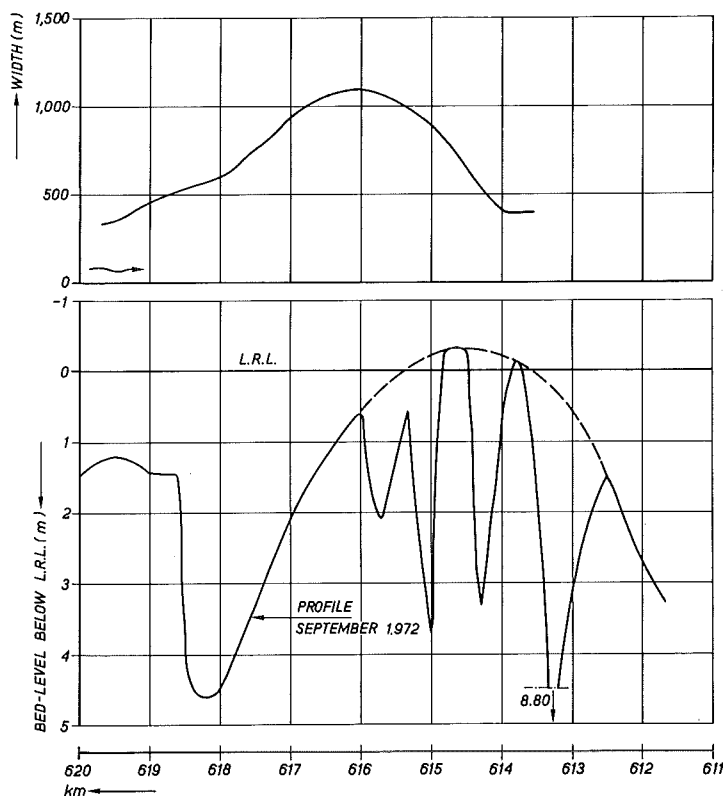


Figure 3.8.2 Schematization of Crossing Upstream of Rfo Sogamoso Confluence

## II, 3.8

The schematization used for the crossing near the Río Sogamoso Confluence is given in Figure 3.8.2. It was based on aerial photographs, soundings and flow-lines as given in Part III, Chapter 3.5.

### Rating\_curve

For the computation the rating curve of Pto. Wilches was used rather than that of Barrancabermeja, although the crossing is situated upstream of the Confluence. It was thought that in view of the short distance to the Confluence and the backwater effects this would give a more realistic water-level. In reality, part of the discharge may come from the Sogamoso River, but in the computation this discharge was neglected because its influence is not very large, as can be concluded from Figure 3.8.3 giving relation-curves for different discharges of the Sogamoso River.

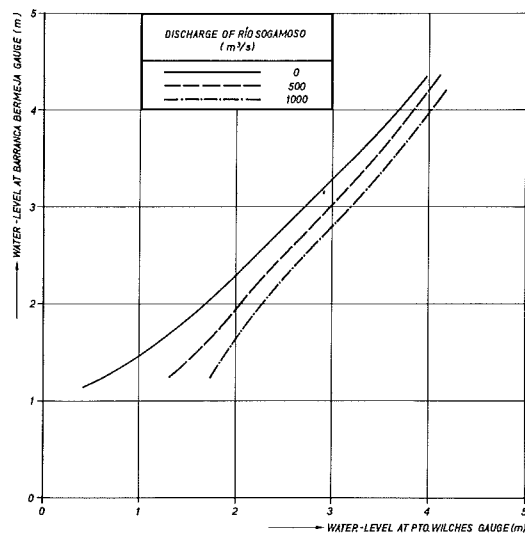


Figure 3.8.3 Relation-curves Barrancabermeja - Pto Wilches

### Regimes

To compute the available depth at L.R.L. a standard regime was introduced as indicated in Figure 3.8.4 (Regime 5), which has been used for all the crossings.

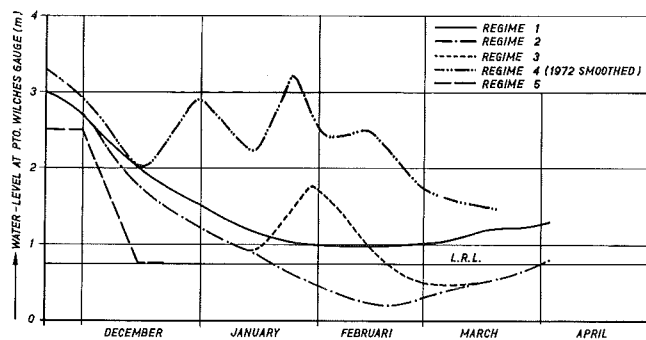
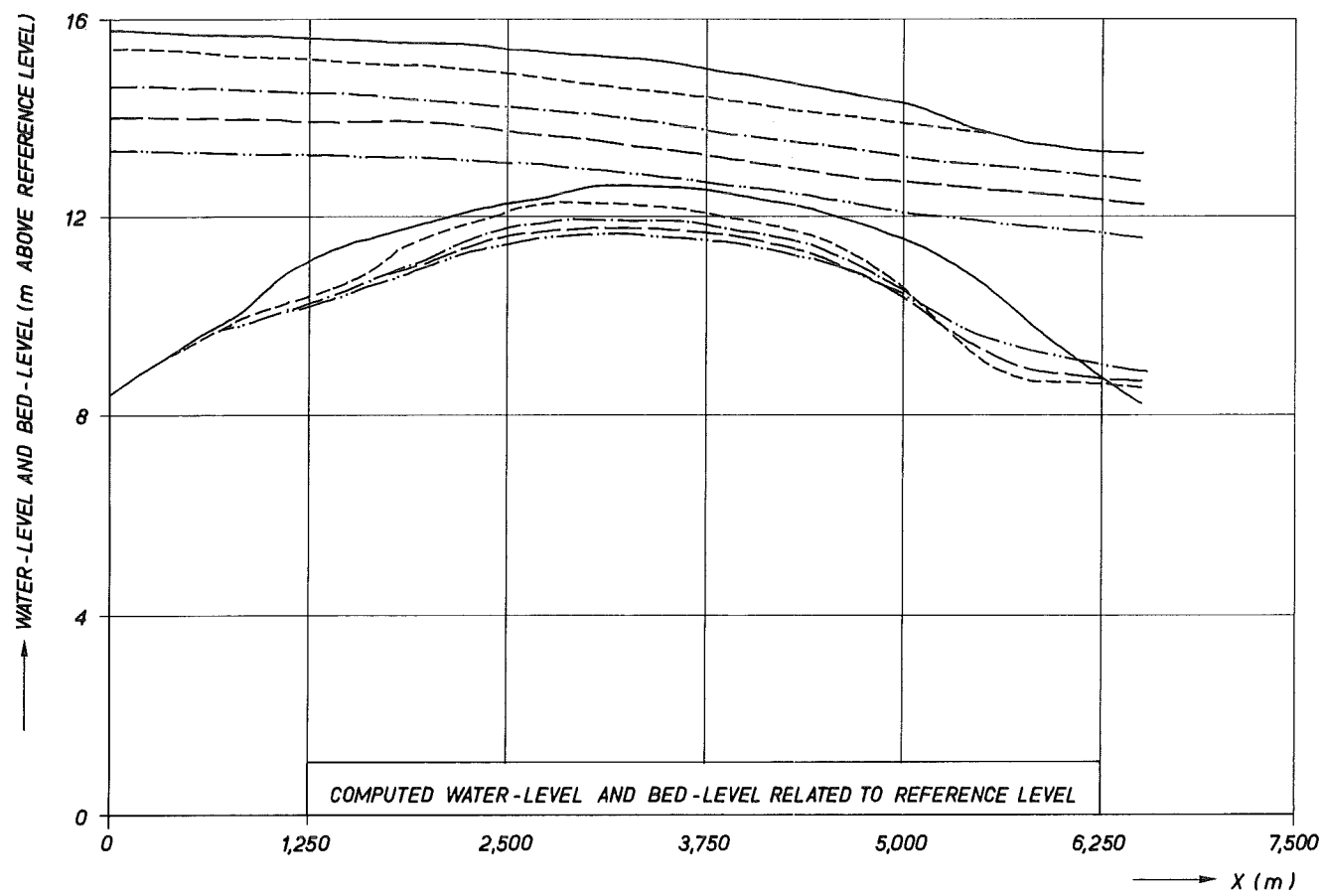
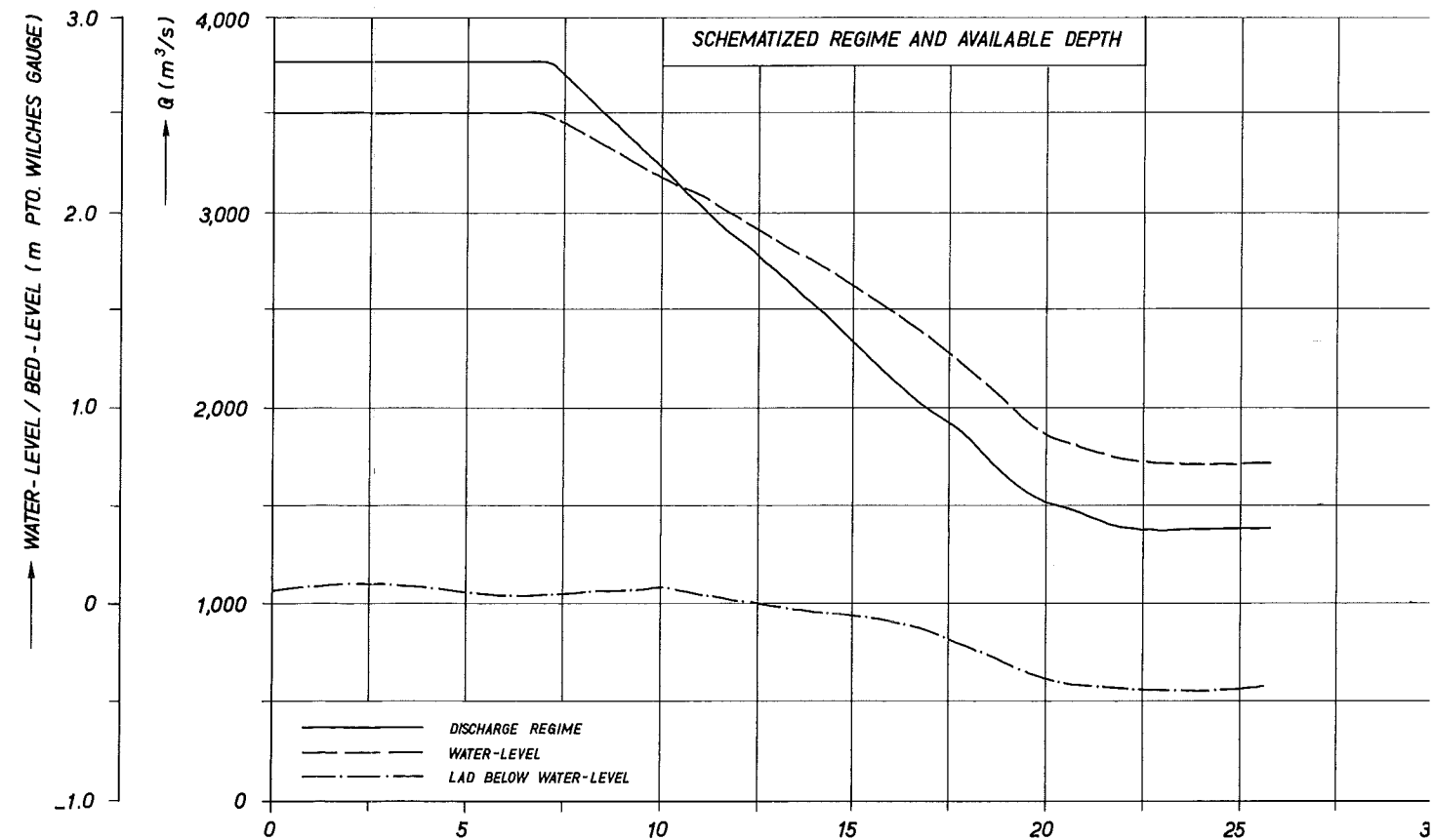
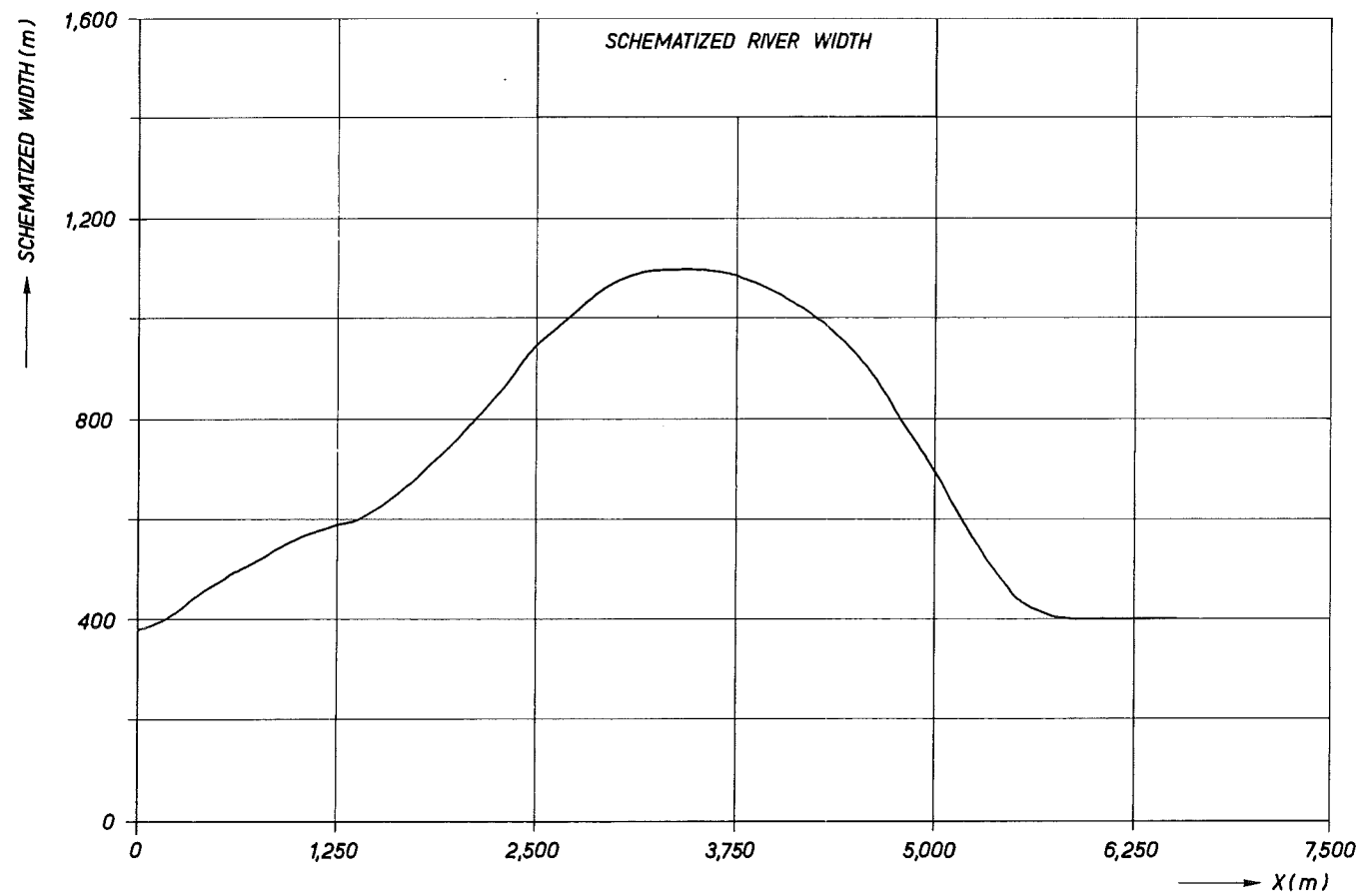
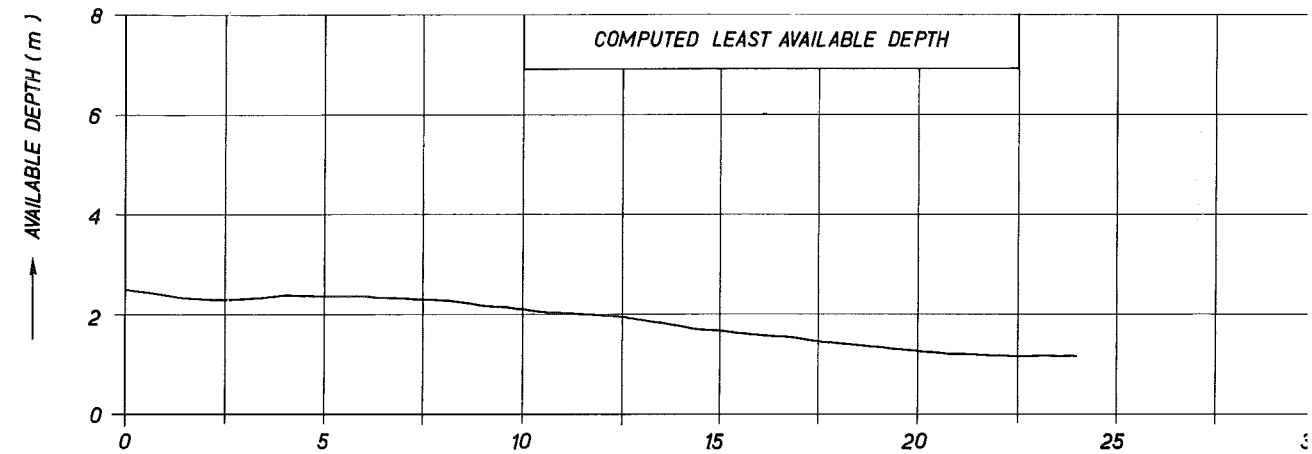


Figure 3.8.4 Regimes used for the Computations



WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—————	0	0	3,761
-----	6	0.5	3,761
-----	12	0.5	2,831
-----	18	0.5	2,111
-----	22	0.5	1,344

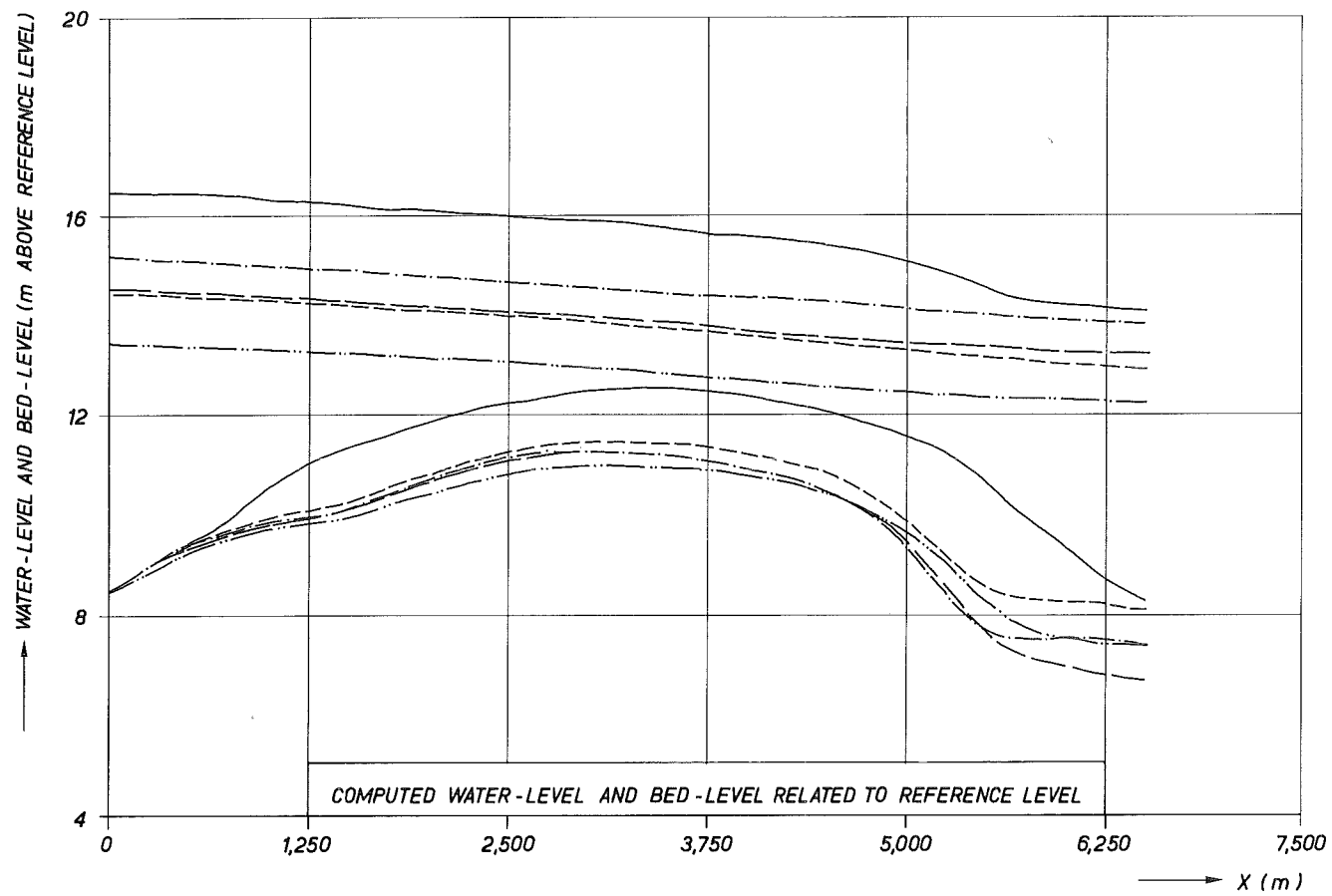


RÍO MAGDALENA km 615 (UPSTREAM RÍO SOGAMOSO), REGIME 5,  $C = 48 \text{ m}^{1/2}/\text{s}$  (CO)

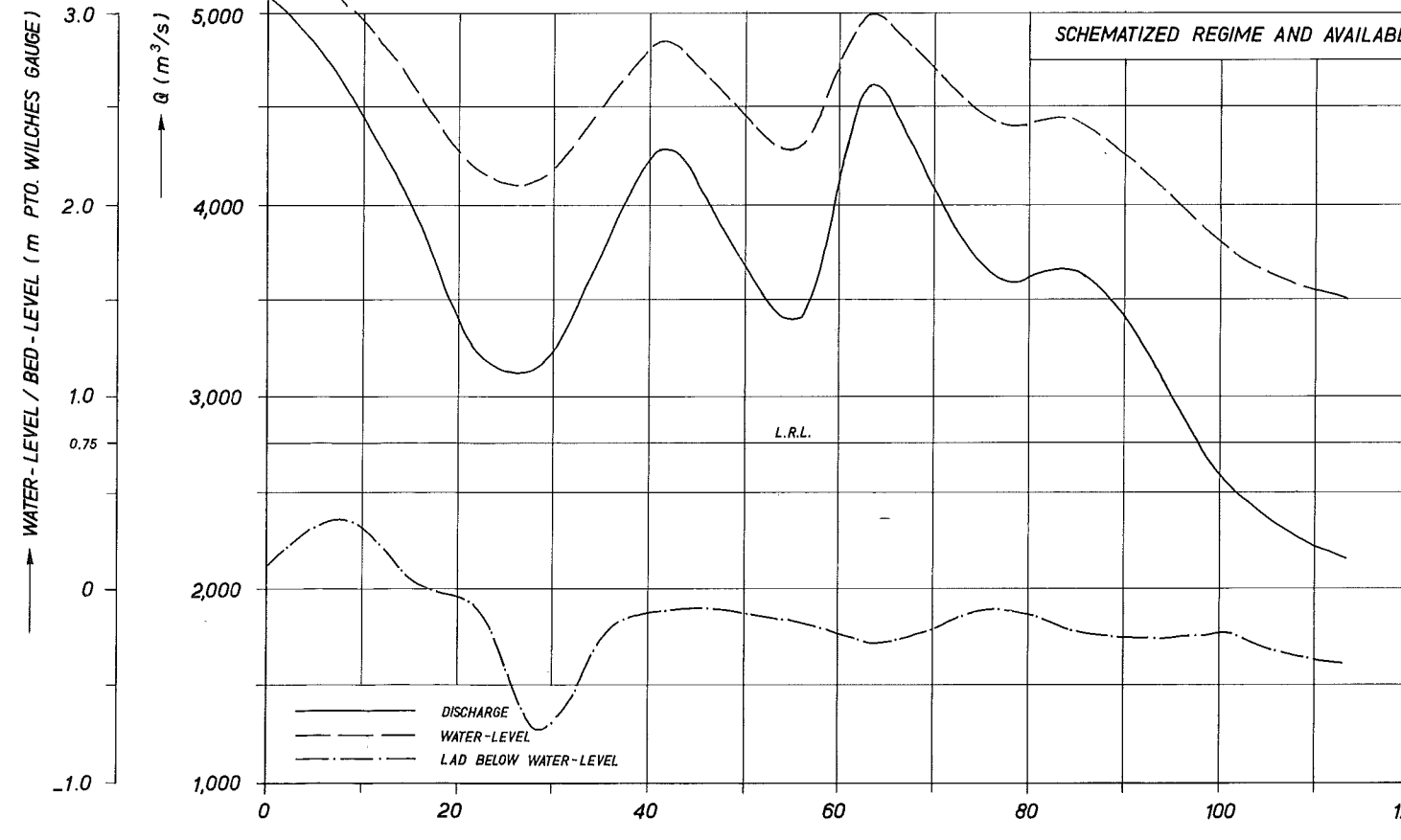
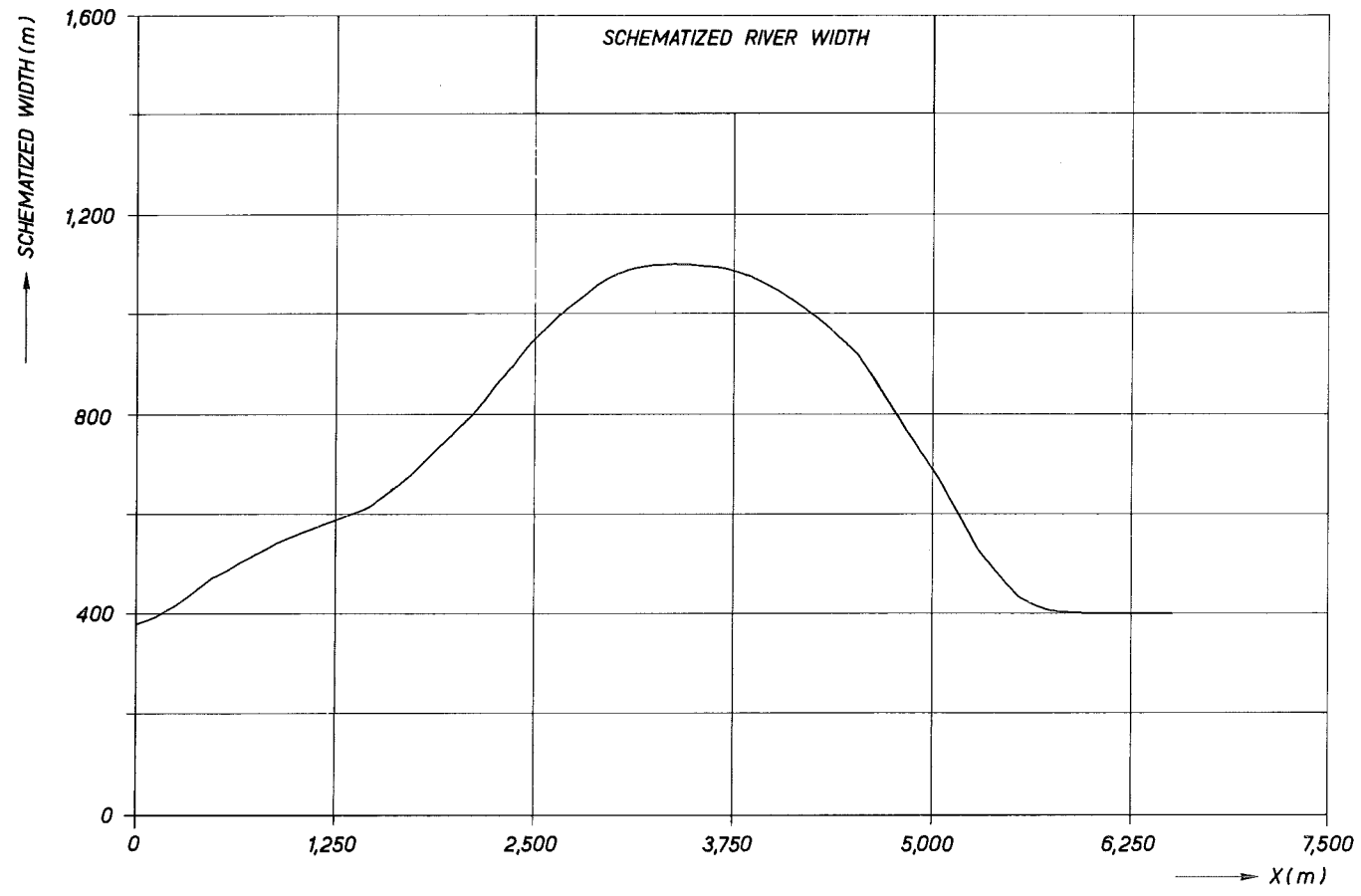
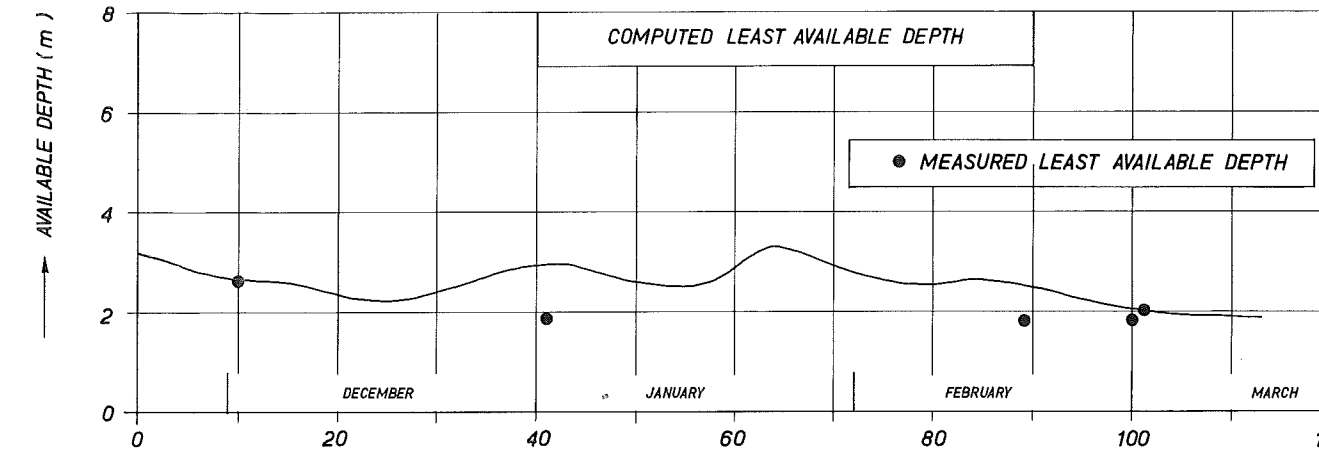
RESULTS OF MORPHOLOGICAL COMPUTATIONS

F.

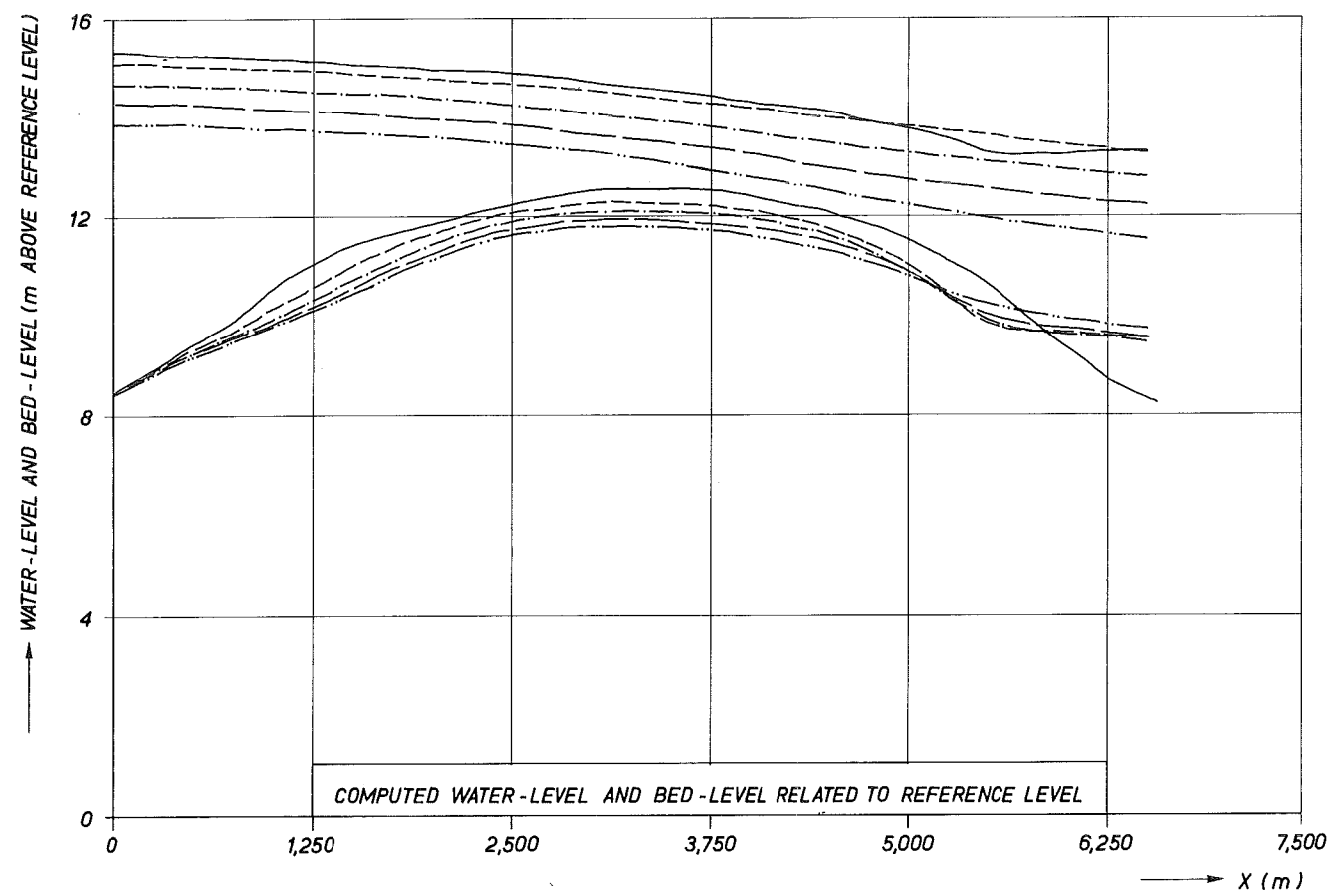




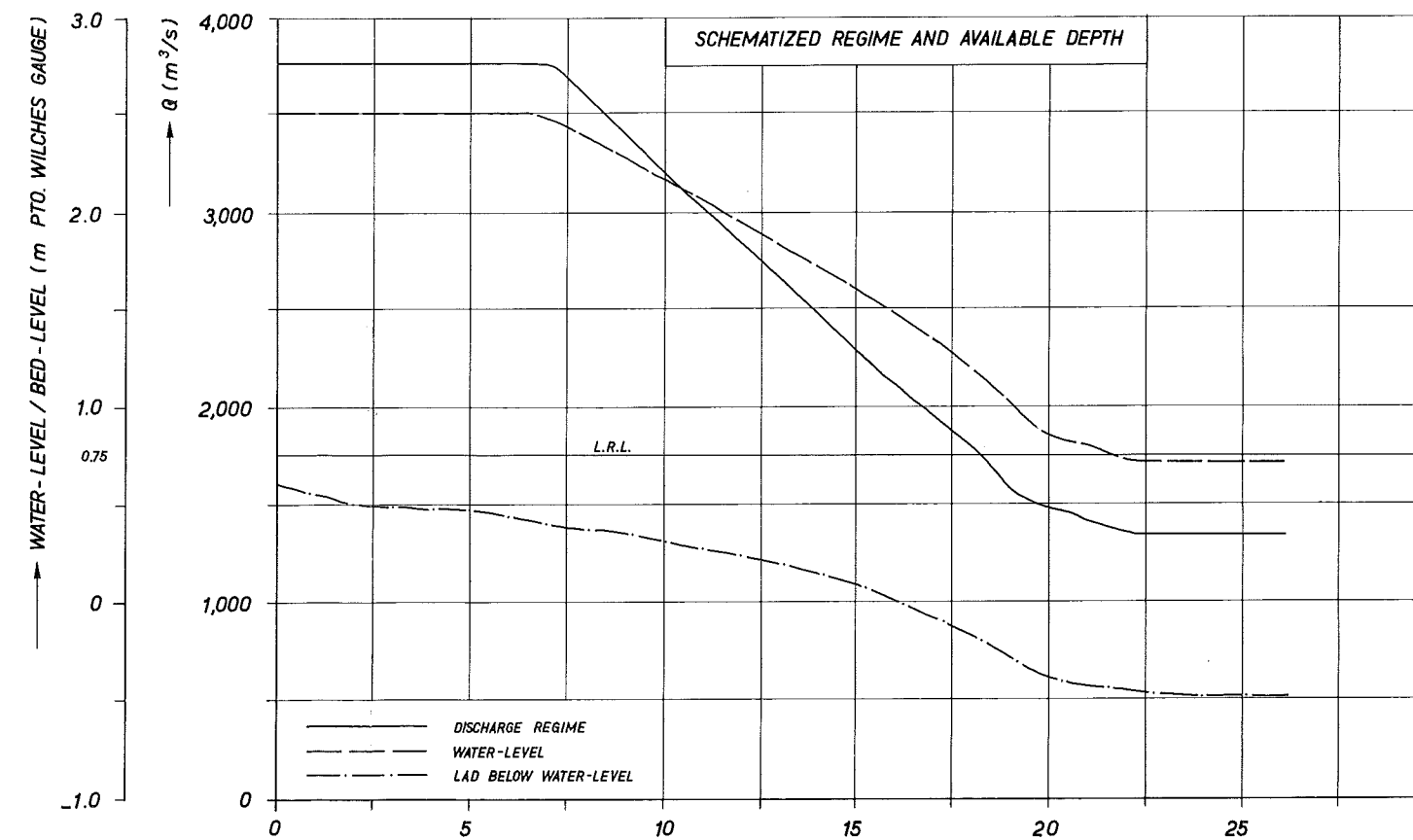
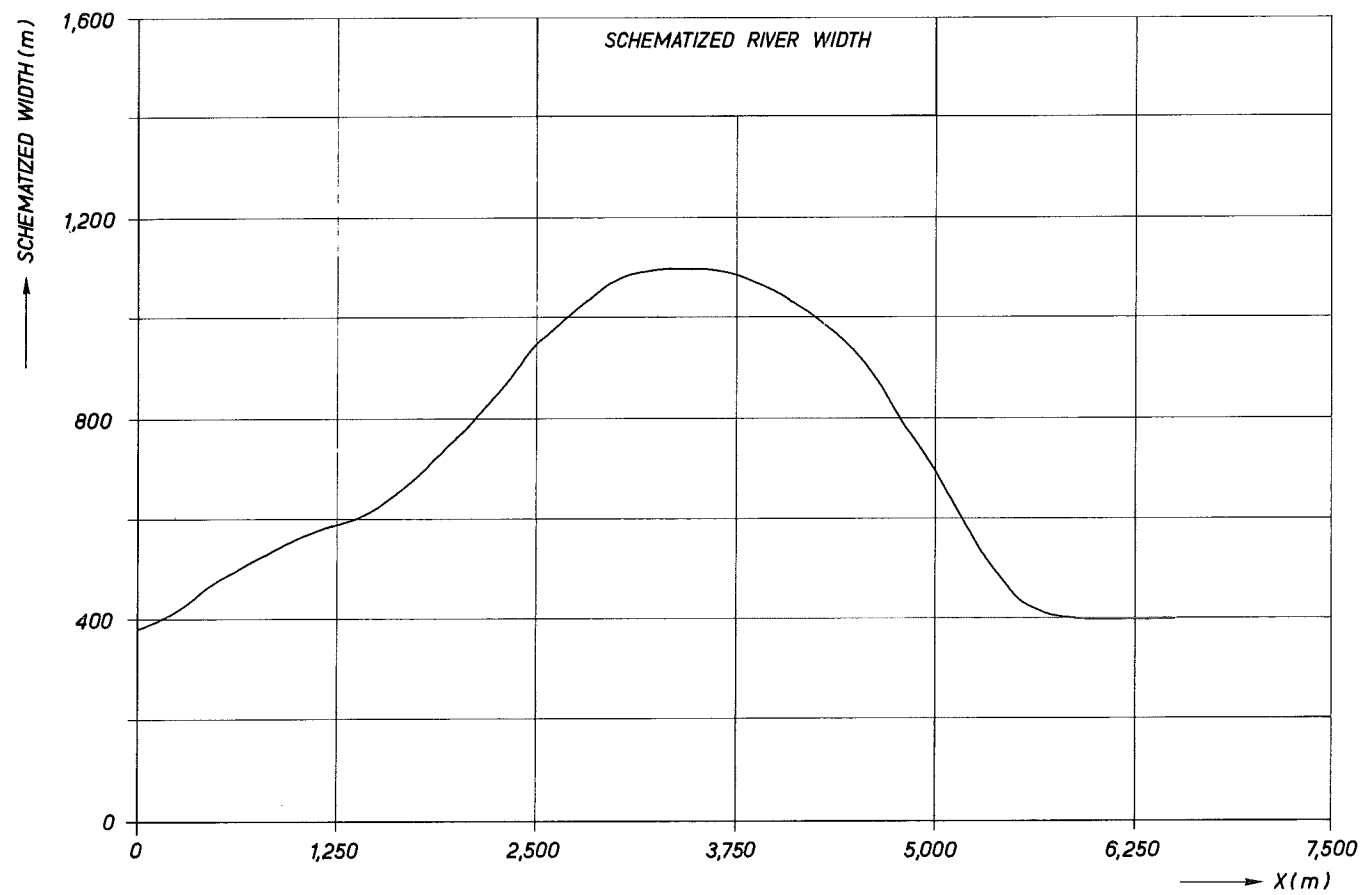
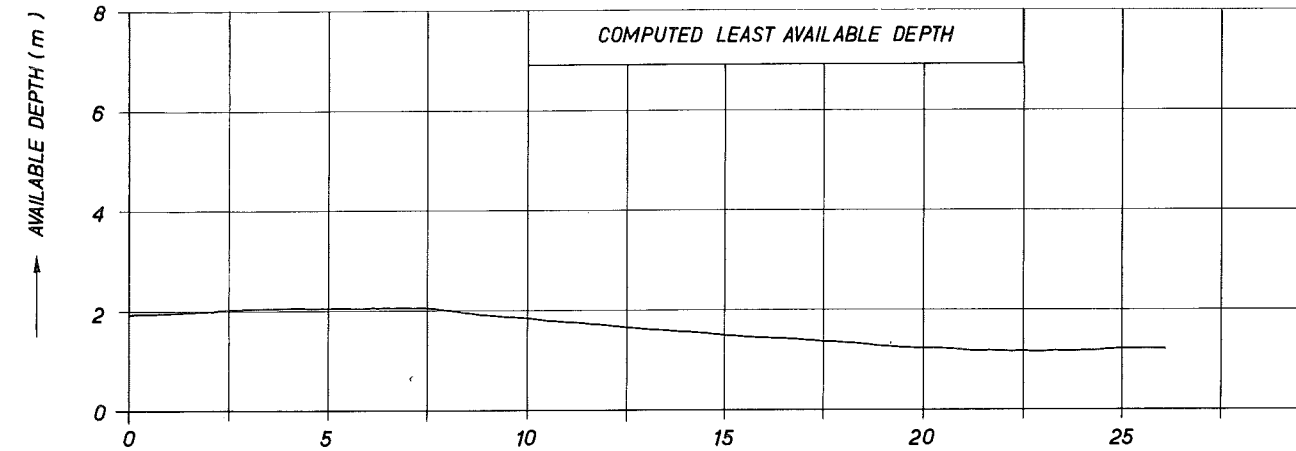
WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—————	0	0	5,083
-----	28	13	1,158
-----	63	14	4,612
-----	84	2	3,667
-----	113	1	2,167



RÍO MAGDALENA km 615 (UPSTREAM RÍO SOGAMOSO), REGIME 4, C = 48 m<sup>1/2</sup>/s (CON  
 RESULTS OF MORPHOLOGICAL COMPUTATIONS



WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—————	0	0	3,761
-----	6	1.5	3,761
-----	11	6	2,973
-----	16	2.5	2,097
-----	22	6.5	1,343



This standard regime, a very pronounced fall in water-level to L.R.L., is thought to give the most unfavourable conditions to be expected. The results of the computation are given in Figure 3.8.5.

The following important conclusions may be drawn:

- The river bed reacts very rapidly to the change in water-level with the result that during the rapid fall the available depth is always more than the depth available at the moment L.R.L. is reached. This means that for a dredging programme only the available depth in relation to L.R.L. is important.
- After L.R.L. is reached a slight continuation of the scour results in a slight increase in depth, which indicates that the conditions immediately after the fall are the worst.
- The retarded scour during the fall is 0.98 m, which means that a fall in water-level of about 2 m results in a reduction in depth of only 1 m.
- As the plan-form schematizations were carried out for low levels (assuming no flow over sand banks, etc.) the bed levels computed for low water are more in agreement with the actual situation than those found for higher levels (this is also of importance in respect of Regime 4 which is dealt with later).

To study the influence of a more irregular regime, computations were also carried out for the other regimes, as given in Figure 3.8.4. Here only the results of Regime 4 are given (Figure 3.8.6), which is a smoothed course of the water-levels of the season 1972-1973 and the results can therefore be compared to those of the prototype.

Also here it can be seen that the bed reacts immediately and depths are always better than those at L.R.L. Good agreement exists with measuring-points (from soundings) except for the point measured at 40 days. Generally the results of the computation can, therefore, be used with confidence. In Part III, Figures 3.2.15 and 3.2.17 the results of the computations at other crossings (km 841 and km 833) are given, where also velocities have been measured. As can be seen, the measured velocities correspond well to those computed.

#### Channel\_roughness

For the computations of the crossings the roughness value was obtained according to the method given in Chapter 3.4. For the crossing near the Rfo Sogamoso Confluence the value found from measurements has been used to prevent an extra cause for uncertainties. To be sure that this difference in method did not cause discrepancies, a number of computations for the Sogamoso were done in both ways. Although the available depths at the beginning of the degradation process differ from those found from a computation with constant roughness (compare Figures 3.8.7 and 3.8.5), the minimum depths found during the later part of the process are practically equal.

#### Rfo\_Regla\_Confluence

In Part III also computations have been carried out to predict the influence of permanent river-works. The plan-form schematization for that case is simpler than for a crossing because the plan-form is more defined. Also for the computations of the river im-

provements near the Río Regla Confluence, the schematization has been based more on the low water situation than the high water situation. As channel roughness, the measured value has been used. The results of these computations are further given in Part III Chapter 3.4.

Test dredging

In the computations of the crossings as used in Part III the depth available at L.R.L. was determined with a one-dimensional schematization. In reality, the river bed contains small channels which generally are not sufficient for navigation, and so the computations generally give values for the available depth which have a good agreement with the prototype. Sometimes, however, several of these small channels join to form one larger channel which is sufficiently wide for navigation, producing navigation conditions which may be better than would follow from the computations. When dredging of crossings is carried out, such a concentration of small channels is made artificially. It will be clear that if the depth dredged would be introduced in the presently used computations, the bed would immediately sediment to the equilibrium level, which is higher (see Figure 3.8.8). This does not mean that a dredged channel deeper than this equilibrium depth is not stable, as in reality the phenomenon is three-dimensional and concentration of a number of small channels into one somewhat larger channel is possible.

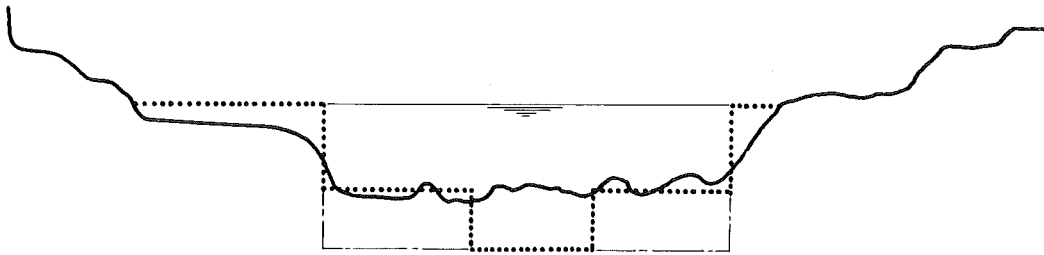


Figure 3.8.8 Dredge Cut in Crossing

To check the stability of such a channel, computations could be made for a composite channel where no exchange of water and sediment takes place perpendicular to the length axis (from I to II in Figure 3.8.9).

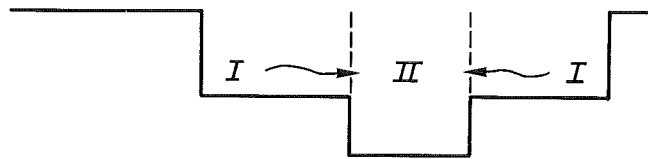


Figure 3.8.9 Schematization of the Crossing with Dredge Cut for Computation

However, such computations have not been carried out because no way of checking the results is available except by means of dredging such a channel in the prototype and studying the developments of the channel. This is the reason test dredging is required, and if this proves that such computations give good results for the Río Magdalena, it would be possible to predict the stability of a dredge cut by means of computations.

**PART III**

**IMPROVEMENT FOR NAVIGATION OF THE  
RÍO MAGDALENA AND THE CANAL DEL DIQUE**

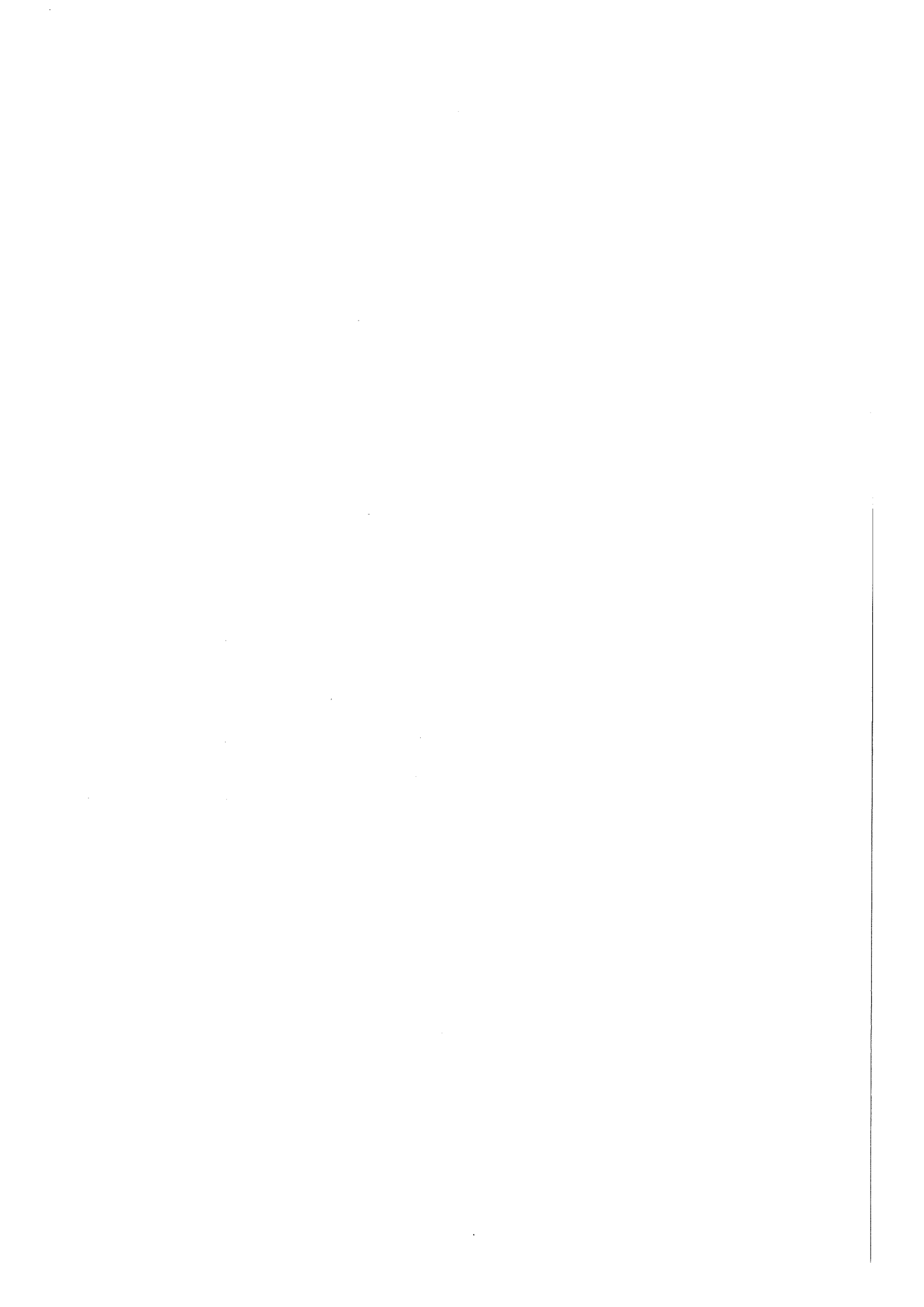


## CONTENTS

PART III IMPROVEMENT FOR NAVIGATION OF THE RÍO MAGDALENA AND THE CANAL DEL DIQUE		Page
Chapter 1	GENERAL	157
Chapter 2	RIVER IMPROVEMENT	158
2.1.	Introduction	158
2.2.	Aids to navigation	158
2.2.1.	Measures taken by river operators	158
2.2.2.	Channel patrol service	160
2.3.	Temporary river-works	168
2.3.1.	Introduction	168
2.3.2.	Temporary constructions	169
2.3.3.	Dredging equipment (general)	169
2.3.4.	Dredging equipment at present in use on the Río Magdalena and the Canal del Dique	180
2.3.5.	New equipment for the Río Magdalena	182
2.3.6.	Organization of dredging	187
2.4.	Permanent river improvement	187
2.4.1.	Introduction	187
2.4.2.	Regulation of the plan-form	190
2.4.3.	Normalization of the river width	199
2.4.4.	Regulation of the river-bed	202
2.5.	Constructions	205
2.5.1.	Introduction	205
2.5.2.	Design criteria of a protection	206
2.5.3.	Closure of secondary branches	216
2.5.4.	Examples of constructions	216
Chapter 3	RÍO MAGDALENA	224
3.1.	Introduction	224
3.2.	La Dorada-Puerto Inmarco (km 884-773)	225
3.2.1.	General description and design criteria	225
3.2.2.	Temporary improvement by means of dredging	233
3.2.3.	Improvement of La Dorada-Pto.Salgar	234
3.2.4.	Confluences of the Río Magdalena and the Ríos Negro and La Miel	253
3.2.5.	River-crossing km 833	256

	Page
3.2.6. Puerto Triunfo	257
3.2.7. Puerto Boyacá	260
3.2.8. River-crossings near km 780	263
3.2.9. Confluence of the Río Magdalena and the Río Nare	264
3.3. Puerto Inmarco-Puerto Berrío (km 773-730)	266
3.3.1. General description and design criteria	266
3.3.2. Temporary improvement by means of dredging	270
3.3.3. Río Magdalena upstream Pto. Berrío (km 735-730.5)	271
3.4. Puerto Berrío-Barrancabermeja (km 730-630)	274
3.4.1. General description and design criteria	274
3.4.2. Temporary improvement by means of dredging	278
3.4.3. The section between Pto. Berrío and the Río Regla Confluence (km 730-710)	278
3.4.4. River improvement near the Río Regla Confluence (km 711-706)	279
3.4.5. The Río Magdalena between the Río Carare Confluence and Chucurí (km 675-659)	285
3.5. Barrancabermeja-Gamarra (km 630-475)	288
3.5.1. General description and design criteria	288
3.5.2. Temporary improvement by means of dredging	294
3.5.3. The access to the Barrancabermeja Port	294
3.5.4. The "Hidro Electrica Lebrija" (km 626) and navigation between km 624 and km 621	295
3.5.5. The Río Sogamoso Confluence	296
3.5.6. The access to Pto. Wilckes	302
Chapter 4 CANAL DEL DIQUE	303
4.1. General	303
4.2. The canal	305
4.2.1. Present dimensions of the canal	305
4.2.2. Required dimensions (MITCH design-section)	306
4.2.3. Bends and bend-radius	310
4.3. Calamar (Canal del Dique bifurcation)	311
4.3.1. Introduction	311
4.3.2. Sedimentation in the present situation	311
4.3.3. Comparison of different solutions	313
4.4. Lower Canal del Dique (Pasacaballos)	318
4.4.1. Introduction	318
4.4.2. Division of discharge and sediment-load along the canal	318
4.4.3. Siltation near Pasacaballos	324
4.4.4. Dredging of the Lower Canal del Dique	331

	Page
Chapter 5 DREDGING PROGRAMME AND PHASING OF RIVER-WORKS	333
5.1. Introduction	333
5.2. Dredging programme	333
5.2.1. The organization	333
5.2.2. Amounts to be dredged and required capacity of the dredge fleet	334
5.3. Phasing of the river-works	336



## Chapter 1

### GENERAL

Whereas Part II outlined the methods which can be used to predict the effect of river-works as well as the required data as far as they were collected or available, Part III contains the applications to the problems encountered along the Río Magdalena.

The Río Magdalena has been divided into four main sections:

- La Dorada-Pto. Inmarco,
- Pto. Inmarco-Pto. Berrío,
- Pto. Berrío-Barrancabermeja, and
- Barrancabermeja-Gamarra.

In Chapter 3 some general information about these sections is given as well as the amounts required to be dredged annually to guarantee a depth of 7'6" downstream of Pto. Berrío and 4'6" upstream of Pto. Berrío. In this Chapter the places along the sections giving problems are systematically dealt with, while in Chapter 4 a similar approach is used for the Canal del Dique.

Before being able to discuss the problems in Chapters 3 and 4, it seemed necessary to give in Chapter 2 a kind of inventory of the types of solutions available, solutions which were of a more general character such as aids to navigation, etc., as well as a number of specific river-works divided into temporary river-works (dredging) and permanent river-works and their constructions. The paragraphs about temporary river-works also contain the description of the type of dredger thought most appropriate for the Río Magdalena.

The dredging programme could not be given in that Chapter as it could only be determined after the amounts to be dredged were gathered from Chapters 3 and 4. The dredging programme is, therefore, given in Chapter 5 as part of the phasing of the river-works to be carried out along the Río Magdalena.

## Chapter 2

### RIVER IMPROVEMENT

#### 2.1. INTRODUCTION

In this Chapter the ways to attain a river improvement are dealt with. The most simple means for improvement are those measures taken by the river operators to adapt the navigation as far as possible to the prevailing river conditions, or those taken by the agency responsible for the river conservancy to define the navigable channel. Such measures, considered as aids to navigation, are treated in Para. 2.2.

Para. 2.3 contains the means for the temporary improvement of bad river sections (crossings) which need to be repeated annually. Basically, this paragraph deals with dredging and, apart from the general dredging techniques, a list of the presently available dredging equipment is given and a description of the type of dredger thought most appropriate for the Rfo Magdalena.

For the best ways to obtain a permanent improvement of a river section, reference is made to Para. 2.4. The types of construction required for such river-works are treated in Para. 2.5, in which some remarks are also made regarding the types of construction used for the river-works recently undertaken along the Rfo Magdalena.

#### 2.2. AIDS TO NAVIGATION

Aids to navigation can be divided into those measures taken by river operators and those to be taken by the agency responsible for the river conservancy (MOP, ADENAVI, Unidad de Estudios fluviales). These last measures can be gathered under the name channel patrols.

##### 2.2.1. Measures taken by river operators

The aim of the river operators is to adapt the navigation as far as possible to the prevailing river conditions by using the correct type of ships and equipment, as well as competent crews. The impression exists that river operators on the Rfo Magdalena have achieved a very high efficiency resulting in extremely low ton/km prices, and therefore that not much can be improved in this field. Of course, the present fleet is based on present conditions, which means mainly that 80% of the transport takes place downstream of Barrancabermeja. If any improvement in efficiency could be achieved, it would be in the La Dorada - Barrancabermeja traffic. As this, however, will be dealt with in a special study (The Magdalena River Area Transport Study), only a few remarks will be made here.

The present fleet has a capacity of about 169,000 tons, of which more than 30% consists of units larger than 3,500 tons, while some units are around 6,000 tons. The barges used generally have relatively large widths and shallow draughts, which is logical under present river conditions. Compare, for instance, the standard European barges with those of the Rfo Magdalena (Figure 2.2.1).

Barge Dimensions	Río Magdalena	Rhine
Width	13 m	9 - 11½ m
Draught	1.5 m (5')	2.5 m (8'3")

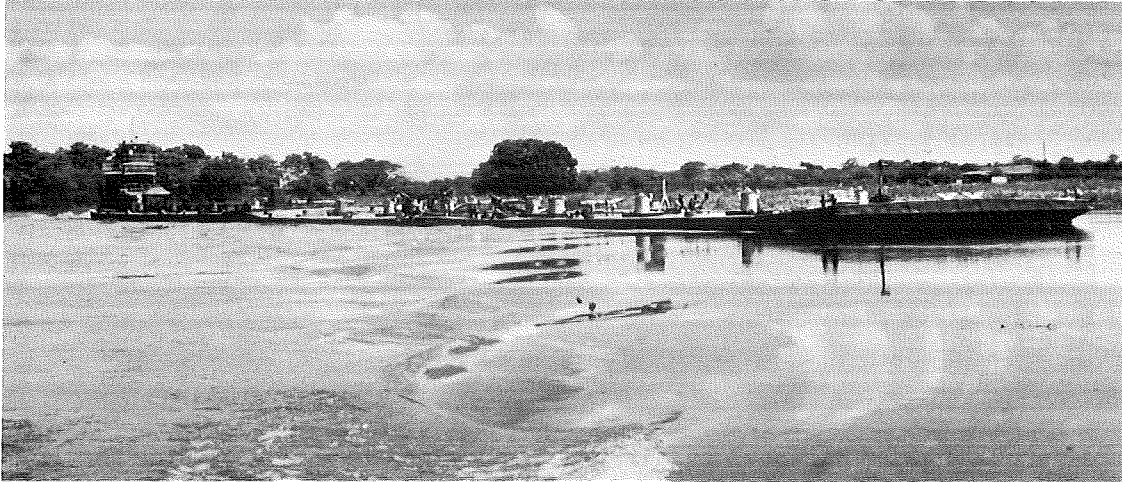


Figure 2.2.1 Barge Train on the Río Magdalena

In view of the high velocities which occur locally upstream of Barrancabermeja, barge trains sometimes have to be broken up to pass such areas, while units sometimes have to wait for a lower water-level with corresponding lower velocities. A system of shuttling may improve navigation upstream of Barrancabermeja. A small but relatively powerful tug could push 1 or 2 barges upstream of Barrancabermeja while in Barrancabermeja 6 or 8 of these barges could be assembled and pushed by one of the larger tugs.

At present no use is made of containers or Lash barges, and it should be studied to ascertain whether the use of containers would have any advantages for the transport along the Río Magdalena.

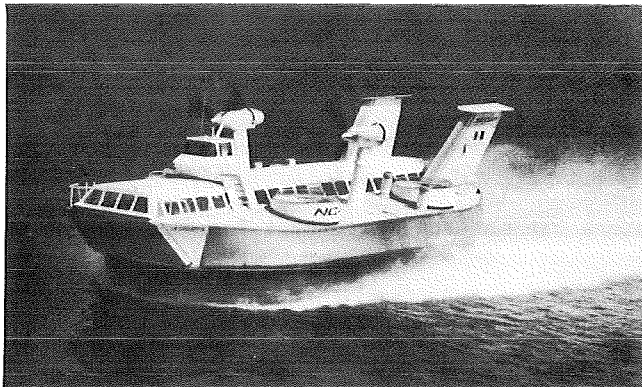


Figure 2.2.2 Hovercraft

### III, 2.2

Nowadays there is no passenger traffic here as in former times, and in this connection the possibility of using hovercraft (carrying passengers from La Dorada to Barranquilla within 8 hours) should be investigated. Although transport by hovercraft is admittedly expensive, it is cheaper than by plane, and no airfields are required. Moreover, many villages along the river could be incorporated in the service.

The so-called aerofoil craft, however, seems less suitable in this area because of the great amount of floating trees and debris.

#### 2.2.2. Channel patrol service

A channel patrol service is required for any river where an appreciable amount of navigation is available; this applies equally to regulated as to non-regulated rivers. In those cases where recurrent dredging of crossings is carried out, for optimal use of the relatively small dredge cut it should be accurately indicated, while the development of the channel (location and depth) should be closely followed and the gathered information transmitted to river operators.

In short, the work of a channel patrol service may be divided into two parts:

- Gathering of channel information and, as far as necessary, transmitting this information to river operators; and
- finding and marking the best channels for navigation and clearing these channels of obstacles.

At present ADENAVI maintains a channel patrol service between La Dorada and Gamarra, but the accent lies mainly on the second type of activities: marking and clearing of the channels.

#### Channel information

For the optimal use of the river, the operators should always have access to up-to-date information on the available depth and the state and marking of the channels in a certain river stretch. In fact, if possible, not only should up-to-date information be available but also the prediction of the available depths some days in advance. This enables the river operator to load the barges exactly to the level permitted by the river.

#### River maps for navigation

River maps for navigation purposes should not be too detailed, as these details usually change so rapidly that the maps quickly become outdated. A better system is to use a kind of basic map which only shows slowly changing features, conspicuous points, etc., and then river operators can mark on these maps the information given by a channel patrol service.

A scale of 1:20,000 seems most suitable for the Río Magdalena and such maps have been prepared from the aerial photographs made in 1972 (Part II, Figures 3.3.17 to 3.3.40 inclusive). About every 5 to 10 years these maps should be renewed according to new photographs which, if taken for this purpose only, could be on the scale 1:60,000 (As weather

conditions often do not permit high altitude photography in the low water season, a smaller scale may be required). A kilometrage (each km) is indicated on the maps. At least every 5 km kilometer boards should be installed on the (high) banks and indicated on the maps. The location of the kilometer indication should remain the same in relation to the high banks even in those cases where the actual distances alter, because of changes in the talweg.

#### Available depths

It should be possible to indicate the available depth for each river section daily. This does not mean that a complete length sounding should be carried out every day; a rough check of the total section two or three times a week and an accurate measurement at the bottlenecks also two or three times weekly will suffice, provided sufficient information is available on the properties of the crossings and the change in water-levels. About once every month a proper route mapping should be carried out (see Part IV).

As already mentioned, optimal use of a river not only requires up-to-date information, but in fact also needs the prediction of available depths and, consequently, also prediction of water-levels some days in advance. For the Río Magdalena, at present, such predictions are not possible, so further study on this subject is required (see Part II, Chapter 2.7).

#### Navigation bulletins

Navigation bulletins should be made available to river operators daily, and should contain the following information:

- Water-levels of the main gauge-stations and the rise or fall at those stations (Although the Canal del Dique is not included in the actual channel patrol service, prediction of high water (level and time) in the Bahía de Cartagena might be included).
- The available depth for the following river sections:
 

	La Dorada - Pto. Inmarco
	Pto. Inmarco - Pto. Berrío
Río Magdalena	Pto. Berrío - Barrancabermeja
	Barrancabermeja - Gamarra
	Gamarra - Calamar
	Calamar - Barranquilla
Canal del Dique	Calamar - Bahía de Cartagena

For each section mentioned the location of the shallowest places should also be given

- General information on changes in channels and channel markers. In sections where buoys are used, these should be positioned correctly and changed immediately when required. Navigation bulletins should, therefore, not contain indications about the necessity of passing a buoy at a certain distance or avoiding certain buoys, etc. However, in stretches which are stable and simple and in which therefore beacons are used, the relocation of beacons is not always necessary immediately a change in the channel is found, provided this is mentioned in the navigation bulletins. For example, it may be stated that, at a certain place the crossing lies 50 m upstream of the crossing indicated by the beacons.

III, 2.2

Other information important to river operators, such as reports about the execution of river-works, the location of dredgers, measuring vessels, etc.

These navigation bulletins should be made available in different ways: in printed form they should be supplied to river captains in the ports, but they should also be broadcast at fixed times every day by radio (to be repeated at dictation speed). This should be done in the morning, about midday and in the afternoon. During the day new information should be included in the next bulletin broadcast. An example of a navigation bulletin is given in Figure 2.2.3.

<u>NAVIGATION BULLETIN</u>					
<u>DATE:</u>					
<u>RIVER SECTION LA DORADA - PTO. BERRÍO</u>					
<u>Water-levels</u>	<u>Yesterday</u>	<u>To-day</u>	<u>General</u>		
Arrancaplumas					
Pto. Salgar					
Pto. Triunfo					
Pto. Inmarco					
Pto. Berrío					
<u>LEAST AVAILABLE DEPTH</u>			<u>RESTRICTING CROSSING</u>		
La Dorada - Pto. Triunfo	ft			km	
Pto. Triunfo - Pto. Boyacá	ft			km	
Pto. Boyacá - Pto. Inmarco	ft			km	
Pto. Inmarco - Pto. Berrío	ft			km	
<u>CHANNEL INFORMATION</u>					
<u>Km</u>	<u>Name</u>	<u>Available Depth</u>	<u>Date</u>	<u>Gauge</u>	<u>Buoys, etc.</u>
<u>Special Notes:</u>					
Officer i/c channel patrols					

Figure 2.2.3 Example of Navigation Bulletin

From the foregoing it will be clear that several times a day radio contact should exist between headquarters and patrol units. Radios should therefore be installed on the patrol vessels in order to make available all information required to prepare the next navigation bulletin in the headquarters. Also the gauge readings on the main gauge-stations should become available immediately by means of radio or telephone. The main gauge-stations have been selected in such a way that this should be possible without much difficulty.

### Channel marking

Orientation on a river can be helped by the use of kilometer boards, beacons and buoys. In this order these navigation aids have an increasing accuracy to indicate the proper shipping route. Kilometer boards in combination with the navigation maps and navigation bulletins are sufficient in those sections where the channel is clearly defined and no actual problems exist. A clear-cut example is the Canal del Dique with the exception of the outlet in the Bahía de Cartagena, but also at parts of the river, mainly downstream of Gamarra, kilometer boards will be sufficient. These boards should be erected at least every five km; visibility at night may be increased by scotchlite tape or paint.

### Beacons and buoys

Generally it may be stated that channel marking by means of beacons is less accurate than by buoys, and the system is therefore more suitable for stretches where channels are simple, relatively stable and the river is not very wide.

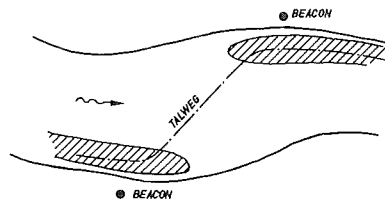


Figure 2.2.4 Crossing Marked by Means of 2 Beacons

A crossing may be indicated by means of two beacons (Figure 2.2.4), or by more beacons (transit beacons) as in Figure 2.2.5. It is obvious that by transit beacons a channel will be indicated more accurately. The installation of transit beacons is, however, much more difficult: often bush has to be cut, and as the beacons should be large in order to make them visible between the trees and bushes, the handling of the beacons becomes more cumbersome.

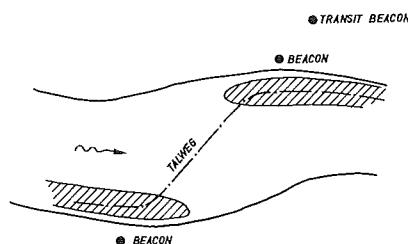


Figure 2.2.5 Crossing Marked by Means of Transit Beacons

It is advised that transit beacons along the Río Magdalena be used only in those cases where the channel is simple and stable, but where the river is so wide that the use of two beacons does not indicate the channel with sufficient accuracy. In all other circumstances buoys should be used.

### III, 2.2

It is thought that, except for transit beacons, the beacons used by ADENAVI are most suitable; the visibility is good, and they do not have the disadvantage of many other beacons which are so heavy and difficult to manage that repositioning is difficult (Figure 2.2.6).

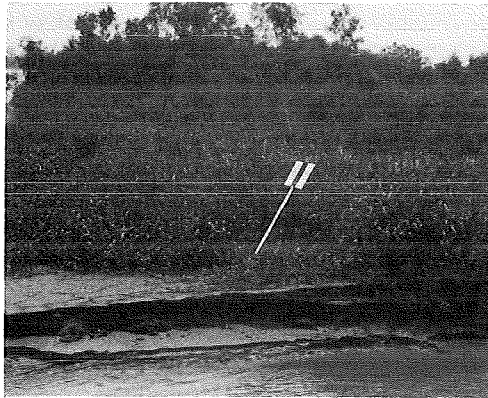


Figure 2.2.6 Beacon used by ADENAVI

At those places where beacons cannot indicate a channel with sufficient accuracy, buoys should be used.

For example, this is the case at the flats upstream of the Río Nare, upstream of Pto. Berrío, between Pto. Berrío and Río Nuevo, some places between Río Nuevo and Barrancabermeja, and near the Río Sogamoso Confluence; all places where a wide expanse of water exists with a badly-defined channel, changing rapidly. However, the sections mentioned often are bottlenecks for the navigation and the shallow depths available should be used as efficiently as possible.

Buoyage on rivers like the Río Magdalena should be by the lateral system indicating the channels (instead of the cardinal system indicating dangers). The buoys in such a lateral system should attract navigation, and are therefore normally required only on one side of the channel. Generally it is best to position the buoys on the steep side of the channel. Buoys on both sides of the channel are only required if the channel is very narrow (e.g., less than 60 m).

It is not good practice to use too many buoys for a crossing. At least two buoys are required to mark the beginning and the end, while in between other buoys can be placed. Using too many buoys tends to indicate channels which are too sinuous to be followed by large ships. Great skill is required to indicate channels providing a maximum depth combined with a minimum sinuosity.

Buoys have to be cleaned regularly of floating debris. The experiments described later have indicated that during rising stages cleaning should be done at least every two days.

Buoyage experiment

As it was expected that buoyage along the Rfo Magdalena would present difficulties in view of the high velocities and the large amounts of floating debris, some experiments were carried out. A number of buoys were made from 55 gallon oil drums. The buoys on the port side were painted red and white while those on the starboard side were painted black and white as indicated on Figure 2.2.7. The starboard buoys were made to float erect by slightly lowering the steel band around each buoy and filling the buoy with sand. These buoys were positioned close to Barrancabermeja and near the Rfo Regla Confluence.

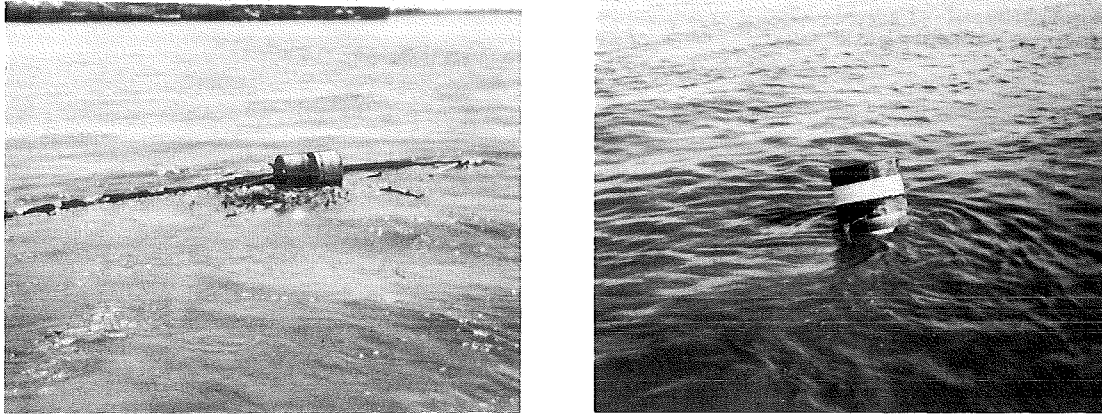


Figure 2.2.7 Port Side Buoy and Starboard Side Buoy

Concrete blocks of 75 kg, 150 kg and 300 kg were used as anchors. It was found that three blocks of 75 kg were insufficient, but three 150 kg blocks ensured the position of the buoys. In fact, it is believed that at some shallow places with small velocities 75 kg blocks would be sufficient, while at a few others with high velocities possibly three anchors of 300 kg will be required. This has to be found out by practice. The connection between buoys and anchors can be by steel wire.

The buoys could easily be placed by a crane; picking up the buoys presented a little more difficulty, but after some experience had been gained, this also could be carried out rapidly.

Specially during rising stages the buoys caught a lot of floating debris, but this did not diminish the visibility much. Visibility of the buoys was good, although the port side buoys were seen more easily than the starboard side buoys. (It is not thought necessary to improve the visibility of the starboard side buoys by means of a top mark, but if this seems required later, it can easily be done).

Two buoys were stolen during a long weekend; so a good system to safeguard the buoys against theft is required. This could be done by welding the steel band around the buoy.

### III, 2.2

Night navigation could be improved by providing buoys, beacons, and the km boards with scotchlite tape and paint. Most tugs are equipped with searchlights. Of course, the net of buoys and beacons should be more dense for night navigation. Experiments in this connection are suggested after a combined system of beacons and buoys has been accepted by the river operators.

#### Organization

The tasks a channel patrol service has to carry out should determine the number of staff and the organization required. These tasks, as indicated earlier, are:

- Gathering of channel information and transmitting this information to river operators; and
- finding and marking the best channels for navigation and keeping them cleared.

Both activities should fall under the responsibility of the same person. The best system is to divide the river into sections which can be patrolled by one officer who carries out both types of work. At the outset a division into two sections seems sufficient: La Dorada - Pto. Berrfo and Pto. Berrfo - Gamarra.

This will require two patrol officers, and during low water they should patrol their sections 3 times weekly. A third officer, the senior officer, should be in charge of all operations carried out by the other two officers. The senior patrol officer should also be able to fulfil normal patrol duties in the absence through illness or holidays of one of the junior officers.

In addition to these three persons, and of course the crews for the ships, a fulltime radio operator should also be appointed. He will maintain contact with the patrol units on the river, prepare the navigation bulletins, and do the actual broadcasting.

The crew for each patrol unit should consist of a captain, 3 deck hands, a cook, an engine driver and a "Johnsista".

The work of a patrol officer will not only be very responsible. It is also difficult, because it requires an insight into the river morphology. He will also spend a large part of his time on the river, year after year. A careful selection of staff is, therefore, required, while his salary should be in relation to his duties (which means equal to or slightly less than an engineer's salary).

A channel patrol service could possibly be incorporated in a larger agency or a special river conservancy department.

#### Vessels and equipment

To be able to carry out the work indicated in the preceding section, two patrol vessels will be required having the following characteristics:

Accommodation: Should be sufficient for the people indicated.

Draught : Should be preferably slightly less than 3'.

Speed : Should be relatively high as at least 3 trips from La Dorada to Pto. Berrfo should be possible weekly (3 x 160 km), buoys

positioned and cleared, soundings made, and channels cleared on the way. This means that going from Pto. Berrfo to La Dorada (upstream) the journey should be made well within a day (say 8 hours) requiring a speed of 20 km/hour going upstream and 25 km/hour in still water. On the other hand, manoeuvrability should be good at low speeds.

**Working space:** The ship should have a low working deck about 40 to 50 cm above the water-level with sufficient space to store and handle buoys and anchors.

**Equipment :** The vessel should be equipped with a speed-boat with outboard engine which may be used, among other things, to clean buoys more often than 3 times a week. The vessel should further have a crane or derrick with a lifting capacity of at least 3 tons for the easy handling of buoys. It should also be equipped with an echo-sounder and radio transmitter/receiver. The characteristics mentioned for this vessel seem to indicate a catamaran-type of vessel.

Besides the two patrol vessels, there should be an inspection launch (speed-boat) capable of about 40 km/hour and with very simple accommodation, but equipped with radio transmitter/receiver and echo-sounder. In Barrancabermeja, a powerful radio station should be available.

#### Summary

Improvements of the channel patrol service suggested here can be divided into those improvements which can be made immediately without much investment and those which require more time to effectuate and also relatively large investments.

#### Immediate improvements

- Frequent distribution of up-to-date river maps for navigation purposes,
- positioning of kilometer boards every 5 km,
- transit beacons at a few crossings, and
- transmission of available information to river operators by means of navigation bulletins.

Although at present it is not possible to transmit all data required, some data, such as water-levels, could be made available; this would have the additional advantage of making river operators acquainted with the navigation bulletins.

#### Future Improvements

It is suggested that 2 patrol vessels and an inspection launch be made available as indicated and that with these patrol vessels and the staff to be appointed, patrol services be extended and improved, while also a combined channel marking system of buoys and beacons be introduced.

As some time will be required to design and build the required vessels, an earlier start could be made by making use of a small barge and tug as a substitute for a patrol vessel. A crane should be mounted on the barge.

2.3. TEMPORARY RIVER-WORKS

2.3.1. Introduction

Temporary river-works are constructions or works in a river of which the effect is intended to last only one season. In the following season similar works have to be repeated because either the original works have been damaged during the high water season or because the works are needed at another place. Problems for navigation seldom occur in exactly the same spot in successive years or seasons, as they will have moved if the river's course is not completely fixed. It will be clear that in those cases temporary works are indicated. Temporary works may also be used for other reasons: for example, they may be cheaper than permanent river-works. Although temporary river-works are sometimes carried out by means of constructions only, generally temporary works consist of dredging, or dredging in combination with some kind of construction (Para. 2.3.2).

On the Rfo Magdalena the following types of temporary works in relation to navigation may be considered:

- Dredging of crossings;
- closing of secondary branches (in combination with dredging of crossings); and
- opening up harbour approaches.

Along the Canal del Dique the following types of work may be considered:

- Dredging of the outlet of the Canal in the Bahía de Cartagena;
- dredging of a sediment trap (Calamar);
- maintenance of distributing Caños (Correa, Matunilla, Lequerica); and
- maintenance of the Canal profile.

These works all relate to navigation. Maintenance dredging can, however, also be carried out for the drainage of "ciénagas", protection of villages, maintenance of channels to water-works, etc. The various types of work often require different types of equipment.

There are many types of dredging equipment, but because of the available depth, currents and type of bed material, only a few types are suitable for the Rfo Magdalena. In fact, for the type of dredging as indicated above a choice should be made between a cutter-dredger and a dustpan-dredger, while for small works and the maintenance of river port approaches also a (floating) crane may be suitable.

In Para. 2.3.3 some characteristics of dredging equipment are dealt with, while in Para. 2.3.4 more specifically the equipment at present available on the Rfo Magdalena and Canal del Dique is considered. In Para. 2.3.5 a description is given of new dredging equipment which is required. Although in this Report mainly the navigation interests have been considered, it was thought that as far as dredging equipment was concerned, it was necessary to keep the possibility in mind of using the equipment also for other purposes, such as high water protection (drainage).

### 2.3.2. Temporary constructions

Temporary constructions may consist of bandals or of bottom and floating panels. Both bandals and panels concentrate the flow. The difference between the two systems is the helicoidal flow induced by the panels with the purpose of moving bed material out of the channel, while with bandals some sediment is brought back into the channel (see Figures 2.3.1 and 2.3.2).

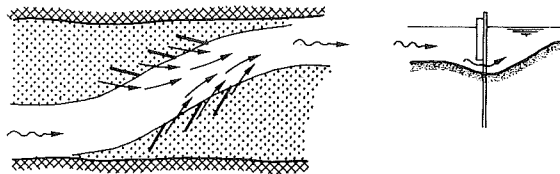


Figure 2.3.1 Bandals

Bandals are made from locally-found material such as cane. Panels are often of a somewhat more permanent construction and may be considered to be a transition between temporary and permanent river-works.

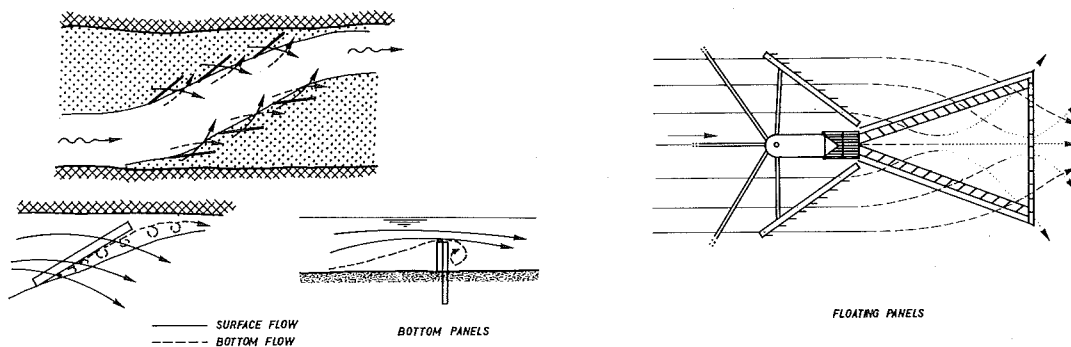


Figure 2.3.2 Bottom and Floating Panels

The construction of bottom panels is relatively expensive, especially in labour, and in most cases dredging will give a cheaper solution. Sometimes a good solution may be found by combining panels with dredging.

### 2.3.3. Dredging equipment (general)

#### Cutter-dredger

A cutter-dredger consists of a pontoon in which a pump is generally driven by a diesel engine, although sometimes it is driven by a steam turbine or a diesel-electric installation. A sand-water mixture is pumped from the bed through a suction pipe and transported through a pipeline to a disposal area (Figure 2.3.3).

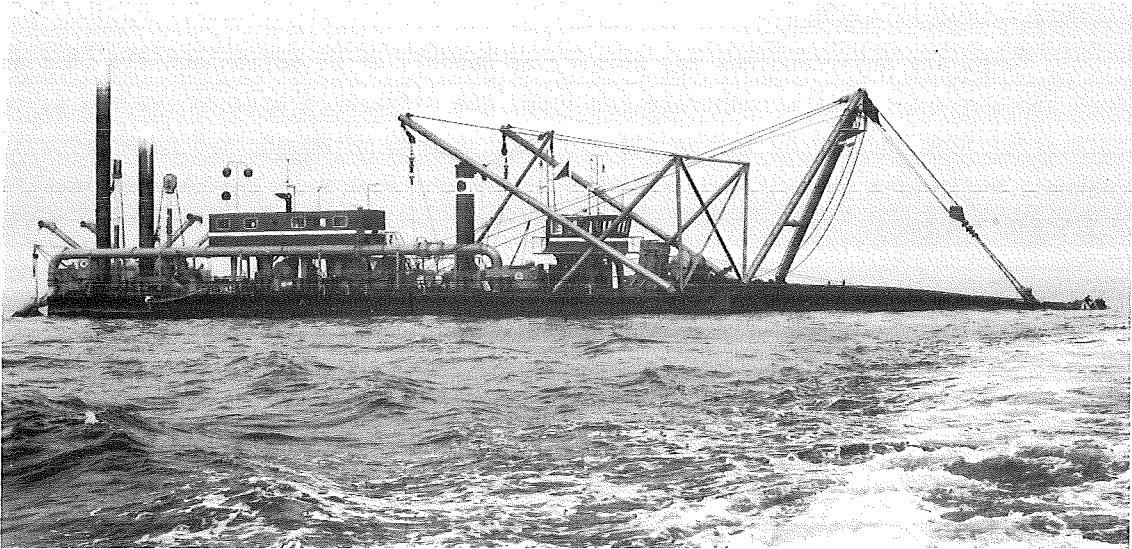


Figure 2.3.3 Cutter-dredger

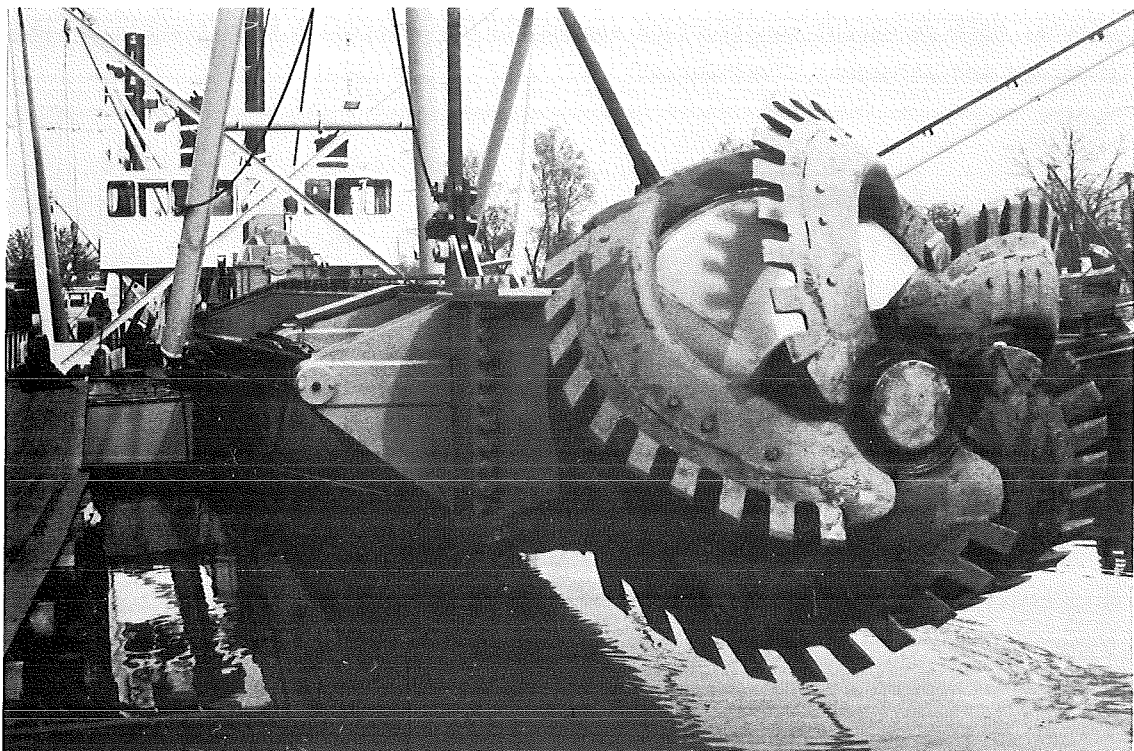


Figure 2.3.4 Cutter head

As the suction pipe has a flexible connection with the pump, dredging can take place, within limits, at any required depth. In front of the suction pipe a cutter head is mounted (Figure 2.3.4) which serves to loosen the material to be dredged. Dependent on the kind of soil, different types of cutters may be used. Cutter-dredgers can be used for sand, clay, or soft rock. Between the suction tube and the pump a trap is installed to catch large

pieces of stone or wood which pass the cutter but are too large to pass through the pump. This debris can be removed by lifting a lid. Pieces of wood jammed in the pump can also be removed in this way.

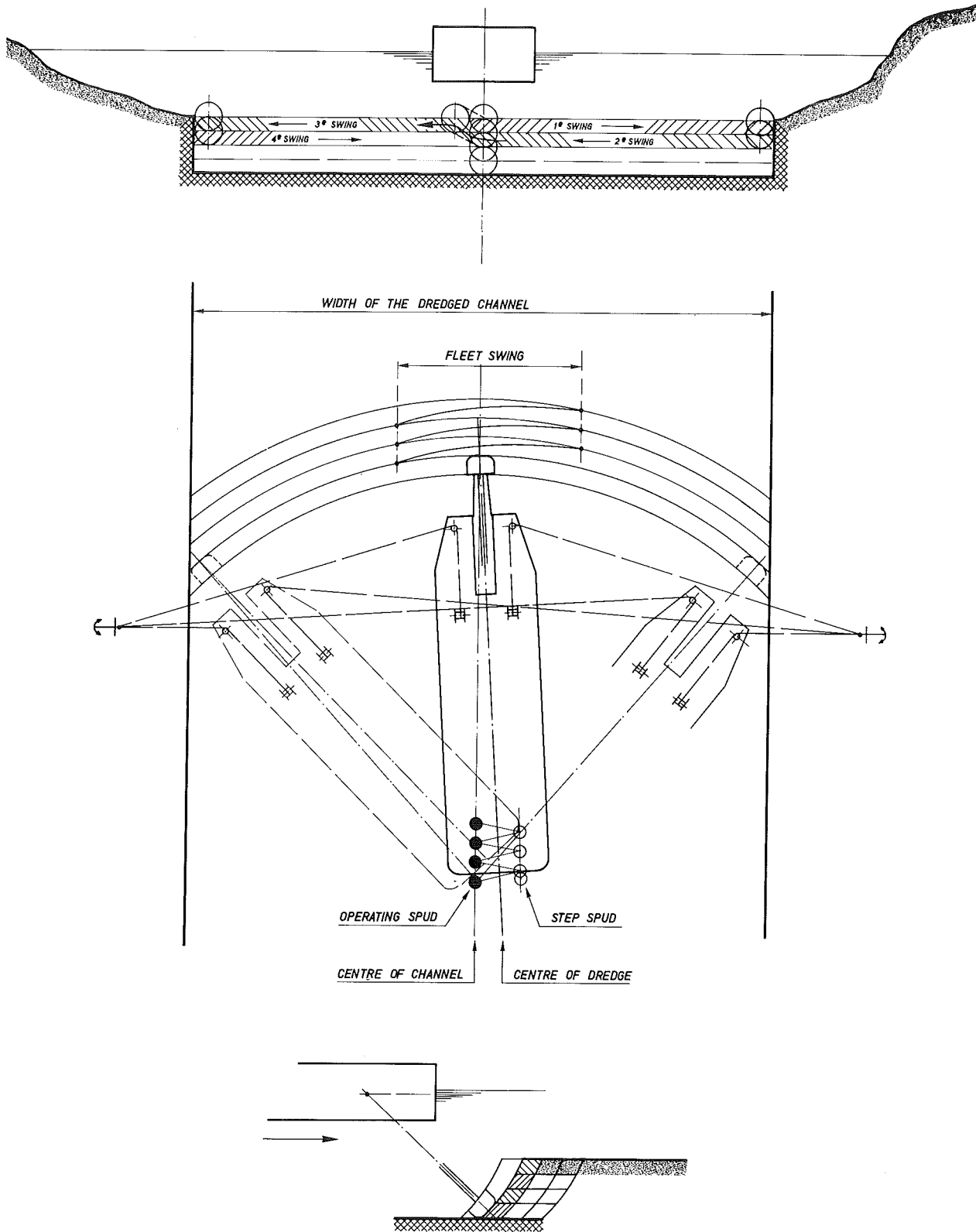


Figure 2.3.5 Operation of Cutter-dredger

### III, 2.3

The operation of a cutter-dredger is shown in Figure 2.3.5. As can be seen, the cutter-dredger moves forward by means of spuds and swings by means of side anchors. Sometimes one of the spuds is mounted on a spud-carrier as shown in Figure 2.3.6, in which case the operation of the dredger is slightly different.

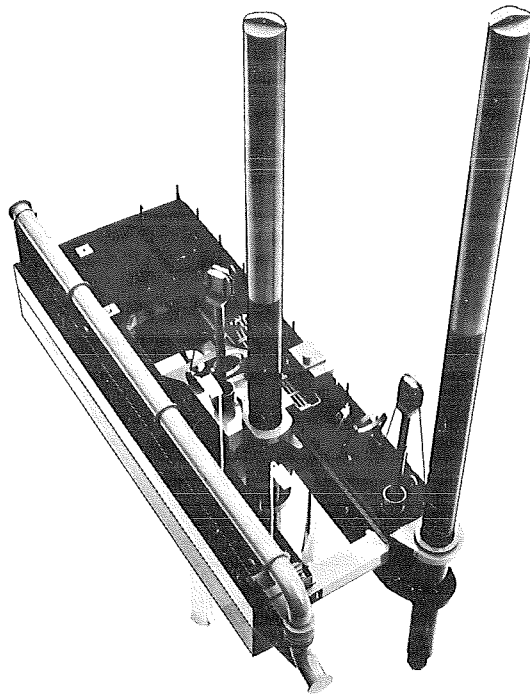


Figure 2.3.6 Spud-carrier

Although, this is not the general practice, on rivers the operation of a cutter-dredger in a downstream direction must be preferred. In that way the spoil and transported sediments do not easily fill up the dredge cut, while it is easier to maintain a high concentration. Some special provisions are required to dredge in a downstream direction, however (for instance, a stern winch).

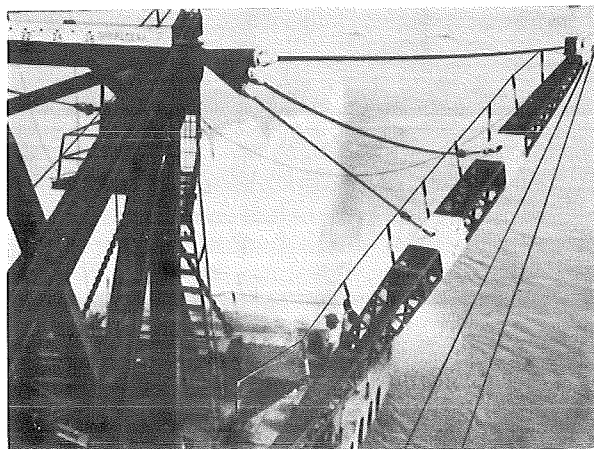
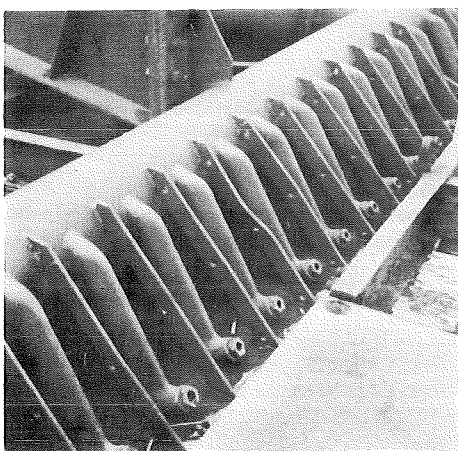


Figure 2.3.7 Water Jets of Dustpan-dredger

### Dustpan-dredger

A dustpan-dredger differs from a cutter-dredger as the sediment is loosened by means of water jets instead of a cutter (see Figure 2.3.7). The mouth of the suction pipe is similar to the intake of a vacuum cleaner. A dustpan-dredger is only suitable for non-cohesive, not-too-coarse material. Operation of a dustpan-dredger differs from that of a cutter-dredger as indicated in Figure 2.3.8.

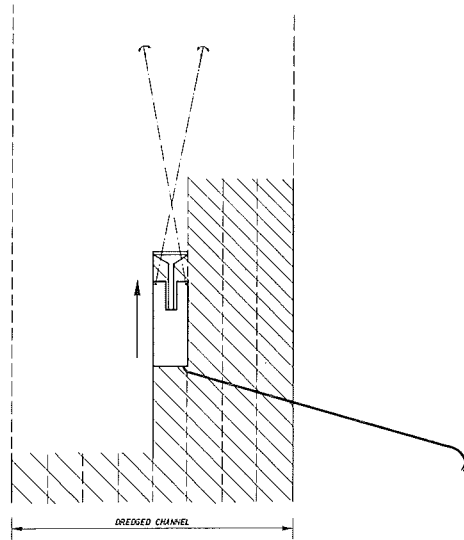


Figure 2.3.8 Operation of Dustpan-dredger

The advantages of a dustpan-dredger are:

- It is possible to remove a relatively thin layer (1') and at the same time maintain a large output; and
- After one run of the dustpan-dredger a small channel is excavated which may attract the current, thus maintaining the dredge cut. A cutter-dredger immediately makes the required width of the channel, but the channel is not maintained, especially when dredging in an upstream direction. This is because no current is drawn unless from the sides, which even may spoil the channel again as relatively large quantities of sediment also enter the channel.

Comparing the dustpan-dredger with a cutter-dredger, the latter can more universally be used.

### Pumps and pump-engines

The sand pump is one of the most important components of the dredger. The construction of a sand pump is always the result of a compromise. The shapes of the impeller and the pump casing should be designed in such a way that a high efficiency is secured, but care must be taken to ensure that large objects can pass through the pump, that wear is at a minimum, and that dismantling and repairs can be carried out easily. These requirements are best answered by a centrifugal pump.

The production of a centrifugal pump driven by an engine with constant rotation speed depends, of course, on the pressure-head that must be overcome. For normal operational conditions, a variation in the discharge only slightly affects the head produced; the interrelation between discharge and head is schematically given in Figure 2.3.9. There is, however, a limit to the performance of the engine. With increasing discharge and only slightly decreasing head, the load of the engine increases and this can only continue until the point of full power has been reached. A further increase of discharge can then no longer be obtained, and the engine will lose speed and power, so that the head decreases sharply (see Figure 2.3.9b). For most pumps, the performance will depend on these two conditions: maximum allowable speed (left part of the line in Figure 2.3.9b) and maximum possible power (right part of the line).

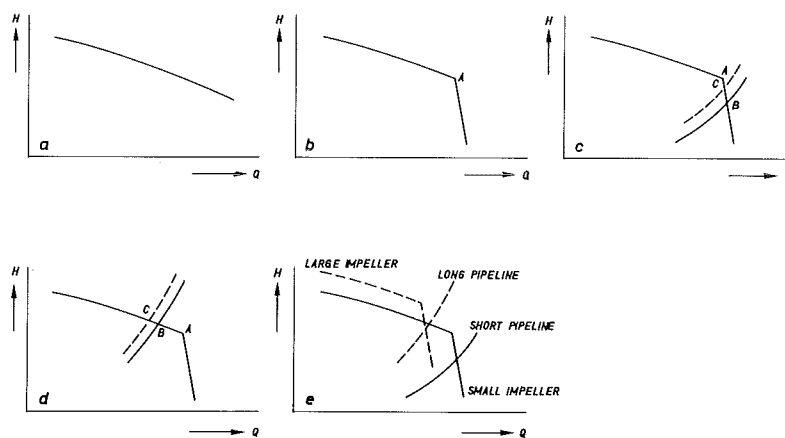


Figure 2.3.9 Behaviour of Sand Pump under Various Conditions

Where along this function the pump operates, mainly depends on the pressure-head required to overcome the pipeline resistance. In general, the resistance of the pipeline is almost proportional to the square of the velocity, and such function is schematically represented in Figure 2.3.9c. The operation point then is where the pushing force equals the resisting force, a condition which is met in the point of intersection B of both curves. But with this theoretical approach, a problem crops up. When the pipeline velocities are high, all the transported sand is in motion. If, for any reason, the discharge decreases (perhaps by a temporary obstruction of the suction tube) the velocity is also reduced and part of the sand will, at a given moment, settle in the pipe, thus decreasing the available profile for the flow. It would be expected that the lower discharge would imply a smaller resistance, but this effect is neutralized, or perhaps even outweighed, by the sedimentation, and the resistance curve shifts upward (dash-line in Figure 2.3.9c). This means that also the operation point of the system shifts upward (point C). The change in operation point from B to C implies a slight decrease of discharge and a reasonable increase of pressure-head, and there will be a small chance of difficulties occurring because the velocity is limited and the increase in pressure-head may succeed in blowing the obstruction out.

For a situation as sketched in Figure 2.3.9d, however, the conditions are different; the shift of the resistance curve involves a further considerable decrease in the discharge, whereas the pressure-head rises only slightly. Consequently, the sedimentation in the pipe tends to increase and the whole pipeline may get blocked. A remedy could be to hoist the ladder so as to pump in pure water and clean the system, but as the process takes place very rapidly it is often extremely difficult to take timely action.

During the dredging operations, the concentration of sand in the mixture is continuously varying, and this, too, causes an unceasing shifting of all the relation curves of Figure 2.3.9. A blocked pipeline is, therefore, certainly not imaginary. It would be good to have the pump and engine of the dredger designed in such a way that the point of operation lies on the steep part of the pump characteristic (i.e., the torque, exerted by the engine upon the pump axle, should be maximum). On the other hand, it should be borne in mind that as the point of intersection A represents the condition of optimum use of engine and pump, the actual operation point should, for reasons of efficiency, not be too far from this optimum point A. Here again, a difficulty arises: dredgers have to be capable of operating under various conditions, and one of the most important variables is the length of the pipeline. Figure 2.3.9e makes it clear that when conditions for a short pipeline are favourable a long pipeline does not produce a satisfactory situation. At present, however, pumps exist which maintain a high efficiency over a large range of conditions, and this range may be increased still further by using a different pump impeller. In Figure 2.3.9e the dash-line represents the pump characteristic for a large-diameter impeller, whereas the unbroken line indicates the characteristic of a small-diameter impeller. The optimum conditions can be restored by choosing the optimum impeller. If the required range of pipeline lengths would be very great, the variation in pump impellers would be excessive and the efficiency of the pump performance would deteriorate. A possible solution would be to use a second pump which could be operated in series with the first one.

### Pipeline

The pipeline serves to transport the soil hydraulically, i.e., as part of a moving mass of water/soil mixture. This means that the particles must be kept in suspension. The condition to satisfy that requirement is not sharply defined, because the pumped soil is not uniform, but it is generally possible to speak about a "critical" velocity below which sedimentation will occur. This critical velocity thus imposes a lower limit to the pipeline velocities.

Figure 2.3.10 gives the approximate relation between the critical velocity and the average particle size of the transported soil for mixtures with a soil concentration of 15% or higher. For lower concentrations the critical velocity decreases.

Much higher velocities are not strictly necessary, as extra velocity not only increases the pipeline resistance, and thus the required engine power, but also causes relatively great wear in pump and pipes. Both effects involve a rise in the production cost. On the other hand, it must be said that high velocities certainly have some advantages. The risk, for instance, of a blocking of the pipeline, as outlined previously, becomes less if

### III, 2.3

the pipeline velocities are well above the critical value. Moreover, the pipeline diameter can be smaller if, for a given required output, higher velocities are allowed; and this makes the pipes more easily manageable.

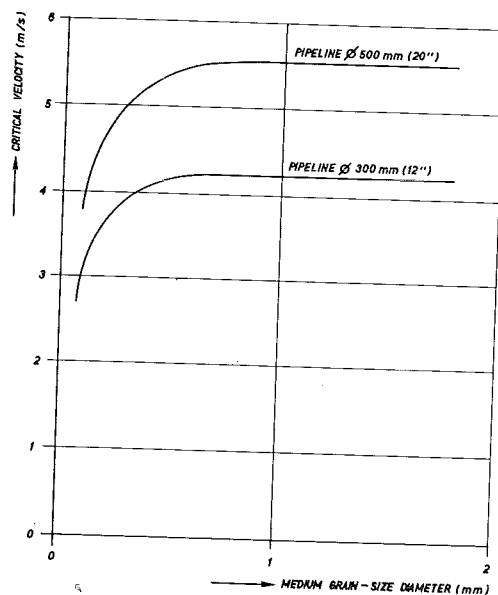


Figure 2.3.10 Critical Velocity in the Pipeline

Especially for the floating pipeline this latter argument is of great significance. Reduction of the pipe diameter also reduces the weight of the pipeline (filled with water) and, consequently, the draught of the supporting elements (pontoons), which is a favourable factor in view of the small available depth on and near a river crossing. Particularly at the end of the pipeline is a shallow draught required. The end-pontoon, therefore, is often equipped with a small derrick from which the last pipe section is suspended, hanging over the shallow water area.

Another aspect of the floating pipeline is its control. In principle, this type of pipeline must be flexible as to allow movement of the dredger with respect to the dumping-place or, if the spoil is pumped ashore, with respect to the shore connection. This flexibility is obtained by movable joints, either ball-joints or hinge-joints (see Figures 2.3.11 and 2.3.12), and a great number of these joints permit the floating pipeline to take a smooth course without being subjected to great bending forces of wind and current. But a high flexibility is not advantageous under all circumstances; for example, serious trouble may arise when current and wind are in opposite directions, or when, in tidal areas, the current direction alternates between flood and ebb. Unwanted motion of the pipeline is then often prevented by anchoring some of the pontoons. But as anchors complicate the operations, it would in that case be better to reduce the number of joints.

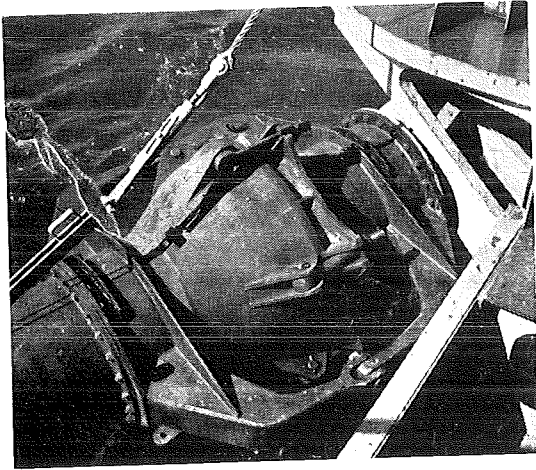


Figure 2.3.11 Ball-joint

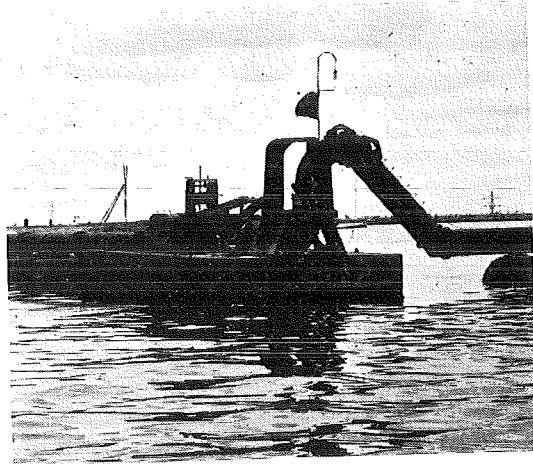


Figure 2.3.12 Hinge-joint

Theoretically, full flexibility between the dredger and the end of the pipeline is obtained with three joints. When the spoil is dumped back into the river, the floating pipeline can even be made entirely rigid with only one movable joint at its connection with the dredger. In that case, an attractive means of controlling the pipeline becomes possible, viz., by making use of the outflow momentum of the jet at the end of the pipeline. A baffle-plate (Figure 2.3.13) serves to give this force the appropriate direction and the pipeline can be steered to the required position.

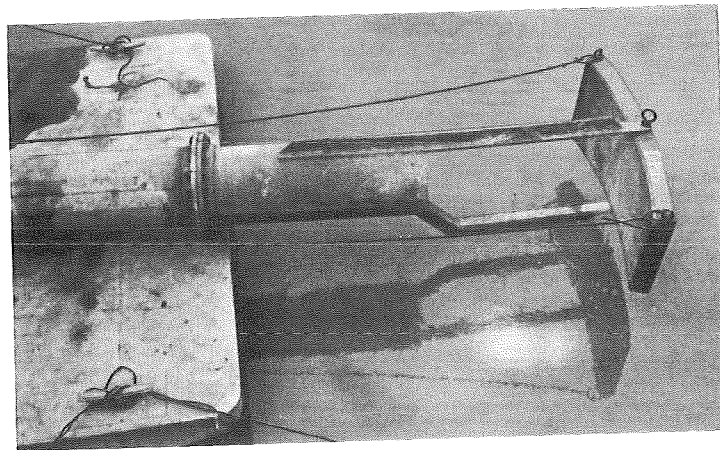


Figure 2.3.13 Baffle-plate

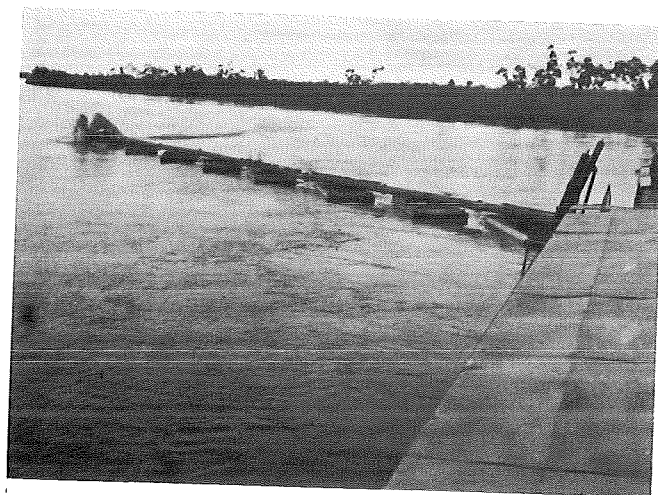
Obviously this method is only feasible when the impulse momentum of the jet exceeds the momentum of the resistance of the pontoons in the flowing water. The outflow impulse of the jet at the end of the pipeline equals:

$$F_j = \frac{1}{4} \rho_m \pi D^2 v_p^2$$

with  $\rho_m$  = density of the water/soil mixture

$D$  = diameter of the pipeline; and

$v_p$  = velocity in the pipeline.



Floating Rigid Pipeline, Steered by Baffle-plate

The pushing force exerted by the river flow on the pontoons can be computed by:

$$F_r = \frac{1}{2} \rho_w C_w n P h v_r^2$$

- with  $\rho_w$  = density of the river water;  
 $C_w$  = the resistance co-efficient of the pontoons in the river flow (for cylindrical pontoons approximately  $\frac{1}{2}$ );  
 $n$  = number of pontoons;  
 $P$  = pontoon diameter;  
 $h$  = pontoon draught, depending on the weight of the floating pipeline filled with water/soil mixture; and  
 $v_r$  = river current velocity.

Steering is possible when  $F_j > \frac{1}{2} F_r$ .

It is sometimes necessary also to take wind influence into consideration (especially on the pipeline itself).

For quick and efficient work on river crossings, this rigid-type pipeline with baffle-plate is very suitable. It is obvious, however, that rigid and semi-rigid pipelines need to be stronger.

Instead of the use of a pipeline, also a nozzle can be used for free spoil discharge over short distances. This principle can also be used at the end of a short floating pipeline, instead of using land lines.

Flexible floating pipelines have recently been developed, made of reinforced synthetic material. These pipelines do not need floats: they float partly emerged. No ball-joints or hinges are required because the pipeline itself is sufficiently flexible. From experience in Europe it follows that the higher investment for such a pipeline is counterbalanced by a higher life-time. For Colombia the fact that it is not constructed locally but still needs to be imported is an additional disadvantage.

Capacity

Theoretically, the output of the dredger can be computed by determining the pump discharge (at a certain required head), and multiplying this by the concentration of solids in suspension. It is, however, very difficult to make any reliable prediction with respect to this latter magnitude. In general, a figure of 20% (in volume) has to be regarded as a maximum, but actual concentrations can be much lower. Although natural conditions (coherency of the soil, dredging depth, river current, etc.) certainly have their influence, much depends in this respect on the skill and feeling of the dredge-master and his crew. Even an experienced dredge-master who makes good use of vacuum and pressure meters, and who can thus respond immediately to any disturbance within the pipeline or to any change of the soil inflow in the suction mouth, will probably not be able to maintain maximum concentration all the time. It would, therefore, be safe to count on an average sand concentration of not more than 15%. Experience, of course, provides figures which roughly indicate the average capacity of a certain size dredger operating in a certain type of bed material, and these experience figures may be useful for a first approach when considering the requirements for the dredger to be obtained. On the Rfo Magdalena such information will have to be gathered from test dredging.

Auxiliary craft

For a proper functioning of the dredging equipment, a few satellite vessels will be indispensable. The purposes, types and numbers of these auxiliary craft depend on the local circumstances, the type and size of dredging equipment, and the traditions that prevail in the country. Without enlarging too much upon this subject, mention may be made in this respect of a small tug, a sounding-boat, and a vessel for the supply of fuel, food, drinking-water and spare parts. However, some additional vessels may be required to provide facilities not available on the dredger itself: an anchor-boat if the dredger is not equipped with anchor booms, and extra craft to serve as living space for crew members not accommodated on the dredger.

All such satellite vessels can best form part of the dredging unit, and be under the command of the dredge-master.

2.3.4. Dredging equipment at present in use on the Rfo Magdalena and the Canal del Dique

The dredgers in the possession of MOP are listed in Table 2.3.1.

In fact, at present for dredging purposes on the Rfo Magdalena and the Canal del Dique the following dredgers are available: DH 1, 6, 7, 8, 9 and 10.

The DH 7 works only on the Canal del Dique and for contracts, while the DH 6 and DH 9 are used on the Canal del Dique most of the time.

Although the DH 1, 8 and 10 work on the Rfo Magdalena, they are only used for the protection of villages, etc., but not for the improvement of navigation. There is not a good coordination between the work of the different dredgers and, in fact, this would be very difficult with the present type of organization.

### III, 2.3

Name	Make	Capacity m <sup>3</sup> /hr (solids)	Base	Present Use	Remarks
DH 1	Ellicot	150	Barranquilla	Rfo Magdalena, protection of villages, etc.	Steam, very old (Panama canal) plans for conversion with parts of DH6
DH 2	Alemana	200	Quibdó		
DH 3	Ellicot	300	Buenaventura		
DH 4	Ellicot	150-200	Buenaventura	Colpuertos	
DH 5	Ellicot	300	Tumaco		Casco needs repairs
DH 6	Ellicot	450	Barranquilla	ADENAVI, mainly Canal del Dique, but also Rfo Magdalena	To be converted to 1,400 m <sup>3</sup> /hr solids
DH 7	Deggendorf	275	Cartagena	Junta Canal del Dique	
DH 8	Deggendorf	325	Barranquilla	Rfo Magdalena, protection of villages, etc.	
DH 9	Deggendorf	300	Barranquilla	ADENAVI, Canal del Dique	
DH 10	Deggendorf	300	Barranquilla	Rfo Magdalena, protection of villages, etc.	
Colombia I.H.C.		1,000	Barranquilla	Colpuertos	

Table 2.3.1 Dredgers in Possession of MOP (data as per January 1973)

A study is being initiated at present for improvement of the dredging organization as well as the technical features of the available dredgers in relation to the required dredging in Colombia. During the Rfo Magdalena and Canal del Dique Survey Project some data have already been obtained concerning the dredgers working along the Rfo Magdalena and the Canal del Dique (DH 1, 6, 7, 8, 9 and 10). The DH 1, 6, 9 and 10 have been inspected by the Mission and some remarks will be made about them. However, the comments on the DH 9 and 10 are also largely valid for the DH 7 and DH 8, being very similar dredgers.

#### DH 1

The decision to recondition the DH 1 with parts of the DH 6 should not be taken too lightly. The pontoon is probably made of mild steel and welding may present difficulties. If this can be overcome and the hull of the DH 1 is still in a good condition, the reconditioning of the DH 1 may be a good proposition.

#### DH 6

Especially when this dredger is converted to a capacity of 1,400 m<sup>3</sup>/hr (solids), it will be a very good general purpose dredger, suitable for use on the Rfo Magdalena as well as on the Canal del Dique.

For test dredging it is the most suitable dredger, as it can also be used as a dustpan-dredger. At present it is not known which type will be more suitable for the dredging of crossings on the Rfo Magdalena: a cutter or a dustpan-dredger. This question should be answered by the test dredging (in 1973).

#### DH 9 and DH 10

The makers of these dredgers, being builders of traditional ships, probably had little experience in the building of dredgers. As a result, there are a number of shortcomings in these dredgers.

- The pumps have a very small diameter and, consequently, high speeds, resulting in excessive wear of pump-house and impeller, risk of damage by larger stones in the pump, while the pump is easily blocked by large pieces of wood entering the small pump-house.

Due to the small pump-house the impeller is of a bad design; the impeller blades being too short and at the wrong angle. The installation of new pumps should be considered.

- The engines (MAN) have been designed for the propulsion of ordinary ships and as used at present are not very suitable for dredging because, with the number of revolutions below the design value, the available torque decreases rapidly (dredge pumps require engines with a nearly constant torque over a large range). The best solution seems to build a reduction gear between the pump and the engine. The practice of the engine-room staff to run the engines at even lower speeds than permitted by the manufacturers reduces the capacity even more. Exhaust temperatures of 300° C were common during the test dredging in 1972, instead of normal working temperatures of about 400° C.

- The cutter-head should be provided with a special knife to cut pieces of wood which come in front of the cutter. A further improvement can be obtained by the installation of a cutter at the entrance of the pump-impeller, which may cut the finer pieces of wood which have still passed the cutter (see Figure 2.3.14).

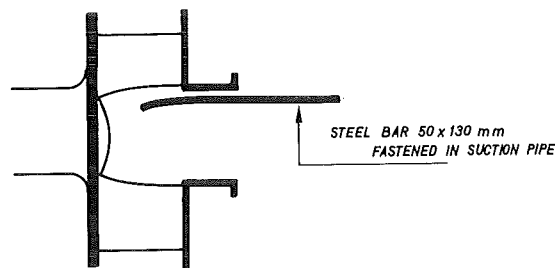


Figure 2.3.14 Wood Chopper

- The spuds are too light and of a wrong design. The spuds of the DH 10 were changed without difficulty during the test dredging in 1972, and these changes are indicated in Figure 2.3.15. The weight of the spuds was increased by filling them with sand and gravel, after which they worked adequately.

- Anchor-winches should be of a type that heave in with a constant speed, even when the load changes. This is impossible with the type used on the DH 9 and DH 10. The torque converter should be removed and a different coupling designed. The swinging speed should be maintained by the operator at such a level that a good vacuum is maintained (8 m). During the test dredging in 1972 extremely low vacuums were maintained with consequent low production. Evidently the anchors should be of sufficient weight and correct design to counteract the forces with large swing speeds; the anchors on the DH 10 were too light.

- The pipeline of the DH 9 and DH 10 is too thin, resulting in much time lost due to damage of the pipeline. New pipes should have a thickness of at least 8 mm.

- During the test dredging none of the instruments (vacuum, pump-pressure) worked properly. Without these instruments it is impossible to make a good production. Installation of production meters should be considered.

It may be concluded that the DH 9 and DH 10 will never be ideal dredgers for use on the Rfo Magdalena, although with minor alterations their usefulness can be greatly improved.

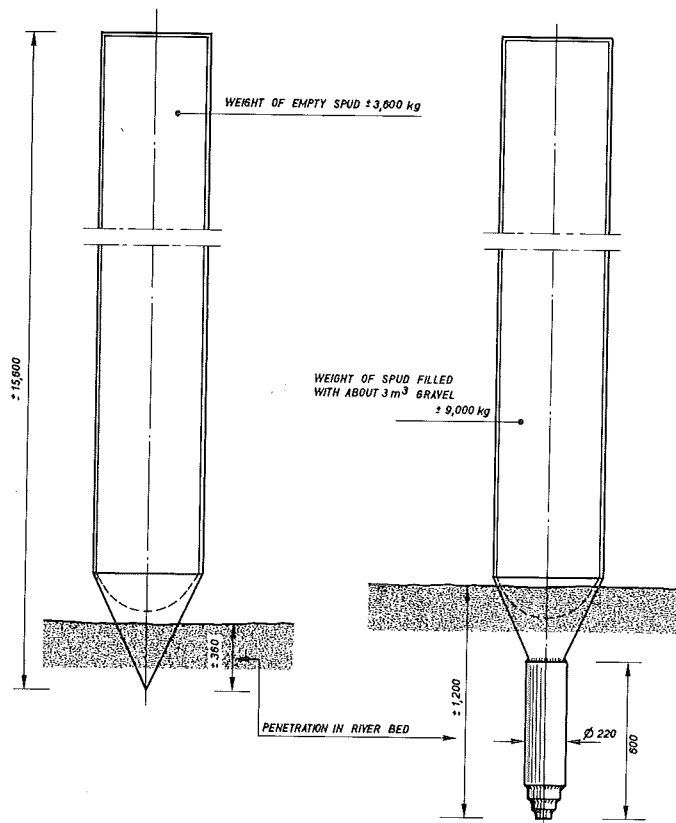


Figure 2.3.15 Conversion of Spuds of DH 10

A general remark can be made about the spare parts for all the dredgers working on the Rfo Magdalena at present. A far greater supply of spare parts should be available. At present, dredgers often have to stop because of lack of spares and consumables such as shackles, wire, etc.

#### 2.3.5. New equipment for the Rfo Magdalena

The most suitable type of equipment for dredging on the Rfo Magdalena will now be discussed. The optimal capacity of such a dredger, when used for dredging crossings, will be based on an assumed average crossing. The number of dredgers required cannot be given here, as this depends on the total amount to be dredged and the time available. This is dealt with in Chapter 5.

It has been assumed that the dredger should be optimally designed for navigation dredging, but still be well equipped to carry out dredging for other purposes. The present custom, however, of using large (expensive) dredgers for work where often a crane or bulldozer would do the job more efficiently, should be abandoned. It is, in fact, advised that a few very small dredgers and floating cranes be purchased to carry out the many necessary small works along the river, and to use large dredgers only where use can be made of their capacity.

Some properties of the optimal dredger for navigation dredging (crossings) are dealt with below, followed by a general description of such a dredger.

Capacity

In Part III Chapter 5 it is shown that between 80 and 100 crossings, varying considerably in size, will have to be dredged at the end of the high season. In the upstream part at some crossings only about 1,000 m<sup>3</sup> or even less has to be removed, while at other places sometimes more than 50,000 m<sup>3</sup> will have to be taken away. It is impossible to have a dredger which is optimal for all of those crossings. A common crossing is about 15,000 m<sup>3</sup>, and this has been taken as a standard crossing. But to have some insight into the influence of the size of the crossing, also crossings of 5,000 m<sup>3</sup> and 25,000 m<sup>3</sup> have been considered. To determine the optimal capacity, the following assumptions have also been made:

- The dredgers are worked in two shifts of 10 hours, while urgent repairs can be done in the remaining 4 hours of the day;
- the two shifts, including crew, auxiliary craft, etc., consist of 20 men;
- on Saturday and Sunday the dredger is operated by one shift (10 hours per day);
- distance between the crossings is 3 to 4 km (sailing time without breaking up and preparation: ½ hour);
- depreciation and interest amounts to 20% per year of the invested capital; and
- an average efficiency of 80% is obtained.

In Table 2.3.2 the time required to dredge one crossing is shown for different dredger capacities.

Crossing (m <sup>3</sup> )	5,000				15,000				25,000			
	300	600	1,200	2,200	300	600	1,200	2,200	300	600	1,200	2,200
Capacity (m <sup>3</sup> /hr; solids in situ)												
Dredging	16.7	8.3	4.2	2.3	50	25	12.5	6.8	83.3	41.7	20.8	11.4
Preparation	2	3	3	4	2	3	3	4	2	3	3	4
Losses for changes, (re)positioning, etc.	2.5	1.3	0.6	0.3	7.5	3.8	1.9	1	12.5	6.3	3.1	1.7
Sailing	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Time lost for week-end	3.1	1.9	1.2	1	8.6	4.6	2.6	1.8	14	7.4	3.9	2.5
Total Time (hours)	24.8	15	9.5	8.1	68.6	36.9	20.5	14.1	112.3	58.9	31.3	20.1

Table 2.3.2 Time Required to Dredge one Crossing for Different Dredger Capacities

In Figure 2.3.16 the total cost of dredging one crossing has been indicated (for different capacities) as found from the hourly running cost and the number of hours required to dredge one crossing (Table 2.3.2). This has been done for a crossing of 5,000 m<sup>3</sup>, 15,000 m<sup>3</sup> and 25,000 m<sup>3</sup>. As can be seen, for a crossing of 15,000 m<sup>3</sup> the minimum cost is obtained by a dredger capacity of about 1,200 m<sup>3</sup>/hour (solids in situ). This minimum, however, is very flat; a deviation between the selected capacity and the optimum capacity is not, therefore, of very great influence. The value of the optimum capacity must, moreover, not be considered as a very exact value, because it is influenced by the assumptions already mentioned. The assumed time of preparation (3 hours) in particular may need to be altered if test dredging data indicate a different value. This would shift the minimum in the graph.

### III, 2.3

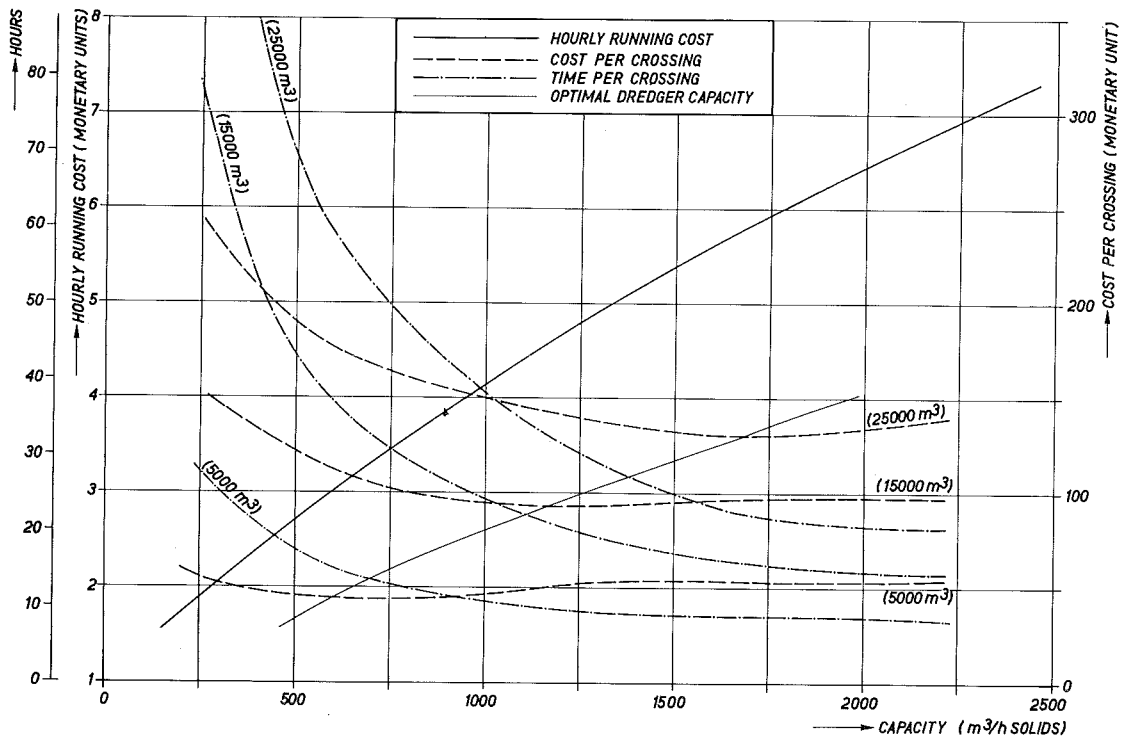


Figure 2.3.16 Optimal Dredger Capacity

It may also be concluded from the graph that for a crossing of 5,000 m<sup>3</sup> the optimum capacity would be about 600 m<sup>3</sup>/hr (solids in situ), while for a crossing of 25,000 m<sup>3</sup> this would be about 1,700 m<sup>3</sup>/hr. As the increase of cost is relatively smaller for a dredger with a larger than optimal capacity than for a dredger with a smaller than optimal capacity, it is better to select a capacity a little too large than too small. For the above computations, an imaginary monetary unit has been used to avoid the influence of devaluation, etc. At present (1973) this monetary unit would show a value between Col \$ 1,000 and Col. \$ 1,500.

#### Cutter-dustpan

Test dredging by a cutter as well as by a dustpan-dredger would have to answer the question as to which dredger is more suitable for the dredging of crossings along the Río Magdalena. As the dredger should also be able to operate as a cutter-dredger, it is thought that a combination dredger (cutter-dustpan) is, based on the present available information, the best choice.

#### Ladder

As the dredger should be able to work at depths ranging from 1.50 m (crossing) to about 15 m, different lengths of the ladder are required.

Description of the optimal dredger for the Rfo Magdalena (not to be regarded as tender specifications)

Hull

The hull should preferably consist of one main pontoon and 4 side pontoons; when fully assembled, the main dimensions should be about: Length 60 m, width 14 m, and draught 0.90 - 1.20 m (3-4 ft).

The draught with full bunkers and provisions should in any case not be more than 4 ft. (The bunker capacity need not be very large, as in the complete dredging unit a fuel barge should be included). The main pontoon should, if possible, be able to operate without the side pontoons when dredging in narrow channels; in that case, of course, the draught will increase.

The dredger should be self-propelled. She should have a bow thruster or a "schottel" for dustpan-dredging purposes. The speed in still water should be about 7 knots.

Pump drive

The pump should be driven by a medium-speed diesel engine (maximum about 1,200 rpm), which can maintain a near constant torque under a wide range of conditions (revolutions). The rated continuous power under tropical conditions should be about 1,500 hp.

Dredge pump

A high-efficiency dredge pump (400 rpm); optimal discharge with a floating pipeline 500 m long. The pump should also be suitable for a 1,500 m long pipeline (partly landline, partly floating). The wearing parts of the pump should be of a high wear-resistant alloy. Spoil output (15% solids, 500 m pipeline) should be 2.4 m<sup>3</sup>/s.

Pipeline

A floating flexible pipeline is required with a length of 500 m, and provided with a baffle-plate installation. The diameter should be suitable for the above pump capacity (about 650 mm). Thickness of the pipeline should be about 8-10 mm.

Spuds

The two main spuds should have a length of 23 m, a diameter of 1 m, and a weight of at least 20 tons. Mounting one of the spuds on a spud-carrier should be considered; spuds and carrier hydraulically driven. A third spud is required on the forward part of the pontoon for dustpan-dredging.

Winches

There must be two main winches suitable for head wires when the dredger is used as a dustpan-dredger and for swing wires when it is used as a cutter-dredger. Constant line-pull of each winch: about 25 tons.

Speed of winches (in two steps): First step 0 - 18 m/min, and second step 18 - 24 m/min.

### III, 2.3

#### Ladder

A derrick hoisted dredge-ladder base is required suitable for mounting:

- Dustpan-head for dredging at depths between 4' and 10';
- a cutter-head suitable for dredging at depths between 5' and 20'; and
- a cutter-head suitable for dredging at depths up to 50'.

#### Anchor booms

The dredger should be equipped with anchor booms about 25 m long.

#### Cutter and jet-pump drive

On the ladder base an electric cutter motor (25 rpm; 600 hp) and a jet-pump for the dustpan should be mounted.

#### Dustpan/cutter-head installation

- A dustpan unit 9 m wide which can be connected to the ladder base.
- An angled down cutter-head (and suction tube) which can be connected to the ladder base and which is suitable for dredging at depths ranging between 5' and 20'. Cutter diameter, 2.50 m (600 hp; 25 rpm).
- Ladder (and suction tube) extension with the necessary connections to dredge at depths up to 50'.

In addition to the dredger, the following auxiliary craft will be required:

#### House-boat

As the dredger would not have sleeping accommodation, a house-boat should be available that can accommodate about 30 people.

#### Tug and work-boat

A tug should be available to move the dredger, handle the pipeline, tow the fuel barge, etc. In addition, a work-boat should be available provided with a derrick for removing snags, lifting a fouled anchor, and for the removal of floating debris which gathers in front of the pipeline. In still water, the speed of the combined fleet of tug, dredger and pipeline should be about 7 knots.

#### Fuel barge

In view of the required shallow draught of the dredger, only small bunkers (to work the dredger for about 10 days) are possible. So a fuel barge should be available with a fuel capacity for about one month.

#### Speed-boat

A speed-boat should be available for transportation, obtaining provisions, etc.

### 2.3.6. Organization of dredging

In Colombia the following agencies are concerned with dredging in the Río Magdalena and the Canal del Dique:

- MOP
- ADENAVI, and
- Junta del Canal del Dique.

There is no proper coordination between these agencies, nor is there a proper division of responsibilities. A study has been initiated to investigate the dredging in Colombia, and the Mission recommends that during this study the following points should also be investigated:

- How a separation can be obtained between the execution of dredging works and their control.
- How a proper preparation of dredging can be obtained.
- How a better motivation and stimulation can be obtained for dredge crews.
- How a more economic use can be made of the equipment by means of a proper job description, working under contract, etc.
- What spare parts should be available to ensure continuous dredging.
- How a reduction of crew can be obtained. It is thought that a dredger as described in Para. 2.3.5 should be able to work with a crew (two shifts) of: 1 Dredge-master, 2 Operators, 2 Engine-drivers, 10 Deck-hands, 4 men (including coxswain) for auxiliary craft, and 1 cook.

## 2.4. PERMANENT RIVER IMPROVEMENT

### 2.4.1. Introduction

In the foregoing paragraph only those methods for river improvement have been discussed which have a temporary character, mostly carried out at the end of the high water period to serve their purpose during the following low water period. The results of such improvements are often lost in the next high water period and therefore must be repeated every year.

Permanent river improvement, on the contrary, includes those methods of stabilizing a river which will serve their purpose for a number of years, such as, for example, the fixation of a river section consisting of different branches, or even a local bank protection. However, it must be stressed that although the works are carried out to serve for a number of years, yearly maintenance is often required to prolong their use.

Permanent river improvement includes all those measures connected with:

- Control of the discharge;
- control of the water-level; and
- regulation of the river-course.

In the next paragraphs only the last type of works will be considered, to be carried out to improve, locally, conditions for navigation. The first two types of improvement require such enormous investments that a more complex purpose needs to be served to be economically justified. Apart from navigation purposes, also irrigation, hydro-electric power-stations, protection against inundations, etc., have then to be served. Such improvements are,

therefore, outside the scope of the present Report and as, moreover, these purposes will be covered in greater detail in the "River Regulation and Flood Defence Study in the Basin of the Rfo Magdalena", only some general remarks will be made here.

#### Control of the river discharge

For those rivers with a great fluctuation in the discharge during the rainy season and the dry season, control of the discharge by means of a reservoir (or a chain of reservoirs) can be considered. During the rainy season part of the run-off is then accumulated in reservoirs to decrease the maximum river discharges in the wet season and to increase the discharge during the dry season. In case of discharge regulation also a change of the water-level will result, which often is of secondary importance. Of course, the possibility for the creation of such reservoirs is completely dependent on the topography of the river valley.

Apart from the general requirement that the construction and the cost should be economically justified, some requirements can be specified in a little more detail:

- Security is required of the construction of the dam, as well as the stability of the sub-soil and the banks, etc. Special attention should be given to the construction when there is the possibility of earth tremors. Emergency spillways should be constructed to prevent overflow of the dam, which would be likely to cause its collapse.

- To be economically justified, the lifetime of reservoirs should be at least 50 years. However, if the river discharges great quantities of sand and/or silt, the sedimentation in the reservoir may reduce its lifetime considerably.

- Leakage and evaporation are mostly not very important if the reservoir is only constructed for protection against inundations, or to increase the available water depth for navigation. However, if the dam is combined with a hydro-electric power-station, both leakage and evaporation give a loss in the possible generation of energy. Dependent on the topography of the river valley it might then be more economical to prevent the water-loss as far as possible; by a narrow and deep instead of a wide and shallow reservoir.

- The regime of the river downstream of the dam will change because the discharge frequency-curve will alter, and the supply of sediments from upstream is completely blocked. Downstream of the dam scour of the river-bed will occur which, however, will be limited by armouring the river-bed [40]. (Armouring means loss of the finer particles of the bed material which is gradually decreasing and even fully stopped by the protection of the remaining, coarser, particles). Moreover, often the plan-form of the river downstream of the dam will change because of the more constant discharge.

#### Control of the water-level

This method implies an artificial change in the water-levels of a river (a setting-up of the water) without a change in the discharge. If by the construction of weirs the discharge is temporarily changed, this is of secondary importance. The canalization of a river for navigation (the Meuse River) is a good example of this method. However, other purposes can also be served, e.g., irrigation, erosion prevention, regulation of groundwater, hydro-electricity generation, and to a less extent fishery and recreation.

The elevation of the water-level is possible by the construction of weirs, which not only set up the water upstream but still discharge water to the river section downstream. The discharge to the river section downstream can be done either over the weir, through (adjustable) openings discharging below the water-level downstream of the weir, or by a combination of both. In Figure 2.4.1 a schematized example is given for a river with a (constant) discharge of water and sediment in which the water-level is elevated by the construction of a fixed weir.

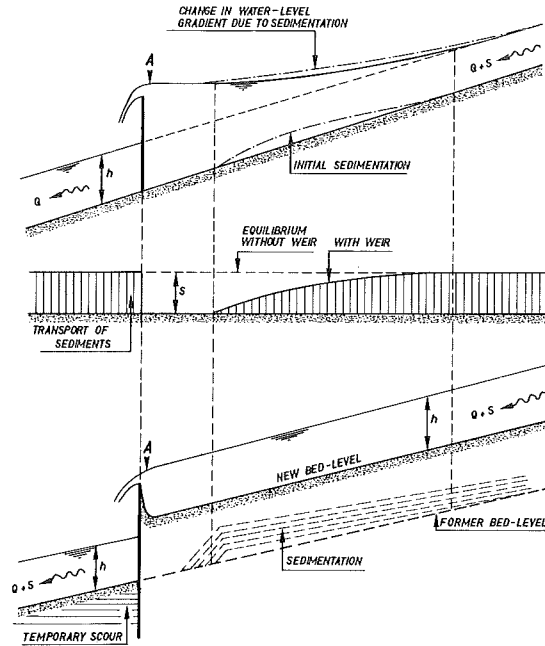


Figure 2.4.1 Influence of Weir on Elevation of River-bed

The setting-up of the water-level causes a decrease of the water-level gradient and the flow-velocity. The transport capacity upstream of the weir diminishes too, resulting in sedimentation. Consequently, the water-level gradient changes again until a new equilibrium is reached (bed-level and water-level parallel). The level of elevation (A) of the water, however, is constant. Below the weir scour occurs till the equilibrium-stage upstream of the weir is reached. From that moment onwards, the same quantity of sediment (S) is also transported downstream of the weir and sedimentation occurs until the former bed-level is reached. Figure 2.4.1 clearly shows that in the new situation no improvement has actually been obtained, because the setting-up of the water-level is nullified by an equal rise of the bed-level. In this simplified example the discharge of water and sediment was assumed to be constant. However, in reality both will vary considerably, resulting in a fluctuation of both the water-level and the elevation of the river-bed.

If only erosion purposes are to be served, the construction of such a fixed weir may be considered. For other purposes, e.g., irrigation or navigation, the sedimentation and the fluctuation of the water-level upstream of the weir (requiring higher dikes to prevent overflow) are strong disadvantages. As far as possible the sedimentation upstream must be prevented by discharging at least part of the sediments.

### III, 2.4

In this respect the project for the construction of a dam in the Rfo Magdalena near Honda for the generation of hydro-electricity should be mentioned. In the light of the foregoing remarks, the possible benefits for navigation or the prevention of inundations which should be served by the construction of this dam, must be regarded with suspicion. Locally, conditions will indeed change, but more downstream the effect of this dam will probably hardly be felt.

To justify the high cost involved in the canalization of a river, preferably a multi-purpose function should be served. For navigation alone, and specifically in view of the rather small cargo flow at present along the Upper Rfo Magdalena, such a scheme will not be feasible. In view of the considerable cargo flow along the Canal del Dique and the high yearly maintenance cost of dredging along the whole length of the Canal, the construction of sluices and, locally, raising the level of the banks may be a solution for the future (this is further elaborated in Chapter 4, dealing with the Canal del Dique).

#### Regulation of the river-course

After these general remarks about the methods to control either the discharge or the water-level in a river, the regulation of the river-course will now be discussed in greater detail. The regulation can be obtained in the plan-form: e.g., by forcing a movable channel or system of channels into one, stable, smoothly-curved channel; by short-cutting or widening of too sharp river-bends ; or by a bank-protection, etc. These methods are described in Para. 2.4.2.

When river-works are carried out along both banks a normalization of the river width will be attained. Some remarks about this are made in Para. 2.4.3.

The regulation of the river-course can also be carried out in a vertical plane by the removal of obstacles or sills, or by the fixation of the bed-level. These methods are treated in Para. 2.4.4.

#### 2.4.2. Regulation of the plan-form

The regulation of the plan-form can refer to the low water bed, the mean water-level bed, or the high water bed of a river. For obvious reasons, i.e., to attain a greater water depth by the concentration of the current into one, smoothly-curved, stable channel instead of a diverting current over various shallow channels, mostly only the low water bed is considered. Stabilizing a river-bed at higher water stages will generally not lead to too favourable a situation at low water stages, as may be seen in Figure 2.4.2.

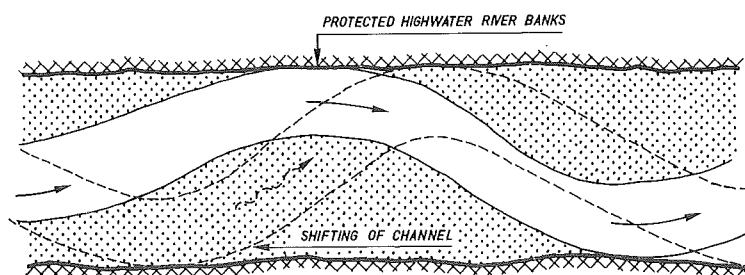


Figure 2.4.2 Mobile Channel in Stabilized High Water Bed

Stabilizing the river-bed by means of protected high water banks does not stabilize the channel as it can still freely meander in between the high banks. The purpose of the regulation is then not served. Even the solution to constrict the river-width by means of groynes will still not serve its purpose completely (see Figure 2.4.3).

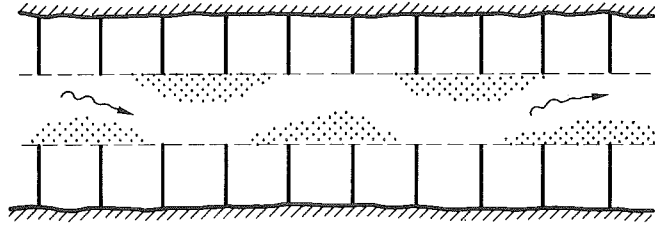
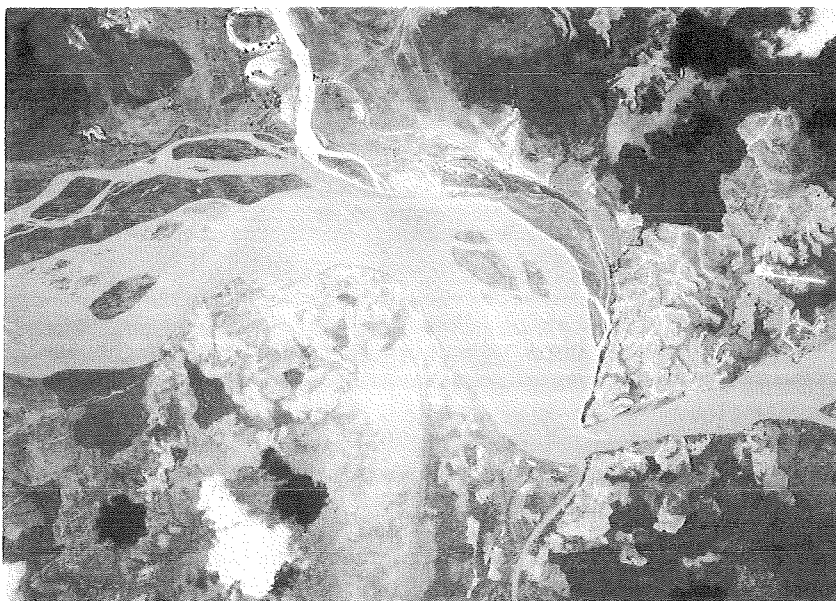


Figure 2.4.3 Constricted High Water Bed by means of Groynes

The constricted width of the channel at low stages tends to give a greater depth, and even when the sedimentation on both sides of the channel does not hamper navigation, a proper regulation is still not reached because the sand-banks will continuously propagate in a downstream direction. Moreover, this normalization is extremely costly, as not only do both sides of the bed have to be defended, but also the periodical scour or sedimentation in front of the groynes demands a heavy protection to a safe foundation depth.

If in a river section the cross-sectional area varies greatly, resulting in heavy sedimentation at high water stages due to the local retardations and accelerations of the flow, the regulation of the high water bed of the river can have great advantages. An example of this can be found at the confluence of the Rfo Magdalena and the Rfo Regla, where downstream of the confluence the width of the river is restricted (where the Rfo Magdalena is called the Rfo Nuevo) due to the rocky banks, while upstream the river is wide, consisting of various channels constantly changing their course so that alternating sedimentation and scour hamper navigation greatly.



What is actually meant by a regulation of a river section is graphically shown in Figure 2.4.4.

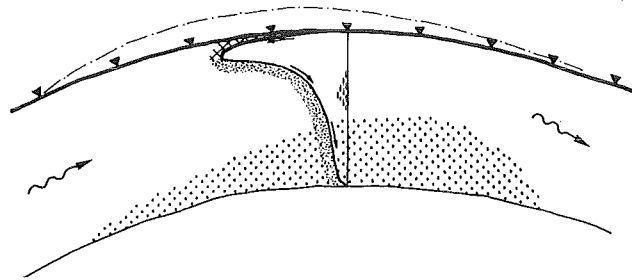


Figure 2.4.4 Principle of River Regulation

The helicoidal flow (also called spiral flow) in a river-bend will cause erosion of the bed along the outer bank and of the bank itself and sedimentation along the inner bank. By the construction of a bank protection or a number of groynes along the outer bank, the erosional forces of the helicoidal flow are arrested and the river-bend remains stable. It is very important to choose the radius of the curves properly. If too great a radius is chosen, the main channel may not follow the protection, while too small a radius implies not only that a greater length of river banks will have to be protected, but that the radial cross-slope of the river-bed will also increase, thus necessitating a protection to a greater depth. Moreover, the increased number of bends may hamper navigation. In Part II, Para. 3.7.3 an equation (3.7.10) was given to compute the radial cross-slope in a river bend with a fixed, protected outer bank. With an estimated depth in front of the protection and a radius of the curve (see Para. 3.7.2) the cross-slope, and thus also the width available for navigation, can be computed.

When considering the regulation of a river section it should be tried as far as possible to design the river-works in accordance with the occurring pattern of channels and shoals. This is not only to make the human intervention as small as possible but also to diminish the quantities of sediments which have to be removed either artificially (dredging) or by the river itself, and finally to save on the cost of the investments. The regulation of two consecutive river-bends should, however, never be so designed that they are curving in the same direction (Figure 2.4.5).

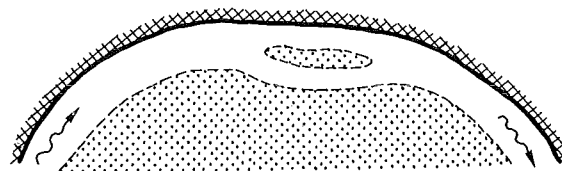


Figure 2.4.5 Regulation of Two Consecutive Bends Curving in the Same Direction

In the straight stretch in between the two bends the flow may show the tendency to separate from the outer bank, resulting in sedimentation or even in a new division of the channel (which is the very thing one is trying to prevent). This phenomenon may occur especially during a long-continued dry season when the natural radius of curvature of the channel is smaller (smaller discharge).

Consequently, the proper design of river regulation shows a series of curves in different directions, separated from each other by smoothly-curved or straight sections (Figure 2.4.6).

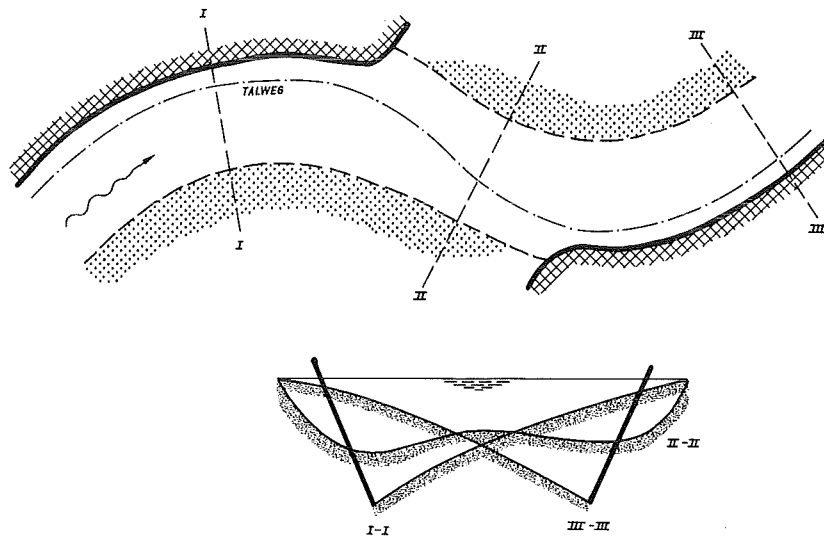


Figure 2.4.6 Consecutive River-bends and Cross-sections after Regulation

The crossing (cross-section II-II) always shows a shallower profile than the bends. For navigation purposes this least available depth (L.A.D.) should be as great as possible. The change from one bend to the other should, however, not be too abrupt. The usually accepted length of the crossing is in the same order as the river width. If the resulting depth on the crossing, however, is too small, improvement can be made by a continuation of the protection over the length of the crossing. Restricting the tendency of a river to increase the width on the crossing results in a deeper profile. (The next step may be to further restrict the width of the channel by the protection of the inner bends too; the effects of such a normalization are discussed in Para. 2.4.3).

As already explained, the aim of the design of regulation works should be to follow the course of the natural channel so as to make human intervention as little as possible. However, additional work will always have to be carried out; for example, if scour by the river itself is too slow. An example is given in Figure 2.4.7.

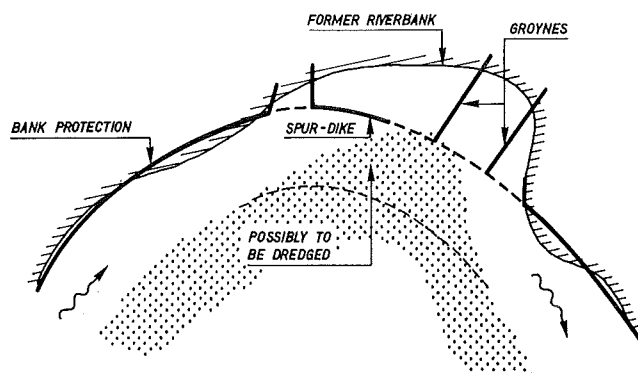


Figure 2.4.7 Combination of Regulation Works

The bank protection (schematically indicated in Figure 2.4.7) as far as it does not follow the former river bank, can either be made "in the dry" and the river will gradually erode the remaining bank in front of the protection, or can be made after the erosion has occurred. Part of the outer bend of the former course has to be filled up. A complete filling and protection will generally be too costly, and a solution with groynes or a spur-dike or a combination of both will likely be more economical. Care should be taken that the spur-dike is properly connected to the former river bank to prevent leakage. The shoal along the inner bend will be eroded by the river; to allow for navigation during the execution of the regulation works. This scour may be accelerated by means of dredging.

The option between the use of either spur-dikes, groynes or a bank protection depends entirely on the river topography. The best (most economical) solution is found by comparing cost. In those countries where labour costs are high, the most economical solution tends more and more to a mechanization of the construction and hence to, e.g., protections over great lengths of the banks. However, if labour charges are relatively cheap, groynes and spur-dikes may well be considered.

The improvement by means of the regulation of outer bends alone will not always lead to a favourable result. Only in the case of a meandering river, the low water bed of which consists of a series of curves separated by more or less straight sections in which the current crosses from one bank to the other, may such improvements be sufficient. If, however, the low water bed shows a more complex pattern of a meandering main channel in a relatively wide bed with secondary channels, or a great number of branches meeting and dividing again with islands in between (braiding), the fixation of the main channel may have to be supported by the closure of one or more of the secondary branches. Such an example is given schematically in Figure 2.4.8.

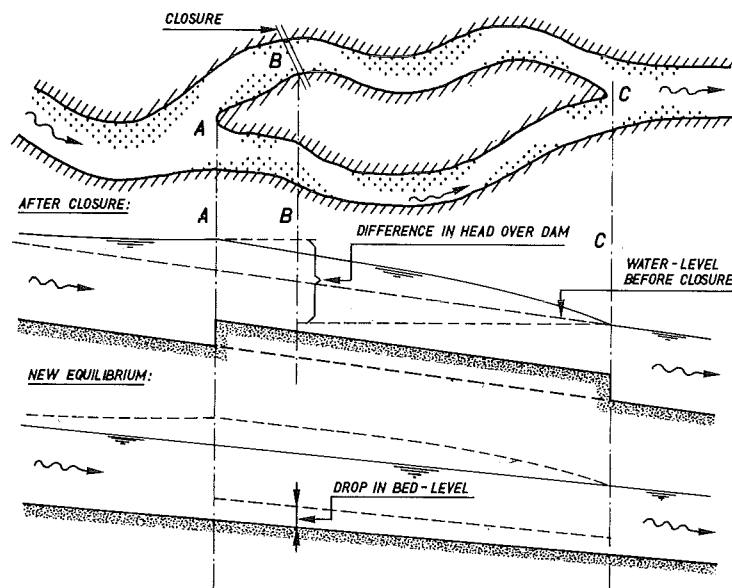


Figure 2.4.8 Effect of Closure of Secondary Branch

The capacity of each of the two branches is smaller than of the undivided channel and, theoretically, the available depth for navigation in both the branches is also smaller. However, it must be stressed that the local conditions at the division of the channel completely govern the behaviour of the two branches downstream. The sand-banks in the main channel may at one time propagate in front of the mouth of one of the two branches which will generally lead to a deterioration of this branch (increase of the sand supply and heavier sedimentation), while the other branch may improve. However, the passing of a new flood-wave may again change the conditions at the division of the channel, reversing the process of deterioration and improvement of both branches. Consequently, the navigable channel in such a region generally shows rapid changes, not only by a shifting of the course in one of the branches, but also by a change from one branch to the other. Unless very frequent surveys are carried out to mark the navigable channel, such a section produces great difficulties for the navigation.

Improvement can be obtained by the closure of one branch. The results of this intervention are also given in Figure 2.4.8. After the closure the discharge in the open branch increases and upstream of C the water-level gradient has to increase. The water-level at C does not change if the conditions downstream are not altered. The increased water-level gradient between A and C results in a higher water-level at A and a setting-up of the water-level upstream of A. Consequently, the smaller flow-velocities upstream of A may well lead to sedimentation, while the increased velocities between A and C will result in scour. This increase in sediment transport will again produce sedimentation downstream of C. The scour between A and C may take place either along the bed or along the banks. During low water stages the scour will mostly occur along the bed, while at higher water stages the banks are often more eroded (unless protected or erosion-resistant). If the particular aim is to increase the depth, initiation by means of dredging is recommended. When the new equilibrium stage is reached and the gradients of the water-level and the bed-level are again parallel, the bed-level between A and C will be lower. In Figure 2.4.8 it was assumed that the initial width of the open branch was equal to the width of the undivided channel; otherwise, a discontinuity in the bed-level is found.

As just explained, the abrupt closure of a branch will lead to a deterioration of the channel downstream until the new equilibrium stage has been reached. Such unfavourable results may be prevented if a more gradual closure of one of the branches in the course of the years is reached by the action of the river itself. By means of dredging, panning, or a combination of both it may be possible to change the local conditions at the division in such a way that an increase of the sand supply to one branch will automatically lead to its gradual deterioration, while in the other branch the deficit in sand will be undone by scour of the bed or the banks. However, if the river section upstream of the division is not yet regulated, a change of the conditions at the division of the branches may overrule the human intervention and, eventually, the improvement which one is trying to obtain. For the prediction of such changes surveys should be regularly carried out so as to be able to adjust the intervention as far as possible. However, some failures will have to be accepted as well.

In Figure 2.4.8 also the difference in head over the dam (at B) is indicated. More information about this is given in Chapter 2.5, where constructions are dealt with.

It has already been mentioned that every effort should be made to make the human intervention as little as possible. This means that the regulation of a certain river section should be carried out only gradually over a number of years, especially if a section of a number of branches separated by islands is concerned. If such an improvement would be carried out in one low water season (if at all possible by technical means) apart from all construction works also the complete dredging of the main channel would be required because the capacity would be too small to transport the quantities of sediment which had to be removed. Consequently, if the channel was not dredged completely, the constructed regulation works would produce such a high resistance in the following high water season that they could be damaged severely. This is explained by the example given in Figure 2.4.9.

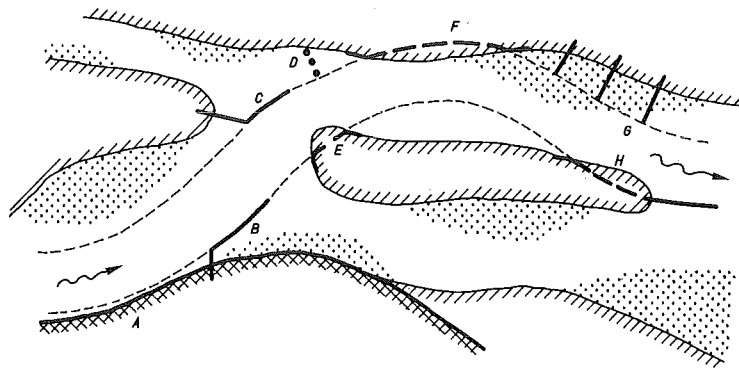


Figure 2.4.9 Phasing of Regulation Works

The closure of the secondary branch can be started by constructing the spur-dike at B (considered as a continuation of the erosion-resistant river bank at A). The crown of the spur-dike must not be too high and the secondary branch must be only partly closed. In this way the secondary branch remains open during high water stages to discharge water. The higher resistance of this branch will result in a gradual sedimentation, whereas the main channel will scour. The scour of the banks at E, F and H will be done by the river itself if sufficient guidance of the flow is provided by constructing the spur-dike at C and the groynes at G and, possibly, at D. Such a scheme may be executed in the course of years, although it must be sufficiently flexible to be revised if meanwhile the river course changes.

A great intervention in the river course will also occur if a meander in the low water bed has to be improved by making a short-cut through the high banks. Such an improvement may be required for navigation purposes, or to improve the discharge capacity at high water stages (Figure 2.4.10).

Again the possibility exists to guide the current sufficiently so that the new channel will partly be scoured by the river itself, or to dredge the short-cut completely. In the first case, a small pilot channel should be dredged. The shorter length (greater water-level gradient and velocity) will induce scour of the short-cut while the old channel

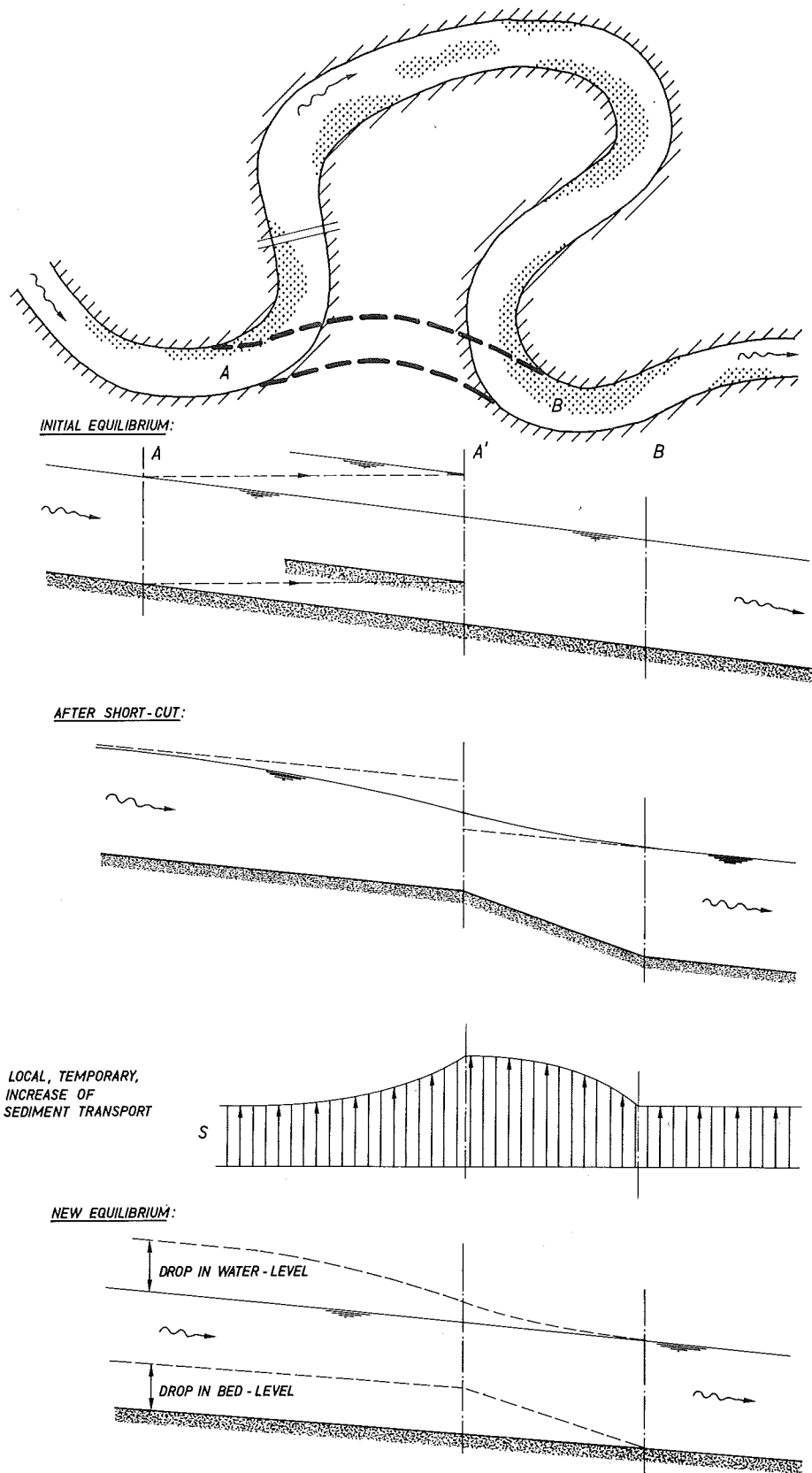


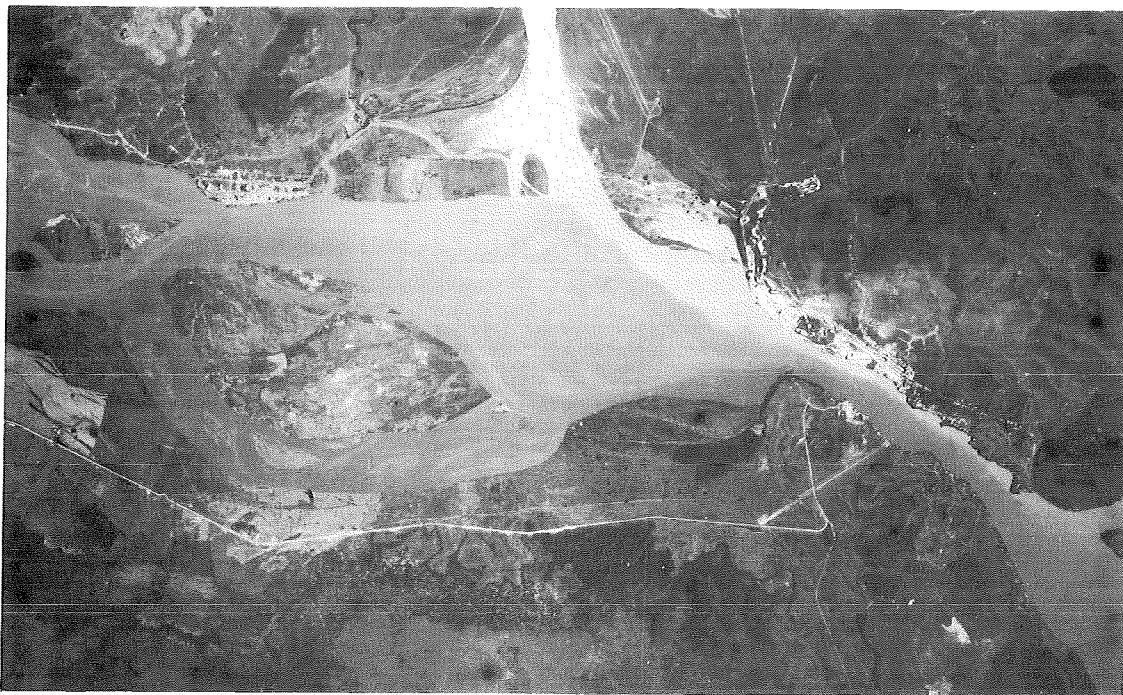
Figure 2.4.10 Short-cut of Meander

### III, 2.4

is silting up. However, if the improvement is carried out for navigation purposes, such a scheme is not to be recommended, because of the sedimentation in the old channel at a moment that in the short-cut still insufficient depth will be available for navigation. Moreover, the sediments picked up in the short-cut may settle again in the navigation channel more downstream of the intervention and thus hamper navigation even more. Therefore, the second solution will be better: to dredge the short-cut completely and, at the last moment, cut through the remaining dam and close the former course. The results of this intervention are shown graphically in Figure 2.4.10.

The shortening of the channel will result in a drop of the water-level upstream while the consequent increase in the transport capacity will lead to a lower bed-level. When the new equilibrium stage is reached, the gradients of the water-level and bed-level will be parallel again. Until this moment sedimentation in the improved river section will have to be accepted.

Especially if it is necessary for a number of loops to be cut off to improve the river, the drop of the bed-level and the water-level and, consequently, of the groundwater-level as well, may cause damage to river-works or agriculture situated upstream. The drop of the groundwater-level which is still acceptable will determine the extent to which the river can be shortened.



Rfo Nare Confluence

A last example of irregularities in the course of a river which, at one time or another, may demand regulation is the abrupt widening or narrowing of the river. By studying the Rfo Magdalena from aerial photographs or topographical maps such examples can easily be found, e.g. the confluence of the Rfo Magdalena and the Rfo Nare. In Figure 2.4.11 an example is given schematically.

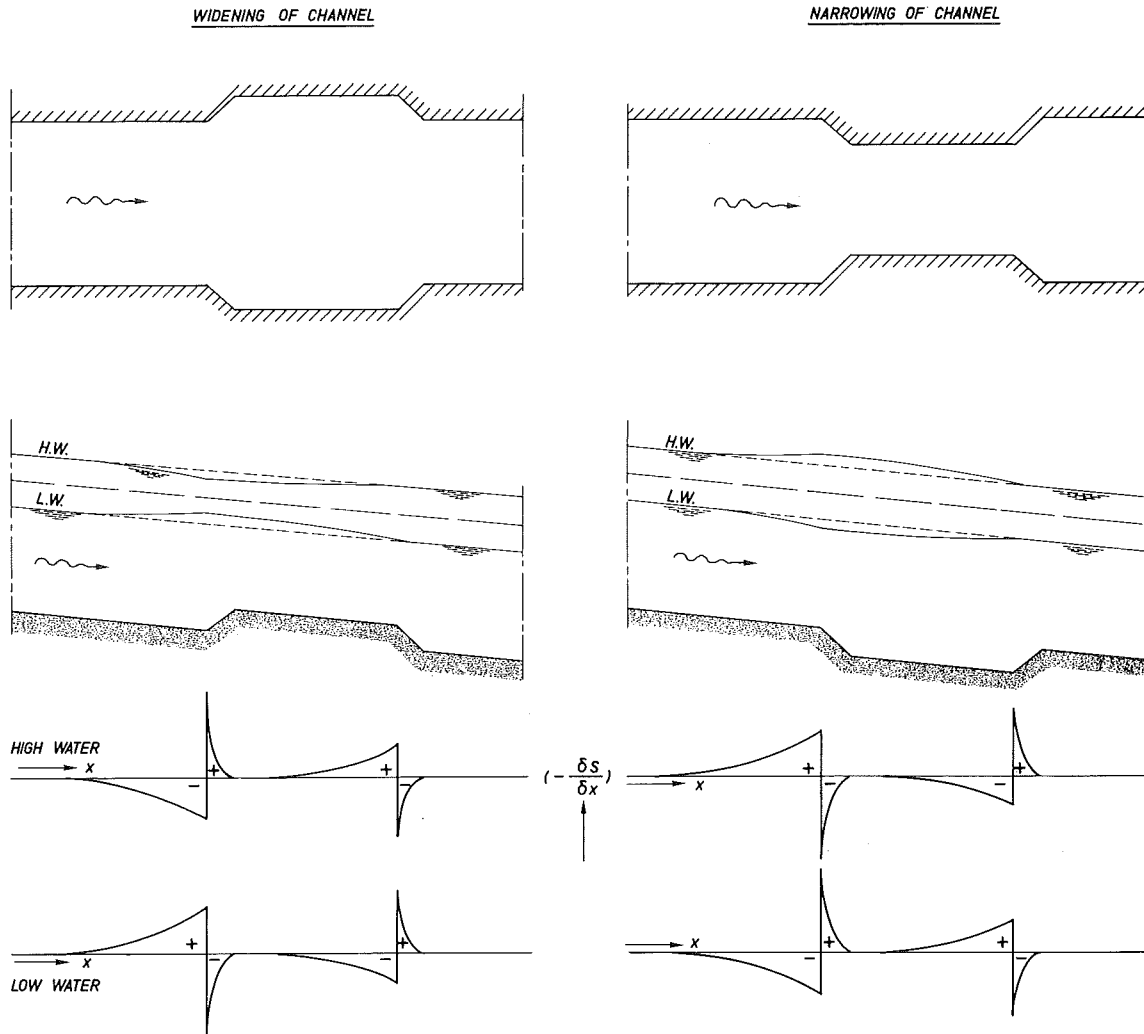


Figure 2.4.11 Widening or Narrowing of River

In the equilibrium stage (bed-level and water-level parallel) it can easily be seen that the water depth in the wider cross-section must be smaller and in the narrow cross-section greater than in the main channel. At water stages higher than the equilibrium stage, the increase or decrease in the flow-velocities will lead to scour and sedimentation as indicated in Figure 2.4.11. It can also be seen that at low water stages the backwater-curves are the opposite of those at high water stages, as well as the locations where scour and sedimentation occur. In the equilibrium stage, the discontinuity in the bed-levels will be evened out after the normalization of the width has been carried out.

2.4.3. Normalization of the river width

It has already been mentioned that the regulation of two consecutive outer bends of a river (Figure 2.4.6) still leaves a transition zone in between the bends in which the main current crosses from one bank to the other. Unless counter-measures are taken, the width on the crossing will likely be greater than in the bends and the available depth smaller. Improvement for navigation can be obtained by restricting the river width over the crossing (Figure 2.4.12).

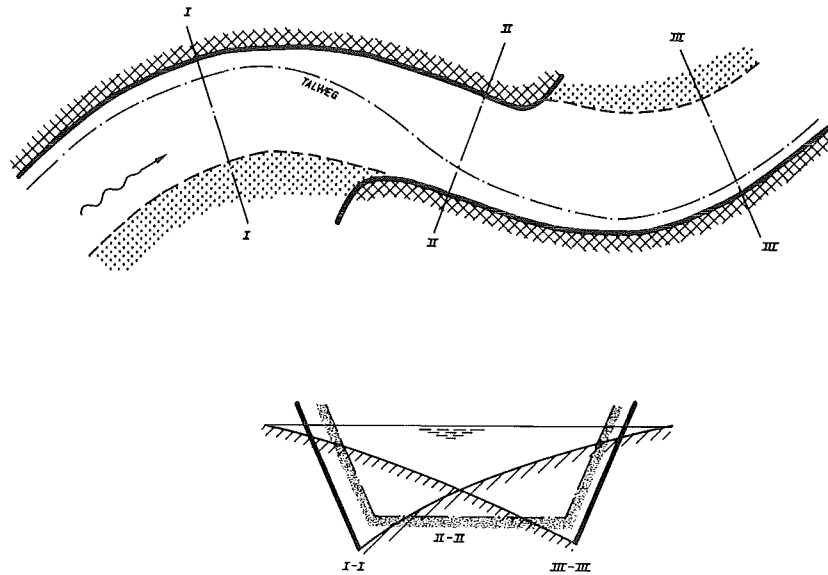


Figure 2.4.12 Normalization of Crossing

It may also happen that the regulation of the outer bends still does not lead to a sufficient improvement for navigation. The helicoidal flow in the river bend results in a radial cross-slope of the bed and it is possible that the required water-depth will not be available over sufficient width. (For the relations of the drift-angle, the speed of the vessel and the bend-radius, reference is made to Para. 4.2.3).

The best way to obtain an increase of depth in a river bend over a greater width of the navigable channel will be by increasing the radius of the curve, as a greater radius will result in a smaller cross-slope of the river bed. However, this solution will not always be feasible if there are considerable investments alongside the river. Another solution may be, to even out the bed-level by filling in the deepest part of the channel and to protect the heightened bed against scour. However, this will not always result in a wider channel; see Figure 2.4.13.

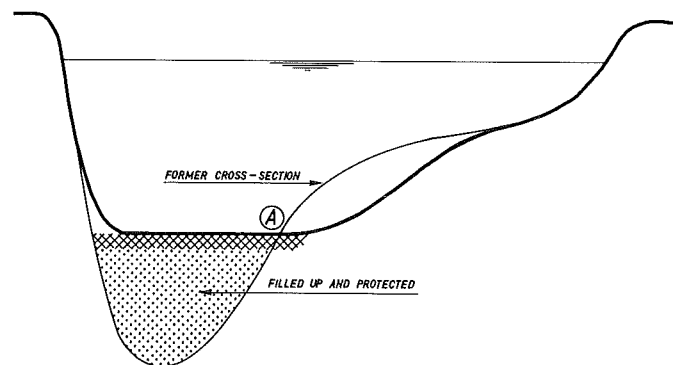


Figure 2.4.13 Widening of a Triangular Cross-section

The increased flow-velocities result in an increase of the transport capacity. However, the load to be transported per unit of width also increases because the heightened bed will have a smaller wetted perimeter than the original profile. The helicoidal flow in the bend

concentrates the load near to the inner bend and when the supply exceeds the transport capacity the widening of the channel will not be reached. Moreover, the required keel-clearance determines the possible heightening of the bed (indicated in Figure 2.4.13 by A) and although the width of the cross-section apparently increases, the required widening of the cross-section may still not be obtained.

A constriction of the river width in the bend by the construction of groynes alongside the inner bend (Figure 2.4.14) may also be considered to provide for an increase of the depth over a greater width. However, to a lesser extent, the above remark has also to be taken in consideration and only accurate model studies can show the feasibility of this method.

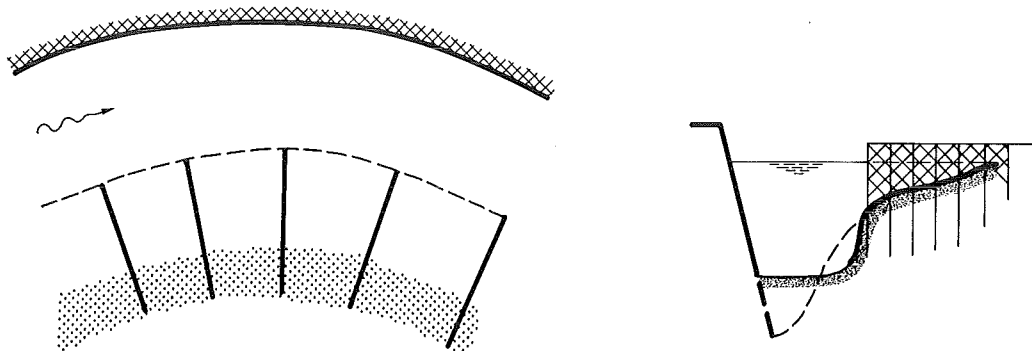


Figure 2.4.14 Normalization of a River-bend

A first impression of the required bed-level can be obtained by applying Chezy's formula:  $Q = Bh^{3/2} CI^{1/2}$ . Assuming, for the time being, the values of  $Q$ ,  $I$  and  $C$  to be constant, the value of the conveyance ( $Bh^{3/2}$ ) of the profile must also be constant and the width of the river which results in the required depth can be calculated. Of course, such an estimate will be a very rough one and the following remarks have to be considered carefully:

- To start with, a steady flow has been assumed. In reality, both the discharge and the sediment transport will fluctuate considerably and, consequently, the water-level gradient will not be constant either.

- If the groynes are constructed in the low water bed of the river only, and will be flooded during high water stages, this formula can be applied by the substitution of average values for  $Q$ ,  $I$  and  $C$  during low water stages. On the other hand, if the groynes are constructed in the high water bed of the river, the new bed-level can only be found by introducing the regime of the river over the year. As calculations of such problems can at present only be carried out as one-dimensional schematizations, and the helicoidal flow in a river-bend greatly determines the bed-level, the best solution is obtained by means of model tests.

- If the width of the normalized channel is narrow compared with the total width of the river, long (and expensive) groynes are required to prevent leakage.

- If the discharge of the river is small and the required depth of the navigable channel results in a too narrow width to provide for safe manoeuvring by navigation, normalization will not be possible. The only solution will then be to set-up the water-level in the whole river by the construction of weirs combined with ship-locks. However, for the Río Magdalena such solutions need not be considered, because there is sufficient discharge.

#### 2.4.4. Regulation of the river-bed

In the foregoing paragraphs solutions concerning the regulation of the plan-form of an alluvial river have been considered aiming at a greater depth for navigation. It is also possible that navigation in a certain river section is hampered by irregularities in the river-bed itself, e.g., rock-sills. To improve navigation the removal of such obstacles may be considered, and in the case of only a small obstacle partly blocking the navigable channel, the removal will have hardly any consequences for the bed-level upstream or downstream. However, in the case of a sill over the full width of the river, removal by dredging or blasting will have far-reaching consequences; see Figure 2.4.15.

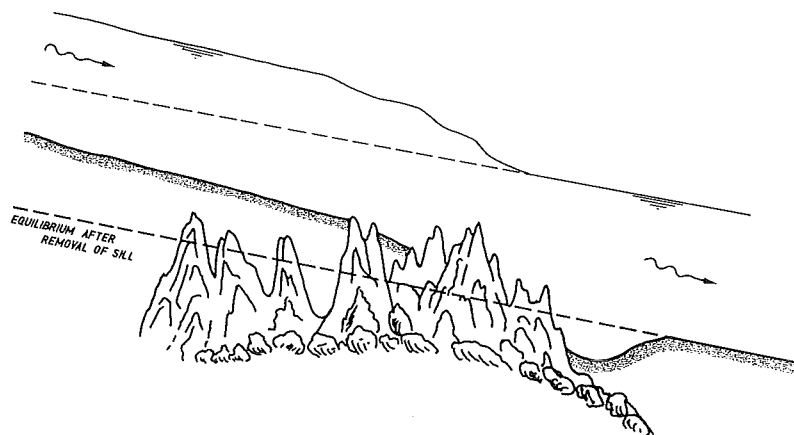


Figure 2.4.15 Removal of Sill

Due to the loss in energy-head over the sill, the water-level and the bed-level upstream are higher. Removal of the sill results in a greater water-level gradient, and the increase of the flow-velocities leads to scour of the bed upstream of the sill. In the equilibrium stage the bed-level and water-level upstream will be lower. The lowering of the groundwater-level, with possible damage to constructions may be unacceptable. Due to the erosion of the upstream river section it may also happen that new and unacceptable rock-sills appear. Downstream of the improved river section a strong (although temporary) sedimentation must also be expected.

A better result is obtained by removing only part of the sill and constructing a spur-dike to even out the difference in head over a greater length and to provide for smaller flow-velocities (than the initial velocities over the sill) in the navigation channel; see Figure 2.4.16.

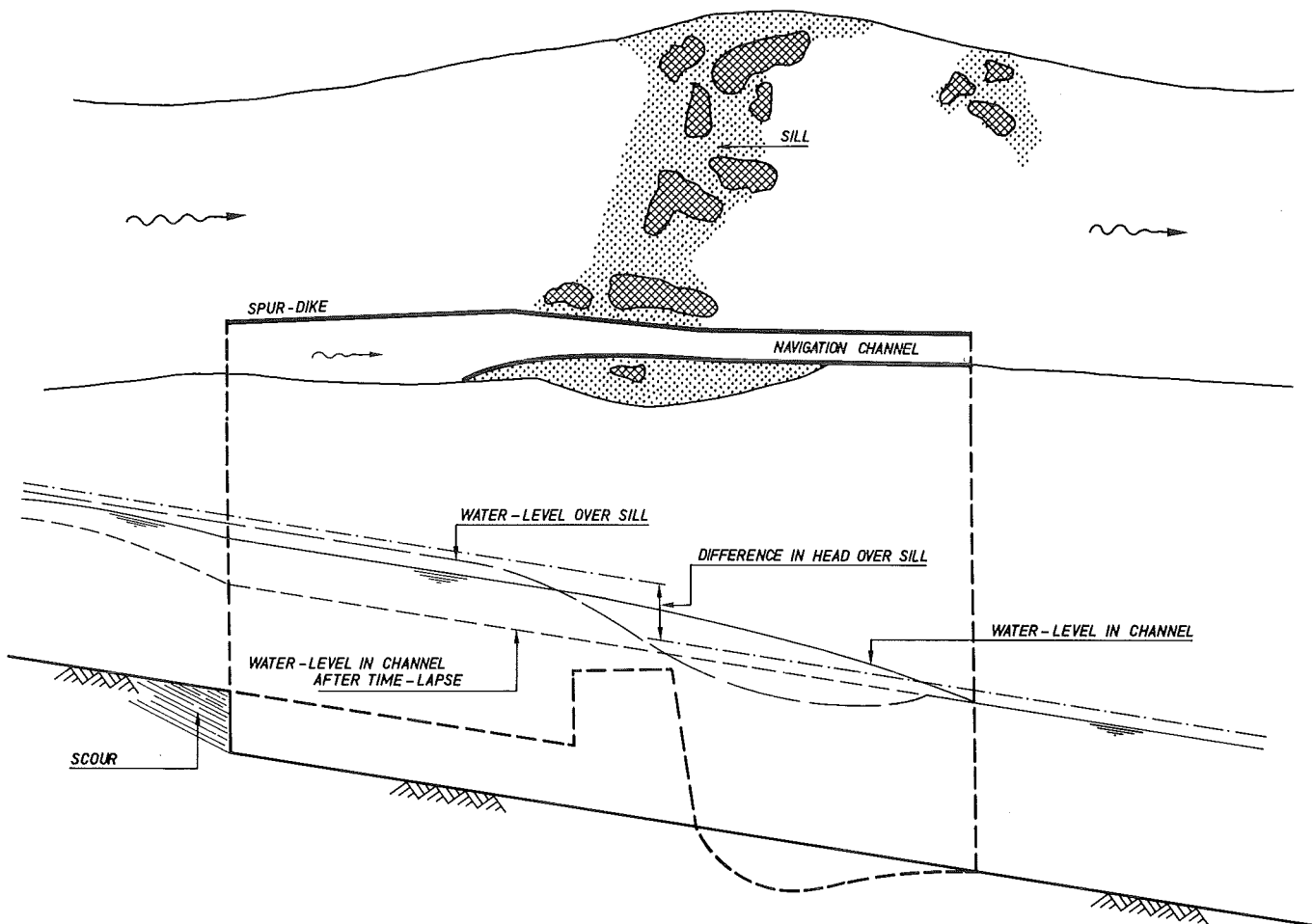


Figure 2.4.16 Navigation Channel through Rock Sill

The greater water-level gradient in the navigation channel results in an increased discharge, while the increase of the flow-velocities and the transport capacity leads to scour of the bed-level in the channel and upstream of the spur-dike in the river. Consequently, the water-level in the river upstream drops and the discharge over the sill decreases, while the increase of the discharge through the navigation channel results in a further scour of the bed. If not prevented by the presence of deeper-lying rocks, the bed-level upstream of the sill will lower over a height more or less equal to the initial difference in head over the sill, and the discharge of the river will finally be concentrated in the navigation channel. To prevent such consequences the river-bed in the navigation channel and just upstream of the spur-dike should be protected against scour. Moreover, the resistance in the channel should be increased to prevent a drop of the water-level upstream and an increase of the discharge through the navigation channel. Both purposes will be served by a bed-protection of coarse stones or the construction of a number of small underwater sills over the full width and evenly spaced over the full length of the navigation channel.

The fixation of the river-bed may thus be required to prevent scour. Such a protection may also be necessary in coastal regions where, for better accessibility for the sea-going vessels, deep navigation channels are required and maintained by recurrent dredging (Figure 2.4.17).

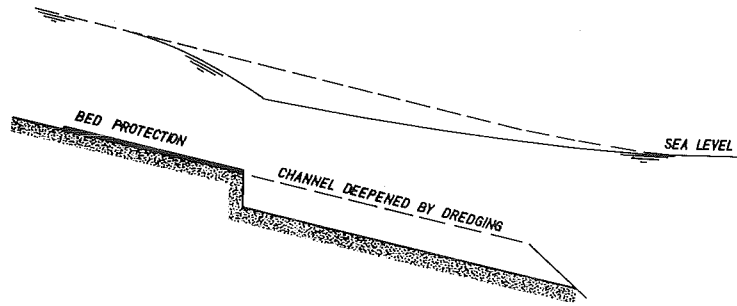


Figure 2.4.17 Bed protection in Coastal Regions

The great cross-sectional area in the access channel results in a smaller water-level gradient and if upstream the river-bed is not protected, the greater flow-velocities induced by the backwater-curve will lead to scour. The saltwater wedge will, consequently, penetrate further inland, which usually has to be prevented in the interests of vegetation, water-supply, etc.

The reverse occurs if the outlet of a river into the sea is narrowed by the construction of spur-dikes to prevent too heavy sedimentation in the access channel. Such an example can be found in Colombia in the outlet of the Rfo Magdalena into the Caribbean near Barranquilla (Bocas de Ceniza); see Figure 2.4.18.

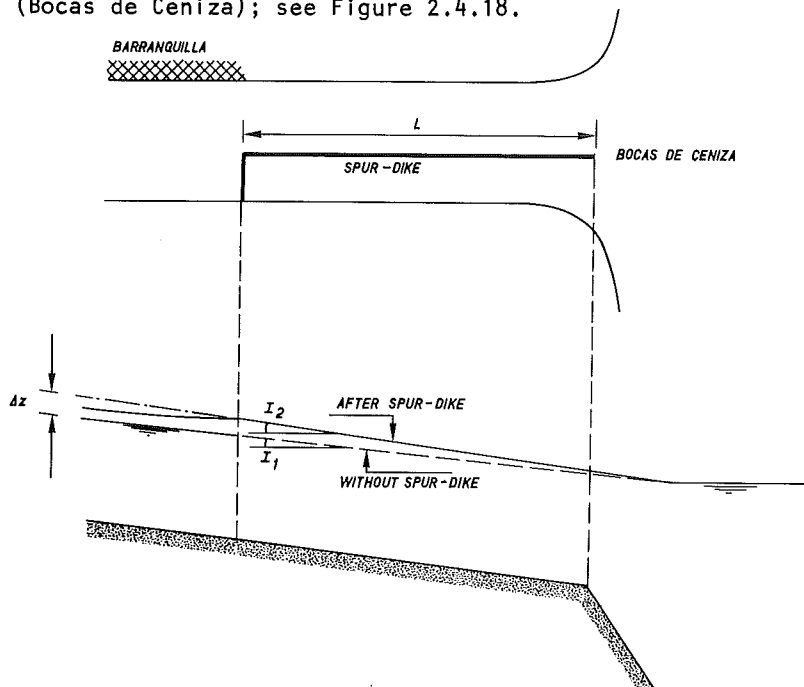


Figure 2.4.18 Schematization of Bocas de Ceniza

The smaller cross-sectional area in the river-mouth results in a greater water-level gradient, and the increase of the flow-velocities will lead to scour, while the set-up of the water-level upstream of the spur-dike results in sedimentation. This set-up is equal to the difference in water-level gradient before and after the construction of the spur-dike, multiplied by the length of the spur-dike. This is expressed in formula:

$$\Delta z = (I_2 - I_1) \times L.$$

Remark: As it is often heard that the water-levels in Calamar will never again reach the low levels of the past because of the river-works already carried out (and still to be carried out) in Bocas de Ceniza, it seems worthwhile to investigate the value of  $\Delta z$  in a little more detail. From Chézy's formula:  $Q = Bh^{3/2} CI^{1/2}$ , it can be seen that the change in the conveyance ( $Bh^{3/2}$ ) is inversely proportional to the root of the change in water-level gradient (assuming that  $Q$  and  $C$  will be constant). A reduction of the conveyance by a factor 2 (which means that the river has to be reduced to less than half the initial width), results in an increase of the water-level gradient by a factor 4. For a water-level gradient of  $2 \times 10^{-5}$  and a length of the spur-dike of about 1 kilometer, the value of  $\Delta z$  will only be in the order of 5 cm, which means that the influence on the water-level at Calamar will be nil.

## 2.5. CONSTRUCTIONS

### 2.5.1. Introduction

In the foregoing paragraph different types of constructions have been dealt with to design river-works, namely: groynes, spur-dikes, bank protection, bed protections, and structures for the closure of secondary branches. The first two types can be considered as dikes constructed to guide the flow: groynes perpendicular to the flow, and spur-dikes parallel to the flow. Both types often consist of a core of earth or gravel (dumped in bulk or in bags) and sometimes reinforced by means of an open piling construction or a sheet-piling, which in total provides the required stability of the construction. A cover prevents the loss of material by the flow. Although these constructions differ from a constructional point of view from the following two types (bank and bed protections), the latter also consist of layers which have to prevent the loss of bank or bed material and which are covered with (heavier) layers to provide for protection against attack by the flow. Therefore, the design criteria for these types of construction are about the same and will be treated together in Para. 2.5.2.

The closure of secondary branches can often be attained by means of dredging. Depending on the topography, a protection may be required. More about this follows in Para. 2.5.3.

A few types of construction have recently been built in Colombia in the Rfo Magdalena and the Canal del Dique. In the Rfo Magdalena the constructions were mostly built to provide for protection of the banks, while in the Canal del Dique mostly navigation purposes were served. Together with a review of possible constructions for future river-works, these types of constructions are discussed in Para. 2.5.4.

A general remark must still be made. The structure of a protection can be either: Open (permeable), or closed (impermeable). Generally, for a protection constructed to prevent erosion by the action of the river, only the open structure type should be considered. The structure needs to be open to prevent too great pressure differences on the protection, due to the (sometimes) rapid fluctuation of the water-level in the river and the more gradual change in groundwater-level in the bank. To resist these pressure differences an impermeable structure needs to be very heavy and the cost will in most cases not be justified. A vertical protection of the bank or a combination of a vertical protection at low water-levels and a protection

### III, 2.4

under slope at higher water-levels must only be considered for harbours or river-ports where ships are frequently mooring alongside for loading or unloading cargo. The high cost involved in e.g., sheet-piling (the impermeability of which and the great scour of the bed in front of the construction are disadvantages) justifies such a type of protection in specific cases only. Therefore, only open, permeable structures will be considered here (For impermeable constructions reference is made to the literature on soil mechanics [55]).

#### 2.5.2. Design criteria of a protection

The most important force acting on constructions is that of the current. To ensure the stability of a protection, the following requirements must be met:

- The bank or bed to be protected needs to be in equilibrium and not slip;
- the protection as a whole needs to be in equilibrium, also after settlements have occurred;
- the permeability of the construction must be maintained, while the pores in the construction should not be so great that particles from underlying layers can be washed away by the current (filter-layers); and
- the components of the layer on top of the filter should be stable.

A protection, however, does not only have to resist the attack by the current but must also be resistant against, e.g., chemical or organical influences. These requirements are further elaborated later.

#### Equilibrium of the bank or bed

In the literature on soil-mechanics [55] the principles are outlined regarding the equilibrium of a slope or bed and the possibilities of the occurrence of slips for different grain-sizes, slopes and pressures. Therefore, only some general remarks about the equilibrium of side slopes will be made here.

Regarding the equilibrium slope the following distinction can be made:

- Side slope above high water stages;
- side slope below low water stages; and
- side slope in the transition zone between high and low water stages.

The single particles of the soil material must not be washed away by seepage of groundwater. The weight of the future protection needs also to be considered, introducing an extra weight in the calculations.

When the slopes found from these considerations are more gentle than 1 : 3 and if the protection has to be carried out over a great height, the total cost of the protection will be high. In such cases the use of one or more berms may be considered, because a reduction in the weight of the top-soil increases the critical slope at which slips occur. Berms have the additional advantage that the protection of the underwater side slope can be secured (anchored) more easily, if mats of twigs or synthetic fibres are used (Moreover, in canals and coastal regions the run-up of waves and ship-waves will also be reduced).

The equilibrium of the side slope must also be considered after the protection has been installed. The extra weight of the protection resting on the sub-soil should not cause slips at the chosen slopes.

#### Equilibrium of the protection as a whole

The protection as a whole must be in equilibrium also after the occurrence of settlements in the sub-soil. Although such settlements should be reduced as much as possible (e.g., by consolidation of the soil), the structure of the protection should still be flexible. For a permeable protection the different components should be thick enough to guarantee the proper function of the underlying filter-layers (to be discussed later on).

Because of the scour in front of the protection, the termination at its toe should be flexible so that equilibrium will not be disturbed. This means that a bank protection should be continued on the river-bed (see Figure 2.5.1). It is also possible to protect the bank immediately to a greater depth by dredging a cunette (which can be filled up again after the protection has been installed).

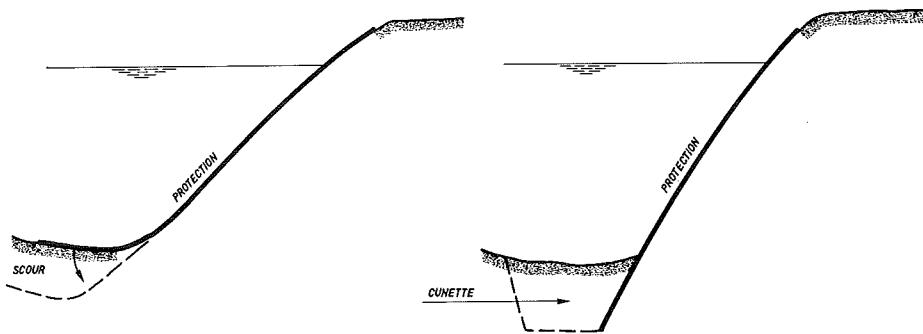


Figure 2.5.1 Termination of Bank Protection

An estimate of the depth in front of a bank protection can be obtained from computations (see Part II, Para. 3.7.3). Carefully conducted experiments in a laboratory often lead to more trustworthy results. However, sometimes it will be unavoidable, although very expensive, to extend the protection downward at a later date; see Figure 2.5.2.

In the case of protection of groyne-heads the total protection will usually be installed right away, the more so if the groynes are constructed to guide in future the main current of the river. A heavier type of protection is often required than for the protection of a river-bank because the attack by the current and the scour in front of the groyne will likely be greater. The new equilibrium level of the river-bed in front of the groyne is difficult to estimate, but an estimate of the depth can be obtained by computing the depth in front of a continuous bank protection (Part II, Eq. 3.7.10). Depending on the distance between the consecutive groynes, an over-depth of the protection must be considered which may well be in the range of 20%-30%. A more accurate estimate of the scour in front of the groyne can be obtained from model-tests, but experience should also be gathered in the field. In this respect, it is recommended that the development of the river-bed in front

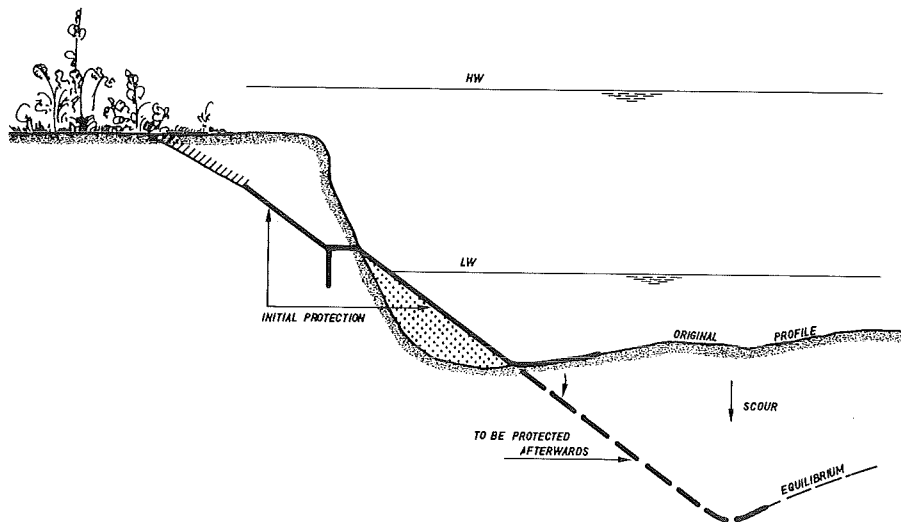


Figure 2.5.2 Protection of a River-bank in Two Phases

of the groyne recently constructed (1972) in Pto. Boyacá be closely followed by means of measurements (local soundings and flow-lines) taken at regular time-intervals (this should also be done when in future the main channel of the Río Magdalena follows the right bank along the protection of the Shell compound at Yarirí).

#### Permeability and prevention of loss of material

It has been mentioned that the protection should have an open structure to ensure that water can flow through it and that no great pressure differences over the construction are created. For example, during a rapid drop of the water-level in the river, the groundwater-level in the bank lags behind. To prevent great pressure differences over the protection, the groundwater should be able to seep through the protection to gradually diminish the difference between the water-levels on both sides of the protection. However, the pores in the protection should not be so great that seepage of groundwater can lead to loss of material, because the consequent settlements may finally result in the complete destruction of the protection.

On the other hand, the pores in the consecutive layers of the protection should not be so small that some loss of the material of the underlying layer (which always occurs) would obstruct the permeability. To ensure these two requirements (permeable construction and prevention of loss of material), a protection can consist of consecutive layers of coarse sand, fine gravel, coarse gravel, small stones and boulders. Each layer must be composed of material somewhat coarser than the underlying layer, while the final cover-layer must be so heavy that the attack by the current can be resisted. An example of such a construction is given in Figure 2.5.3.

Different formulae for the diameters and the thickness of the consecutive layers have (experimentally) been developed to ensure the permeability of such constructions and to prevent the loss of material from the underlying layers. However, such formulae are valid for a great range of conditions and therefore it seems appropriate to draw attention to the

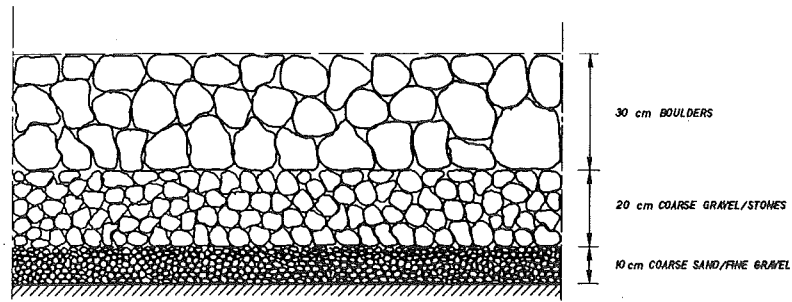


Figure 2.5.3 Schematic Example of a Protection

experimental data of the United States Corps of Engineers regarding bank protections of graded stones (also called riprap) along the Mississippi River. (For a review of the design criteria of protections and different types of constructions used along the Mississippi River, reference is made to, amongst other authors, Shen (1971) [56]).

#### General specifications

Thickness:  $0.25 \text{ m} \pm 0.05 \text{ m}$  and at least  $1\frac{1}{2}$  times the average stone-size (to be increased below the water-level).

Density:  $\rho_{\text{stone}} \geq 2,240 \text{ kg/m}^3$ .

Diameter of stones: smaller than 0.60 m.

#### Riprap above low water-level

Individual weight of stones: between 3 kg and 60 kg.

Approximate gradation: 35-60 kg 10% maximum

10-35 kg 40-60%

3-10 kg 20-40%

< 3 kg 15% maximum (greater than 2.5 cm).

Total weight of riprap:  $\geq 500 \text{ kg/m}^2$ .

#### Riprap below low water-level (applied on filter of, e.g., gravel)

Individual weight of stones: between 3 kg and 100 kg

Approximate gradation: 70 - 100 kg 5% maximum

60 - 70 kg 5-15%

35 - 60 kg 15-40%

10 - 35 kg 40-55%

3 - 10 kg 10% maximum

Total weight of riprap:  $\geq 800 \text{ kg/m}^2$

If such a type of construction has been chosen, the permeability can easily be tested by means of the model schematically given in Figure 2.5.4. By changing the total difference in head ( $\Delta H$ ), the time-intervals required for the adaptation of the pressures in the consecutive layers can be found. Consequently, the pressure difference on each layer is also known.

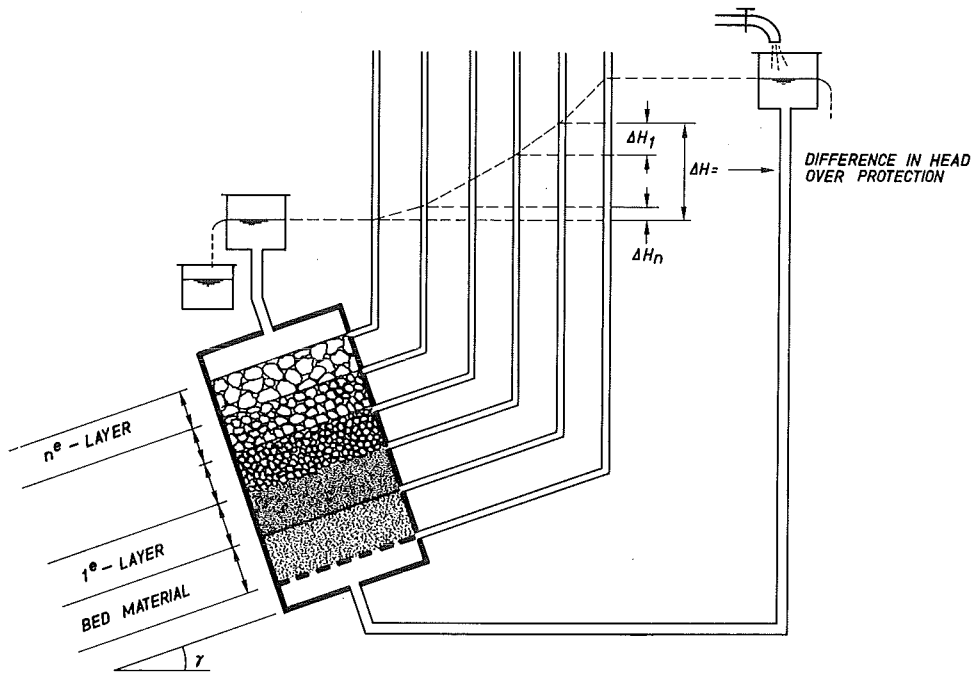


Figure 2.5.4 Testing Permeability of Protection

To prevent the obstruction of the filter by loose soil material, not more than 5% of the filter material should be smaller than 750  $\mu\text{m}$ . Moreover, the sieve-curves of the filter and soil material should be more or less parallel in the range of the small diameters (see Figure 2.5.5), because the angle between the curves influences the stability and the permeable function of the filter.

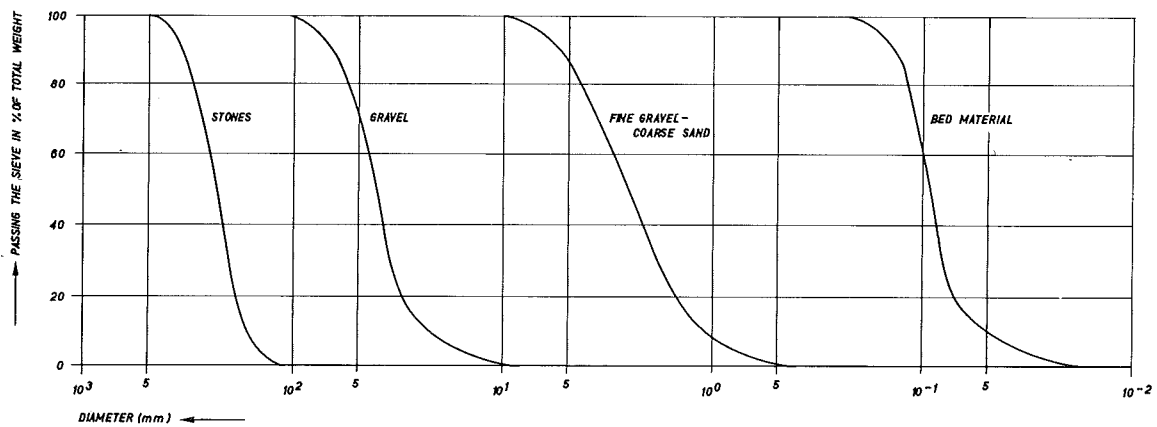


Fig. 2.5.5 Relation between Diameters of Filter Material

The type of construction presented in Figure 2.5.3 has two main disadvantages:  
 - A great number of layers have to be dumped to bring about an effective protection; and

- execution "in the wet" increases considerably the amount of material required for each consecutive layer. Due to the greater fall-velocity, the coarser particles will settle first, with the lighter particles on top, but as these are likely to be washed away by the current, consequently rather thick layers need to be dumped.

An attempt has been made to overcome these disadvantages by the use of other materials, bearing in mind, at the same time, the possibility of cutting costs. A number of such alternative solutions are schematically presented in Figure 2.5.6.

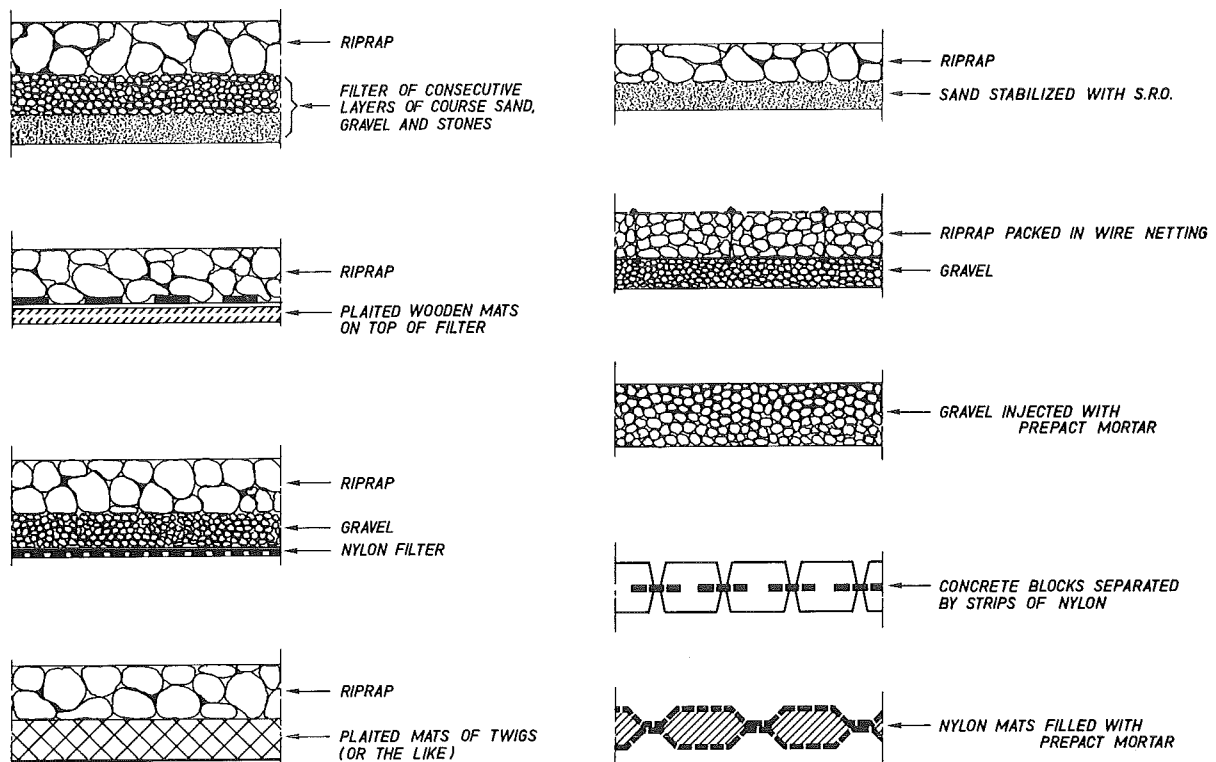


Figure 2.5.6 Examples of Permeable Protections

Stability of the cover

Cover-layers are required on top of the filter, and these must be heavy to provide resistance against attack by the current (and/or waves). In the case of a permeable protection the single components of the cover must be so heavy that equilibrium is not disturbed. (The components of the cover of an impermeable protection can generally be smaller and less heavy, because by means of, e.g., asphalt-penetration, they adhere to greater and heavier units).

- The stability of the single components of the cover can be disturbed in three ways:
- When the hydrodynamic force ( $F: \frac{1}{2}\rho v^2$ ) acting on a component which is placed in the current exceeds the internal friction induced by the neighbouring components, the single component can slip.

III, 2.5

- Due to local turbulences in the water, pressure differences are generated between the bottom and top of the components. When this lifting force exceeds the gravity force, the single components can be lifted from the protection.
- When the momentum of the hydrodynamic forces in respect of the downstream edge of the component exceeds the counteracting momentum of the gravity force, the single component can roll down.

Obviously, the stability of the components depends on their shape, their weight, the internal friction induced by neighbouring components, the slope of the sub-soil and the direction of the current. The diameter (D) of the components required to ensure stability when placed on a horizontal bed can be expressed as:

$$D \geq \frac{\alpha}{\Delta} \frac{v^2}{2g} \tag{2.5.1}$$

in which:  $\alpha$  = coefficient.

For example, tests carried out by the United States Bureau of Reclamation have shown that for strongly turbulent flow, equilibrium is observed for  $\alpha \approx 1.4$ . For different values of the relative density ( $\Delta$ ) of the components and the factor  $\alpha$  of Eq. 2.5.1, the relation between the flow-velocity and the required diameter of the components is represented in Figure 2.5.7.

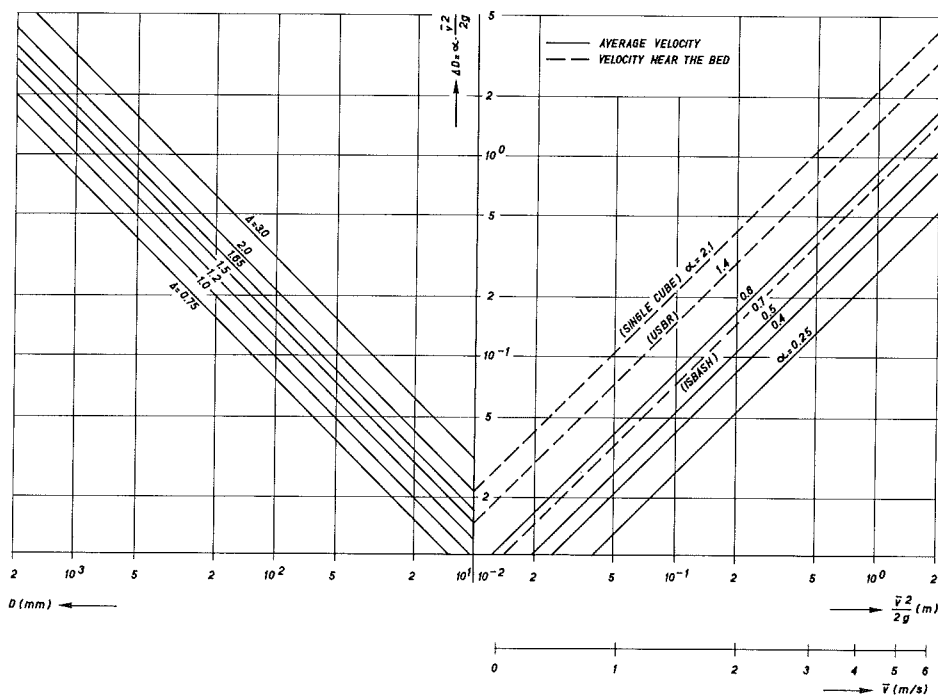


Figure 2.5.7 Required Dimensions of Stones

Stones are usually specified in kg-weight, instead of in kg-mass in accordance with the dimensions used in the formulae. Consequently, in the formulae also the specific weight ( $\gamma = \rho g$ ) should be introduced. Estimating the average weight of a rough-edged stone to be:  $G = 0.5 \rho g D^3$ , Eq. 2.5.1 yields:

$$G \geq 0.5 \rho g \left( \frac{\alpha}{\Delta} \cdot \frac{v^2}{2g} \right) \tag{2.5.2}$$

For different values of the relative density ( $\Delta$ ) the relation between the diameter and the mass of the stones is given in Figure 2.5.8.

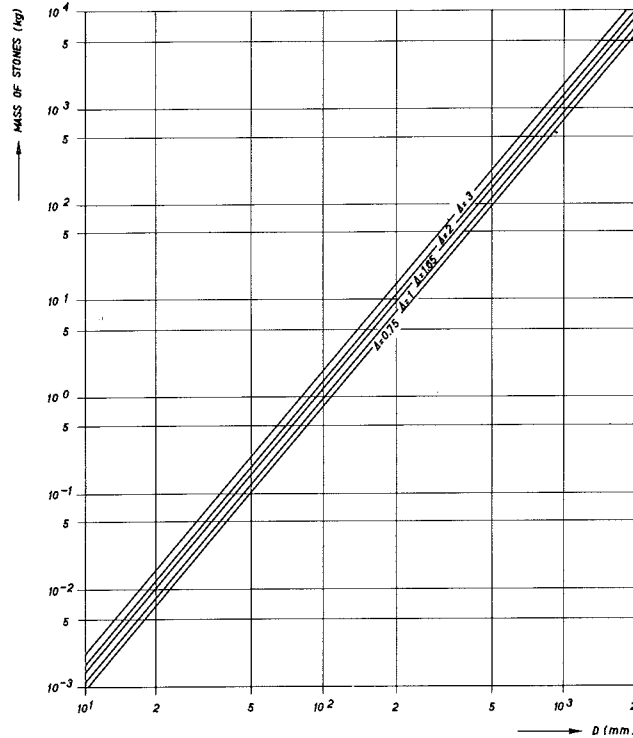


Figure 2.5.8 Relation between Diameter and Mass of Stones

It must be remarked that the diameter (mass) of the components can often be reduced by the use of specially-shaped units. For example, tetrapods can be interwoven to form greater units so that the stability of the single component can hardly be disturbed.

When the protection of a river bank is considered, the components of the cover have to be placed on a side slope. Consequently, the stability of the single component of the cover is also influenced by the component of the gravity force directed downward along the side slope. An increase of the diameter of the stones is then required, which can be expressed by introducing a correction-factor ( $f$ ) in Eq. 2.5.1 (an equation derived for a component placed on a horizontal bed):

$$D \geq f \frac{\alpha}{\Delta} \frac{v^2}{2g} \tag{2.5.3}$$

The value of the correction-factor depends on the angle of the side slope ( $\gamma$ ), the angle of internal friction ( $\phi$ ) and the direction of the current. In the case of flow parallel to the side slope the value of the correction-factor ( $f_1$ ) is given in Figure 2.5.9.

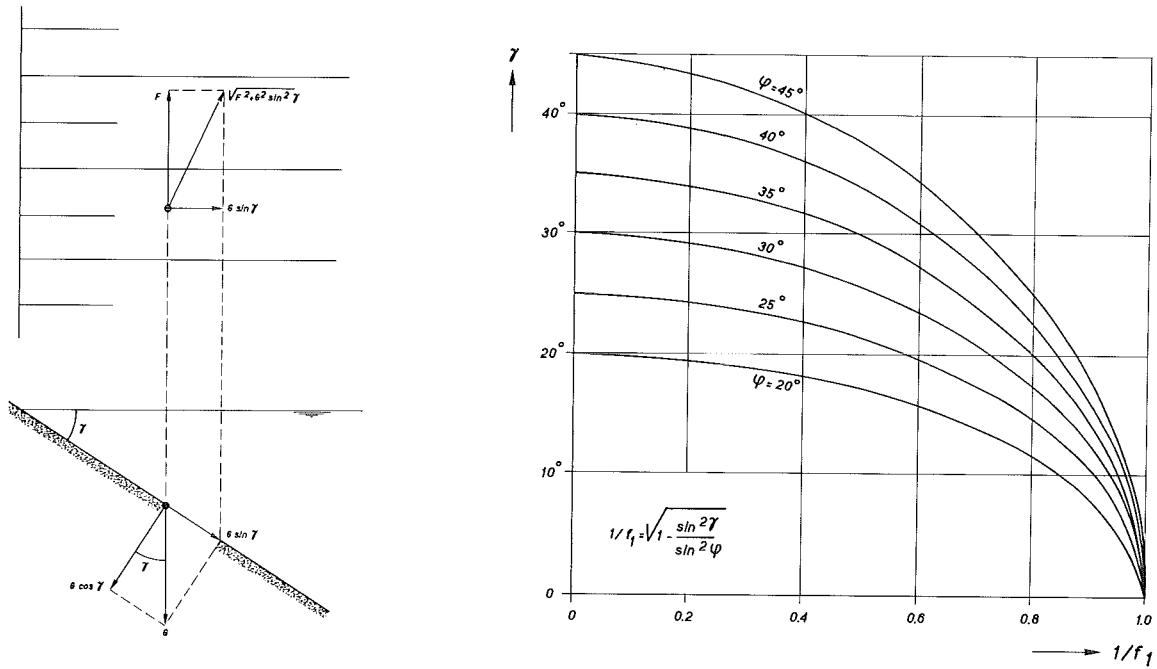


Figure 2.5.9 Correction-factor ( $f_1$ ) for Flow Parallel to a Side Slope

In the case of flow parallel to the side slope but perpendicular to the length axis of the side slope (for example, in case of overflow over a dam), the value of the correction-factor ( $f_2$ ) is given in Figure 2.5.10.

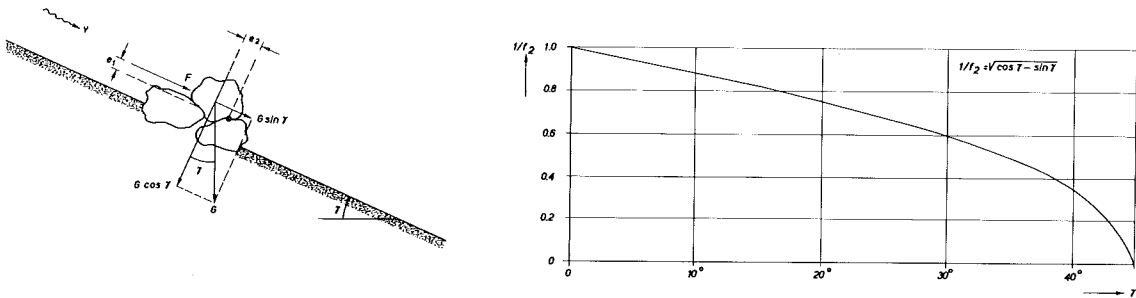


Figure 2.5.10 Correction-factor ( $f_2$ ) for Flow Perpendicular to a Side Slope

The helicoidal flow in river-bends introduces both a tractive force ( $F_1$ ) on the component of the cover parallel to the side slope and a tractive force ( $F_2$ ) perpendicular to the length axis of the side slope (see Figure 2.5.11).

Consequently, the equilibrium condition of the components of the cover is ensured by a combination of the two foregoing correction-factors. However, the velocity perpendicular to the side slope ( $u$ ) is very small compared with the velocity parallel to the side slope ( $v$ ). Hence the tractive force  $F_2$  is small compared with  $F_1$  and usually only very small values of the angle  $\beta$  need to be considered. It then appears that the correction-factor for the diameter of the stones is reduced to about the correction-factor as presented in the case of flow parallel to the side slope only. If a bank-protection in a river-bend is considered, it is therefore advised to apply the correction-factor ( $f_1$ ) as presented in Figure 2.5.9.

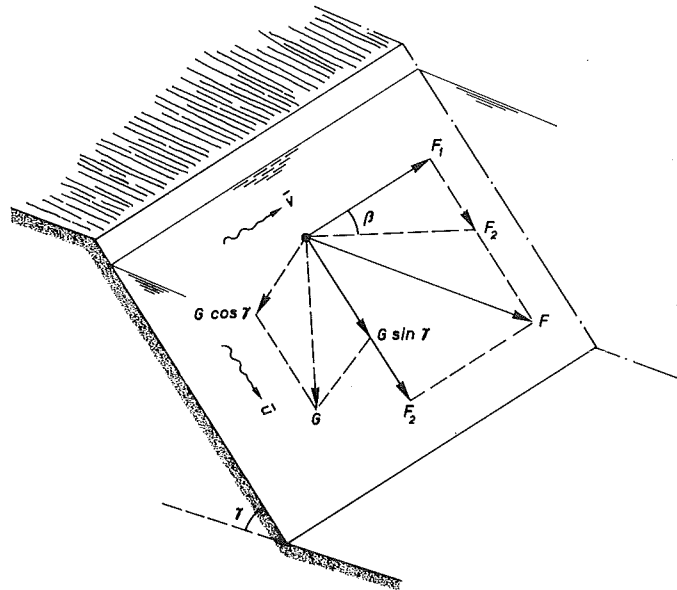


Figure 2.5.11 Forces Induced by Helicoidal Flow

#### Resistance against other influences than those caused by the current

In the foregoing the attack of the current on the protection has been discussed. Other types of influence, however, may also cause damage which can reduce the lifetime of the protection. Amongst others, the following can be mentioned:

- Mechanical influences, such as differences in temperature in the construction, or damage caused by ships (anchors or propulsion-stream);
- chemical influences, such as aggressive liquids or oil, the interaction with sea water, or electro-chemical action if different types of metal are used in one construction; and
- organic influences, such as vegetation above the water-level, or algae underwater.

The lifetime of the construction is mainly determined by the materials which are used. A number of such materials, as far as they appear to be suitable for use in Colombia, is listed below (see also Figure 2.5.6):

- Natural fibres (grass, twigs, cane, etc.);
- synthetic fibres (plastic, nylon, etc.);
- wood;
- plaited mats of twigs or tropical wood;
- gravel or stones;
- bricks;
- concrete blocks (cubes, tetrapods, hexapods, etc.);
- steel (sheet-piling, wire netting, etc.); and
- permeable suspensions (prepack-mortar, sand mixed with Standard Road Oil (S.R.O)).

A few remarks should perhaps be made about some of these materials. Wooden structures must generally be used below the low water-level to prevent their decay; although some kinds of tropical wood can also be used above the water-level. During its growing period grass needs oxygen and thus may not be flooded for more than about two weeks consecutively (although later

### III, 2.5

on flooding is acceptable during longer periods). Grass should, therefore, not be used below a level which is on the average exceeded during about 15 days per year. The synthetic fibres usually need to be covered from the sunlight to prevent reduction of their strength; but some special type of fibres have now been developed which no longer have this disadvantage.

#### 2.5.3. Closure of secondary branches

In Paragraphs 2.3 and 2.4 it was mentioned that the closure of secondary branches can often be considered to improve (either temporarily or permanent) the navigability of the main channel (such a closure was for example carried out in 1966 near the Río Sogamoso Confluence).

Dredging of the main channel is usually required, and this spoil can be used for the closure of the secondary branches. If the spoil has to be transported over a great distance, a combination of pumping and transport in trucks or in barges may be considered.

The secondary branches should not be closed too near to the upstream division of the channel. If the branch is closed more downstream, sedimentation will occur upstream of the dam, building a natural bar to protect the dam. If the branch which is to be closed still carries water, the velocities can increase considerably during the closure. The velocity may be approximated at:

$$v = \sqrt{2 g z}$$

with  $z$  = difference in head over the dam (see Figure 2.4.8). For values of  $z$  greater than about 0.50 m ( $v \approx 3$  m/s) the loss of sand by the increased flow-velocity will be considerable and a very great dredge capacity will be required to bring about the closure of the final gap. It may then be considered to close the branch temporarily at the upstream division of the channel where the flow-velocities are likely to be smaller (greater width and smaller water depth), and afterwards to built the dam further inward. (Or closure by means of stones or sand-bags may be chosen, to be dumped on a bed-protection to prevent great scour of the bed.)

The crest-level of the dam is also of importance. If the dam must function at low water stages only, while the branch will still discharge water at high stages, the protection of the dam and the river-bed must be heavy. Especially on the downstream side of the dam, the depression along the side slope caused by the overflowing water may well lift the stones from their setting and destroy the dam. If the dam should also fully close the branch at high water stages, the crest-level of the dam can best be determined on the basis of the occurrence of extreme high water-levels. Overflow of the surrounding river-banks should also be prevented, because this may result in a diversion of the current around the site of the dam.

#### 2.5.4. Examples of constructions

A number of constructions have been carried out along the Río Magdalena and the Canal del Dique, either for the protection of the established investments along the bank (e.g., La Dorada, Pto. Boyacá) or for navigation purposes (Canal del Dique). These constructions will now be discussed briefly.

Wooden open groynes

Wooden open groynes were constructed by ADENAVI mostly along the Canal del Dique although in the beginning of the 1960's one groyne was also constructed along the left bank of the Río Magdalena, upstream of La Dorada, while in 1971 6 groynes were constructed along the right bank of the Río Negro, just downstream of the village Puerto Libre, to protect the bases of overhead cables. Such groynes mostly consist of a single row of wooden piles connected with each other by wire or twigs (see Figure 2.5.12) or, in stronger currents, of two rows of piles more or less 1 m apart and filled in between with bundles of twigs.



Figure 2.5.12 Open Groyne along the Canal del Dique

Such open groynes can be carried out as a rather light type of construction because initially they will not concentrate the flow completely in front but permit part of the current to still pass through the groynes. The higher resistance induced by the construction results in smaller flow-velocities downstream where sedimentation occurs. On the upstream side of the groyne the floating debris carried by the river is collected and after a time-lapse is completely interwoven with the groyne. The groyne gradually changes into a more or less closed (but often still permeable) type of construction.

In the Canal del Dique a number of these groynes have been damaged, mostly by navigation. When two barge-trains meet each other, the narrow width of the Canal means that the one going upstream has to give way to the other by mooring along the bank. The damage is also partly caused by scour of the bed in front of the groynes. The groynes should, therefore, be inspected regularly and be properly maintained.

The use of such type of groynes in the Río Magdalena should generally be considered with suspicion. The often stronger currents, especially during high water stages, may well wash them away. Nevertheless, the construction of such a groyne near La Dorada, upstream of

### III, 2.5

the "Vuelta del Conejo", to prevent further scour of the left bank of the Río Magdalena proved its applicability in the past, although the groyne has now been almost completely washed away due to the lack of maintenance.

#### Bank protection upstream of La Dorada

Upstream of La Dorada the left bank of the Río Magdalena is attacked by the current, the more so because just downstream a rock is protruding into the river and upstream a strong eddy is generated which is gradually eroding the left bank. As this may endanger the railway connection La Dorada - Manizales (via Honda), the Ferrocarril Nacional (FCN) has constructed locally a bank protection consisting of dilapidated waggons filled with stones (see Figure 2.5.13).

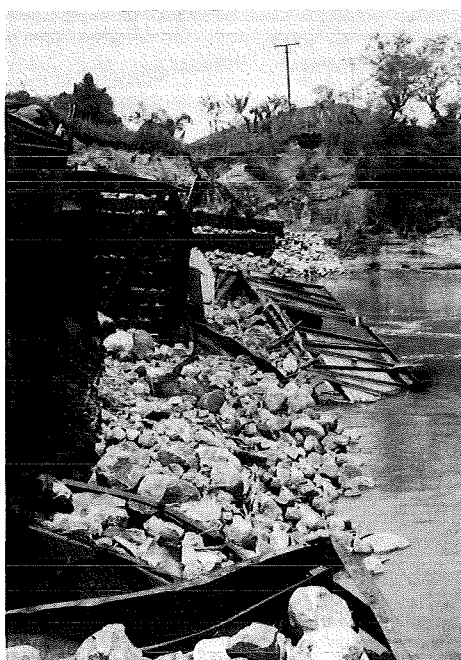


Figure 2.5.13 Bank Protection Upstream of La Dorada

The waggons form a permeable structure and may serve as the required filter for the prevention of loss of bank material. However, as they have not been firmly positioned along the side slope and have already partly fallen down, the flow-velocities may not have been reduced sufficiently to prevent the loss of bank material. Great quantities of stones will then have to be dumped to obtain the required protection. It is therefore recommended to use in future more appropriate types of constructions.

#### Groynes in Pto. Boyacá

In the high water season of 1972 the river-front at Pto. Boyacá was partly inundated and eroded by the Río Magdalena. To protect the village, the construction of groynes was undertaken by the "Departamento de Boyacá" to divert the main channel from the right bank (see Figure 2.5.14).

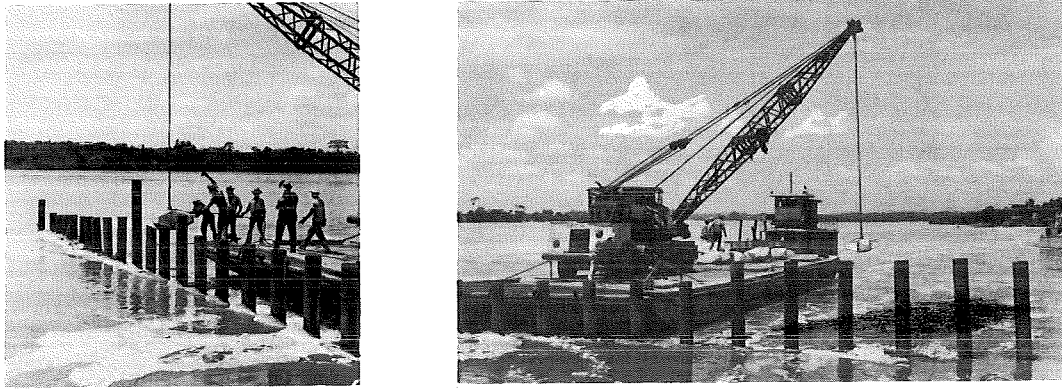


Figure 2.5.14 Construction of Groyne at Pto. Boyacá

The groynes consist of hollow steel tubes about half a meter apart and filled "in situ" with sand. Prefabricated tetrapods have been dumped in between and alongside the tubes. During the construction the steel tubes were used for the mooring of the barge which carried the crane and for guidance of the tetrapods during the dumping (see Figure 2.5.14). After completion, however, the tubes have no further function.

No filter was applied on the bed to prevent the loss of bed material, while a slight change in the direction of the upstream river-crossing may cause leakage as the groynes were barely continued into the bank. To maintain the groynes, probably great quantities of tetrapods (or stones) will still have to be dumped in future.

To study the stability of these groynes it is recommended to prepare local soundings and flow-lines at regular time-intervals. Gradually some experience will then be gathered regarding the applicability of and rate of maintenance required by such types of construction.

Further reference is made to Para. 3.2.7 in which a forecast is made of the possible development of the river course upstream of Pto. Boyacá.

#### Bank protection and groynes at Yarirí

In front of the Shell compound at Yarirí along the right bank of the Rfo Magdalena, a bank protection and a number of short groynes were constructed in 1972. The construction consists of hexapods (prefabricated in the nearby Shell compound) placed "in situ" by means of a crane (see Figure 2.5.15). These works are more or less a continuation of the existing protection of synthetic bags filled with stabilized sand which was executed in the 1950's.

Initially, a gravel filter was designed to prevent the loss of soil material on which the hexapods would be placed. Unfortunately, this filter was omitted (presumably in view of cost) and the hexapods were placed directly on the bed and the side slope of the bank. Although at present (1973) the main channel of the Rfo Magdalena follows the opposite bank, these protection works were already initiated to provide a future protection when the main channel shifts back again to the right bank. (That this phenomenon is not unlikely can be seen in Figure 3.5.6, where the meandering main channel of the Rfo Magdalena indeed follows the right bank at Yarirí). However, at present the depth along the right bank is very limited,

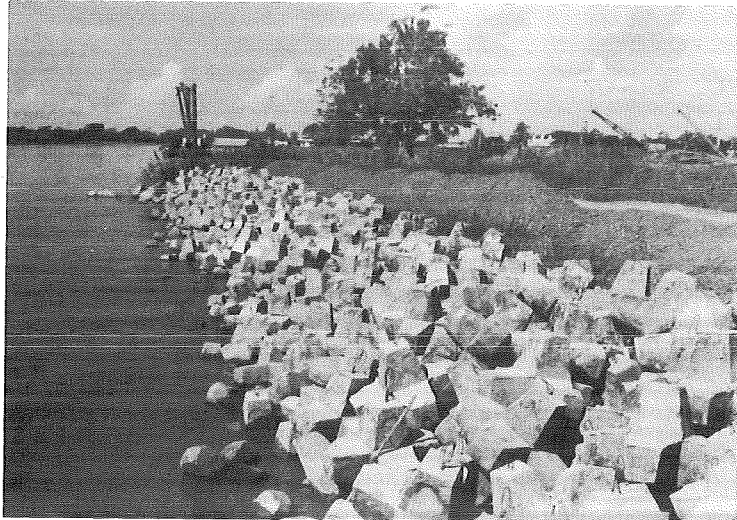


Figure 2.5.15 Protection of Shell Compound at Yarirf

and as no excavation works were carried out, the protection was only constructed near the water-level over a height of 3-4 m. When in future scour of the river-bed occurs, the hexapods will likely slip as a result of the loss of soil material and the increased water depth in front of the protection.

The hexapods were prefabricated in the Shell compound and consist of a mixture of concrete and locally-excavated river-bed material (mixture about 1:4). The whole scheme was very well organized and in about 3½ months 11,500 hexapods were fabricated and placed "in situ".

After these remarks regarding the river-works recently undertaken in Colombia, it must be mentioned that these constructions were carried out near the water-level only and that no proper measures were taken to prevent the loss of soil material. Consequently, the stability of these works in the course of time must be regarded with suspicion. So it seems worthwhile to conclude this Paragraph with a review of constructions which may be considered for future river-works.

Dumping of graded stones (riprap)

Advantage : Loss of soil material can be prevented.

Disadvantages: Execution mainly "in the dry" or in small water depths.

The weight of the stones determines the fall-velocity, which means that by execution "in the wet" in great water depths the heaviest stones will settle first with the smaller stones on top. This finer material may be taken away by the current so that great quantities are required to have a sufficiently thick, current-resistant protection.

Riprap packed in wire-netting (see Figure 2.5.16)

Advantage: "Armouring" of the protection during the execution is not necessary. (To be applied on gravel filter against loss of soil material.)

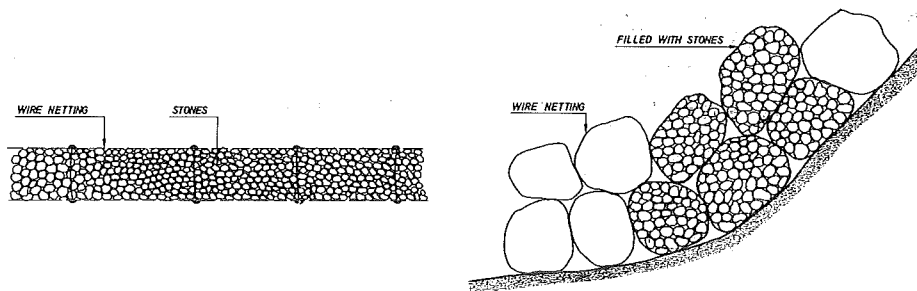


Figure 2.5.16 Stones Packed in Wire-netting

**Disadvantages:** Corrosion of the wire-netting may finally lead to the destruction of the protection if the stones are then carried away by the current. Scour in front of the protection results in the shifting of the gabions. This may be prevented by drilling wooden piles (see Figure 2.5.17), although fixation and stability of the toe remains difficult.

Riprap packed in plaited mats (Figure 2.5.17)

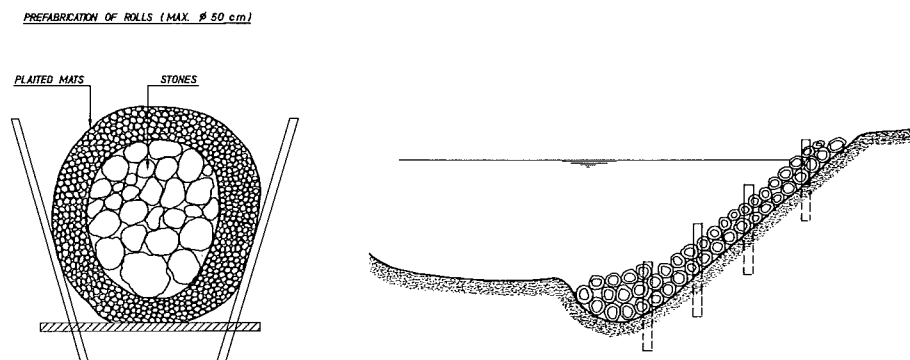


Figure 2.5.17 Stones Packed in Plaited Mats

**Advantages:** The filter of the protection consists of plaited mats of twigs or wood, while the weight is provided by stones. Prefabrication of the rolls with a maximum length of about 10 m is possible at the shore, after which they can easily be rolled down the side slope.

**Disadvantages:** Erosion in front of the protection results in a shifting of the rolls. Prevention is possible by drilling wooden piles through the rolls, although, especially under water, this is rather difficult.

Bags

**Advantage:** The bags can be made either of jute (which is, however, not very resistant) or nylon. Comparatively cheap construction in view of the low cost of filling material (sand). However, jute and nylon bags should be very finely woven to prevent the loss of sand from the bags. Dumping of the bags is possible from relatively great heights, especially when reinforced nylon bags are used.

### III, 2.5

#### Bags filled with suspensions

**Advantages:** The same as mentioned for sand-bags. Moreover, the loss of sand from the bags is prevented by injection of the filling material by means of, e.g., prepaqt mortar or Standard Road Oil (S.R.O.).

#### Gravel filter covered with riprap

**Advantages:** If properly executed, a good filter is obtained. The consecutive layers of the filter can be dumped from a temporary jetty (Figure 2.5.18) or from a barge.

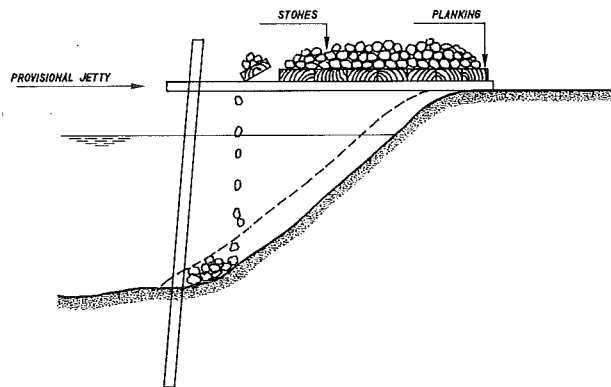


Figure 2.5.18 Side Slope Protection Dumped from a Jetty

**Disadvantages:** If the protection is executed "in the wet" or on a side slope of a river-bank, rather thick layers of the filter are required to prevent the loss of bank material. In the case of a bed protection dumped from a barge, the current will carry the smaller particles further downstream than the greater ones. A bed protection should therefore always be made in a downstream direction to adequately obtain a filter.

#### Nylon filter covered with riprap

**Advantages:** A simple construction of a nylon mat weighted down with stones. Irregularities in the soil can easily be followed by the mat.

**Disadvantages:** Due to the great flexibility of nylon, special barges and experience are required for the execution. An initial weighting by means of, e.g., sand-bags is required during the installation. If the components of the cover-layer have to be dumped from a great height, the fabric can be damaged and it may be better to start with a gravel layer. To overcome these disadvantages, nylon filters which can be weighted with filling material are available (see Figure 2.5.6).

#### Facine mattress covered with riprap

**Advantages:** In general, suitable for all kinds of protection under water and, if special measures are taken (impregnation or the use of tropical wood), also applicable above the water-level.

Disadvantages: Skilled labour is required and the execution is highly laborious. The mattress (filter) must be prefabricated, e.g., on a low-lying shore and be transported to the site by floating at higher water stages. To prevent damage during the transportation, the filter is towed behind a barge which is positioned perpendicular to the flow direction. On the spot gravel and stones are evenly dumped on the mattress till it is just about to sink, after which, dumping of the stones is continued from the upstream end onward, so that the actual sinking to the river-bed will be partly induced by the current. For bank protections the filters are attached to the side slope (preferably by drilling piles through the filter along a berm), and the dumping of the stones can be carried out from the bank.

## Chapter 3

### RÍO MAGDALENA

#### 3.1. INTRODUCTION

This Chapter contains the proposed improvement for navigation of the Río Magdalena between La Dorada and Gamarra.

For this purpose the Río Magdalena has been divided into four main sections: La Dorada (km 885) - Pto. Inmarco (km 773); Pto. Inmarco - Pto. Berrío (km 730); Pto. Berrío - Barrancabermeja (km 631); and Barrancabermeja - Gamarra (km 473).

This division into sections is mainly based on the requirements for navigation, and for the proposed improvement the required Least Available Depth was taken at 7'6" below L.R.L. downstream of Pto. Berrío, at 4'6" (possibly 6') below L.R.L. between Pto. Inmarco and Pto. Berrío, and at 4'6" below L.R.L. upstream of Pto. Inmarco. However, as the pertaining hydraulic and morphological conditions in the river are also based on affluents of the Río Magdalena, the main sections are sometimes sub-divided. For example, the La Dorada - Pto. Inmarco section consists of two sub-sections, one upstream and one downstream of the Río Negro and Río La Miel Confluences.

Firstly, a general description of each section is given. The collected data have been interpreted to arrive at design criteria for each section and especially attention has been paid to the presented (computed) cross-sections of the river in bends and at crossings which must be pursued in future when considering execution of river-works.

Secondly, a temporary improvement by means of dredging each section is dealt with. The volumes which have to be dredged recurrently in the low water season are mainly based on the longitudinal profiles presented in Part II of this Report. For an estimate of the retarded scour to be expected at crossings at a water-level corresponding to L.R.L., the morphological computations (the principles of which are outlined in Para. 3.6 of Part II and partly already applied to specific problems like the Río Sogamoso Confluence in Para. 3.8) have been used.

Thirdly, each section has been analysed and specific problems (concerning not only navigation aspects) are treated separately.

In Para. 3.2 the section La Dorada - Pto. Inmarco is dealt with; Para. 3.3 treats the Pto. Inmarco - Pto. Berrío section (and also gives a review of the results of the morphological computations presented in Para. 3.6 of Part II); and Para. 3.4 covers the Pto. Berrío - Barrancabermeja section. Considerations regarding the Barrancabermeja - Gamarra section can be found in Para. 3.5.

3.2. LA DORADA - PUERTO INMARCO (KM 884-773)3.2.1. General description and design criteriaAvailable cross-section

According to Part II, Para. 3.5.4, this river section can best be sub-divided into two sections, viz., a section between La Dorada and the confluences of the Río Magdalena and the Ríos Negro and La Miel (about km 840), and the section downstream of these confluences to Pto. Inmarco. The cross-sections which were taken in these two sections have been compiled in Figures 3.2.1 and 3.2.2, where they are given as mass curves. The mass curve of the schematized measuring cross-section near La Dorada is also included.

Schematized cross-section

The cross-sections given in Figures 3.2.1 and 3.2.2 have been schematized into average cross-sections which are also presented as mass curves in the figures. The depth of the average mass cross-section has been determined as an average of the depth in the single cross-sections. (For the determination of the average depth not very much attention has been paid to the included measuring cross-sections, because these sections were not chosen at random but selected as good measuring sections with, generally, greater depth). The determination of the width of the average mass cross-section was made in accordance with the width of the single cross-sections and the mean river-width deduced from the aerial photographs. Thereafter the relation between the water-level and, respectively, the average water-depth ( $\bar{h}$ ), the value of  $(\bar{h}^{3/2})^{2/3}$  and of  $(\bar{h}^{5/2})^{2/5}$  has been determined.

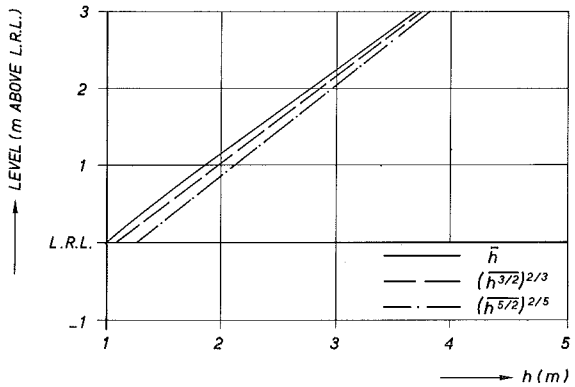
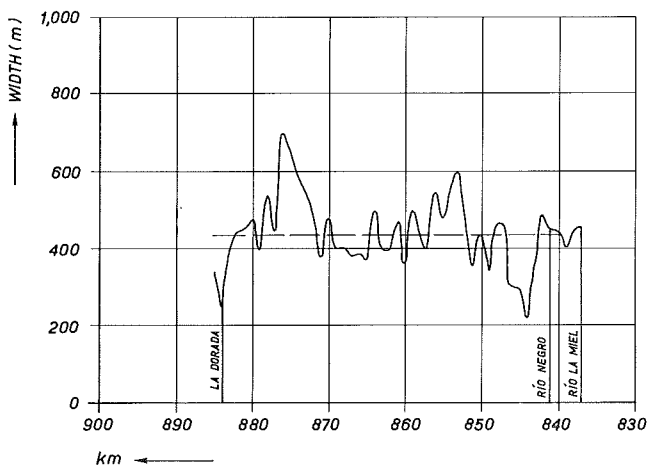
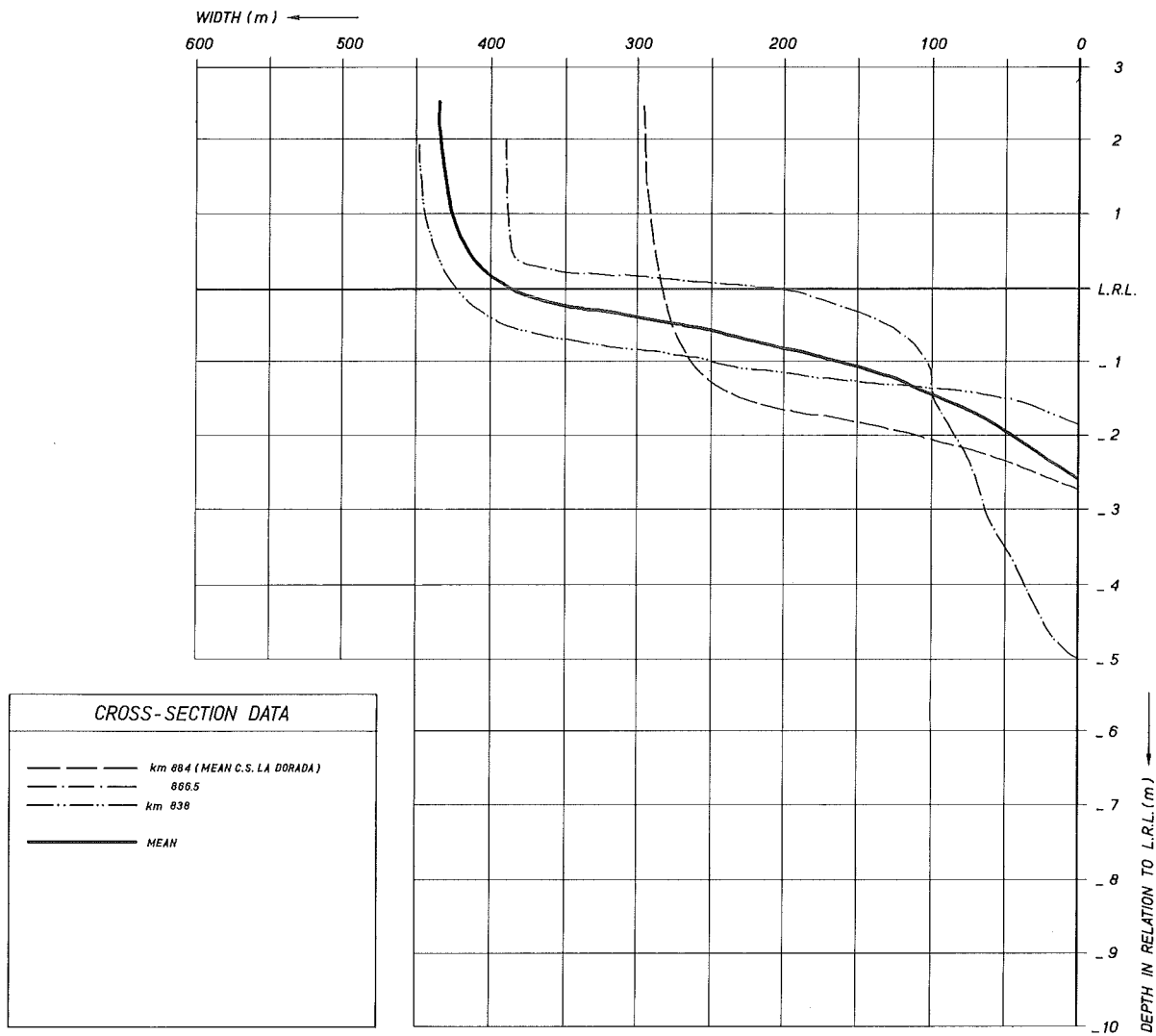
Water-level gradient

From the water-level data of the gauges at Pto. Salgar, Pto. Inmarco and Pto. Berrío, it was found that the average water-level gradient is in the order of  $40 \times 10^{-5}$ . Although the local gradients may differ considerably from this average value (e.g., in the La Dorada measuring section  $\bar{I} = 22 \times 10^{-5}$ ), for the design of river-works the average water-level gradient should be used. However, it should be kept in mind that a change in width generally also leads to a change in water-level gradient. If an improvement of a river-section is considered, and the morphological computations will show a change in the water-level gradient which will lead to unacceptable changes of the water-level upstream of this improved section, a study must be made to find out if this unfavourable result of the improvement can be counterbalanced by other measures (e.g., the cutting of meander-bends) or, possibly, by an adaptation of the design of the improvement. In the following considerations the average water-level gradient has been used.

Design bend-radius and water depth in outer bend

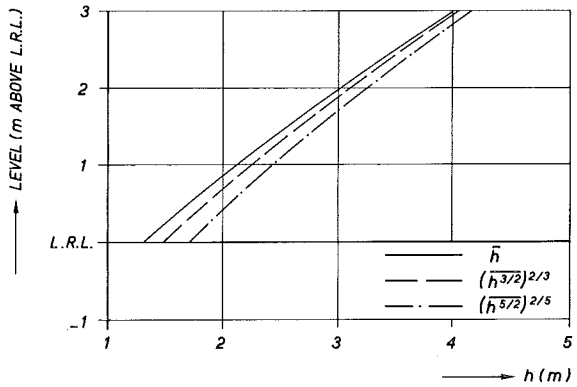
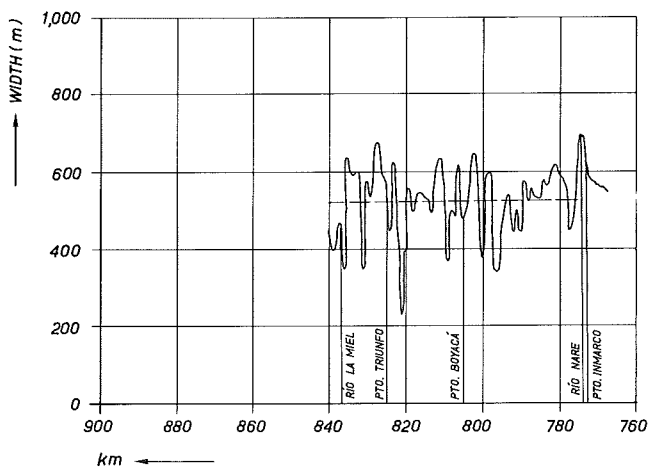
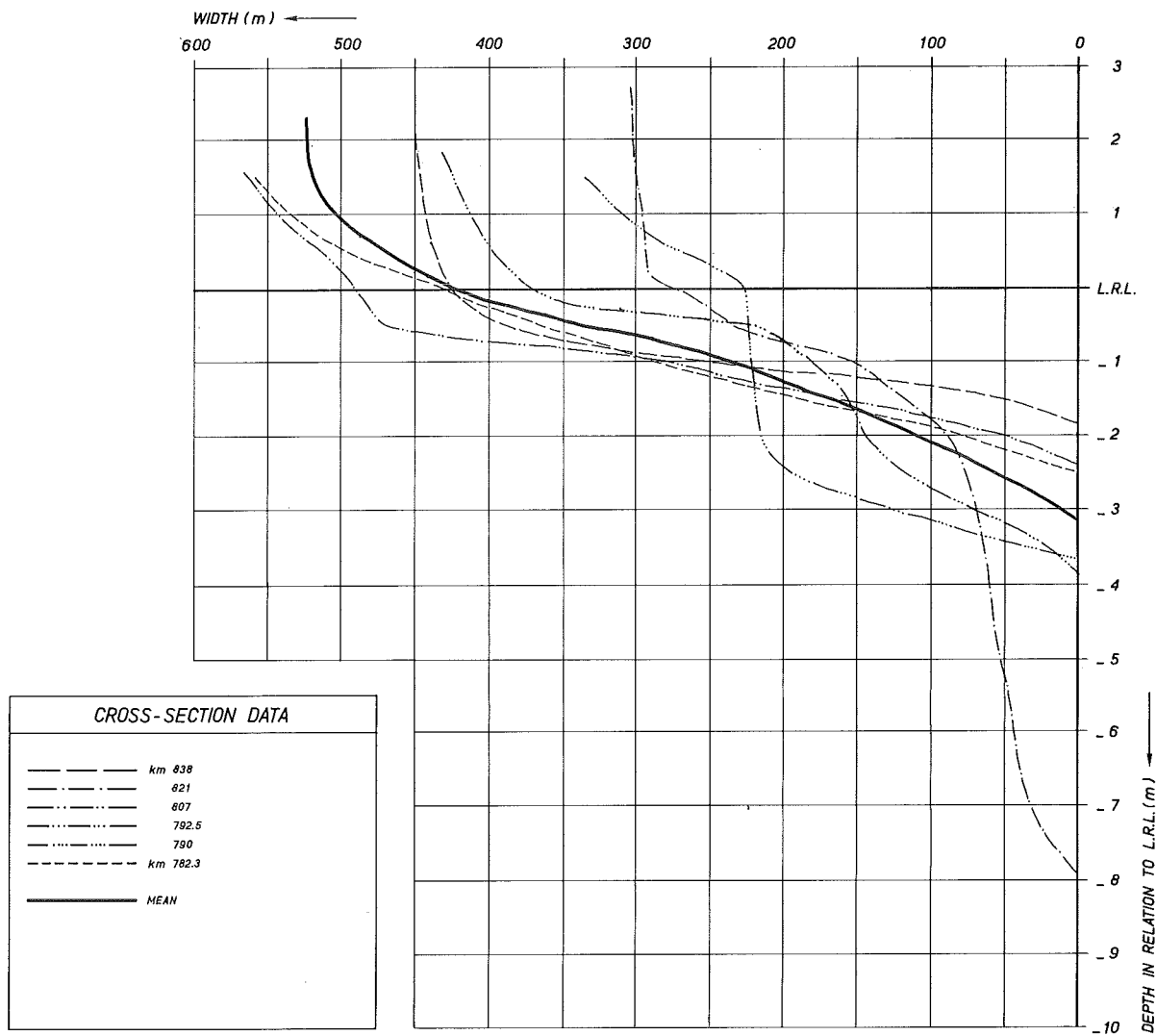
In Part II, Para. 3.7.3, a formula was deduced to calculate the radial bed-level slope in a bend as a function of the bend-radius and the water depth:

$$\frac{dh}{dR} = \frac{A I_0 R_0}{\Delta D} \cdot \frac{h^2}{R^2} \quad (\text{Part II, 3.7.10})$$



SCHEMATIZATION OF THE RÍO MAGDALENA km 885 - km 840

FIG. 3.2.1



SCHEMATIZATION OF THE RÍO MAGDALENA km 840 - km 774

FIG. 3.2.2

### III, 3.2

An example was given (Part II, Figure 3.7.10) for a cross-section in the Río Magdalena (km 821.5) downstream of Pto. Triunfo which showed that for a value of  $A = 10$  the computed cross-section fitted the measured cross-section reasonably well. This value will also be applied here (while another check on the value  $A = 10$  is made in Para. 3.2.3 for a cross-section in the Vuelta del Conejo, upstream of La Dorada). For a given bend-radius and an estimate of the water depth in front of the protected outer bank, the radial bed-level slope can be computed by the "trial-and-error" method. The conveyance ( $Bh^{3/2}$ ) of the computed cross-section at the water-level corresponding to the dominant discharge needs to be of the same order of magnitude as the conveyance of the schematized cross-section. When this is not compatible, the computation must be carried out again by selecting another bend-radius. Re-estimating the water depth in front of the protected outer bank is not very effective, as this influence is felt only in the first steps of the computations. For the estimation of the water depth in front of the protection, it should be considered that to a certain extent a smaller water depth and, possibly, a greater bend-radius may lead to lower cost.

The computations of the radial bed-level slope of the river section between km 885 and km 840 can be followed in Table 3.2.1 in which only the final results of each computed step are given.

$$\frac{dh}{dR} = \frac{A I_o R_o}{\Delta D} \cdot \frac{h^2}{R^2}$$

Assumptions:  $A = 10$   
 $I_o = 40 \times 10^{-5}$   
 $\Delta D = 672 \times 10^{-6} \text{ m}$  ( $\bar{D}_{50} = 400 \text{ }\mu\text{m}$ , see Part II, Figure 3.3.15)  
 Dominant water-level = L.R.L. + 2 m  
 Water depth in outer bend = L.R.L. - 8 m  
 Average width in river-bend = 375 m (see Figure 3.2.1)

$R_o = 1,500 \text{ m } (R_o = 4B)$					$R_o = 2,250 \text{ m } (R_o = 6B)$				
$R_i$	$\frac{dh_i}{dR_i}$	$R_{i+1}$	$h_{i+1}$	$\frac{dh_{i+1}}{dR_{i+1}}$	$R_i$	$\frac{dh_i}{dR_i}$	$R_{i+1}$	$h_{i+1}$	$\frac{dh_{i+1}}{dR_{i+1}}$
(m)		(m)	(m)		(m)		(m)	(m)	
1,500	0.235	1,490	7.65	0.235	2,250	0.18	2,240	8.2	0.18
1,490	0.152	1,480	6.53	0.153	2,240	0.13	2,230	6.9	0.128
1,480	0.092	1,460	4.69	0.092	2,230	0.078	2,210	5.34	0.078
1,460	0.037	1,410	2.84	0.036	2,210	0.036	2,160	3.54	0.036
1,410	0.018	1,360	1.94	0.018	2,160	0.02	2,110	2.54	0.0194
1,360	0.0077	1,260	1.17	0.0077	2,110	0.009	2,010	1.64	0.009
1,260	0.004	1,160	0.77	0.004	2,010	0.005	1,910	1.14	0.0048

Table 3.2.1 Computation of Radial Bed-level Slope (km 885 - km 840)

The results of the computations are given in Figure 3.2.3. The conveyance of the schematized cross-section (Figure 3.2.1) at a water-level of 2 m above L.R.L. is about  $Bh^{3/2} = 2,100 \text{ m}^5/2$ . In the bend the conveyance will reach this value for about  $R = 1,750 \text{ m}$  (Figure 3.2.3).

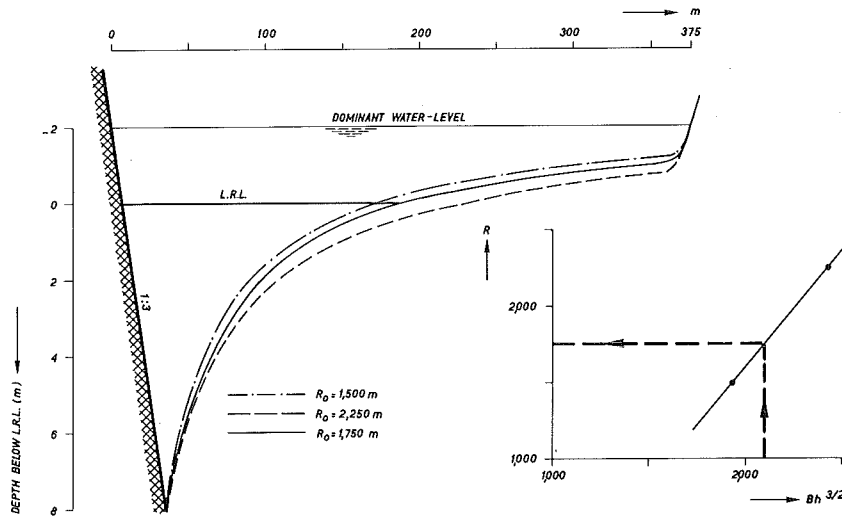


Figure 3.2.3 Computed Cross-section in River-bend (km 885 - 840)

Similarly to the computations outlined above, the cross-section in a river-bend for the river section between km 840 and km 774 has been computed (Figure 3.2.4). The following assumptions have been made:

$A = 10$ ,  $I = 40 \times 10^{-5}$ ,  $R = 1,750$  m,  $D_{50} = 800 \mu\text{m}$  and the water depth in the outer bend is 8 m below L.R.L. The conveyance of this profile at the dominant water-level (L.R.L. + 2 m) is about  $3,050 \text{ m}^5/2$  while the conveyance of the schematized cross-section (Figure 3.2.2) at this level reaches the value of  $2,850 \text{ m}^5/2$ .

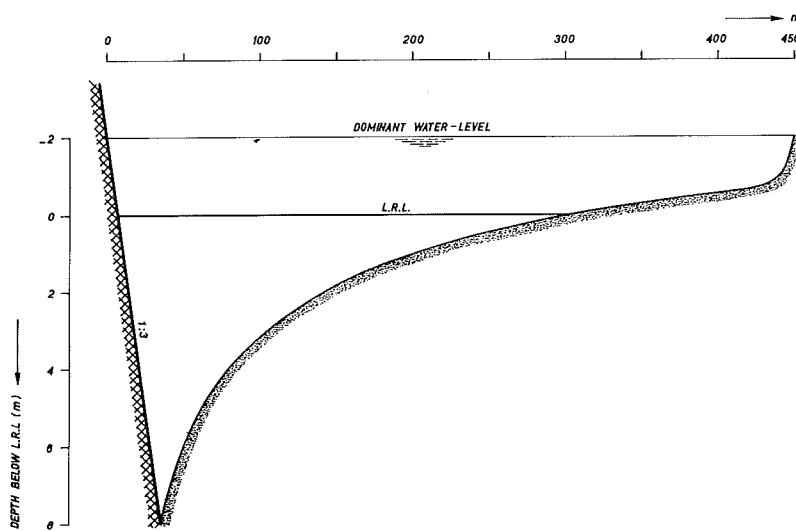


Figure 3.2.4 Computed Cross-section in River-bend (km 840 - 774)

From Figures 3.2.3 and 3.2.4 it can be read that in the bends the minimum required depth of 4'6" below L.R.L. will be available over a width of 110 m or more, which is sufficient for two-way traffic.

Design width and water depth on crossing

The question arises what must be the width of the river on the crossings to ensure that at L.R.L. the minimum available water depth will be equal to 4'6" (this means that no maintenance dredging would be required). To answer this question an assumption has to be made regarding the shape of the cross-section on the crossing. Usually, the cross-section on the crossing is assumed to be flat. However, in a braided river the sinuous channel mostly consists of a great number of consecutive bends curved in opposite directions and divided by relatively short transition zones: the crossings. The shape of the cross-section on the crossing can, therefore, also assumed to be a superposition of the cross-sections in the two neighbouring bends as shown in Figure 3.2.5.

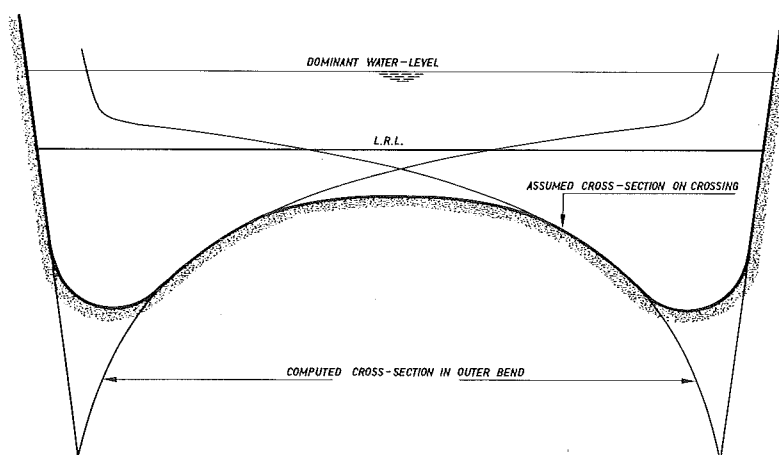


Figure 3.2.5 Schematized Cross-sections in River-bends and Crossing

On the crossing the channel of the upstream bend will not have faded away completely, while the channel of the downstream bend will already be partly developed. The cross-section will then consist of a channel along both banks (each channel in its turn shallower than the fully-developed channel in the outer bend), divided from each other by a shallow part which has to be encountered by navigation crossing from one channel into the other.

Although this second assumption may be somewhat more realistic, it should be kept in mind that in the one-dimensional morphological computations outlined in Part II, Para. 3.6, the assumption is implicitly made that the cross-section is of rectangular shape. It is recalled from Part II, Para. 3.8.2 that the available depth on the crossing at the beginning of the low water season is considerably more than would be expected from the data at higher water stages, due to the retarded scour during the fall of the water-levels. The example given in Part II, Figure 3.8.2, shows that the extent of this retarded scour is in the order of 1 m (which is confirmed by the examples given later on in this Chapter).

From these considerations, it can be concluded that in reality the cross-section will be shaped neither rectangularly nor as a superposition of the cross-section in two consecutive bends curved in opposite directions (schematically given in Figure 3.2.5). Nevertheless, to be able to determine the required width on the crossing in such a way that also at low water stages the available water depth will be sufficient for navigation (4'6" = 1,35 m), the following procedure is followed:

- In the first instance, the width on the crossing is predicted from the computed shape of the cross-section in the outer bend. In this cross-section the width is determined at a level of 0.35 m below L.R.L. (allowing for an increase of the depth of about 1 m due to the retarded scour during the fall of the water-level from the dominant stage to L.R.L.). Multiplication by a factor 2 yields the width of the cross-section on the crossing.

- Secondly, the cross-section on the crossing is assumed to be rectangular in shape. The width of a rectangular profile with an average bed-level at the dominant stage of L.R.L. - 0.35 m (allowing again for a local increase of the depth of 1 m at L.R.L.) can then be computed.

It is likely that when the width of the cross-section on the crossing found by these two methods is compared, a difference will be found (the width found by the first method will be too small, while that in the rectangular profile will be too large). As the cross-section on the crossing is usually composed of a channel along both banks, it is therefore advised to rely more on the result found by the first method.

For the river section between km 885 and km 840, Figure 3.2.3 shows that the width on the crossing should be about 300 m. From the given conveyance of the schematized cross-section ( $Bh^{3/2} = 2,100 \text{ m}^{5/2}$ ; see Figure 3.2.1), it follows that a rectangularly-shaped profile should have a width of about 580 m (the average water depth at the dominant stage will then be 2.35 m and locally  $0.35 + 1 \text{ m} = 1.35 \text{ m}$  at the water-level coinciding with L.R.L.). For the width on the crossing a value of about 400 m should, therefore, be taken.

For the river section between km 840 and km 775, Figure 3.2.4 shows that the width on the crossing should be about 500 m. The conveyance of the schematized cross-section reaches a value of  $Bh^{3/2} = 2,850 \text{ m}^{5/2}$  at the dominant stage (Figure 3.2.2). The width of a rectangularly-shaped profile should then be about 800 m. For the crossing a width of about 600 m is considered to be sufficient.

#### River stretches requiring improvement

After the foregoing considerations regarding the available cross-sectional area and the design width and depth of the river section between La Dorada and Pto. Inmarco, the stretches which require improvement need to be studied more closely. In the Schedule of Operations only the La Dorada - Pto. Salgar area and the Río Nare Confluence are mentioned, but a better insight can now be obtained when also the longitudinal soundings of this river section (Part II, Figures 3.3.17 to 3.3.22 inclusive) are considered.

A number of crossings can be distinguished which will likely hamper navigation in the dry season because the available water depth is less than 4'6" below L.R.L. However, most of the crossings are not located at fixed places, but change along the river axis. Apart from the changes in the bed-level resulting from a change in the water stage, this may also be caused by the development of new channels and the deterioration of the older ones (compare, e.g., the course and longitudinal profile of the sounding in September, 1972, between km 875 and km 868 with the course and profiles of older dates). The temporary improvement of the whole river section by means of recurrent dredging is treated in Para. 3.2.2.

### III, 3.2

In addition to such temporary improvements, a more permanent improvement of certain river stretches must also be considered. The following division can be made:

#### La Dorada - Pto. Salgar

The accessibility to the available river-port facilities in La Dorada and Pto. Salgar needs to be improved. As such river improvements are closely related to the defense against erosion by the river of, respectively, La Dorada town and the German Olano (Palanquero) air-base, this whole region is further discussed in Para. 3.2.3.

#### Confluences of the Río Magdalena and the Ríos Negro and La Miel

Comparing the present situation with, e.g., the situation during the survey of the Julius Berger Konsortium (1922-1924), it is clear that the main course of the Río Magdalena has shifted considerably. This area is discussed in Para. 3.2.4, along with the crossing upstream of the Río Negro Confluence.

#### Crossing near km 833

The morphological computations of the schematized crossing near km 833 are presented in Para. 3.2.5.

#### Pto. Triunfo

During the Missions' study ADENAVI proposed that the port facilities of La Dorada/Pto. Salgar be moved downstream to Pto. Triunfo. The feasibility of such a scheme, together with the advantages in view of the lower maintenance cost of the Río Magdalena between La Dorada and Pto. Triunfo, is dealt with in Para. 3.2.6.

#### Pto. Boyacá

During the high water season in April-May 1972 the low-lying right bank of the Río Magdalena in front of Pto. Boyacá eroded, causing damage to the town area. The construction of groynes undertaken with the aid of personnel and equipment of the Texaco Oil Company (Pto. Niño) is treated in Para. 3.2.7.

#### River-crossings near km 780

Some information regarding the river-crossings near km 780 in the Río Magdalena is given in Para. 3.2.8.

#### Confluence of the Río Magdalena and the Río Nare

Near to this Confluence the main channel of the Río Magdalena is constantly shifting in a braided river-bed, while downstream of this confluence the width of the river is strongly restricted by diluvial formations (Pto. Inmarco). Together with the crossing upstream of Nare town this stretch is discussed in Para. 3.2.9.

### 3.2.2. Temporary improvement by means of dredging

For the determination of the total quantity which has to be dredged recurrently to bring about a temporary improvement of the river sections between La Dorada and Pto. Inmarco, two questions still need to be answered.

In Figures 3.3.17 to 3.3.22 inclusive, presented in Part II, a number of longitudinal profiles are given. The first question is: Which of these profiles should be taken for the determination of the quantities to be dredged? To answer this question, it should be considered that the longitudinal sounding should not have been made during a too high water stage, when the talweg is more difficult to find and a too shallow depth may have been recorded. On the other hand, the longitudinal sounding should not have been made during a too low water stage, because too great a depth may then have been recorded due to the retarded scour, while at an earlier date (higher water stage) the available water depth could have been less. For the determination of the quantities which should be dredged recurrently, the longitudinal sounding recorded in February, 1972, has been used. This sounding was made at the end of a period of relatively high water stages, but before the onset of the period of low water stages. As indicated on the route maps the water-level during the survey was about 3 ft. above L.R.L. For the river section upstream of the confluence of the Rfo Magdalena and the Rfo La Miel, a minimum water depth of about 3 ft (bed-level at L.R.L.) was recorded at km 864 and in between km 859 and 860. For the river section downstream of the Rfo La Miel Confluence, the minimum recorded water depth was about 4'6" (bed-level 1'6" below L.R.L.) at km 833 and in between km 819 and 820.

The second question which arises is: To what extent can retarded scour be expected if after the survey the water-level would have fallen to L.R.L.? For the two river sections under consideration, the extent of the retarded scour will be demonstrated by means of morphological computations for the schematized crossings near km 840 and km 833 (Paras. 3.2.4 and 3.2.5). Anticipating the results given in these paragraphs, the schematized regimes which have been used in these computations show scour of the highest level of the bed of about 3 ft. during the fall of the water-level to about L.R.L.

The total quantities which have to be dredged can thus be determined in two ways. Firstly, by computing the quantity enclosed by the recorded bed-level and the level of 4'6" below the L.R.L.; but such a procedure will lead to a too high amount because no allowance is made for scour of the bed during the fall of the water-level. Secondly, by considering a scour of the bed of about 3 ft. during the fall of the water-level and computing the quantity enclosed by an imaginary bed-level (situated roughly 3 ft. below the recorded bed-level) and the level of 4'6" below the L.R.L. This procedure will, generally, lead to too low an amount because it is likely that the retarded scour will be less than 3 ft. when the initial bed-level was already situated well below L.R.L. However, for those crossings where the bed-level was recorded at L.R.L. (km 864 and in between km 859 and 860), the least available depth was already less than 4'6" and dredging should, in fact, have already taken place. Therefore a number of crossings can be distinguished which need to be dredged completely, a number of crossings which will partly be eroded by retarded scour (about 2 ft) while the remaining part needs to be dredged, and some crossings which will sufficiently erode by retarded scour.

### III, 3.2

The total quantities which need to be dredged have been computed in the two ways just outlined and are given in Table 3.2.2. For the river section between the Río La Miel Confluence and Pto. Inmarco the quantities are sub-divided for the river sections upstream and downstream of Pto. Triunfo; the reason for which is explained in Para. 3.2.6.

River-sections	Kilo- meters	Retarded scour not included			Retarded scour (2') included		
		Volume (m <sup>3</sup> )	Extra (25%) (m <sup>3</sup> )	Total Volume (m <sup>3</sup> )	Volume (m <sup>3</sup> )	Extra (25%) (m <sup>3</sup> )	Total Volume (m <sup>3</sup> )
La Dorada - Río La Miel	889-837	364,000	91,000	455,000	156,000	39,000	195,000
Río La Miel - Pto. Triunfo	837-823	72,000	18,000	90,000	17,600	4,400	22,000
Pto. Triunfo - Pto. Inmarco	823-773	188,000	47,000	235,000	42,500	10,500	53,000
			Total	780,000		Total	270,000

Table 3.2.2 Quantities to be Dredged between La Dorada and Pto. Inmarco

The volumes have been determined for a 50 m wide channel with a bed-level at 4'6" below L.R.L. An allowance of 25% has been made: for waste and side-slopes of the channel 10%, and for the assumption that the locally recorded depth is taken to be representative for that part of the cross-section where the channel (50 m wide) will be dredged, 15%. The total volume which needs to be dredged in the river section between La Dorada and Pto. Inmarco (retarded scour included) amounts to 270,000 m<sup>3</sup>, of which 217,000 m<sup>3</sup> must be dredged upstream of Pto. Triunfo and 53,000 m<sup>3</sup> between Pto. Triunfo and Pto. Inmarco.

#### 3.2.3. Improvement of La Dorada - Pto. Salgar

##### Introduction

La Dorada is situated on the left bank of the Río Magdalena (km 888) just downstream of the so-called "Vuelta del Conejo". Pto. Salgar is on the right bank of the Río Magdalena (km 887), opposite La Dorada, while immediately downstream of Pto. Salgar is the important air-base German Olano. A railway-road bridge spans the river at the downstream end of La Dorada town, giving this region, apart from river transport, access by rail to the Caribbean coast, Bogotá and Neiva (via Ibagué), as well as access by road to Bogotá, Medellín and Manizales.

Upstream of La Dorada the Río Magdalena flows in a narrow valley bordered on both sides by mountain ranges of the Andes. The river has steep water-level gradients, sharp bends and high flow-velocities. Although more upstream the river section between Purificación (km 1,148) and Girardot (km 1,085) appears to be suitable for navigation with small craft, the aforementioned conditions and, above all, the rapids near Honda (km 930), show that La Dorada - Pto. Salgar can be considered to be the most upstream site for port facilities and navigation as far as the middle and lower courses of the Río Magdalena are concerned. Downstream of La Dorada the river valley widens considerably, the water-level gradients and flow-velocities decrease, and the river is flowing mostly through its own (alluvial) deposits.

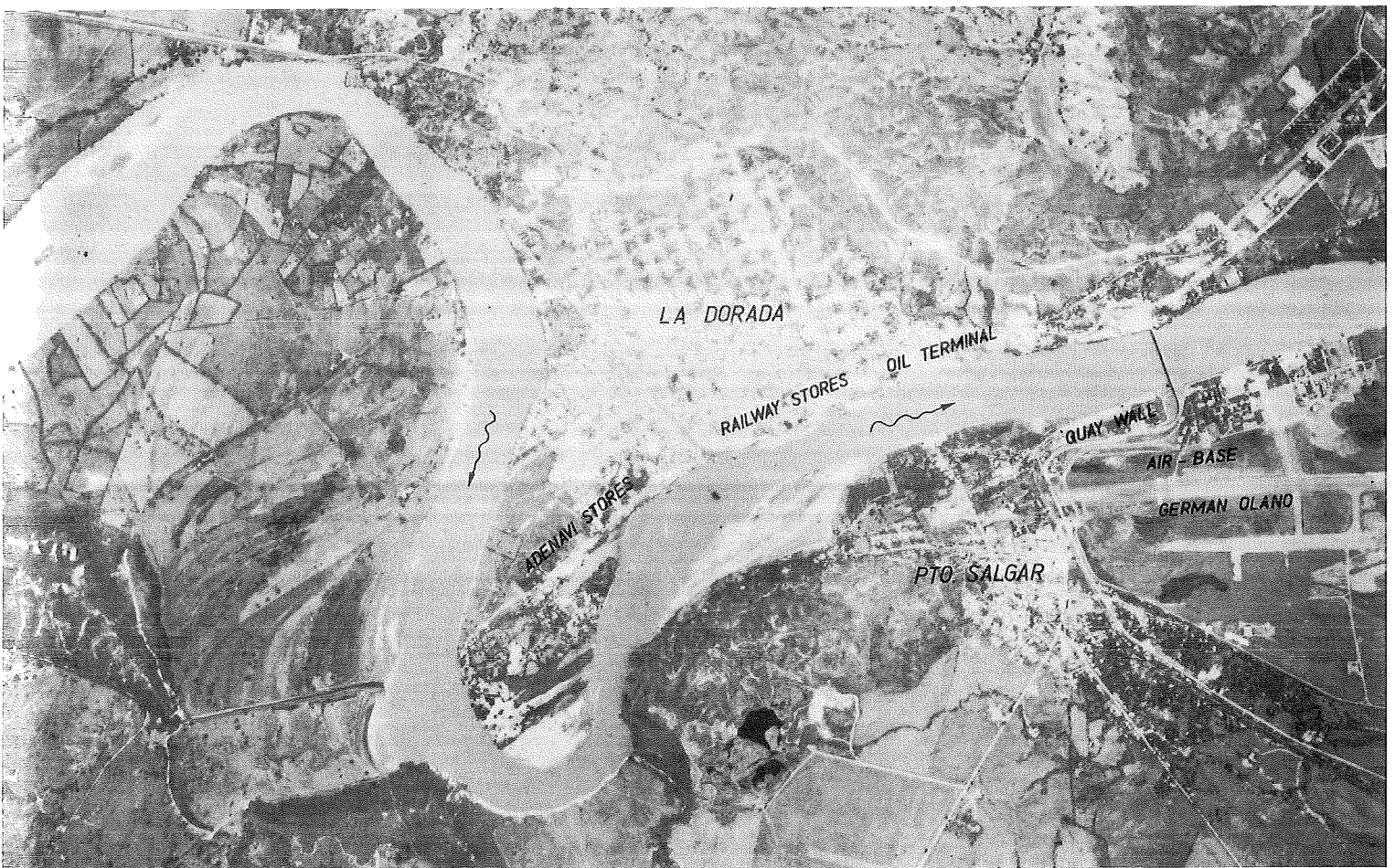


Figure 3.2.6 La Dorada - Pto. Salgar Area

The existing river-port facilities in La Dorada and Pto. Salgar and their defects can be listed as (going downstream, see Figure 3.2.6):

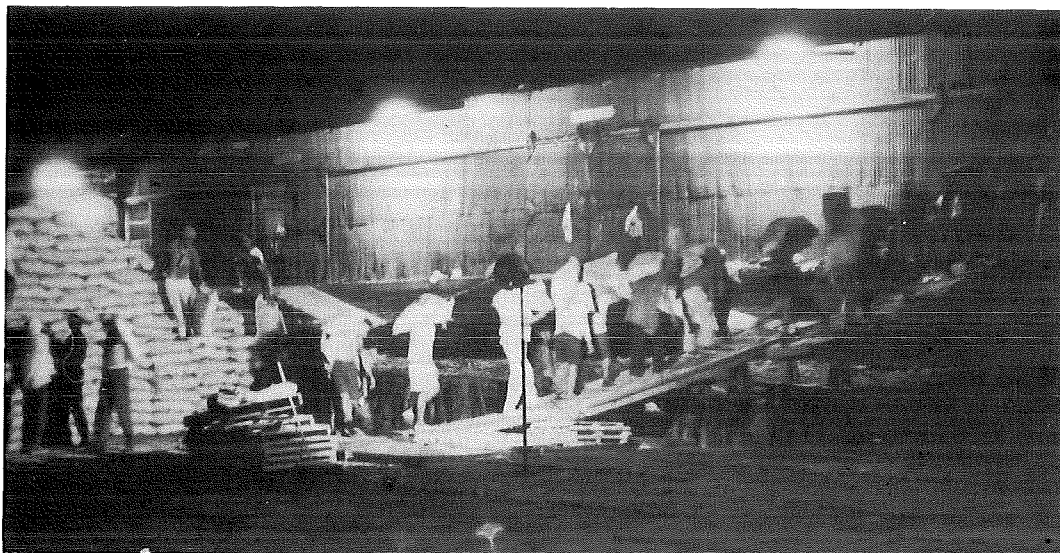
- The ADENAVI stores (left bank)

Situated on the South-Eastern side of La Dorada town, originally along the left bank of the Rfo Magdalena but, due to an alteration in the course of the river at the Vuelta del Conejo, they are at present situated far inland. In the past when the stores were still situated along the Rfo Magdalena, ADENAVI constructed a kind of spur-dike in front of the stores, and connected to the left bank of the river upstream of the stores, to reduce the sedimentation. Although it appears unlikely that the construction of this spur-dike has resulted in the alteration in the course of the Rfo Magdalena, it is possible that the sedimentation along the inner bend of the river in the Vuelta del Conejo has been activated by this construction. In the present situation the stores can only be reached by an artificial inlet. If this inlet is not kept open regularly, the port facilities are not accessible to river transport. The stores are accessible by road through La Dorada town, while a side-track of the railway between La Dorada and Honda provides for accessibility by rail. There is no equipment for cargo-handling.

### III, 3.2

- The La Dorada water-front (left bank)

Railway stores and oil terminals are situated here alongside deep water. However, for navigation purposes, manoeuvrability is hampered by the high flow-velocities (up to 3 m/s at high water stages). Due to the erosion of the river-bank and the lack of proper maintenance, the water-front is rather dilapidated. There is good accessibility by railway and road, and sufficient storage capacity in the railway stores. There is, however, no equipment for cargo-handling and at present loading and unloading of cargo is carried out by manual labour.



Cargo-handling at La Dorada water-front by night

- The quay wall in Pto. Salgar (right bank)

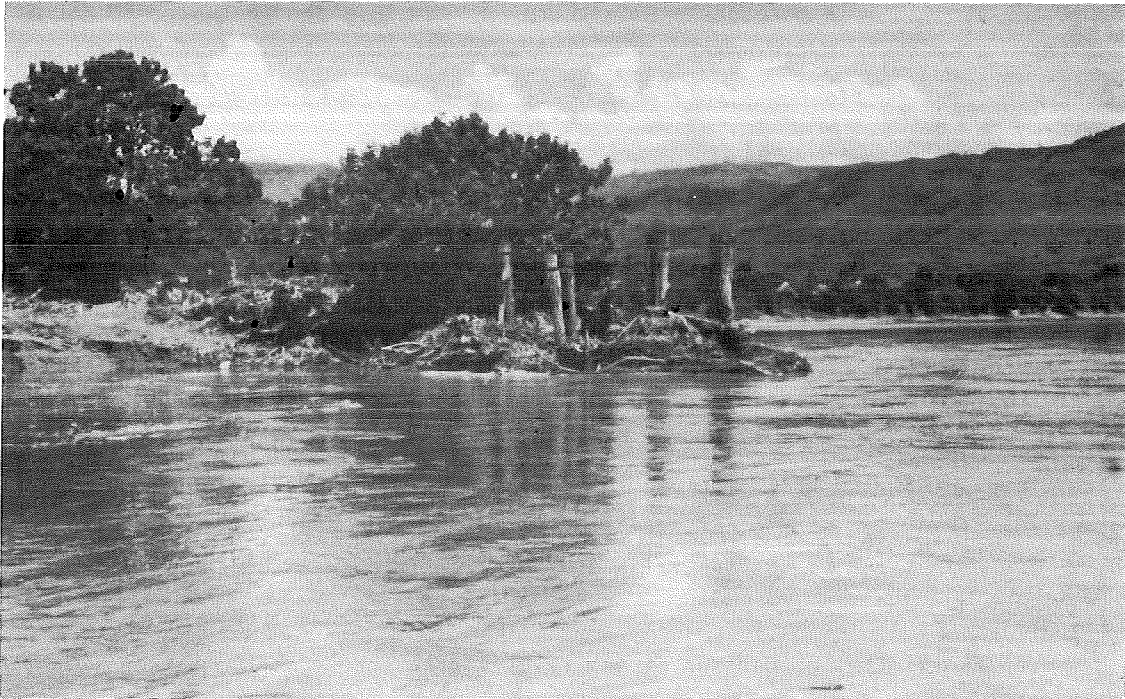
Railway and ADENAVI stores are situated here. In the year 1971, access for river transport was restricted, due to a shoal in front of the quay wall and heavy sedimentation at its upstream end, where an island along the bank was rapidly growing not only into the river itself but also in a downstream direction. After the high water season (April-May) in 1972, the current had partly eroded the shoal in front of the quay wall and accessibility for river transport was improved. There is good accessibility for railway and road, sufficient storage capacity in the stores and equipment for cargo-handling.

In addition to these defects in the existing port facilities (mostly as far as navigation purposes are concerned), two other problems have to be mentioned.

- The protection of La Dorada town (left bank)

The southern side of La Dorada town is situated along an outer bend of the Río Magdalena, upstream of the Vuelta del Conejo. In the past, erosion occurred which endangered the town and future extension areas of La Dorada (projected on the spit of land in the Vuelta del Conejo). To discover means of preventing erosion, a model study was carried out by the Laboratoire Central d'Hydraulique de France in Bogotá

and the advice was given to construct groynes along the left bank of the Rfo Magdalena to divert the main current from the bank and, consequently, to prevent further scour. One groyne was constructed in the beginning of 1960's and although by now this groyne has been almost washed away by the current due to lack of maintenance, the construction of the groyne appeared to be successful at the time and proved its usefulness.



Groyne upstream of La Dorada

- The defence of the air-base German Olano (right bank)

During the high water stages in 1970 and 1971, the downstream end of the sheet-piling, protecting the water-front of the air-base German Olano (situated downstream of the railway bridge along the right bank of the Rfo Magdalena), gave way under the strong attack of the current and about 50 m had to be reinforced by driving a second row of sheet-piling inside the original one.

#### Case history

On the basis of aerial photographs a case history has been compiled of the course of the Rfo Magdalena in the La Dorada-Pto. Salgar region.

The oldest available information dates back to the survey which was carried out by the Julius Berger Konsortium in the years 1922-1924. All the information has been compared with the present situation (aerial photographs taken in May, 1972) (Figure 3.2.7). In the most recent aerial photographs former courses of the Rfo Magdalena in the Vuelta del Conejo can still be clearly distinguished (Figure 3.2.7a). The deposits to the West of this former course are apparently from a still older date.

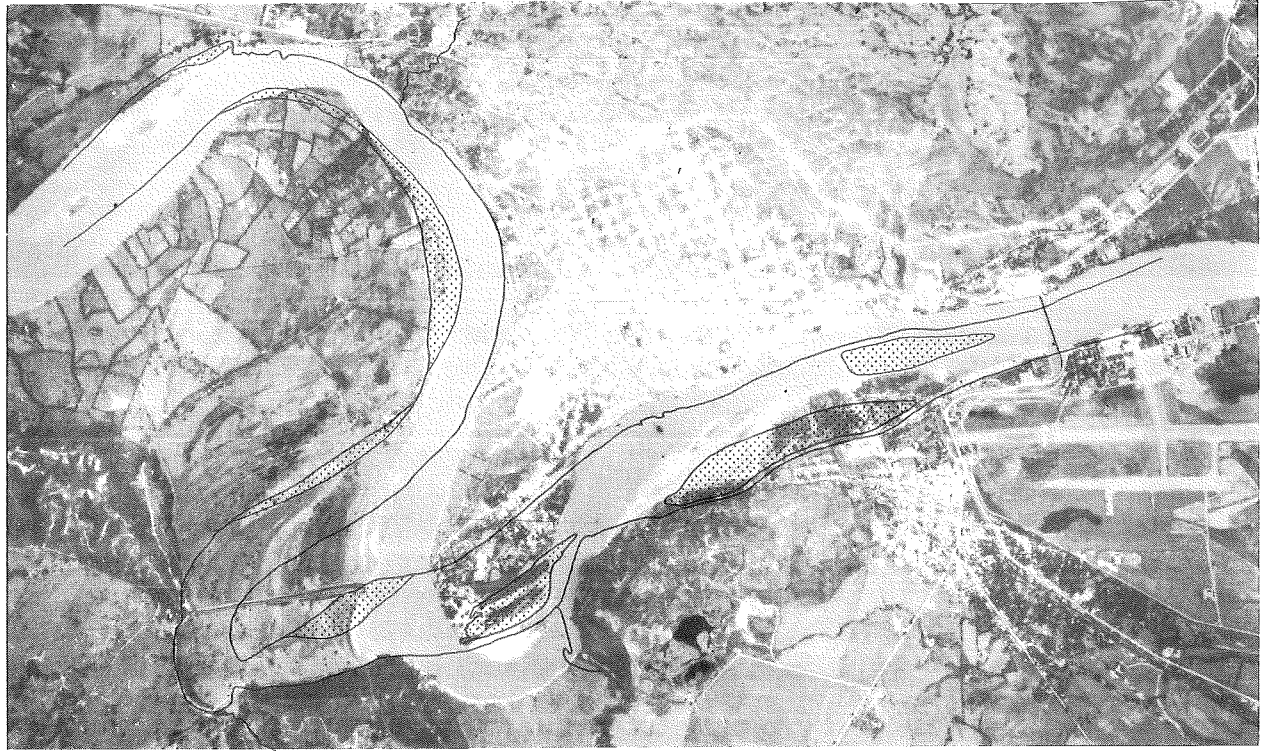
Between the years 1950 and 1957 the Río Magdalena shifted its course at the Vuelta del Conejo, eroding a loop in a more easterly direction (Figures 3.2.7c and d). In the beginning the erosion in the outer bend was considerable (about 20 m per year), but has at present diminished to about 5 m per year (see Figure 3.2.7f). On comparing the present course of the Río Magdalena with the former course of the river before 1950 (Figure 3.2.7c), it will appear that the former course was more favourable as far as navigation purposes and the location of the port facilities are concerned. Formerly, the river flowed downstream of the Vuelta del Conejo in a more or less northerly direction along the water-fronts of La Dorada and Pto. Salgar (and Palanquero) and most likely an adequate access for water transport could have been maintained with simple means for both the La Dorada and Pto. Salgar port facilities.

At present, however, the main current crosses from the right bank in the Vuelta del Conejo to the left bank near the railway stores, locally causing a strong eddying of the current in front of the stores. Further downstream, the current crosses again to the right bank, attacking the sheet-piling of Palanquero (1970-1971). A strong sedimentation in front of the quay wall at Pto. Salgar hampers navigation. In the course of 1972 the crossing of the current shifted upstream and part of the main current again flowed along the quay wall at Pto. Salgar (compare Figures 3.2.11 and 3.2.13). Another disadvantage of the present situation is that the erosion along the outer banks of the Vuelta del Conejo tends in the direction of an even more pronounced loop in future and possibly, unless strong protection works are undertaken, a short-cutting of the river through La Dorada town and the town's extension area.

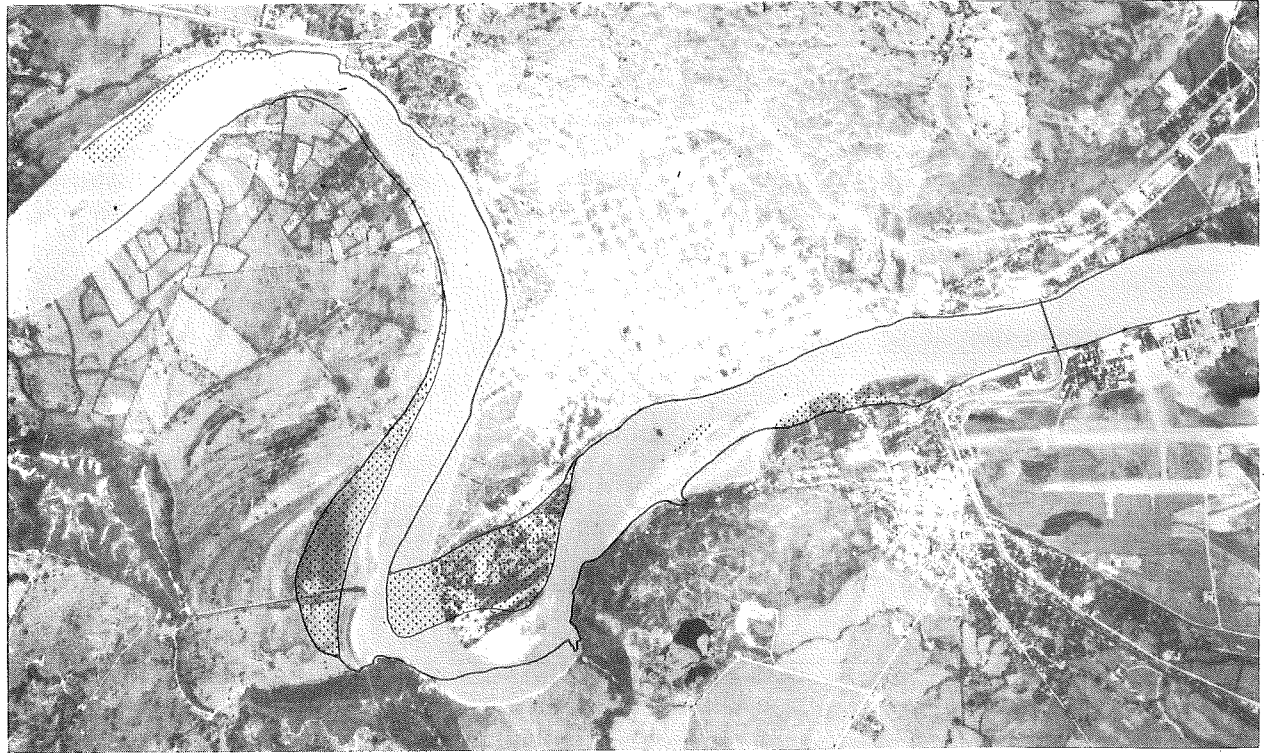
Apart from the comparison of the river courses presented in Figure 3.2.7, the elevation of the river bed-level can also be compared from different soundings. In Figure 3.2.8 the sounding of 1963 is given. (This sounding was originally prepared by ADENAVI and the depth contour-lines were given in relation to M.S.L. To enable this sounding to be compared with data of a later date, the sounding has been related to L.R.L. by the Mission). Although there are no flow-lines available, the direction of the current can be assumed to be more or less parallel to the depth contour-lines. It can be seen that the quay wall at Pto. Salgar was only accessible at high water stages, while the shoal in front of the La Dorada water-front probably caused even higher flow-velocities along the left bank than at present.

As in 1970 the situation in front of the quay wall at Pto. Salgar hardly differed from the situation as given in Figure 3.2.8, it was decided by ADENAVI to move the dredger DH 9 to this area in August 1970 to dredge the approach and the area in front of the quay wall at Pto. Salgar. The spoil was to be used to make a connection between the right bank of the Río Magdalena upstream of the quay wall and the island in front of the quay wall. By means of such a spur-dike a more or less protected harbour basin would have been created, accessible only from a downstream direction. A mechanical breakdown of the dredger, however, prevented the execution of this scheme.

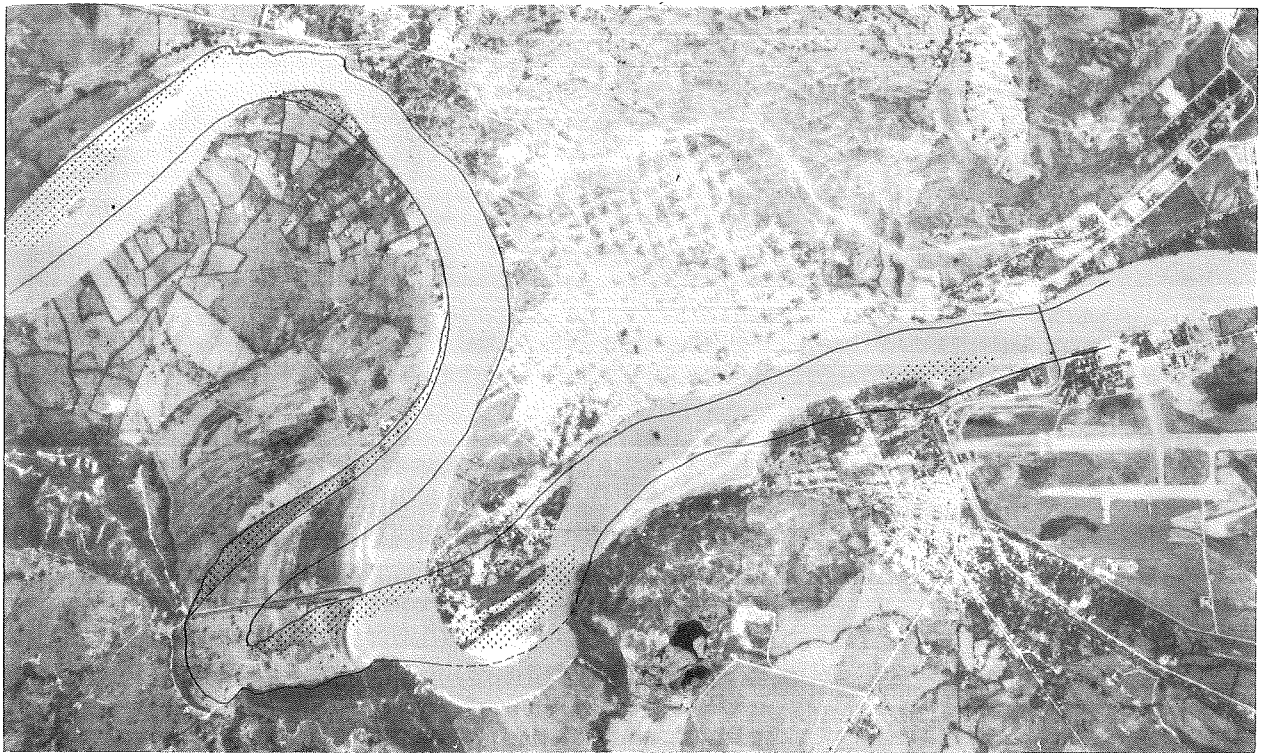
In Figure 3.2.9 the sounding is given of July 1971 in front of the quay wall at Pto. Salgar. In the Figures 3.2.10 and 3.2.11 the soundings and flow-lines are given of September-November 1971. It can be seen in Figure 3.2.11 that the dredging in front of Pto.



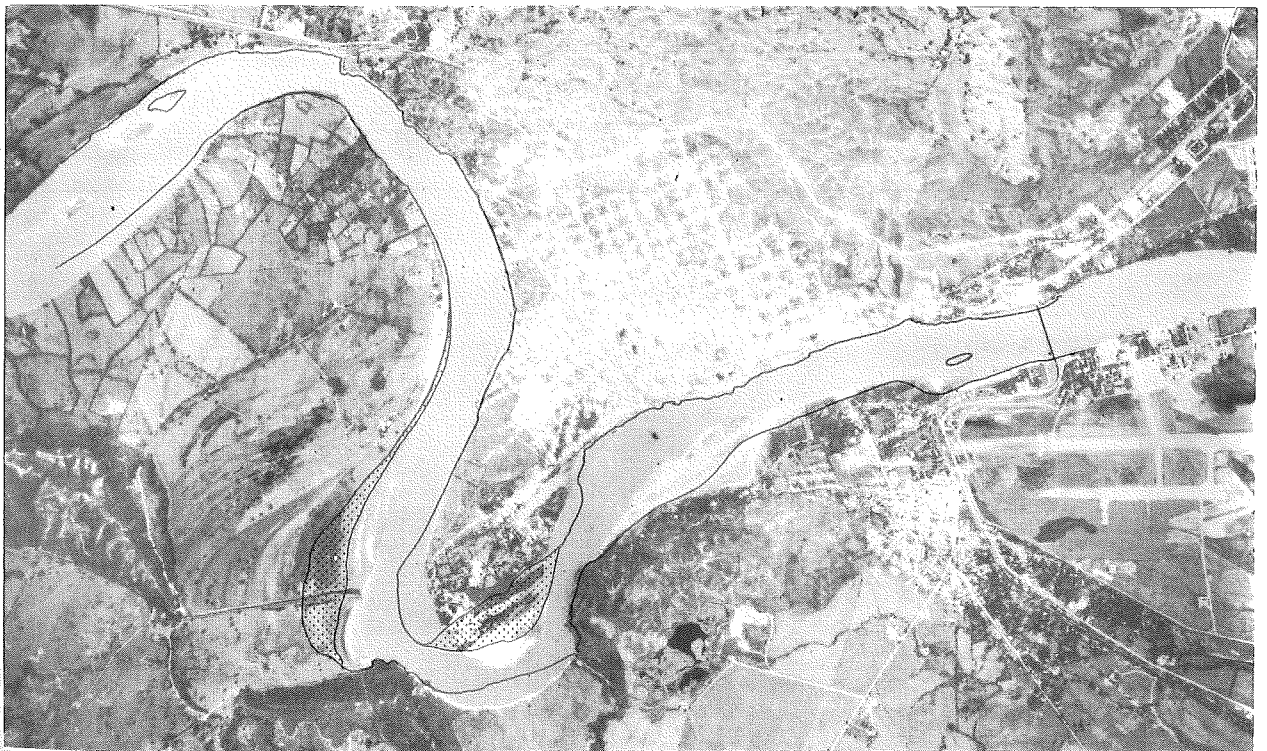
a 1.924



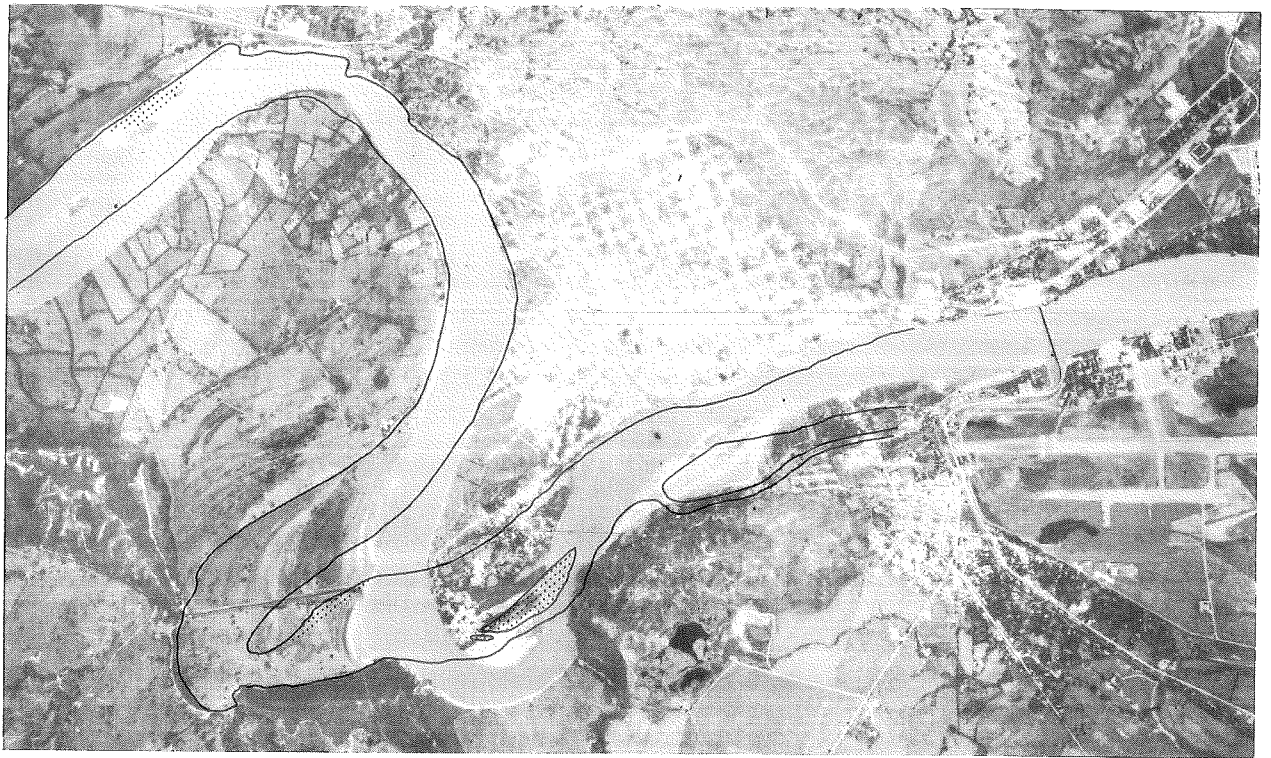
d 1.957



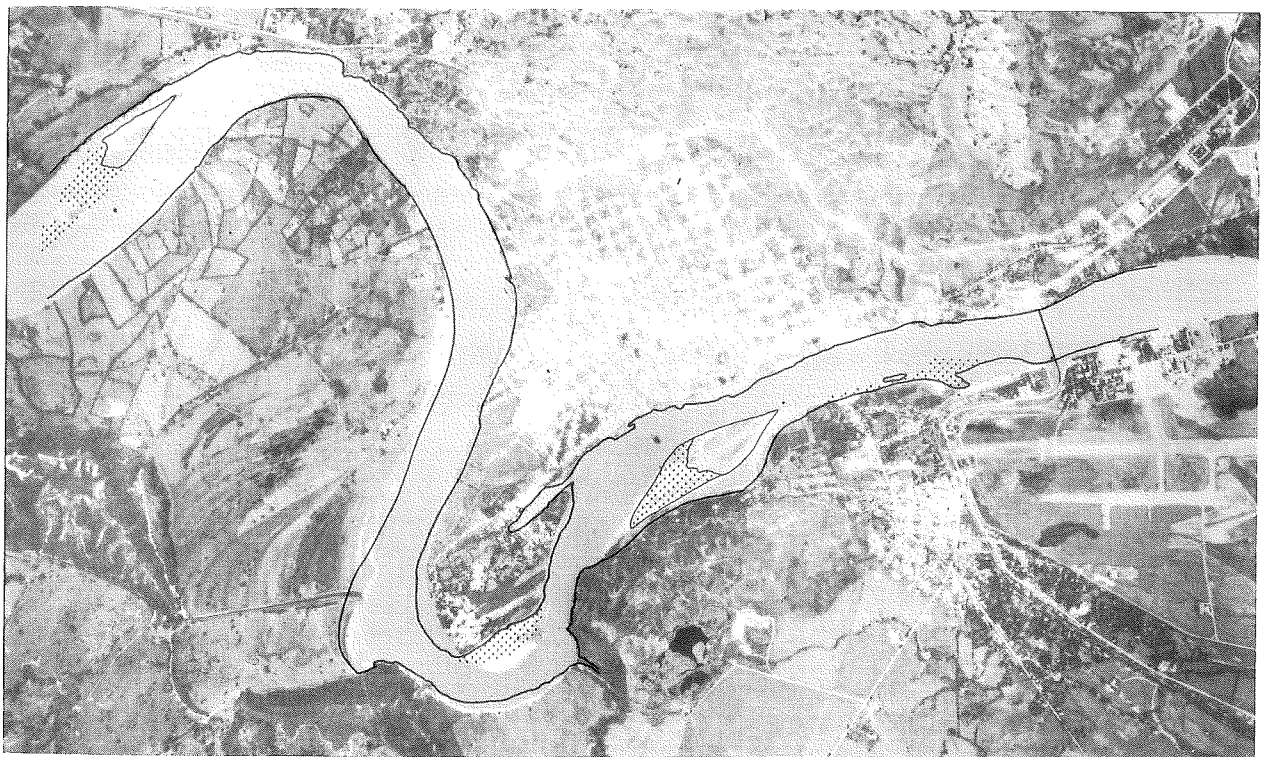
b 1,948



e ±1967



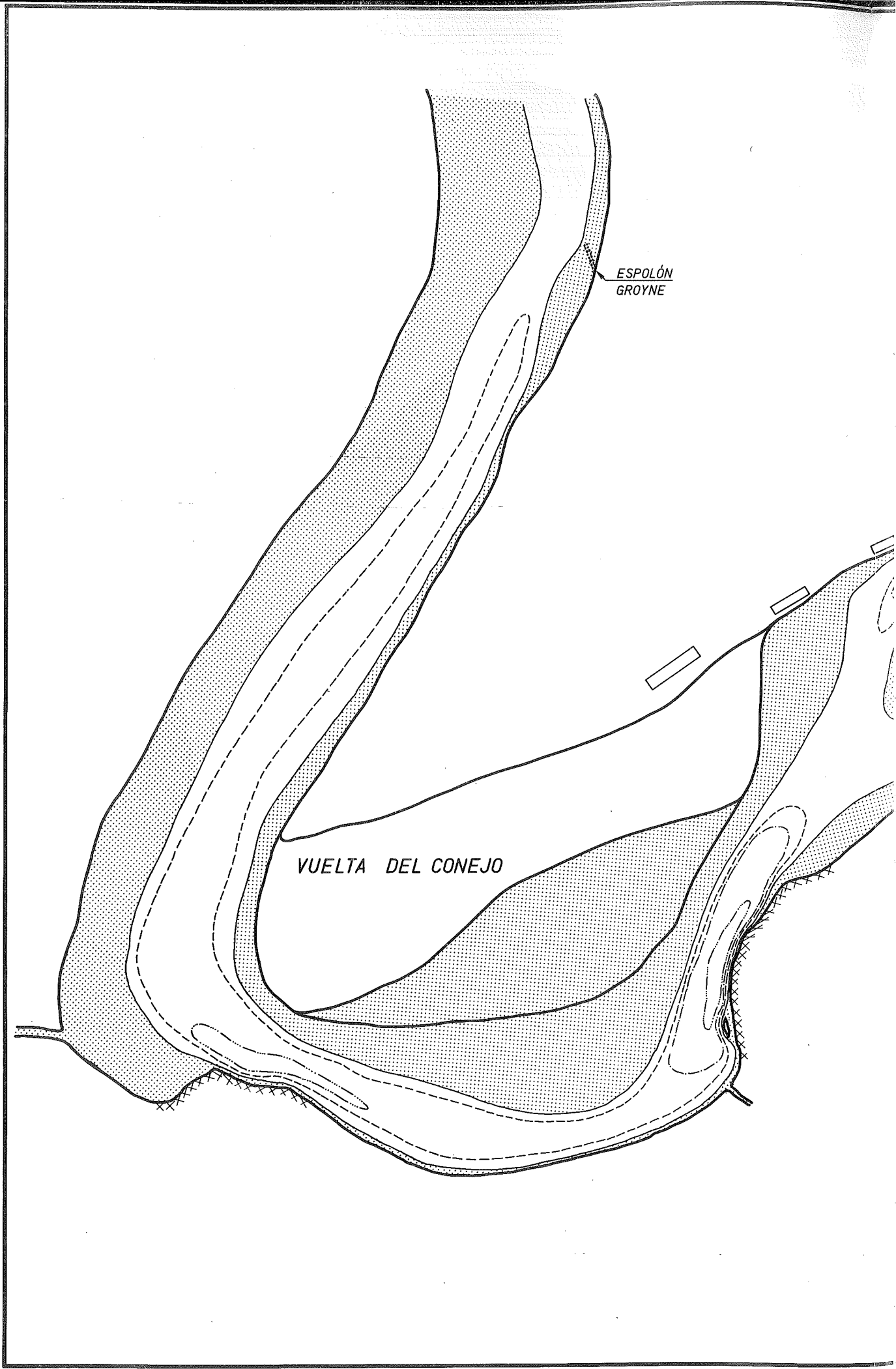
c 1950



f 1969

CASE HISTORY OF THE RÍO MAGDALENA NEAR LA DORADA

FIG. 3.27

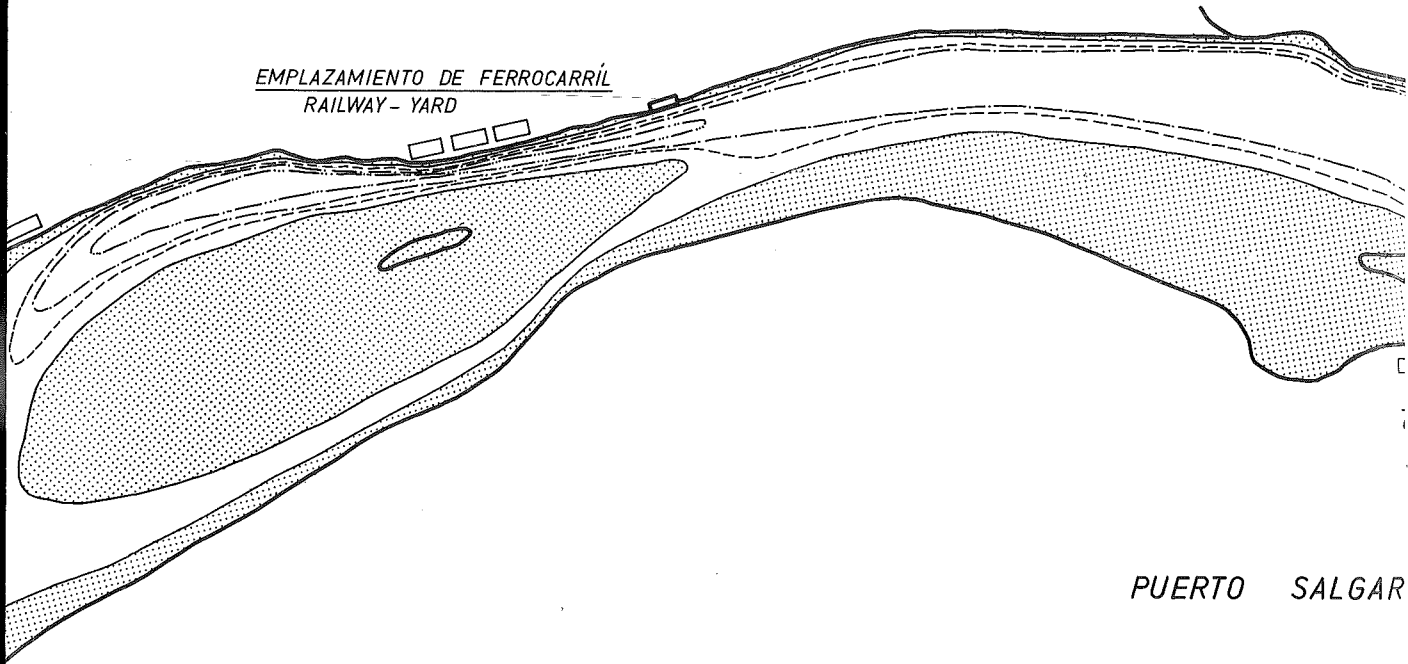


ESPOLÓN  
GROYNE

VUELTA DEL CONEJO

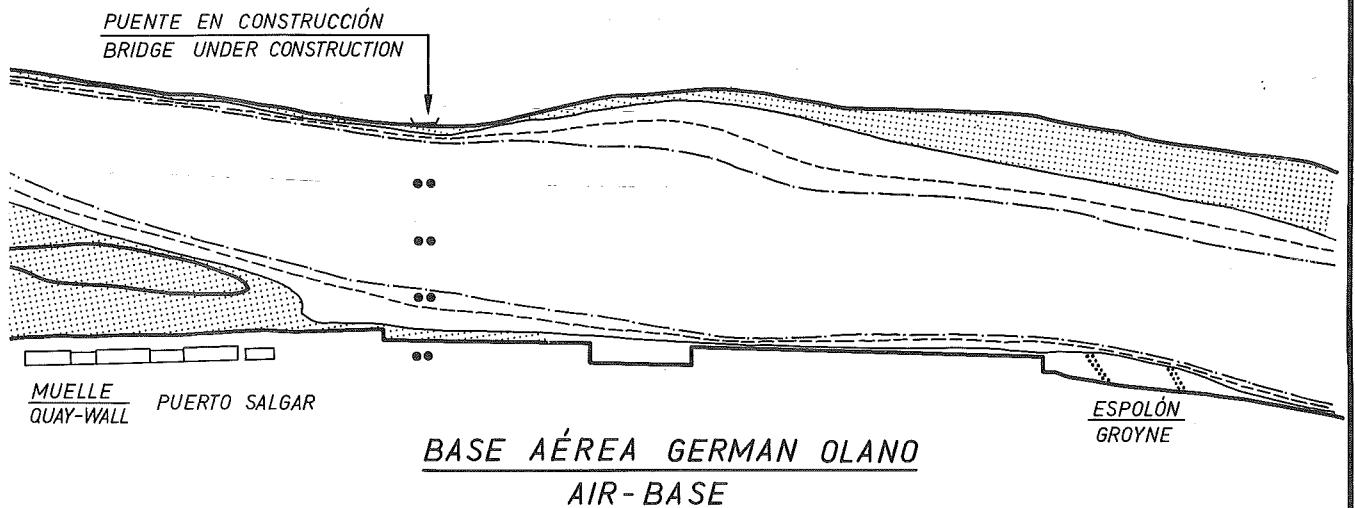
LA DORADA

EMPLAZAMIENTO DE FERROCARRIL  
RAILWAY - YARD

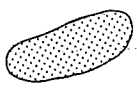


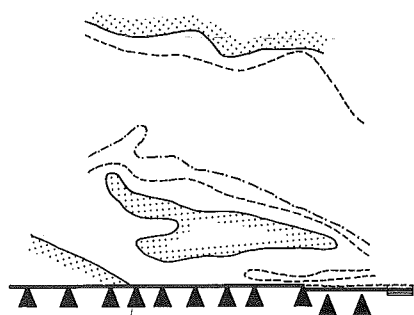
PUERTO SALGAR

S



R

<u>SONDEO</u> SOUNDING	<b>RÍO MAGDALENA</b>	<u>CERCA</u> NEAR	<b>LA DORADA-PUERTO SALGAR</b>
ESCALA/SCALE 1:10.000			
FECHA/DATE 26-X-1963 (SONDEO ORIGINAL RELACIONADO AL NIVEL DEL MAR, HECHO POR LA SECCIÓN TÉCNICA DE ADENAVI)			
ORIGINAL SOUNDING RELATED TO MSL., MADE BY THE TECHNICAL DEPARTMENT OF ADENAVI			
NIVEL DE REDUCCIÓN: 168 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PUERTO SALGAR			
CHART DATUM: 168 metres ABOVE ZERO OF THE PUERTO SALGAR GAUGE			
<u>CURVAS ISOBATAS</u> DEPTH CONTOURS		0 m ----- 1.5 m - - - - - 2.5 m . . . . . 5 m - - - - - 7.5 m - - - - - 10 m - - - - -	XXXXXXXX  <u>ROCAS</u> ROCK
		<u>PLAYA, SOBRE EL DATUM</u> DRY, ABOVE DATUM	
NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT			FIG. 3.2.8



PUERTO SALGAR

SONDEO  
SOUNDING

RÍO MAGDALENA

CERCA  
NEAR

LA DORADA-PUERTO SALGAR

ESCALA/SCALE 1:10.000

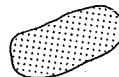
FECHA/DATE 9, 10 - VII - 1971

NIVEL DE REDUCCIÓN: 1.68 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PUERTO SALGAR

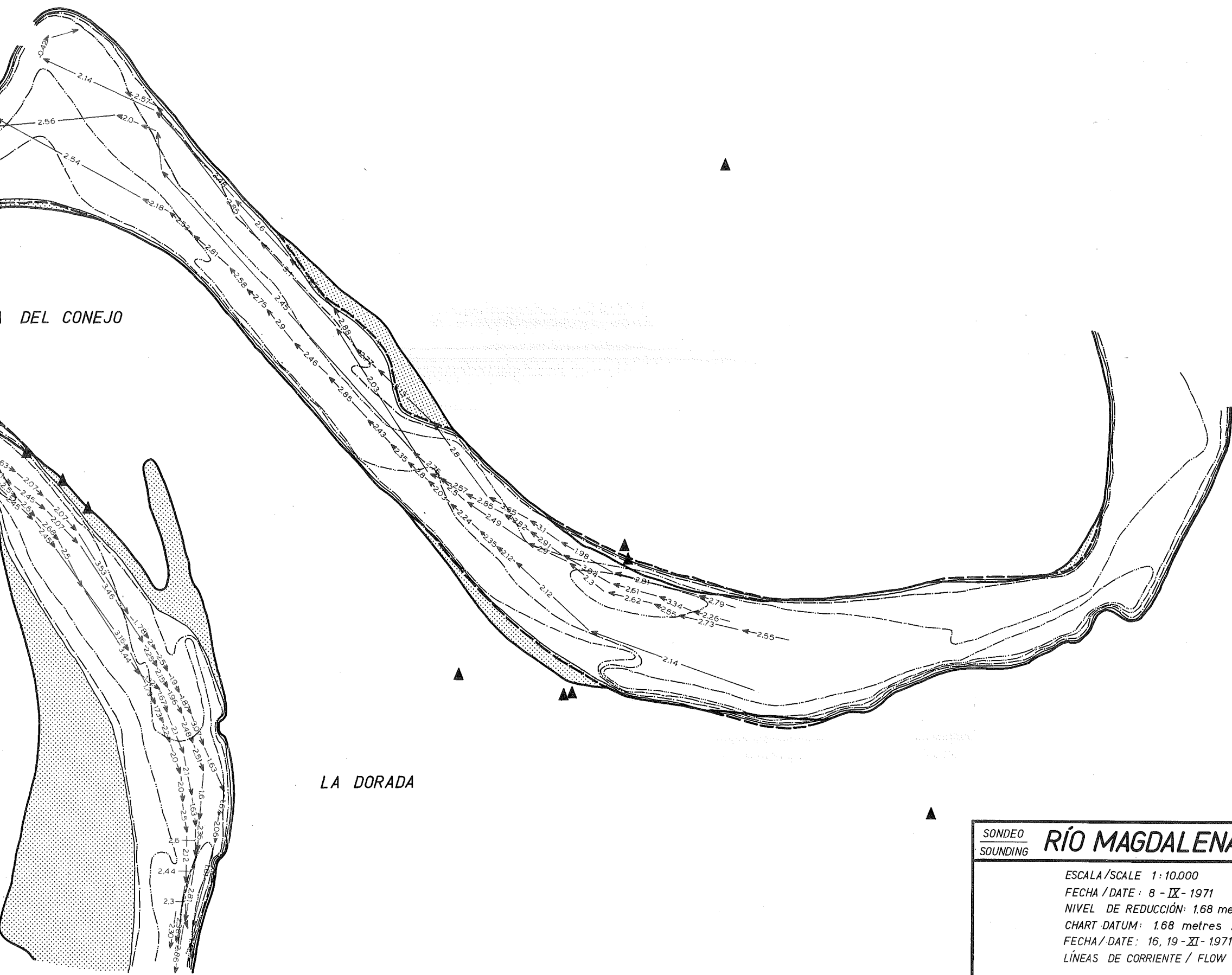
CHART DATUM: 1.68 metres ABOVE ZERO OF THE PUERTO SALGAR GAUGE


CURVAS ISOBATAS  
DEPTH CONTOURS

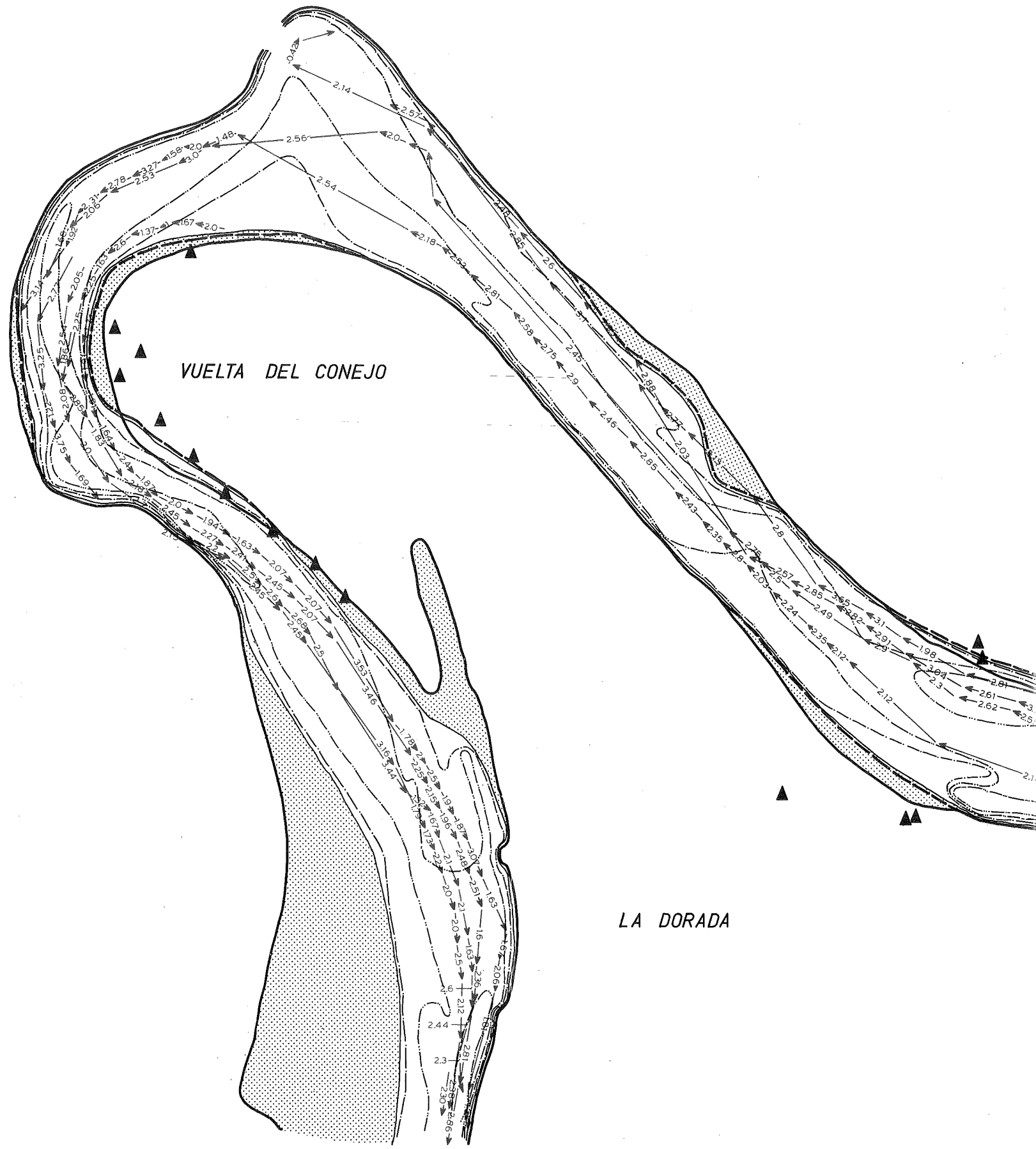
————— 0 m  
- - - - - 1.5 m  
- · - · - 2.5 m

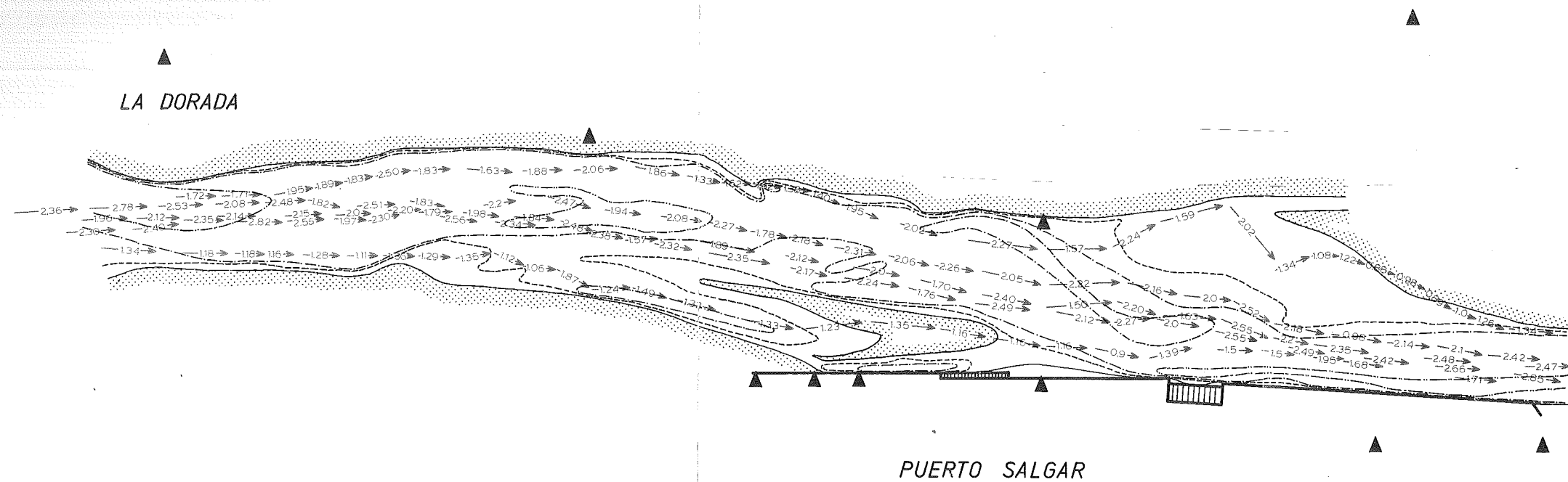


PLAYA, SOBRE EL DATUM  
DRY, ABOVE DATUM



SONDEO SOUNDING	<b>RÍO MAGDALENA</b>	CERCA NEAR	<b>LA DORADA-PUERTO SALGAR</b>
ESCALA/SCALE 1:10.000 FECHA/DATE: 8 - IX - 1971 NIVEL DE REDUCCIÓN: 1.68 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PUERTO SALGAR CHART DATUM: 1.68 metres ABOVE ZERO OF THE PUERTO SALGAR GAUGE FECHA/DATE: 16, 19 - XI - 1971 NIVEL DE AGUA/WATERLEVEL: 1.50 m SOBRE EL DATUM/ABOVE DATUM LÍNEAS DE CORRIENTE / FLOW LINES — 1:10 —→ VELOCIDAD EN m/seg. / VELOCITY IN m/s			
CURVAS ISOBATAS DEPTH CONTOURS		0 m 1.0 m 2.5 m 5.0 m	 PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM





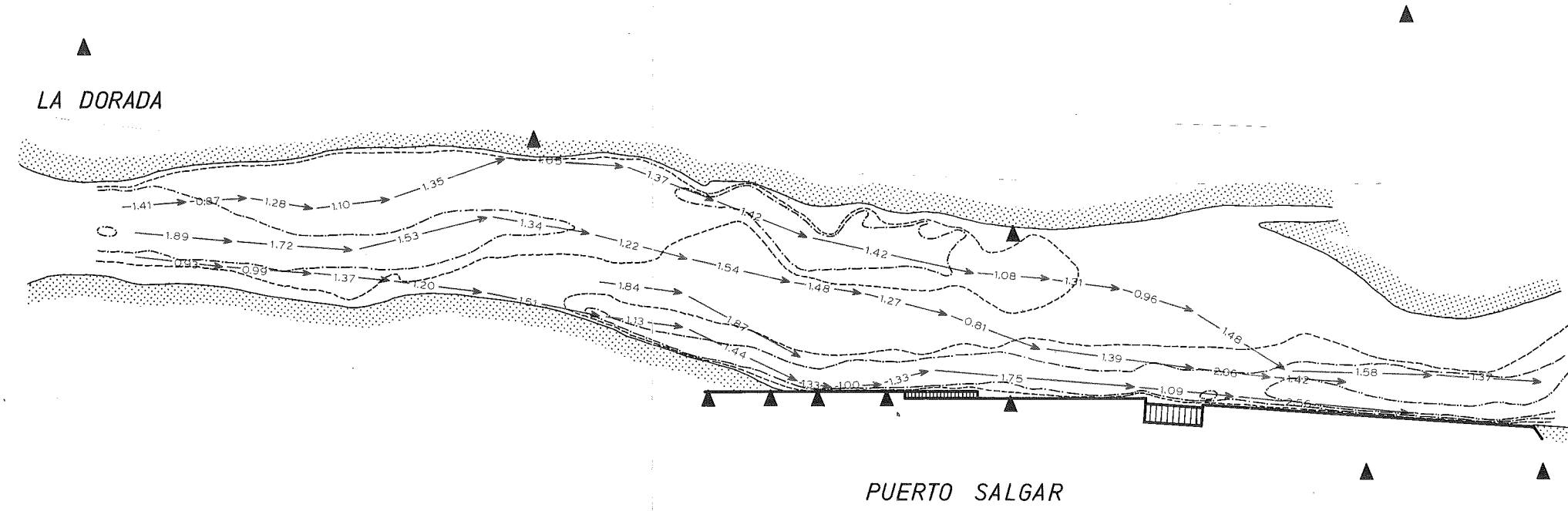
SONDEO / SOUNDING **RÍO MAGDALENA** CERCA / NEAR **LA DORADA - PUERTO SALGAR**

ESCALA / SCALE 1:10,000  
 FECHA / DATE 17 - XI - 1971

NIVEL DE REDUCCIÓN: 1.68 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PUERTO SALGAR  
 CHART DATUM: 1.68 metres ABOVE ZERO OF THE PUERTO SALGAR GAUGE

FECHA / DATE: 16, 19 - XI - 1971 NIVEL DE AGUA / WATERLEVEL: 1.23 m SOBRE EL DATUM / ABOVE DATUM  
 LÍNEAS DE CORRIENTE / FLOW LINES — 1.18 —> VELOCIDAD EN m/seg / VELOCITY IN m/s

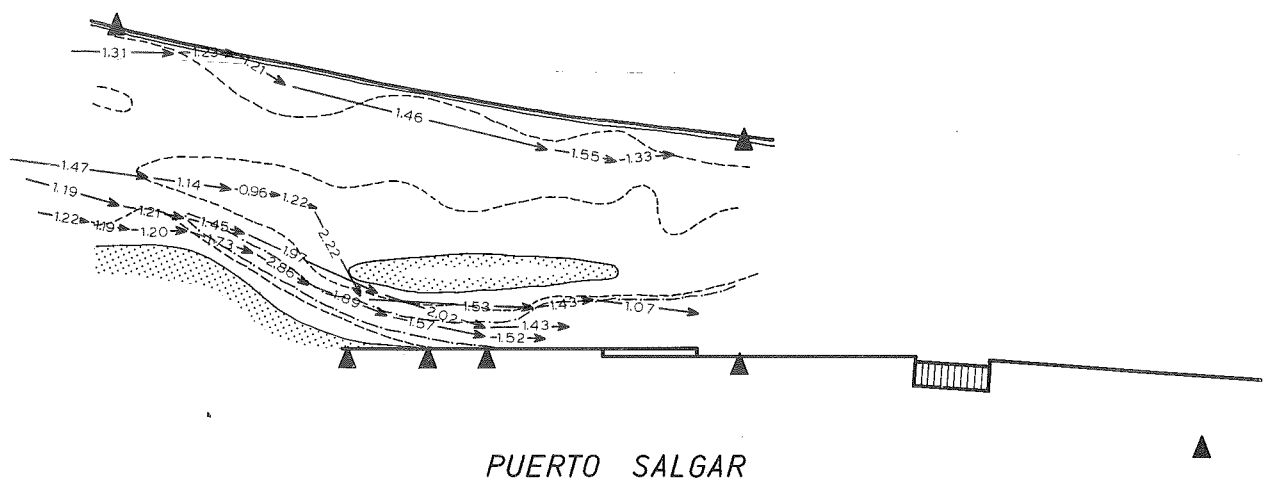
CURVAS ISOBATAS	-----	0 m	PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
DEPTH CONTOURS	-----	1.5 m	
	-----	2.5 m	
	-----	5 m	



SONDEO / SOUNDING **RÍO MAGDALENA** CERCA / NEAR **LA DORADA - PUERTO SALGAR**

ESCALA / SCALE 1:10.000  
 FECHA / DATE 31 - VIII - 1972  
 NIVEL DE REDUCCIÓN: 1.68 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PUERTO SALGAR  
 CHART DATUM: 1.68 metres ABOVE ZERO OF THE PUERTO SALGAR GAUGE  
 FECHA / DATE: 31 - VIII - 1972 NIVEL DE AGUA / WATERLEVEL: 0.65 m SOBRE EL DATUM / ABOVE DATUM  
 LÍNEAS DE CORRIENTE / FLOW LINES — 1.18 —> VELOCIDAD EN m/seg. / VELOCITY IN m/s

CURVAS ISOBATAS	————— 0 m		PLAYA, SOBRE EL DATUM
DEPTH CONTOURS	- - - - - 1.5 m		DRY, ABOVE DATUM
	————— 2.5 m		
	————— 5 m		



SONDEO  
SOUNDING

**RÍO MAGDALENA**

CERCA  
NEAR

**LA DORADA-PUERTO SALGAR**

ESCALA/SCALE 1:10.000

FECHA/DATE 12-X-1972

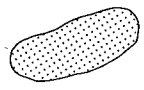
NIVEL DE REDUCCIÓN: 1.68 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PUERTO SALGAR

CHART DATUM: 1.68 metres ABOVE ZERO OF THE PUERTO SALGAR GAUGE

FECHA/ DATE: 11, 12-X-1972 NIVEL DE AGUA / WATERLEVEL: 0.20m SOBRE EL DATUM / ABOVE DATUM

LÍNEAS DE CORRIENTE / FLOW LINES — 1.18 —> VELOCIDAD EN m/seg. / VELOCITY IN m/s

CURVAS ISOBATAS ——— 0 m  
DEPTH CONTOURS - - - - - 1.5 m  
                                  - · - · - 2.5 m



PLAYA, SOBRE EL DATUM  
DRY, ABOVE DATUM

Salgar provided a better access to the quay wall, although the channel is still very narrow. In the Figures 3.2.12 and 3.2.13 the soundings and flow-lines near Pto. Salgar are given of August and October 1972 respectively.

When the situation near Pto. Salgar in 1971 (Figure 3.2.11) is compared with the situation in October 1972 (Figure 3.2.13), it can be seen that the crossing from the left bank of the Río Magdalena to the right bank shifted upstream, and that at present at least part of the main current flows along the quay wall at Pto. Salgar, although the island (consisting of pebbles) in front of the quay wall is still present. (According to Figure 3.2.12, this island would be well below L.R.L. in August 1972; however, this must be regarded with suspicion).

#### Proposed river improvement

The river improvement which is required in the La Dorada - Pto. Salgar area can be characterized as:

##### - Improvement for navigation purposes

To provide a better approach to the Pto. Salgar quay wall and an easier manoeuvrability along the La Dorada water-front. In this respect, it must be mentioned that in the present situation (1973), the total cargo destined to or shipped from this area can easily be handled along the Pto. Salgar quay wall and the La Dorada water-front. For the time being it is advised that other uses be found for the surplus capacity in port facilities represented by the stores situated inland on the south-eastern side of La Dorada town. Unless other cargo sources can be found, the present quantities of cargo handled in this area do not justify the cost required to open up and maintain the accessibility of these stores. It is considered advisable to concentrate the port activities at the Pto. Salgar quay wall and the La Dorada water-front. The proposed scheme for the river improvement is also based on this principle. (However, as in future the port facilities of the ADENAVI stores may again be required, the scheme for the improvement should be so flexible that a proper access for river transport can then still be provided).

##### Improvement for protection purposes

This would be to provide for a proper defence of the southern and eastern sides of La Dorada town and the air-base Palanquero.

These purposes can be only partly fulfilled in the present topography (1973). It has already been mentioned that for navigation purposes only, the situation as it was during the survey of the Julius Berger Konsortium appears to have been more favourable, while for the protection of La Dorada a smoother course of the Río Magdalena is required. The combination of these requirements tends to the situation as present in about 1957 (see Figure 3.2.7d). Because of the fact that in this situation the meander in the Vuelta del Conejo is not as pronounced as in the past (1920 - 1950) or at present (1969 and onwards), the river-works can be maintained at lower cost, while downstream of the La Dorada water-front the main current can be diverted to the right bank (e.g., by means of groynes) to prevent sedimentation in front of the Pto. Salgar quay wall.

If, however, less attention could be paid to the navigational requirements, the intervention in the course of the Rfo Magdalena would be smaller and defence of La Dorada town and Palanquero can be pursued by protection of the present course. If therefore the development of a new port area near Pto. Triunfo is considered to replace the existing port facilities in La Dorada and Pto. Salgar, an alternative and cheaper solution for the La Dorada - Pto. Salgar area will be possible. Moreover, the yearly maintenance dredging in the Rfo Magdalena will be reduced. An alternative solution for the improvement of the La Dorada - Pto. Salgar region is drawn up in Para. 3.2.6.

Before the proposed scheme of river improvement is dealt with, a final remark must still be made. A total design of river-works is generally accompanied by model studies. To some extent such river-works can be designed from the knowledge of the river under consideration and on the base of experience. However, the extent to which the river-works have to be carried out (e.g., the length of a bank protection) and what type of river-works (e.g., groynes or spur-dikes, etc.) will serve the purpose best, needs to be found out from model tests. Therefore, the proposed scheme must be seen only as a first design of the river improvement, whereas its details and general applicability need to be confirmed by tests in the laboratory.

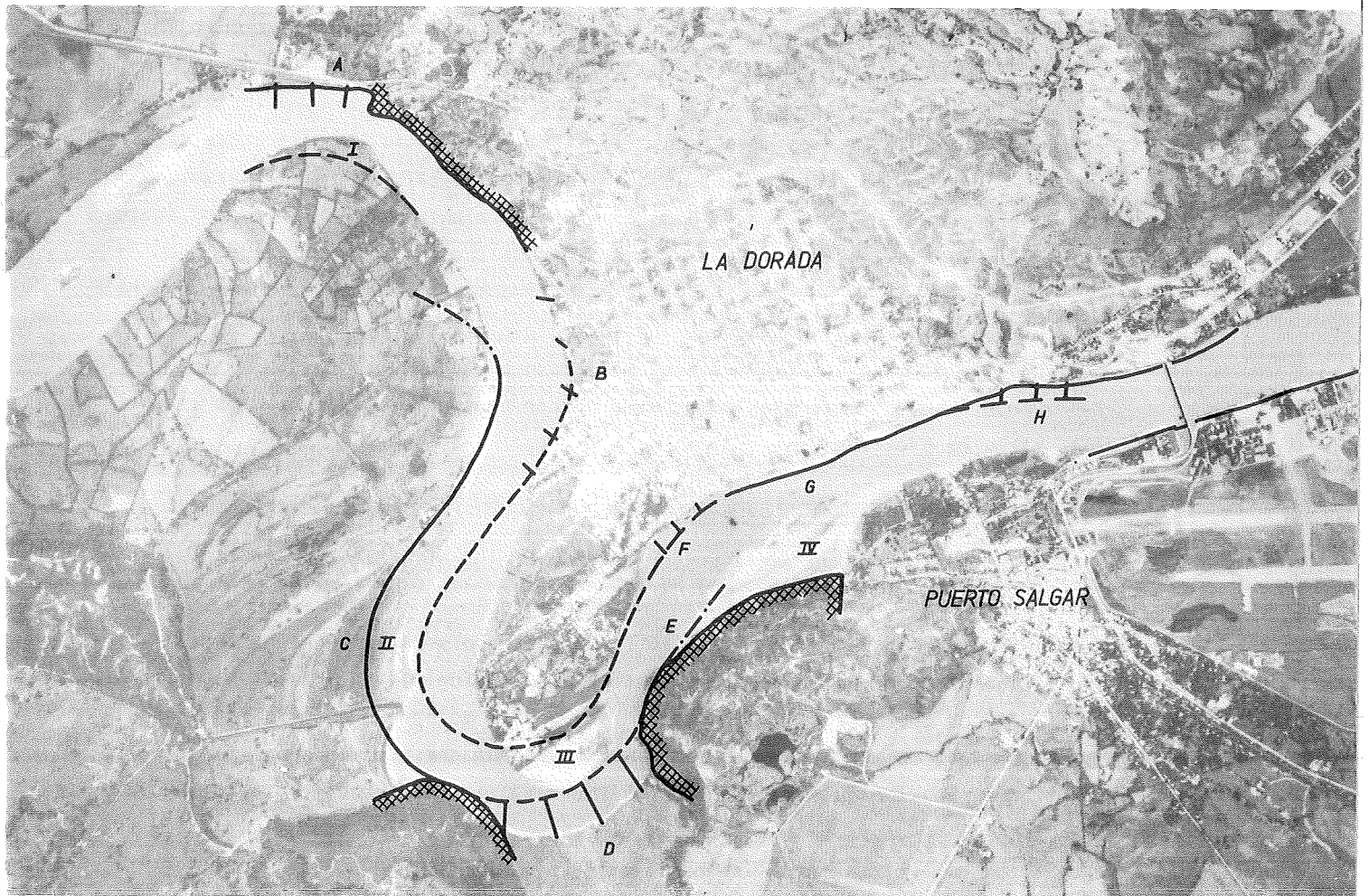
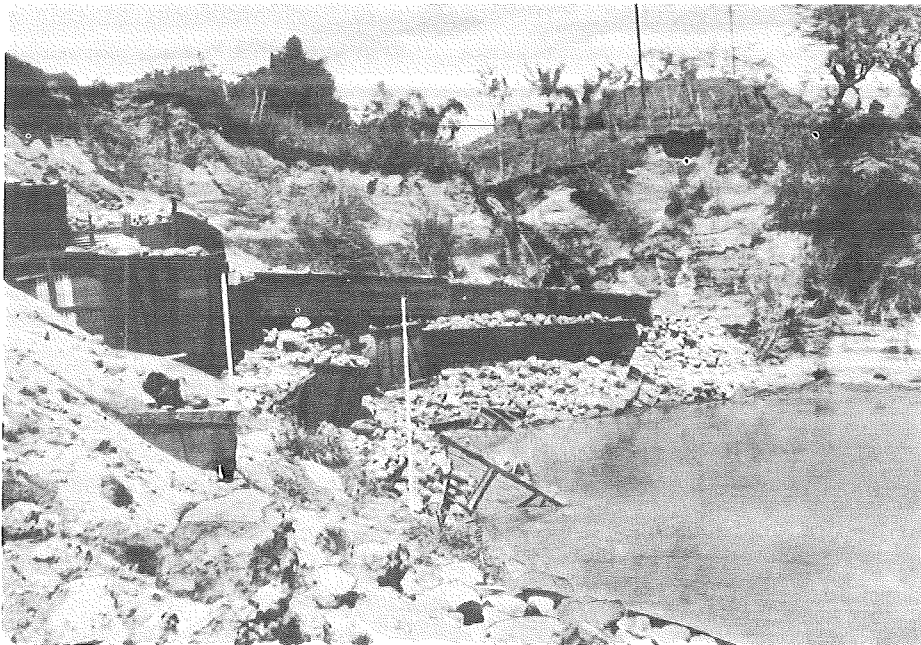


Figure 3.2.14 Proposed River Improvement near La Dorada - Pto. Salgar

The proposed scheme for river improvement in the La Dorada - Pto. Salgar area is presented in Figure 3.2.14. As a reference, the present plan-form of the Río Magdalena is also given. The various sites which need to be improved are listed below (in a downstream direction):

A. River-bend to the south-west of La Dorada

At present this outer bend is deteriorating rapidly. Upstream of the rock the turbulent flow in the eddy is causing erosion of the river-bank. The railway-line connecting La Dorada and Honda may be endangered by this erosion of the bank and therefore the FCN has constructed locally a bank protection, consisting of old railway waggons filled with boulders.



Providing that such a protection is constructed with permeable waggons (see remarks in Para. 2.5.2 regarding the filter of a protection) and the waggons themselves are placed in a stable position along the slope of the bank, the protection may well serve its purpose. However, it is felt by the Mission that a smoother flow in the outer bend is also required.

For this reason, three groynes are projected along the outer bend, each about 200 m apart. Part of the inner bend (I) must then be removed, either by the river itself or by means of dredging.

B. River-bend to the south of La Dorada

The model study carried out by the Laboratoire d'Hydraulique de France in Bogotá showed that the erosion in this bend could be prevented by the construction of a number of groynes. Only one has been constructed by MOP, and although at present this groyne needs urgent repairs, it has proved its usefulness against erosion in the past years.

To prevent future erosion and to guide the flow downstream in a somewhat southerly direction, four more groynes are projected, each again about 200 m apart.

C+D+E The Vuelta del Conejo

The right bank of the Rfo Magdalena in the upstream part of the Vuelta del Conejo (C) consists mostly of fine alluvial deposits which have sedimented since about 1950 when the river gradually changed its course to the more or less present situation. (However, in the river-bed also gravel and stones were encountered by the Mission). The bank-level lies mostly above the high water stages, although the most recent deposits are still low. In the outer bend at D the right bank of the river is about 13 m above L.R.L. Due to the strong meandering of the river this bank is still eroding (about 5 m per year, although this is less than in the past). On both sides of D the river-bank is protected by rock.

The bank protection (C) and the groynes (D) projected in the outer bend of the Vuelta del Conejo have a key-function in the complete river improvement of this area. On the one hand, the bank protection, in combination with the groynes mentioned under B, fixes the course of the river somewhat to the south of the present course, while on the other hand the guidance of the flow initiated by the bank protection and followed up by the protection at D will determine the angle of attack of the current on the water-front at La Dorada (although some correction is still possible at F).

The bank protection (C), however, also determines the type and the extent of the protection required at D. Although in Figure 3.2.14 four groynes are indicated at D to continue the guidance of the flow, it may also be possible that the guidance of the flow at C is such that the main current crosses directly to the rock just downstream of D. In that case an eddy will be generated by the flow along the outer bend, and in view of the fact that the flow-velocity in the eddy will be considerably lower than in the main channel, only a relatively light protection of the right bank at D may then suffice. It should be recalled that on the basis of the model study the Laboratoire d'Hydraulique de France also advised the construction of a number of groynes.

The spur-dike (E), provisionally indicated in Figure 3.2.14, provides for the crossing of the current from the right bank in the Vuelta del Conejo to the left bank in front of La Dorada town. Every means must be used to prevent the main channel remaining along the right bank, as this would probably produce sedimentation in front of the La Dorada water-front. However, in view of the foregoing considerations regarding the required guidance of the flow in the Vuelta del Conejo, it will be clear that only a model study can provide the correct answer if a further guidance of the flow at E will at all be required.

Remark 1: The excavation of the shoal opposite La Dorada (indicated in Figure 3.2.14 by IV) by means of drag-lines by the FCN must be mentioned. It is advised that in future all the issues of such leases will fall under the authority of a River Conservancy Department, because such schemes can have far-fetching consequences. For example, this excavation could result in a decrease of the water depth along the La Dorada water-front. As a consequence, the construction of a spur-dike or groyne at E may then be required.

Remark 2: In view of the importance of the protection in the outer bend of the Vuelta del Conejo and the availability of two short stretches where the outer bank is fixed (rock) in the present situation, the application of Eq. 3.7.10 presented in Part II, Para. 3.7.3, to compute the radial bed-level slope has also been tested on the measured bed-level slope in the present situation. From the sounding made in 1971 (Figure 3.2.10) the average profile in front of the two rocks has been drawn (Figure 3.2.15). Then by using:

$$\frac{dh}{dr} = \frac{A I_o R_o}{\Delta D} \cdot \frac{h^2}{R^2} \quad (\text{Part II, 3.7.10})$$

the radial bed-level slope can be computed. The following values have been used for this computation:

$$A = 10$$

$$I_o = 60 \times 10^{-5} \text{ (an average water-level gradient determined from the available records of water-levels at the Arrancaplumas and Pto. Salgar gauges)}$$

$$R_o = 400 \text{ m (from the aerial photographs of May, 1972); and}$$

$$D = 10,000 \mu\text{m (this is a rather rough estimate, because no bed-samples are available of this area).}$$

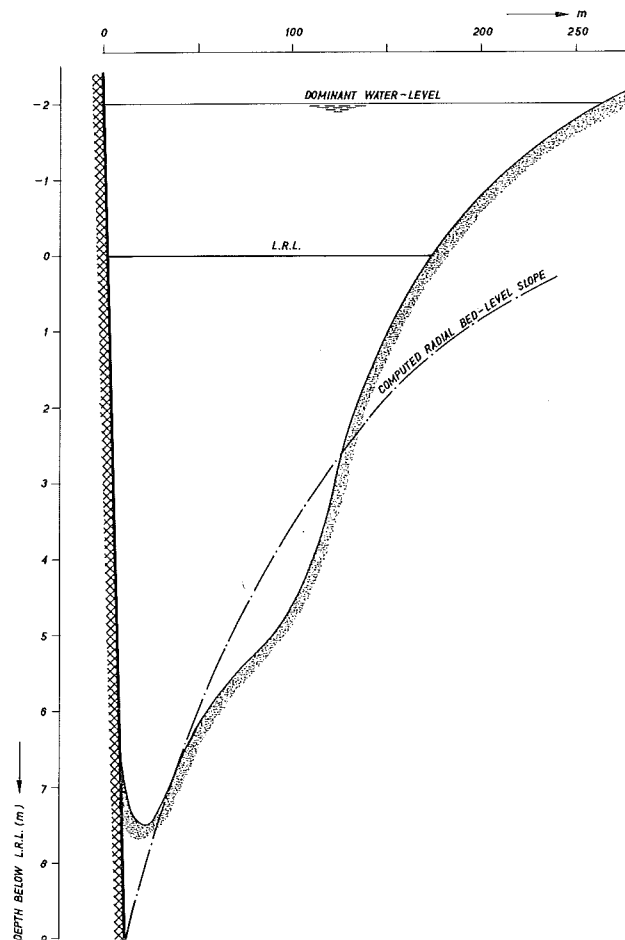


Figure 3.2.15 Computed Cross-section in the Vuelta del Conejo

### III, 3.2

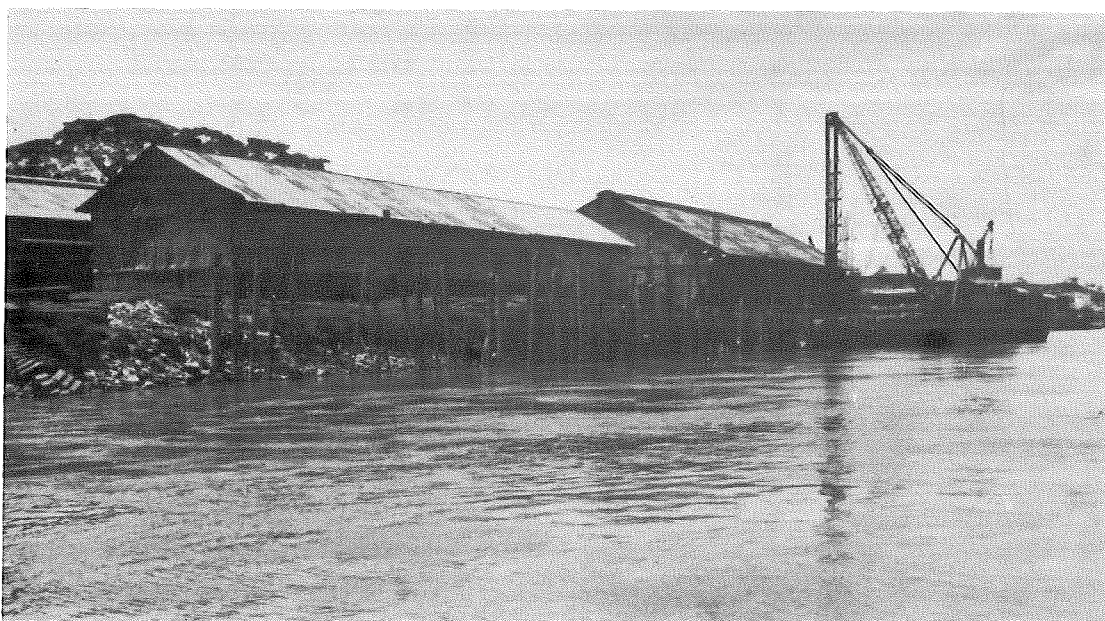
Close to the outer bend the computed bed-level slope agrees well with the measured one, and it can therefore be concluded that for a selected bend-radius the depth in front of a bank protection can be computed by means of Eq. II. 3.7.10 with reasonable confidence.

#### F. River-bend to the east of La Dorada

The protection of the La Dorada water-front needs to be extended somewhat in a southerly direction. For this purpose three groynes are schematically indicated in Figure 3.2.11. Additional advantages of this protection are a smoother water-front and a more uniform distribution of the attack by the current on the left bank. For example, in the present situation (1973) the most upstream end of the railway-yard is under strong attack by the current and the eddy which is generated by the flow upstream is eroding the left bank.

#### G. The La Dorada water-front

A scheme for the temporary improvement of the La Dorada water-front was initiated in 1972 by M.O.P. and ADENAVI.



Temporary Improvement of the La Dorada Water-front

The wooden piles will be connected by a concrete beam on top. However, because of the high flow-velocities, it is felt that this type of construction will be too light to give an effective improvement.

It seems to be possible that in the proposed scheme the main flow may separate from the La Dorada water-front so that the crossing of the channel to the right bank already occurs in front of La Dorada instead of more downstream (at H). In that case, the opposite (right) bank should also be protected (the construction of groynes over the foreshore at IV may also be considered, but will probably require higher investments).

#### H. Provisions for the Pto. Salgar quay wall

Downstream of the La Dorada water-front the main current should cross from the left bank to the right bank to provide for deep water along the Pto. Salgar quay wall. It has already been mentioned that in 1972 this crossing shifted in a more upstream direction which improved the approach to and the available water depth in front of the quay wall.



Pto. Salgar Quay Wall

However, this is not a stable situation (which can only be reached if a guidance of the flow at H (see Figure 2.3.14) is provided for, but whether the three groynes indicated in the figure will indeed be required needs to be answered by model tests.

A final remark must be made regarding the air-base Palanquero. The present sheet-piling will provide a proper defence against erosion by the current after the execution of the proposed scheme. At high water stages, however, part of the right bank downstream of the sheet-piling is inundated. This may be prevented by a further heightening of the right bank. The improvement of the drainage of the air-base area, which is already being carried out by the construction of drainage canals, will not be hampered by a further heightening of the right bank, if the canals can be temporarily closed during high water stages in the Río Magdalena by means of, e.g., removable coffer dams or inflatable weirs.

#### Phasing of the proposed scheme

It has already been mentioned that the scheme presented for the improvement of the La Dorada - Pto. Salgar area can be considered as a first design only. Model tests have to answer the extent to which river-works have to be carried out and what type of river-works best serve the required purposes. In the first instance, the flow-pattern must, therefore, be studied. This can be done in a model with a "fixed" bed (adaptation of the bed-level to

### III, 3.2

the flow-velocities must, however, be possible to study its influence on the flow-pattern). The scales of such a model will mainly depend on the water depth and the bed roughness. The minimum water-depth in the model should still be large enough (3-4 cm) for velocities to be measured, while the bed roughness should also be reproduced correctly. The following scales therefore seem suitable:

$n_h = 40$ ,  $n_l \approx 60$ ,  $n_v = \sqrt{40}$ ,  $n_c \approx 1.2$  and  $n_Q \approx 15,000$ . These geometrical scales also allow for additional tests in the same model with movable bed.

Apart from this general model, it is also advised to study the erosion of the river-bed in front of a groyne head. Earlier in this paragraph, an estimate has been given of the water depth which can be expected in front of a bank protection. In Para. 2.5.2 it was mentioned that the depth in front of a groyne may be 20% to 30% greater than in front of a bank protection. In view of the high cost involved in adequately protecting a groyne head, it is worthwhile studying this problem in a separate model.

Summarizing the foregoing considerations, the priority of execution of the various components of the river improvement can be listed (in a downstream direction) thus:

#### The construction of the groynes at B and the bank protection at C

If the model tests show that the bank protection needs to be continued at D, the construction of a number of groynes or a spur-dike could be considered. It is advised to start the execution of the project with the construction of the bank protection, working in an upstream direction. The greater part of this protection can be executed "in the dry". The length of the bank-protection is given in Figure 3.2.14 schematically. At both ends measures should be taken to prevent leakage. If the model tests do not answer absolutely what length of the right bank should be protected, it is also possible to construct this protection in two phases. The upstream end of the bank protection can then be continued at a later date when the requirements for its extension have been determined from measurements in the prototype.

Thereafter the groynes at B can be constructed. The crossing of the main current to the right bank, which will be the result of the construction of the groynes at B, will lead to scour of the right bank in front of the bank protection and sedimentation along the left bank. However, part of the right bank will have to be removed by means of dredging. The change in the course of the Río Magdalena which will then result will already lead to scour along the inner bank of the Vuelta del Conejo and reduce the flow-velocities and scour at D. If a further guidance of the flow at D will still be required, this should only be undertaken after the complete removal of the former inner bank in the Vuelta del Conejo (indicated in Figure 3.2.14 by III), to obtain the greatest reduction of the flow-velocities. In view of the great changes which will be achieved upstream, it is also possible that the inner bend of the Vuelta del Conejo will have to be dredged a number of times.

#### The construction of the groynes at A and H

The construction of the groynes at A are required to create a smoother river-bend for the further protection of the railway connection between La Dorada and Honda. For navi-

gation purposes the groynes at A will not be necessary. However, if the groynes at A are not constructed, a total change in the river course is not unimaginable and their construction is therefore required.

The construction of the groynes at H will be required to guide the main channel along the quay wall at Pto. Salgar. If the model tests show that the crossing of the current will occur upstream of the quay-wall, dredging of the shallow shoal in front of the quay wall will be required to safeguard the approach to and the manoeuvrability in front of the quay wall. If, however, the location of the crossing will not be stable, or the crossing will only occur downstream, the construction of the groynes at H must be undertaken.

#### The construction of the spur-dike and/or groynes at E and F

It is not very likely that after the bank protection at C and, possibly, groynes at D, a further guidance of the flow at E will be required. If the model tests, however, show that this cannot be omitted, the construction of a spur-dike along the shoal (indicated with IV in Figure 3.2.14) may be the most economical solution.

It is also possible that the construction of the groynes at F will not be required, but that the smoother curve of the Vuelta del Conejo will already provide for a more gradual crossing of the main channel to the left bank and, consequently, a more uniform distribution of the current attack. If this situation, however, can only be achieved by the construction of the groynes at F, the following consequences must be considered: The accessibility to the ADENAVI stores, situated at the south-eastern end of La Dorada, will be blocked by the construction of the groynes and the sedimentation between the groynes. If an extension of the existing port facilities is required in future, a more upstream access to the basin in front of these stores can still be kept open by means of dredging. The accessibility for river transport to the stores situated upstream of the railway-yard will also be hampered by the construction of the groynes at F. However, relatively simple wooden jetties constructed over the foreshore and under the lee of the groynes will provide sufficient landing facilities.

#### Recommended types of construction and estimate of cost

##### Bank protection (C)

Upstream of the Vuelta del Conejo a length of about 3 km needs to be protected along the right bank. This work can almost completely be constructed "in the dry". As can be seen in Figure 3.2.15, the present depth along the outer bend in the Vuelta del Conejo is about 8 m below L.R.L. As the radius of the projected curve of the future course of the Río Magdalena ( $R \approx 700$  m, see Figure 3.2.14), is greater than that used for the computation of the radial bed-level slope ( $R = 400$  m), the depth along the bank protection will probably be smaller. Locally, however, a greater depth may still occur. For this reason the depth along the bank protection has been estimated at 8 m below L.R.L., with an over-depth of 2 m for local scour (see Figure 3.2.16). The bank must be protected up to about 4 m above L.R.L. which level will on the average be exceeded once every five years.

III, 3.2

The recommended type of construction consists of a nylon filter on a side slope 1 : 3 (a steeper side slope can endanger the stability of the protection), covered with a gravel layer of about 10 cm (diameter stones 2-10 cm) and 3 layers of hexapods (or the like) on top. According to Figure 2.5.7 the diameter of the components of the cover should be about 40 cm to resist a flow-velocity of 3 m/s. A correction-factor ( $f_1 \approx 1.5$ , see Figure 2.5.9) is required because of the side slope of the bank and the helicoidal flow in the bend. The components should, therefore, be larger than 60 cm and have a weight greater than 300 kg (see Figure 2.5.8). The hexapods which were used in the bank protection at Yarirf have an average weight of 320 kg and a diameter of about 0.90 m. In view of the fact that such hexapods can be interwoven (especially when constructed "in the dry") these specifications appear to be sufficient. If graded stones are used (which are available near La Dorada according to the survey of the Julius Berger Konsortium), the weight should be increased because the single stones may be lifted from the construction. It is then advised that stones with a weight of between 300 kg and 500 kg are used.

To start with, a trench should be dug to 8 m below L.R.L. The spoil can be used for heightening of the right bank and the prefabrication of the hexapods. The nylon mat needs to be continued horizontally at the toe to allow for local future scour. The gravel can be dumped on the mat and afterwards evenly spread to a thickness of the layer of about 10 cm. The hexapods are to be placed on top of the gravel (see Figure 3.2.16).

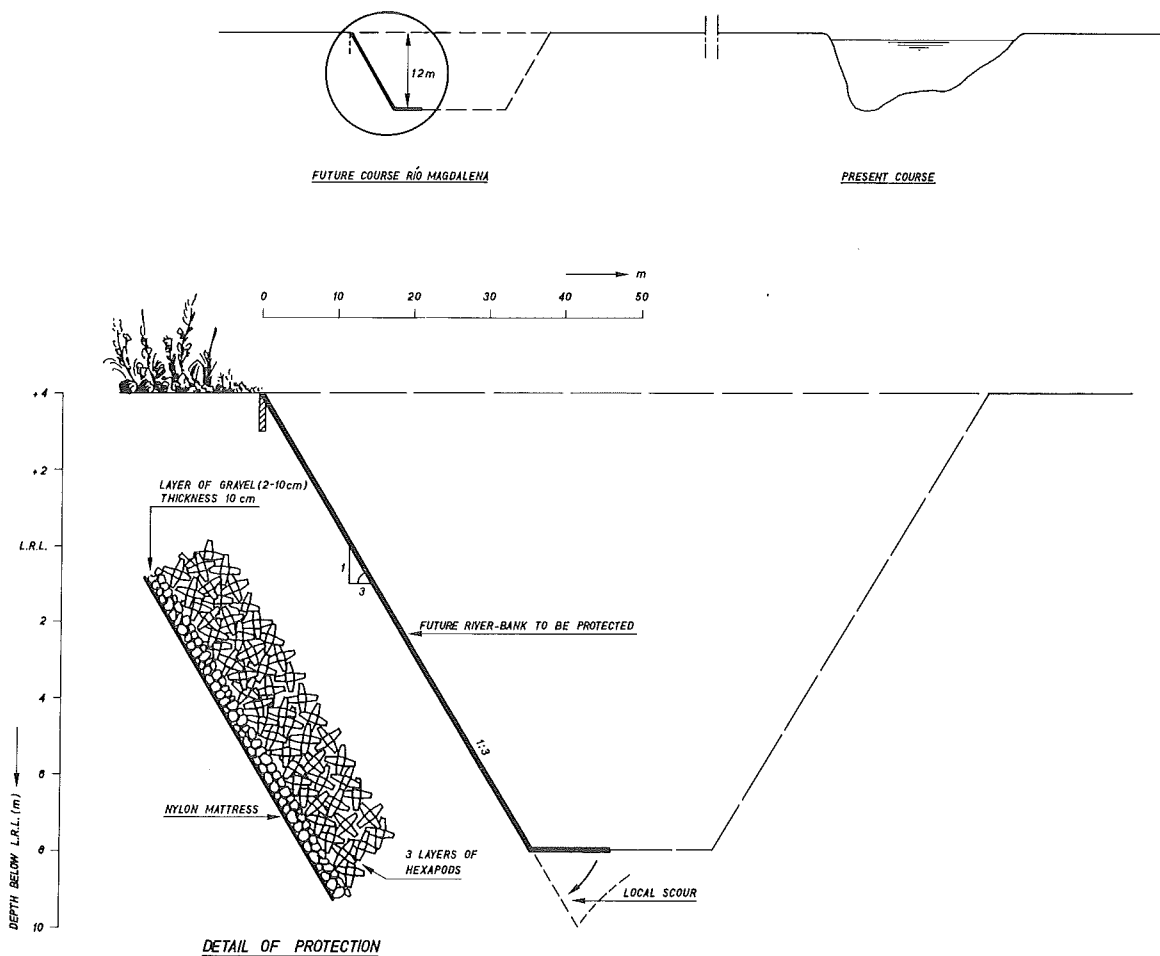


Figure 3.2.16 Construction of Bank Protection

An estimate of the cost of such a protection constructed over a length of about 3,000 m is given below. This estimate is based on the 1972 prices and gives only an impression of the total cost. After the model tests have been carried out, the type of construction can be designed in greater detail and the cost mentioned should be revised.

Trench	(2,000,000 m <sup>3</sup> )	Col \$	20,000,000
Nylon mattress	( 130,000 m <sup>2</sup> )		8,500,000
Gravel	( 15,000 m <sup>3</sup> )		1,500,000
Hexapods	( 400,000 pieces)		<u>40,000,000</u>
	Total cost:	Col \$	70,000,000

### Groynes (B)

On the southern side of La Dorada town five groynes are projected along the left bank of the Río Magdalena for defence of the town area and to bring about a shifting of the river course downstream in a more southerly direction. The existent groyne needs to be rebuilt.

According to the sounding of 1971 (see Figure 3.2.10) the greatest water depth along the outer bend near the projected groynes is about 8 m below L.R.L. If constructed on a proper filter, the groynes may well be built directly on the river-bed. However, at the head of the groynes allowance must be made for future scour, because such (local) fixed points are usually more heavily attacked by the current than a bank protection. The groynes should be constructed up to about 4 m above L.R.L. The required length of the groyne is estimated at 50 m and they will be about 200 m apart.

For the filter a nylon mattress is again advised. Such flexible slabs are, however, difficult to handle at high flow-velocities. It is therefore advised to use a mattress with stuck-on lateral cells which can be filled with sand stabilized with Standard Road Oil (S.R.O.-sand). (This type of construction was also used in the 1950's near Pto. Wilches to protect drilling sites). The further weighting down of these mattresses can be done with nylon bags of about 1 m<sup>3</sup> filled with S.R.O.-sand. On top of the bags hexapods can be dumped and the underlying sand-bags will prevent damage of the nylon mattress. The dimensions of the hexapods are the same as used for the bank protection at C.

For the construction of the groynes near Pto. Boyacá (see Figure 2.5.14) hollow steel tubes were applied in the centre line of the groyne. From a constructional point of view, these tubes will certainly have advantages (mooring of barge alongside during execution, and guidance for the dumping of the tetrapods), but after the execution these tubes no longer function. It may be more economical to study the possibility of anchoring a barge in the river in the centre-line of the groyne and to work from both sides (see Figure 3.2.17).

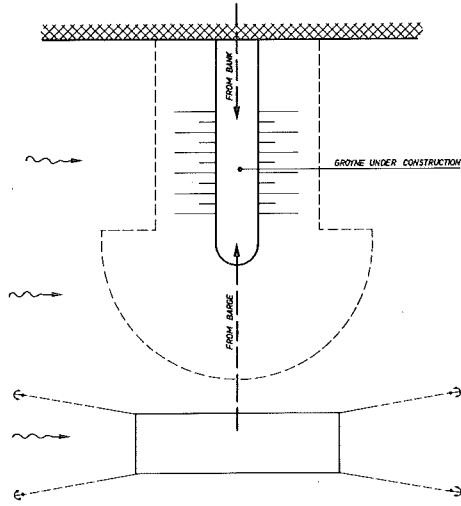


Figure 3.2.17 Construction of Groyne from Two Sides

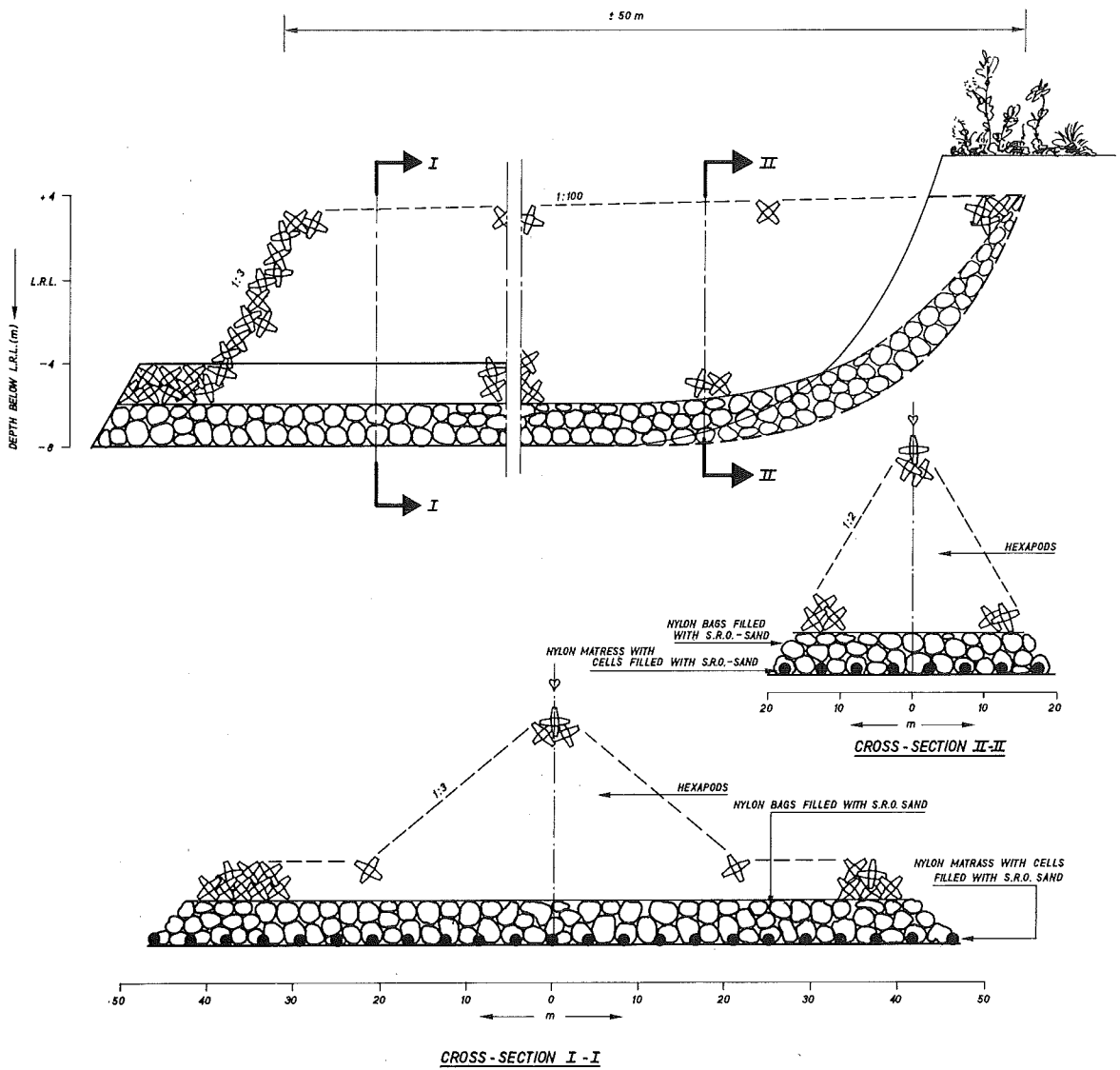


Figure 3.2.18 Construction of Groyne

The construction is schematically indicated in Figure 3.2,18. It can be seen that near the groyne-head an allowance is made for future scour. In view of possible settlements the side slope is rather gentle (1:3) while closer to the bank a steeper side slope (1:2) can be permitted.

An impression of the cost of each groyne can be obtained from the following estimate:

Nylon mattress (4,000 m <sup>2</sup> )	Col. \$ 400,000
Nylon bags (8,000 pieces)	1,500,000
S.R.O.-sand (8,000 m <sup>3</sup> )	100,000
Hexapods (40,000 pieces)	<u>4,000,000</u>
Total cost per groyne:	Col. \$ 6,000,000

For the projected five groynes a total investment of Col. \$ 30,000,000 will be required.

#### Protection at D

It has already been mentioned that the type of construction required for the protection of the right bank at D will depend on the results of the model tests. Either groynes (or a spur-dike) are required for a further guidance of the flow, or a relatively light bank protection will already be sufficient if the main current crosses directly from the bank protection at C to the rock downstream of D generating an eddy in front of the present right bank at D. In the first case, the same type of groynes should be used as indicated above, although they are considerably longer (about 200 m). For the four groynes schematically indicated in Figure 3.2.14 a total investment of roughly Col. \$ 100,000,000 will be required.

When a bank protection will suffice, a similar type of construction as indicated for the bank protection at C is advised. However, this protection needs then to be executed "in the wet" and therefore some modifications are necessary. A nylon mattress partly filled with S.R.O.-sand (the same type as used for groynes) must then be chosen as filter. On top of this filter one layer of nylon bags filled with S.R.O.-sand can be dumped, covered by two layers of hexapods. Because of the smaller flow-velocities, the cover may also consist of lighter components (e.g., stones with an approximate gradation of 35-60 kg). For the total length of about 800 m which needs to be protected, this type of bank protection amounts to Col. \$ 12,500,000.

A comparison of the cost of these two types of construction (groynes or a bank protection) clearly shows that carefully conducted experiments in a model can lead to enormous financial saving. If the length and the direction of the bank protection at C can be designed in such a way that only a light attack of the current on the bank at D is to be expected, a more economical type of construction can be chosen.

#### Groynes (A and H)

The construction of these groynes is again identical to that designed at B and the cost will amount roughly to Col. \$ 36,000,000.

III, 3.2

Dredging (II and III)

The present low-lying inner bend in the Vuelta del Conejo needs to be dredged completely. Roughly 500,000 m<sup>3</sup> must be removed here, requiring an investment of Col. \$ 5,000,000.

After the construction of the bank protection at C and the groynes at B, erosion will occur along the right bank of the Río Magdalena (in front of the bank protection, indicated by II in Figure 3.2.14), but dredging will also be required. The spoil can be used to heighten part of the spit of land in the inner bend of the Vuelta del Conejo, for the pre-fabrication of hexapods, or be dumped alongside the left bank. It is estimated that in total about 1,000,000 m<sup>3</sup> needs to be dredged, at a total cost of Col. \$ 10,000,000.

Remark: The spit of land in the inner bend of the Vuelta del Conejo is presently reserved for the future town extension of La Dorada. However, at high water stages the low-lying inner bend is flooded, and it should be studied whether reclamation of this bank will not result in an unacceptable setting-up of the water-level upstream in periods of high discharges. It might be better to use this area for general and less vulnerable purposes, and to project the future extension of La Dorada town in a more northerly and westerly (up-hill) direction.

Estimate of total cost for the proposed scheme

An estimate of the total cost for the proposed improvement of the La Dorada - Pto. Salgar region is given in Table 3.2.3.

Priority	River-works	Cost (1972 Prices, in million Col. pesos)	Total
First phase	Bank protection (C)	70	
	Groynes (B)	30	
	Dredging (II and III)	15	
	Bank protection (D)	<u>12.5</u>	127.5
Second phase	Groynes (H)	18	
	Groynes (A)	18	
	Dredging (I)	<u>p.m.</u>	36
Third phase	Groynes (F)	p.m.	
	Spur-dike (E)	p.m.	

Table 3.2.3 Estimate of Total Cost of Proposed Improvement

### 3.2.4. Confluences of the Río Magdalena and the Ríos Negro and La Miel

#### Introduction

At km 840 the Río Negro unites with the Río Magdalena along the right bank, while a little way downstream (km 837) the Río La Miel debouches into the Río Magdalena along the left bank. The name of the Río Negro originates from its colour, as the river discharges the black particles of the cretaceous settlements in the upper region of the Río Negro. On the contrary, the Río La Miel discharges at low water stages almost clear water, although at higher stages the turbidity increases. The difference in concentration and origin of the wash-load carried by the Río Magdalena and the Ríos Negro and La Miel can be seen on the aerial photographs on which, downstream of the confluences, the respective flows are clearly marked. The run-off of the rainfall in the Río Negro and Río La Miel basins causes rapid fluctuations of the water-level in both these rivers, and it is likely that this will have a backwater effect on the Río Magdalena. This may explain the great changes which have occurred in the course of the Río Magdalena in this region in the past.

#### Case history

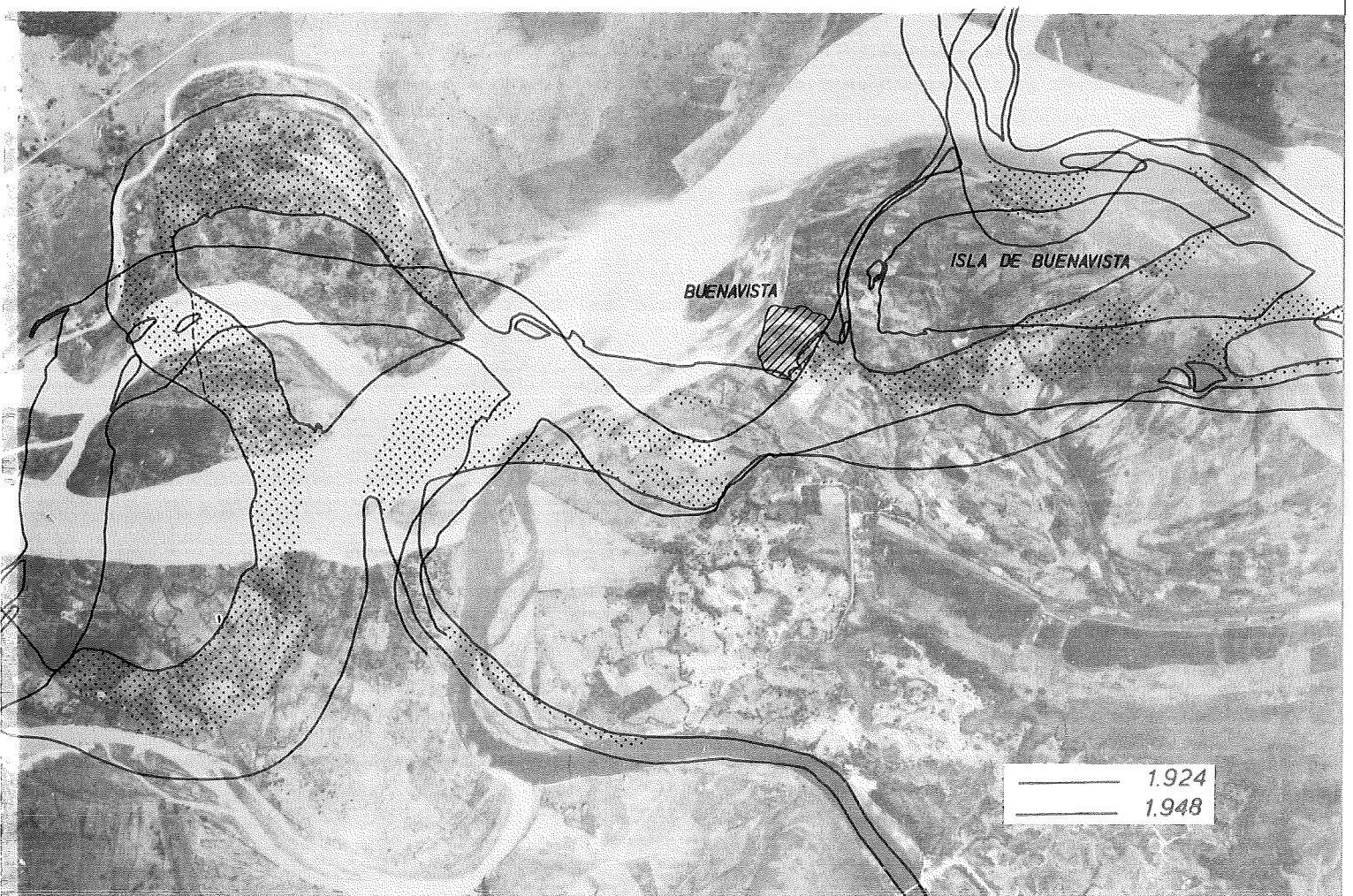


Figure 3.2.19 Case History of the Ríos Negro and La Miel Confluences

III, 3.2

The oldest available information dates back to the survey of the Julius Berger Konsortium. In Figure 3.2.19 a comparison has been made of the course of the Rfo Magdalena near the confluences with the Rios Negro and La Miel in the years 1924, 1948 and 1972. The data originate from aerial photographs. This comparison shows that the outlet of the Rfo Negro is still located at about the same place as during the survey of the Julius Berger Konsortium. More upstream, however, the Rfo Negro is very mobile, and in the photographs of 1972 dried-up meanders of former courses of the river are clearly marked. The confluence of the Rios Magdalena and La Miel shifted in a westerly direction since about 1960 (compared with the 1:25,000 topographical maps of IGAC) and the former village Buenavista as well as the island of the same name were wiped out. In the present topography, relatively young deposits can clearly be distinguished opposite the outlet of the Rfo La Miel and upstream of the Rfo Negro. The great changes in the course of this stretch of the Rfo Magdalena may partly be the result of these two affluents. The Rios Negro and La Miel carry together at high water stages probably half of the discharge of the Rfo Magdalena, which may cause a backwater effect on the latter river.

In the present situation (1973) a number of bad crossings have to be encountered by navigation upstream of the Rfo Negro Confluence (km 844 - km 840). As can be seen on the route map (Part II, Figure 3.3.19) the least available depth (LAD) in this area was about 1'6" below L.R.L. during the survey in February 1972. At higher water stages the talweg is often difficult to find, also as a result of the lack of proper beaconing (which may perhaps explain the higher bed-level recorded during the other surveys). The LAD recorded on the river-crossing between km 842 and 840 has been presented in Figure 3.2.20. In September 1972, also a number of velocity-verticals were measured near this crossing which showed that at high water stages (about 4 ft above L.R.L.) the highest velocities occur in the deeper channels and the lowest velocities on the crossing itself (greater width). This means that sedimentation will then probably occur on the crossing. It is likely that at lower water stages the velocity in the deeper channels decreases, while the velocity on the crossing itself increases, causing the so-called retarded scour. It is recommended that in future at low water stages these measurements are repeated to confirm this phenomenon. Also the velocity-verticals as measured in September 1972, are given in Figure 3.2.20.

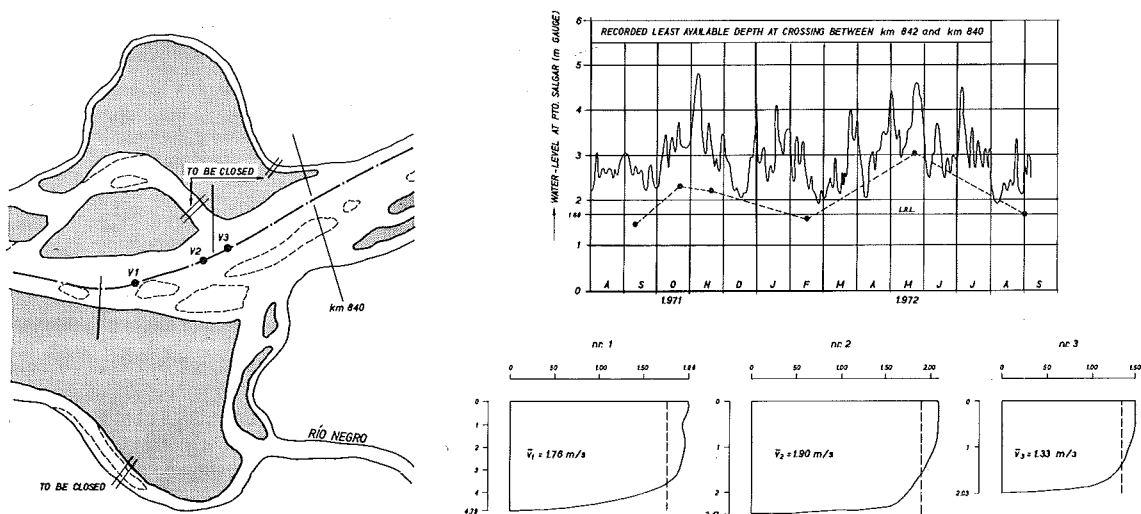
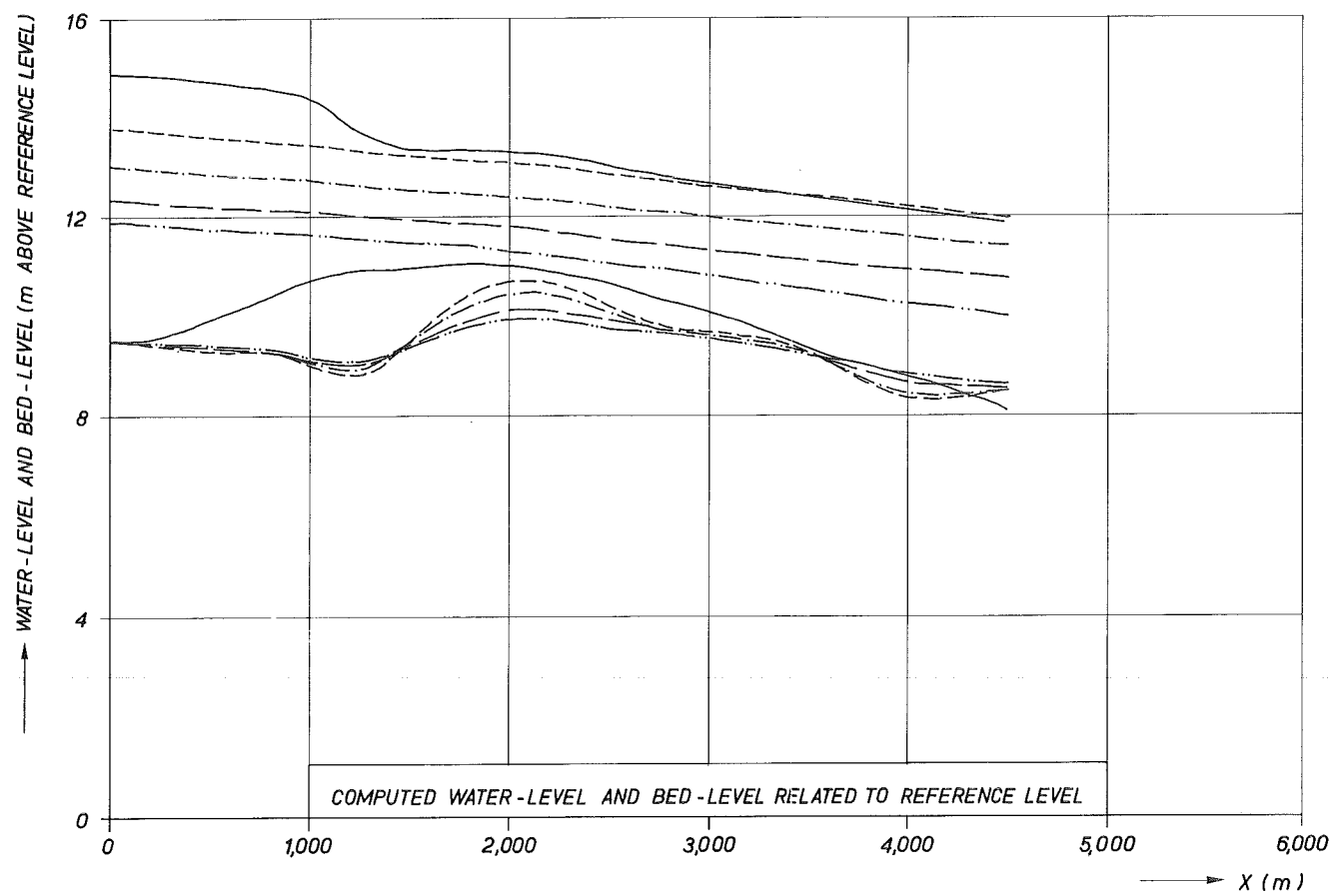
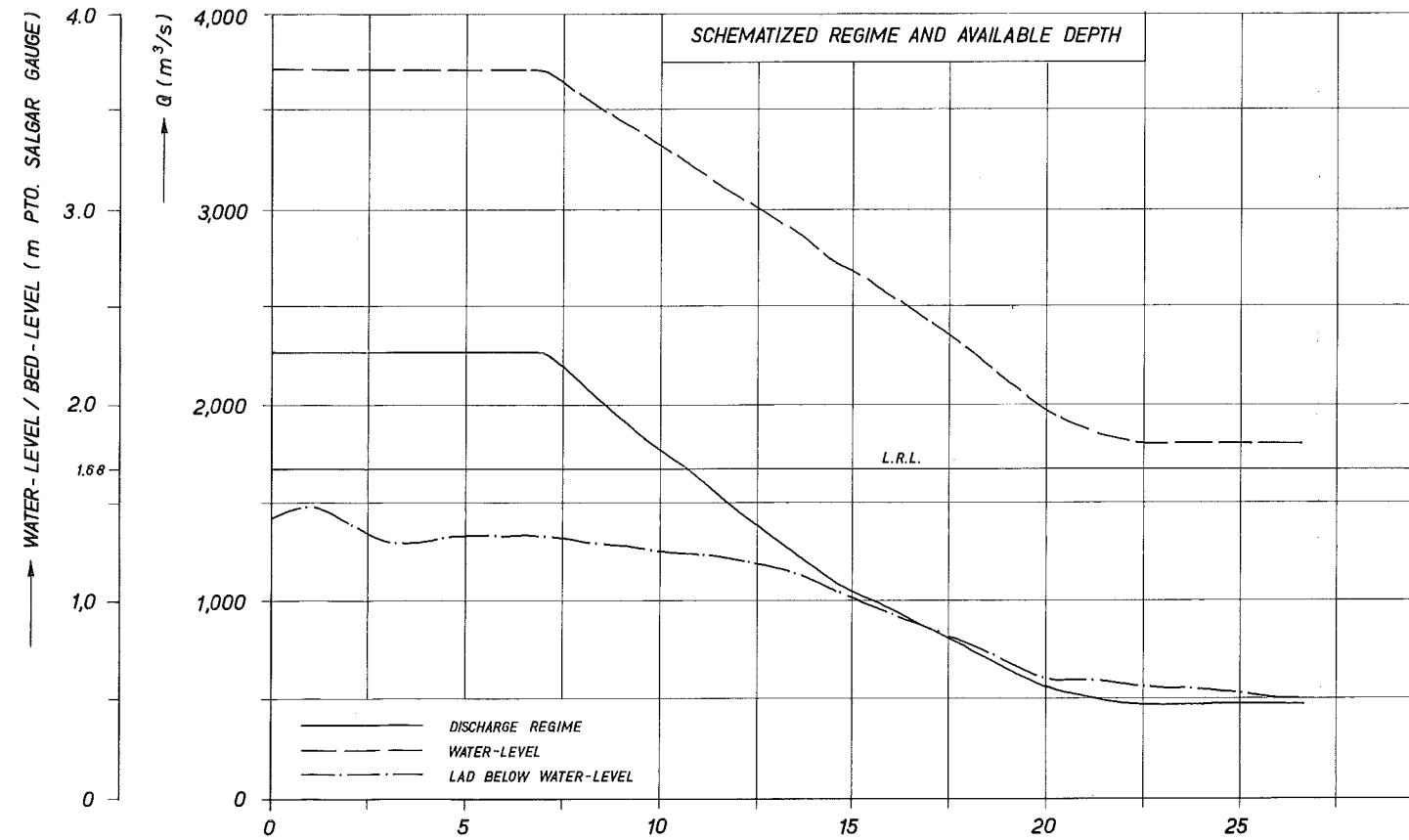
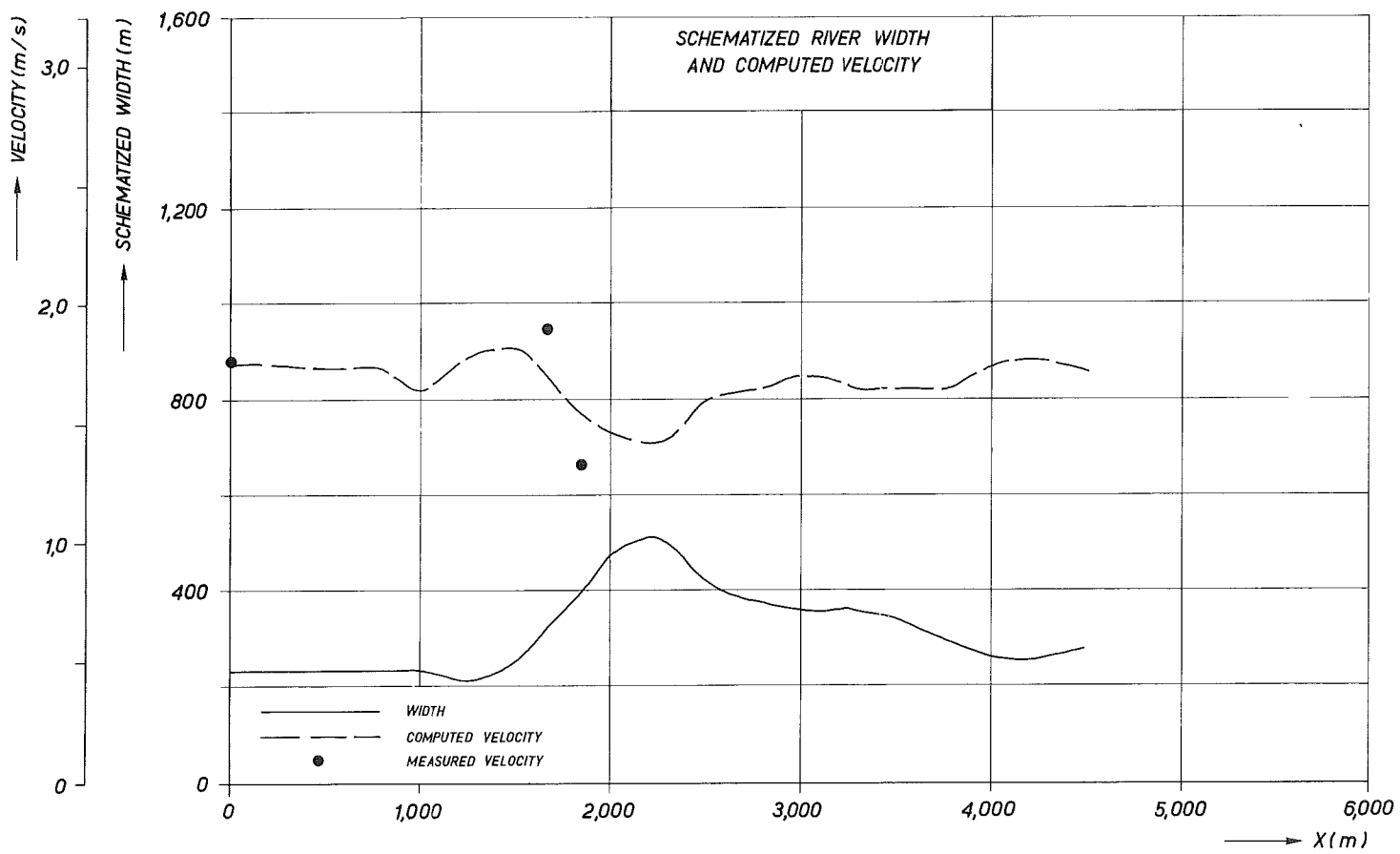
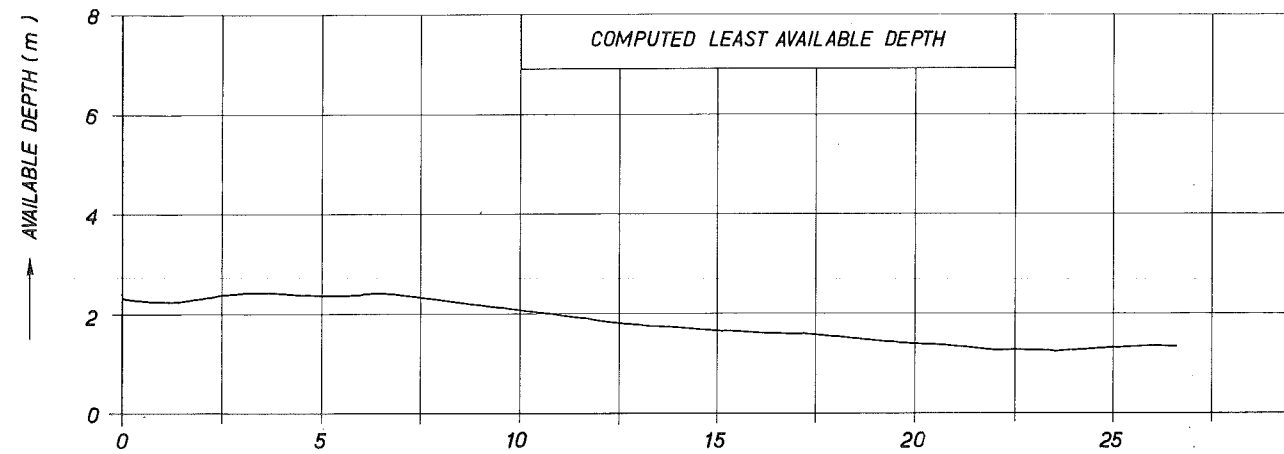


Figure 3.2.20 Recorded LAD on Crossing Between km 842 and km 840



WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—————	0	0	2,267
-----	6	1	2,267
- - - - -	11	6	1,579
— · — · —	16	6	928
· · · · ·	22	9.5	466



RÍO MAGDALENA CROSSING BETWEEN km 842 AND km 840

RESULTS OF MORPHOLOGICAL COMPUTATIONS

This crossing has also been used for morphological computations to predict the changes in bed-level as a result of the decrease of the regime from about the dominant stage ( $Q \approx 2,250 \text{ m}^3/\text{s}$ ) to L.R.L. ( $Q \approx 500 \text{ m}^3/\text{s}$ ). The principles for such computations as well as the chosen boundary conditions, have been outlined in Part II, Paras. 3.6 and 3.7. The results of the computations are laid down in Figure 3.2.21, where the schematization of the river width, the changes in bed-level and water-level in the time, and the LAD on the crossing have been given. It can be noted that according to these computations the LAD is never less than 4 ft, while the retarded scour causes an erosion of the highest part of the bed-level of roughly 3 ft (As indicated, this information has been used in Para. 3.2.2).

The average velocity in the measured verticals is compared with the computed velocity over the crossing. The computed velocities presented in Figure 3.2.21 were chosen at the time of occurrence of the same water-level as recorded at the time of the measurements, while the information of the measured velocity-verticals is plotted at those locations where the computed water depth corresponds to the measured one.

#### Improvement for navigation

In view of the great movability of the Rfo Magdalena (which would require the execution of river-works over a great length) and the considerable extent of retarded scour of the bed-level which can be expected at low water stages, it is recommended to improve the Rfo Magdalena section near the Rfo Negro and Rfo La Miel Confluences by means of recurrent dredging, accompanied by a proper beaconing of the talweg (probably in some of the wide branches the positioning of some buoys may be useful). The spoil of the dredging can, of course, be used to provide a guidance of the flow into the dredged channel at low water stages, but the river's topography is very suitable for the closure of secondary branches (schematically indicated in Figure 3.2.20).

For example, the former branch of the Rfo Magdalena, which connects the Rfo Negro about 2 km upstream of its confluence with the Rfo Magdalena, is already partly closed by sedimentation. A dam through this branch, covered with a light protection, will be sufficient to provide a closure for several years. The secondary branch along the left bank can also be closed to provide an even more concentrated flow in the remaining channel at low water stages. However, it is advised to close this branch in the first instance only temporarily, because at present no information is available of the discharge distribution over the two branches at high water stages. When more information has become available, a more permanent closure of one branch may then possibly be considered. According to the present topography the remaining channel will have a width of about 400 m, which is considered sufficient for this river section (see Para. 3.2.1).

3.2.5. River-crossing km 833

Also for the river-crossing near km 833 in the Rio Magdalena morphological computations were carried out. In addition to the longitudinal profile of the crossing given in Part II, Figure 3.3.19, the LAD on the crossing has been plotted in Figure 3.2.22, together with two velocity-verticals measured in September 1972. Again it appears that at high water stages the bed-level is heightened as a result of sedimentation (lower flow-velocities), while at the lower water stages the bed-level is lowered by retarded scour (higher flow-velocities).

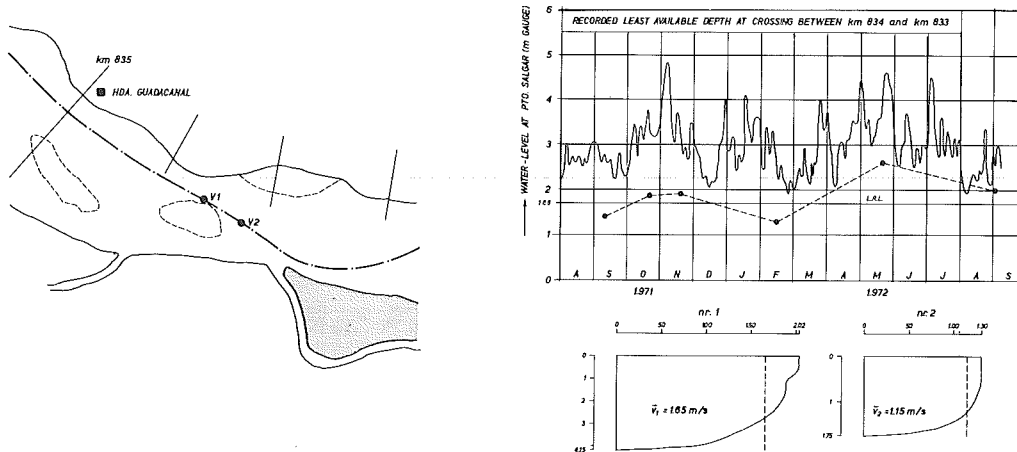


Figure 3.2.22 Recorded LAD on Crossing near km 833

The results of the computations are given in Figure 3.2.23, which shows a schematization of the river width, the changes in bed-level and water-level in the time, and the LAD on the crossing. It appears that with the given (schematized) regime of the river, the LAD on the crossing will be 3 ft or more, while the highest level of the bed will be lowered by at least 3 ft as a result of the retarded scour (This information was used in Para. 3.2.2). The average velocity in the measured verticals is compared with the computed velocity over the crossing. The computed velocities presented in Figure 3.2.23 were chosen at the time of occurrence of the same water-level as recorded at the time of the measurements, while the information of the measured velocity-verticals was plotted at those locations where the computed water depth corresponds to the measured one.

The river topography gives very few possibilities for improvement by means of the closure of secondary branches. The spoil of (recurrent) dredging works should be used to provide for the required guidance of the flow at low water stages.

### 3.2.6. Puerto Triunfo

#### Introduction

The village Pto. Triunfo is situated along the left bank of the Río Magdalena at km 825. The railway track "Atlantico" from the Caribbean Coast to Bogotá passes the village about 500 m inland. Good access by road will also be available in the future, when the project for the construction of a bridge near the Hacienda San Fernando (km 822.5) in the road connection Bogotá - Medellín will have been executed, and in connection with which high water-levels of the Río Magdalena were collected at the site by INTEGRAL (presented in Part II, Figure 2.3.10). If this project is actually carried out, Pto. Triunfo will also be a favourable site for the construction of port facilities. This scheme was provisionally proposed by ADENAVI in the course of the study and will have the following great advantages:

- For the improvement of the La Dorada - Pto. Salgar region not such expensive river-works will be required as outlined in Para. 3.2.3;
- the maintenance of the river-section La Dorada - Pto. Triunfo by means of re-current dredging in the dry season can more or less be omitted and this section will remain navigable during high water stages;
  - the possibilities for access to the new port area for the three modes of transport (river, rail and road) will be the same as for the present port area La Dorada - Pto. Salgar, but the accessibility for river transport will be cheaper and easier to maintain; and
  - the topography and conditions of the river near Pto. Triunfo probably permits the construction of a new port area.

Some of the advantages of a development of a port area near Pto. Triunfo will now be discussed.

#### Alternative solution for the improvement of La Dorada - Pto. Salgar

In Para. 3.2.3 a scheme was proposed for the improvement of the Río Magdalena in the La Dorada - Pto. Salgar region. This scheme was based on two requirements: an improvement of the accessibility for river transport to the existing port facilities and the provision of protection of the established investments on the river-banks. If, however, the requirement to improve the accessibility for river transport can be dropped by the development of new port areas near Pto. Triunfo, an alternative solution for the improvement of La Dorada - Pto. Salgar can be drawn up. It will then be sufficient to fix more or less the present course of the Río Magdalena; see Figure 3.2.24 (to enable comparison with Figure 3.2.14 the same code has been used). For the protection of La Dorada town the construction of groynes (or the like) at B, D and F will be sufficient. The groynes at B are required to protect La Dorada town to the south, but now shorter groynes will already suffice because no shifting of the course of the Río Magdalena in a somewhat southerly direction will be necessary. The groynes at F are required to protect La Dorada town to the east.

It is still recommended, however, to shift the inner bend at the Vuelta del Conejo in a somewhat westerly direction by dredging the present low-lying foreshore along the inner bend. This will result in a reduction of the flow-velocities at D and the construction of the bank protection can then be undertaken.

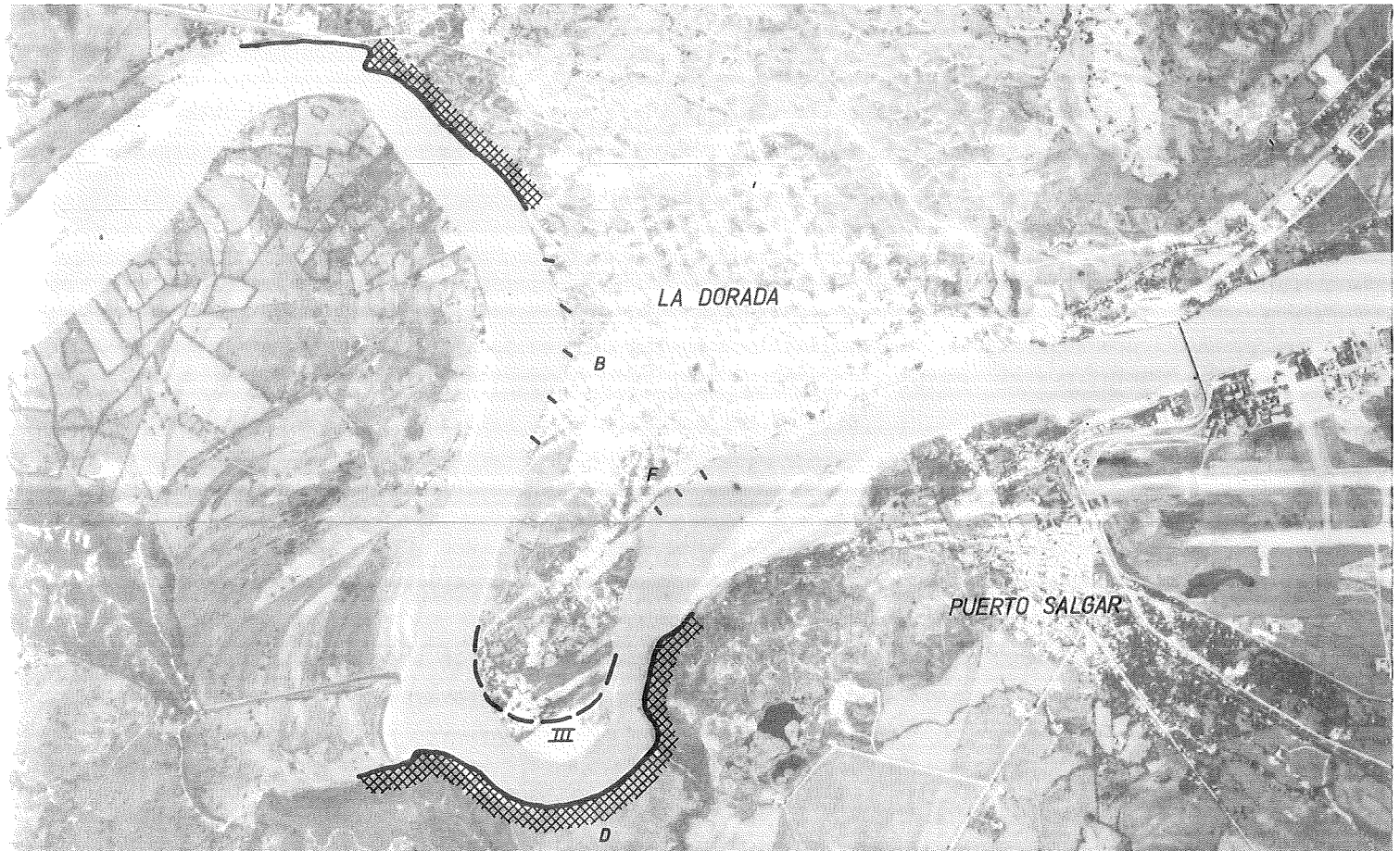


Figure 3.2.24 Alternative Solution for River Improvement near La Dorada - Pto. Salgar

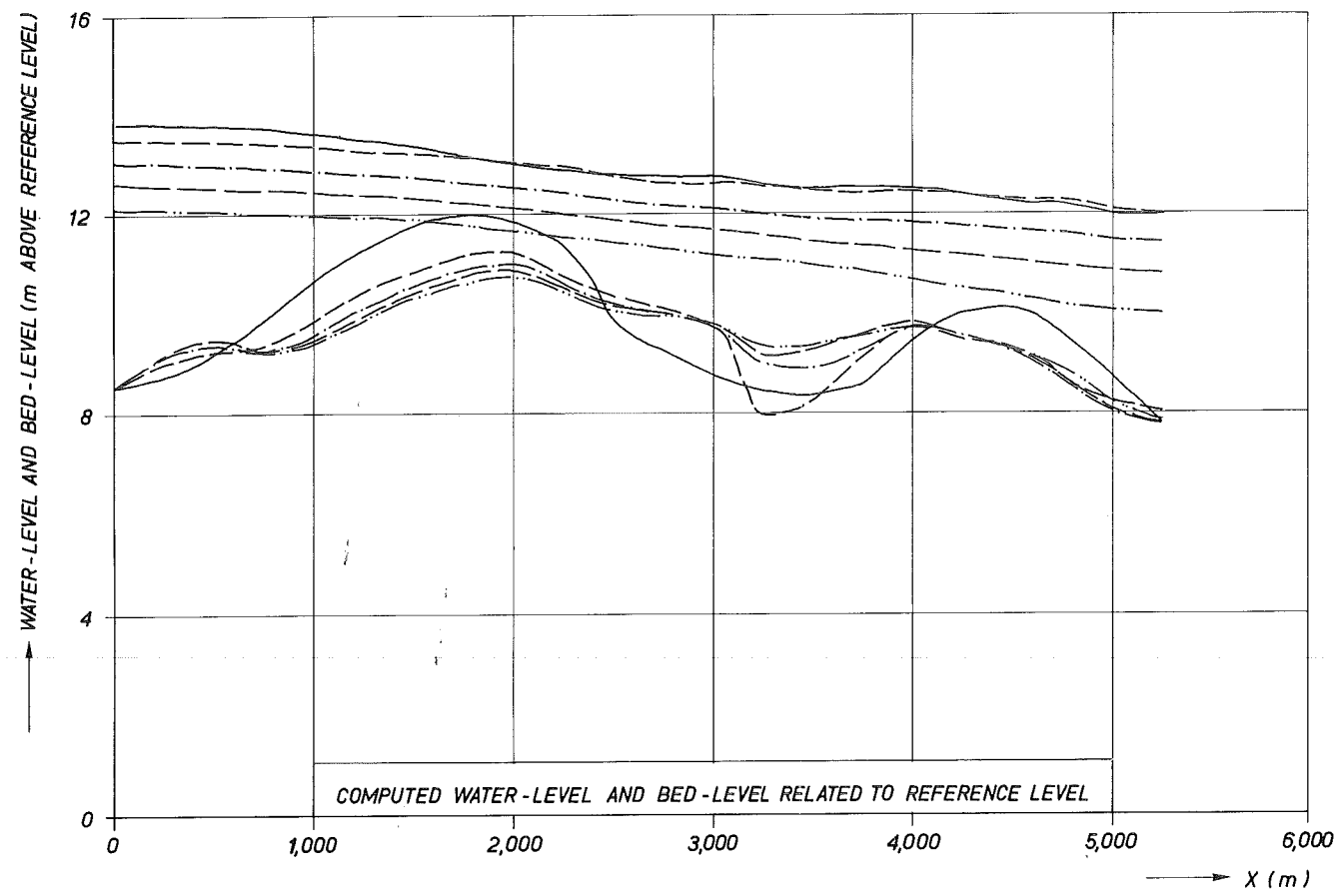
For the type of constructions to be applied reference is made to Para. 3.2.3. (Figures 3.2.16 and 3.2.18). An estimate of the cost of this alternative scheme is:

Groynes (B)	Col \$	20,000,000
Bank protection (D)		20,000,000
Dredging (III)		5,000,000
Groynes (F)		<u>10,000,000</u>
Total cost:	Col. \$	55,000,000

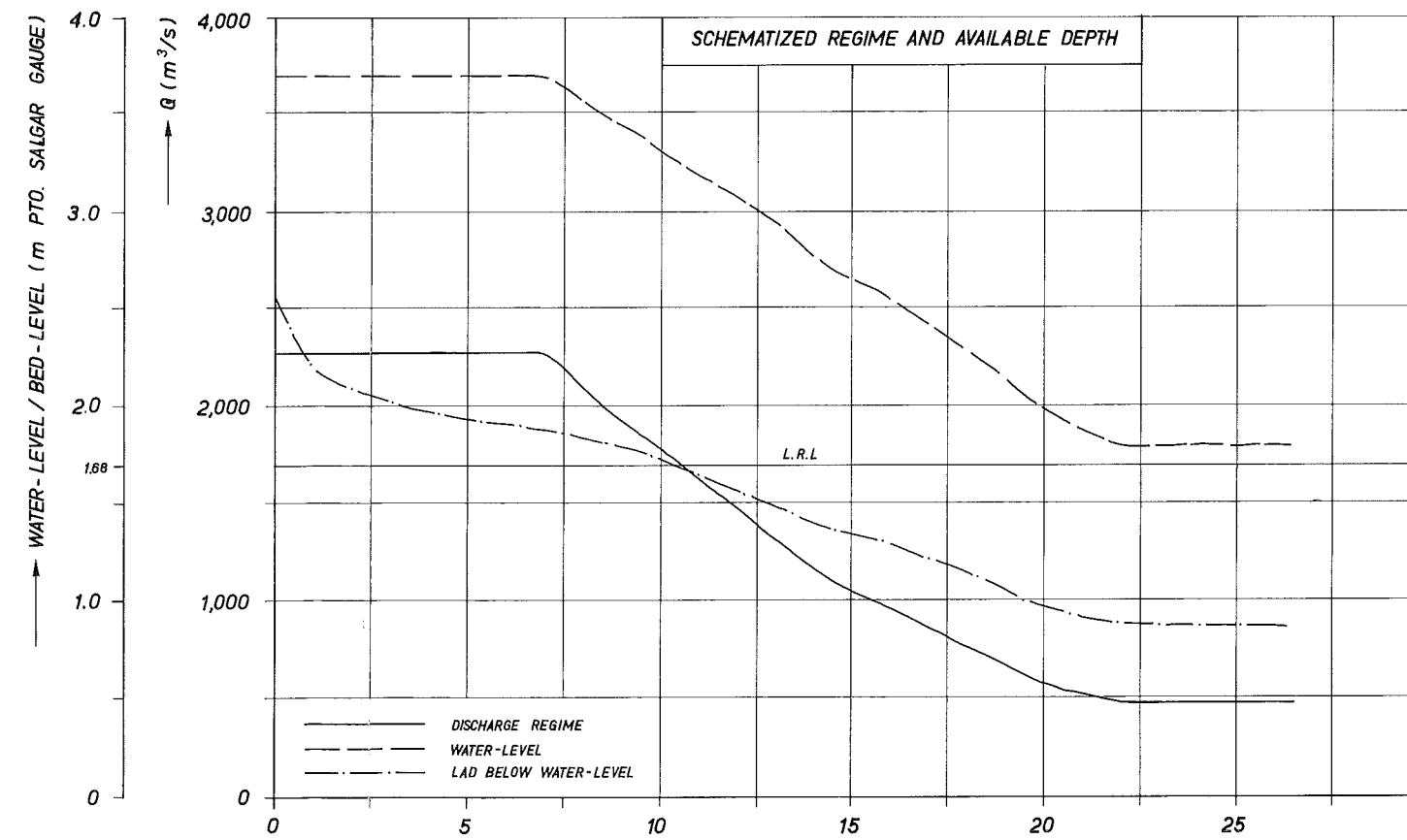
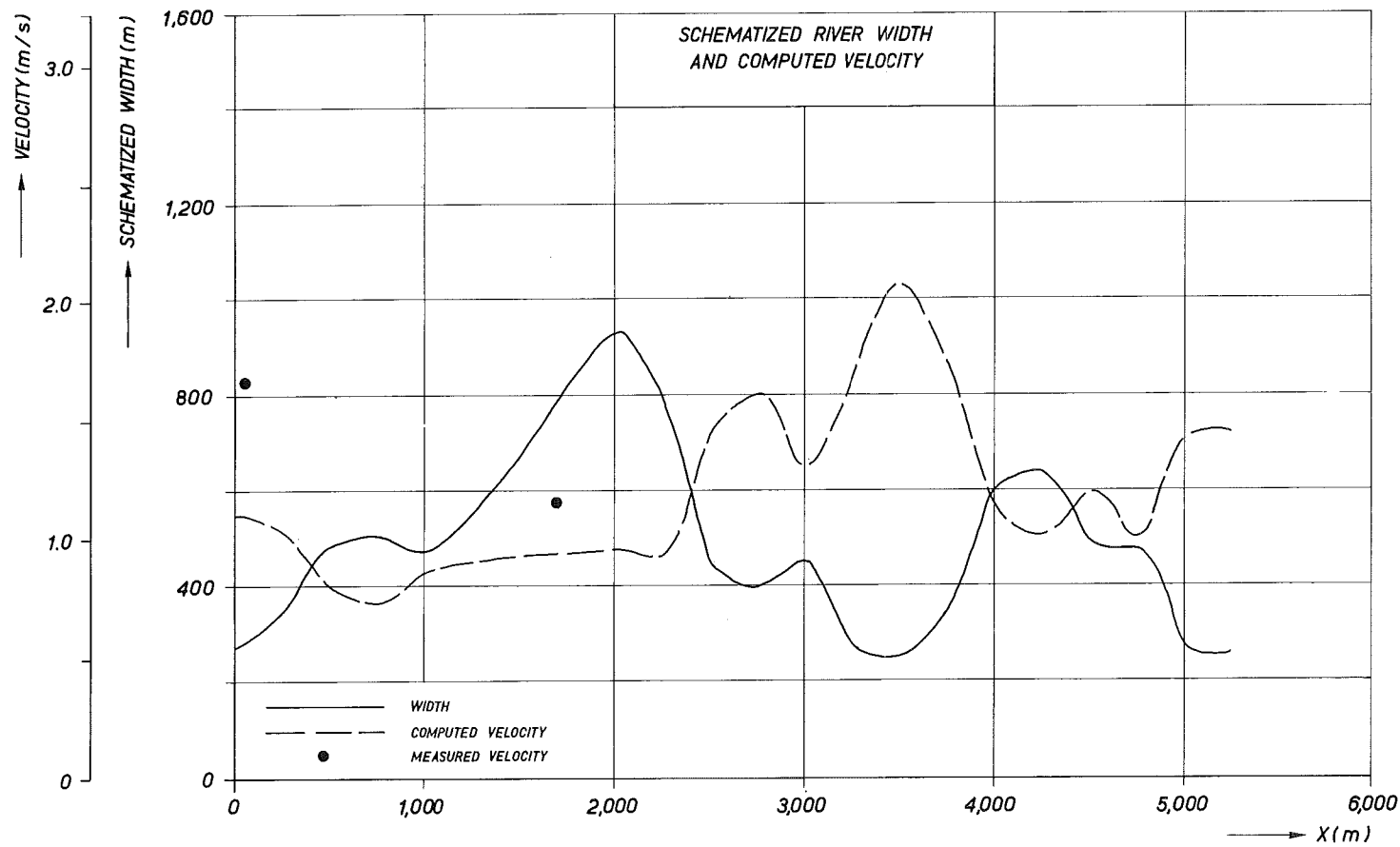
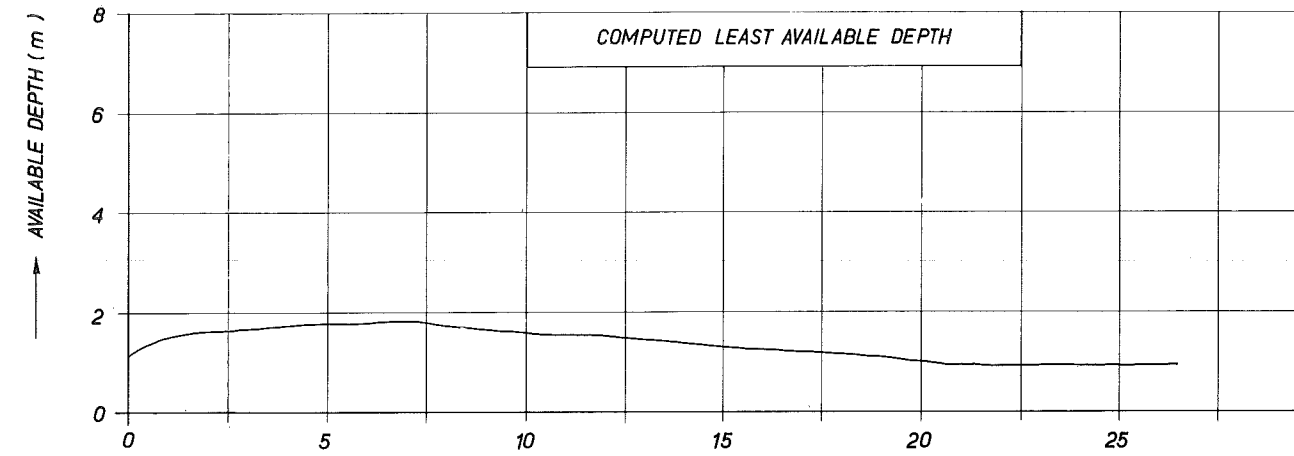
(Compare this estimate of cost with the one presented in Para. 3.2.3, for the first phase of improvement only: Col \$ 127,500,000).

Saving of cost for maintenance dredging

In Para. 3.2.2 the quantities which need to be dredged recurrently in the river section La Dorada - Pto. Inmarco have been determined. It appeared that in total 270,000 m<sup>3</sup> must be dredged, of which about 220,000 m<sup>3</sup> are in the section between La Dorada and Pto. Triunfo. This means that by the development of a new port area near Pto. Triunfo, about \$ 2,200,000 pesos can be saved annually (the La Dorada - Pto. Salgar port facilities remain accessible for river transport at high water stages as in the present situation). These savings must, of course, be balanced against the required investments for the new port facilities.



WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—————	0	0	2,267
-----	6	2.5	2,267
- - - - -	11	7	1,572
—————	16	0	952
-----	22	3	466



RÍO MAGDALENA CROSSING NEAR km 833

RESULTS OF MORPHOLOGICAL COMPUTATIONS

Site for future port development

A new port area near Pto. Triunfo can best be situated along the left bank of the Río Magdalena just downstream of the village. The site which appears suitable has been indicated in Figure 3.2.25. It must be mentioned, however, that this site has been chosen mainly from a river-morphological point of view and by studying the aerial photographs. A topographical survey will still have to prove whether this site is also suitable from the constructional and soil-mechanics point of view.

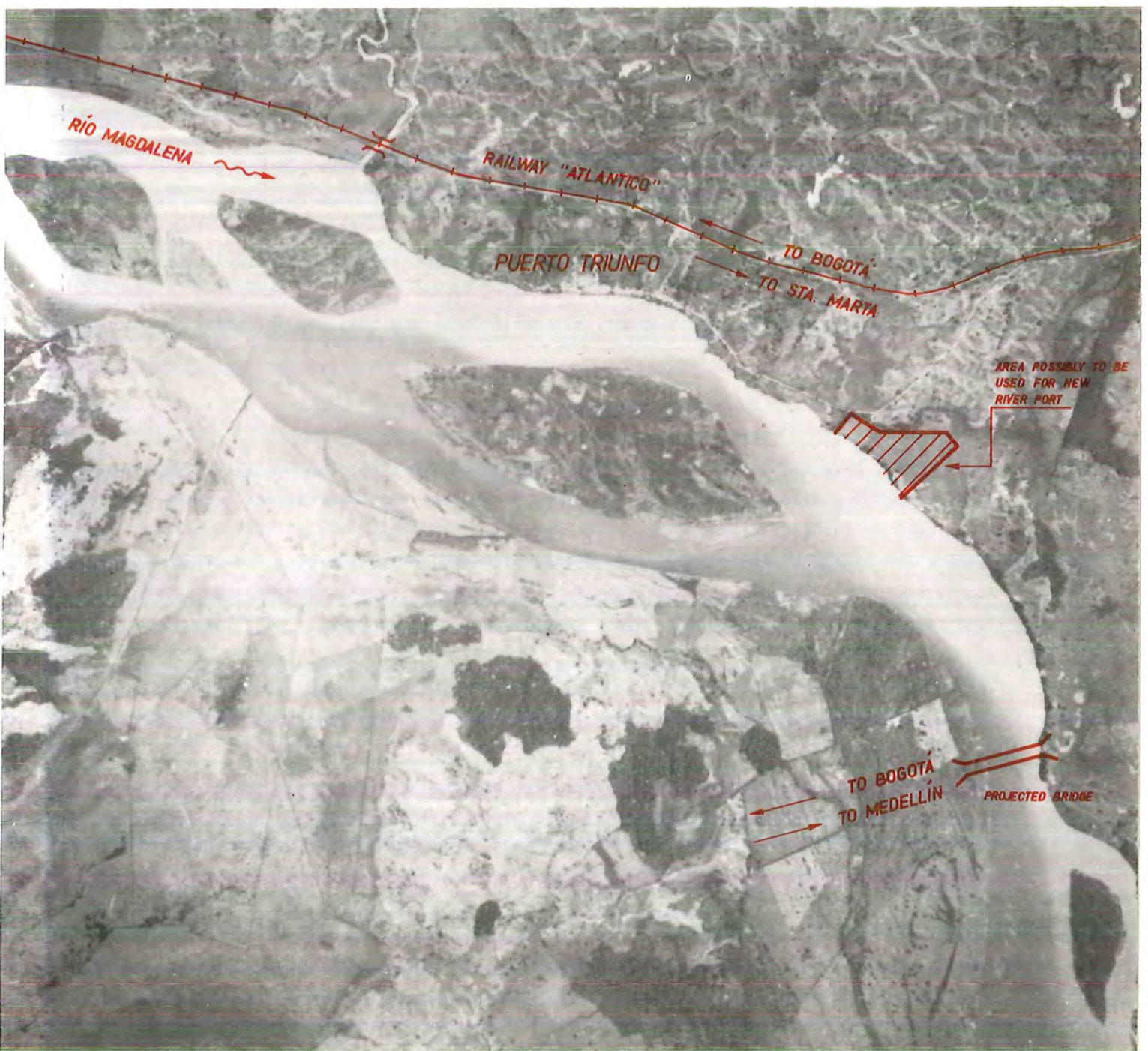


Figure 3.2.25 Site for Future Port Development near Pto. Triunfo

### III, 3.2

A study of the aerial photographs revealed that at present (1973) the course of the Río Magdalena lies more to the west than in the past. For example, if the present situation is compared with the situation in 1948, it appears that at that time the Hda. San Fernando was situated about 300 m inland, while more upstream there existed a wide and low-lying foreshore along the left bank. This means that if in future the chosen site is indeed developed for port facilities, the possibility exists that at a later date (after an alteration in the course of the Río Magdalena) river-works may be required for the protection of this area or to improve the accessibility to the new port. It must, however, be considered that such river-works will not be so expensive as the investments required for the present port facilities at La Dorada - Pto. Salgar (see Para. 3.2.3), because of the smoother course of the Río Magdalena and the lower flow-velocities. For an estimate of the future depth along a protected water-front, reference is made to the example given in Part II, Figure 3.7.10.

It is recommended that the development of a new port area near Pto. Triunfo be studied in greater detail in the near future. The existing port facilities in La Dorada - Pto. Salgar can still be used at high water stages, but the transshipment of cargo destined for the interior or the coastal regions (and abroad) should then be concentrated in the new port. It will be clear that the feasibility of the development of this port area will greatly depend on results of the "Magdalena River Area Transport Study" and on the execution of the projected bridge on the highway Bogotá - Medellín near the Hda. San Fernando.

#### 3.2.7. Puerto Boyacá

##### Introduction

Pto. Boyacá is situated at about km 805 along the right bank of the Río Magdalena, just downstream of a strong meander. In front of the village the main channel crosses from the left bank to the right bank. In the high water season (April-May) of 1972 erosion and inundation of the river-front of the village occurred. For the protection of Pto. Boyacá the construction of groynes was undertaken in a joint effort by the "Departamento de Boyacá" and the Texaco Oil Company (Pto. Niño, km 813), about which more is said later.

A general remark, however, must first be made regarding the comparison of the aerial photographs of the upper part of the middle course of the Río Magdalena in the past and present. From the case history of the plan-form of the Vuelta del Conejo near La Dorada (Figure 3.2.7) it can be seen that the greatest morphological changes in the course of the Río Magdalena occurred in the 1950's. This can also be noticed in the aerial photographs of Pto. Triunfo and the case history of the Pto. Boyacá area (see Figure 3.2.26 presented further on). To a lesser extent this is also confirmed by the case history of the plan-form of the Río Negro and Río La Miel Confluences (see Figure 3.2.19) and the still-to-be-presented case history of the Río Nare Confluence (see Figure 3.2.28). In these two latter examples this phenomenon may be less pronounced, because backwater effect of the affluents on the Río Magdalena must also be accounted for. An explanation of why the greatest morphological changes in the plan-form occurred in the 1950's can be seen by studying Figure 2.3.15 presented in Part II. In this graph the 50% (182 days) and 5% (18 days) exceeded water-levels of the yearly duration-curves of, amongst other places Pto. Salgar and Pto. Berrfo were given. It can be seen that in the 1950's consistently higher water-levels were

recorded at the Pto. Salgar gauge than in the foregoing and following periods, while the water-levels recorded at the Pto. Berrío gauge show a more gradual path. It seems logical to accept that these higher water stages also brought about great changes in the plan-form of the river.

#### Case history

A comparison of the plan-form of the Río Magdalena in the past years is presented in Figure 3.2.26. The available information dates back to the survey of the Julius Berger Konsortium (1924), aerial photographs taken in 1948, the topographical maps (scale 1:25,000) of IGAC and the photographs taken in 1972. From Figure 3.2.26 a prediction regarding the

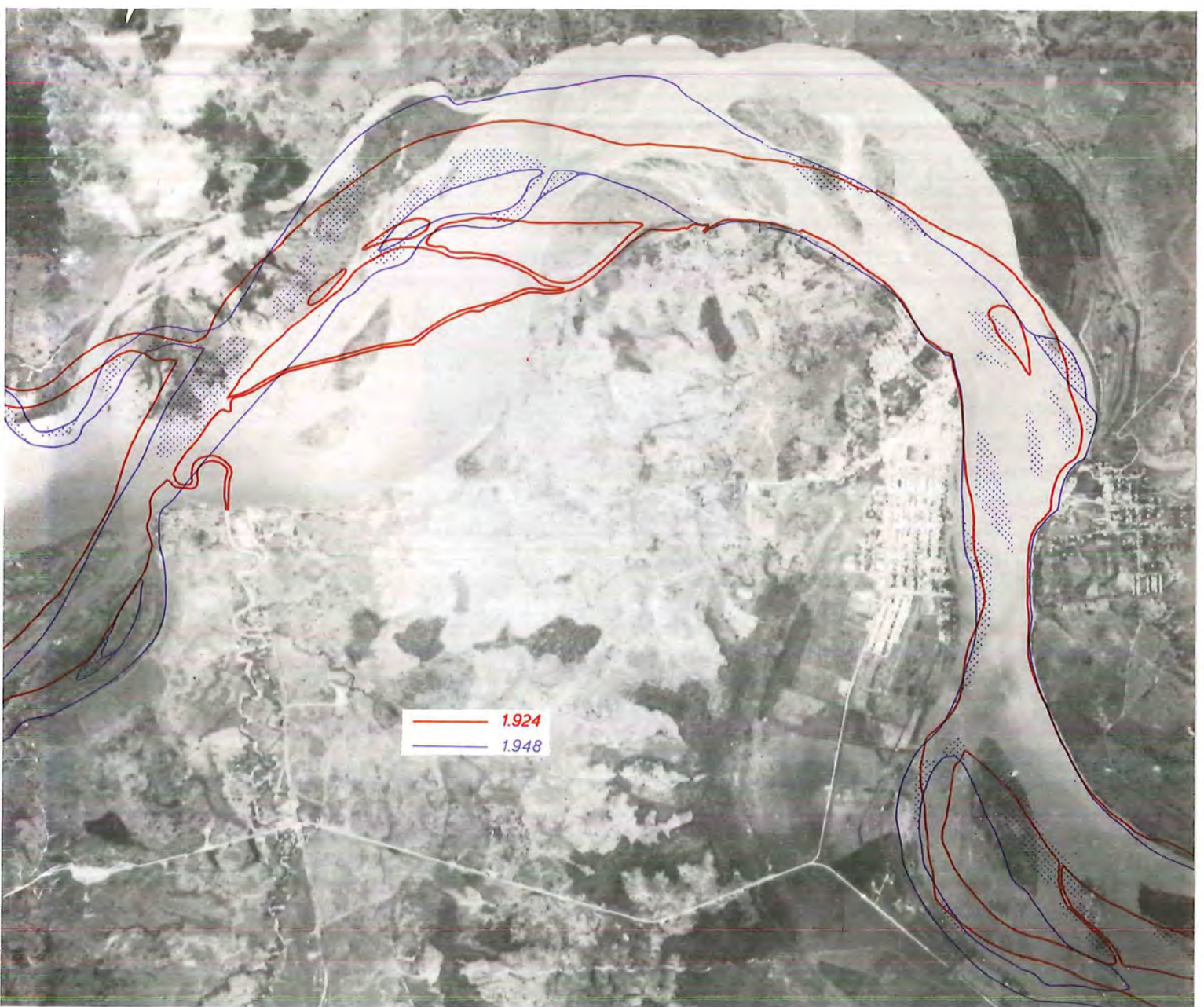


Figure 3.2.26 Case History of the Río Magdalena near Pto. Boyacá

### III, 3.2

future development of the plan-form of the Río Magdalena near Pto. Boyacá can be made. A further scour of the right bank downstream of Pto. Niño must be expected, as well as an advance of the erosion of the left bank in the meander loop and the right bank near the river-front of Pto. Boyacá. This development will depend, among other things, on the possible extent of the erosion of the left bank of the Río Magdalena in the meander loop. The aerial photographs reveal the presence of rocks which are probably erosion resistant. However, upstream of El Reboso (situated along the left bank, opposite to Pto. Boyacá) former plan-forms of the Río Magdalena can still be distinguished in the (low-lying) alluvial deposits. Consequently, unless counter-measures are taken, a further scour of the river-bank at Pto. Boyacá must be expected (compared with the survey of the Julius Berger Konsortium, the river bank at Pto. Boyacá has already eroded some 200 m).

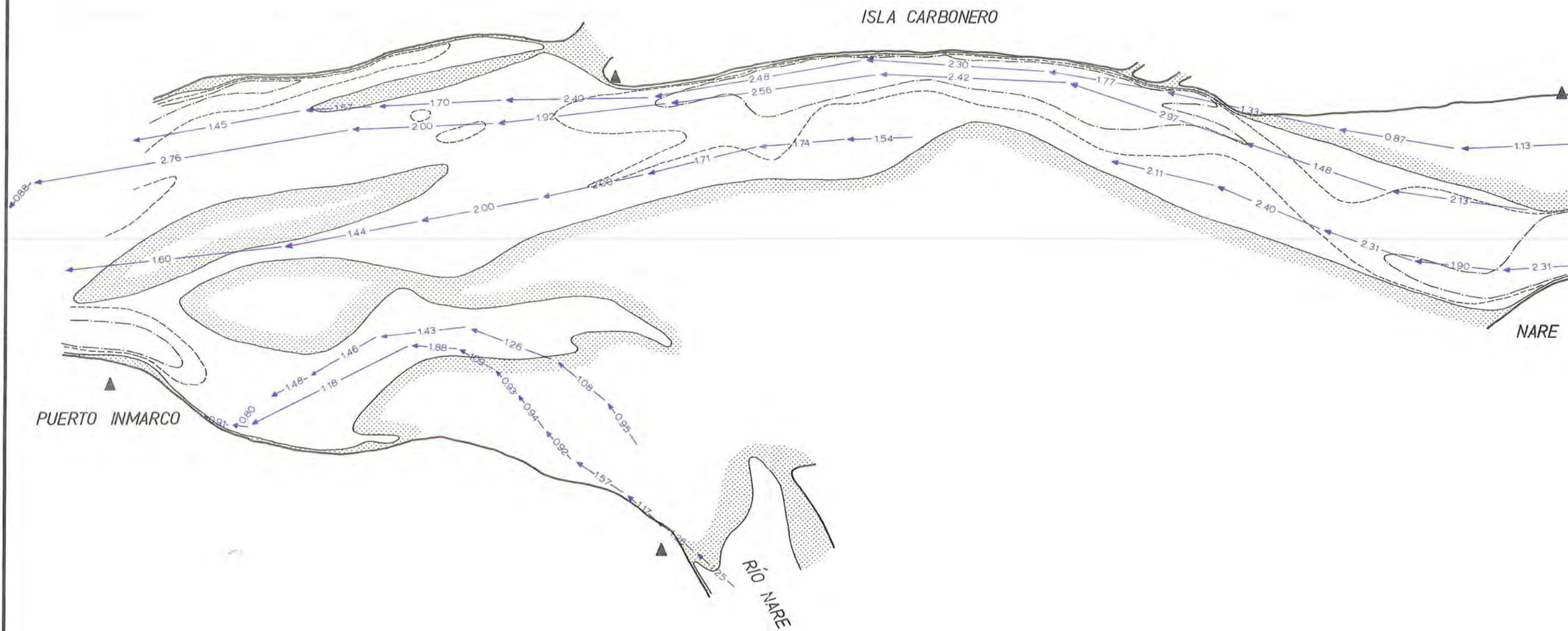
#### River improvement

The inundation and erosion of the river-front at Pto. Boyacá in the high water season in 1972 actuated the execution of a scheme for the protection of the village. This scheme was initiated by the Departamento de Boyacá, while for its execution the staff and equipment of the Texaco Oil Company were enlisted.



The project comprises the construction of a number of groynes along the right bank of the Río Magdalena in front of Pto. Boyacá. The groyne consist of hollow steel tubes, drilled about half a meter apart, and filled "in situ" with sand. Prefabricated tetrapods are dumped in between the tubes and alongside the groynes. The construction itself appears to be sound; however, as no filter is used on the bed to prevent the loss of bed material, the stability of the groynes can incur a risk in the course of time (this may lead to the necessity of an excessive dumping of stones in future). Moreover, a slight change in the upstream river-crossing may cause leakage, because the groynes are scarcely continued into the river-bank itself.





SONDEO SOUNDING **RÍO MAGDALENA** km 773 - km 776 CONFLUENCIA DEL CONFLUENCE OF **RÍO NARE**

ESCALA/SCALE 1:10.000

FECHA/DATE 19.20.21-VIII-1.971

NIVEL DE REDUCCIÓN: 0.20 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. INMARCO

CHART DATUM: 0.20 metres ABOVE ZERO OF THE PTO. INMARCO GAUGE

FECHA/DATE 23-VIII-1.971 NIVEL DE AGUA/WATERLEVEL: 2.00 m SOBRE EL DATUM/ABOVE DATUM

LÍNEAS DE CORRIENTE/FLOW LINES ←-1.18- VELOCIDAD EN m/seg / VELOCITY IN m/s

CURVAS ISOBATAS	— 0 m	PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
DEPTH CONTOURS	- - - 1.5 m	
	— 2.5 m	
	— 5 m	

NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.2.30

If the foregoing prediction regarding the future development of the plan-form of the Río Magdalena indeed comes true, it will be clear that the construction of the groynes in front of Pto. Boyacá only will not be sufficient for the defence of the village. A future extension of the river-works in an upstream direction will be required; namely, along the outer bank just downstream of Pto. Niño and along the opposite bank in the outer bend of the meander (upstream of El Reboso). By that time some experience will have been gathered regarding the stability of the groynes near Pto. Boyacá, so that this type of construction may again be considered. However, the Mission feels that for future river-works some kind of filter, to prevent the loss of bed material, should not be omitted.

3.2.8. River-crossings near km 780

Although the crossings near km 780 were not chosen for morphological computations to predict the bed-level at water stages corresponding to L.R.L., some information was gathered in the course of the study which is worth being included in this Report. The LAD of three crossings measured during the longitudinal soundings was plotted against the water-level as read on the gauges of Pto. Salgar and Pto. Inmarco (see Figure 3.2.27). It can be seen again that a change in the water-level results in a change of the bed-level too, and that the LAD at low water stages (February 1972) is at least not smaller than the LAD at high water stages.

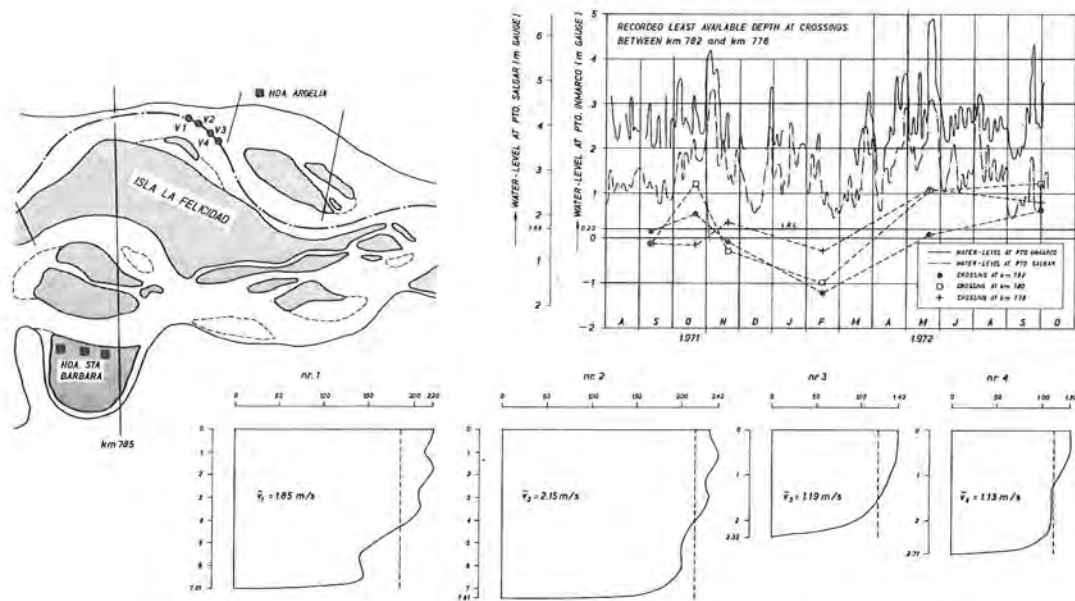


Figure 3.2.27 Recorded LAD on Crossings near km 780

On the crossing between km 785 and km 784 four velocity-verticals were measured in September 1972. Again it appears that the water-level was so high that the greatest velocities occurred in the deeper channels, while on the crossing (greater width) the velocity decreased, so that sedimentation is likely to occur. A repetition of such measurements at low water stages may prove that then the greatest velocities occur at the crossing itself, causing the retarded scour.

3.2.9. Confluence of the Río Magdalena and the Río NareIntroduction

At km 774 the Río Nare debouches into the Río Magdalena, just upstream of Puerto Inmarco (km 773) where an important cement factory is located. The raw material for this cement factory is found upstream in the Río Nare basin and transported to the factory by small craft. The Río Nare is sufficiently deep, but at high water stages overhead cables and the railway bridge (situated 3.5 km upstream of the confluence with the Río Magdalena) limit the clearance. The run-off of the rainfall in the Río Nare basin causes rapid fluctuations of the water-level in the Río Nare, an effect which is still partly recorded at the gauge in Pto. Inmarco. Regarding the local influence of the Río Nare on the water-level gradient in the Río Magdalena near Pto. Inmarco, reference is made to Part II, Para. 3.5.3, regarding the Pto. Inmarco Section.

Downstream of Pto. Inmarco the Río Magdalena is very narrow (some 200 m only) and deep, because of the high, diluvial and erosion resistant river-banks (the influence of this narrow river-stretch on the record of water-levels at the Pto. Inmarco gauge has already been discussed in Part II, Para. 3.5.4). Upstream of Pto. Inmarco the Río Magdalena is wide and shallow, consisting of a number of branches divided by islands and shoals.

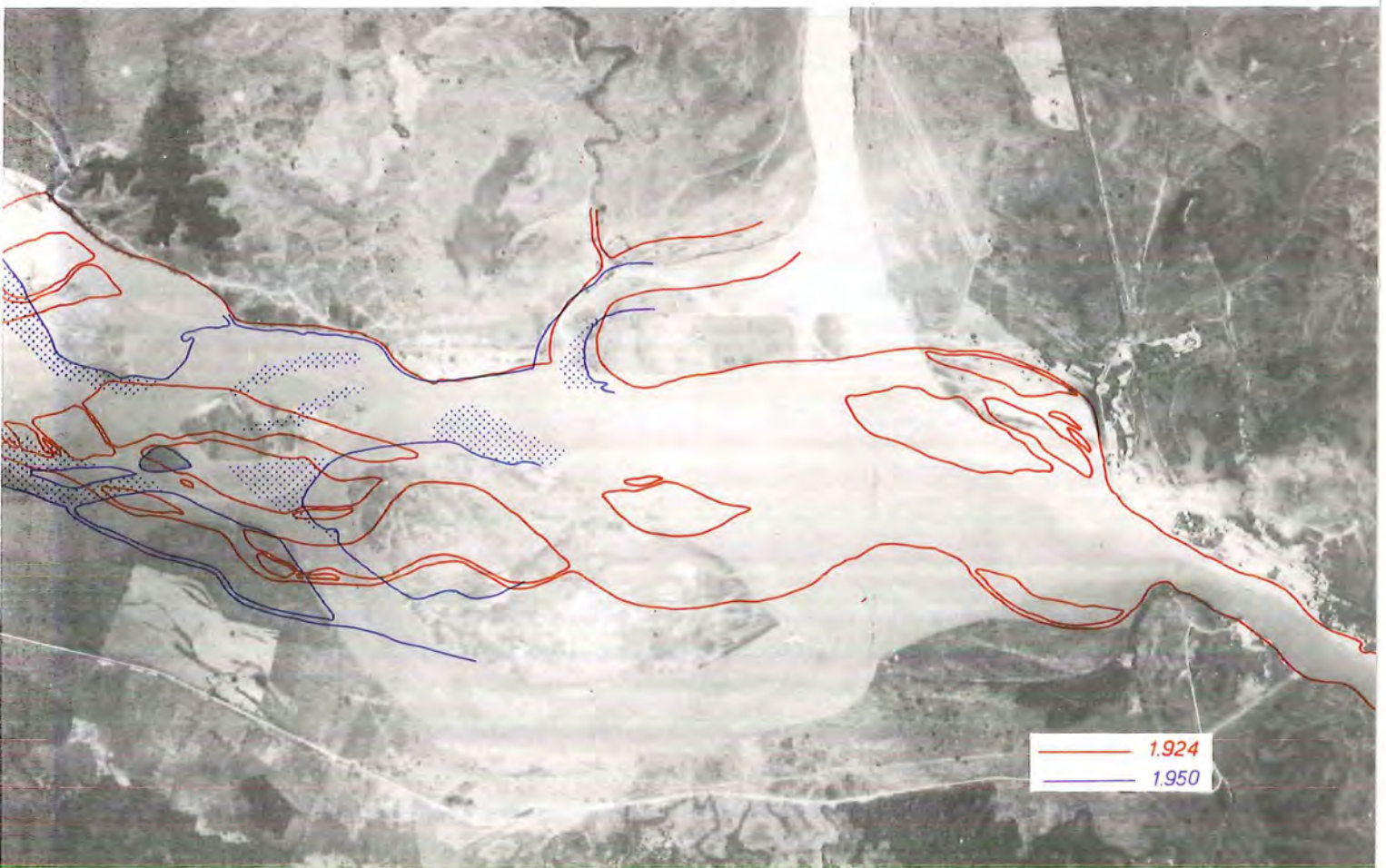


Figure 3.2.28 Case History of the Río Nare Confluence

### Case history

The available information regarding the plan-form of the Río Magdalena near the Río Nare Confluence has been compiled in Figure 3.2.28. The information dates back to the survey of the Julius Berger Konsortium (1924), aerial photographs of a short river stretch upstream of Nare village in 1950, the topographical maps (scale 1 : 25,000) prepared by IGAC, and the aerial photographs of 1972. During the survey of the Julius Berger Konsortium the Río Nare Confluence was situated just downstream of Nare village, about 1.5 km further upstream than its present location.

A detailed sounding of the Río Nare Confluence was made in August 1971, and the information obtained, together with the flow-lines, is presented in Figures 3.2.29 and 3.2.30. Attention has already been drawn to the LAD on the crossings just upstream of Nare village (see Para. 3.2.8).

### River improvement

In view of the similarity between the topographical features of the Río Nare Confluence and the Río Regla Confluence (near to the confluence the Río Magdalena is very wide and shallow, while a few kilometers downstream the width is strongly restricted), the future improvement of the Río Nare Confluence can be based on the same principles as the scheme presented for the improvement of the Río Regla Confluence (see Para. 3.4.4).



Figure 3.2.31 Possible Future Improvement of the Río Nare Confluence

### III, 3.3

However, three remarks must be made. Firstly, the restriction of the width of the Río Magdalena near the Río Nare Confluence is still about twice as large as near the Río Regla Confluence; secondly, the discharge of the Río Nare is far greater than that of the Río Regla; and, thirdly, navigation on the Río Nare probably requires the execution of additional river-works in the Río Nare itself. In view of the fact that the present navigation conditions near this confluence are rather good (apart from the crossings upstream of Nare village which can easily be improved by means of recurrent dredging), it is recommended that for the time being the permanent improvement of the Río Nare Confluence is not considered. It should be postponed to a later date when the traffic flow between La Dorada (or Pto. Triunfo) and Pto. Berrío will have increased considerably. For the time being it is recommended that the required improvement of the Río Nare Confluence be brought about by means of recurrent dredging; which means that in the present situation (1973) only the crossings near km 780 need dredging.

An impression of the river-works which may be executed in the far future to attain a permanent improvement of the Río Nare Confluence may be obtained from Figure 3.2.31. This improvement is, of course, based on the present topography and should only be considered as a first impression of the nature and scope of river-works which may eventually be required.

### 3.3. PUERTO INMARCO - PUERTO BERRÍO (KM 773 -730)

#### 3.3.1. General description and design criteria

The contents of this paragraph are similar to those in Para. 3.2.1, so for a more detailed explanation of the data presented here, reference is therefore made to that paragraph.

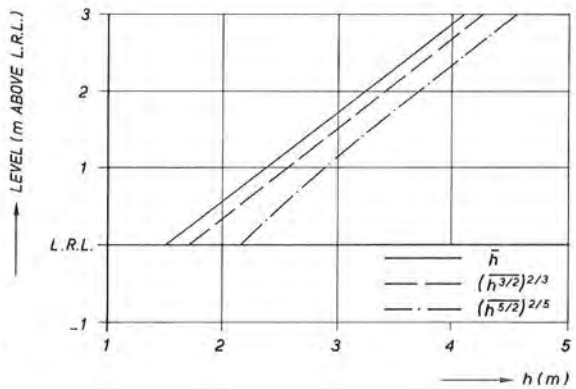
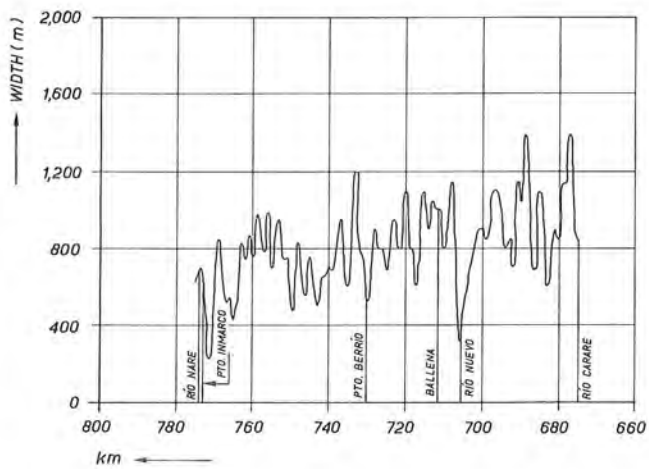
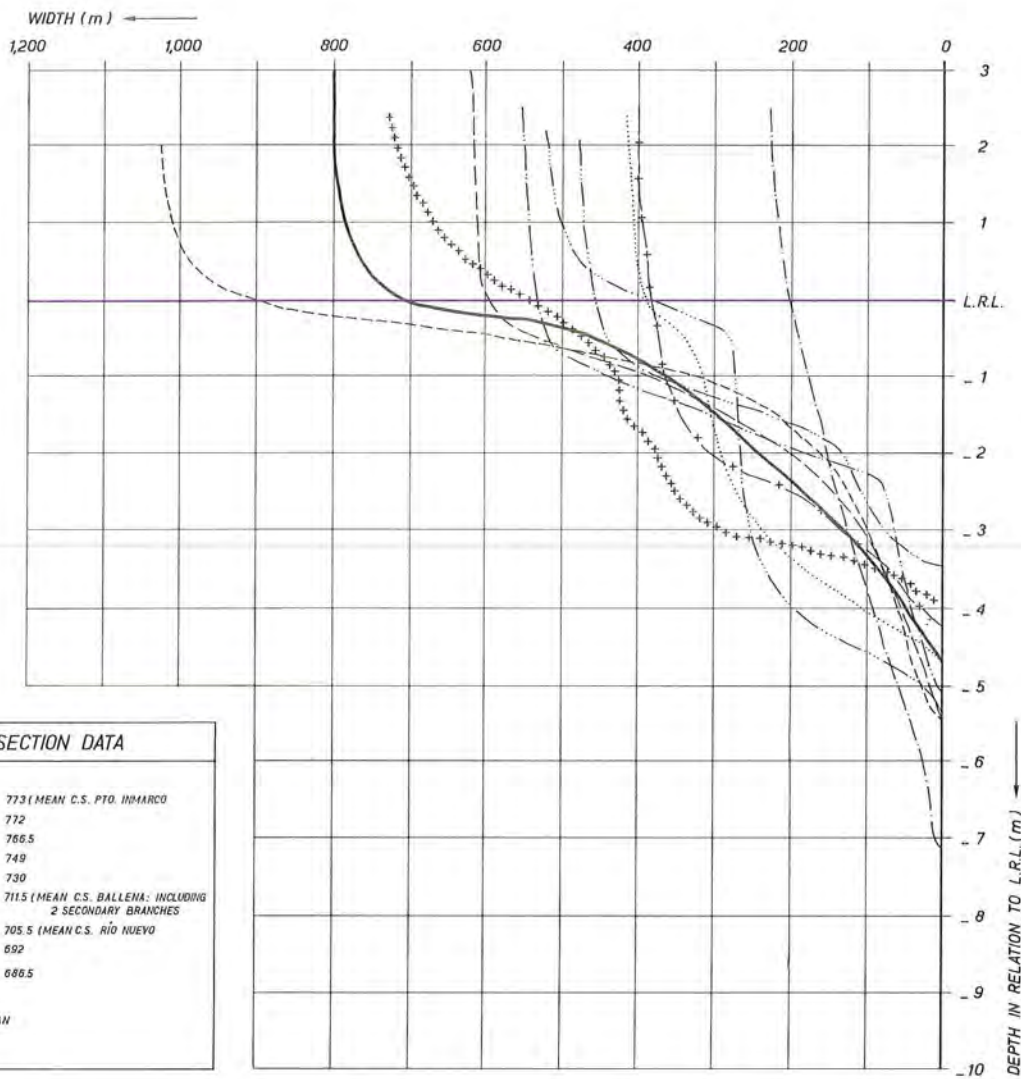
In Part II, Para. 3.5.4, it was concluded that the river section Pto. Inmarco - Pto. Berrío could be treated together with the river section Pto. Berrío - Río Carare Confluence (km 675). For that reason, the available information about the river section between Pto. Berrío and the Río Carare Confluence has also been used in this paragraph.

#### Available cross-section

The cross-sections which were measured in the river-section between Pto. Inmarco and Pto. Berrío are given as mass curves in Figure 3.3.1, which also includes the mass curve of the schematized measuring cross-section in Pto. Inmarco.

#### Schematized cross-section

The cross-sections given in Figure 3.3.1 have been schematized into an average cross-section which is also presented in the figure as a mass curve. The depth of the schematized mass cross-section has been determined as an average of the depth in the single cross-sections. The determination of the width of the mass cross-section was made in accordance with the width of the single cross-sections and the mean river width deduced from the aerial photographs. Thereafter the relation between the water-level and, respectively, the average water-depth ( $\bar{h}$ ), the value of  $(\bar{h}^{3/2})^{2/3}$  and of  $(\bar{h}^{5/2})^{2/5}$  was determined.



SCHEMATIZATION OF THE RÍO MAGDALENA km 774 - km 675

FIG. 3.3.1

Water-level gradient

From the water-level data of the gauges at Pto. Salgar, Pto. Inmarco and Pto. Berrío it was found that the average water-level gradient is in the order of  $40 \times 10^{-5}$ .

Design bend-radius and water depth in outer bend

At the dominant water-level (L.R.L. + 2 m) the conveyance of the schematized cross-section is in the order of  $Bh^{3/2} = 5,000 \text{ m}^{5/2}$ . The computation of the radial bed-level slope in an outer bend was carried out by means of Eq. 3.7.10, presented in Part II. The following values have been inserted in this equation:  $A = 10$ ,  $I_o = 40 \times 10^{-5}$ ,  $R_o = 4,000 \text{ m}$  ( $\approx 6 B$ ),  $B = 650 \text{ m}$  (see Figure 3.3.1), and  $\Delta D = 1,000 \times 10^{-6} \text{ } \mu\text{m}$  (see Part II, Figure 3.3.15). The computed cross-section is presented in Figure 3.3.2. The conveyance of this cross-section is in the order of  $Bh^{3/2} = 4,800 \text{ m}^{5/2}$ .

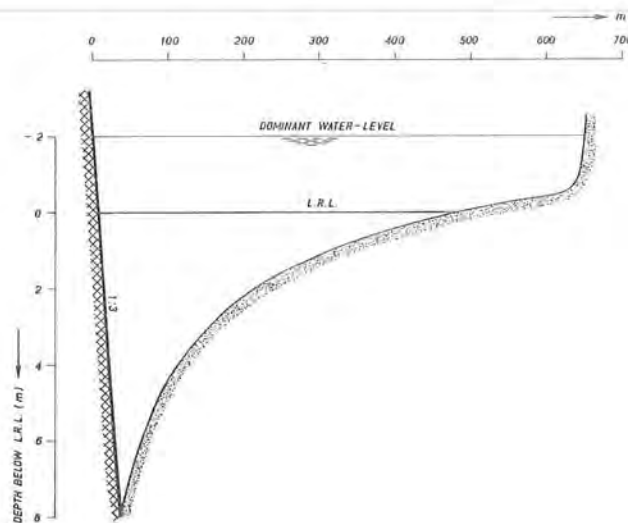


Figure 3.3.2 Computed Cross-section in River-bend (km 773-675)

From Figure 3.3.2 it can be seen that in the bends the minimum required depth of 4'6" below L.R.L. will be available over a width of about 250 m, which is amply sufficient. Even an increase of the minimum required depth to 6' below L.R.L. will provide a width of the navigation channel (about 200 m) sufficient for two-way traffic.

Design width and water depth on crossing

For the present traffic on the Río Magdalena a minimum depth of 4'6" below L.R.L. is considered sufficient for the river section Pto. Inmarco - Pto. Berrío. However, a minimum water-depth of 6' below L.R.L. in this river section is also being taken into account, in view of a possible increase of the traffic flow to and from Pto. Inmarco.

The extent of the retarded scour is taken at 2'6" (= 0.75 m), which is confirmed by the measurements presented later on in this paragraph and the morphological computations of the Río Regla Confluence given in Para. 3.4.

Minimum water depth 4'6" (= 1.35 m) below L.R.L.

If the shape of the cross-section is considered to be a superposition of the computed shape of the cross-sections in two consecutive bends, it follows from Figure 3.3.2 that the width at the crossing should be about 700 m (determined at a level of 0.60 m below L.R.L. with an allowance of 0.75 m for local retarded scour). A rectangularly-shaped cross-section with a conveyance of about  $Bh^{3/2} = 5,000 \text{ m}^{5/2}$  (see Figure 3.3.1) would result in a width at the crossing of about 1,200 m. As the first assumption regarding the shape of the cross-section on the crossing is somewhat more reliable (see Para. 3.2.1), a value of about 850-900 m should, therefore, be kept for the width at the crossing.

Minimum water depth 6' (= 1.80 m) below L.R.L.

If an increase of the minimum water depth to 6' below L.R.L. will be required, the extent of the retarded scour which can be expected needs to be reconsidered. In the examples used for the morphological computations, the initial bed-level was located at about L.R.L., and it will be clear that when the initial bed-level is lower, the retarded scour will be less. No computations have been carried out for such cases from which the extent of the retarded scour could be found. Nevertheless, an estimated retarded scour of 1'6" is being taken into account as well as no retarded scour at all.

If a retarded scour of 1'6" (0.45 m) is assumed, it follows from Figure 3.3.2 that for a shape of the cross-section composed from the computed profile in an outer bend, the width on the crossing should be about 500 m (determined at a level of 1.35 m below L.R.L. with an allowance of 0.45 m for local scour). If no retarded scour at all is taken into account, this width reduces to about 400 m. For a rectangularly-shaped cross-section a width of about 800 m can be computed for a 1'6" retarded scour, against 675 m without retarded scour. The final width on the crossing must be chosen in this range of computed values, although it appears likely that a solution based on a composed profile of the crossing with a channel along both banks is more reliable. Also the type of construction of the river-works is important. As the length of groynes can still be adjusted at a later date (when it is known that the required depth is still not reached), a width of about 600 m seems to be sufficient. However, if a solution by means of a spur-dike is projected, the width of the river will be less adjustable and a smaller width (500 m) can then better be taken.

River stretches requiring improvement

In the Schedule of Operations no special mention is made of stretches in the Pto. Inmarco - Pto. Berrfo river section which require improvement, which is more or less confirmed by the longitudinal soundings given in Part II, Figures 3.3.23 to 3.3.25.

The temporary improvement of the whole river section by means of recurrent dredging is discussed in Para. 3.3.2. Upstream of the bridge at Pto. Berrfo the Rfo Magdalena is very wide and shallow, and the navigation channel is often difficult to find. Some remarks regarding the possible means of improvement are made in Para. 3.3.3, while reference is made to Para. 3.4.3.

3.3.2. Temporary improvement by means of dredging

In compliance with the remarks made in Para. 3.2.2, the longitudinal sounding recorded in February 1972 is again being used for the determination of the total amount which needs to be dredged recurrently in this river-section. The least available depth during this survey was recorded near km 734 where the bed-level was found at about L.R.L.

A distinction is again made for an improvement of this river-section to 4'6" and 6' respectively below L.R.L. Moreover, the total volumes which need to be dredged have been determined, firstly, without any retarded scour and, secondly, with some extent of retarded scour. For the crossings where the bed-level was recorded at L.R.L. (near km 734), the least available depth was already less than 4'6" and dredging should in fact have already been done. For those crossings no local improvement due to retarded scour has been considered. For crossings with an initial bed-level above 4' below L.R.L. a retarded scour of 2' is being taken into account, while for still deeper crossings no erosion is considered during the fall of the water-level to L.R.L. (this means, in fact, that the reduction by the retarded scour of the volumes to be dredged is in the same order of magnitude as for improvement to 4'6" below L.R.L. or to 6' below L.R.L.).

Minimum water depth 4'6" (= 1.35 m) below L.R.L.

The total volumes which need to be dredged for a 50 m wide channel have been computed in the two ways outlined above and are given in Table 3.3.1. An allowance of 25% has been made in these figures: 10% for waste and side-slopes of the channel and 15% for the assumption that the recorded depth is taken to be representative for that part of the cross-section (50 m wide) where the channel will be dredged.

River section	Kilometers	Retarded scour not included			Retarded scour (2') included		
		Volume (m <sup>3</sup> )	Extra (25%) (m <sup>3</sup> )	Total Volume (m <sup>3</sup> )	Volume (m <sup>3</sup> )	Extra (25%) (m <sup>3</sup> )	Total Volume (m <sup>3</sup> )
Pto. Inmarco - Pto. Berrfo	773-730	240,000	60,000	300,000	125,600	31,400	157,000

Table 3.3.1 Quantities to be Dredged between Pto. Inmarco and Pto. Berrfo for Improvement to 4'6" below L.R.L.

The total volume which needs to be dredged annually in the river section between Pto. Inmarco and Pto. Berrfo amounts to about 160,000 m<sup>3</sup> for an improvement to 4'6" below L.R.L.

Minimum water depth 6' (= 1.80 m) below L.R.L.

For a 50 m wide channel the total volumes which need to be dredged to attain this improvement are given in Table 3.3.2. In view of the, generally, deeper cuts of the dredger an allowance of 30% has been made: 15% for waste and side-slopes of the channel and 15% for the assumption that the recorded depth is taken to be representative for that part of the cross-section (50 m wide) where the channel will be dredged.

River section	Kilometers	Retarded scour not included			Retarded scour (2') included		
		Volume (m <sup>3</sup> )	Extra (30%) (m <sup>3</sup> )	Total Volume (m <sup>3</sup> )	Volume (m <sup>3</sup> )	Extra (30%) (m <sup>3</sup> )	Total Volume (m <sup>3</sup> )
Pto. Inmarco - Pto. Berrío	773-730	562,000	169,000	731,000	421,000	126,000	547,000

Table 3.3.2 Quantities to be Dredged between Pto. Inmarco and Pto. Berrío for Improvement to 6' below L.R.L.

The total volume which needs to be dredged annually in the river-section between Pto. Inmarco and Pto. Berrío amounts to about 550,000 m<sup>3</sup> for an improvement to 6' below L.R.L.

### 3.3.3. Río Magdalena upstream Pto. Berrío (km 735 - 730.5)

#### Introduction

In front of Pto. Berrío the Río Magdalena is rather narrow (some 700 m) and offers good conditions for navigation. This is important in view of the investments in port facilities at the Pto. Berrío water-front along the left bank of the Río Magdalena (km 730). As can be seen on the route map (Part II, Figure 3.3.25) the approach to Pto. Berrío from a downstream direction is not too bad, although locally some improvement is required. Downstream of km 725 the least available depth decreases due to the wide river-bed and the number of parallel branches. The river section downstream of Pto. Berrío is treated in the next paragraph (3.4).



### III, 3.3

The approach to Pto. Berrío from an upstream direction is more difficult. Upstream of the bridge, which spans the Río Magdalena on the southern side of Pto. Berrío, the river widens considerably, resulting in a smaller depth. However, this represents only part of the problem, because the talweg can often not be found. ADENAVI's system of marking the navigation channel by means only of beacons does not suffice in this region. Here, a combination of buoyage and beaconing should be pursued in future, which would immediately result in a considerable improvement.

The execution of river-works to achieve a permanent improvement of this area can also be studied. The river's topography is such (as, e.g., at Pto. Inmarco and Río Nuevo) that in future river-works can be projected from Pto. Berrío in an upstream direction. However, at present (1973) the transport flow between Pto. Berrío and Pto. Inmarco is such that a navigable channel with a least available depth of 4'6" below L.R.L. will meet the requirements. Such a channel can best be maintained by means of recurrent dredging.

A navigable channel with a least available depth of 6' below L.R.L. was also considered, which would increase the total volume to be dredged annually between Pto. Berrío and Pto. Inmarco from about 160,000 m<sup>3</sup> (for 4'6") to about 550,000 m<sup>3</sup> (for 6'). For the river section just upstream of Pto. Berrío (km 735 - km 730) these volumes are respectively about 100,000 m<sup>3</sup> for 4'6" channel and about 180,000 m<sup>3</sup> for a 6' channel. A comparison between these figures shows that for navigation purposes only, the permanent river improvement upstream of Pto. Berrío is not yet justified.

Nevertheless, the approach to the river port of Pto. Berrío should not be left out of account. At present, the main channel passes in front of Pto. Berrío along the left bank of the Río Magdalena, offering sufficiently deep water in front of the quay wall. A change in the river's course upstream of Pto. Berrío may, however, result in the main channel shifting to the right bank, causing sedimentation in front of the quay wall. A study of the depth-contour lines of the sounding prepared by the Julius Berger Konsortium in 1923 more or less reveals this phenomenon. Therefore, the execution of river-works may still be considered in future to even out the locally strong reduction of the river's width just upstream of the bridge. For that reason, the topographical features of the river stretch near Pto. Berrío have been schematized and used for the three types of morphological computations outlined in Part II, Para. 3.6.4.

Obviously, an improvement of the river near Pto. Berrío will result in changes downstream. Such an improvement is, therefore, inter-related to the improvement of the river section between Pto. Berrío and Río Nuevo (km 706.5). As this subject is further treated in Para. 3.4.3, the outline of a possible scheme for improvement upstream of Pto. Berrío is also postponed to that paragraph. Here only the results given in Part II, Para. 3.6.4 are reconsidered, together with some additional information about the river-crossing near km 733.

#### Results of improvement

In Part II, Para. 3.6.4 a schematization is given of the river's topography near Pto. Berrío. The width of the river downstream of the bridge was normalized to 700 m, while upstream a constant width of 1,200 m was assumed. First of all, the question was put whether the narrow section of 700 m actually offers sufficient depth when the water-level falls to

L.R.L. This question was, in the first instance, answered by means of a computation (type a, see Part II, Para. 3.6.4) for equilibrium conditions, which showed an available depth of 1.88 m below L.R.L. which is, indeed, sufficient for a 6 ft channel. It is recalled from Para. 3.3.1 that the width of a crossing for a rectangularly-shaped cross-section was computed as 675 m if no retarded scour was included (an assumption illustrated later on when considering the non-permanent computations: type c). Obviously these results must agree, because the same assumptions were made for both, namely, one-dimensional computations with parallel flow-lines.

Thereafter, river-works were carried out over a length of about 2,000 m upstream of the bridge, reducing the width from the initial 1,200 m to 700 m. It followed that the water-level gradient in the improved section would decrease to  $25 \times 10^{-5}$  (equal to the water-level gradient downstream of the bridge), bringing about a drop in water-level and, consequently, in the bed-level too of 0.15 m. By means of an error function, the required time interval can be computed (type b computation) before this drop in bed-level and water-level will reach a certain place along the river upstream of the executed river-works.

Finally, the influence of the river's regime was studied, as well as the time interval required to obtain an equilibrium condition in the improved river section (type c computation). The influence of the river's regime was studied in two ways: with an average regime based on the 50% frequency-curve of the water-level at the Pto. Berrío gauge, and then with a steep, schematized, fall of the water-level to about L.R.L. at the end of such a regime. The change in bed-level for these two conditions was found to be very small. It was also found that in the total river section with a width of 700 m, the minimum available water depth was 6 ft when the water-level falls to L.R.L., which is in good agreement with the results found in steady flow (computation a), where no retarded scour was considered. Moreover, the computation showed that the change in bed-level is about equal to the change in water-level. This is not strange in view of the schematizations which have to be made for such computations, including one-dimensional (which implies a straight river section with parallel flow-lines) and uniform depth (Part II, Para. 3.6.2).

However, a study of the plan-form of the river shows that this assumption does not hold. About 1,500 m upstream of the bridge (Hacienda Sebastopol) the Río Magdalena curves in an easterly direction. Consequently, after improvement the plan-form will also show a curve followed by a crossing of the flow to the right bank in the narrowed river section. This implies a greater depth along the outer bank and a smaller depth (less than 1.80 m) at the crossing. It is therefore logical that in Para. 3.3.1 a still smaller width of the river was found.

The least available depth found at the crossing near km 773 was plotted against the water-levels recorded at the Pto. Berrío gauge. This additional information is presented in Figure 3.3.3.

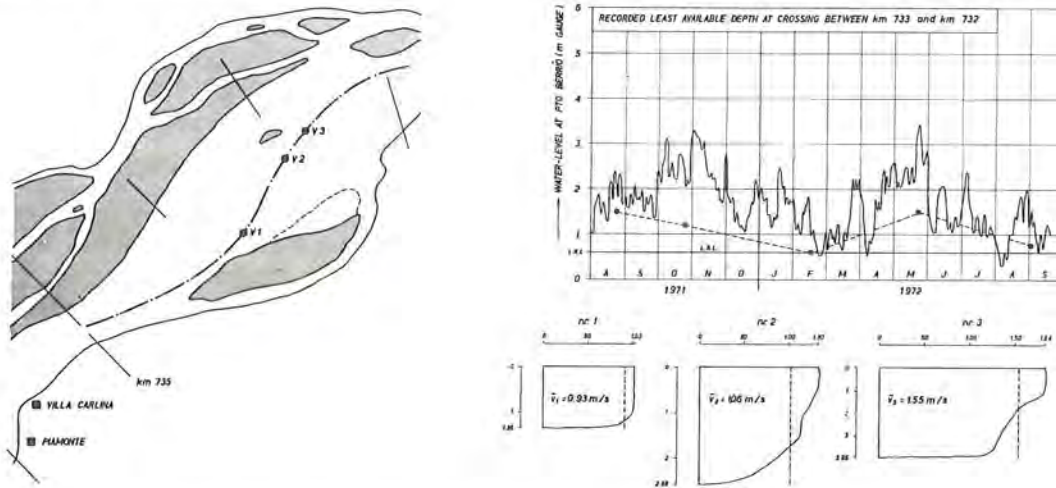


Figure 3.3.3 Recorded LAD on Crossing near km 773

3.4. PTO. BERRÍO - BARRANCABERMEJA (KM 730 - 630)

3.4.1. General description and design criteria

Distinction is made between the river sections upstream and downstream of the Rfo Carare Confluence. In Para. 3.3.1, it has already been mentioned that the Pto. Inmarco - Pto. Berrío section could be extended to the Rfo Carare Confluence. The section downstream of the Rfo Carare Confluence does not change much until the (backwater) influence of the Rfo Sogamoso becomes apparent.

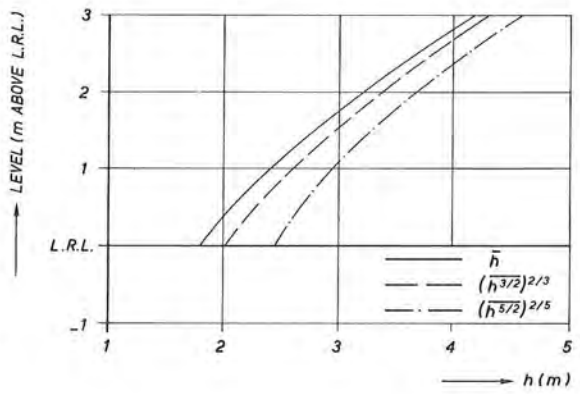
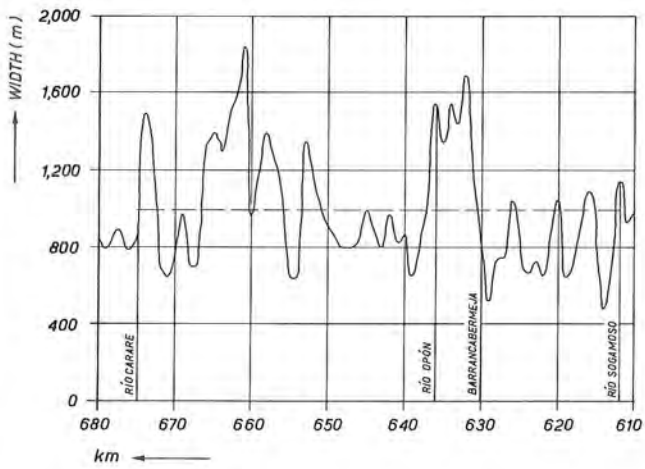
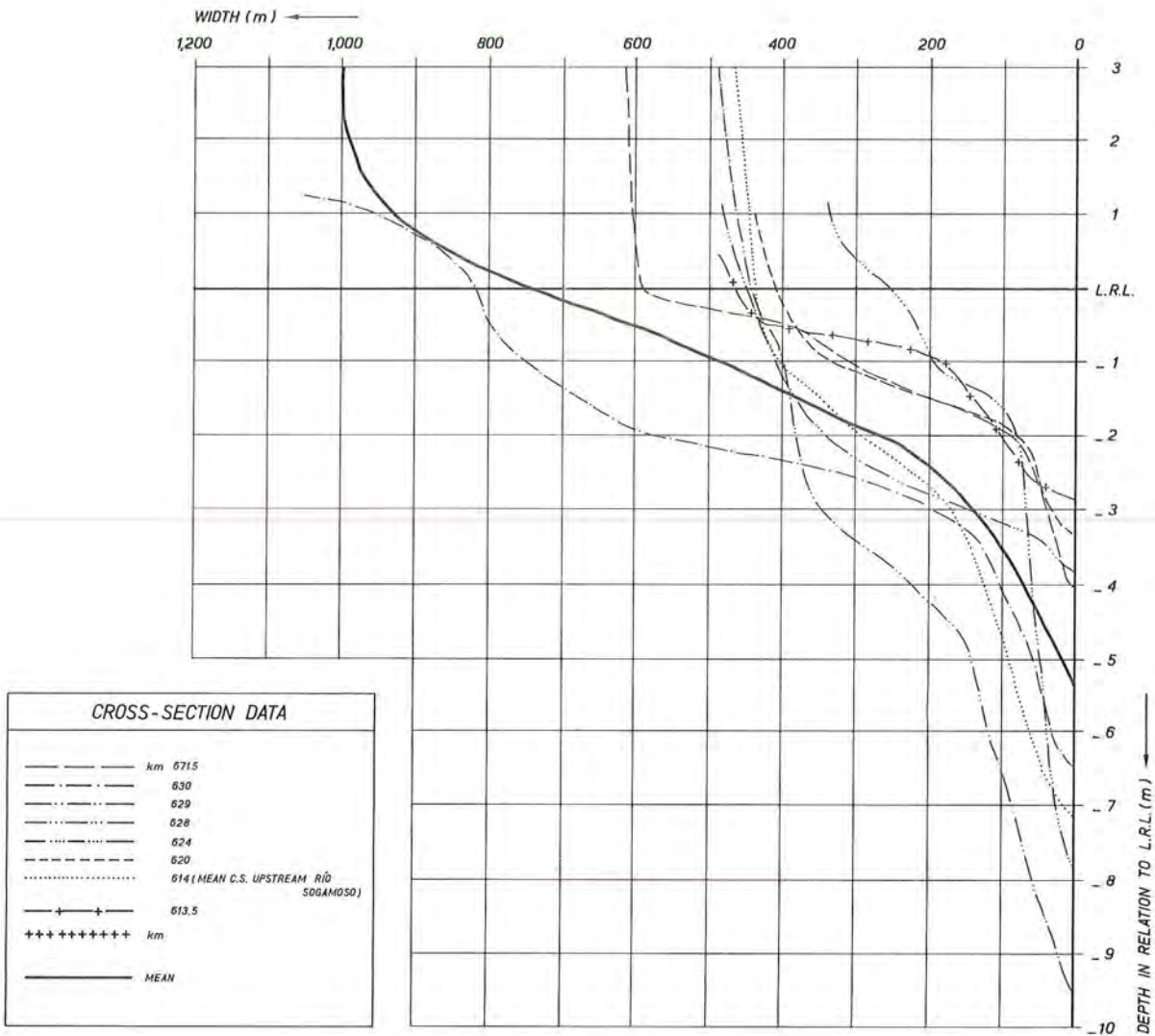
In Part II Para 3.5.4, it was concluded that for the morphological computations, the stage discharge curve for Pto. Berrío could be used upstream of the backwater effects of the Rfo Carare, while for the section downstream of this confluence till upstream of the Rfo Sogamoso Confluence, the rating curve for Barrancabermeja must be preferred.

Available cross-section

The available cross-sections of the section upstream of the Rfo Carare Confluence have already been presented in Figure 3.3.1. The available cross-sections of the section downstream of the Rfo Carare Confluence are given in Figure 3.4.1.

Schematized cross-section

In the Figures 3.3.1 and 3.4.1, the schematized cross-sections have also been given, which data were gathered in the same way as explained in Para. 3.2.1. It may be seen in those figures, that the average width upstream of the Rfo Carare Confluence is 800 m, while the average width downstream increases to 1,000 m (which is more or less the same as the width found downstream of the Rfo Sogamoso Confluence; see Figure 3.5.1).



-SCHEMATIZATION OF THE RÍO MAGDALENA km 675 - km 612

FIG. 3.4.1

Water-level gradient

From the water-level data of the gauges at Pto. Berrío and Barrancabermeja, it was found that the average water-level gradient is about  $35 \times 10^{-5}$ . However, the local water-level gradients may differ considerably from this average value; for example, in the wide section near Ballena (upstream of the Río Regla) gradients of  $40$  to  $60 \times 10^{-5}$  have been found, while in the narrow Río Nuevo (downstream of the Río Regla) the gradients decreased and were found to be  $20$  to  $25 \times 10^{-5}$ . For the computations of the river improvements near the Río Regla (which aim at an increase of the available water depth, by narrowing the width), a value of  $25 \times 10^{-5}$  has been used but, generally, for the section a value of  $35 \times 10^{-5}$  should be taken.

Design bend-radius and water depth in outer bend

As the average water-level gradient ( $35 \times 10^{-5}$ ) in the section upstream of the Río Carare Confluence is more or less the same as that in the Pto. Inmarco - Pto. Berrío section, the same values for the bend-radius and the depth in the outer bend may be used ( $R = 4,000$  m, depth in the outer bend is  $8.0$  m below L.R.L.; see Para. 3.3.1).

For the possible river-works near the Río Regla Confluence, more specific data were available from the measurements and those have been used in the computations presented in Para. 3.4.4.

For the section downstream of the Río Carare Confluence, the conditions are different again, and for that section the computed radial bed-level slope is presented in Figure 3.4.2.

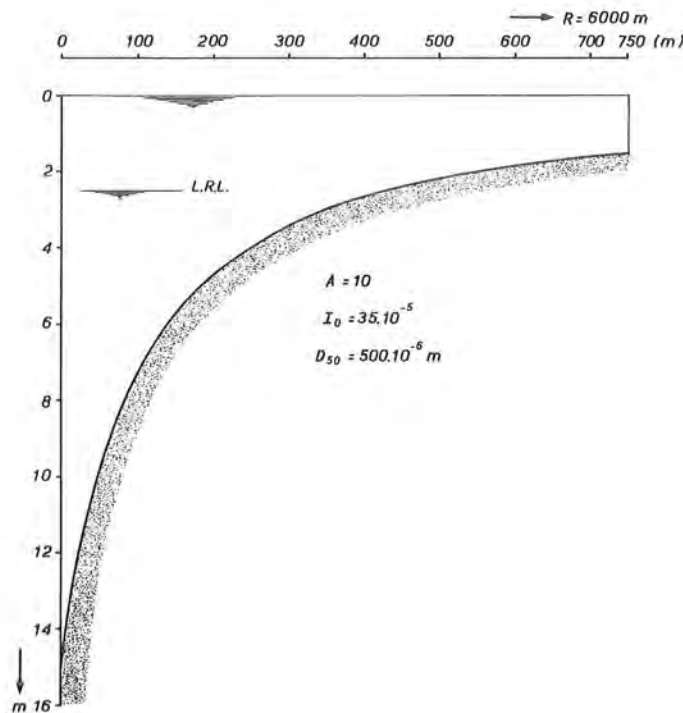


Figure 3.4.2 Computed Cross-section in Outer Bend, Downstream of the Río Carare Confluence

Design width and water depth on crossing

The required width on a crossing between Pto. Berrío and the Río Regla Confluence, cannot be taken as equal to that found for the section Pto. Inmarco - Pto. Berrío because the required least available depth is greater and the amount of retarded scour differs. The L.A.D. was taken as 7'6", but as this may not be warranted by the present river traffic, possibly a depth of 6' is economically more realistic. The retarded scour computed for a crossing near km 667 is found to be 1.20 m (see Para. 3.4.5) but this amount seems rather high when compared with the computations near the Río Sogamoso Confluence (0.60 m) and in the Pto. Inmarco - Pto. Berrío section (0.75 m). This is possibly due to a wrong schematization of the crossing, or that the initial bed-level was chosen badly, although the measured variations in bed-level at this crossing are also very large (see Figure 3.4.17).

At the crossing near the Río Regla Confluence, the amount of retarded scour was computed as about 0.50 m (see Figure 3.4.12). This value was found, when after a complete regime, the water-level dropped drastically to L.R.L. As this value appears to be more realistic, for the whole section a value of 0.60 m has been used as retarded scour.

The required width on a crossing, as found by the various methods outlined in Para. 3.2.1, are given in Table 3.4.1.

Shape of cross-section on crossing	L.A.D. (ft)	width on crossing (m)	
		retarded scour not included	retarded scour included
Composite	6'	450	550
	7'6"	400	480
Rectangular	6'	830	1,040
	7'6"	700	860

Table 3.4.1 Computed Width on Crossing

For a channel with a least available depth of 7'6", a width of 600 m can be taken as a starting point.

River stretches requiring improvement

According to the Schedule of Operation, in the section Pto. Berrío - Barrancabermeja, there exist three bad locations: a short stretch of about 4 km just downstream of Pto. Berrío, the Río Regla Confluence, and the Río Carare Confluence. This is in agreement with the measured length profiles, as presented in Part II. Those locations were during the Study also difficult for navigation, although occasionally they may shift a few kilometers.

Downstream of the Río Carare Confluence, a few bad crossings may also be present (often located near Chucurí), but these form only a temporary restriction to navigation.

The section downstream of Pto. Berrío is treated in Para. 3.4.3. Although, in fact, the complete stretch between Pto. Berrío and the Río Regla Confluence offers problems, in the Schedule of Operation only the stretch between km 720 and 730 was mentioned.

### III, 3.4

An improvement of this section, however, cannot be considered, without taking into account an improvement near the Río Regla Confluence at the same time.

The Río Regla Confluence is dealt with in Para. 3.4.4. As explained, the stretch of the Río Magdalena near this confluence is one of the most outstanding of a number of such narrows along the whole river. Downstream of the Río Regla, the narrow stretch (Río Nuevo) is fixed by rock and/or hard soil, and such a location is therefore considered to be very suitable to start the execution of permanent river-works. Such river-works can be extended in an upstream direction in the course of years, until the next narrow river stretch (Pto. Berrío) has been reached, and in such a way an improvement of the river can gradually be achieved. Although the execution of these river-works is at present not yet warranted by the river traffic, the required measurements and morphological computations for the design of such works are treated in Para. 3.4.4, because a future increase of the navigation between Barrancabermeja and Pto. Berrío may warrant their execution there, or at similar locations.

The Río Carare Confluence is discussed in Para. 3.4.5. In this area there are a number of long secondary branches, which often offer sufficient depth for navigation. The entrance to and exit of these channels, however, are often blocked by sand banks, which either will have to be removed by dredging, or by the execution of more permanent river-works.

#### 3.4.2. Temporary improvement by means of dredging

The amounts to be dredged in the Pto. Berrío - Barrancabermeja section, have been determined for a least available depth of 6' and 7'6" respectively, and are shown in Table 3.4.2. The length profile measured in February 1972 has been used, while a retarded scour of 2' was taken into account. 30% has been added to the computed volumes to allow for waste and side-slopes of the dredge cut (15%), and for the assumption that the recorded depth is representative for the full width (50 m) of the cut (15%).

River-section	Kilo- meters	L.A.D. below L.R.L.	Retarded scour not included			Retarded scour (2') included		
			Volumes Extra (30%) (m <sup>3</sup> )	Total Volumes (m <sup>3</sup> )	Total Volumes (m <sup>3</sup> )	Volumes Extra (30%) (m <sup>3</sup> )	Total Volumes (m <sup>3</sup> )	Total Volumes (m <sup>3</sup> )
Pto. Berrío - Barrancabermeja	730-630	6'	730,000	220,000	950,000	384,000	116,000	500,000
		7'6"	1,335,000	374,000	1,709,000	777,000	223,000	1,000,000

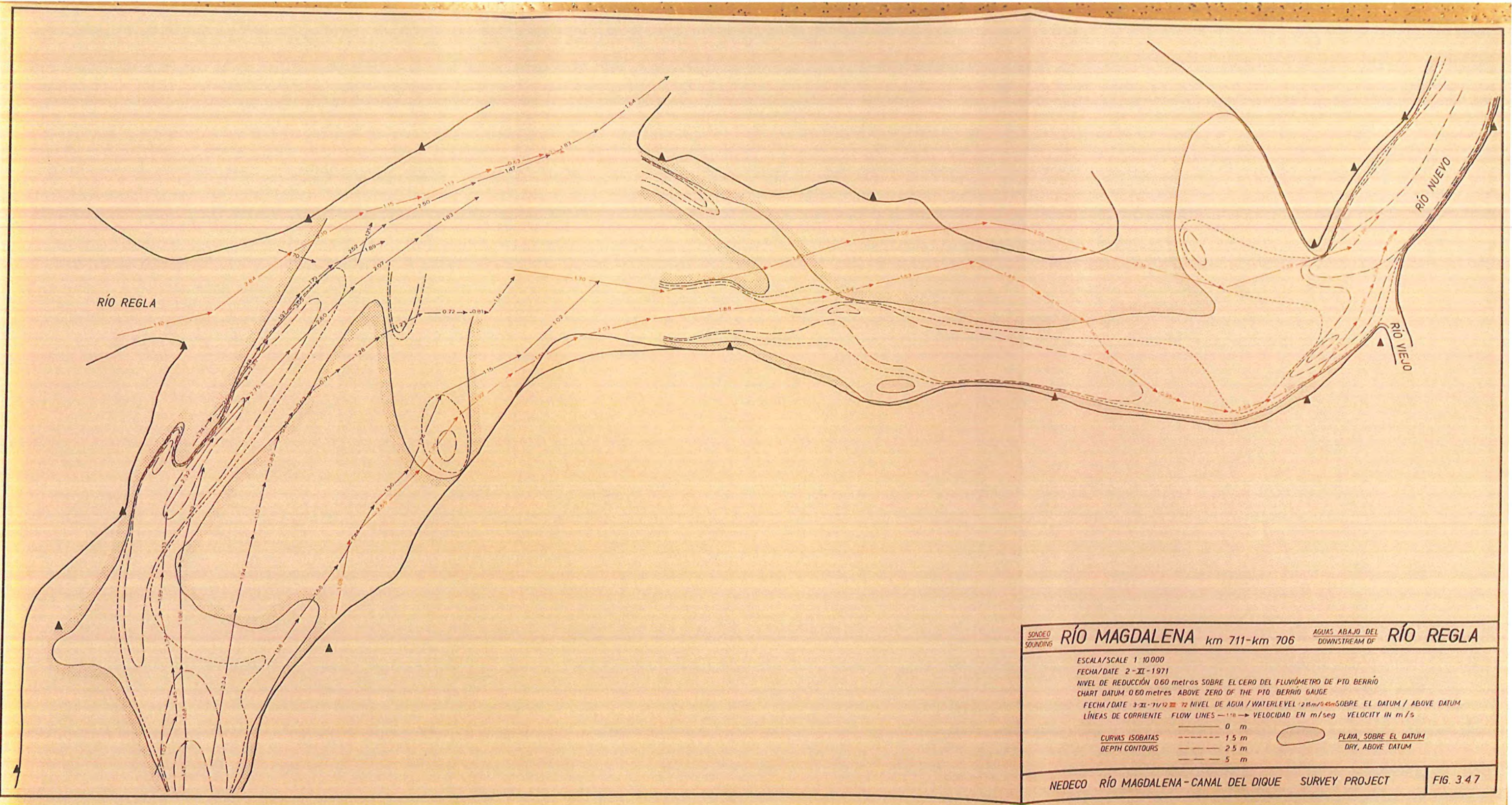
Table 3.4.2 Quantities to be Dredged Between Pto. Berrío and Barrancabermeja

#### 3.4.3. The section between Pto. Berrío and the Río Regla Confluence (km 730 - km 710)

It is difficult to give a case history of this section, because of lack of good aerial photographs, while some data of the Julius Berger Survey are missing (at least not available to the Mission). The general impression about this section is, that it is rather stable. This is not surprising, as at both ends the river is fixed by diluvial deposits, while at



PERMANENT IMPROVEMENT BETWEEN PTO. BERRÍO AND RÍO REGLA CONFLUENCE | FIG. 3.4.3

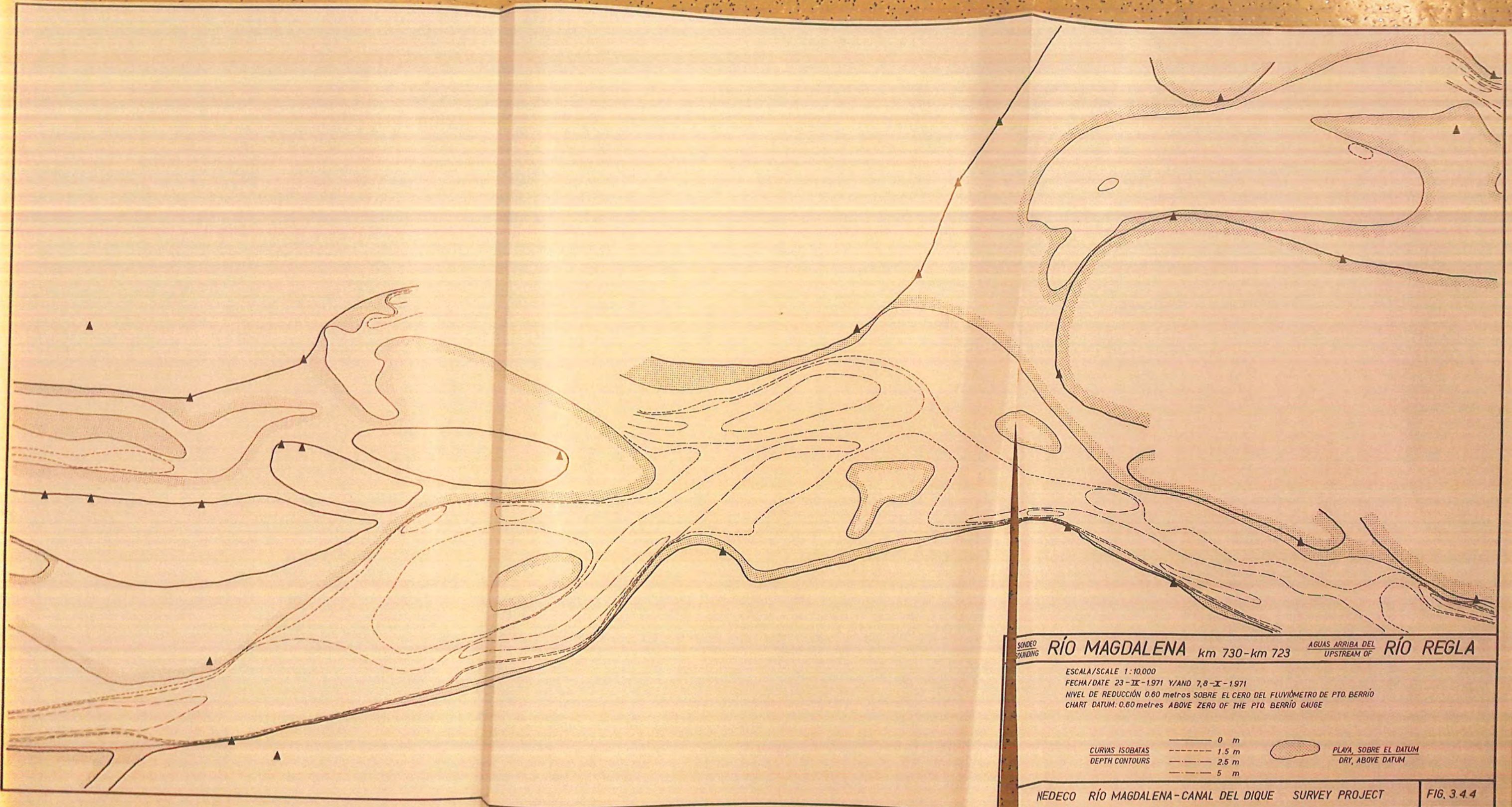


**RÍO MAGDALENA** km 711-km 706 AGUAS ABAJO DEL RÍO REGLA  
SONDEO SOUNDINGS DOWNSTREAM OF

ESCALA/SCALE 1 10000  
 FECHA/DATE 2-XI-1971  
 NIVEL DE REDUCCIÓN 0.60 metros SOBRE EL CERO DEL FLUJÓMETRO DE PTO BERRÍO  
 CHART DATUM 0.60 metres ABOVE ZERO OF THE PTO BERRÍO GAUGE  
 FECHA/DATE 3-XI-71/12 72 NIVEL DE AGUA / WATERLEVEL 2.15m/0.45m SOBRE EL DATUM / ABOVE DATUM  
 LÍNEAS DE CORRIENTE FLOW LINES ———→ VELOCIDAD EN m/seg VELOCITY IN m/s

CURVAS ISOBATAS	-----	1.5 m	PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
DEPTH CONTOURS	-----	2.5 m	
	-----	5 m	

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT FIG. 3.47



SONDEO  
SOUNDING

**RÍO MAGDALENA** km 730 - km 723

AGUAS ARRIBA DEL  
UPSTREAM OF

**RÍO REGLA**

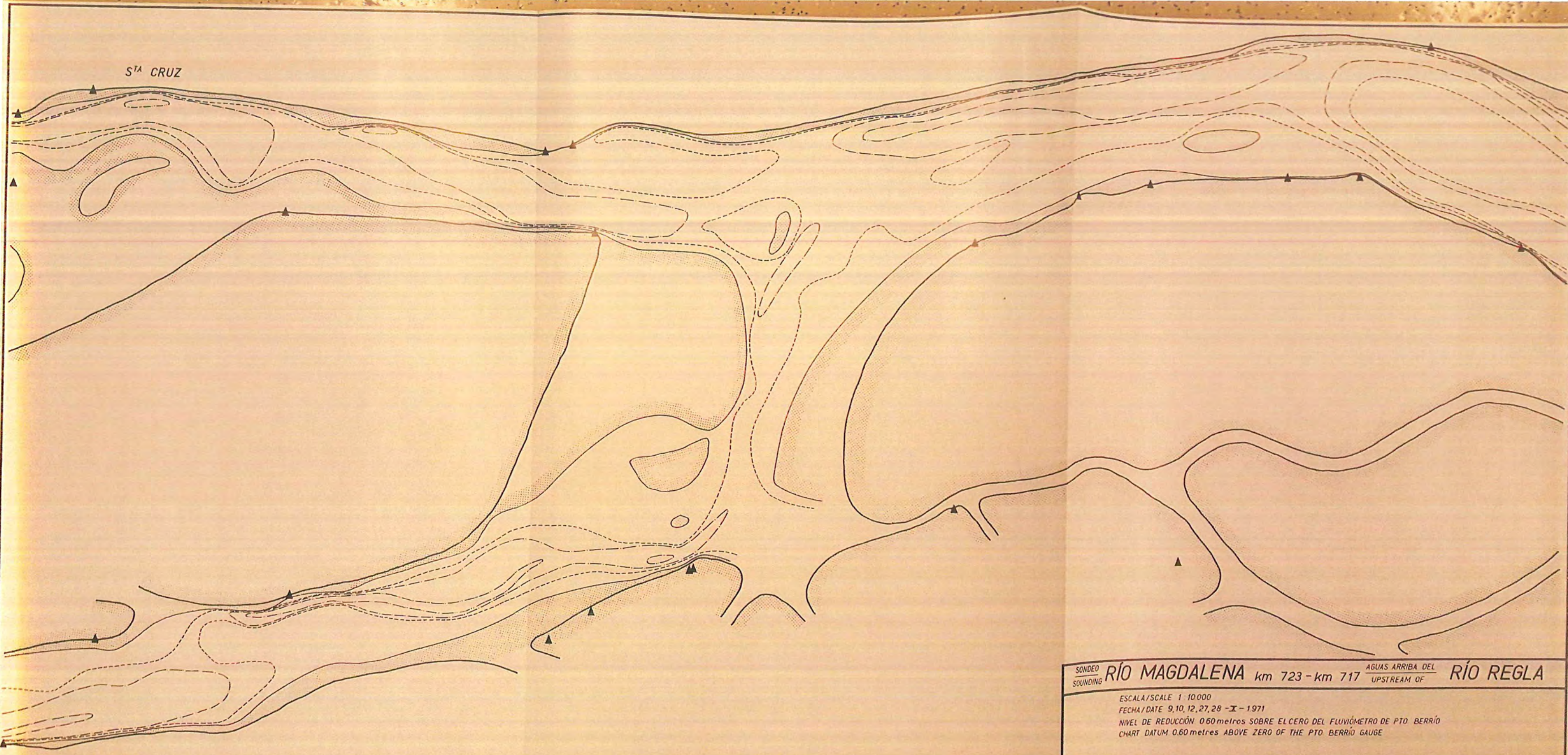
ESCALA/SCALE 1:10,000  
 FECHA/DATE 23-IX-1971 Y/AND 7,8-X-1971  
 NIVEL DE REDUCCIÓN 0.60 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. BERRÍO  
 CHART DATUM: 0.60 metres ABOVE ZERO OF THE PTO. BERRÍO GAUGE

CURVAS ISOBATAS	— 0 m	PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
DEPTH CONTOURS	- - - 1.5 m	
	- - - 2.5 m	
	- - - 5 m	

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.4.4

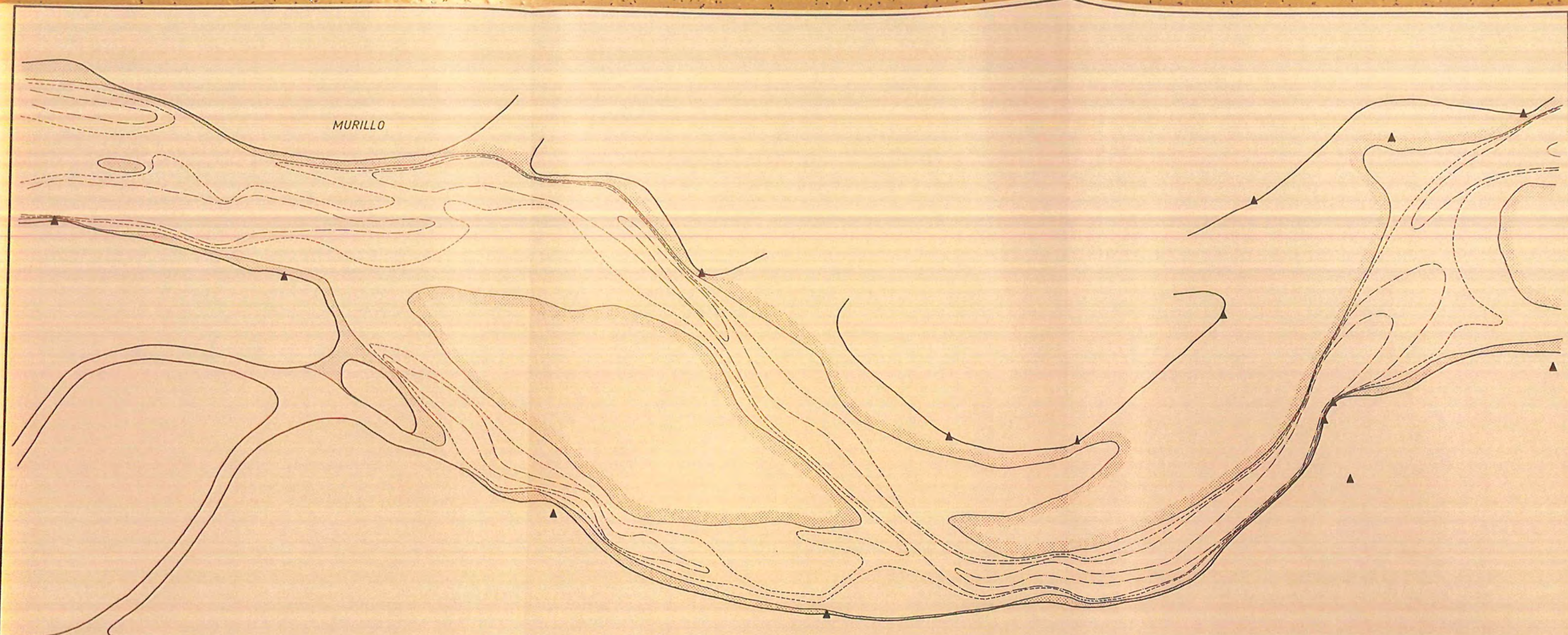
S<sup>TA</sup> CRUZ




SONDEO / SOUNDING **RÍO MAGDALENA** km 723 - km 717 AGUAS ARRIBA DEL / UPSTREAM OF **RÍO REGLA**

ESCALA / SCALE 1 10 000  
FECHA / DATE 9, 10, 12, 27, 28 -X- 1971  
NIVEL DE REDUCCIÓN 0.60 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. BERRÍO  
CHART DATUM 0.60 metres ABOVE ZERO OF THE PTO BERRÍO GAUGE

CURVAS ISOBATAS	— 0 m	○ PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
DEPTH CONTOURS	- - - 1.5 m	
	- - - 2.5 m	
	- - - 5 m	



SONDEO SOUNDING	<b>RÍO MAGDALENA</b> km 717-km 711	AGUAS ARRIBA DEL UPSTREAM OF	<b>RÍO REGLA</b>
ESCALA/SCALE 1:10000			
FECHA/DATE 28.29.30.31-X-1971 Y/AND 2-XI-1971			
NIVEL DE REDUCCIÓN: 0.60 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. BERRÍO			
CHART DATUM 0.60 metres ABOVE ZERO OF THE PTO. BERRÍO GAUGE			
CURVAS ISOBATAS DEPTH CONTOURS	<ul style="list-style-type: none"> <li>—— 0 m</li> <li>- - - - 1.5 m</li> <li>- - - - 2.5 m</li> <li>- - - - 5 m</li> </ul>		PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT			FIG. 3.4.6

regular intervals the river has a hard bank on one side. Between km 725 and 728 the river bank on the right-hand side is erosion resistant, while from km 721 to km 716 the left bank is hard. From km 712 to km 709 the right bank is hard again, thus forcing the main channel of the river from one side to the other.

When improvement of this section is considered, as far as possible use should be made of the presence of these erosion resistant deposits, while at both ends of the section (Pto. Berrío and the Río Nuevo), a proper connection to the adjoining river sections should be made. How this can be done, is shown on the alignment given in Figure 3.4.3. As a base for this alignment, the soundings given in Figures 3.4.4, 3.4.5, 3.4.6 and 3.4.7 have been used. It can be seen that the bend-radius used, differs at some places from the computed bend-radius (4,000 m), in order to follow the hard banks. To bring this into alignment with the river-works near the Río Regla and the Pto. Berrío Port, it is necessary to follow the channel opposite Sta. Cruz, and not the present navigation channel along Sta. Cruz.

#### 3.4.4. River improvement near the Río Regla Confluence (km 711 - km 706)

##### General

Together with Sebastopol, Pto. Inmarco and other places, the Río Nuevo is one of the locations along the Río Magdalena, where a narrow river section is rigidly fixed by rock or hard soil. Upstream of such a fixation, navigation problems may be expected, due to backwater effects and the rapid shifting of the channels. Near the Río Nuevo and Pto. Inmarco this situation is aggravated by the inflow from affluents, the Río Regla and Río Nare respectively.

As these navigation problems always occur at the same place and have a naturally fixed downstream boundary, they present excellent conditions to commence permanent river-works. Such river-works, however, shift the navigation problems further upstream, but when a more gentle transition to the wide (and shallow) upstream section is provided, these problems will be reduced. Once, started with the execution of permanent river-works near the Río Nuevo, this implies that eventually an extension of these works will have to be made as far as Pto. Berrío (Sebastopol), where another narrow and properly fixed section exists. In view of the short distance between the Río Nuevo and Pto. Berrío (about 25 km), this section offers good possibilities to start permanent river-works.

As is shown later, a simple comparison between the cost of recurrent dredging and of permanent river-works turns out in favour of recurrent dredging, but there are some considerations in favour of permanent river-works which have to be taken into account too:

- Permanent river-works will offer better navigation conditions than dredging.
- At several places along the Río Magdalena permanent river-works will certainly be required, not only for navigation purposes, but to protect properties, etc. Such works are at present being carried out, or have already been carried out, amongst others near La Dorada, Pto. Boyacá and Pto. Wilches. Of these works, many have been lost against appreciable cost. It would therefore have great advantages if somewhere

in this region experience could be gained in the execution of large river-works, and a tradition could be built up. In fact, to maintain the experience gained, it will be necessary for a group of workers to be continuously employed in the execution and maintenance of river-works.

- When comparing dredging with permanent river-works, it should be realised that dredging requires a large percentage of the cost in foreign exchange (the dredger) while permanent river-works can mainly be executed with locally available materials.

A few observations about the morphology near the Río Regla Confluence will be helpful before indicating a solution in the form of permanent river-works.

#### Case history

It was possible to study the morphological changes over the past 50 years, by comparing the maps prepared by the Julius Berger Konsortium (1923), the aerial photographs of April 1954, and recent photographs taken in May 1972 (see Figure 3.4.8). In 1923 as well as in 1954 the main channel followed the right bank, a situation which is much more favourable for navigation. Between 1923 and 1954 the main channel apparently shifted to the left bank, causing considerable scour, while along the right bank there was some sedimentation. The photographs reveal also, that in former times there existed situations where the left bank had scoured about 300 to 400 m more landward than in 1954. Therefore the conclusion seems logical, that the main channel switches every so many years from one side to the other, with

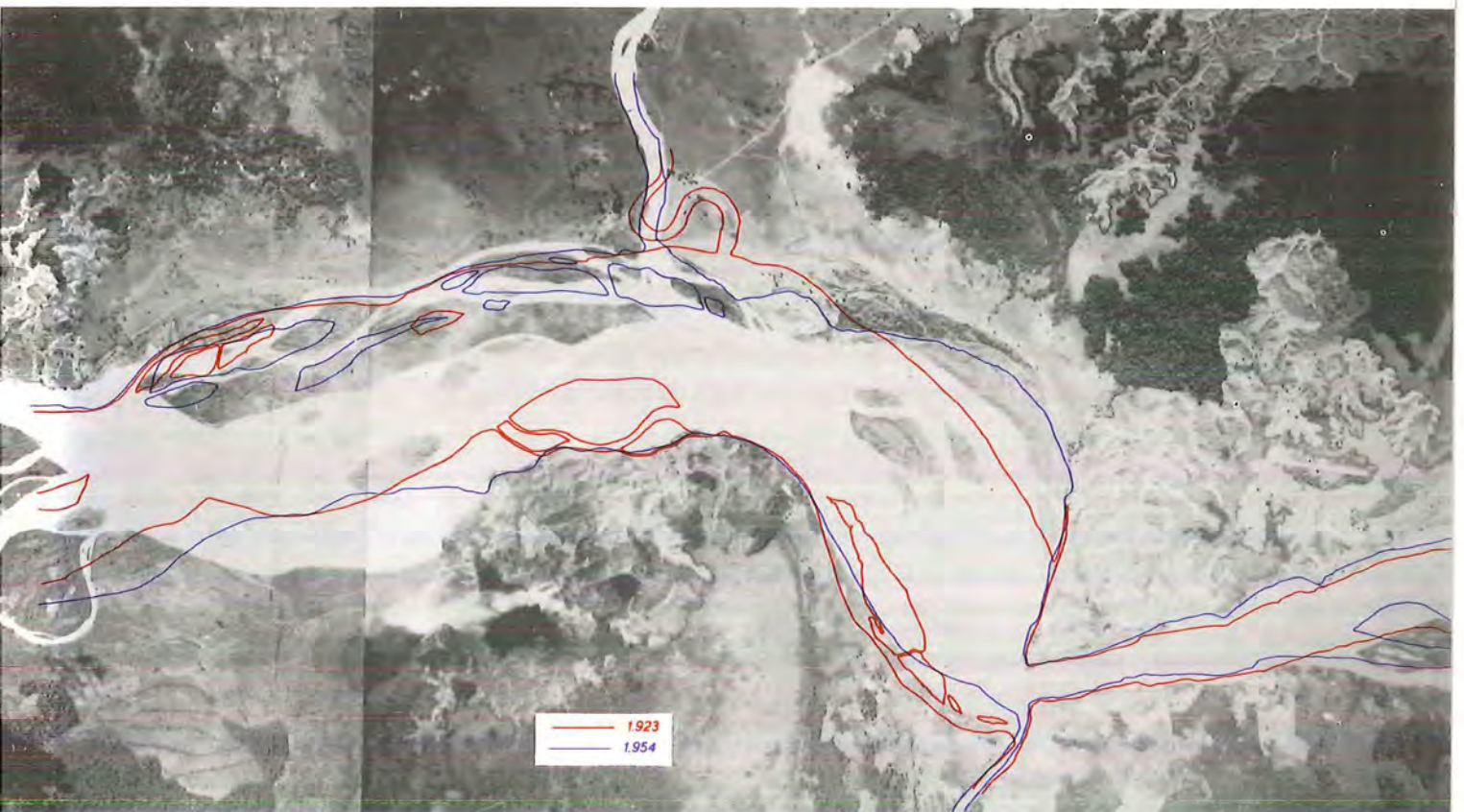


Figure 3.4.8 Case History of the Río Regla Confluence

a slight preference for the left bank. This means that the present situation, in which the main channel follows the right bank (preferable for navigation), offers good conditions for the execution of river-works, as a large portion of those works can then be made in shallow water, without heavy attack by the current (see Figure 3.4.10).

#### The alignment of the river-works

For the alignment, the design data given in Para. 3.4.1 have been used as much as possible:

Bend-radius  $R = 4,000$  m; and  
width  $B = 600$  m.

Locally (km 708), this width has been increased to nearly 700 m to obtain a smoother alignment (Figure 3.4.10).

The conveyance ( $Bh^{3/2}$ ) of a computed cross-section with a bend radius of 4,000 m, appears to be slightly smaller than the available conveyance of  $6,485 \text{ m}^{5/2}$  (compare Figures 3.4.9 and 3.3.1). In fact, a radius of 4,500 to 5,000 m would have been required (using the locally measured data:  $C = 40 \text{ m}^{1/2}/\text{s}$ ;  $I = 25 \times 10^{-5}$ , and  $Q_{\text{dominant}} = 4,100 \text{ m}^3/\text{s}$ ). With that value for the bend-radius, a depth in the outer bend may be expected of 15 m below the level corresponding with  $Q = 4,100 \text{ m}^3/\text{s}$ . With the used value of  $R = 4,000$  m, the depth will be somewhat more, but as the depths found in outer bends are generally somewhat smaller than the computed values, the depth of 15 m has been used for the design of the river-works.

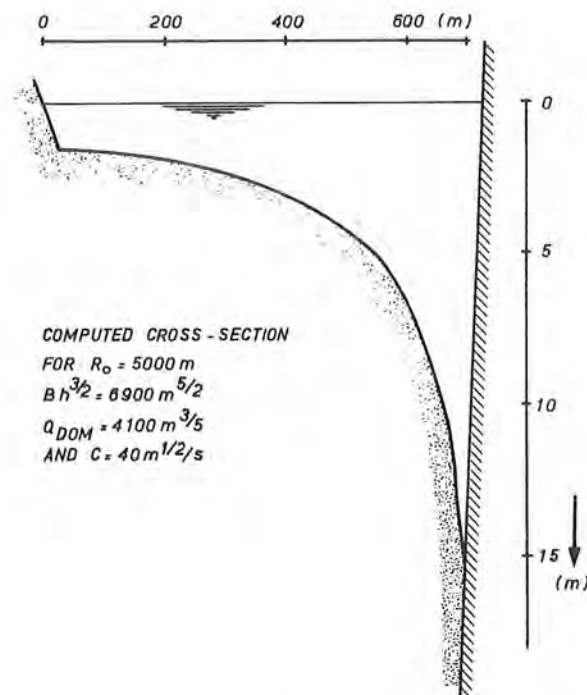


Figure 3.4.9 Computed Cross-section in Outer Bend near the Río Regla Confluence

If in future the depth will have to be increased, a narrower section can be obtained by the extension of the groyne along the right bank. It is interesting to note that Julius Berger indicated a width of 400 m for this section.

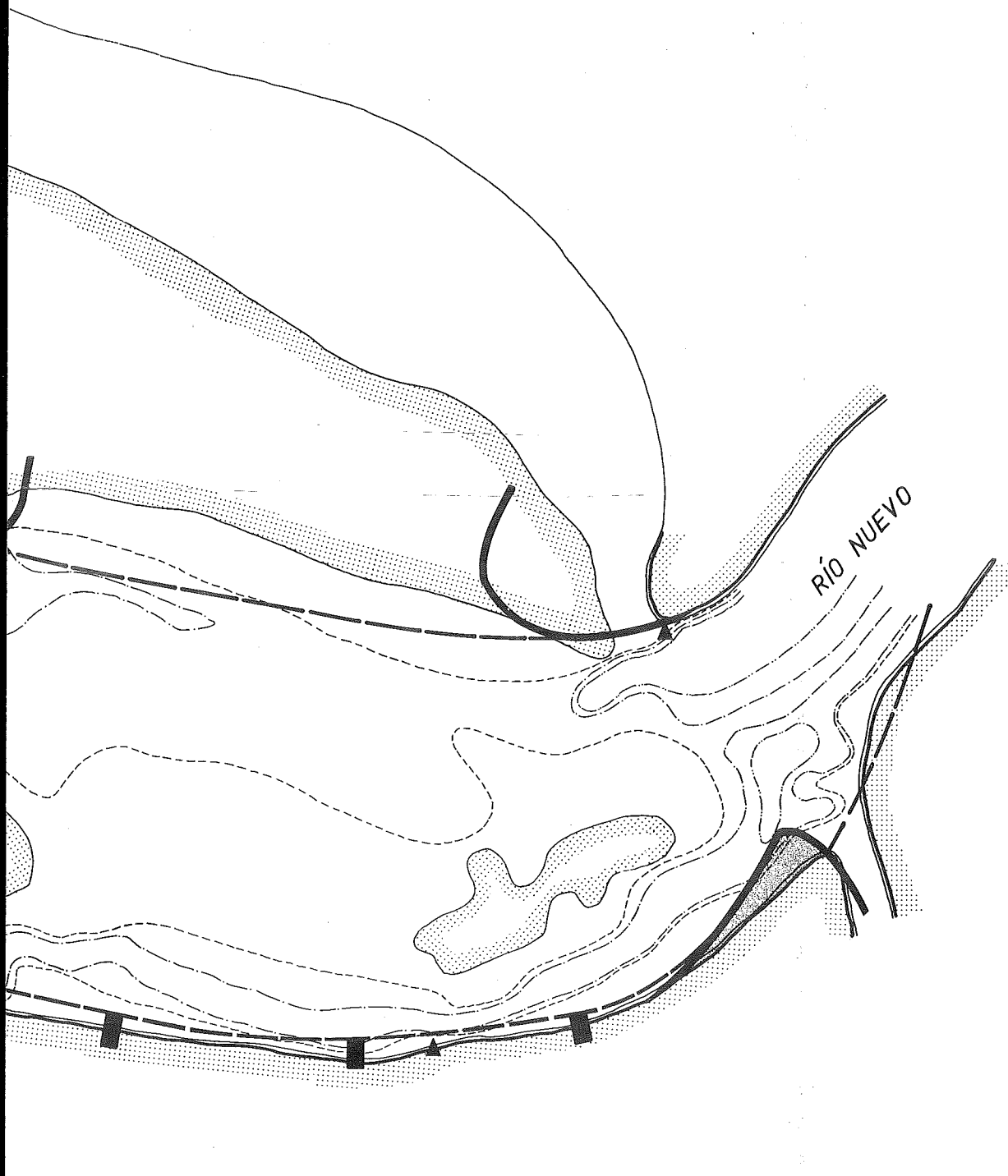
The alignment of the permanent river-works, given in Figure 3.4.10, is based on the 1971/1972 situation. Before these works are actually executed, it will be necessary to adapt the alignment in relation to the developments since 1972, while for the elaboration of the design and its details, a model study seems required. In such a model, the necessity and possibility of closing the Río Viejo may also be studied, in combination with a slightly different alignment, requiring that part of the right bank near the Río Nuevo narrows be removed, as this will probably improve the navigability (see Figure 3.4.10).

A non-steady flow computation has been carried out for this new alignment, and its results are given in Figure 3.4.11. For these computations the locally measured data have been used: for the bed material  $D_{50} = 405 \times 10^{-6}$ , while as regime the schematized 50%-discharge of Pto. Berrfo was used during one year, followed by a drastic drop of the water-level to L.R.L. (regime 5 as given in Para. 3.8 of Part II).

Similar computations have been carried out for the (non-improved) situation of 1971, and of that of 1923 as taken from the Julius Berger river maps. These results are given in Figures 3.4.12 and 3.4.13.

The following conclusion may be drawn from these computations:

- The average depth at L.R.L. in the improved section after application of regime 5 is 2.45 m (slightly more than the required 7'6"). In reality, the depth in the outer bends will be more and, possibly, on the crossings slightly less.
- The average depth for the (non-improved) situation of 1971 was found to be 1.65 m and for 1924 1.25 m. This means that after the improvement the depth increases with 0.80 m to 1.20 m.
- According to the sounding made in July 1972, the minimum depth on the crossing (related to L.R.L.) was found to be 1.40 m. This is in good agreement with the computations, considering that the regime used differs from the regime occurring in 1971-1972 and, moreover, that this minimum depth of 1.40 m was measured locally, whilst the computations give average depths over the width of the channel. The longitudinal sounding of November 1971 (also used as initial condition, see Figure 3.3.26), indicates a depth about 0.50 m less than that found in the computations. During the higher water-levels it may, however, be, that during the sounding the channel was not correctly followed.
- The 1924-situation is worse compared to 1971, not only with regard to the course the ships have to follow, but also the available depths are less.
- In 1924 the difference in head between the water-level at the upstream and downstream end of the section is found to be 1.40 m after 400 days; in 1971 this value is 1.35 m, and with the new alignment 1.20 m. This indicates a reduction of about 0.17 m, or a reduction in the water-level gradient of about  $2.7 \times 10^{-5}$  ( $\approx 0.17/6,400$ ). This is in accordance with the example of Pto. Berrfo, given in Para. 3.6 of Part II, showing that these types of river-work result in a reduction of the water-level gradient.



SONDEO  
SOUNDING

**RÍO MAGDALENA** km 711-km 706

AGUAS ABAJO DEL  
DOWNSTREAM OF

**RÍO REGLA**

ESCALA/SCALE 1:10,000

FECHA/DATE 11, 12-VII-1972

NIVEL DE REDUCCIÓN 0.60 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. BERRÍO

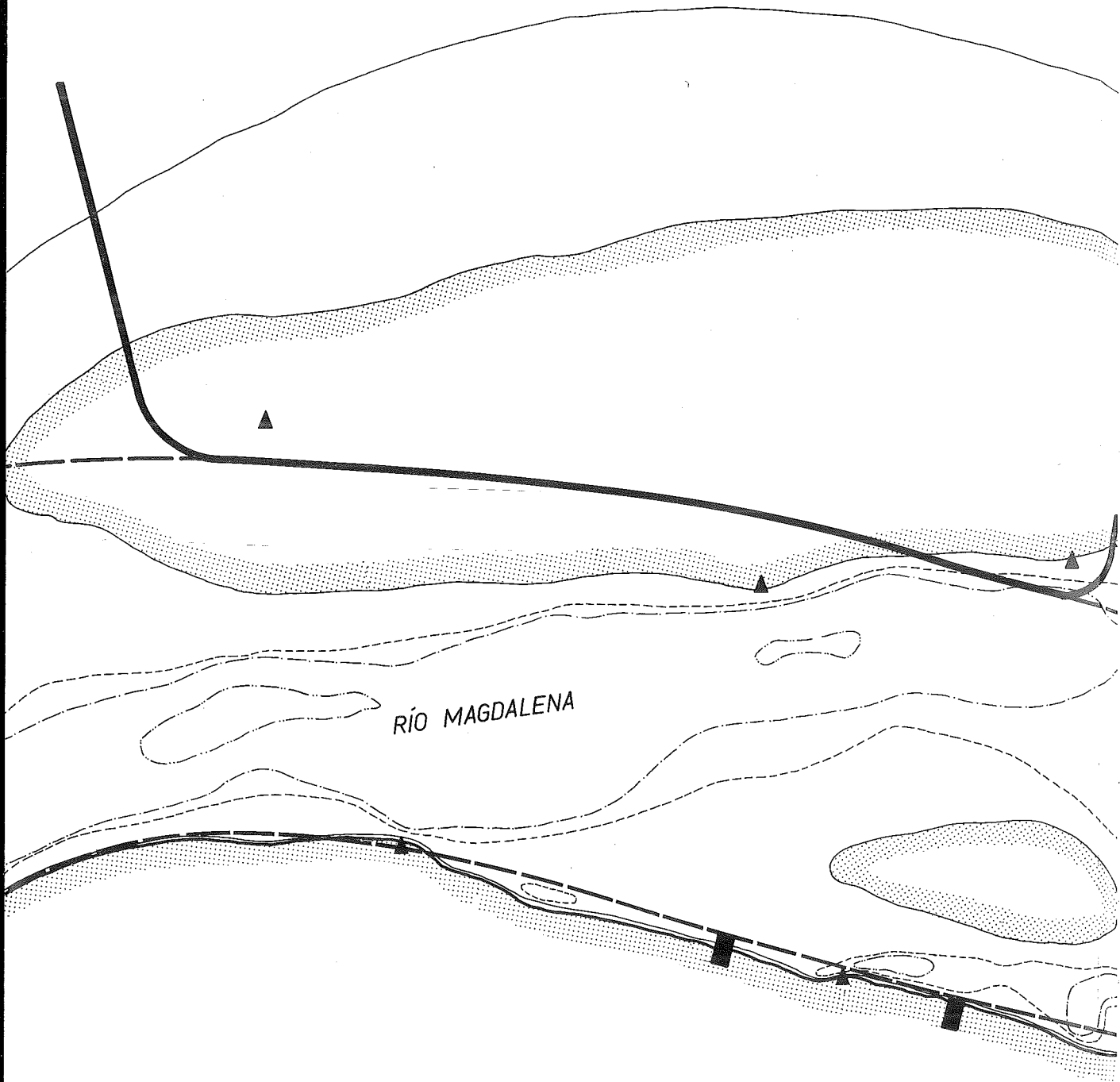
CHART DATUM: 0.60 metres ABOVE ZERO OF THE PTO. BERRÍO GAUGE

CURVAS ISOBATAS  
DEPTH CONTOURS

— 0 m  
- - - 1.5 m  
- · - · 2.5 m  
- · - · - 5 m



PLAYA, SOBRE EL DATUM  
DRY, ABOVE DATUM



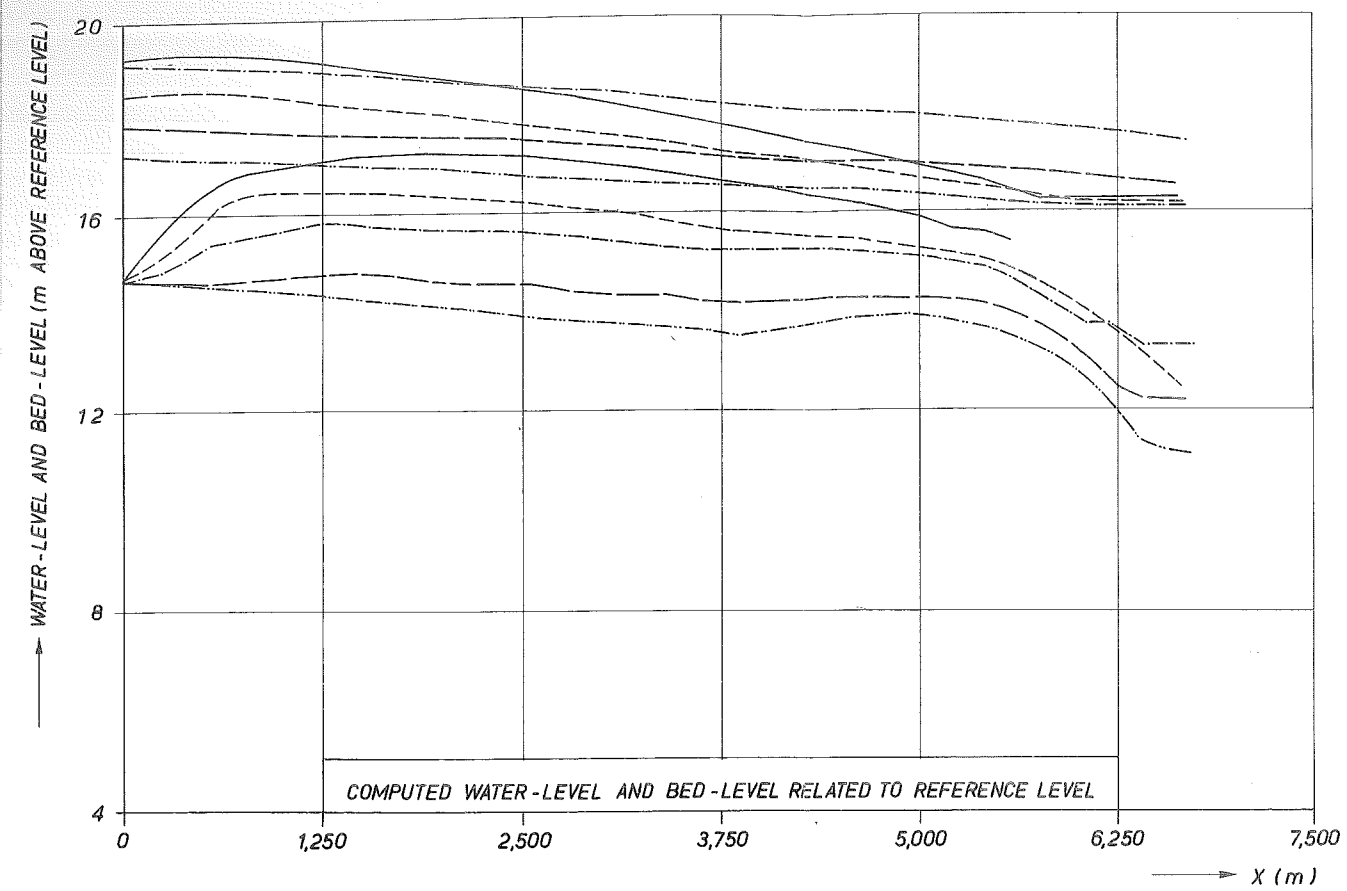
RÍO MAGDALENA

SC  
SO

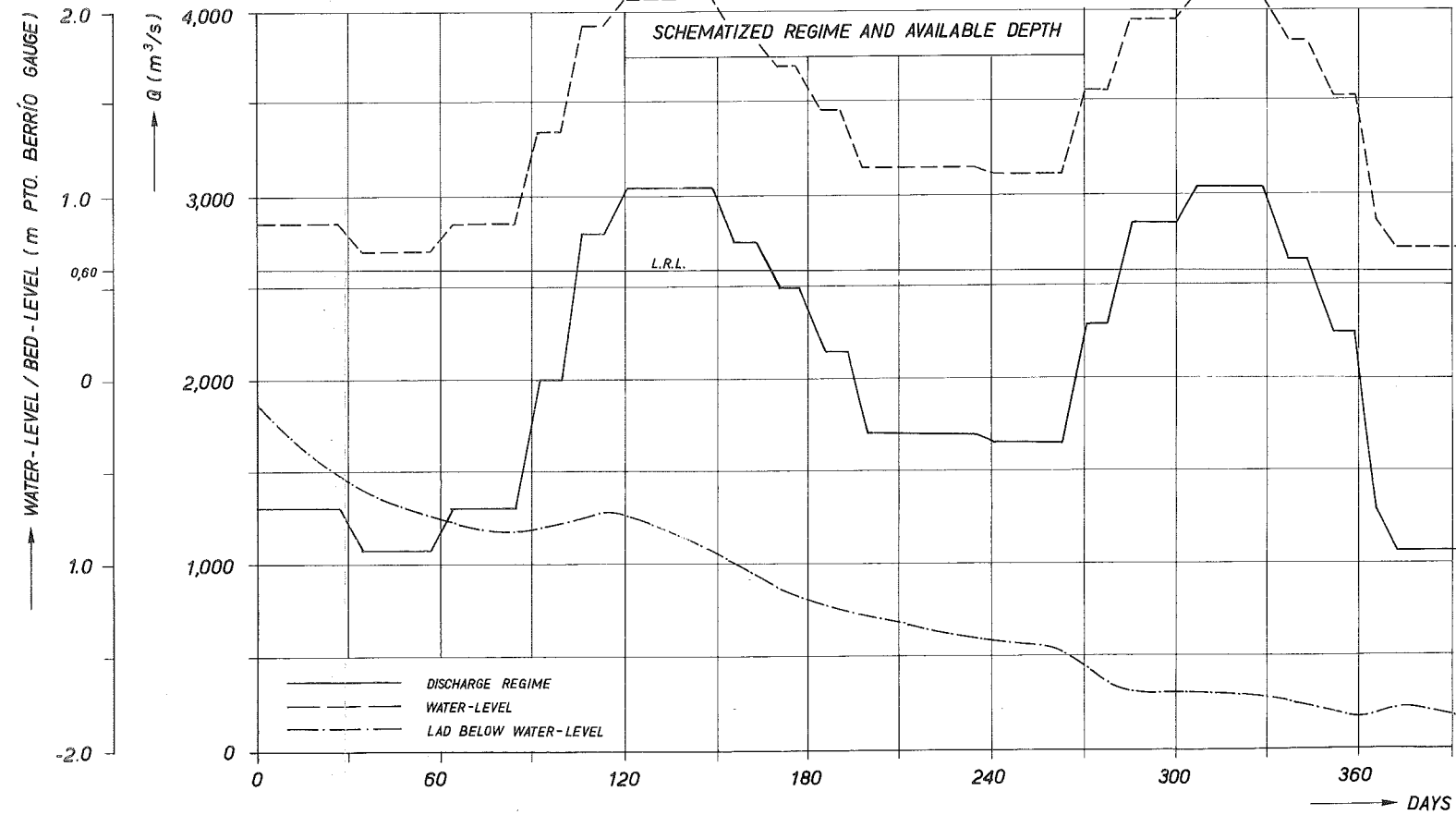
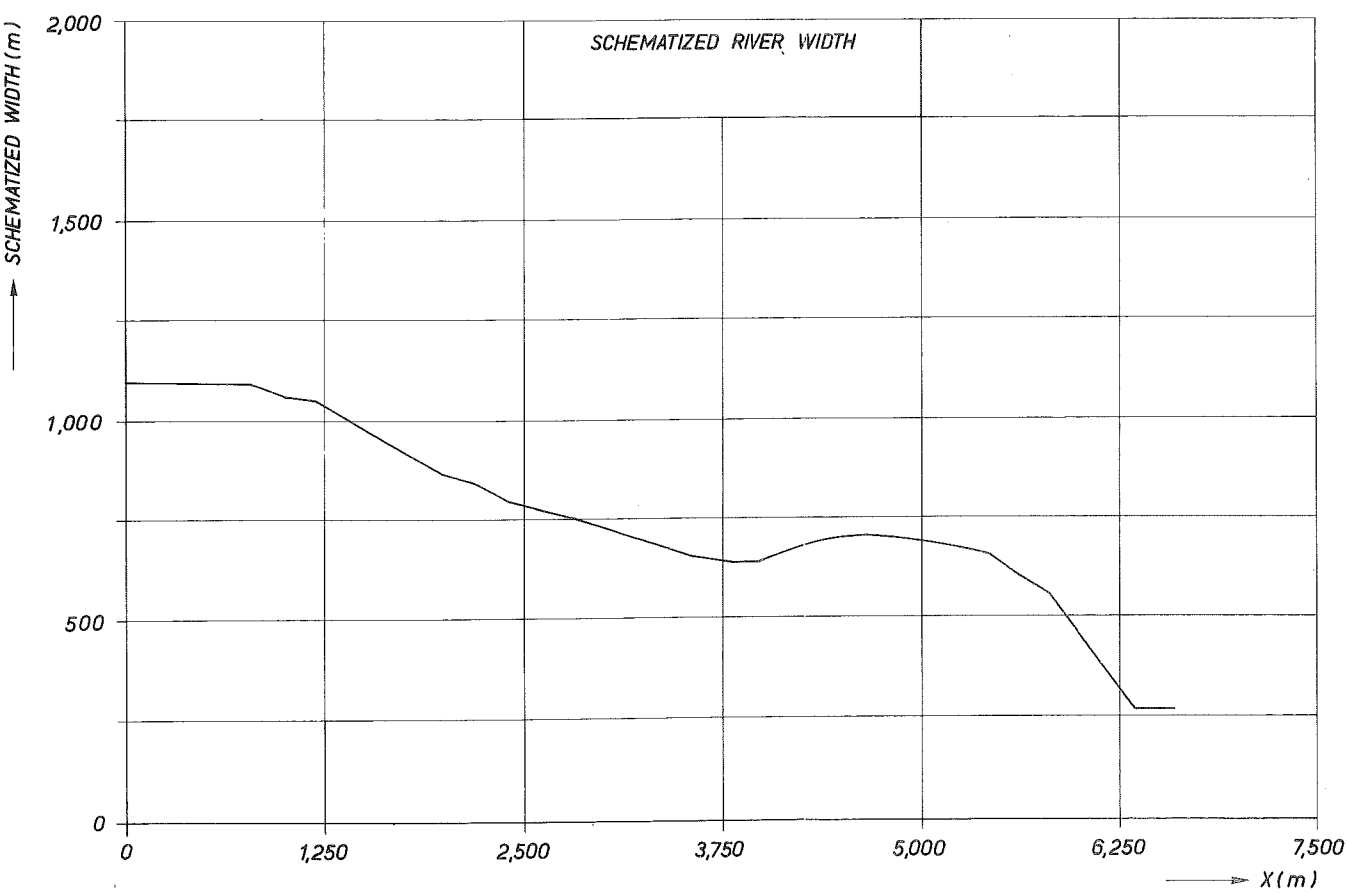
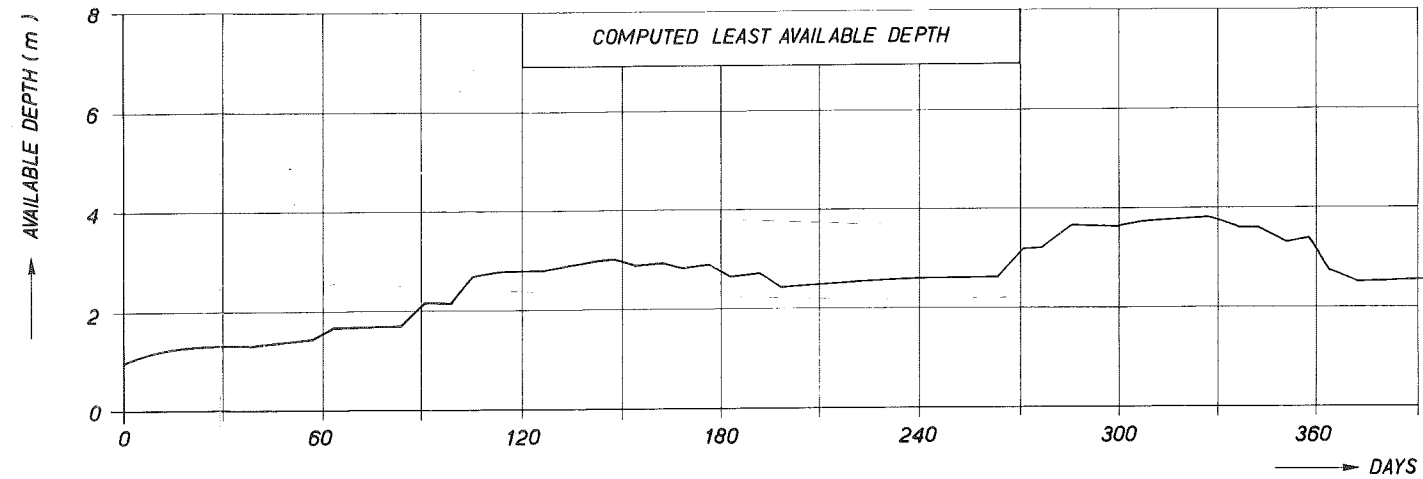
RÍO REGLA

BALLENA





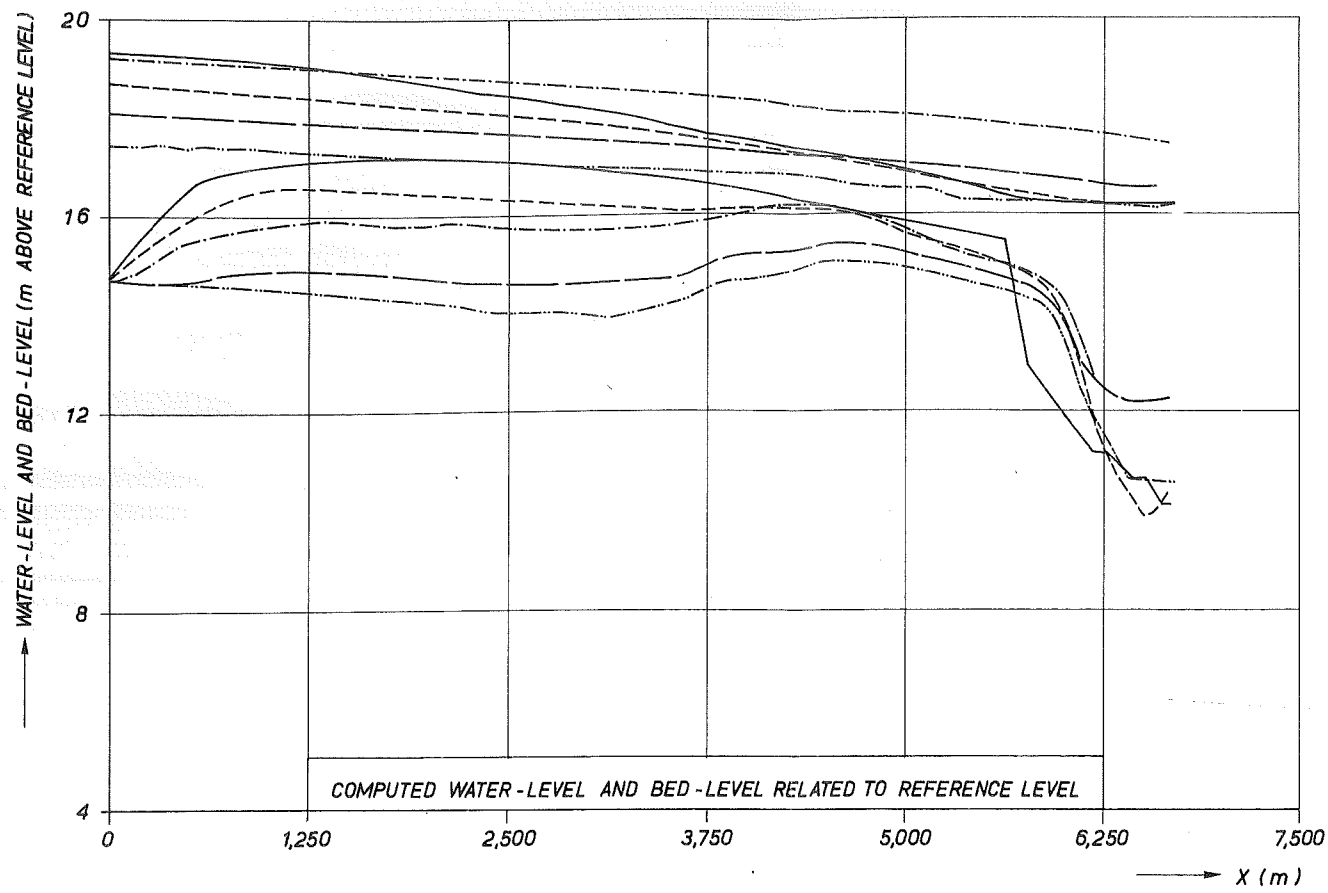
WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—————	0	0	Q = 1,300
-----	51	0.5	Q = 1,070
-----	120	4	Q = 3,050
-----	263	3	Q = 1,650
-----	402	2	Q = 1,070



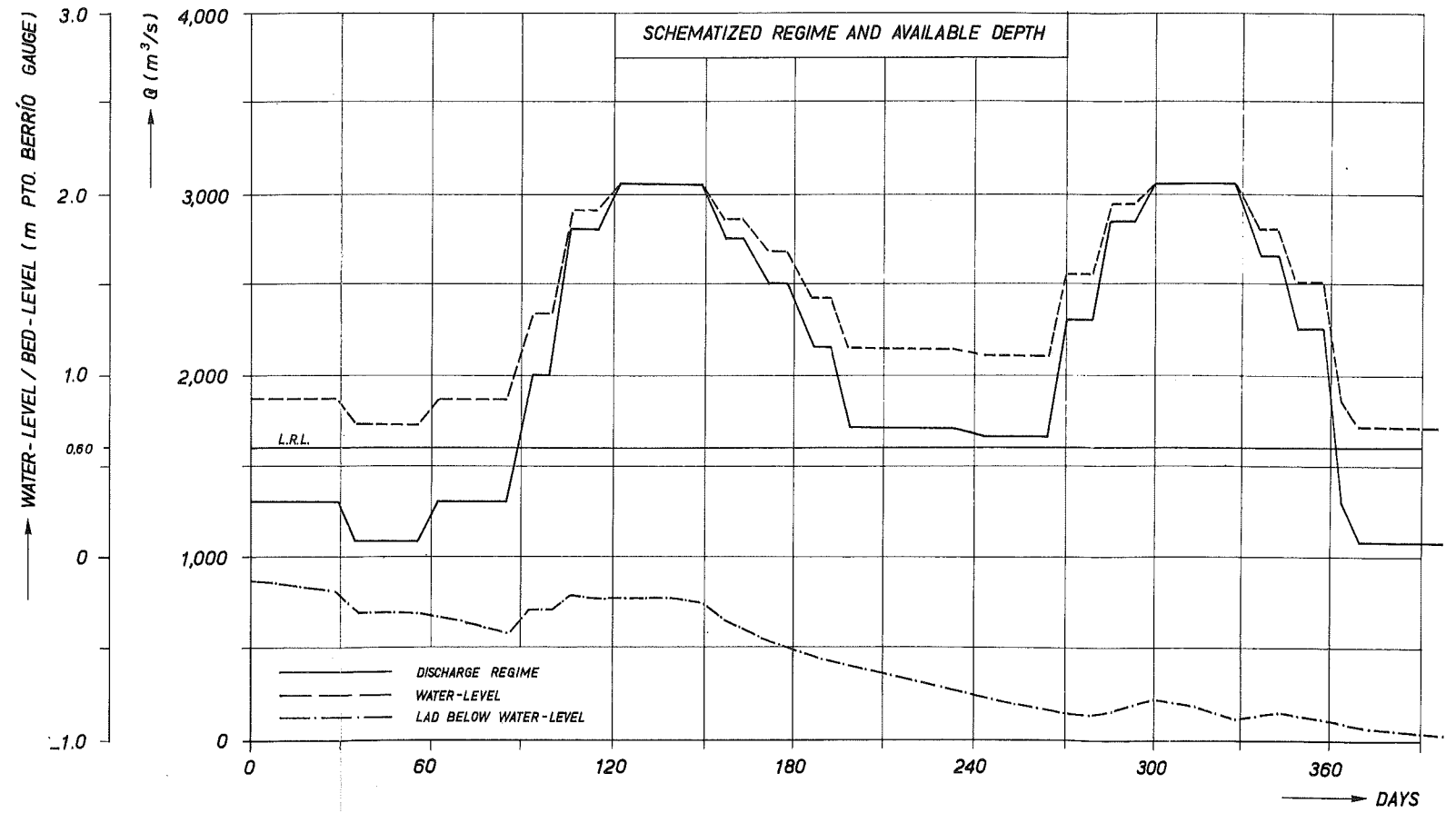
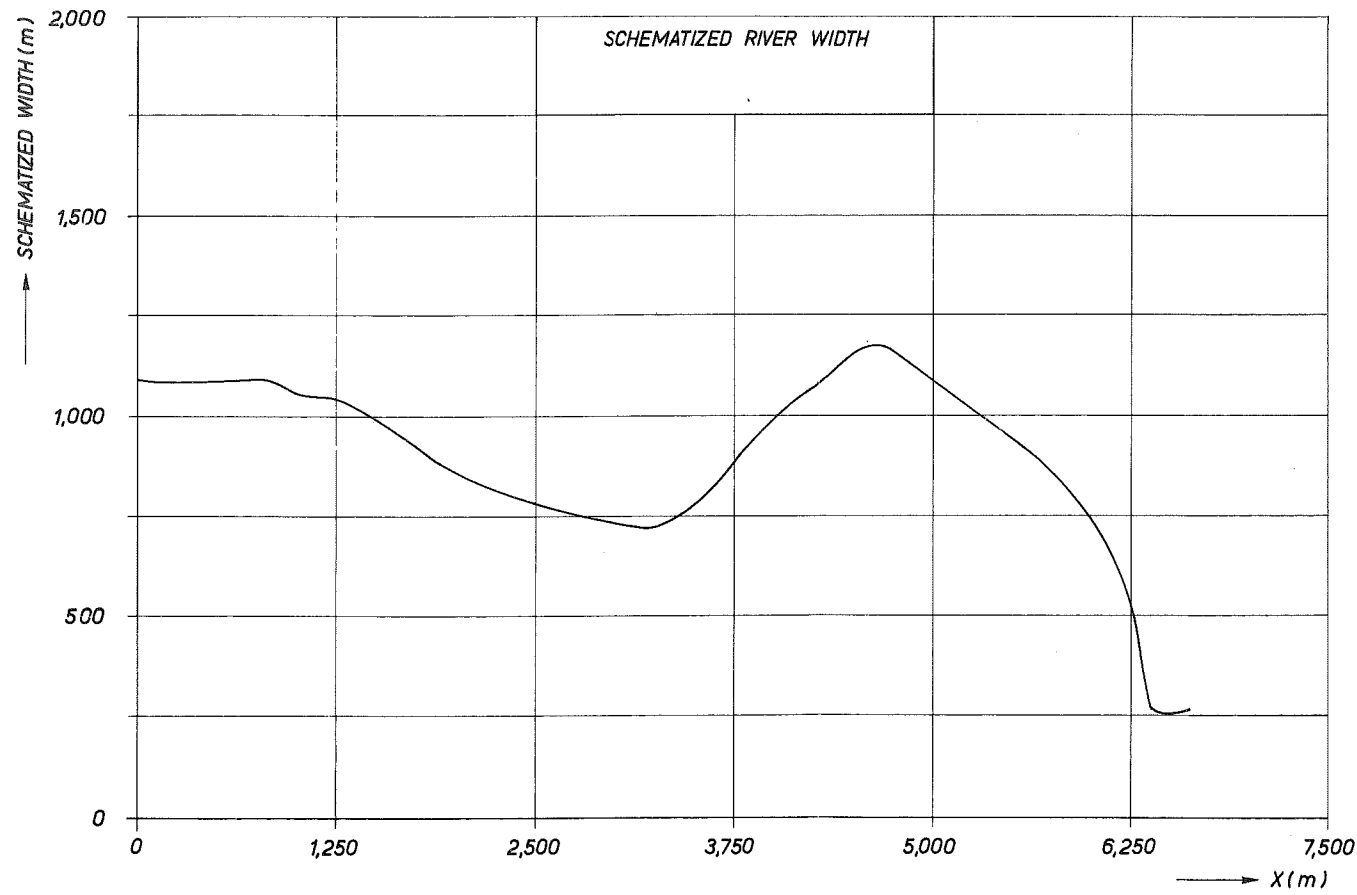
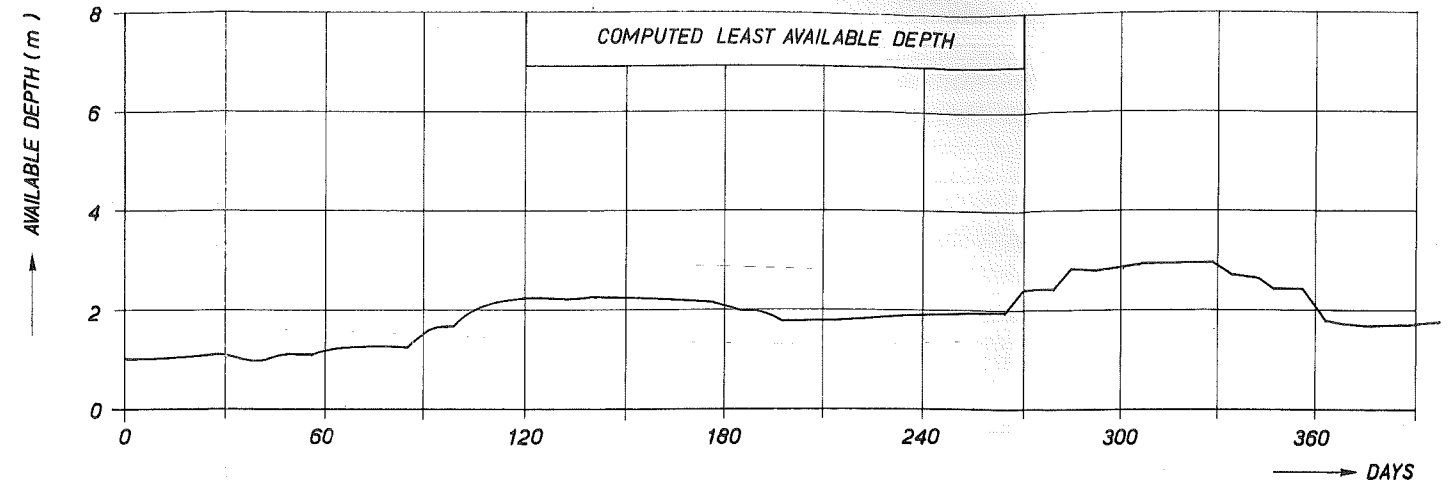
RÍO REGLA CONFLUENCE, AFTER IMPROVEMENT AFTER EXECUTION OF RIVER-WORKS

RESULTS OF MORPHOLOGICAL COMPUTATIONS

FIG. 3.4.11



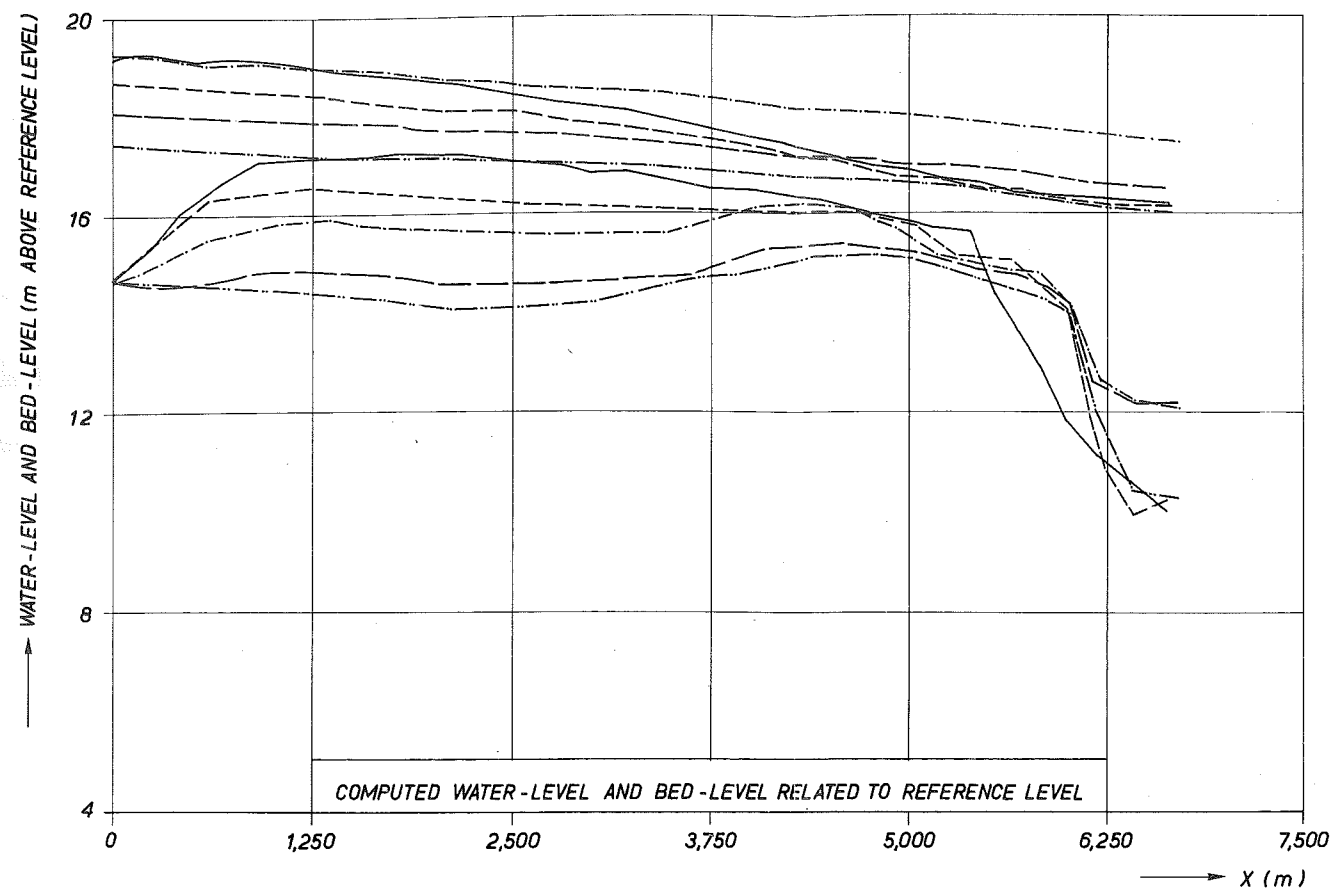
WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—	0	0	Q = 1,300
—	64	2	Q = 1,300
—	121	8	Q = 3,050
—	264	17.5	Q = 1,650
—	398	22.5	Q = 1,070



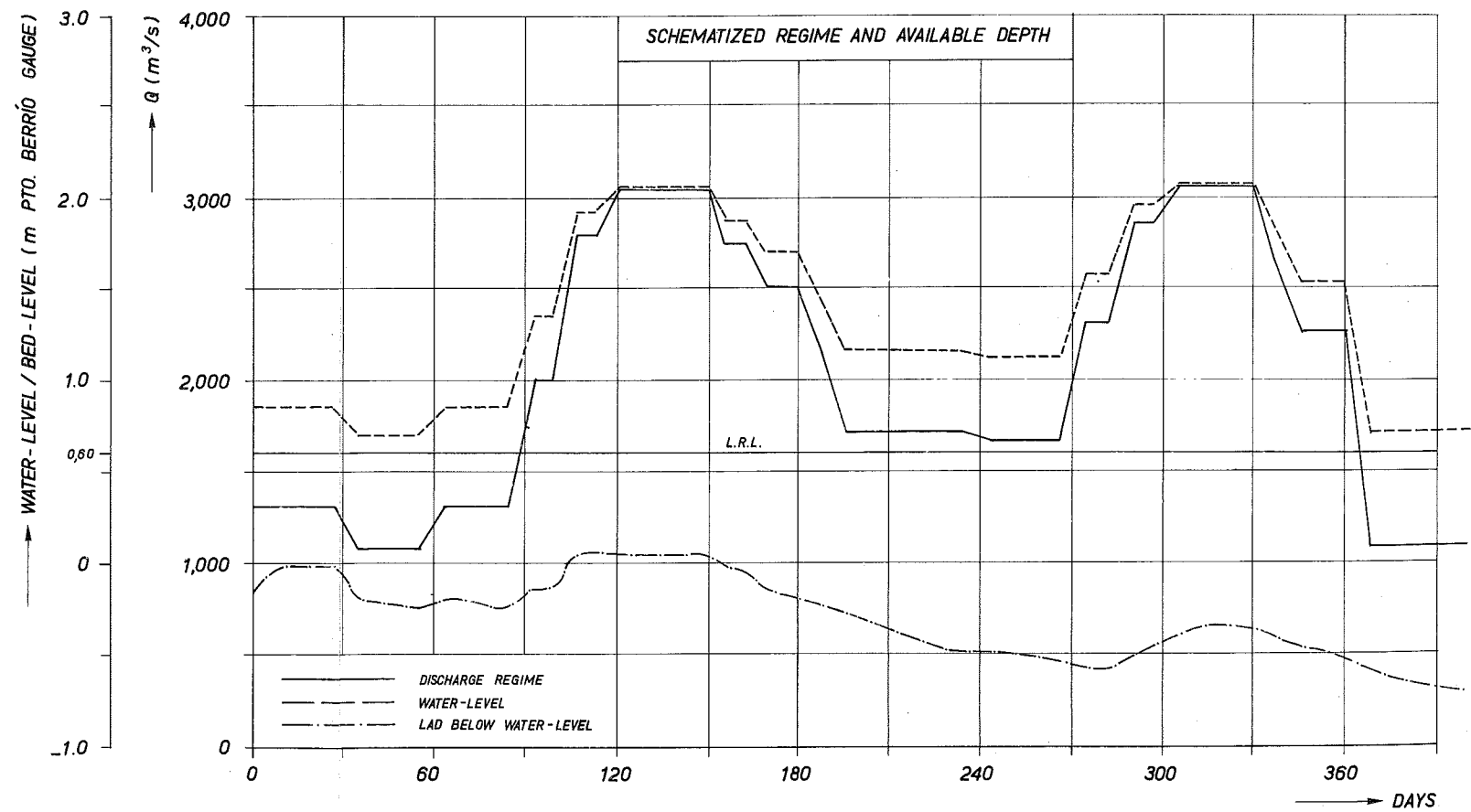
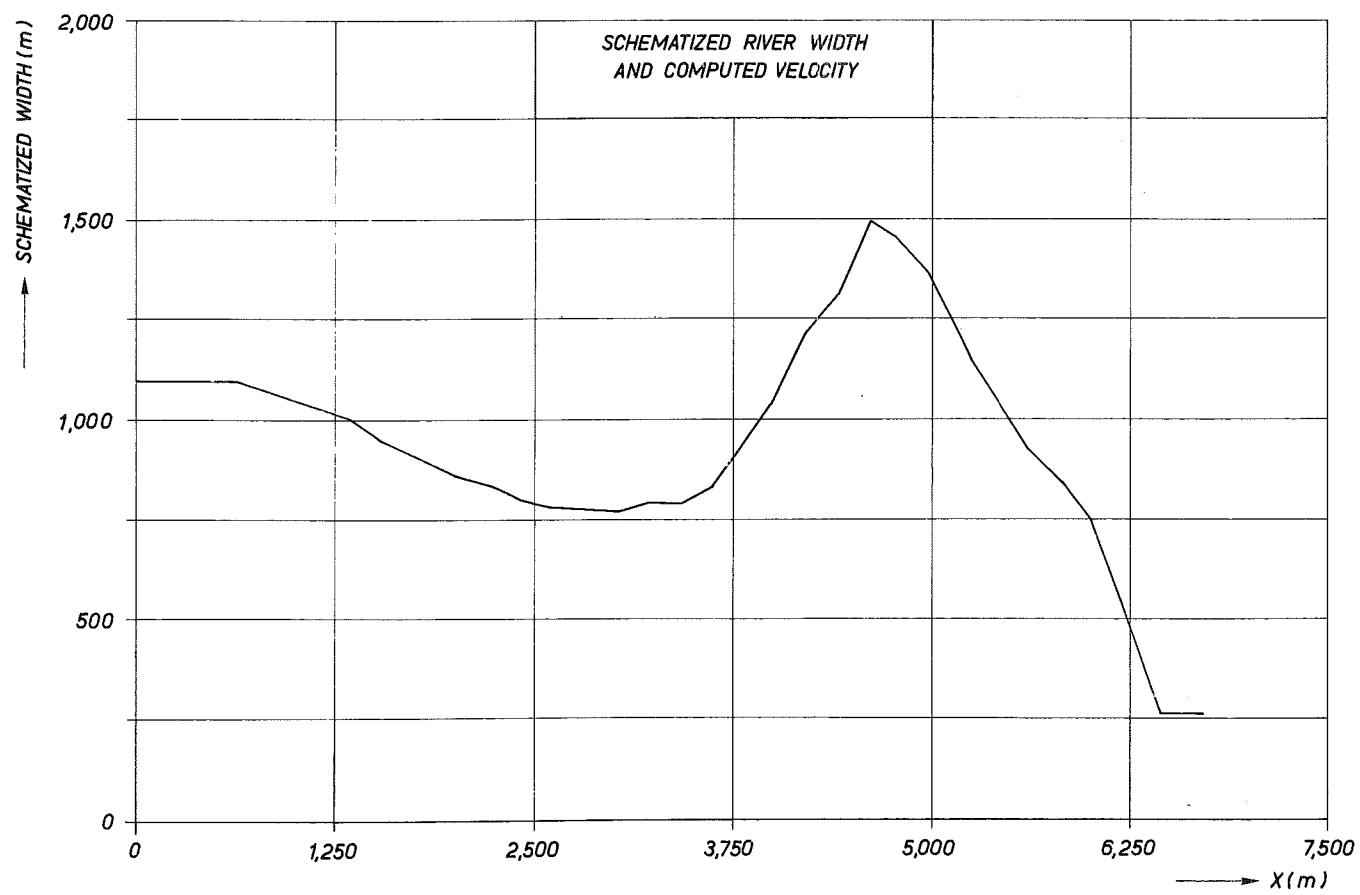
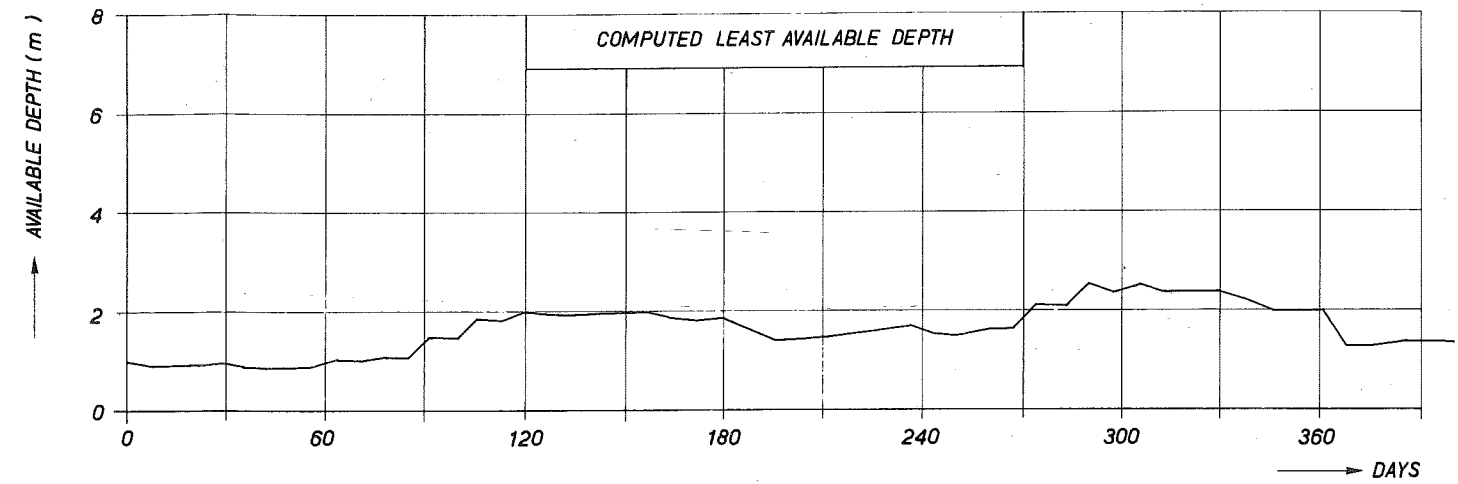
RÍO REGLA CONFLUENCE, SITUATION 1971

RESULTS OF MORPHOLOGICAL COMPUTATIONS

FIG. 3.4.12



WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—	0	0	Q = 1,300
- - -	63	22	Q = 1,300
— · —	120	17	Q = 3,050
— · —	166	2	Q = 1,650
— · —	400	35	Q = 1,070



RÍO REGLA CONFLUENCE, SITUATION 1924

RESULTS OF MORPHOLOGICAL COMPUTATIONS

FIG. 3.4.13

The construction

Bank protections will have to be carried out under differing circumstances: at places which are heavily attacked during the construction and places which lie more protected. The use of two different types of construction is therefore logical, these are indicated in the Figures 3.4.14 and 3.4.16. Their locations have been indicated in Figure 3.4.10, as section I - I and section II - II respectively.

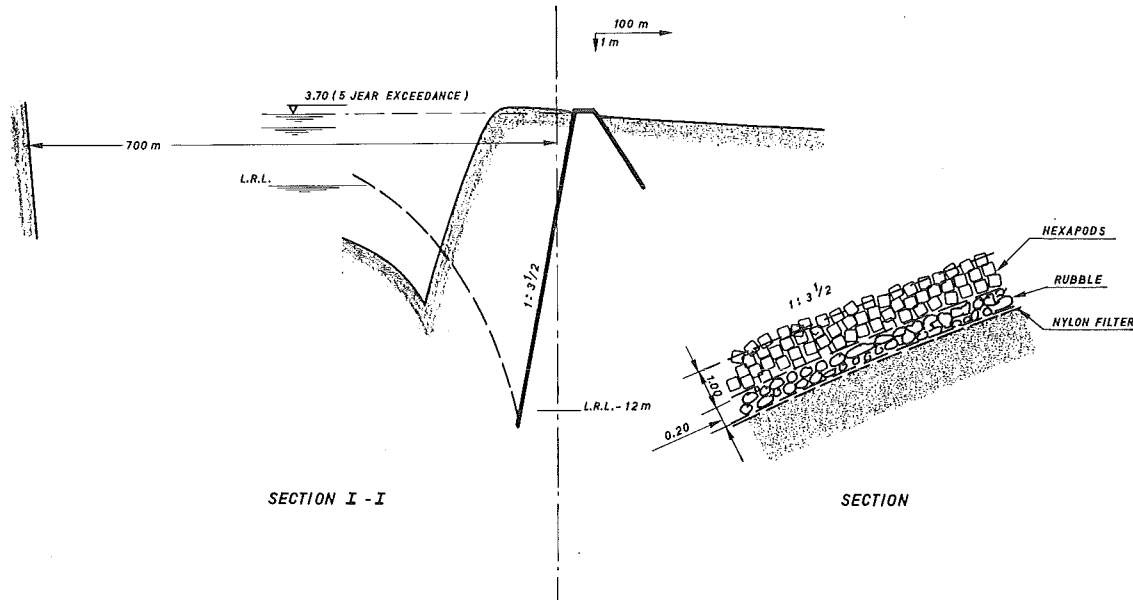


Figure 3.4.14 Cross-section of Groyne (Section I-I)

Section I - I requires a trench to be dug. This construction consists of a nylon filter weighed down by concrete blocks or rock, for which it has been assumed that hexapods can be used. Between the hexapods and the nylon filter, a layer of rubble is indicated as protection against mechanical damage when the hexapods are dumped (see Figure 3.4.14). According to Figure 2.5.7, concrete blocks should have a diameter of at least 0.28 m (for  $v = 2.5$  m/s and  $\Delta = 1.4$ ), to be slightly increased in view of the side-slope (see Figure 2.5.9). Hexapods (with a diameter of 0.90 m) will therefore have sufficient weight. For hexapods a steeper side-slope than the indicated  $3\frac{1}{2} : 1$  would be permitted, as far as their stability is concerned but, during the execution it would be difficult to maintain the unprotected side-slope at a steeper angle.

The sinking of the nylon filter will not present too many difficulties, if the filter contains cells filled with sand, stabilized with S.R.O. (Standard Road Oil; see Figure 3.4.15). The nylon filter proved its applicability along the Rfo Magdalena, as similar constructions were used about 20 years ago near Pto. Wilches, which are still in a reasonable good condition. According to the Julius Berger Survey, stone to be used for rubble can be found in the Rfo Nare.



Figure 3.4.15 Nylon Filter

### III, 3.4

The crest-level of the construction lies at 3.70 m above L.R.L., a level exceeded on the average once every five years. Although some damage of the construction may be expected during overflow of some duration, this damage will not be excessive and can easily be repaired. The toe of the protection is designed at a level of 12 m below L.R.L. This will, however, not be necessary along the complete length. A model study could give more information about this.

Excavation of the trench could be avoided by making only the higher part of the construction given in Figure 3.4.14, and waiting with the construction of the toe until the river attacks the construction and thus removes the fore-shore. But in such a case, once the fore-shore will be eroded, the protection must be carried out rapidly to avoid losing the complete work. This is a more risky solution, but would save about 10 million pesos in dredging work.

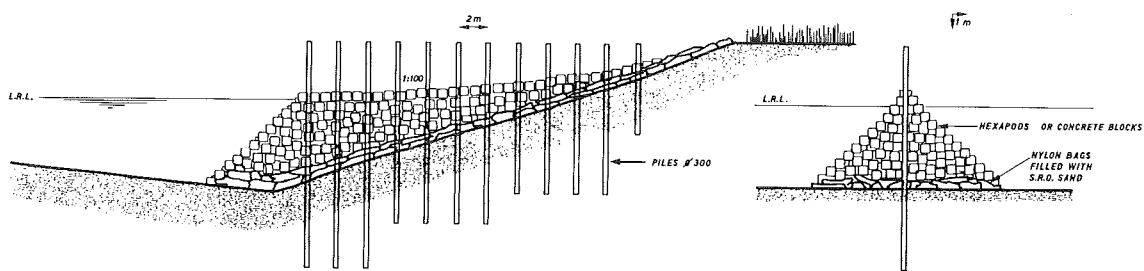


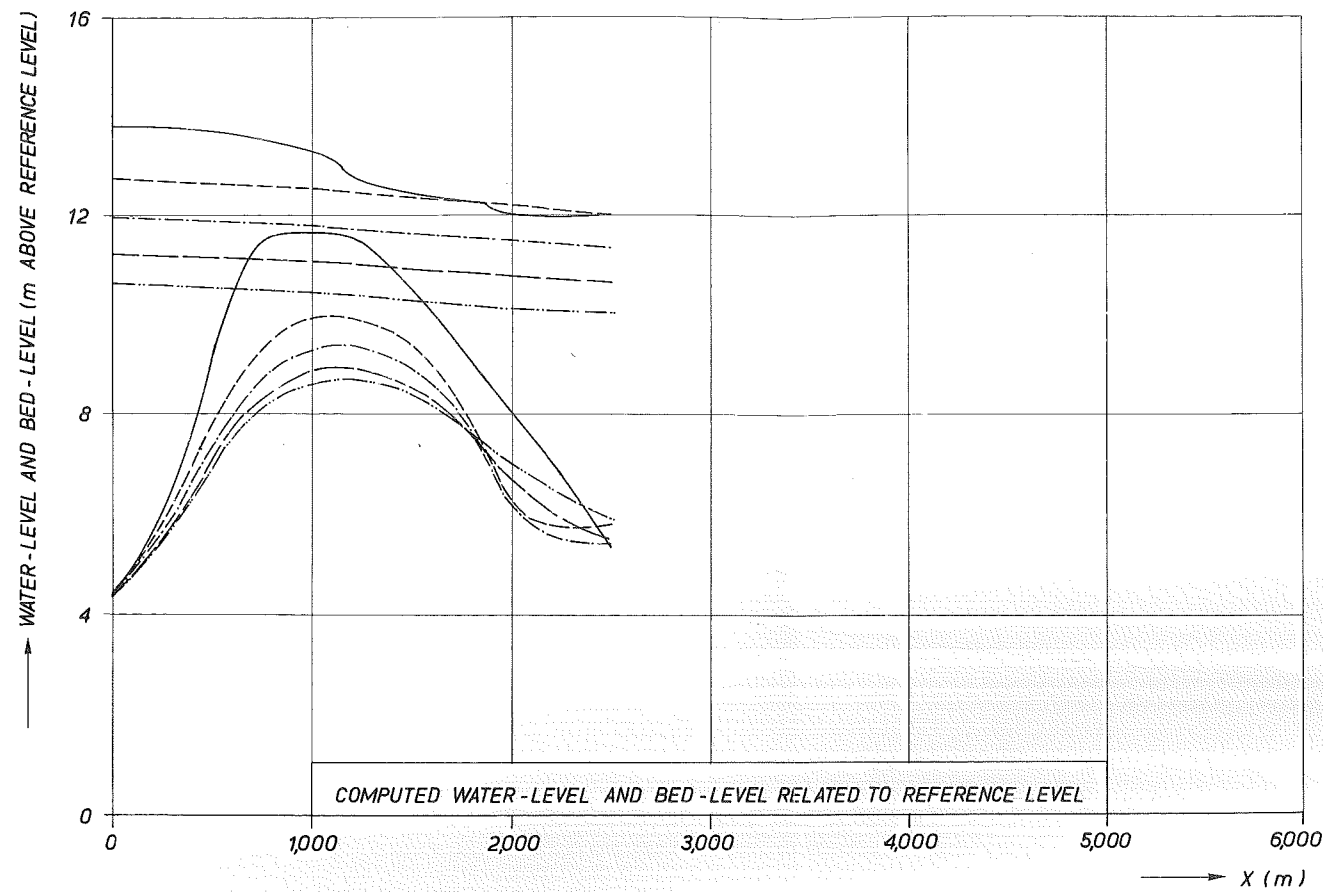
Figure 3.4.16 Cross-section of Groyne (Section 11-11)

The construction of the groynes given in Figure 3.4.16 have a filter base, consisting of two layers of large nylon sandbags, filled with S.R.O. sand. These bags can be positioned by a crane. The piles which are indicated, serve as guides during the positioning of the sandbags and the hexapods which are to be dumped on top of the bags (as an alternative, the construction given in Figure 3.2.18 can be considered).

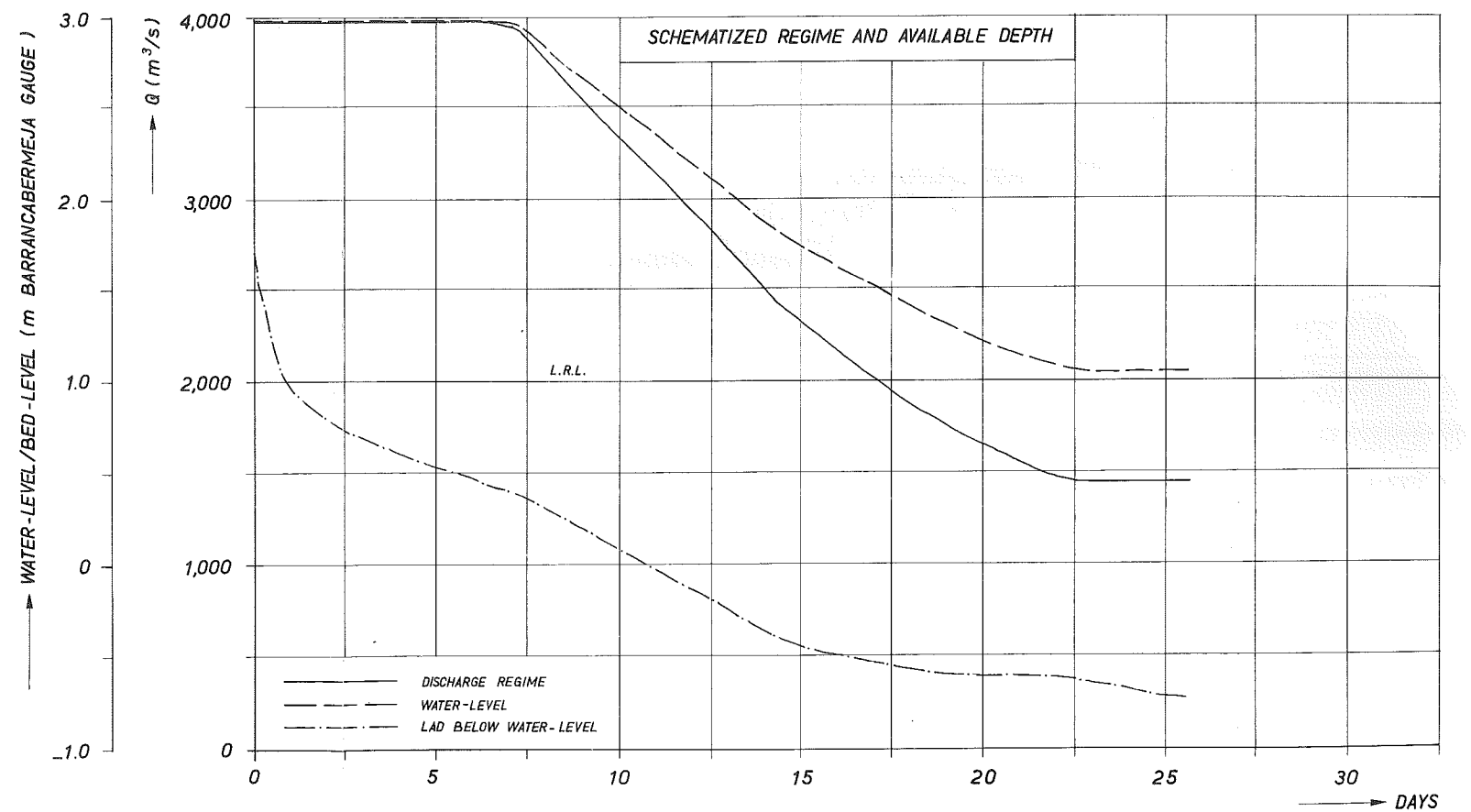
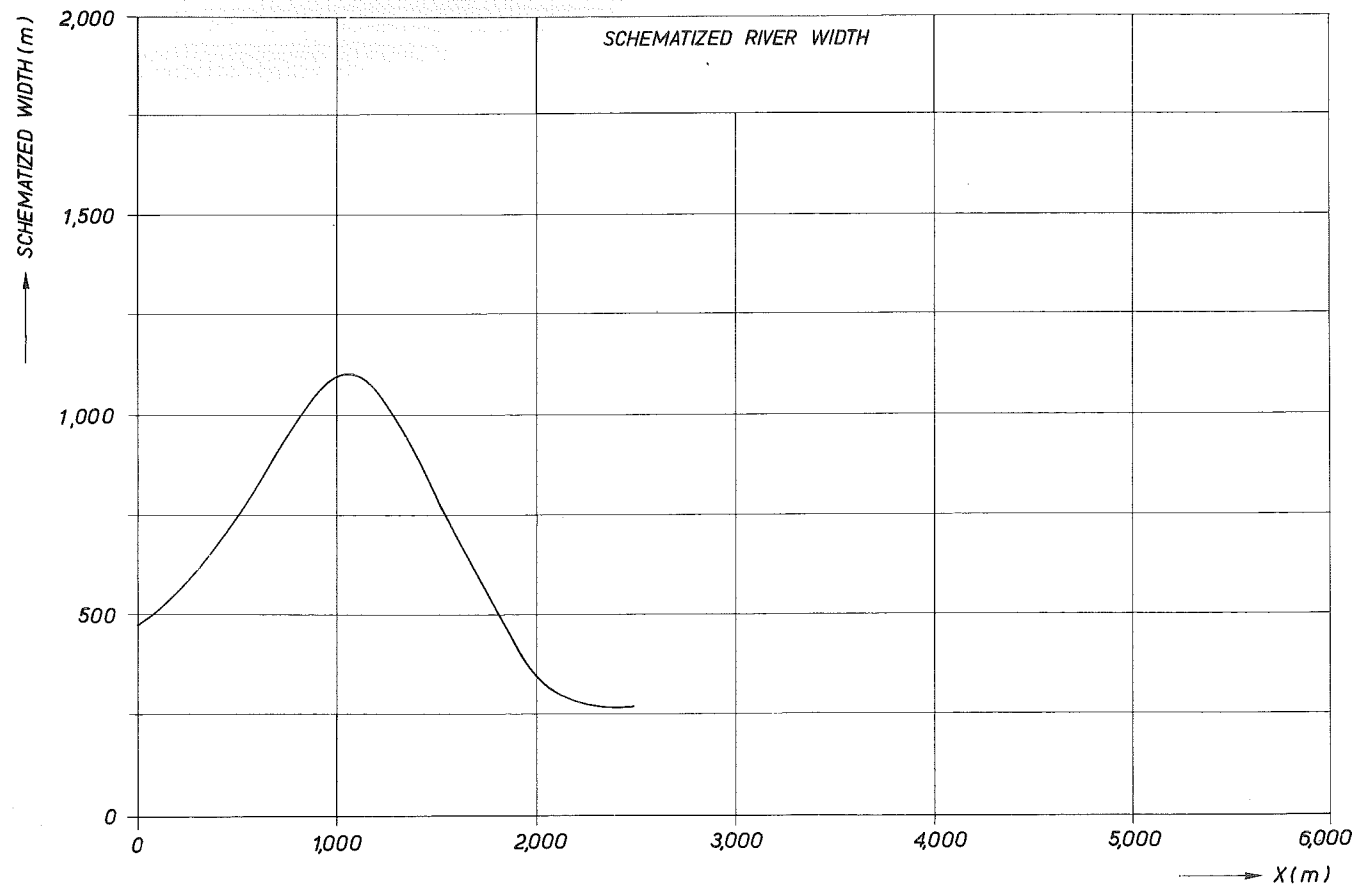
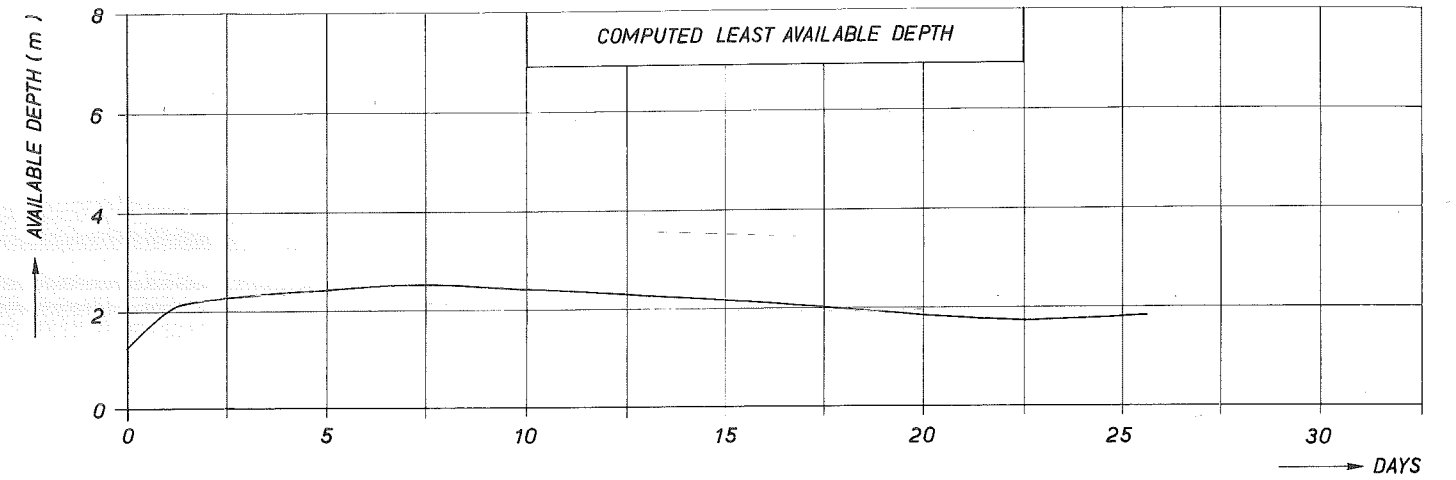
The constructions given here could be carried out by a Colombian contractor, but some advice from a contractor with experience in this type of river-works seems required during the execution.

#### Phasing of the river-works

The extent of the river-works indicated, will make it (probably) impossible to finish the complete project in one low water season. The execution of the river-works should therefore be phased in such a way, that a high water does not destroy the already completed part of the work. The execution should start in the first year with the works at the upstream end along the left bank, thus forcing the main current already somewhat to the right bank, and some of the minor secondary branches should be closed. During the second year, the long revetment on the sand-bank along the left-hand side of the future navigation channel can be made. The groynes along the right bank and the works just upstream of the present narrows, can then be completed in a third year. Of course, the phasing of these works should be flexible, so that it is possible to adapt the sequence of execution as well as the actual alignment to changing conditions, when part of the project is already completed.



WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE ( $m^3/s$ )
	DAYS	HOURS	
—————	0	0	$Q=3975$
-----	6	3.5	$Q=3975$
- - - - -	11	16.5	$Q=2976$
—————	17	0	$Q=2011$
-----	22	17.5	$Q=1441$



RÍO MAGDALENA CROSSING NEAR km 667

RESULTS OF MORPHOLOGICAL COMPUTATIONS

FIG. 3.4.17

Costs

A detailed cost analysis has not been made, as this can only be done after the construction has been worked out more fully. Moreover, costs change rapidly over the years. A rough estimate based on 1972 prices can be taken as Col. \$ 40 million (or \$ 30 million if the trench is not made). For a break-down of the cost, the reader is referred to Para. 3.2.3. Without river works, local maintenance dredging twice a year will amount up to about Col. \$ 500,000 - \$ 1,000,000 annually.

3.4.5. The Río Magdalena between the Río Carare Confluence and Chucurí (km 675 - km 659)

The Río Magdalena between the Río Carare Confluence and Chucurí has changed rather drastically during the years, as far as the navigation circumstances are concerned. These changes were so pronounced that during the reconnaissance surveys in 1970 and 1971, it was impossible to recognize any resemblance of the river to the aerial photographs of 1954, while these photographs differed again greatly from the charts as given by Julius Berger.

Between km 666 and km 659, at present a long and rather narrow island can be seen. On the left-hand side of this island a narrow but good navigation channel is available, which shows a tendency to meander, while on the right-hand side of the island the river is braided and offers generally bad navigation conditions. From photographs of 1954 it appears that this narrow island is the remnant of a group of large islands, which were partly incorporated into the left bank. At that time the navigation followed the right bank where a good navigation channel existed. In 1923, the navigation channel followed the left bank, but navigation conditions must have been rather bad.

For the crossing at km 667 a computation has been carried out to determine the retarded scour, and the result is shown in Figure 3.4.17. The computed scour of 1.20 m appears to be very large, possibly as a result of a less successful schematization. The measurements on this crossing, however, also show a rapid and large movement of the river-bed (see Figure 3.4.18).

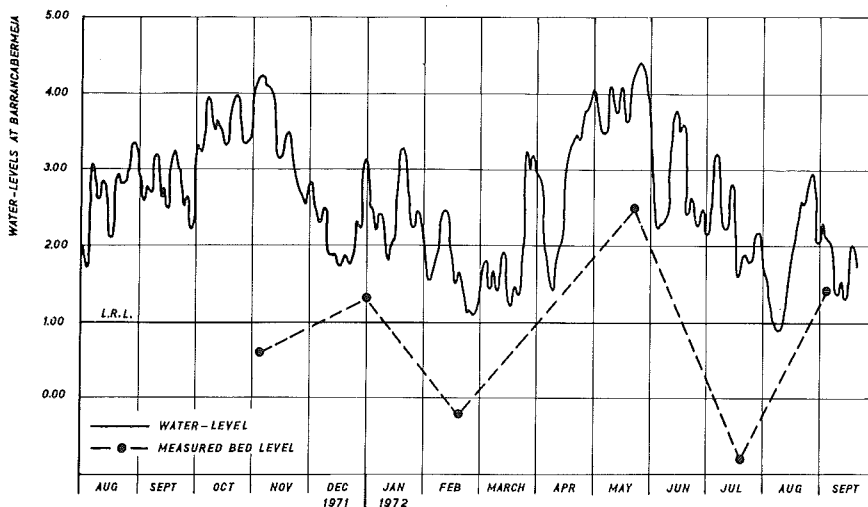


Figure 3.4.18 Recorded LAD on Crossing near km 667

### III, 3.4

Although at present, the section between km 675 and km 659 is rather good, potentially it is a bottle-neck for navigation, due to the rapid changes which do not give channels time to develop sufficiently. Improvement by recurrent dredging may often fail, due to the great part of the current that may be drawn to other channels than the dredged one. The best solution, therefore, seems to be, to try and stabilize the situation by means of temporary or permanent river-works.

The narrow and deep navigation channel on one side of a long island, and a braided shallow channel on the other side, is a rather typical example of what is encountered frequently, and not only on the Río Magdalena. Generally, a navigation problem exists at the crossing upstream of the island to the entrance of the channel. However, sometimes the crossing may be rather good, while in other years the entrance of the navigation channel is blocked by sandbanks propagating downstream (see Figure 3.4.19).

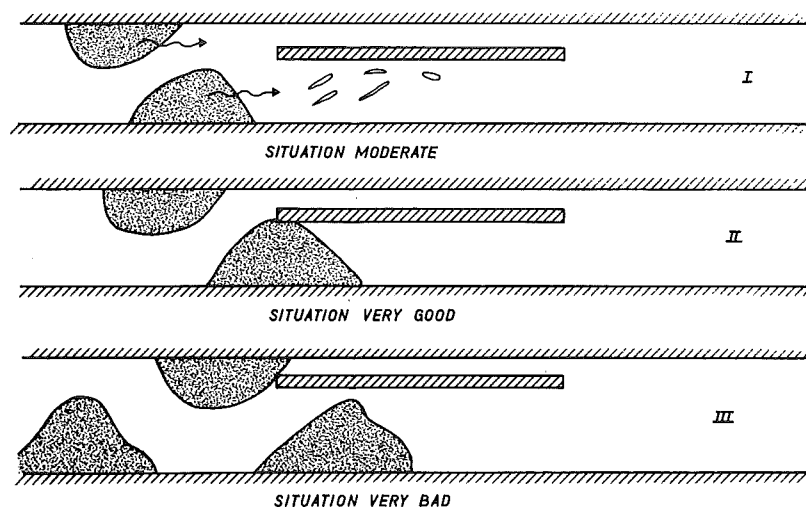


Figure 3.4.19 Entrance of Navigation Channel Alternatively Blocked and Left Open by Propagating Sand-banks

Three types of solution can be used for this problem:

- a. The braided channel can be closed off completely. However, this only gives a temporary improvement if no extensive (permanent) river-works are carried out to fix the complete configuration, because otherwise the narrow channel will widen gradually and become braided again. Such a solution is, therefore, costly.
- b. The development of the situation as indicated in Figure 3.4.19 by III can be awaited, to make a dredge cut through the island. This solution will only be worthwhile if the island is indeed narrow, because in a subsequent season this opening possibly may have to be closed again (which may be difficult), and another one opened.
- c. By means of temporary or permanent river-works a configuration can be enforced, whereby no sandbank can develop at the entrance of the narrow channel.

Solution c is in the long run probably the cheapest and safest solution, while for a short-term improvement solution b may offer good conditions. Solution c requires some further clarification. For example, river-works may be carried out in agreement with the alignment given in Figure 3.4.20. If these river-works are extended too far, the solution is very similar to solution a and extensive river-works may be required to safeguard the configuration and the original structures. For the same reason, care should be taken that the wide braided channel on the right of the island maintains a sufficiently large capacity, particularly during higher levels. This may mean that in practice (to be on the safe side) a somewhat greater capacity of the right-hand channel will be maintained (at the cost of the discharge of the left-hand channel) and that some dredging may be required in the navigation channel.

A solution by means of bottom panels may also be mentioned here (Figure 3.4.21).

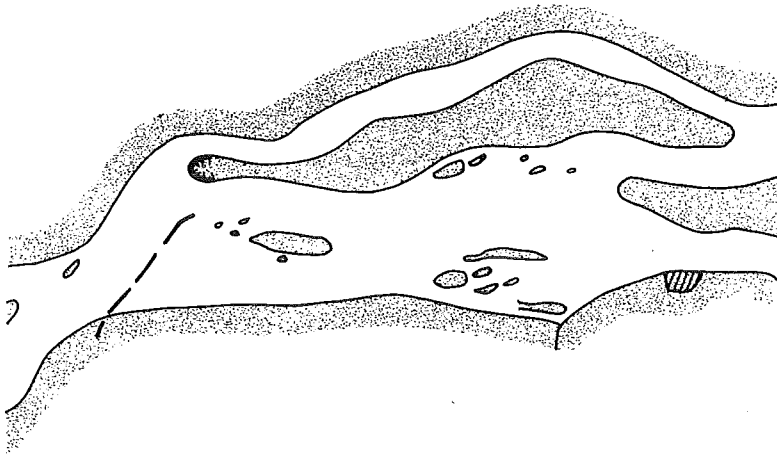


Figure 3.4.20 Possible Alignment of River-works

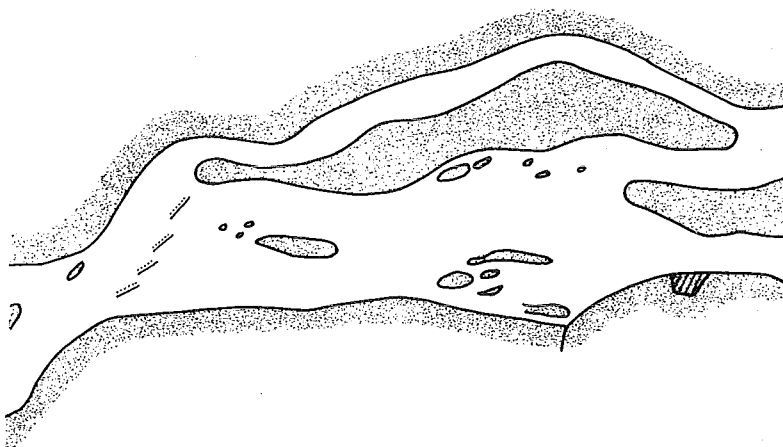


Figure 3.4.21 Possible Solution with Bottom Panels

3.5. BARRANCABERMEJA - GAMARRA (KM 630 - 475)

3.5.1. General description and design criteria

Available cross-section

As previously stated, the division of the river between La Dorada and Gamarra into the sections La Dorada - Pto. Inmarco, Pto. Inmarco - Pto. Berrfo, Pto. Berrfo - Barrancabermeja and Barrancabermeja - Gamarra is based more on the division of the present traffic flow than on the river morphology. In the section Barrancabermeja - Gamarra, the river changes in character near the confluence of the Rfo Magdalena and the Rfo Sogamoso, and near the bifurcations with the Brazo Simitf and the Brazo Morales.

Three different sections must, therefore, be considered:

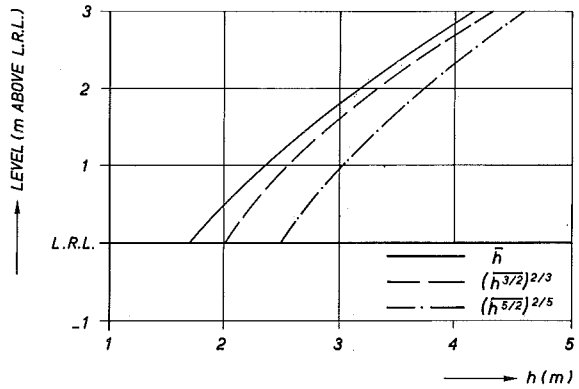
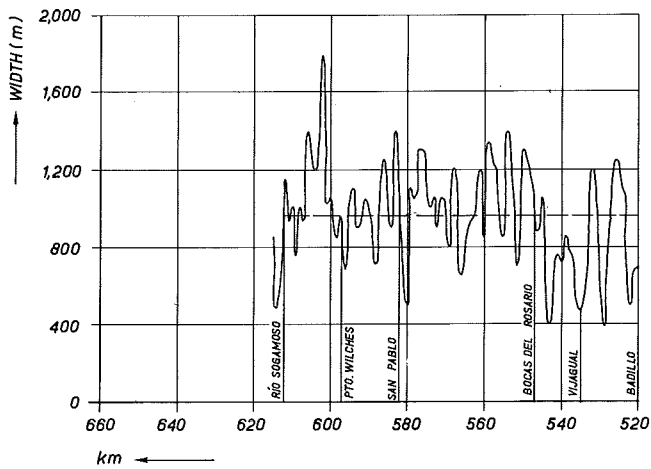
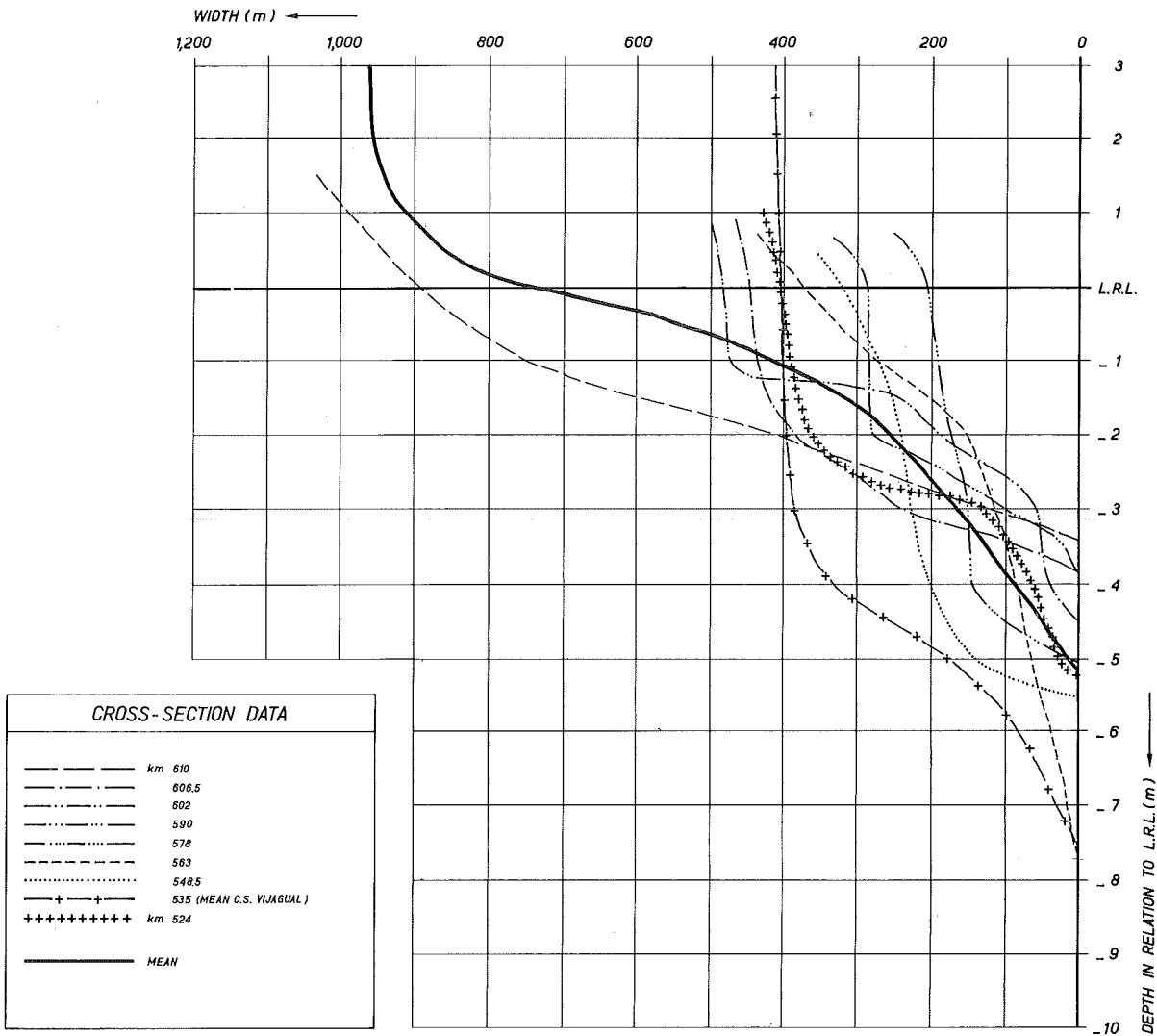
- The section upstream of the influence of the Rfo Sogamoso, where the cross-section is similar to that between the Rfo Carare Confluence and Barrancabermeja. This cross-section was already given in Para. 3.4, Figure 3.4.1.
- The section between the Rfo Sogamoso Confluence and the bifurcation with the Brazo Simitf near Badillo (Figure 3.5.1).
- The section downstream of Badillo (actually below the bifurcation with the Brazo Morales), which continues downstream of Gamarra, until the Brazo Morales again joins the Rfo Magdalena (Figure 3.5.2). About this section, information is less specific because it was studied in less detail; particularly is the division of the discharge over the different branches not known. It seems advisable to study this division in more detail at some future time.

Although there is a change in morphological conditions at the Rfo Sogamoso Confluence (which already starts near Barrancabermeja) this does not express itself very much in the schematized cross-sections (Figures 3.4.1. and 3.5.1), because the width and depth in both sections are about the same. The section downstream of Badillo is, however, much smaller; the average width being 600 m against 950 m upstream, while the average depth at L.R.L. is about 2.20 m against 1.80 m upstream of Badillo. It appears, in fact, that below Badillo the river offers better conditions for navigation, although the discharge is divided over two channels.

Downstream of Badillo the river changes from braided to meandering. As was already pointed out in Chapter 3.7 of Part II (Figure 3.7.4), a reduction of the discharge may cause, with the same water-level gradient, a transition from braiding to meandering of the river. (Also some reduction in the water-level gradient may, together with the discharge reduction, cause such a transition).

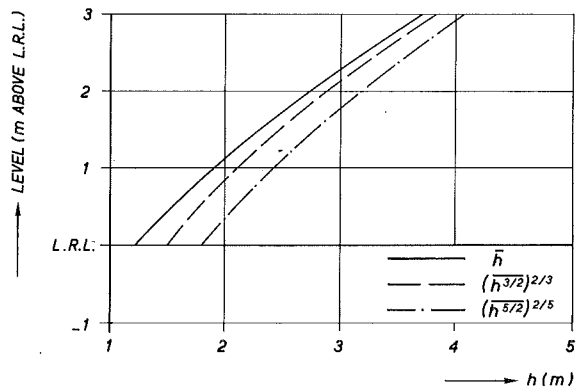
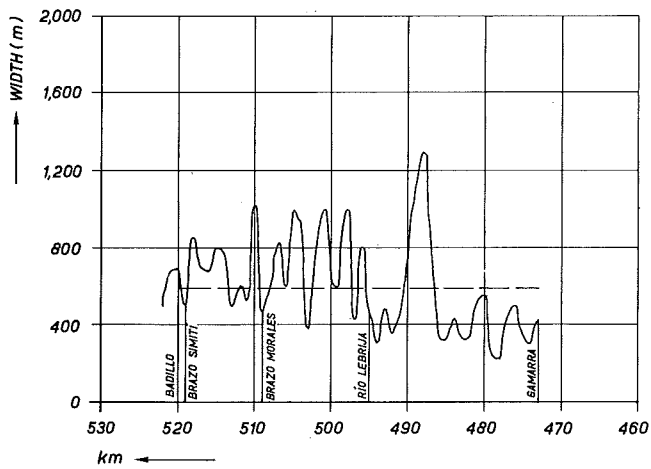
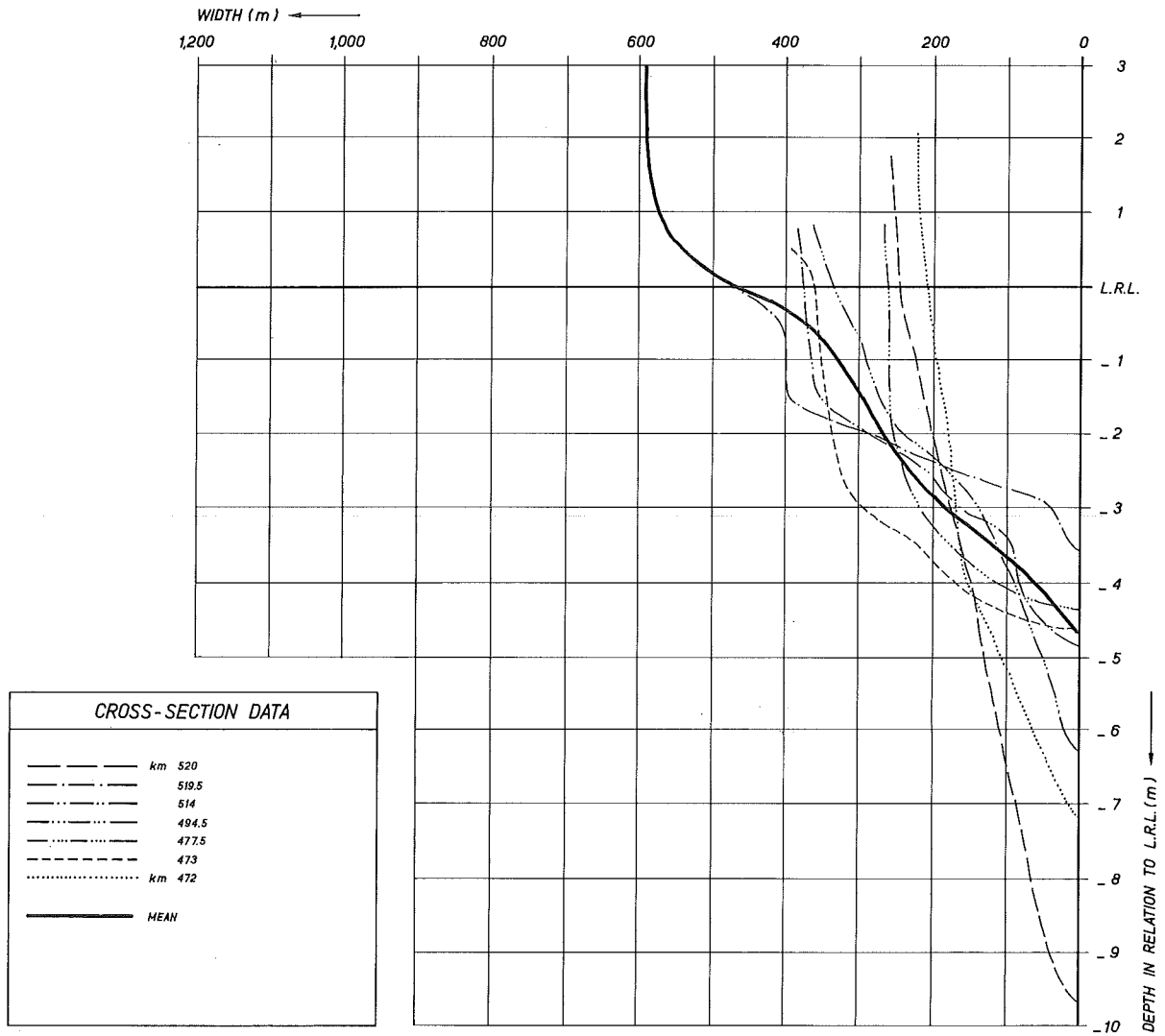
Water-level gradient

As in the other sections, the water-level gradient may differ considerably from place to place; for the section upstream of the confluence with the Rfo Sogamoso, the average water-level gradient as found from the water-levels at the Barrancabermeja and Pto. Wilches gauge-stations is about  $30 \times 10^{-5}$ . Because of the gaps in the water-levels at Gamarra, the water-level gradient between the Rfo Sogamoso Confluence and Gamarra is less accurate. This value was found to be  $20 \times 10^{-5}$  (from water-level data at Pto. Wilches and Gamarra).



SCHMATIZATION OF THE RÍO MAGDALENA km 612 - km 520

FIG. 3.5.1



SCHEMATIZATION OF THE RÍO MAGDALENA km 520 - km 473

FIG. 3.5.2

### Design bend-radius and water depth in outer bend

As for the other sections the design bend-radius and water depth in the outer bend have been determined. Upstream of the influence of the Rfo Sogamoso, the same data can be used as those found for the section Rfo Carare Confluence - Barrancabermeja. For the section just upstream of the Rfo Sogamoso Confluence to Badillo, the main data as given in Figure 3.5.3 are: Bend-radius  $R = 5,500$  m and the depth in the outer bend is 12.50 m below L.R.L.

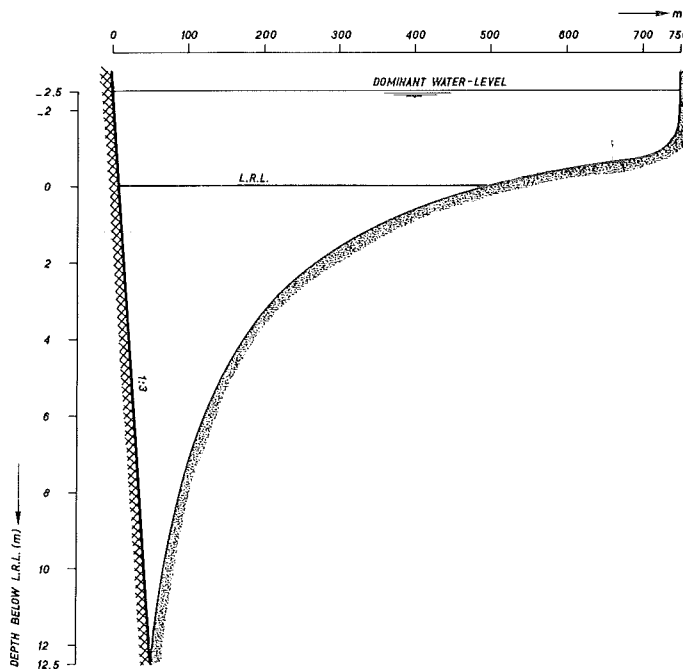


Figure 3.5.3 Computed Cross-section in Outer Bend between km 613 and km 520

For the section downstream of Badillo these data could not be computed as the division of the discharge over the different branches was not known. However, as this section is much more regular and meandering, the bend-radius can be found by studying the charts; a radius of about 4,000 m seems predominant. For the depth in front of river-works, the same depth as in the section upstream of Badillo may be used (12.50 m below L.R.L.).

### Design width and water depth on crossing

The width and water depth on the crossing for the section upstream of the Rfo Sogamoso Confluence are again the same as those found for the section Rfo Carare Confluence - Barrancabermeja.

For the section downstream of Badillo the width and depth on a crossing are not given, as for this section insufficient data concerning crossings are available. It may be recalled, that downstream of the Rfo Sogamoso Confluence the river has been studied in a more general way. Although downstream of Badillo there are still some crossings that present difficulties to navigation, these problems are not very severe, and generally this section offers good conditions for navigation.

### III, 3.5

The design width and depth for the crossings between the Rfo Sogamoso Confluence and Badillo (also to be used just upstream of this confluence), have been determined in the same way as in the foregoing paragraphs. The retarded scour as found from Figure 3.5.4 is 0.50 m, and the design depth below L.R.L. is taken as 2.35 m (7'6"). Taking into account the retarded scour, this means that with a dominant water-level (taken to be 2.50 m above L.R.L.) the bed-level should be  $2.35 - 0.50 \text{ m} = 1.85 \text{ m}$  below L.R.L. From the composite profile (superposition of the cross-sections in two consecutive bends), it follows that a width of 520 m is required. If the retarded scour is not taken into account, this would be 400 m.

From Figure 3.5.2 it follows, that for the dominant water-level the conveyance of the schematized cross-section is  $7,100 \text{ m}^{5/2}$ . Assuming a rectangularly shaped cross-section, this means that the required width to obtain a bed-level of 1.85 m below L.R.L. is 780 m, while neglecting the retarded scour this would be 660 m.

Summarizing: The width on the crossing required to obtain a depth of 7'6" below L.R.L., according to:

composite cross-section is	400 m,
including retarded scour is	520 m,
rectangular cross-section is	660 m, and
including retarded scour is	780 m.

It is difficult to obtain an insight into the rate of retarded scour in a composite channel when the initial depth is already relatively large. As some retarded scour can, however, still be expected, a design width of 600 m therefore seems sufficient. If a river improvement is considered, by the construction of groynes, the length of which can still be adjusted at a later date (when it is known that the required depth is still not reached), a width of about 700 m seems to be sufficient.

#### River stretches requiring improvement

As already mentioned, the section Barrancabermeja - Gamarra has been studied in a somewhat different way from the other sections as far as the lower stretch is concerned (downstream of Badillo). In that stretch there are still a few difficult crossings, but these do not differ appreciably from the crossings further upstream, while the river in general offers better navigation conditions. No measurements have been carried out, neither in the Brazo Simití and the Brazo Morales, nor in the main river. As has been mentioned, the three channels (Simití, Morales and Magdalena) have a stronger tendency to meander than the joined channels would have (see also Part II, Figure 3.7.4).

According to river operators, it seems at present that the Brazos Simití and Morales gain in importance at the cost of the main river channel. This may well be true, as the system is in a rather delicate state of equilibrium, and a slight increase in the resistance of the main channel may already cause a disturbance of this equilibrium situation. To make sure that the main channel (the Rfo Magdalena) will be the major shipping route in future too, the resistance should be kept as low as possible (by means of dredging), while the resistance of the other channels may even be increased. This subject, however, has not been studied in detail by the Mission.

The whole stretch between Barrancabermeja (km 630) and just downstream of the Río Sogamoso Confluence (km 610) in fact offers bad conditions for navigation, largely due to the influence of the Río Sogamoso. It should, therefore, be considered as one problem. In this Chapter a possible alignment of the future navigation channel between Barrancabermeja and Pto. Wilches will therefore be given. This alignment was drawn, taking into account the required accessibility for navigation of the Barrancabermeja Port, and the morphological conditions of the Río Sogamoso Confluence. The access to the Pto. Wilches quay wall, although considered of secondary importance, can possibly be maintained with a limited amount of dredging. However, the "Hidro Electrica Lebrija" power station is situated so unfavourably, near km 626 along the right bank of the Río Magdalena, that the intake of the required quantity of cooling water will probably be difficult.

For convenience, the consequent problems are treated separately.

#### The access to the Barrancabermeja Port

Although at present (1973) the accessibility of the port is not bad, only small changes in the course of the main channel upstream of Barrancabermeja may already cause a deterioration of the existing situation. Further scour of the right bank upstream of Barrancabermeja, will likely cause the crossing of the current to shift upstream, with the ultimate result that near Barrancabermeja the main channel will follow the left bank, causing sedimentation in front of the Barrancabermeja quays. A certain tendency in this direction can already be noted in the present situation. This problem is treated in Para. 3.5.3.

#### The "Hidro Electrica Lebrija" (km 626)

At km 626 a power station is situated on the right bank behind an island. The channel in front of this bank is blocked by sedimentation and carries only a small discharge, which is almost reduced to nil during low water. Consequently, the cooling of the generators presents difficulties, which will even be aggravated when a planned extension of the power station is carried out. Something more is said about this in Para. 3.5.4.

#### The Río Magdalena between km 624 and km 621

In this stretch there is a large island in the middle of the river. At present the channel along the right-hand bank is the main navigation channel, but in 1970 the main navigation route still followed the left bank. In particular at the upstream end, both channels generally offer problems for navigation. This problem is also dealt with in Para 3.5.4.

#### The Río Sogamoso Confluence (km 619 - km 610)

During relatively high stages of the Río Sogamoso (compared with the Río Magdalena), sedimentation occurs downstream of the confluence because the Río Magdalena cannot transport the large amounts of sediment brought down by the Río Sogamoso. Sedimentation also occurs upstream of the confluence, due to a positive backwater-curve in the Río Magdalena and, consequently, lower flow velocities. When the discharge of the Río Sogamoso again diminishes, the Río Magdalena offers bad navigation conditions in this area. More is said about this problem in Para. 3.5.5.

### III, 3.5

#### The access to Pto. Wilches

In front of Pto. Wilches the navigation conditions are generally not bad, but the channel often shifts its course from one side of the river to the other. When the channel does follow the left bank (as at present), the access to the quay wall in Pto. Wilches is blocked by sedimentation. In Para. 3.5.6 it is considered whether the situation can be improved or whether it is better to abandon the facilities at Pto. Wilches altogether.

#### 3.5.2. Temporary improvement by means of dredging

As for all the other sections, the amount to be dredged has to be determined. A required minimum depth of 7'6" has been considered (in agreement with the Schedule of Operations).

In this section the crossings are generally deeper and farther apart, because the river is wider (especially below the Rfo Sogamoso Confluence). However, it was assumed that on all crossings shallower than 7'6", retarded scour would still occur. According to the computations (see Figure 3.5.4), the retarded scour would be in the order of 2'. As the amount to be dredged was based on the sounding of March 1972, some scour would already have taken place, so only part of the scour (1'6") has been taken into account. The shallowest crossing during this sounding was measured at km 611.3 at the mouth of the Rfo Sogamoso, where the depth was 2' below L.R.L.

Again 30% has been added to the amount to be dredged: 15% for waste and side-slopes of the channel and 15% for the assumption that the recorded depth is representative for the full width (50 m) of the channel.

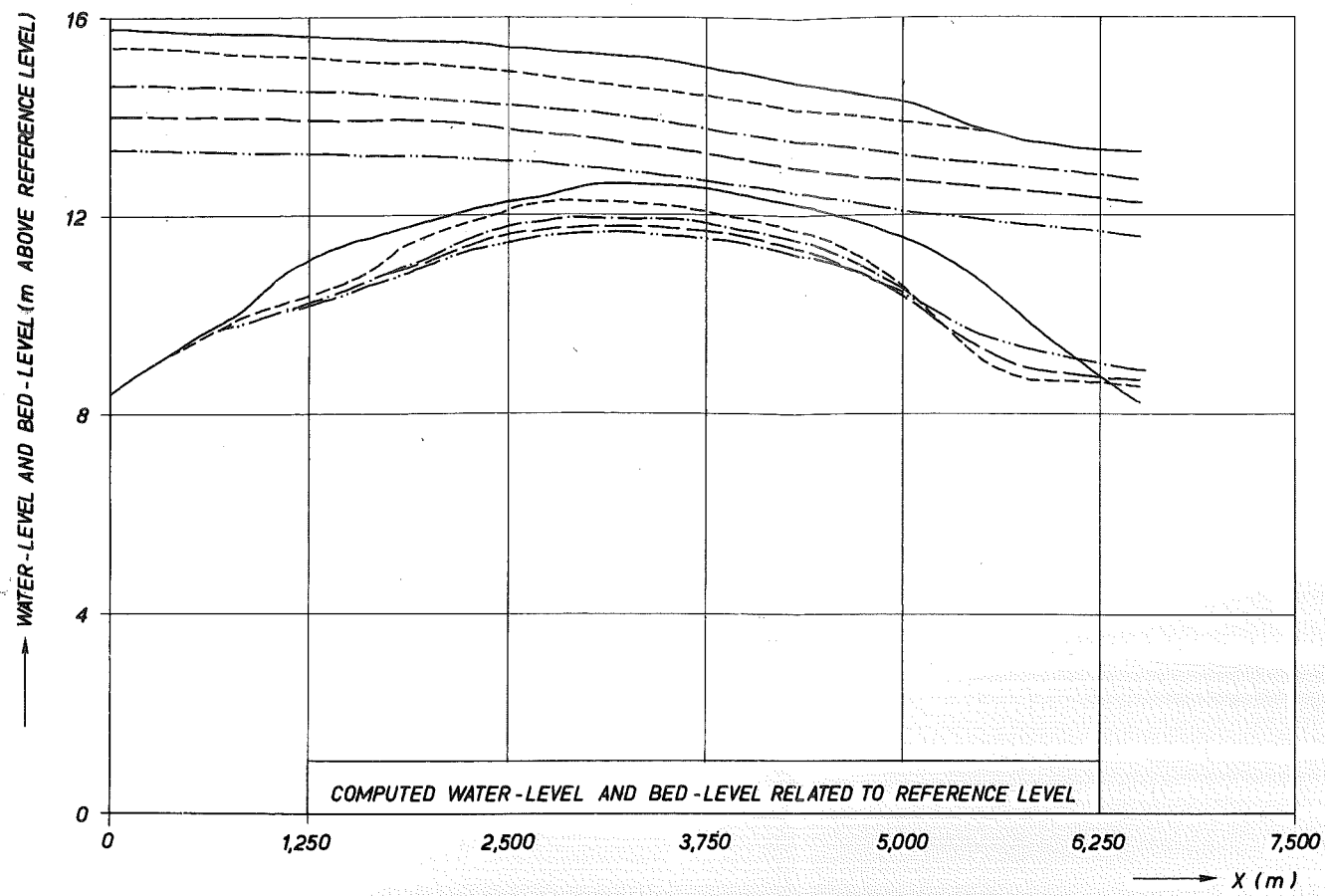
The total volumes to be dredged, are given in Table 3.5.1.

River sections	Kilo- meters	Retarded scour not included			Retarded scour included		
		Volume (m <sup>3</sup> )	Extra (30%) (m <sup>3</sup> )	Total volume (m <sup>3</sup> )	Volume (m <sup>3</sup> )	Extra (30%) (m <sup>3</sup> )	Total volume (m <sup>3</sup> )
Barrancabermeja - Pto. Wilches	630-597	321,000	96,000	417,000	194,000	58,000	252,000
Pto. Wilches - Gamarra	597-475	419,000	126,000	545,000	189,000	57,000	246,000
				Total			Total
							498,000

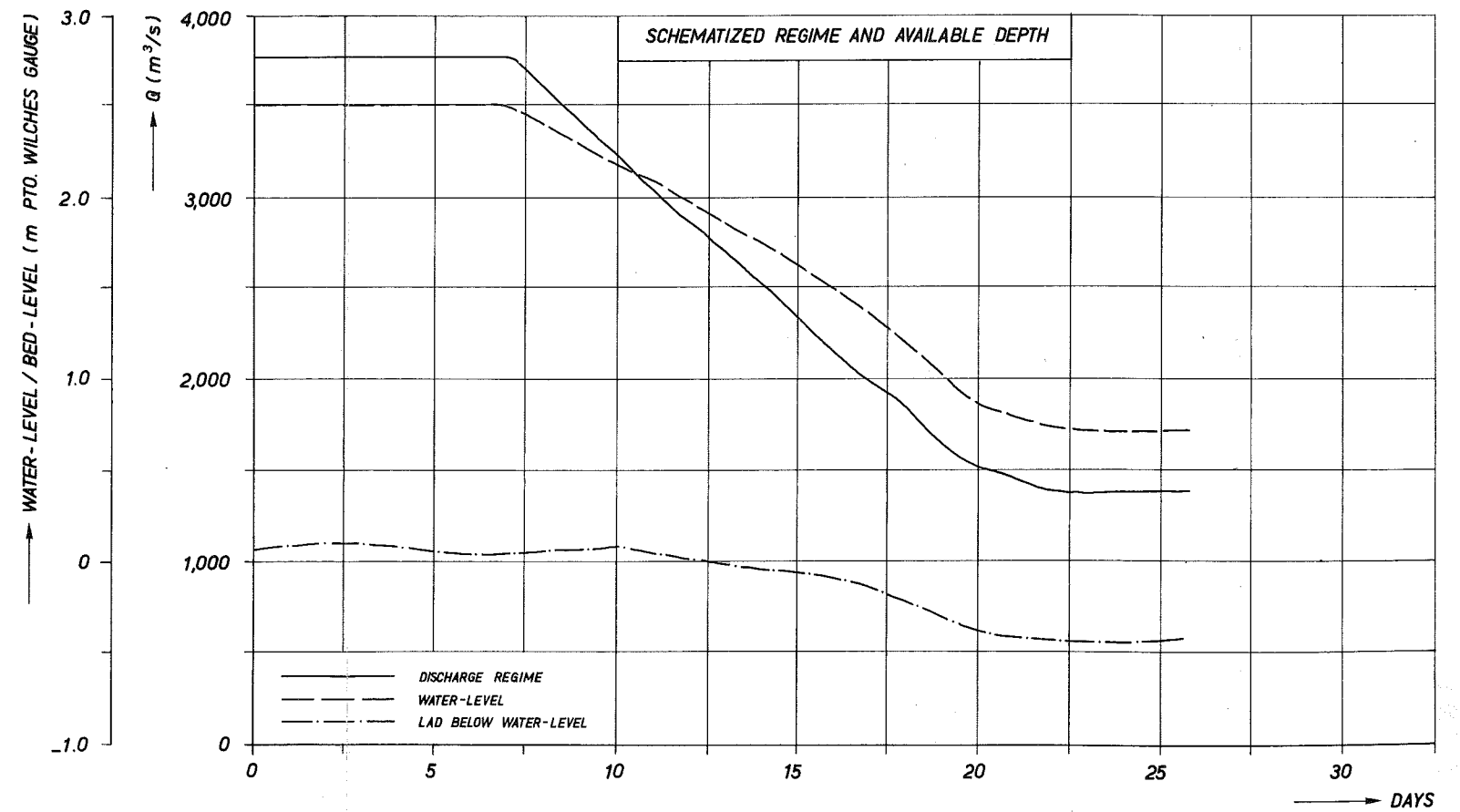
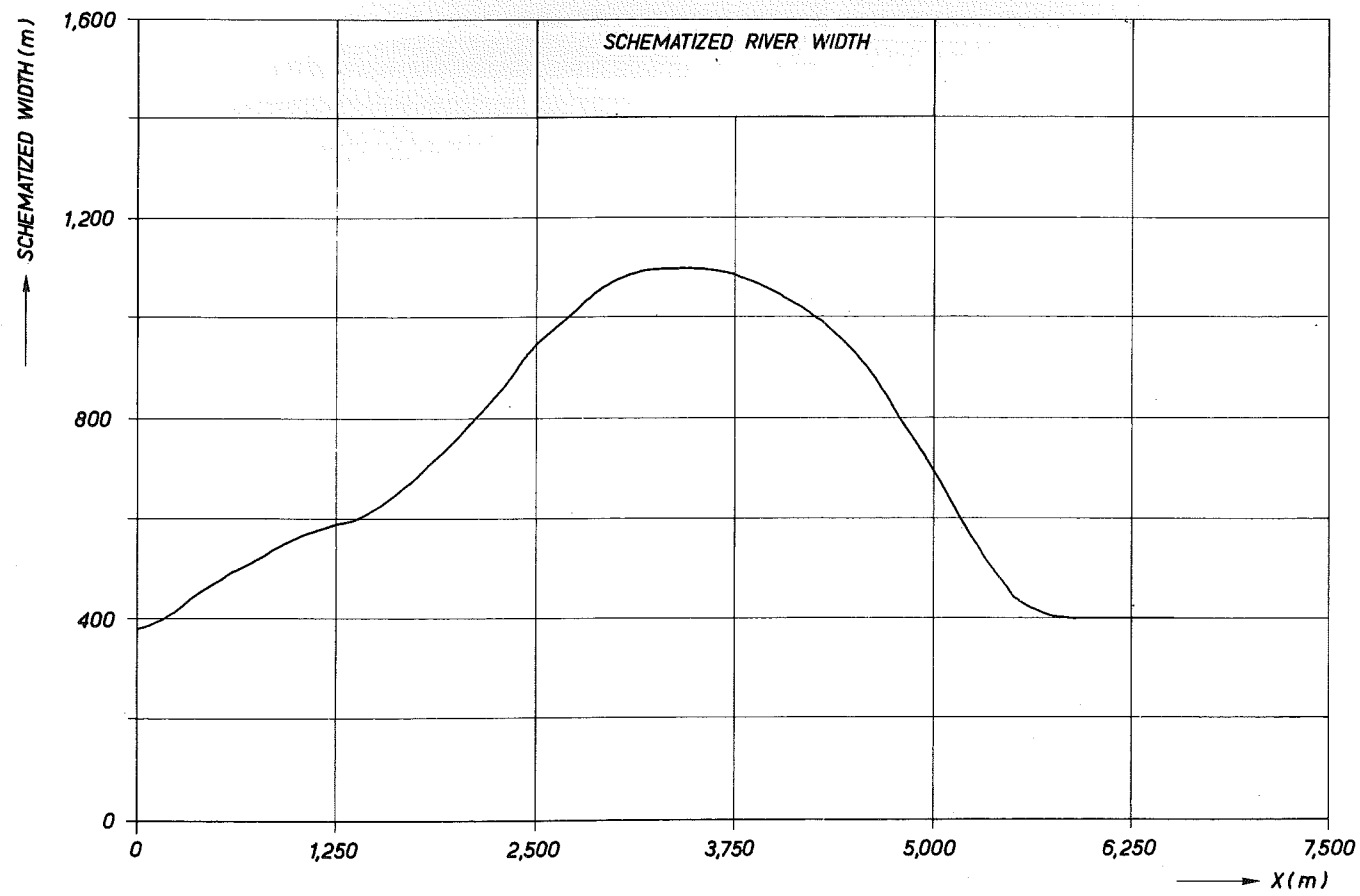
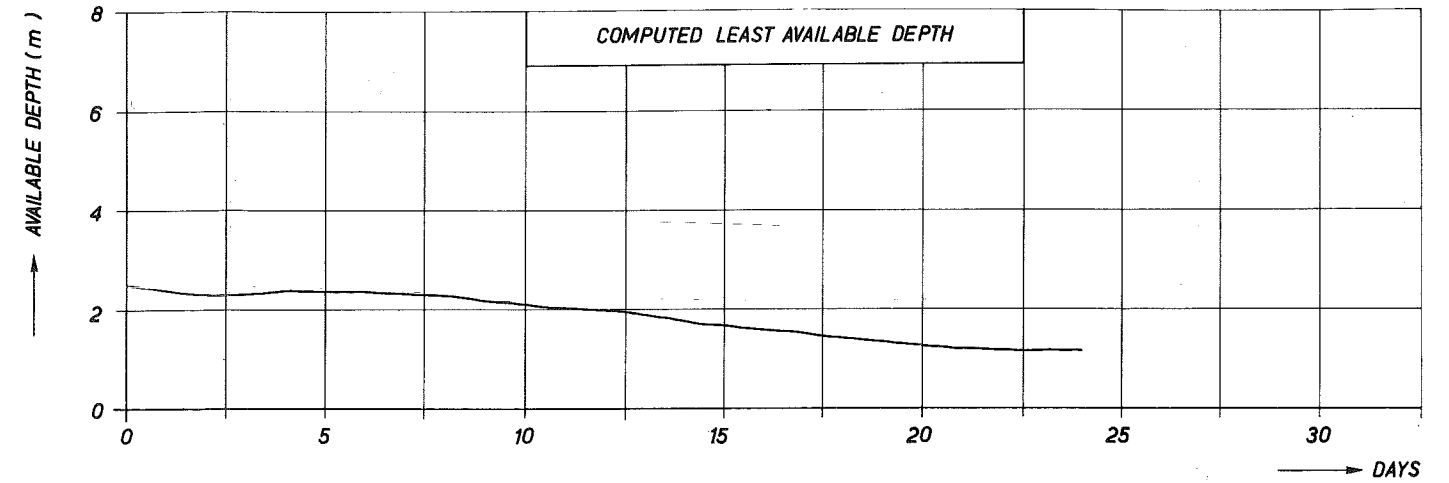
Table 3.5.1 Quantities to be Dredged between Barrancabermeja and Gamarra

#### 3.5.3. The access to the Barrancabermeja Port

The access to the Barrancabermeja Port is governed by the configuration upstream of the port. As upstream of Barrancabermeja the river is free to move in its high water bed, a good as well as a bad situation may develop. In Figure 3.5.5 it can be seen, that the present situation (1972/1973) is much better than that in 1923 and in 1954.



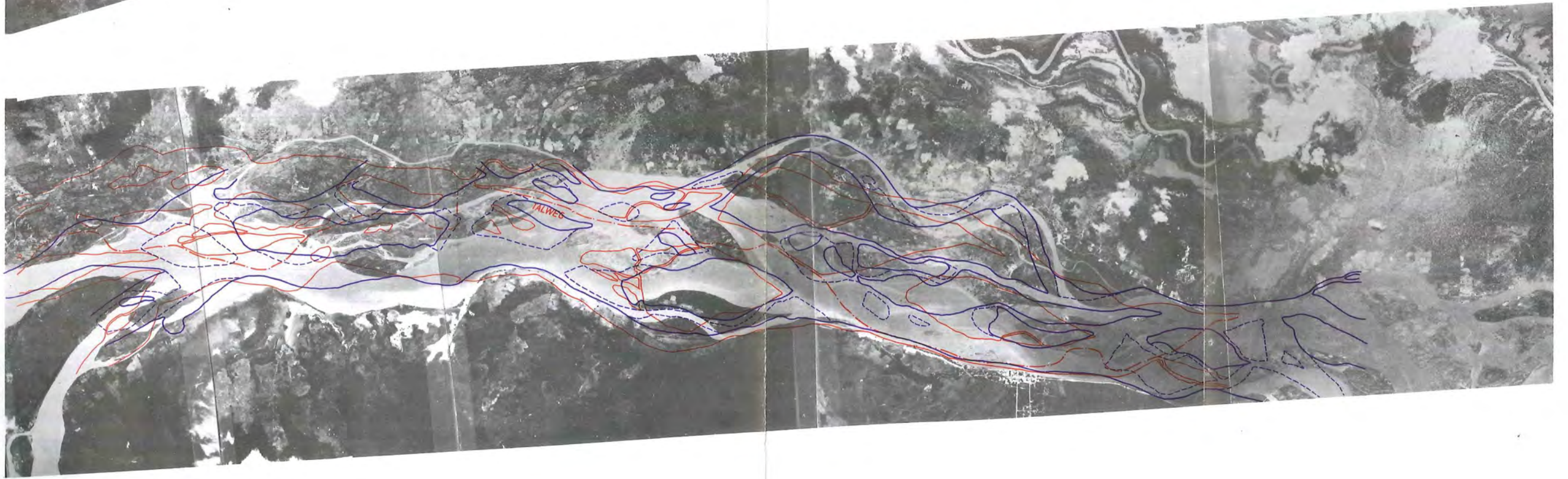
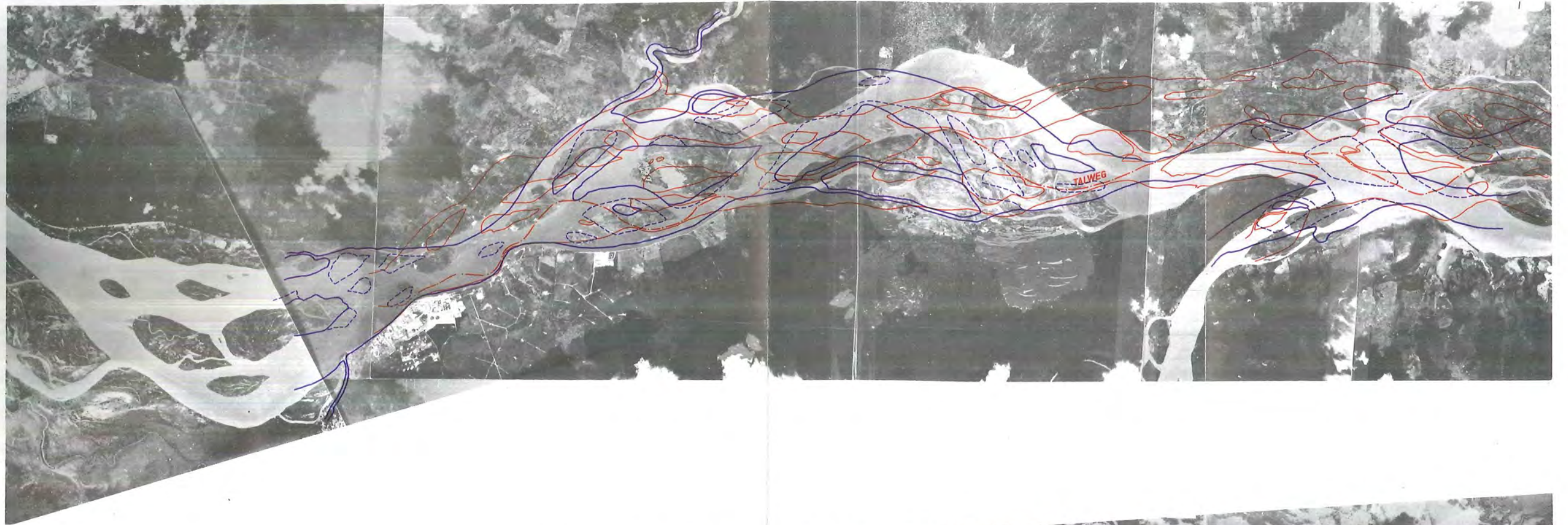
WATER-LEVEL AND BED-LEVEL	TIME		DISCHARGE (m <sup>3</sup> /s)
	DAYS	HOURS	
—————	0	0	3,761
—————	6	0.5	3,761
—————	12	0.5	2,831
—————	18	0.5	2,111
—————	22	0.5	1,344



RÍO MAGDALENA km 615 (UPSTREAM RÍO SOGAMOSO), REGIME 5,  $C = 48 \text{ m}^{1/2}/\text{s}$  (CONSTANT)

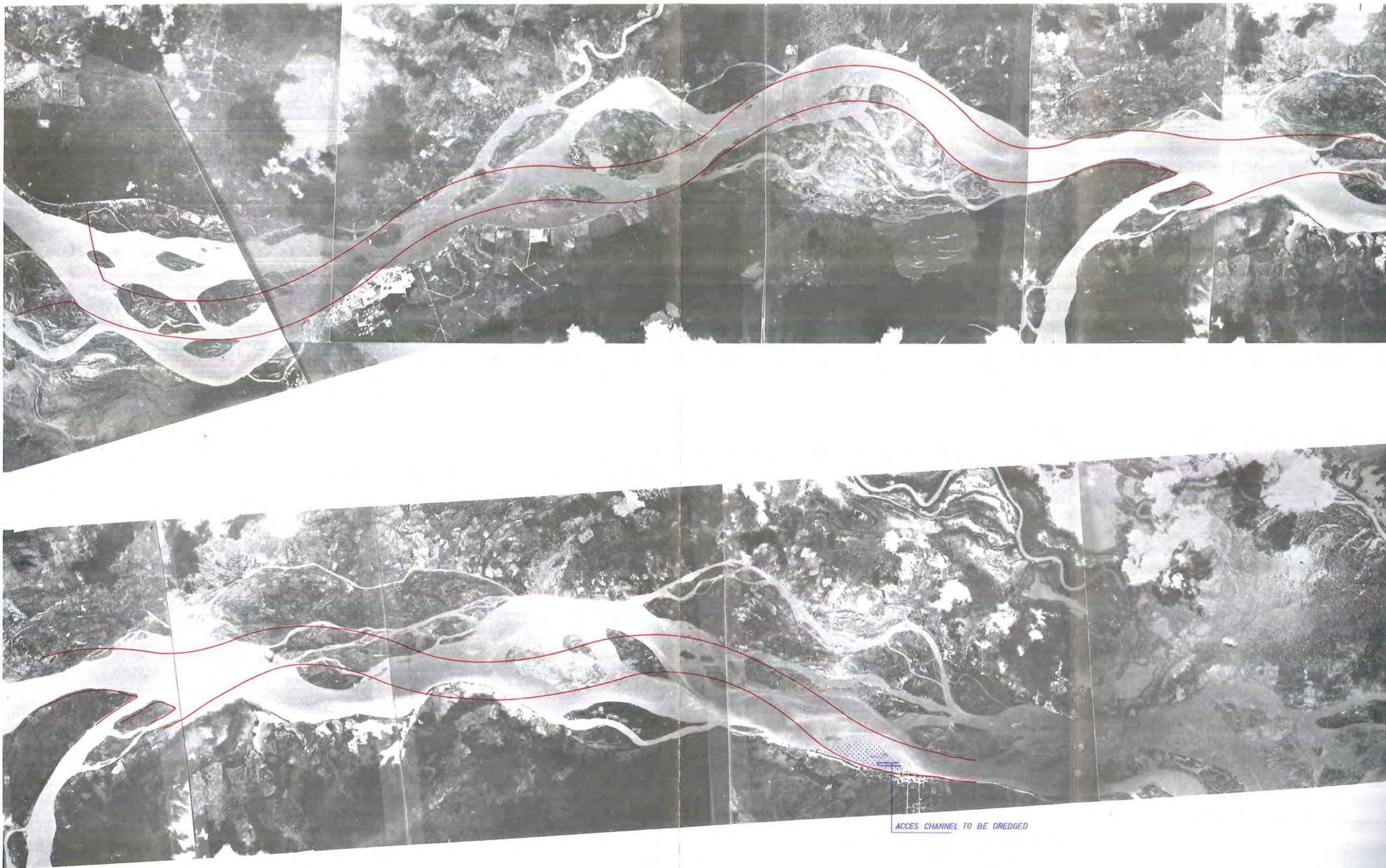
RESULTS OF MORPHOLOGICAL COMPUTATIONS

FIG. 3.5.4



— 1.923  
— 1.954

CASE HISTORY RÍO MAGDALENA BETWEEN BARRANCABERMEJA AND PTO. WILCHES FIG. 3.5.5



ALIGNMENT FOR PERMANENT IMPROVEMENT BARRANCABERMEJA-PTO. WILCHES REACH FIG. 3.5.6

In 1923, the main channel crossed just upstream of Barrancabermeja from the right bank to the islands along the left bank, with a tendency to split and form an island in the middle (Galán). One channel then crossed back to Barranquita and followed the present channel in front of the power station, while the other went along the left-hand side of the "Isla de Galera". In this way, the quays at Barrancabermeja tended to be situated in an inner bend, with only small depths. In 1954, the situation was very complicated: the large island upstream of Barrancabermeja ("Isla de San Antonio") had moved downstream, partly in front of the quays, blocking their access. Upstream of the island there were flats.

Compared to this, the 1972-situation appears to be more favourable for navigation purposes but, dangerous in view of its instability, because the right bank upstream of Barrancabermeja has scoured considerably (and in fact is still eroding). Although part of the current still follows the right-hand bank along the quay wall, it may be expected that if scour of this bank upstream is not stopped, after some time the crossing of the main channel will shift upstream of the quays (Hotel Pipaton). The quays will then again be situated in an inner bend and possibly a large sand-bank will be formed there. To prevent this unfavourable development, the scour of the right-hand bank upstream of Barrancabermeja should not only be stopped, but the alignment of this bank should partly be brought back as indicated in Figure 3.5.6. (As has already been explained, the alignment given in this figure also takes into account the situations near the Sogamoso Confluence and near Pto. Wilches). In order to ensure such an alignment, at the upstream end provisions should be made to trap the current in any direction from which the current would come. However, as the risk will always exist that a new channel will be formed outside (behind) the indicated alignment, this must be prevented at all costs.

For the types of construction to be used for these river-works, reference is made to Chapter 2 and to Paras. 3.2.3 and 3.4.4.

The cost of the river-works as indicated in Figure 3.5.6, from upstream of Barrancabermeja to Pto. Wilches, will be extremely high. They will not all have to be made at the same time, but a start is required in any case to ensure access to the Barrancabermeja Port. An estimate of the total cost may be in the order of Col. \$ 1,000,000,000.

#### 3.5.4. The "Hidro Electrica Lebrija" (km 626) and navigation between km 624 and km 621

A large island ("Galera") is situated in the middle of the river, with a channel on both sides of about equal importance. Sometimes the best navigation channel is along the left bank (as in 1970), and sometimes along the right bank (1973). For navigation purposes, it would be best to increase the resistance of the left-hand channel (see Figure 3.5.6), and thus obtain a permanent preference for the right-hand channel.

In connection with the cooling water problem of the power station, however, probably both solutions are equally bad. If the right-hand channel is maintained, the power station is situated along an inner bend of the river and even without the island in front of the power station, a sand-bank would form. If the left-hand channel would be maintained, the lower end of the minor branch in front of the power station would sediment (see

### III, 3.5

Figure 3.5.6). (It may be seen in Figure 3.5.5, that in 1954 and 1972 the situation was not very favourable, while also in 1923 the situation would have been difficult).

In Figure 3.5.6 a possible alignment of the future navigation channel is given, primarily based on the required accessibility of the Barrancabermeja Port and the morphological conditions of the Río Sogamoso Confluence. In fact, an alignment where the Barrancabermeja quays are situated along an outer bend, will generally have the power station along an inner bend. This means, that the cooling problem of the power station should be adapted to the indicated future situation. Plans exist to obtain cooling water near Barranquita, after the extension of the power station, and the required amount of cooling water ( $6 \text{ m}^3/\text{s}$ ), will then be transported to the power station through a pipeline. The alignment given in Figure 3.5.6, should guarantee the availability of water at the water intake. Some river-works will, however, be required because the present situation does not seem stable and, if no such works are carried out, availability of water at the intake for the power station cannot be guaranteed. It is questionable whether the planned intake near Barranquita is very suitable, because in the given alignment the intake would be situated in an inner bend and a special provision would be required (for example, a mobile water intake).

#### 3.5.5. The Río Sogamoso Confluence

##### Introduction

The Río Sogamoso is, after the Río Cauca, the largest tributary of the Río Magdalena, and it brings large quantities of sediment into the Río Magdalena, which partly sediment there. The extra water discharge also has its effect on the river, not only downstream but also upstream of the confluence, due to backwater effects (the regimes of Río Magdalena and the Río Sogamoso are often not in phase). The course of the Río Magdalena changes rapidly in this area; not only as far as the low water bed is concerned, but also the high water banks shift frequently (a case history is given in Figure 3.5.5).

This area is the main bottle-neck for river traffic between the Caribbean Coast and Barrancabermeja, due to the following facts:

- A high discharge of the Río Sogamoso, combined with a relatively low discharge of the Río Magdalena, results upstream of the confluence in a positive backwater-curve (depth larger than normal), causing sedimentation. When the discharge of the Río Sogamoso again drops, the Río Magdalena removes this sedimentation, but the navigation channel has already been spoilt and time is required before the river has again scoured a new channel through the sedimentation.
- The same high discharge of the Río Sogamoso brings more sediment into the Río Magdalena than this river can transport. Again, this sedimentation will be removed during higher discharges of the Río Magdalena, but until that time the navigation channel downstream of the confluence is spoilt.
- Even when good channels exist, they are difficult to find for navigation because of their rapid changes and high sinuosity.

Due to large expanse of water, orientation is difficult, which is yet another reason why the best channel may be missed (even when indicated by beacons). Buoyage, therefore, seems to be required to mark the navigable channel.

Permanent river-works are not suitable to solve the problems in this area, in view of the changes that occur so rapidly. To be able to adapt the river improvement to these changing conditions, temporary works and semi-permanent river-works will be more suitable. Recurrent dredging will be required, but also the closure of secondary branches as a semi-permanent work may improve the conditions for navigation. All temporary works should, however, take into account the required accessibility of the Barrancabermeja Port and the alignment of future permanent river-works. The usefulness of these semi-permanent works in this area has been proved by the closure of a secondary branch by ADENAVI in 1966 and which, according to ADENAVI, improved shipping conditions considerably for some years (see Figure 3.5.7).

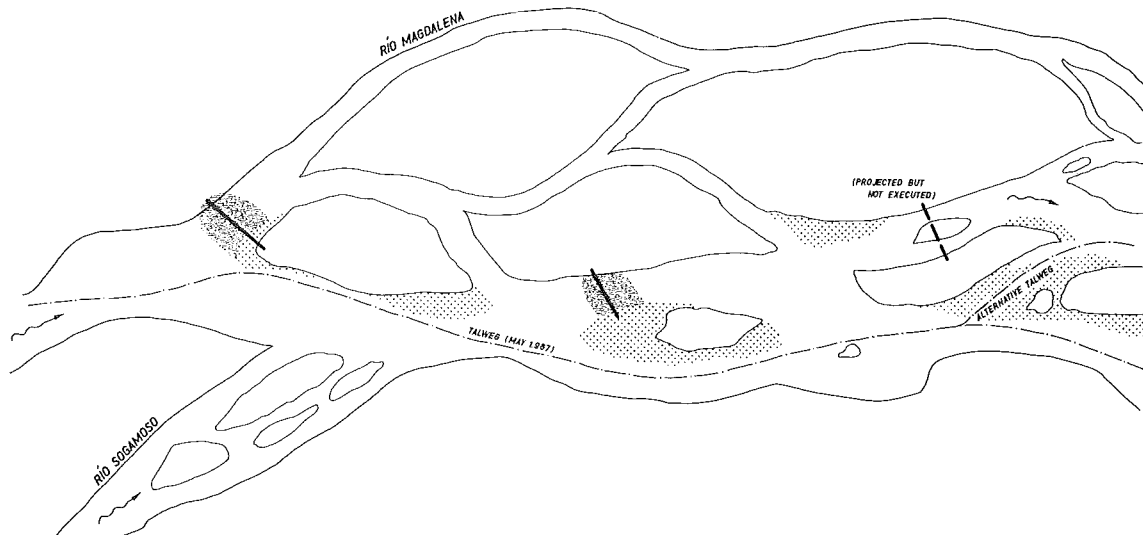


Figure 3.5.7 River-works Carried out by ADENAVI in 1966

The area near the Río Sogamoso Confluence was also selected to carry out test dredging in 1972; not so much because technically it was the most suitable area for test dredging, but because it was the bottle-neck for navigation between Calamar and Barrancabermeja and successful dredging would have given the most direct improvement for shipping. Moreover, the area is close to Barrancabermeja (MITCH office), so that it was easier to supervise the dredging and carry out the required measurements. As the test dredging in 1972 must be considered a complete failure, it was recommended to repeat the test dredging in 1973 in this area.

In view of the available data gathered for the test dredging, this area has also been used as a test case for the computation of the scour on a crossing, although technically more suitable places (less complicated) could have been found (Chapter 3.8 of Part II).

Case history

It is possible to obtain an insight into the morphological changes over the past 50 years by comparing the soundings made by the Julius Berger Konsortium (1923), the aerial photographs of 1954 and those of 1972. Although the main course of the river did not change over very large distances, the changes have been such, that it was impossible to make a comparison of merely the area near the Río Sogamoso Confluence. Only by comparing the complete section between Barrancabermeja and Pto. Wilches, could the aerial photographs be adjusted and the shifts of the river be judged (see Figure 3.5.5). The rapid shifts can also be deduced from the bank just opposite the mouth of the Río Sogamoso, where many old banks may be seen crossing one another (instead of having a tendency to be parallel), although it is impossible to say which bank is the oldest one. The mouth of the Río Sogamoso must have shifted several times, but not over very large distances. The secondary channels downstream of the confluence (along the left bank), which are given on the 1923 map (and for a large part also on the photographs of 1954), were almost completely sedimented in 1972, possibly partly due to the works carried out by ADENAVI.

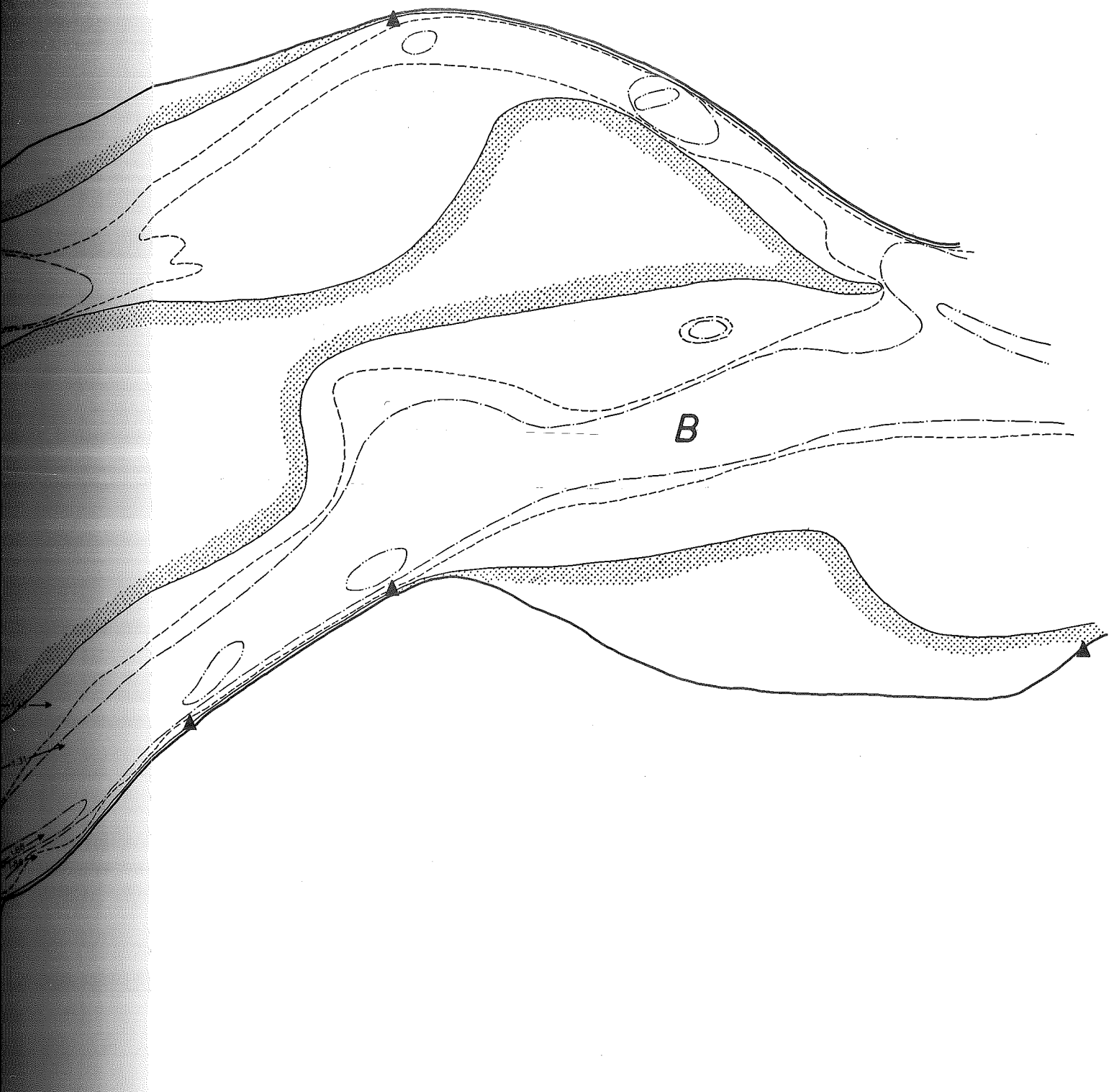
The low island in front of the mouth of the Río Sogamoso, which appeared on the 1954 photographs, was in 1971 actually much larger (see Figure 3.5.8), but disappeared again in 1972

In 1966, with the help of a dredger (DH 6), ADENAVI closed a number of secondary channels, opposite the mouth of the Río Sogamoso. Remnants of these closures can still be seen on the photographs of 1972 (see also Figure 3.5.5). As a result of these works, the main current was forced along the right bank, while along the left-hand side a new bank was formed of the connected islands. The conditions for navigation improved temporarily, but due to scour of the unprotected right bank, the river widened and started to braid again, counteracting the original improvement. It is not known, where the major navigation problems existed before the main channel was forced along the right bank; at present, however, this often occurs near km 615, upstream of the confluence (see also Figure 3.5.9).

The more recent changes can be studied in greater detail, with the help of the available soundings and flow-lines. This is done later when discussing the test dredging of 1972.


Improvement of the section

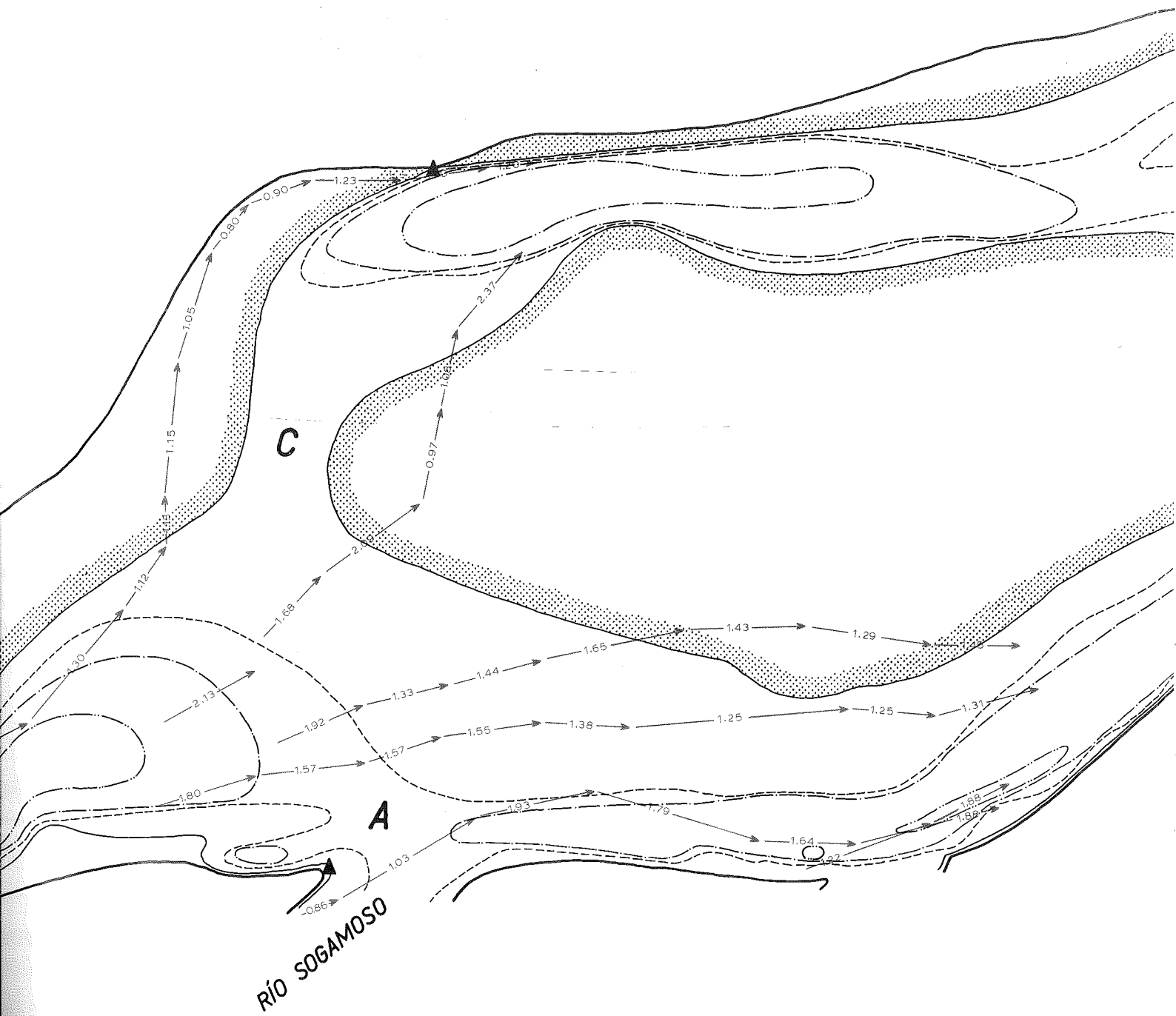
In view of the rapid changes and the many different channels, it seems worthwhile to try and obtain a more stable situation with fewer secondary channels, more or less in the way already tried by ADENAVI in 1966. However, due to these rapid changes, it is impossible to predict exactly how the river will react to such works and, as the river's reaction also may differ from one season to another, it is necessary to use a flexible system of river-works. This is only possible by means of temporary and semi-permanent river-works, while dredging will always be required. In future, these temporary works may have developed a more stable situation, which may then be fixed by means of permanent river-works.



SONDEO / SOUNDING **RÍO MAGDALENA** km 614-km 609 AGUAS ARRIBA DEL / UPSTREAM OF **RÍO SOGAMOSO**

ESCALA/SCALE 1:10.000  
 FECHA/DATE 4.5.9.10 -VIII- 1.971  
 NIVEL DE REDUCCIÓN: 0.70 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. WILCHES  
 CHART DATUM: 0.70 metres ABOVE ZERO OF THE PTO. WILCHES GAUGE  
 FECHA/DATE: 31 - VIII - 1.971 NIVEL DE AGUA/WATERLEVEL: 2.05 m SOBRE EL DATUM / ABOVE DATUM  
 LÍNEAS DE CORRIENTE / FLOW LINES — 1.18 → VELOCIDAD EN m/seg. / VELOCITY IN m/s

CURVAS ISOBATAS	——— 0 m		PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
DEPTH CONTOURS	- - - - 1.5 m		
	— · — · 2.5 m		
	— · — · 5 m		
	— · — · 10 m		

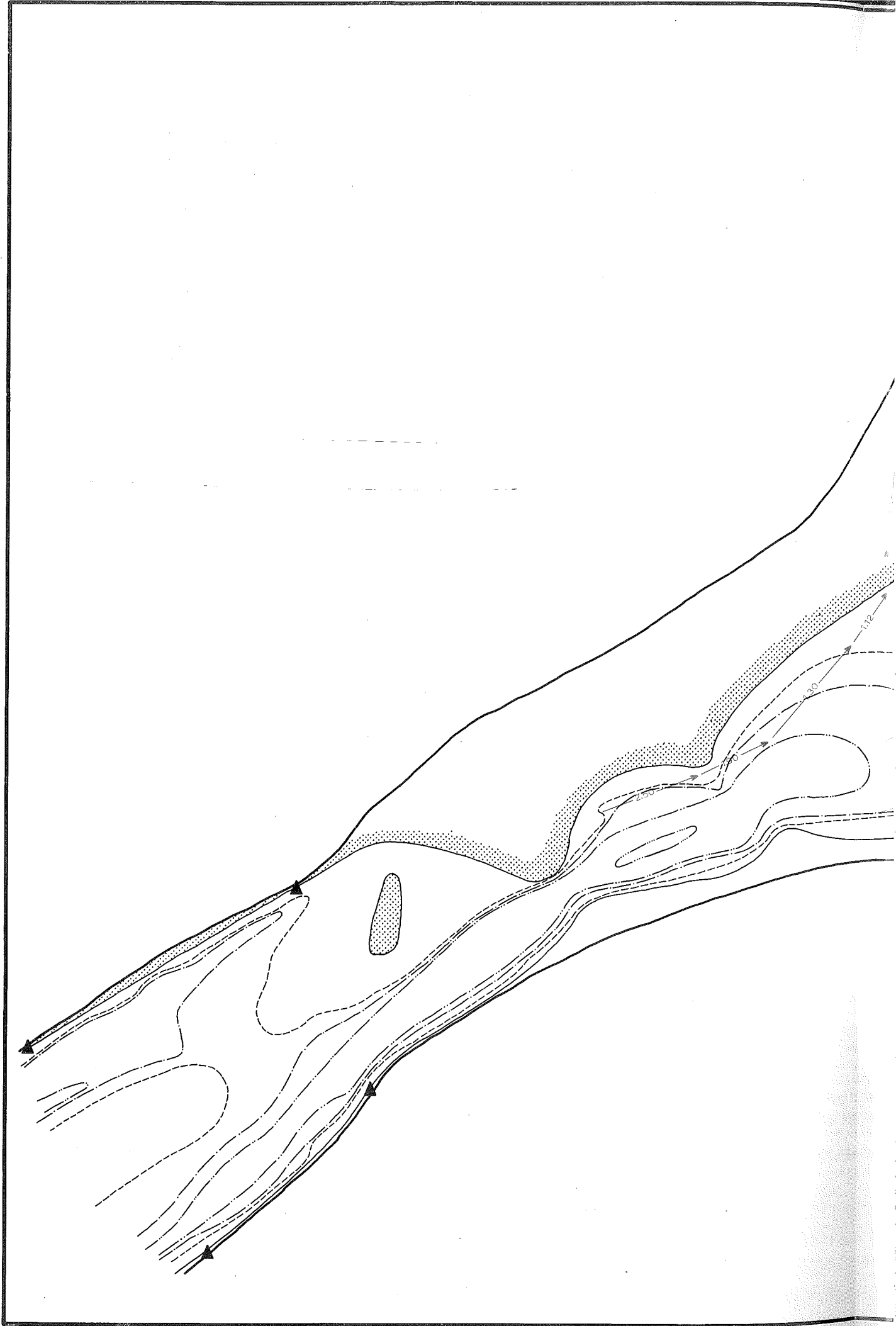


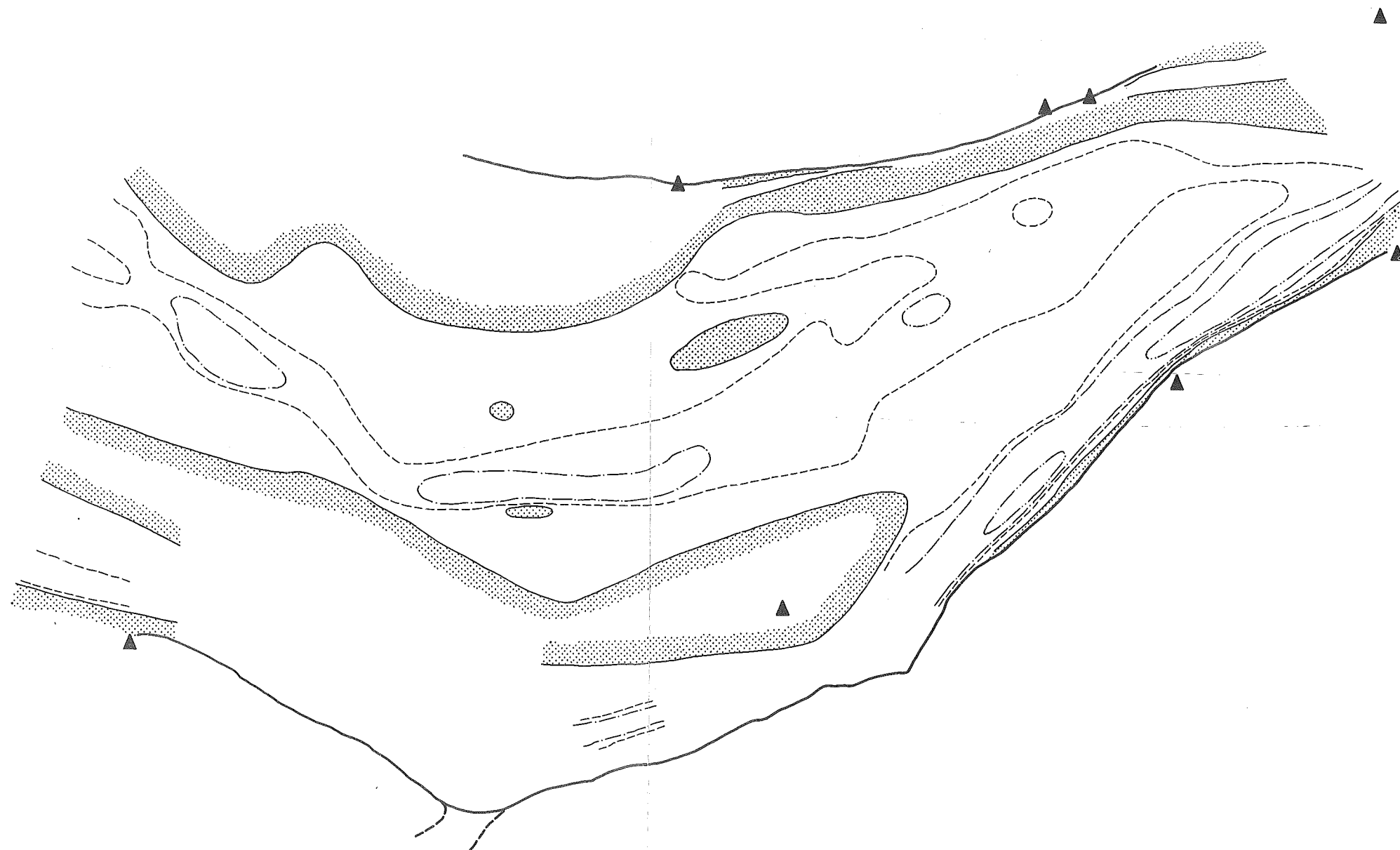
RÍO SOGAMOSO

C

A

S  
SU





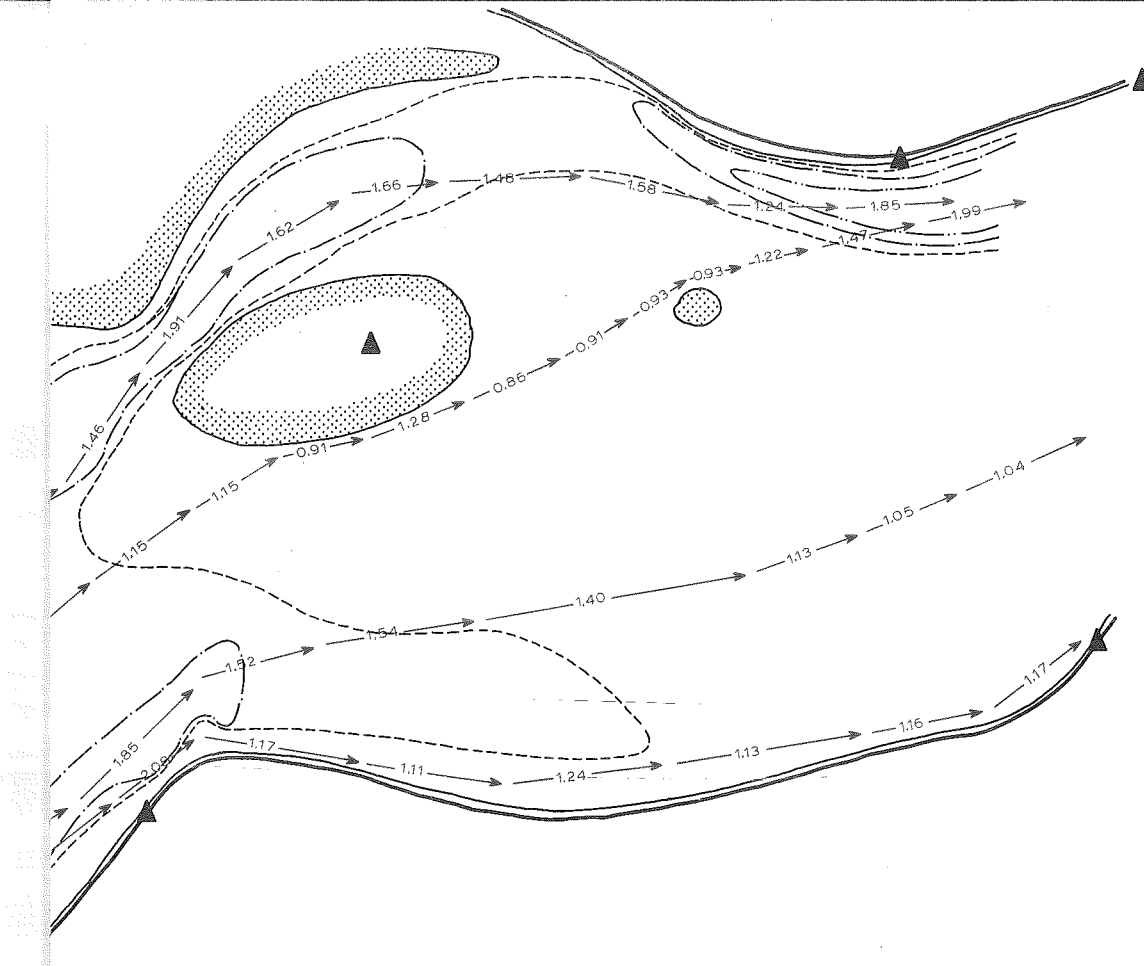
SONDEO / SOUNDING **RÍO MAGDALENA** km 616 - km 614 AGUAS ARRIBA DEL / UPSTREAM OF **RÍO SOGAMOSO**

ESCALA/SCALE 1:10,000  
 FECHA/DATE 15 -XII- 1.971  
 NIVEL DE REDUCCIÓN: 0.70 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. WILCHES  
 CHART DATUM: 0.70 metres ABOVE ZERO OF THE PTO. WILCHES GAUGE

CURVAS ISOBATAS DEPTH CONTOURS	—————	0 m	PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
	- - - - -	1.5 m	
	- - - - -	2.5 m	
	- - - - -	5 m	


NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

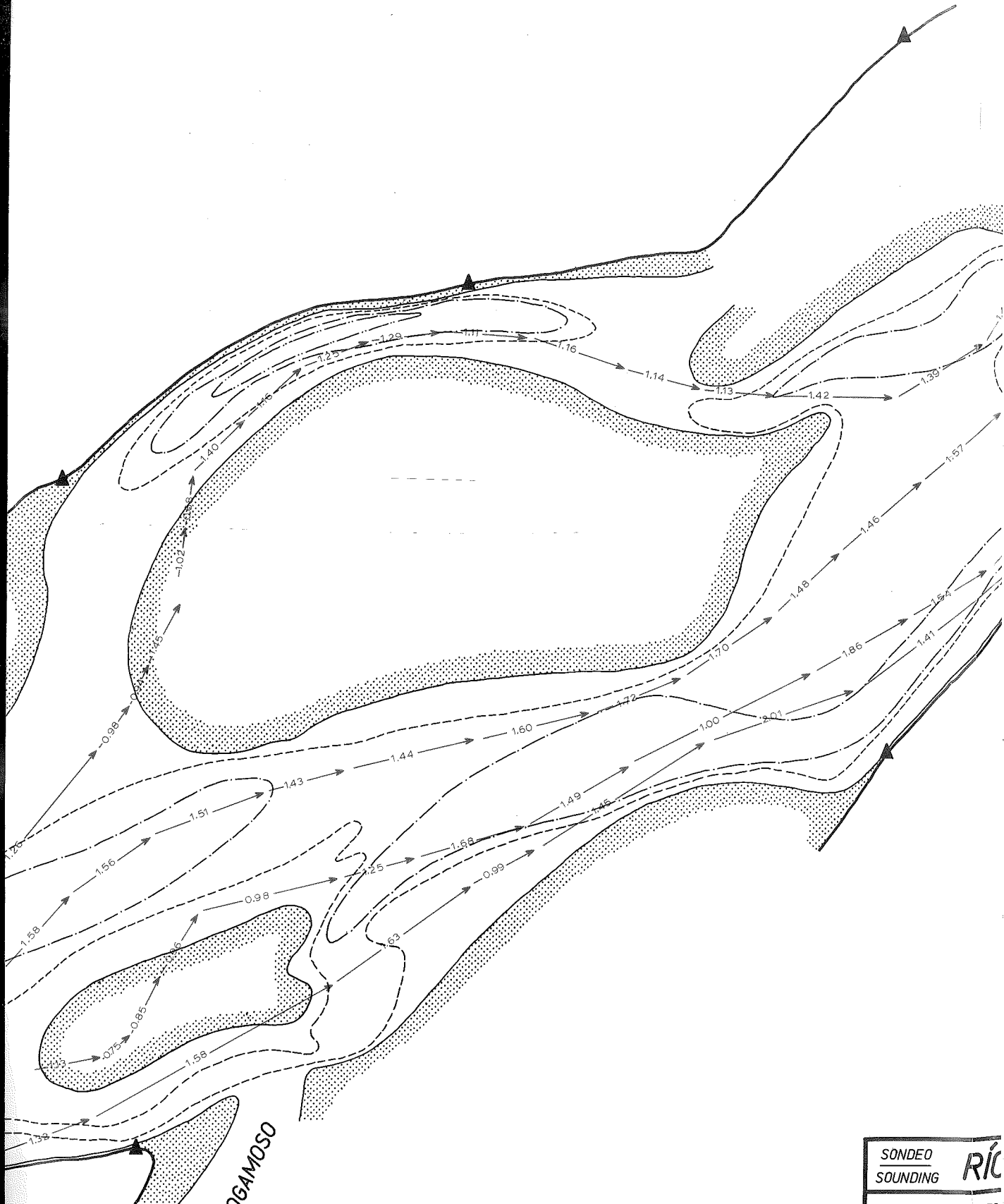
FIG. 3.5.9



**RÍO MAGDALENA** km 614 - km 609 AGUAS ARRIBA DEL RÍO SOGAMOSO  
UPSTREAM OF

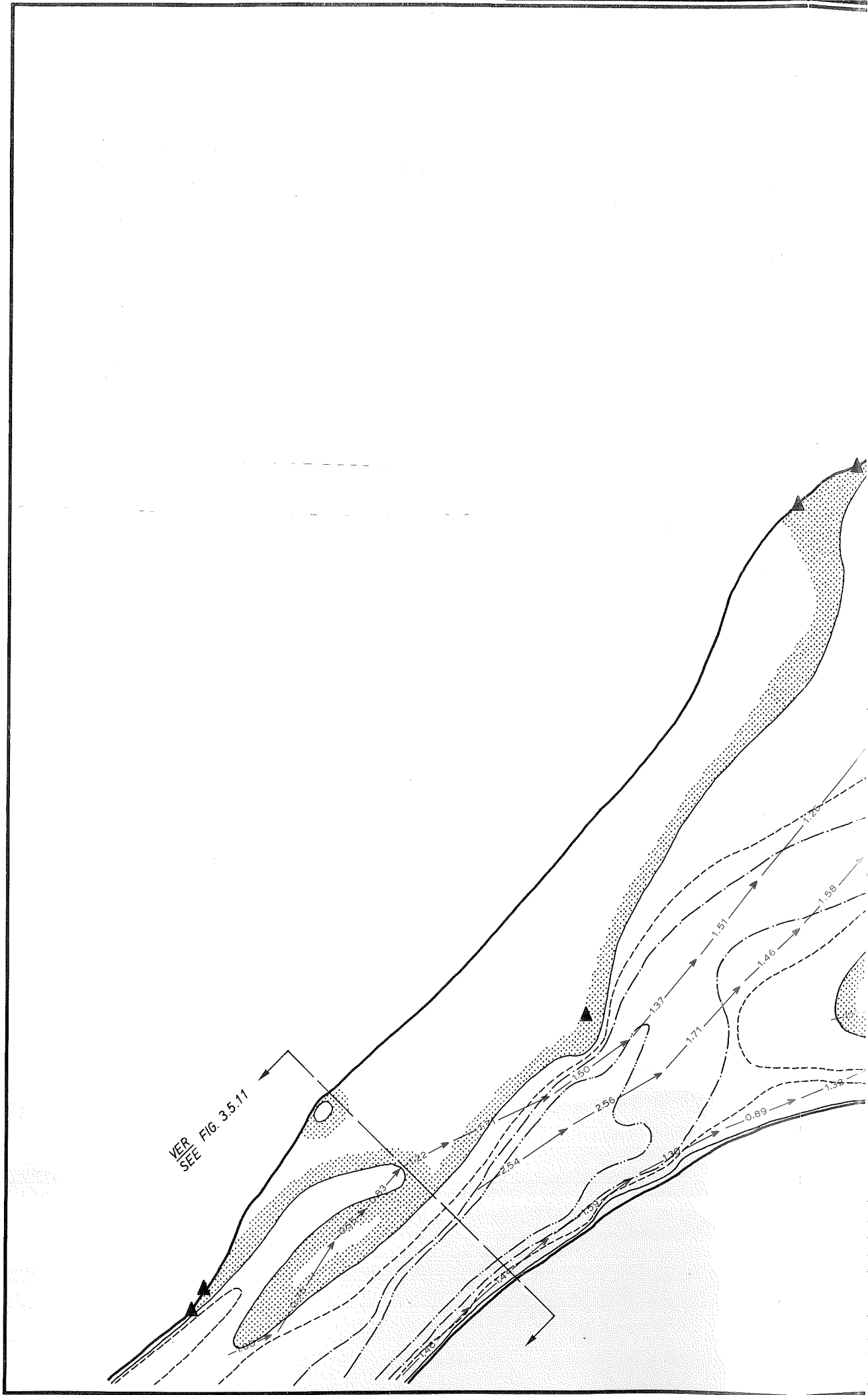
ESCALA/SCALE 1:10,000  
 FECHA/DATE 11-I-1972  
 NIVEL DE REDUCCIÓN: 0.70 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. WILCHES  
 CHART DATUM: 0.70 metres ABOVE ZERO OF THE PTO. WILCHES GAUGE  
 FECHA/DATE: 12-I-1972 NIVEL DE AGUA/WATERLEVEL: 1.55m SOBRE EL DATUM / ABOVE DATUM  
 LÍNEAS DE CORRIENTE / FLOW LINES — 1.18 → VELOCIDAD EN m/seg. / VELOCITY IN m/s

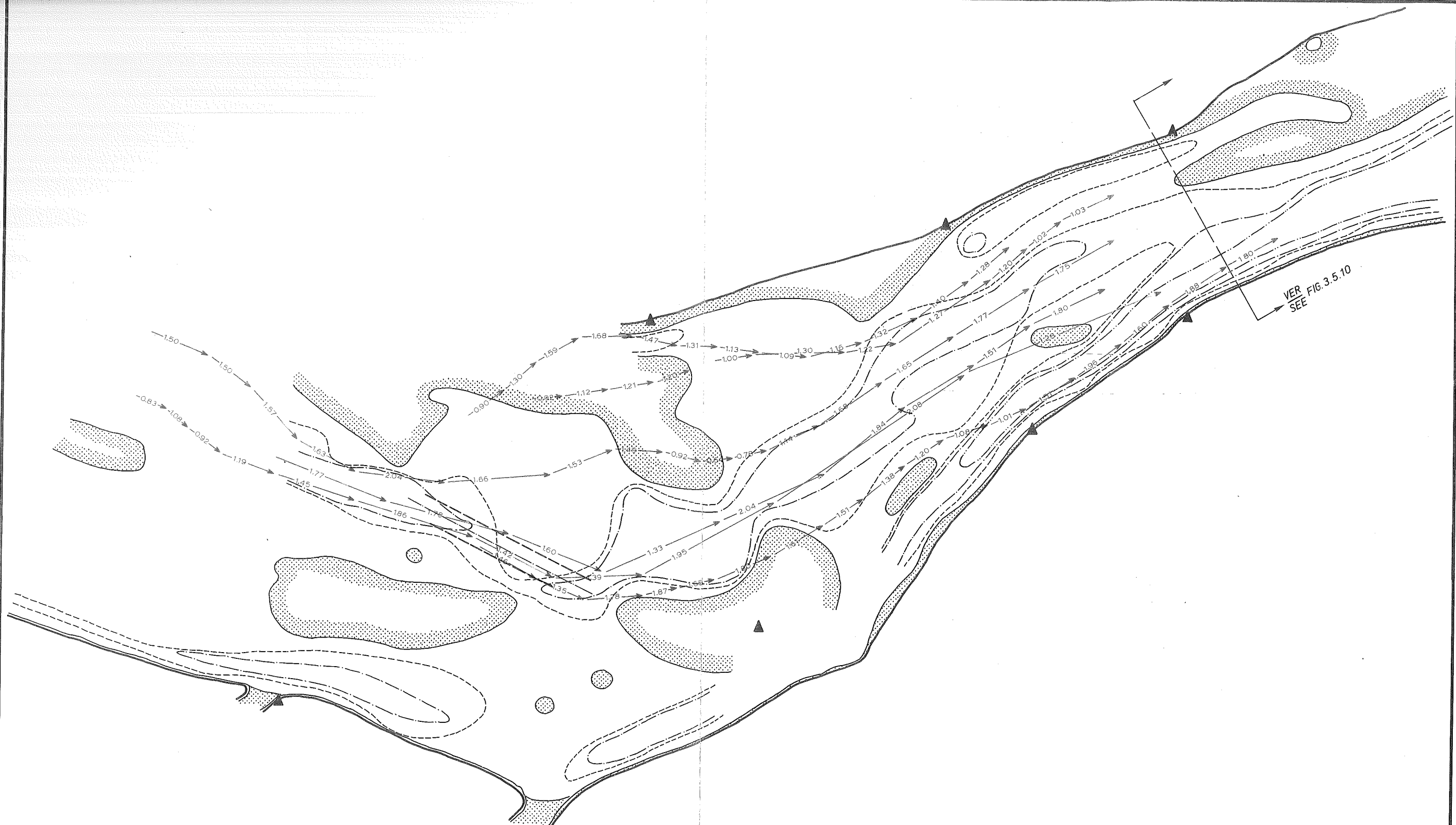
CURVAS ISOBATAS	—————	0 m		PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
DEPTH CONTOURS	- - - - -	1.5 m		
	—————	2.5 m		
	—————	5 m		



RÍO SOGAMOSO

SONDEO	RÍO
SOUNDING	
ES	
FEC	
NIV	
CHA	
FEC	
LIN	
NEDECO	



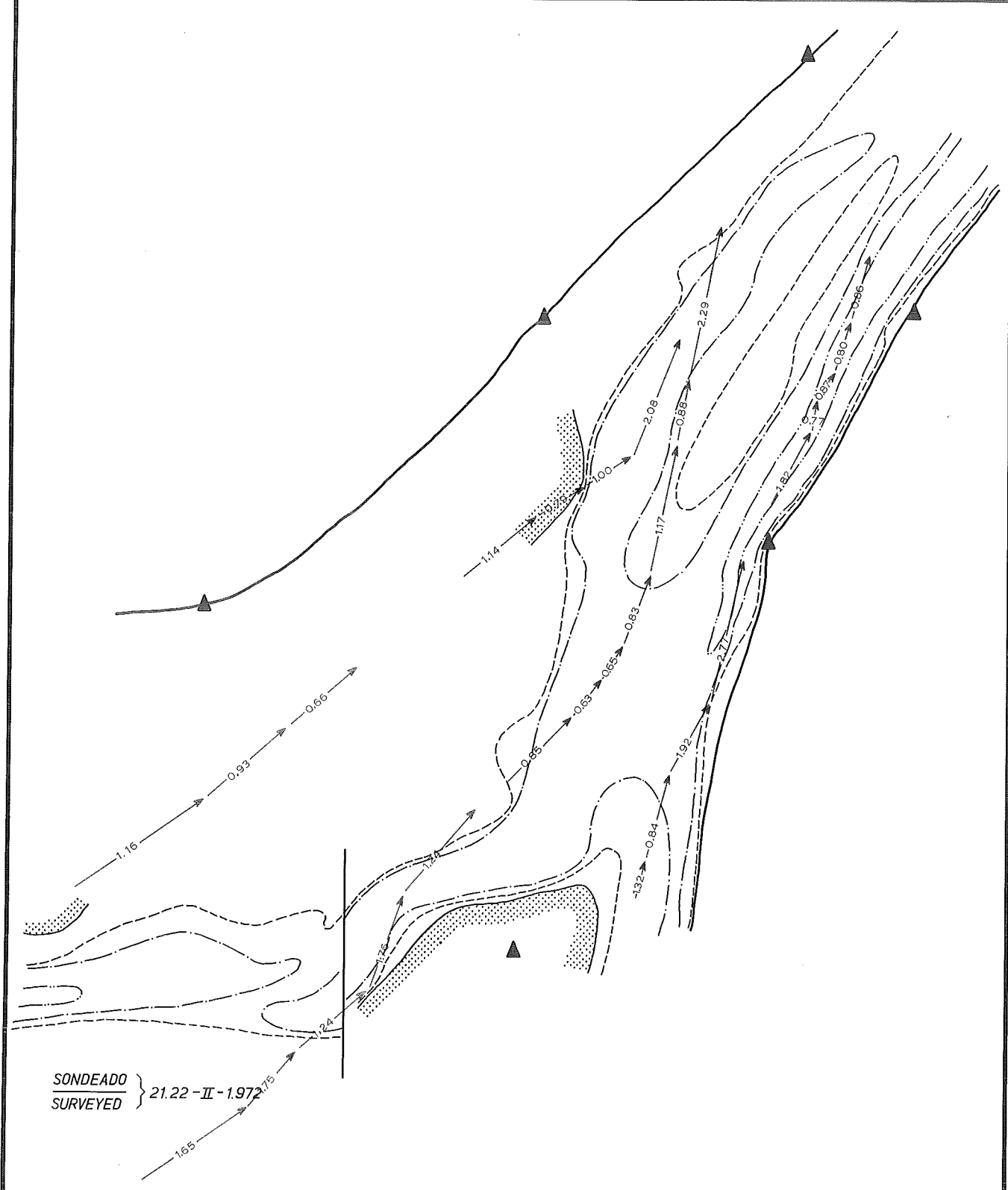


VER FIG. 3.5.10  
SEE

SONDEO / SOUNDING **RÍO MAGDALENA** km 616 - km 614 **AGUAS ARRIBA DEL / UPSTREAM OF** **RÍO SOGAMOSO**

ESCALA / SCALE 1:10,000  
 FECHA / DATE 10-I-1972  
 NIVEL DE REDUCCIÓN: 0.70 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. WILCHES  
 CHART DATUM: 0.70 metres ABOVE ZERO OF THE PTO. WILCHES GAUGE  
 FECHA / DATE: 12-I-1972 NIVEL DE AGUA / WATERLEVEL: 1.55m SOBRE EL DATUM / ABOVE DATUM  
 LÍNEAS DE CORRIENTE / FLOW LINES  $\rightarrow$  VELOCIDAD EN m/seg. / VELOCITY IN m/s

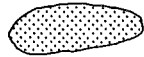
- |                 |             |   |                       |
|-----------------|-------------|---|-----------------------|
| CURVAS ISOBATAS | — 0 m       | ● | PLAYA, SOBRE EL DATUM |
| DEPTH CONTOURS  | - - - 1.5 m | ○ | DRY, ABOVE DATUM      |
|                 | - - - 2.5 m |   |                       |
|                 | - - - 5 m   |   |                       |

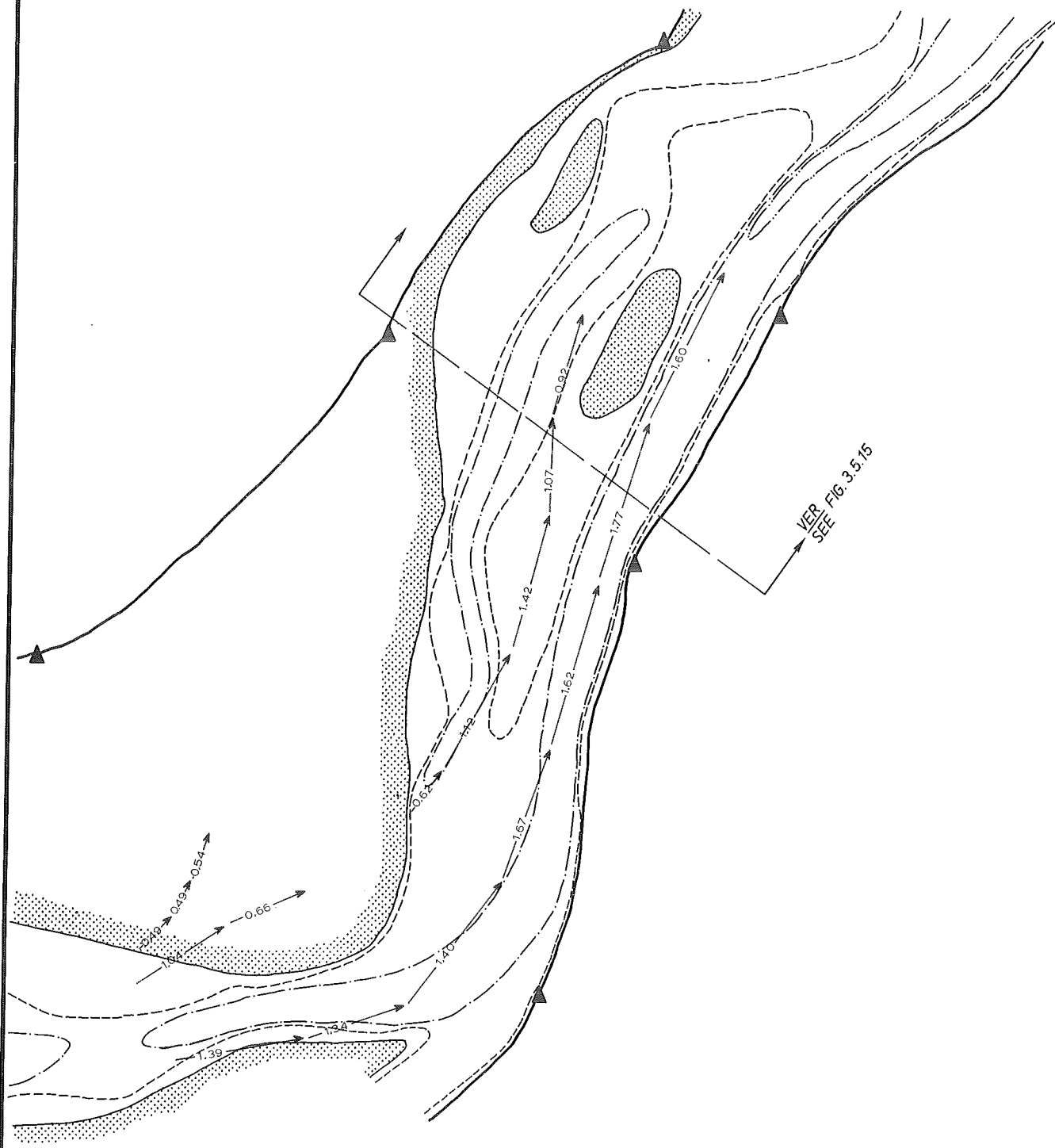


SONDEADO } 21.22-II-1972  
 SURVEYED } 21.22-II-1972

SONDEO / SOUNDING **RÍO MAGDALENA** km 616 - km 614 **AGUAS ARRIBA DEL RÍO SOGAMOSO**  
 UPSTREAM OF

ESCALA/SCALE 1:10,000  
 FECHA/DATE 15-II-1972  
 NIVEL DE REDUCCIÓN: 0.70 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. WILCHES  
 CHART DATUM: 0.70 metres ABOVE ZERO OF THE PTO. WILCHES GAUGE  
 FECHA/DATE: 10-II-1972 NIVEL DE AGUA / WATERLEVEL: 0.90 m SOBRE EL DATUM / ABOVE DATUM  
 LÍNEAS DE CORRIENTE / FLOW LINES —1.18— VELOCIDAD EN m/seg. / VELOCITY IN m/s

- |                 |           |       |   |                       |
|-----------------|-----------|-------|---|-----------------------|
|                 | —————     | 0 m   |   |                       |
| CURVAS ISOBATAS | - - - - - | 1.5 m |  | PLAYA, SOBRE EL DATUM |
| DEPTH CONTOURS  | - · - · - | 2.5 m |   | DRY, ABOVE DATUM      |
|                 | —————     | 5 m   |   |                       |



SONDEO  
SOUNDING

**RÍO MAGDALENA** km 616 - km 614

AGUAS ARRIBA DEL  
UPSTREAM OF

**RÍO SOGAMOSO**

ESCALA/SCALE 1:10,000

FECHA/DATE 29-II-1972. 1.2.3-III-1972

NIVEL DE REDUCCIÓN: 0.70 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. WILCHES

CHART DATUM: 0.70 metres ABOVE ZERO OF THE PTO. WILCHES GAUGE

FECHA/DATE: 3-III-1972 NIVEL DE AGUA / WATERLEVEL: 1.10m SOBRE EL DATUM / ABOVE DATUM

LÍNEAS DE CORRIENTE / FLOW LINES — 1.18 —> VELOCIDAD EN m/seg. / VELOCITY IN m/s

CURVAS ISOBATAS  
DEPTH CONTOURS

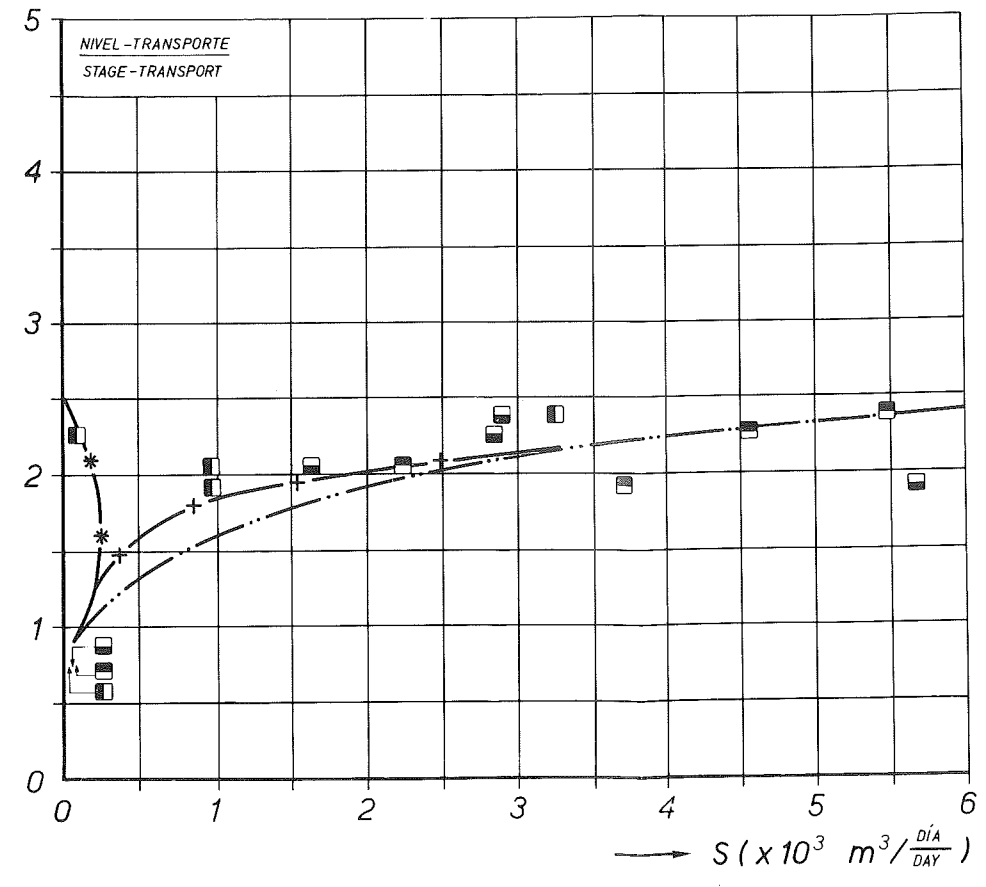
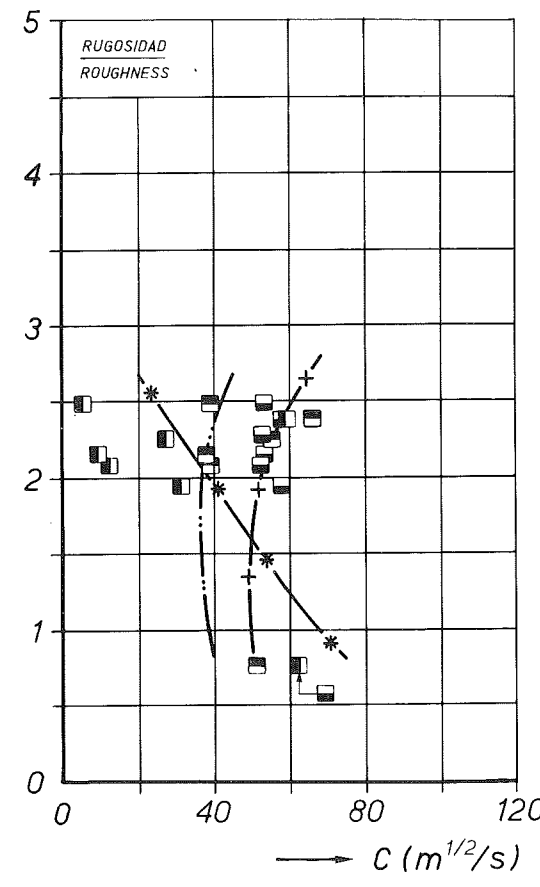
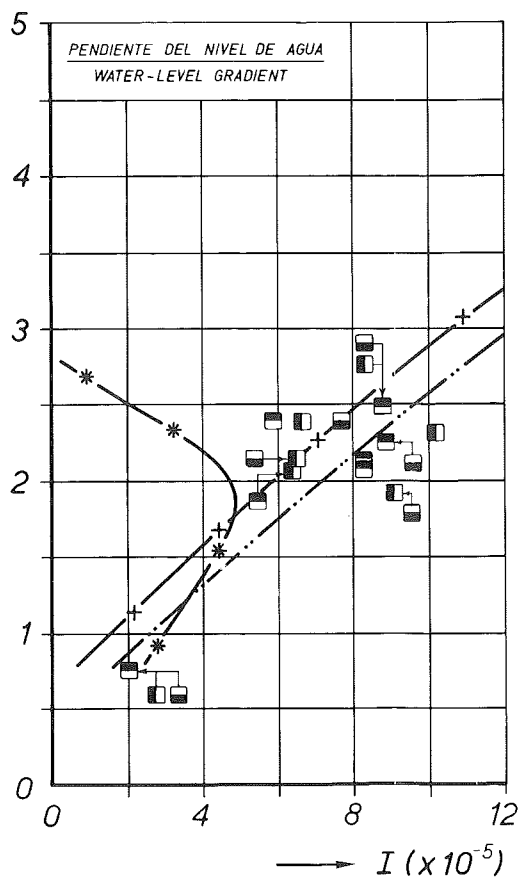
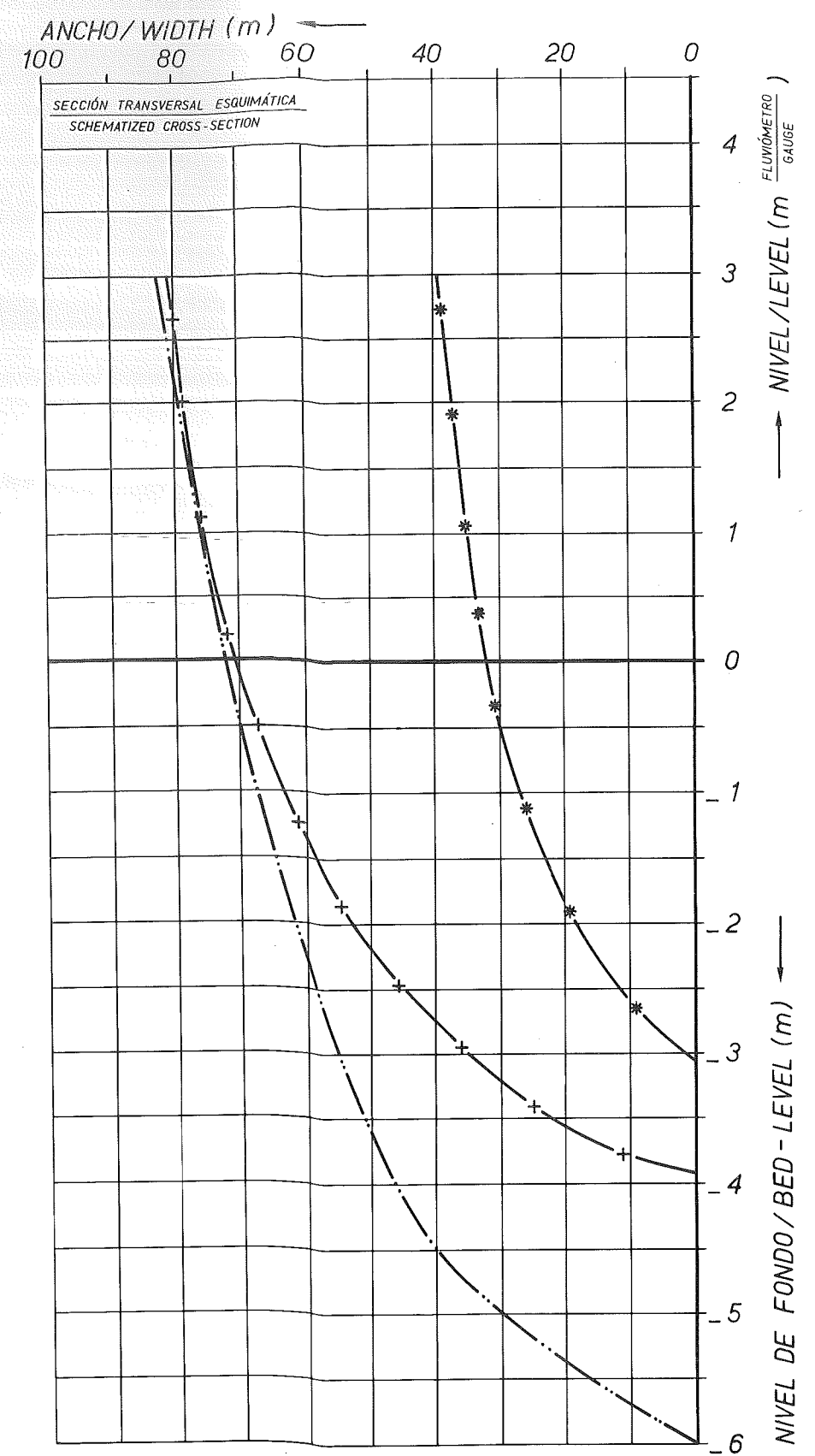
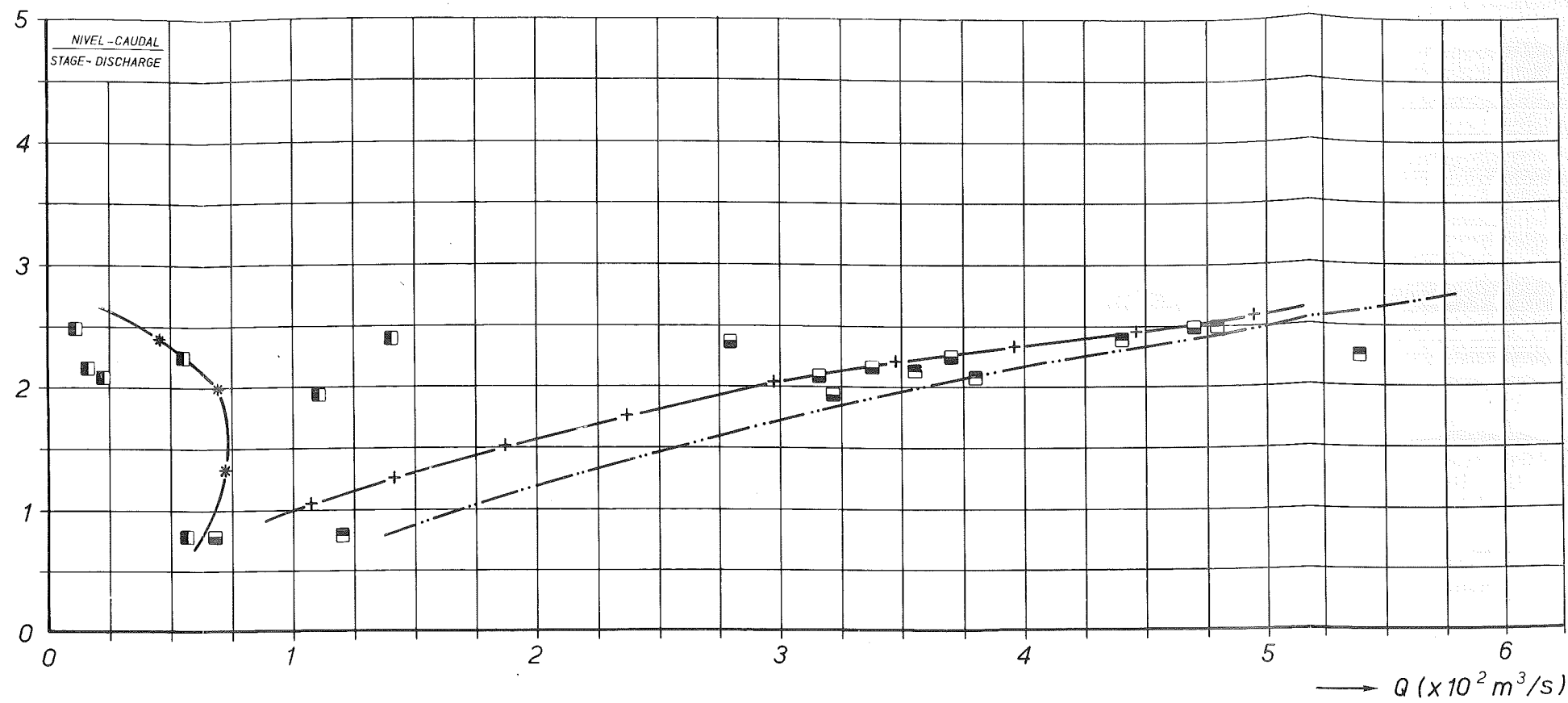
—————	0 m
- - - - -	1.5 m
—————	2.5 m
—————	5 m



PLAYA, SOBRE EL DATUM  
DRY, ABOVE DATUM



M J



**S / BED-MATERIAL DATA**

$\bar{D}_{50} = 150 \mu m$	$\bar{D}_{65} = 170 \mu m$
$\bar{D}_{50} = 100 \mu m$	$\bar{D}_{65} = 115 \mu m$
$\bar{D}_{50} = 120 \mu m$	$\bar{D}_{65} = 130 \mu m$

**DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA**

PROMEDIO / MEAN	—*— AGUAS ARRIBA / UPSTREAM km 82
	—*— CAÑO CORREA
	—+— AGUAS ABAJO / DOWNSTREAM km 83

**DATOS AFOROS / DISCHARGE DATA**

MITCH 1.971-1.972

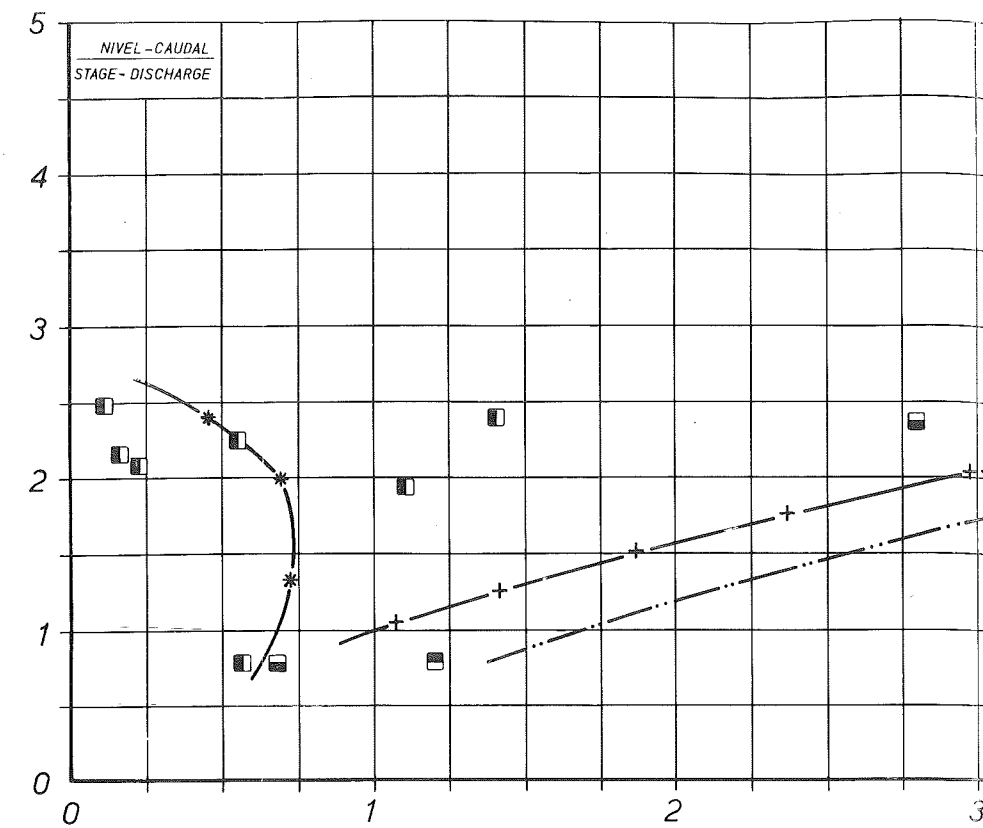
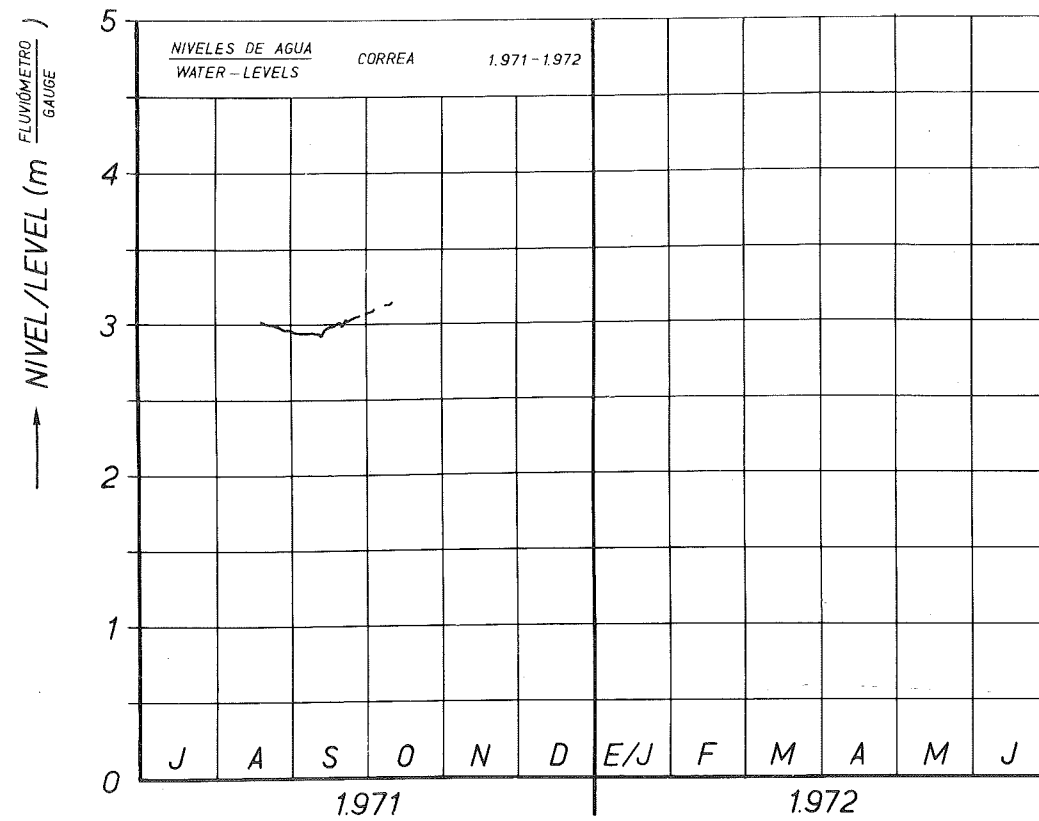
□	—*— AGUAS ARRIBA / UPSTREAM
□	—*— CAÑO CORREA
□	—+— AGUAS ABAJO / DOWNSTREAM

**BIFURCACIÓN / BIFURCATION** CAÑO CORREA CANAL DEL DIQUE km 82.5

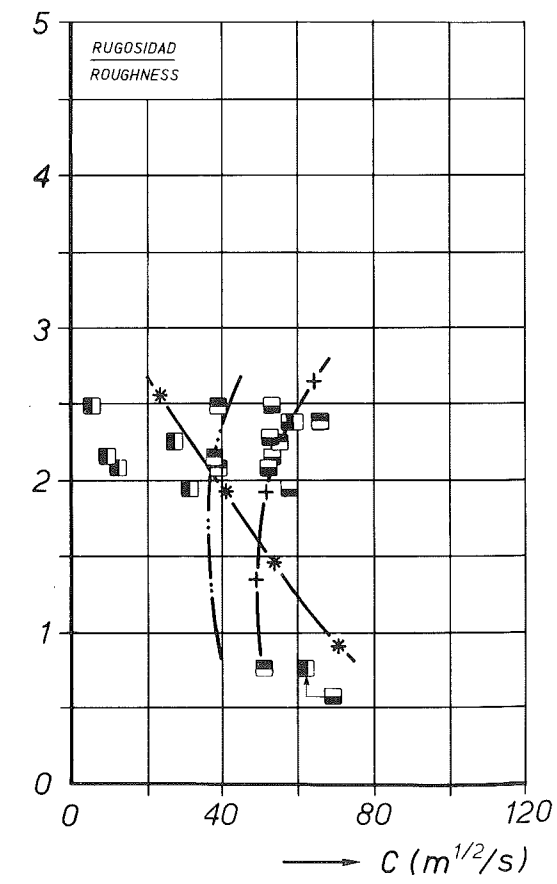
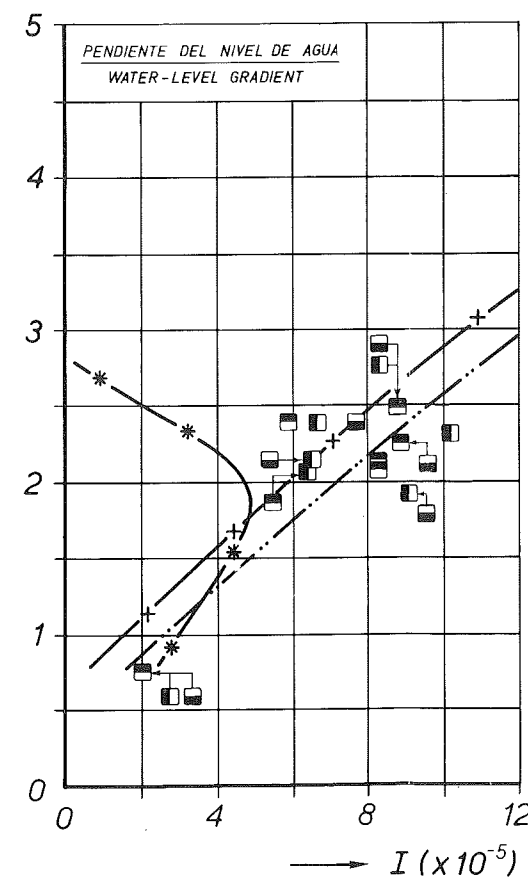
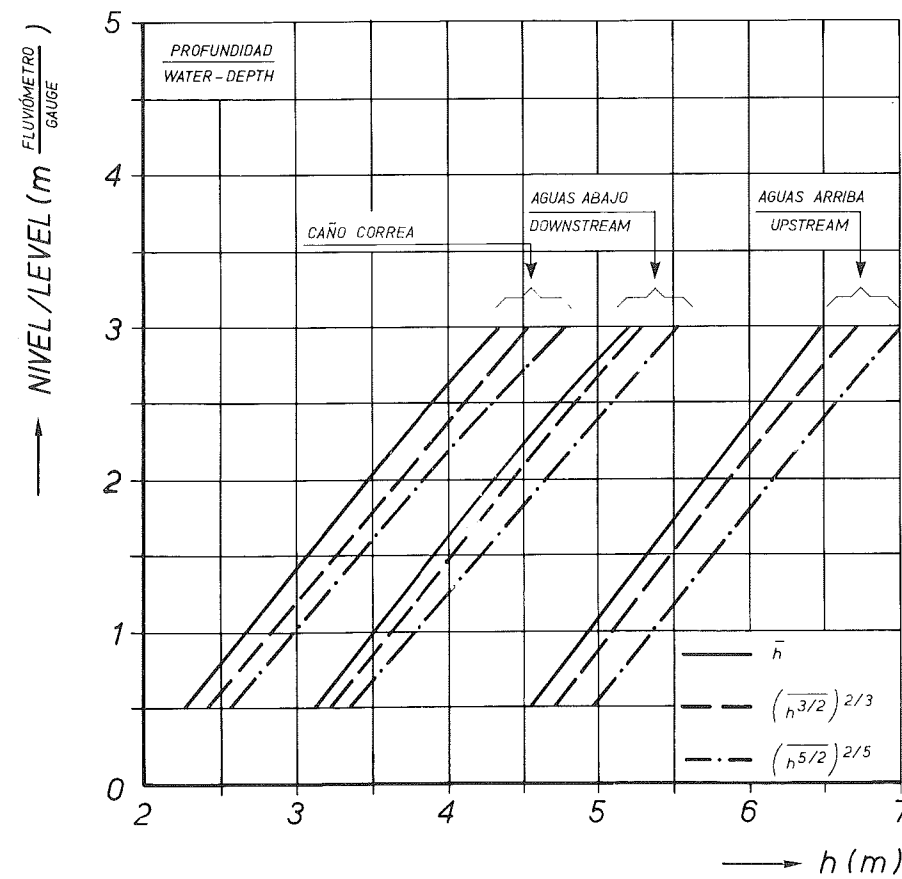
DATOS DE AFOROS Y TRANSPORTE DE ARENAS / DISCHARGE AND SEDIMENT-TRANSPORT DATA

NEDECO RÍO MAGDALENA-CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.5.15



CERO FLUVIÓMETRO  
ZERO GAUGE CAÑO CORREA -0.62 m SNM  
MSL

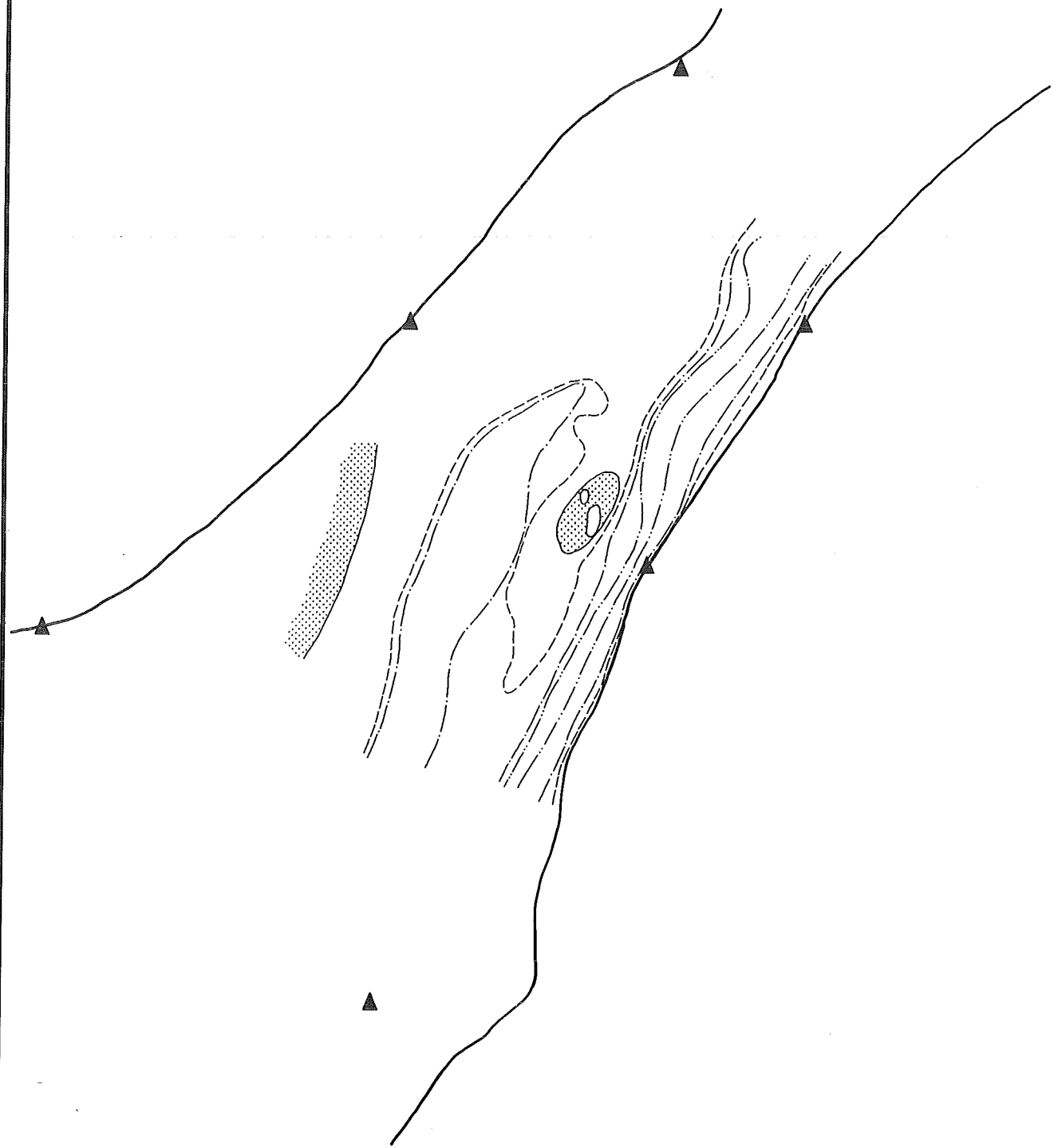


DATOS GRANULOMÉTRICOS / BED-MATERIAL DATA

ARRIBA / UPSTREAM  $\bar{D}_{35} = 140 \mu m$ ;  $\bar{D}_{50} = 150 \mu m$ ;  $\bar{D}_{65} = 170 \mu m$   
 CAÑO CORREA  $\bar{D}_{35} = 90 \mu m$ ;  $\bar{D}_{50} = 100 \mu m$ ;  $\bar{D}_{65} = 115 \mu m$   
 ABAJO / DOWNSTREAM  $\bar{D}_{35} = 105 \mu m$ ;  $\bar{D}_{50} = 120 \mu m$ ;  $\bar{D}_{65} = 130 \mu m$

DATOS SECCIONES TRANSVERSALES / CROSS-SECTION DATA

PROMEDIO / MEAN { —·—·— AGUAS ARRIBA / UPSTREAM km 82  
 \* — CAÑO CORREA  
 + — AGUAS ABAJO / DOWNSTREAM km 83



SONDEO  
SOUNDING

**RÍO MAGDALENA** km 616 - km 614

AGUAS ARRIBA DEL  
UPSTREAM OF

**RÍO SOGAMOSO**

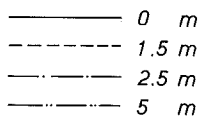
ESCALA/SCALE 1:10.000

FECHA/DATE 2-II-1.972

NIVEL DE REDUCCIÓN: 0.70 metros SOBRE EL CERO DEL FLUVIÓMETRO DE PTO. WILCHES

CHART DATUM: 0.70 metres ABOVE ZERO OF THE PTO. WILCHES GAUGE

CURVAS ISOBATAS  
DEPTH CONTOURS



PLAYA, SOBRE EL DATUM  
DRY, ABOVE DATUM

NEDECO RÍO MAGDALENA - CANAL DEL DIQUE SURVEY PROJECT

FIG. 3.5.16

### III, 3.5

The unpredictability of the reaction on (even temporary) river-works in this section, is mainly caused by the influence of the direction of the current entering the improved section. This direction depends on developments further upstream, and the inflow at the upstream end should therefore be somewhat more stable. This can be obtained by a bank protection along the eroding left bank between km 620 and km 616. Some protection may, however, also be required along the right bank near km 614. Some of the secondary channels along the right bank (km 615) may then be closed. Thereafter, it is considered advisable that developments in this section be studied, before any new river-works are carried out. In the meantime, dredging will be necessary to ensure sufficient depth for navigation.

#### Dredging

One of the ideas on which the present Report is based, is that it will be possible to maintain a good navigation channel by means of the recurrent dredging of crossings. Whether this is indeed possible, can only be affirmed by means of test dredging, the purpose of which should be in the first place, to study the stability of a number of properly aligned dredge cuts. NEDECO advised such test dredging to be carried out in the months January and February 1972, and a trial has been made with the dredger DH 10.

Although much experience has been gathered about dredging in general and the available dredgers in Colombia, the 1972 test dredging must be considered a failure, mainly due to organizational faults. Fortunately, the Colombian Government decided to carry out another test dredging in the months June to August, 1973, with a more suitable dredger (DH 6). If it will appear during this test dredging, that a stable dredge cut cannot be made, the present Report would have to be re-evaluated. Some faith in the stability of such a dredge cut can, however, be had, because the channel chosen to be dredged in 1972, scoured after a month by the river itself and was maintained until the next high water period (April - May, 1972), whereafter the channel silted up completely.

In Figure 3.5.8 a sounding made in August 1971, was given of the area near to the Rfo Sogamoso Confluence. This sounding was used to obtain a first impression of the best place to carry out the test dredging. At that time the worst place for navigation was found just in front of the mouth of the Rfo Sogamoso, where the least available depth was less than 5' below L.R.L. The navigation did not have many problems at that time, because the water-levels were well above L.R.L.

More characteristic as a crossing is the one at km 610 (indicated by B in Figure 3.5.8), which, according to the sounding, did offer sufficient depth. At the same time (August 1971), a crossing further upstream (km 614) presented some problems, but no soundings were made in that area.

It was thought that two or three of the crossings would be suitable for test dredging. As may be seen, a rather deep channel existed along the left bank, separated from the right-hand channel by a large island. The entrance to the left-hand channel (indicated by C in Figure 3.5.8), was blocked (depths less than 3' below L.R.L. were

observed). It was considered, whether the left-hand channel could be opened by dredging this entrance, or the other crossings be dredged and this left-hand channel be closed, because it was drawing with appreciable velocities a relatively large amount of water (see the flow-lines of Figure 3.5.8).

During a reconnaissance survey, made in December 1971, it appeared that the area between km 614 and km 616 again offered the worst navigation conditions of this region. A sounding of that area was therefore made, which is given in Figure 3.5.9. Not only was the least available depth less than 5' below L.R.L., but the channel appeared to be rather unstable. In January 1972, both regions were therefore again sounded (Figures 3.5.10 and 3.5.11).

Comparing Figures 3.5.8 and 3.5.10, it can be seen that the three crossings, indicated by A, B and C in Figure 3.5.8, deteriorated. In front of the mouth of the Río Sogamoso, a large sand-bank developed. Near km 615 the situation deteriorated even more (compare Figures 3.5.9 and 3.5.11), and only a 30 m wide and rather curved channel of about 5' below L.R.L. remained. It was decided, that in view of this bad situation, this crossing would be dredged first. In Figure 3.5.11 it can be seen, that the current concentrated on the crossing, and good results of the dredging were therefore expected if the flow-lines could be followed properly.

The designed dredge-cut is shown in Figure 3.5.11. Although the dredger arrived in Barrancabermeja on January 21, 1972, for several reasons (organization, lack of auxiliary equipment, etc.), the first attempt to dredge at km 615 was not made before February 10, when the dredger worked for only 2 hours. Between February 10 and 26, the dredger worked in total about 10 hours, and no dredge cut could therefore be made. During and after the dredging soundings were made and flow-lines measured locally, to study the development of the channel to be dredged. In the sounding of February 15 (Figure 3.5.12), the channel was practically open and further dredging served no purpose. The fact that the channel opened, may be a reason for some optimism regarding the possibilities of dredging crossings.

In Figure 3.5.13 a very slight shifting of the channel appears, but principally there is not much difference. In Figures 3.5.12 and 3.5.13, however, it can be seen that the flow-lines are less concentrated, corresponding to an irregular and slowly rising water-level.

A final sounding of the crossing was made in May 1972, at the end of the high water season (Figure 3.5.14). The situation had then changed drastically, with the main navigation route following the left bank. This meant that dredging would have had to be repeated in June 1972. Although the flow-lines taken in May 1972 (Figure 3.5.14) had not been extended sufficiently, also a separation of the flow-lines seems apparent, with a large quantity of water flowing along the left bank.

At the end of February 1972, also the complete downstream area was again sounded (Figure 3.5.15), and this should be compared with that of Figure 3.5.10. As may be seen, the most striking difference is the shifting of the crossing at km 610. The flow-lines have shifted slightly at km 610 and are more concentrated.

The development of the crossing at km 614, which is even worse than that at km 616, is also interesting. The crossing was, however, not used by shipping at that time. Comparing the Figures 3.5.9, 3.5.11, 3.5.12, 3.5.13, 3.5.14 and 3.5.16 it may be seen that this crossing first moved somewhat upstream and later on downstream again, always having depths of less than 5' below L.R.L. In May 1972 (Figure 3.5.14), when the channel had shifted to the left bank, this crossing had become the limiting one for navigation. This appeared more strongly during a reconnaissance in July 1972, than indicated in the figure. At that time, the depth at this crossing was less than 3', but because it was a very short one (about 50 m), it could have been dredged easily in one day, while at that time 4 or 5 barge trains were waiting to pass.

#### 3.5.6. The access to Pto. Wilches

Up till 1961, Pto. Wilches handled more than 150,000 tons of cargo annually. As from that year, this amount diminished first rapidly to about 50,000 tons annually, which was maintained till 1966, but then again fell until at present the amount of cargo is negligible. The rapid diminishing of the cargo-handling from 150,000 to about 50,000 tons can be attributed to the opening of the railway "Atlantico", although it is not clear (and outside the scope of this Report), why the total amount of cargo handled in 1966 was still some 60,000 tons, while in 1967 this had fallen to 14,000 tons. During the last few years the accessibility of the port has been blocked by a large sand-bank, which may be the cause of this reduction in cargo-handling. A few kilometers downstream, however, the channel again follows the right bank, and small amounts of cargo could be handled there during low water, if it was taken there by truck.

In view of the small amounts of cargo handled at Pto. Wilches, a solution may require only small investment. Dredging of a channel towards the quays may be considered. The main pattern of the river course between the Río Sogamoso Confluence and Pto. Wilches consists of a number of rather regular consecutive bends, one of which is at present situated just opposite Pto. Wilches, leaving a sand-bank right in front of the port. As long as the river upstream of Pto. Wilches is not fixed, these bends will not be stable but will move. The present situation with a sand-bank in front of Pto. Wilches, may be followed again by a situation in which the main channel follows the right bank. Permanent local solutions are, therefore, not possible, unless combined with a fixation of the river between the Río Sogamoso Confluence and Pto. Wilches. If in future, it is considered to fix this river section, this will undoubtedly be done to improve the main traffic flow from Barrancabermeja to the Caribbean Coast, and not for the cargo handled at Pto. Wilches, but it is recommended that at that time the interests of Pto. Wilches be kept in mind. The fixation should be done in such a way, that the crossing of the current from the left bank to the right bank will not occur upstream of Pto. Wilches, and in a slightly different direction than at present. A schematic indication of the river alignment required, is given in Figure 3.5.6.

For the time being the port can be kept open, if necessary, by the recurrent dredging of a channel (during low water) as indicated in Figure 3.5.6. As soon as during higher water-levels the sand-bank in front of Pto. Wilches is flooded, probably this access channel will also silt up again. The amount to be dredged annually, will be in the order of 25,000 m<sup>3</sup>.

## Chapter 4

### CANAL DEL DIQUE

#### 4.1. GENERAL

The Canal del Dique (Figure 4.1.1) was dug in 1650 by connecting up a number of cienagas which must have been formed along an old Río Magdalena branch. Although the Canal was made artificially, it may be treated as a river in view of the fact that it carries water and sediment from Calamar to the sea and is free to change its course when there is no human intervention.

Three problems may be distinguished:

- The Canal cross-section, its navigation requirements, local sedimentation, etc.;
- sedimentation at the entrance of the Canal (Calamar); and
- sedimentation near the mouth of the Canal (Pasacaballos).

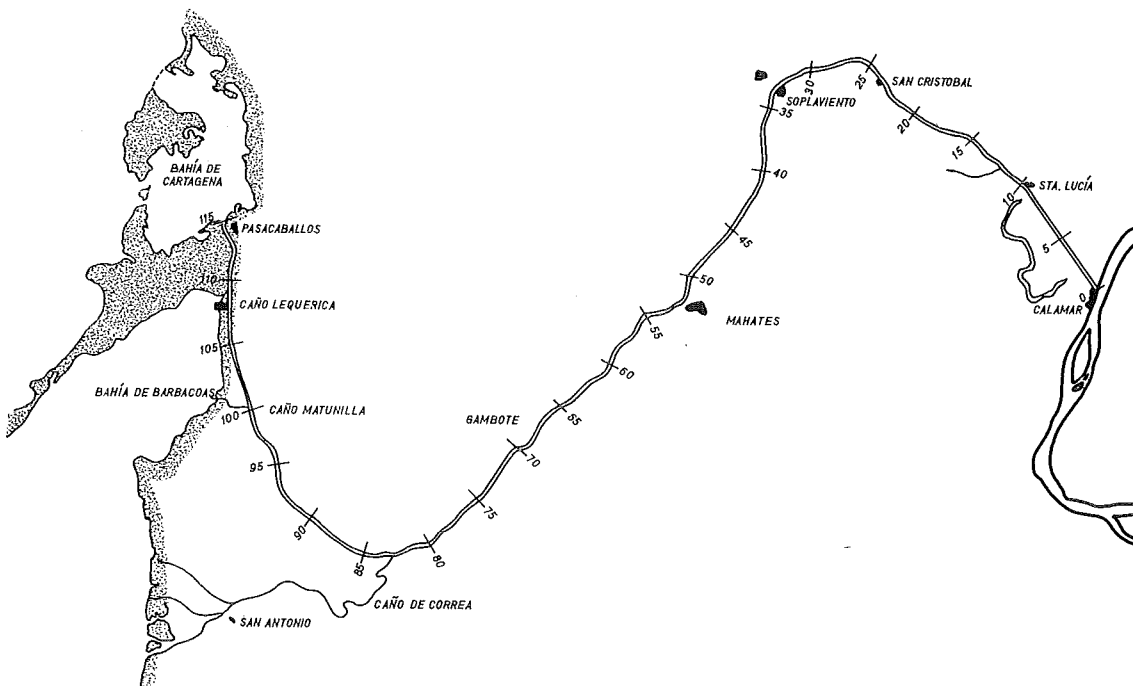


Figure 4.1.1 Canal del Dique

The problem of the Canal cross-section is dealt with in Para. 4.2. When the design cross-section of Mantilla is compared with the present navigation requirements, it is seen that this cross-section is too small. However, in reality the available cross-section as maintained by ADENAVI is often larger and will prove sufficient at many places, although there are also sections which require an increase of the available depth and width. In this respect it should be realized that although the cross-section should be sufficiently large

### III, 4.1

for navigation purposes, it should also be as small as possible because a Canal with a larger cross-section will draw more water and sediment, thus increasing the sedimentation near Calamar and in the lower Canal del Dique.

The sedimentation at Calamar is treated in Para. 4.3. This sedimentation is due to the fact that the sediment-load carried to the Canal is much larger than the transport capacity. The model study carried out by CETIH (Centro de Estudios Técnicos e Investigaciones Hidráulicas of the Universidad de los Andes in Bogotá) indicated a solution by means of a submerged groyne. At the request of MITCH an extension of this model investigation was carried out to study the efficiency of a dredged sand-trap in the mouth of the Canal in order to concentrate the dredging at such a distance from the Río Magdalena that the spoil could easily be pumped back into the Río Magdalena. It appeared from the model study that such a sand-trap ensured that a large part of the required dredging could indeed take place in the very upstream end of the Canal del Dique.

The sedimentation in the lower Canal del Dique is dealt with in Para. 4.4, in which it is shown that the maintenance of the Caños Matunilla and Lequerica keeps the amount of sedimentation near Pasacaballos as low as possible. A solution by closing the Canal del Dique below the Caño Matunilla will have adverse effects, as the amount of dredging would increase considerably, while all dredging would have to be carried out in the shipping channel.

It must be pointed out that all levels used in this Chapter are related to the Mantilla net, to which gauges have been levelled as far as possible. An exception was made for levels near Calamar which have been related to the zero of the gauge at Calamar to make comparison with ADENAVI and CETIH data easier. According to the levelling done by the Mission, the zero-level of this gauge lies 0.27 m above the zero of the net of Mantilla (zero of the gauge is -0.35 m in relation to the net of IGAC; see also Part II, Para. 2.3.4). This means that L.R.L., which has been defined as 2.13 m on the Calamar gauge, is 2.40 m in relation to the net of Mantilla (and 1.78 m in relation to the net of IGAC). See Table 4.1.1.

Description	Reference-Levels				
	MITCH (zero-level 0 m)	ADENAVI/CETIH (zero-level 100 m)	JUNTA del Canal del Dique (zero-level 0 m)	MANTILLA/DICON (zero-level 100 m)	IGAC (zero-level 0 m)
Zero-level gauge Calamar	0	100	0.27	100.27	- 0.35
L.R.L. Calamar	2.13	102.13	2.40	102.40	1.78
L.R.L. Sta. Lucía	1.95		1.95	101.95	1.33
L.R.L. Soplaviento	1.44		1.44	101.44	0.82
L.R.L. Gambote	0.70		0.70	100.70	0.08
L.R.L. Correa	0.51		0.51	100.51	- 0.11
L.R.L. Matunilla	0.30		0.30	100.30	- 0.32
L.R.L. Lequerica	0.21		0.21	100.21	- 0.41
L.R.L. Bahía de Barbacoas	0.12		0.12	100.12	- 0.50
L.R.L. Bahía de Cartagena	0.12		0.12	100.12	- 0.50

Remark: For the zero-level of the gauges along the Canal del Dique, reference should be made to Part II, Tables 2.3.1 and 2.3.9.

Table 4.1.1 Comparison of Reference-levels used along the Canal del Dique

4.2. THE CANAL4.2.1. Present dimensions of the Canal

The main requirement for the Canal is that its dimensions are sufficient for navigation by the ships at present in use. With these dimensions every effort must be made to keep sedimentation at a minimum, and those sedimentations still occurring will have to be dredged. In a river with a constant cross-section (fixed banks) and a constant discharge no sedimentation occurs, as the transport capacity is equal in each section. Along the Canal del Dique, therefore, problems occur at those places where the cross-sectional area increases and where water is lost into cienagas or distributaries (the Caños Correa, Matunilla and Lequerica).

Of course, it would be possible to reduce the cross-section downstream at those places where water is lost into distributaries but, as is shown further on, the cross-section is already small for navigation purposes. Therefore the acceptance of a certain amount of sedimentation is unavoidable.

The design dimensions for the Canal in its present form have been given by Mantilla (1951) [ 7 ] and slightly modified by ADENAVI (Table 4.2.1).

	Mantilla	ADENAVI
Length (km)	114.5	113.7
Number of bends	93	68
Width at bed-level (m)	45	45
Side slope	2 : 1	2 : 1
Depth (m)	2.40	2.40
Minimum radius of bends (m)	600	996
Length of tangents between bends (m)	150	260

Table 4.2.1 Design Dimensions of the Canal del Dique

These dimensions have no hydraulic background and are based solely on navigation requirements. The depth is given in relation to a "minimum" water-level as given by Mantilla. As actually a water-level should be related to a frequency or a duration (because a real minimum water-level does not exist), the Mission has assumed a Low River Level (L.R.L.) which is surpassed on the average during 95% of the time. In part of the Canal this level lies somewhat higher than the minimum level of Mantilla, and in that part the depth is consequently somewhat more than 2.40 m below L.R.L. The bed-level according to Mantilla in Calamar, for instance, should be on 99.10 m, which results in a depth below L.R.L. (= 102.40) of 3.30 m. In the lower part of the Canal, however, only a depth of about 2.20 m below L.R.L. is available. On the average, the available depth along the Canal will be about 2.60 m below L.R.L., which is just sufficient as is shown in Para. 4.2.2. The bed-level as advised by the Mission (2.60 m below L.R.L.) has been given in Figure 4.2.1.

The present Canal dimensions differ from the ADENAVI design-dimensions. At some places greater depths are available while at other places the Canal is shallower, as may be seen in Figures 3.3.39 and 3.3.40, presented in Part II (In these figures several length-soundings are given which at some places differ from each other, due to discrepancies in

### III, 4.2

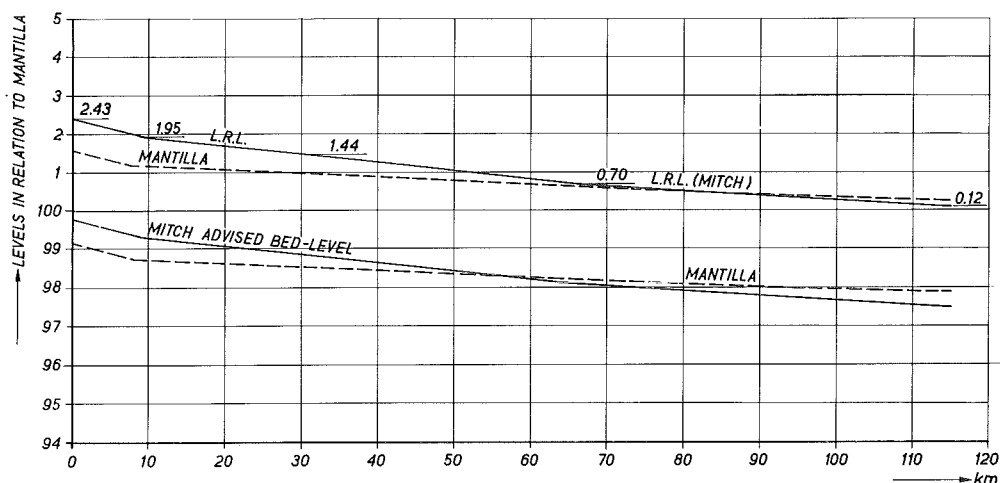


Figure 4.2.1 Advised Bed-level of the Canal del Dique

the water-levels and different courses followed when sounding). The amount to be dredged should not be found from the length profile only, as the cross-sectional profiles will also have to be studied. Many cross-sections are wider than the ADENAVI design-section and (partly) shallower. By comparing the required cross-section with the available sections, the amount to be dredged can be found.

In the following paragraph the available sections as well as the ADENAVI design-section are compared with the navigation requirements.

#### 4.2.2. Required dimensions (MITCH design-section)

To judge a navigation channel it is necessary to adopt a "design ship". Such a ship should be able to pass without too great reductions of speed or other difficulties during low levels (L.R.L.). The design ship should, therefore, not be an extremely large vessel which uses the channel infrequently but a ship of more or less average dimensions. The following dimensions used for such a ship (Figure 4.2.2) are those of commonly-used (large) ships on the Río Magdalena:

- Width 27 m,
- draught 1.60 m (5'2") (not fully loaded),
- length 160 m,
- speed 12 km/hr (3.3 m/s),
- net capacity about 3,500 tons.

As in future on the Río Magdalena an available depth will possibly be maintained which is sufficient for ships with a draught up to 7'6", it is thought that a ship with the same width and length as the design ship, but a draught of 7'6", should also be able to pass through the Canal. An appreciable speed reduction may, however, be accepted, especially as problems mainly arise for vessels going upstream (as will be indicated later), while much more cargo is transported in a downstream direction than in an upstream direction. It should be realized that it would, of course, be easy to design a very large canal which would not

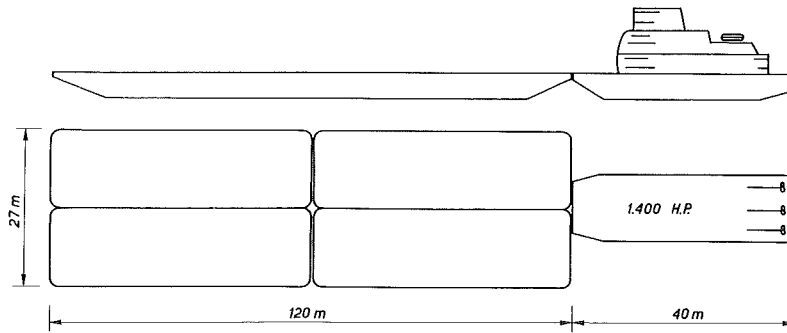


Figure 4.2.2 "Design Ship" for Required Dimensions of the Canal del Dique

give difficulties even for a ship with a draught of 7'6", but the larger cross-section would draw more water and sediment from the Río Magdalena and this would result in increased sedimentation near Calamar and Pasacaballos.

The remainder of this paragraph first checks the ADENAVI design-section (and proves it insufficient), and then gives a new design-section as advised by MITCH.

ADENAVI design-section

The passing of a ship through a (narrow) canal cross-section will cause return flow and a drop in the local water-level. Due to the return flow the ground speed is reduced, and due to the water-level drop the keel clearance is reduced. Both depend on the ratio  $f/F$  between the cross-section of the ship ( $f$ ) and that of the Canal ( $F$ ). The maximum speed in the Canal (independent of horse-power) and the water-level drop may be found from Figure 4.2.3, although of course the maximum speed cannot economically be maintained. Also a curve is given for an (economical) speed of 0.9 times the maximum speed.

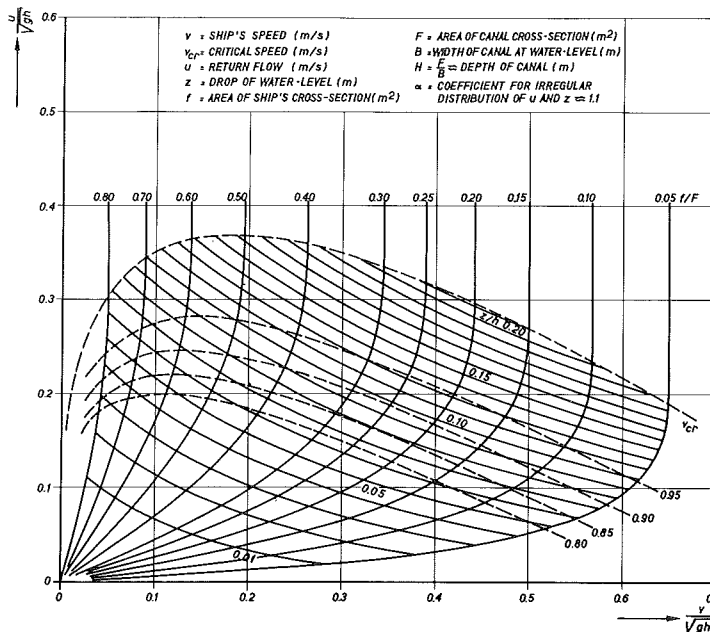


Figure 4.2.3 Economical Speed of Vessel and Drop in Water-level

### III, 4.2

For the design ship ( $f = 43 \text{ m}^2$ ) and the ADENAVI design-section ( $F = 120 \text{ m}^2$ ) the value of  $f/F = 0.36$ . From Figure 4.2.3 it can be read that the economical speed ( $0.9 \times$  maximum speed) is  $0.26 \times \sqrt{gh} = 0.26 \times \sqrt{9.81 \times 2.4} = 1.26 \text{ m/s}$  ( $= 4.5 \text{ km/hr}$ ). This means that in the Canal the speed through the water is reduced from  $12 \text{ km/hr}$  to  $4.5 \text{ km/hr}$ . As the flow velocity in the Canal, during low levels, is about  $0.6 \text{ m/s}$ , the maintained ground speed is about  $1.26 \text{ m/s} - 0.6 \text{ m/s} = 0.66 \text{ m/s}$  ( $2.4 \text{ km/hr}$ ), so that the time required to travel from Pasacaballos to Calamar would be 50 hrs. This speed reduction seems so excessive that it is not worth testing the canal also for the larger ship (7'6"). The ADENAVI design-section is too small and a larger cross-section should be maintained.

#### MITCH design-section

As already mentioned, many cross-sections are already larger than the ADENAVI design-section. The width at the bed is often 55-60 m and at L.R.L. the width at the water surface is nearly always more than 75 m. It will now be shown that a cross-section maintained with the above width and a depth of 2.60 m below L.R.L. will answer the navigation requirements.

The width is sufficient for the design ship to pass (1.3 times the width of the barge train is a minimum width of the bed). Manoeuvres when ships meet will be difficult and one ship will have to move alongside a bank. However, the number of ships passing per day makes this acceptable. One barge-train overtaking another should not be allowed.

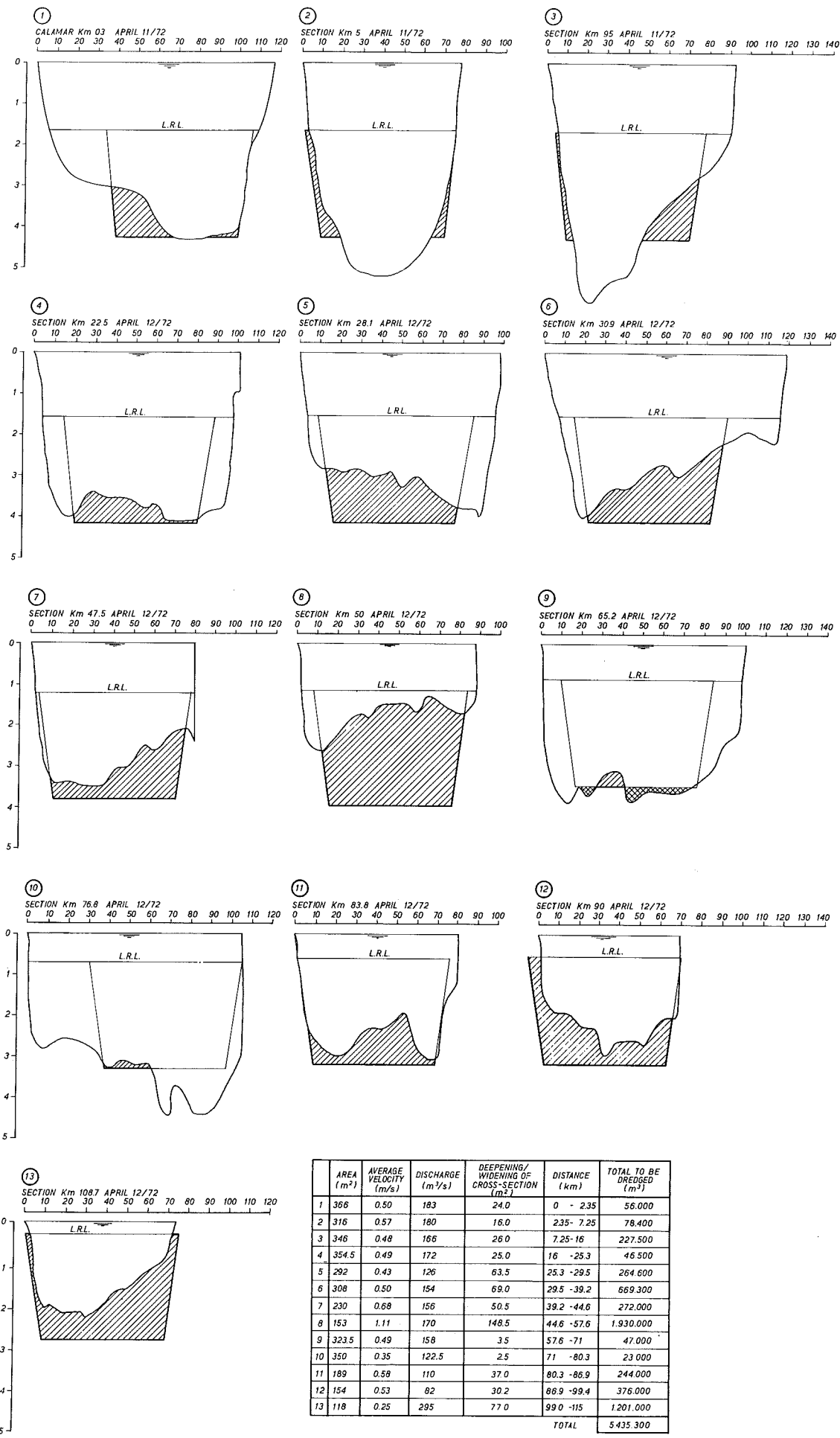
Description of vessel	f/F	Economical	Ground speed	Travelling time	Ground speed	Travelling time	Drop in	Keel clearance
		speed	upstream	Pasacaballos- Calamar	downstream	Calamar-Pasa Caballos	Water-level	(m)
		(km/hr)	(km/hr)	(hr)	(km/hr)	(hr)	(m)	(m)
"Design ship" (160x27x1.60 m)	0.25	6.5	4.3	28	8.6	14	0.25	0.75
Ship with 7'6" draught (160x27x2.28 m)	0.35	4.7	2.5	48	6.9	17	0.25	0.07
		4.0	1.8	60			0.09	0.23
Empty ship	0.063	10	7.8	15	12.1	10		

Table 4.2.2 Speeds and Drop in Water-level for Different Types of Vessel

In Table 4.2.2 speeds and drop in water-level for the "design ship", the larger (7'6") ship, and an empty ship have been collected. As may be seen, the "design ship" maintains a speed through the water of  $6.5 \text{ km/hr}$ , which is still an appreciable reduction of speed but acceptable in view of the fact that L.R.L. has an average duration of only 5% of the time. Also the ship with a 7'6" draught can pass, even though very slowly. In the last column the keel clearance is given.

A ship needs a greater water depth than the depth corresponding with its draught and the drop in water-level because:

- When moving, the after-ship lies deeper than the fore-ship;
- there may be some waves (in the Canal del Dique very small);
- some small bed-forms can be present, reducing the available depth; and
- after taking into account these three points a sufficiently large keel clearance is required for manoeuvrability. Van der Made, in his Report (1955) [57], showed that when the keel clearance is less than 30% of the draught, steering power is rapidly reduced.



	AREA (m <sup>2</sup> )	AVERAGE VELOCITY (m/s)	DISCHARGE (m <sup>3</sup> /s)	DEEPENING/WIDENING OF CROSS-SECTION (m <sup>2</sup> )	DISTANCE (km)	TOTAL TO BE DREDGED (m <sup>3</sup> )
1	366	0.50	183	24.0	0 - 2.35	56,000
2	316	0.57	180	16.0	2.35 - 7.25	78,400
3	346	0.48	166	26.0	7.25 - 16	227,500
4	354.5	0.49	172	25.0	16 - 25.3	46,500
5	292	0.43	126	63.5	25.3 - 29.5	264,600
6	308	0.50	154	69.0	29.5 - 39.2	669,300
7	230	0.68	156	50.5	39.2 - 44.6	272,000
8	153	1.11	170	148.5	44.6 - 57.6	1,930,000
9	323.5	0.49	158	3.5	57.6 - 71	47,000
10	350	0.35	122.5	2.5	71 - 80.3	23,000
11	189	0.58	110	37.0	80.3 - 86.9	244,000
12	154	0.53	82	30.2	86.9 - 99.4	376,000
13	118	0.25	295	77.0	99.0 - 115	1,201,000
TOTAL						5,435,300

MEASURED CROSS-SECTIONS COMPARED WITH "MITCH" DESIGN-SECTION

FIG. 4.2.4

### III, 4.2

For the design ship a sufficient keel clearance (to counteract the above points) of nearly 50% is available. The large ship (draught 7'6"), moving with the economical speed has only a keel clearance of 0.07 m, which is too small. Going upstream a ground speed of 0.5 m/s (= 1.8 km/hr) seems still acceptable and with that speed the keel clearance is 0.23 m. Admittedly this keel clearance is very small but it is sufficient. Such a ship will only very seldom go upstream with low discharge, as generally the cargo can be divided amongst other ships and, in any case, more cargo is brought downstream than is taken upstream.

From a comparison of the available cross-sections with the MITCH design-section, it follows that the amount to be dredged in the Canal del Dique is 5,435,000 m<sup>3</sup> (see Figure 4.2.4 and Figures 3.3.39 and 3.3.40, presented in Part II).

#### 4.2.3. Bends and bend-radius

Ships moving through a bend in a river or canal have a drift angle  $\beta$ , which depends on the speed and the bend-radius (Schäle 1969 [58] and Kuhn 1968 [59]) as indicated in Figures 4.2.5 and 4.2.6 (Rhine-Main-Danube Canal). Because of this drift angle, in bends a wider cross-section is required than in a straight section. As this widening, however, will result in sedimentation (which has to be dredged) the widening and the number of bends should be kept at a minimum. In Figure 4.2.5 it may be seen that if the ADENAVI radius of 996 m is used, no widening of the bend will be necessary.

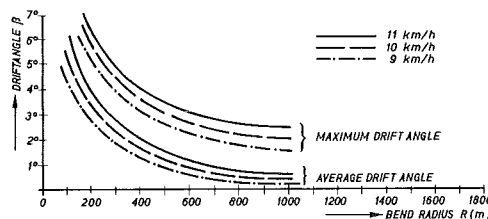


Figure 4.2.5 Drift Angle versus Bend-radius

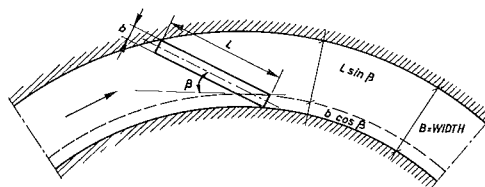


Figure 4.2.6 Widening in Bends

Some maintenance dredging in bends cannot be avoided, namely, the dredging of the sedimentation along the inner bend due to the helicoidal flow (see Part II, Para. 3.7). These amounts will be very small.

### 4.3. CALAMAR (CANAL DEL DIQUE BIFURCATION)

#### 4.3.1. Introduction

The Canal del Dique has been made by following an old Río Magdalena course. The river shifted its course to a new outlet at Barranquilla when the resistance along the old course increased and the Cartagena channel became sedimented. When (in 1650) the old branch was opened again artificially by connecting the remaining cienagas, this was not a stable situation and maintenance was required. Small pile-works constructed by "enemies" of the Canal were therefore sufficient to close it again [6]. In 1724 the Canal was closed completely, to be opened again in 1726, however, with the entrance shifted some miles (from the Caño de las Mulas to Barranca Nueva about 4 miles south of the present entrance).

Also at present the situation is not stable, and a large amount of water and sand enters the Canal. The sediment-transport capacity of the Canal is, however, much smaller than the amount of sediment entering into it. This difference causes sedimentation in the first kilometers of the Canal. It may therefore be concluded that the only practically complete solution of all the Canal del Dique problems would be to built a number of navigation locks, to prevent the entry of water and sediment. This would not only solve the problem near Calamar but also at Pasacaballos and the other bifurcations, although it would entail large expenditure. More is said about this solution in Para. 4.3.3.

If no locks are made and it is accepted that water and sediment enter the Canal, the possible solution may be divided into two parts:

- By reducing as much as possible the amount of bed-material load entering the Canal, and
- by ensuring that sedimentation takes place where removal is easiest (or cheapest).

#### 4.3.2. Sedimentation in the present situation

Sedimentation at the Canal del Dique entrance is caused by bed-material load, as the wash-load only settles in cienagas and the outlets of the Canal into the sea. On comparing measured transports and the computed transport capacity at Calamar (Figure 4.3.1), a large difference is found. By means of these two curves the average yearly transport and the average yearly capacity have been computed thus:

- Average yearly capacity (computed)  $0.23 \times 10^6 \text{m}^3$ , and
- average yearly transport (measured)  $1.26 \times 10^6 \text{m}^3$ .

The difference between the measured transport (the supply as measured at the entrance) and the transport capacity results in sedimentation in the Canal. As at the entrance only suspended-load is present, only a part will settle instantaneously near Calamar, the rest over the first ten to twenty kilometers. This is also indicated by Figure 4.3.2, which shows a slowly diminishing transport between km 0 and km 10. (The works at Sta. Lucía and San Christobal may have shifted the sedimentation to a somewhat more downstream section). It thus follows that annually an amount of about one million cubic meters would have to be dredged on the first 10 - 20 km to maintain the Canal, which seems a very high figure.

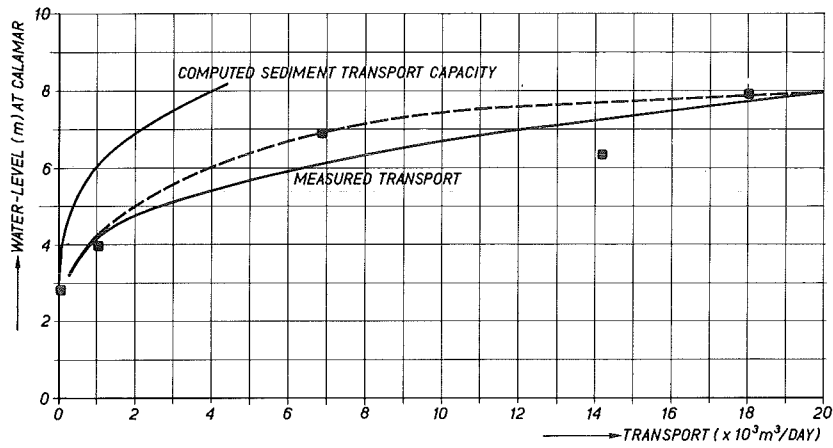


Figure 4.3.1 Computed Transport Capacity Compared with Measured Transport in the Canal del Dique near Calamar

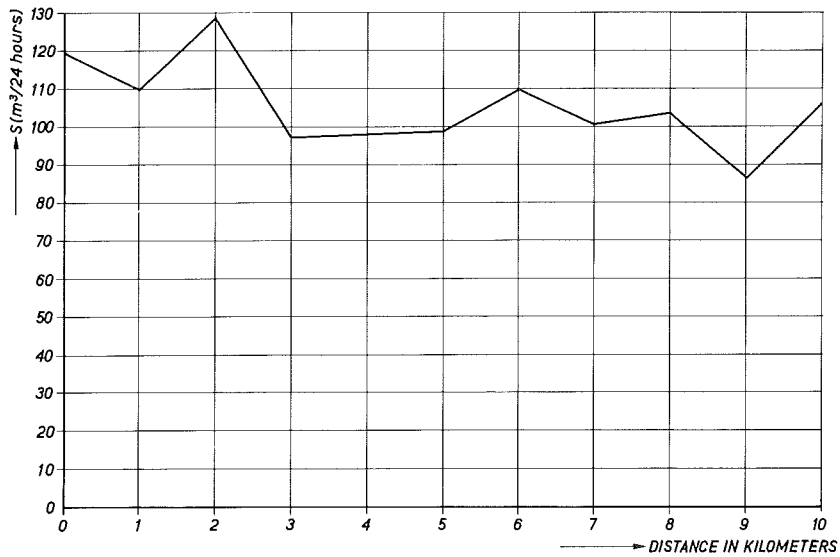


Figure 4.3.2 Measured Sediment Transport over the First 10 Km of the Canal del Dique

Reconsidering Figure 4.3.1 it can be seen that the result is strongly influenced by one measuring point at a level of about 6.20 m. Omitting this point would give a bit steeper relation for the measured transport and result in a considerable reduction of the amount annually to be dredged. It seems better, therefore, to estimate the amount at about 700,000  $\text{m}^3$  for the time being. Some additional measurements are advised.

4.3.3. Comparison of different solutions

The following solutions may be considered:

- i Navigation locks at Calamar and Pasacaballos.
- ii Reducing the amount of sediment entering the Canal by a submerged groyne at the entrance of the Canal del Dique.
- iii Ensuring sedimentation close to the entrance of the Canal, where the material can be dredged and pumped back to the Río Magdalena downstream of the Canal del Dique Entrance.
- iv A combination of ii and iii.

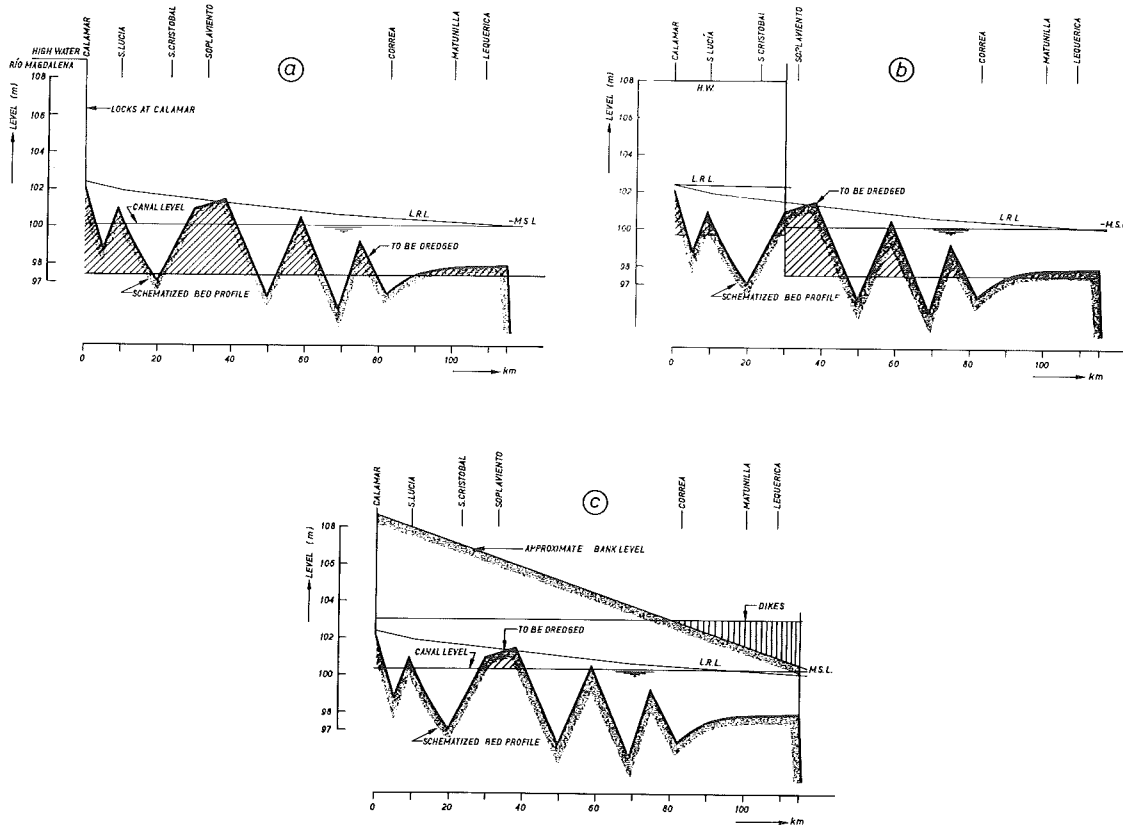


Figure 4.3.3 Solution by Means of Navigation Locks

Navigation locks (solution i)

By means of a sluice at the entrance of the Canal del Dique, practically all sedimentation in the canal and at the canal outlets into the sea would be prevented, as only a negligible amount of water (and sediment) would enter the canal through the sluices. A small amount of sedimentation would take place on the river side of the locks by means of secondary currents.

However, such a sluice would, in addition to a number of other advantages which will be mentioned further on, also have a number of disadvantages (see also Figure 4.3.3a). Some of these are:

- The canal water would become salt, and consequently the water could not be used for agriculture (as, e.g., near El Limón).

### III, 4.3

- The water-level in the canal would drop to about 7 m below the present high water-level at Calamar (equal to about 2 m below the present low water-level), which would have adverse effects on agriculture.
- Many cienagas might fall dry or become salt; the latter becoming stagnant and therefore dangerous to health.
- Fresh water fish would die.
- A very large amount of initial dredging would be required.

Some of these disadvantages could be prevented by constructing the locks some distance downstream of Calamar (e.g., km 30). The upstream part of the canal would then contain fresh water, making irrigation possible. The amount of initial dredging would be considerably less (Figure 4.3.3b).

Practically all the disadvantages mentioned can be avoided by building two locks, one at Calamar (or somewhat downstream of Calamar), and the other at Pasacaballos (Figure 4.3.3c). The amount of initial dredging would be very small, but some dikes would be required between km 80 and the Bahía de Cartagena, while the Caños Correa, Matunilla and Lequerica should be closed.

As a first approximation, the required dimension of the locks could be as follows:

- Length 200 m,
- width 29 m,
- bottom 2.60 m below canal-level (= 103 m) at Calamar and 2.60 m below L.L.W.S. at Pasacaballos,
- the gates at Calamar should reach up to a level of 9 m (at the Calamar gauge), and
- the gates at Pasacaballos should reach to 0.50 m above the canal-level (103.50 m).

An estimate of the annual cost of this latter project is in the order of 50 million pesos (including interest on invested capital, maintenance, etc., against about 15 million pesos annually for maintenance dredging of the canal.

Construction of the locks does not, therefore, seem economically justified from a navigation point of view. However, this is not the only thing to be taken into account, as also the following advantages should be considered:

- Two extra road traffic connections between the North and South banks of the canal would become available.
- Also in the far future, there would be no danger to the Mammonal seaway of being silted up by material brought down by the Canal del Dique.
- A possible pollution of the water and beaches near Cartagena by Río Magdalena water would be avoided.
- Control of the water-level in the canal would make gravity irrigation possible in the lower canal part (drainage could be directly into the sea, or through the Caños Correa, Matunilla and Lequerica). Further upstream, pump irrigation would be possible.

Further study of these aspects may result in a different economic evaluation, especially if, in view of an increase in river traffic, larger canal dimensions would be required (increasing the sedimentation if no locks were present). Before such a project was started, the following points (amongst others) would need further study:

Cienagas

Which cienagas will lose connections with the canal and what will happen to the water-level in the cienagas; which cienagas will become salt, and what will be the consequences?

Fishery

What will be the effect on the present fish stock? Can fishery be improved?

Secondary structures

What secondary structures will be required for irrigation and drainage of the land?

Sedimentation at Calamar

Can sedimentation in front of the sluice at the Rfo Magdalena side be diminished by introducing an eddy (see Figure 4.3.4)?

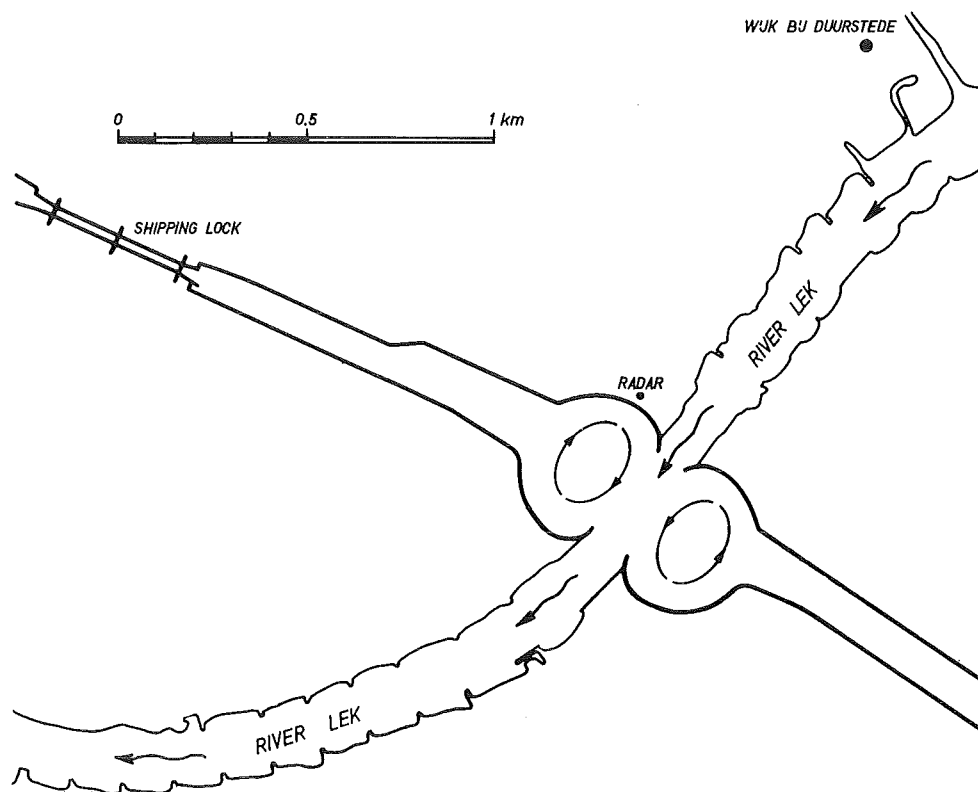


Figure 4.3.4 Induction of Secondary Currents [after Thijsse]

Solutions ii, iii and iv

These three solutions will be treated together. In short, they are all meant to reduce the amount of sediment entering the Canal and/or to localize sedimentation near the entrance to the Canal.

### III, 4.3

The Centro de Estudios Técnicos e Investigaciones Hydráulicas (CETIH) of the Universidad de los Andes in Bogotá carried out a model-study (1970) about the sedimentation at Calamar. At that time mainly the reduction of sedimentation was studied. CETIH [60] found that a submerged groyne (crest-level 99 m = 1 m below the zero-level of the gauge) would reduce the amount of sedimentation in the Canal considerably. The Mission submitted an Interim Report evaluating this solution and requested an extension of the investigation to study the efficiency of a dredged sand-trap, possibly in combination with a submerged groyne. In this Interim Report, amongst others, the following suggestions were given:

- The reconsideration of the crest-level of the dam in view of navigation; and
- that the constructional design of the dam be studied.

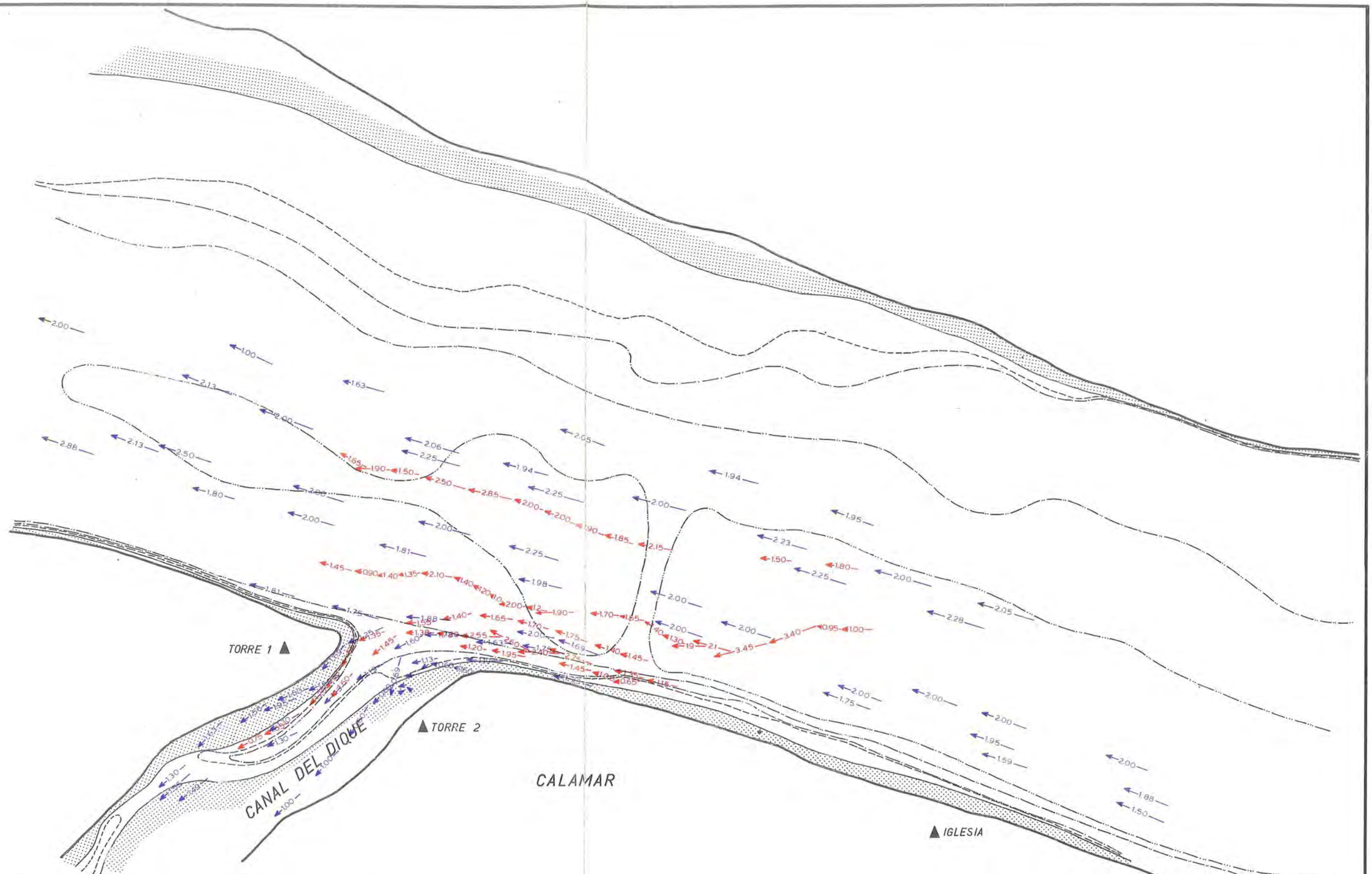
An extension of the model-study was carried out by CETIH in 1972 only in relation to the sand-trap. The model-study could be based on prototype-data which had become available during the navigation-study. CETIH studied several traps with lengths varying between 200 and 600 m, the longer trap catching about 300,000 m<sup>3</sup>. In an Interim Report [61] CETIH presented the following results:

- A sand-trap between km 0.2 and 0.6 with bed-level of 95 m (5 m below the zero-level of the gauge) and 50 m wide will catch the majority of the sediment during high water (200,000 m<sup>3</sup>).
- A submerged groyne will reduce the amount to be dredged considerably.

The following must be said about these recommendations. As already computed, the amount to be caught by the sand-trap should be about 700,000 m<sup>3</sup>. The longer sand-trap as located between km 0.24 and km 0.84 of the Canal del Dique, is therefore advised by the Mission. This sandtrap should be made as deep as possible with the available dredger. Nevertheless, such a sand-trap probably cannot contain 700,000 m<sup>3</sup>. The best would therefore be to dredge at Calamar twice yearly, once at the beginning of the dry season (as early as possible) to such a level that navigation is possible (level 100 m, corresponding to the zero-level of the gauge) and thereafter the actual sand-trap. The sand-trap could then be dredged again early May before the water has risen too much. Some sediment may still settle more downstream in the Canal del Dique itself and will have to be dredged there.

The effect of a submerged groyne is that the water cannot enter the Canal along the river bed, but only at the upper layers. This would mean that when there is a large amount of bed-load the amount of sediment entering the Canal would be considerably reduced. In the Río Magdalena there is predominantly suspended-load; a submerged groyne might then still work, because concentrations are generally larger in the lower part of the vertical. In Part II, however, it was pointed out that in practice, along the Río Magdalena, the concentration verticals differed from those found in theory (see Figures 3.3.7 and 3.3.8 of Part II), the concentrations in the upper part of the vertical being much larger than indicated by the value of  $v_*/w$ .

In model investigations with predominantly suspended-load one normally tries to obtain a value for  $v_*/w$  equal to that in the prototype. This was impossible for the Calamar model because of the fine bed material available in the prototype. As a result, the value in the model was only about half that in prototype, which has the effect that the sediment is



SONDEO / SOUNDING **RÍO MAGDALENA** CERCA LA BIFURCACIÓN DEL / NEAR BIFURCATION OF **CANAL DEL DIQUE**

ESCALA / SCALE: 1:5.000

FECHA / DATE: 17-VI-1971

NIVEL DE REDUCCIÓN: 2.13 metros SOBRE EL CERO DEL FLUVIÓMETRO DE CALAMAR

CHART DATUM: 2.13 metres ABOVE ZERO OF THE CALAMAR GAUGE

FECHA / DATE: 16-VI-71 / 3-IX-71 NIVEL DE AGUA / WATERLEVEL: 5.87m/4.80m SOBRE EL DATUM / ABOVE DATUM

LÍNEAS DE CORRIENTE / FLOW LINES — 1.18 —> VELOCIDAD EN m/seg. / VELOCITY IN m/s

CURVAS ISOBATAS	— 0 m	PLAYA, SOBRE EL DATUM DRY, ABOVE DATUM
DEPTH CONTOURS	- - - 1.5 m	
	— 2.5 m	
	— 5 m	
	— 10 m	



transported nearer to the bed with larger concentrations in the lower part of the vertical. In view of this, with the concentration verticals in the Rio Magdalena differing from theory and the scale for  $v_*/w$  not being unity, the results obtained from the model regarding the submerged groyne should be regarded with some caution. In fact, this is also valid, but in a much less degree, regarding the rate of sedimentation in the sand-trap. The solution with only a submerged groyne must, therefore, be abandoned although in combination with dredging, it still might be a solution.

If by the construction of the groyne the amount of dredging could be reduced by about 100,000 m<sup>3</sup>/year, the groyne seems economically justified. CETIH based a reduction on the fact that less sediment was caught in the sand-trap. This, however, is not necessarily due to less sediment entering the Canal, but might also be due to a lower efficiency of the sand-trap. As the capacity of the sand-trap is already small, the Mission feels that a submerged groyne should not be built, unless more definite information can be gathered. The more so because the soundings given in Figures 4.3.5 and 4.3.6 locally differ appreciably from the model results. In the prototype the advised groyne would be as indicated in Figure 4.3.7. This groyne is so low in relation to the bed-level that in prototype its effect will most probably be less than in the model. Construction of the groyne is, therefore, not advised.

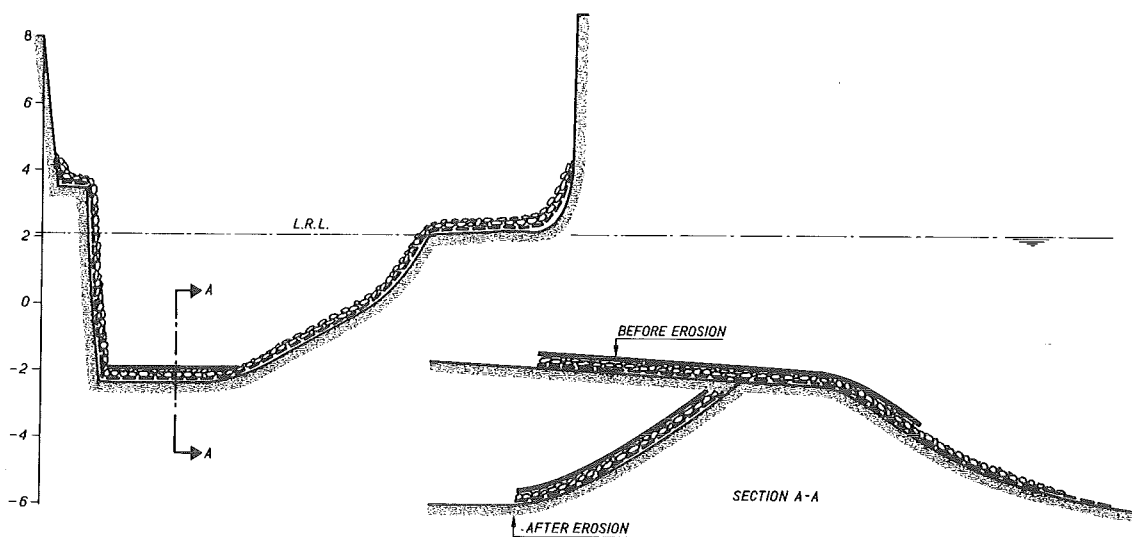


Figure 4.3.7 Dimensions of Submerged Groyne at Calamar

In view of the frequent dredging required, the use of a permanent landline may have advantages, but the cost of the pipeline should be compared with the cost of relaying the landline every year. In any case, sufficient landline should be laid (Figure 4.3.8) before the dredging commences (to reduce the waiting time of the dredger to a minimum).

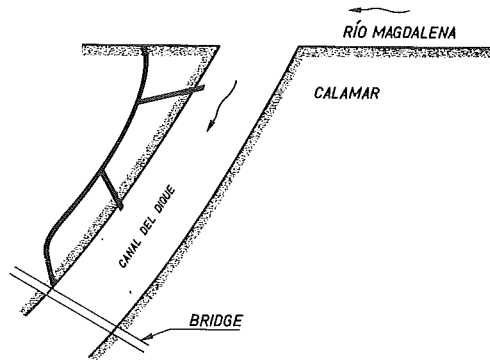


Figure 4.3.8 Permanent Landline for Dredging near Calamar

#### 4.4. LOWER CANAL DEL DIQUE (PASACABALLOS)

##### 4.4.1. Introduction

The silt carried by the Canal del Dique settles for a relatively large part at the mouths in the Bahía de Cartagena and the Bahía de Barbacoas but the sand also settles below the bifurcations and in the bifurcating caños. The settling of these sediments hampers navigation coming down the Canal del Dique and going to Mamonal and other Cartagena port areas, and the mouth at Pasacaballos is only kept open by dredging. It is feared that the silt coming from the Canal del Dique will fill up the navigation route for sea-going vessels from Boca Chica to Mamonal.

In the following paragraphs the movement of these sediments will be considered and the amounts of sedimentation estimated. An analysis will be made of the way the amounts to be dredged can be kept at a minimum, while still providing sufficient depth for navigation.

However, before the movement of sediment can be studied, the division of water will first have to be considered.

##### 4.4.2. Division of discharge and sediment-load along the Canal

###### Division of the discharge

Discharge rating curves for the Canal del Dique are available at Calamar, Correa (3 branches), Matunilla (3 branches), and Lequerica (3 branches). In Part II these rating curves were given against the local water-levels, and in Figure 4.4.1 these rating curves are given again but now related to the water-levels at Gambote, as at this station data are available over a much longer period, while no tidal effects are present at that station. From these rating curves it is now possible to determine for each level at Gambote the discharge for all distributaries. The result is given in Figure 4.4.2 in which also some measurements carried out by the Junta del Canal del Dique and by Mantilla have been included. These measurements, carried out before the dredging of the Caños Matunilla and Lequerica, indicate that at that time more than 40% of the discharge reached the Bahía de Cartagena. After the opening of both caños, this discharge was reduced to about 20%.

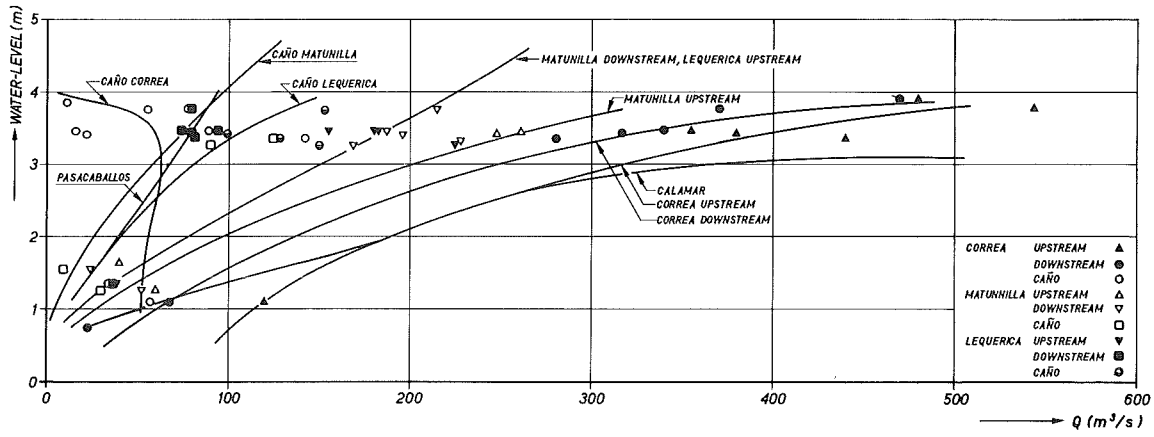


Figure 4.4.1 Discharge Rating Curves for the Lower Canal del Dique

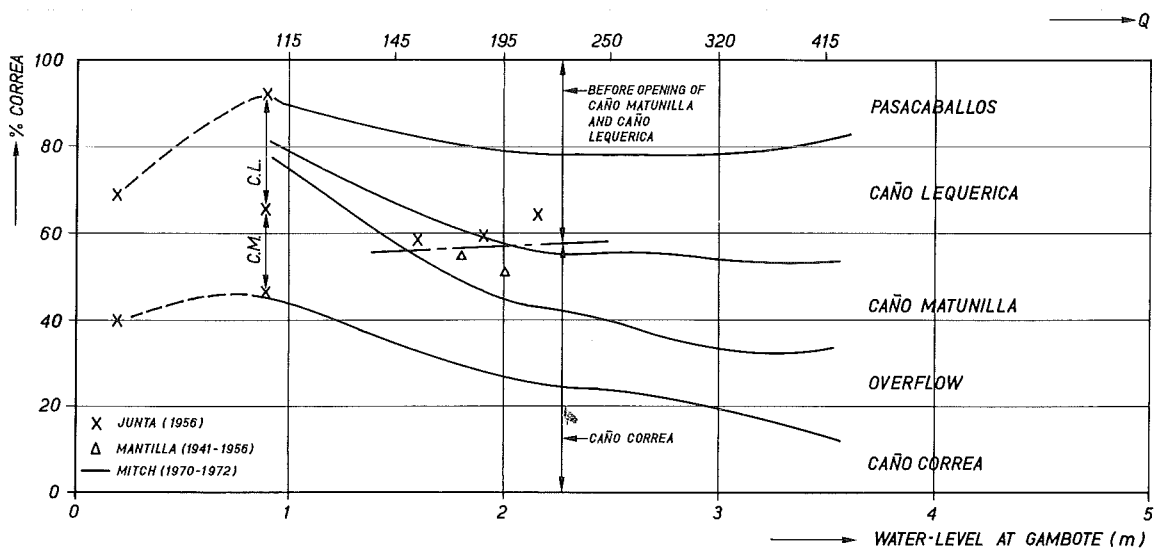


Figure 4.4.2 Distribution of Discharge over Bifurcating Caños

Division of the bed-material load

In Part II stage-transport curves (bed-material load) have been given for Calamar, Correa (3 branches) and Matunilla (3 branches). It may be seen that for Correa and Matunilla good agreement exists between the computed and measured transports.

In Table 4.4.1 the average weekly water-levels have been given for Calamar (taken from the 50% frequency-curve) and for Gambote (taken from a 5 years' average). With the help of the stage-transport curves the average weekly sediment transport was determined (also given in Table 4.4.1). From these data average yearly transports for the pertinent sections have been calculated, and have been gathered in Table 4.4.2.

Week	Water-level (m)				Sediment transport (m <sup>3</sup> /day)							
	Calamar	Gambote	Correa	Matunilla	Calamar computed	Calamar measured	Upstream Correa	Caño Correa	Downstream Correa	Upstream Matunilla	Caño Matunilla	Downstream Matunilla
1	4.40	2.89	1.71	0.81	200	1,200	1,300	240	675	480	310	100
2	4.75	2.70	1.56	0.77	300	1,850	900	240	450	370	250	175
3	4.95	2.46	1.40	0.72	400	2,500	600	220	300	280	180	50
4	5.20	2.26	1.27	0.67	500	3,000	430	200	200	190	130	30
5	5.50	2.05	1.15	0.63	650	4,200	270	150	130	150	90	15
6	5.75	1.80	1.03	0.57	800	5,000	150	100	75	100	60	10
7	5.80	1.58	0.92	0.52	800	5,500	50	40	50	30	10	0
8	5.90	1.35	0.82	0.46	900	5,800	0	0	0	0	0	0
9	5.95	1.20	0.76	0.43	900	6,000	0	0	0	0	0	0
10	5.85	1.14	0.73	0.42	800	5,500	0	0	0	0	0	0
11	5.70	1.05	0.69	0.40	750	4,700	0	0	0	0	0	0
12	5.60	0.99	0.66	0.38	700	4,300	0	0	0	0	0	0
13	5.45	0.94	0.64	0.37	650	4,000	0	0	0	0	0	0
14	5.15	0.95	0.65	0.37	500	2,900	0	0	0	0	0	0
15	5.15	1.09	0.71	0.40	500	2,900	0	0	0	0	0	0
16	5.20	1.12	0.72	0.41	500	3,000	0	0	0	0	0	0
17	5.30	1.44	0.86	0.48	550	3,500	0	0	0	10	0	0
18	5.40	1.74	1.00	0.56	600	4,000	140	80	70	70	40	5
19	5.45	1.90	1.08	0.60	650	4,000	200	120	100	110	65	10
20	5.35	1.99	1.13	0.61	550	3,500	250	150	120	120	70	10
21	5.35	2.11	1.19	0.64	550	3,500	370	180	170	160	100	20
22	5.60	2.26	1.27	0.67	700	4,300	430	200	200	200	130	30
23	5.60	2.38	1.35	0.70	700	4,300	530	210	250	245	160	40
24	5.60	2.49	1.42	0.72	700	4,300	620	220	300	280	180	50
25	5.75	2.55	1.46	0.74	800	5,000	700	230	350	305	205	55
26	6.05	2.61	1.50	0.75	1,000	6,500	750	240	360	310	220	60
27	6.20	2.63	1.51	0.76	1,100	7,200	780	240	380	340	235	70
28	6.30	2.64	1.52	0.76	1,200	7,400	800	240	400	340	235	70
29	6.40	2.59	1.49	0.75	1,300	8,300	730	230	360	310	220	60
30	6.65	2.53	1.45	0.73	1,600	9,500	680	225	330	290	190	55
31	6.75	2.48	1.42	0.72	1,750	10,500	620	220	300	280	180	50
32	6.80	2.47	1.41	0.72	1,800	11,000	610	220	300	280	180	50
33	6.95	2.53	1.45	0.83	2,050	11,700	680	225	330	530	360	130
34	6.40	2.57	1.48	0.74	1,300	8,300	730	230	350	305	205	55
35	6.30	2.57	1.48	0.74	1,200	7,400	730	230	350	305	205	55
36	5.70	2.57	1.48	0.74	750	4,700	730	230	350	305	205	55
37	5.25	2.58	1.48	0.74	500	3,000	730	230	350	305	205	55
38	4.55	2.60	1.50	0.75	300	1,500	750	240	360	310	220	60
39	4.15	2.63	1.52	0.76	200	1,000	800	240	400	340	235	70
40	3.75	2.70	1.56	0.77	100	600	900	240	430	360	250	75
41	3.15	2.73	1.58	0.78	0	250	930	240	460	400	265	80
42	2.90	2.77	1.62	0.79	0	150	1,020	245	500	420	280	90
43	2.70	2.83	1.66	0.80	0	100	1,150	245	570	450	300	95
44	2.65	2.89	1.71	0.81	0	0	1,300	240	660	470	320	100
45	2.85	2.93	1.74	0.82	0	150	1,400	235	730	500	335	110
46	2.80	2.97	1.77	0.83	0	100	1,500	230	780	530	360	130
47	2.85	3.01	1.80	0.84	0	150	1,560	230	850	560	390	140
48	2.90	3.07	1.85	0.85	0	150	1,750	225	1,010	600	420	160
49	3.05	3.09	1.87	0.86	0	200	1,820	220	1,150	630	440	170
50	3.25	3.11	1.89	0.86	0	300	1,950	220	1,250	630	440	170
51	3.70	3.13	1.90	0.87	100	600	1,960	220	1,300	670	470	190
52	4.20	3.09	1.87	0.85	200	1,000	1,820	220	1,150	600	420	160
Total sediment transport (m <sup>3</sup> /year)					224,700	1,403,500	252,840	61,810	134,400	101,290	68,355	22,155

Table 4.4.1 Average Weekly Sediment Transport

Location	Average yearly bed-material transport (m <sup>3</sup> )	Sedimentation (m <sup>3</sup> )
Calamar	225,000	
Upstream Caño Correa	253,000	
Caño Correa	62,000	57,000
Downstream Caño Correa	134,000	
Upstream Caño Matunilla	101,000	33,000
Caño Matunilla	68,000	
Downstream Caño Matunilla	22,000	12,000

Table 4.4.2 Average Yearly Bed-material Transport

It may be seen from these computations that the amount transported upstream of the Caño Correa is larger than the transport at Calamar. Although this is not impossible, it is more plausible that this is due to inaccuracies in the methods used, particularly the fact that for Calamar the 50% frequency-line has been used (based on a large number of years) and for Correa the five years' average. It has, therefore, been assumed that between Calamar and Caño Correa in fact no scour (or sedimentation) takes place.

Although measurements at the Lequerica Bifurcation are available it is difficult to make good stage-discharge curves because of tidal influences. From Figure 4.4.3 it may be seen that this is not really important, as from the 253,000 m<sup>3</sup> supplied upstream of the Caño Correa only 22,000 m<sup>3</sup> is transported downstream of the Caño Matunilla. Of this a small amount goes to the Bahía de Barbacoas via the Caño Lequerica, and the rest settles at Pasacaballos and upstream of Pasacaballos. The total amount reaching the Bahía de Barbacoas (see Figure 4.4.3) is about 130,000 m<sup>3</sup>/year. The rest of the amount supplied (123,000 m<sup>3</sup>/year) settles in the Canal del Dique and the caños downstream of the bifurcations, and at Pasacaballos. These amounts will have to be dredged.

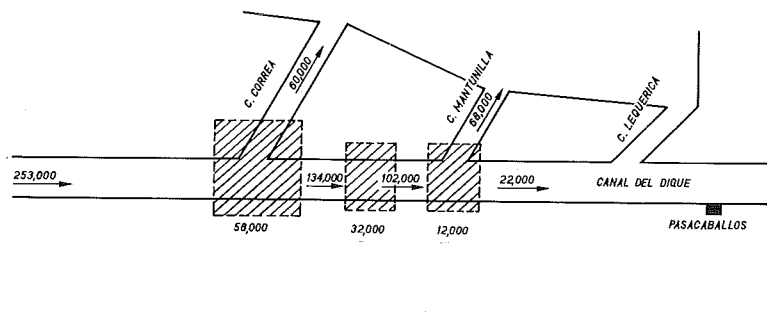


Figure 4.4.3 Division of Bed-Material Transport

Transport of wash-load

Turbidity measurements carried out by DICON give information about the daily variation in turbidity. This information has been plotted for the same five years' period as that for which the water-levels have been given (Figure 4.4.4).

III, 4.4

Week	Water-level		Turbidity		Discharge Correa Bifurcation			Discharge Hatunilla Bifurcation			Discharge Lequerica Bifurcation			Transport Correa Bifurcation			Transport Hatunilla Bifurcation			Transport Lequerica Bifurcation		
	Gambote		Gambote		(m <sup>3</sup> /s)			(m <sup>3</sup> /s)			(m <sup>3</sup> /s)			(kg/s)			(kg/s)			(kg/s)		
	(m)	(ppm)	Upstream	Caño	Overflow	Caño	Caño	Overflow	Upstream	Caño	Overflow	Caño	Caño	Overflow	Caño	Caño	Overflow					
1	2.89	130	310	62	43	62	71	68	40.3	8	5.5	8	9.2	8.8								
2	2.70	155	275	61	41	50	61	61	42.6	9.4	6.3	7.7	9.4	9.4								
3	2.46	150	245	59	39	37	56	54	36.8	8.8	5.8	5.5	8.4	8.1								
4	2.26	175	220	55	37	26	51	48	38.5	9.6	6.4	4.5	8.9	8.4								
5	2.05	200	200	52	34	25	42	42	40	10.4	6.8	5	8.4	8.4								
6	1.80	165	175	52	36	14	34	36	28.9	8.5	5.9	2.3	5.6	5.9								
7	1.58	175	155	51	36	16	26	28	27.1	8.9	6.3	2.8	4.5	4.9								
8	1.35	190	130	51	35	8	18	18	24.7	9.6	6.6	1.5	3.4	3.4								
9	1.20	215	125	50	36	6	16	15	26.9	10.7	7.7	1.2	3.4	3.2								
10	1.14	285	120	52	36	5	13	13	34.2	14.8	10.2	1.4	3.7	3.7								
11	1.05	325	117	50	35	5	13	13	38	16.3	11.4	1.6	4.2	4.2								
12	0.99	345	115	50	35	5	13	12	39.7	17.2	12	1.7	4.4	4.1								
13	0.90	375	110	50	35	3	11	9	41.3	18.7	13.1	1.1	4.1	3.3								
14	0.95	330	110	50	35	3	11	9	36.3	16.5	11.5	0.9	3.6	2.9								
15	1.09	395	120	52	36	5	13	13	47.4	20.5	14.2	1.9	5.1	5.1								
16	1.12	545	120	52	36	5	13	13	65.4	28.3	19.6	2.7	7	7								
17	1.44	760	140	50	35	12	22	22	106.4	38	26.6	9.1	16.7	16.7								
18	1.74	1,000	170	49	34	20	34	34	170	49	34	20	34	34								
19	1.90	750	85	52	35	22	37	39	138.8	39	26.2	16.5	27.7	29.2								
20	1.99	660	195	52	35	23	43	41	128.7	34.3	23.1	15.2	28.4	27.1								
21	2.11	605	205	51	37	27	45	45	124	30.9	22.4	16.3	27.2	27.2								
22	2.26	515	220	53	40	26	51	48	115.5	27.8	21	13.7	26.8	25.2								
23	2.38	485	235	56	40	33	54	52	107.6	25.6	18.3	15.1	24.7	23.8								
24	2.49	400	250	60	40	38	55	55	100	24	16	15.2	22	22								
25	2.55	345	260	60	42	42	57	57	89.7	20.7	14.5	14.5	19.7	19.7								
26	2.61	285	270	62	41	46	62	59	77	17.7	11.7	13.1	17.7	16.8								
27	2.63	290	270	62	41	46	62	59	78.3	18	11.9	13.3	18	17.1								
28	2.64	265	275	61	41	50	61	61	68.1	16.2	10.9	13.3	16.2	16.2								
29	2.59	215	270	62	41	46	62	59	58.1	13.3	8.8	9.9	13.3	12.7								
30	2.53	210	260	66	42	42	57	57	54.6	12.6	8.8	8.8	12	12								
31	2.48	225	250	60	40	38	55	55	56.3	13.5	9	8.6	12.4	12.4								
32	2.47	220	250	60	40	38	55	55	55	13.2	8.8	8.4	12.1	12.1								
33	2.53	240	260	60	42	42	57	57	62.4	14.4	10.1	10.1	13.7	13.7								
34	2.57	245	265	61	40	45	58	58	65	14.9	9.8	11	14.2	14.2								
35	2.57	205	265	61	40	45	58	58	54.3	12.5	8.2	11.9	11.9	11.9								
36	2.57	225	265	61	40	45	58	58	59.6	13.7	9	10.1	13.1	13.1								
37	2.58	245	270	62	41	46	62	59	66.2	15.2	10	11.3	15.2	14.5								
38	2.60	215	270	62	41	46	62	59	58.1	13.3	8.8	9.9	13.3	12.7								
39	2.63	225	275	61	41	50	61	61	61.9	13.7	9.2	11.3	13.7	13.7								
40	2.70	220	285	63	43	51	66	63	62.7	13.9	9.5	11.2	14.5	13.9								
41	2.73	250	290	64	44	52	67	64	72.5	16	11	13	16.8	16								
42	2.77	285	300	63	39	57	69	66	85.5	18	11.1	16.2	19.7	18.8								
43	2.83	320	300	63	39	57	69	66	96	20.2	12.5	18.2	22.1	21.1								
44	2.89	280	300	63	39	57	69	66	84	17.6	10.9	16	19.3	18.5								
45	2.93	300	310	62	43	68	74	68	93	18.6	12.9	20.4	22.2	20.4								
46	2.97	240	315	63	43	68	75	68	75.6	15.1	10.3	16.3	18	16.3								
47	3.01	245	320	61	45	67	77	70	78.4	15	11	16.4	18.9	17.2								
48	3.07	250	330	59	50	67	83	73	82.5	14.8	12.5	16.8	20.8	18.3								
49	3.09	240	340	58	51	71	88	72	81.6	13.9	12.2	17	21.1	17.3								
50	3.11	175	345	55	55	72	90	72	60.4	9.6	12.6	15.8	15.8	12.6								
51	3.13	150	350	56	56	74	91	74	52.5	8.4	8.4	11.1	13.7	11.1								
52	3.09	135	345	55	55	72	90	72	46.6	7.4	7.4	9.7	12.2	9.7								
α =									3,535	896	629	534	752	720								

Table 4.4.3 Average Weekly Quantities of Wash-load

Location	Transport (x10 <sup>6</sup> m <sup>3</sup> /year)	Reduction factor	Reduced transport (x10 <sup>6</sup> m <sup>3</sup> /year)
Upstream Caño Correa	1.527	1	1.527
Caño Correa	0.387	1	0.387
Overflow	0.272	0.75	0.204
Caño Hatunilla	0.231	0.75	0.173
Caño Lequerica	0.325	0.75	0.244
Pasacaballos	0.233	0.75	0.175

Table 4.4.4 Yearly Transport of Wash-load

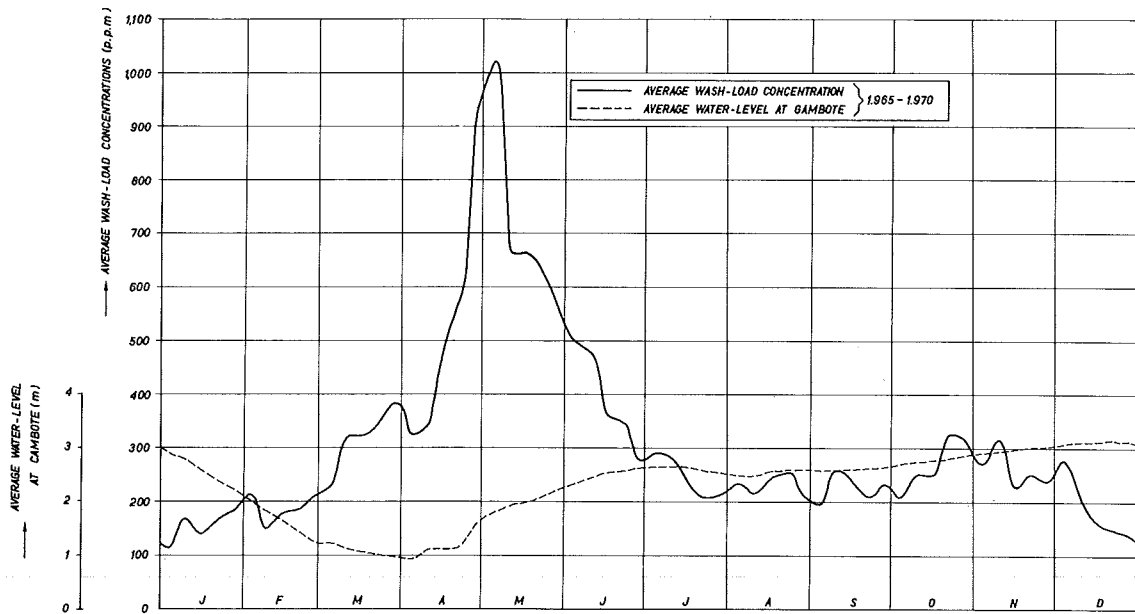


Figure 4.4.4 Wash-load Concentrations in Gambote

From the information given in Figure 4.4.4 (valid for Gambote) average weekly quantities of wash-load have been calculated for the various branches (Table 4.4.3). For the conversion of the average weekly quantities (in kg/s; Table 4.4.3) to the yearly transport of wash-load (in m<sup>3</sup>/year; Table 4.4.4), the total values ( $\alpha$ ) found in Table 4.4.3 have to be multiplied with a factor 432 (=  $24 \times 3,600 \times 7 / 10^3 \times 1.4$ ).

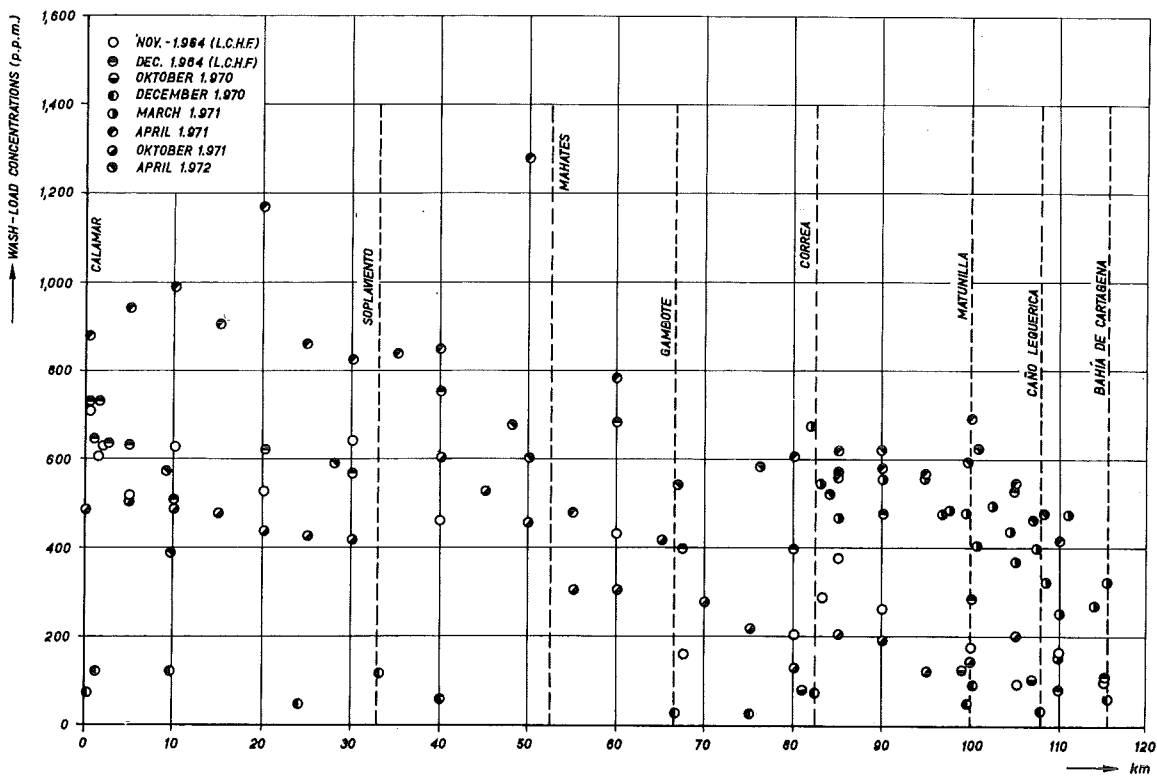


Figure 4.4.5 Wash-load Concentrations Along the Canal del Dique

In Part II it has been mentioned that the turbidity was also measured along the Canal del Dique (see Figure 4.4.5, which was already presented in Part II, Para. 3.3.5). From this figure it can be concluded that along the Canal del Dique the turbidity gradually reduces going in a downstream direction, although between Calamar and upstream of Gambote not much difference is found. Downstream of Gambote, however, the turbidity is significantly less and therefore a reduction factor (0.75) has been used for the sections downstream of the Caño Correa (As may be seen from the next paragraph, the application of this reduction factor gives results different from those found from the comparison of soundings and aerial photographs).

#### 4.4.3. Siltation near Pasacaballos (Figure 4.4.6)

In Figure 4.4.2 the division of the discharge over the various branches has been given. As it was assumed that the amount of silt is divided over the various channels proportional to the water, it will be useful to examine the water division somewhat more closely. The most striking feature is that only about 20% of the discharge upstream of the Caño Correa reaches the Bahía de Cartagena, the remainder mostly going to the Bahía de Barbacoas, although a part is lost by overflow (which partly is again lost by evaporation). Some measurements carried out by the Junta del Canal del Dique (1956) and by Mantilla (1941, 1956) indicate that, before the opening of the Caños Lequerica and Matunilla (1958) about 40% of the upland discharge reached the Bahía de Cartagena. This will probably have caused a sedimentation about twice as large as at present.

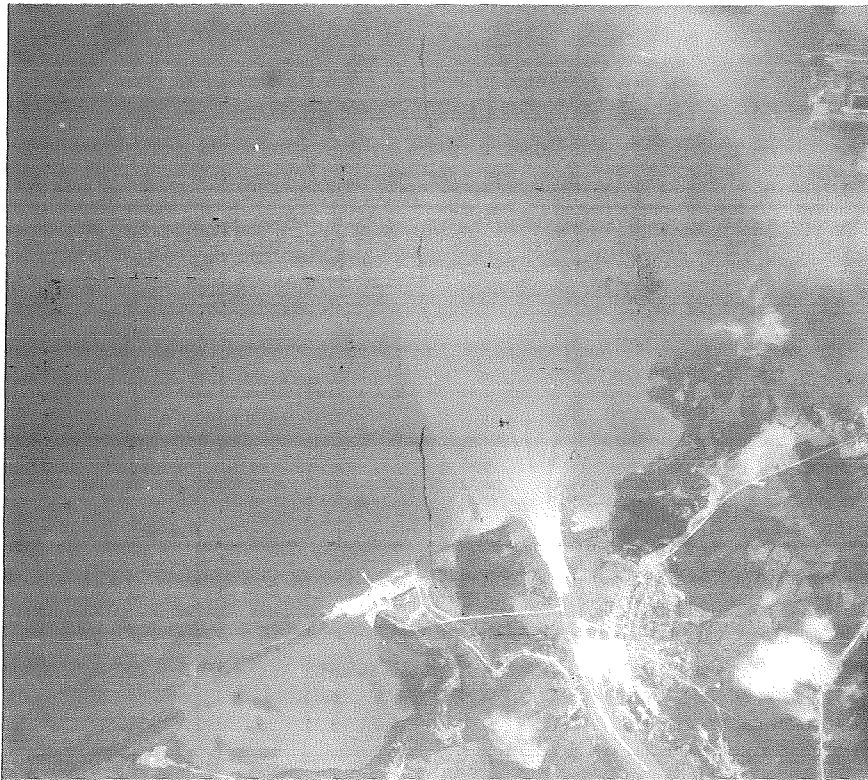


Figure 4.4.6 Aerial View of the Canal del Dique Mouth

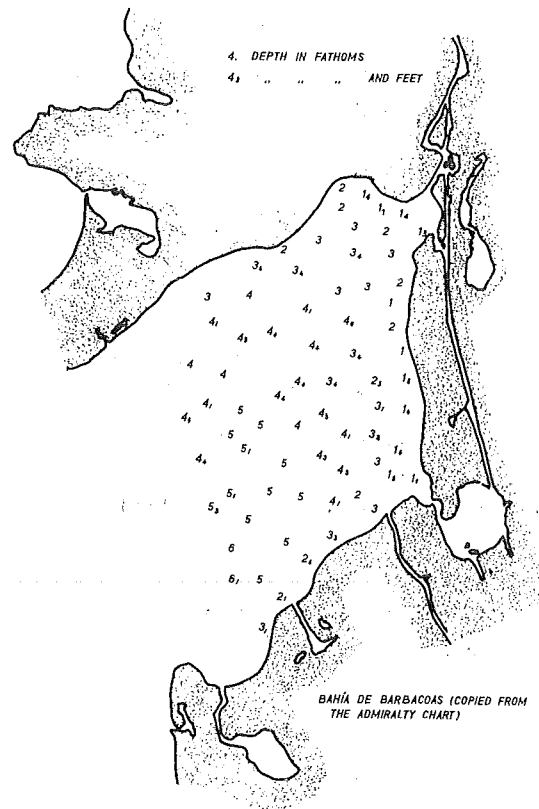


Figure 4.4.7 Sounding of Bahía de Barbacoas

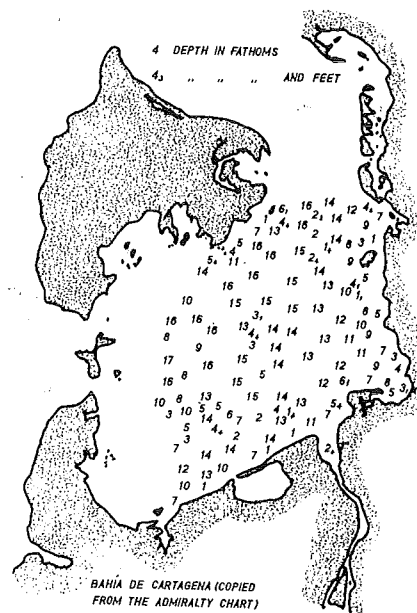


Figure 4.4.8 Sounding of Bahía de Cartagena

From soundings (see Figures 4.4.7 and 4.4.8) it may be seen that depths just outside the sedimentation area in the Bahía de Barbacoas are considerably less than in the Bahía de Cartagena (5m and 15 m respectively). Nevertheless, there is still a large space for the storage of silt in the Bahía de Barbacoas. A guaranteed continuation of the present sedimentation in the Bahía de Barbacoas instead of in the Bahía de Cartagena requires maintenance

III, 4.4

of the channels as they are. This means that in those cases where caños tend to silt up, the mouth and the caño itself should be cleaned by dredging, always ensuring that there are good possibilities for outflow and sedimentation.

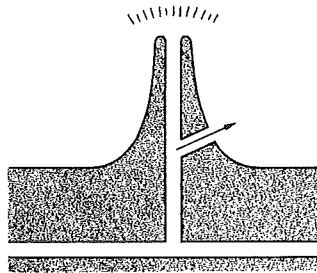


Figure 4.4.9 New Outlet to be Dredged at Lequerica Mouth

The siltation of Lequerica as an example is given schematically in Figure 4.4.9. As soon as the building up of natural levees causes the resistance of the channel to increase, a cut should be made as indicated.

In future also a new outlet may be considered besides Matunilla and Lequerica, branching off near Recreo where the distance to the Bahía de Barbacoas is only short. To keep the outlets open, sedimentation of bed material-load should be dredged in the caños (and not only in the Canal). The works to replace sedimentation of bed material from the main Canal to the caños, as carried out at Correa, may be advantageous, as dredging can be carried out in the caño instead of in the Canal, thus not hampering navigation. By these works, however, the total amount to be dredged will not be reduced.

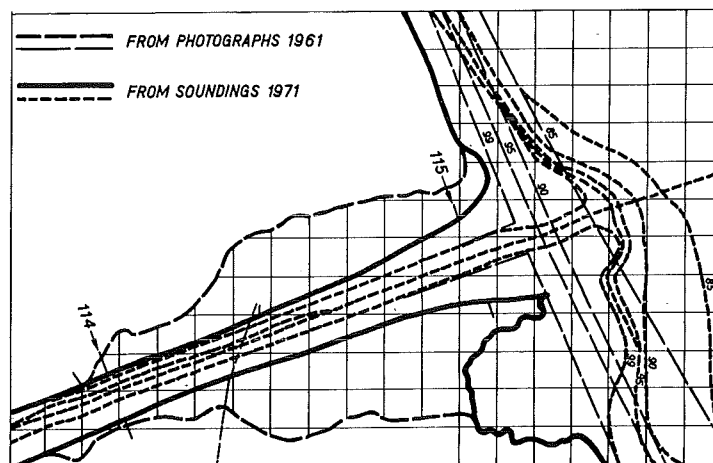


Figure 4.4.10 Sedimentation Pasacaballos (from photographs 1961 and sounding 1971)

From the foregoing it may also be concluded that the plans to close the Canal del Dique downstream of the Caño Matunilla, with a navigation route through the Caño Matunilla, the Bahía de Barbacoas, the Caño Lequerica and the Canal del Dique to Cartagena, should not be favoured. The amounts settling in the Bahía de Barbacoas at the mouth of the Caño

Matunilla would be three times as large as at present in the Bahía de Cartagena. As, moreover, the Bahía de Barbacoas is shallower, the amount to be dredged would increase considerably, while this dredging must then be carried out in the navigation channel itself.

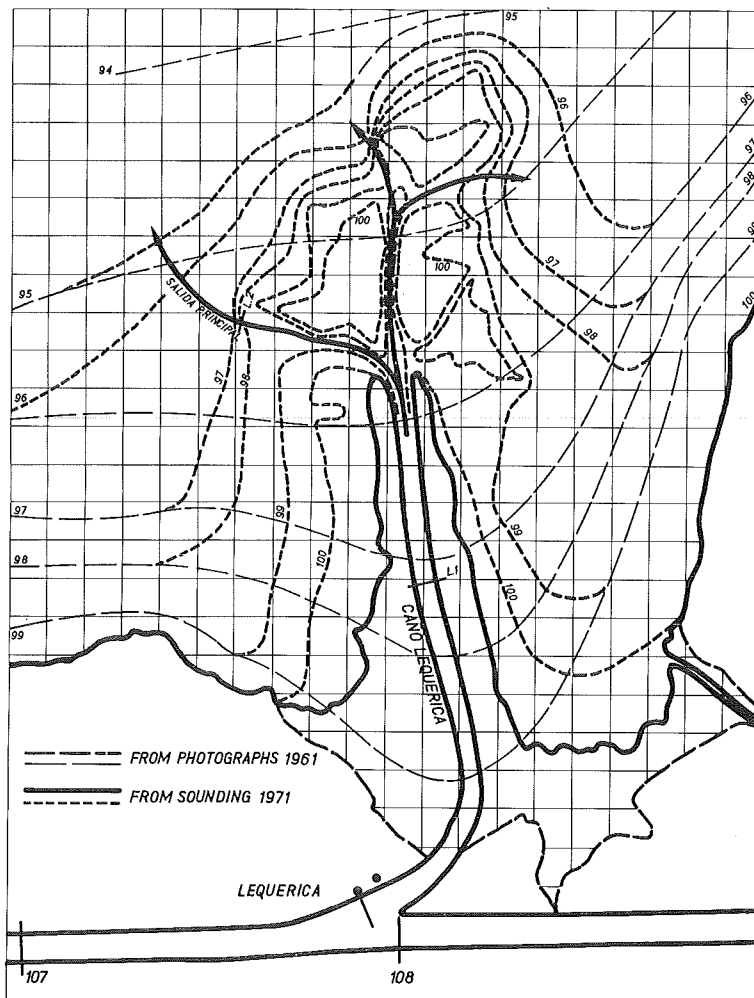


Figure 4.4.11 Sedimentation Caño Lequerica (from photographs 1961 and sounding 1971)

If the navigation route is maintained along the Canal, the amount to be dredged will be considerably less and can partly be done outside the navigation channel. Recurrent dredging in the canal itself will, however, still be necessary.

To estimate the amounts to be dredged, three methods were available of which the results can be compared:

- 1) The amounts could be determined from the silt and sediment-load division as computed in Para. 4.4.2.
- 2) From a comparison of aerial photographs taken in 1961 and soundings made in 1971 the amount of sedimentation at the outlets of the canal (Pasacaballos, Caños Lequerica and Matunilla) could be found. This has been done as indicated in Figures 4.4.10, 4.4.11 and 4.4.12 by means of drawing a net of squares and noting the difference in height (the contour levels had to be estimated with the help of the configuration).

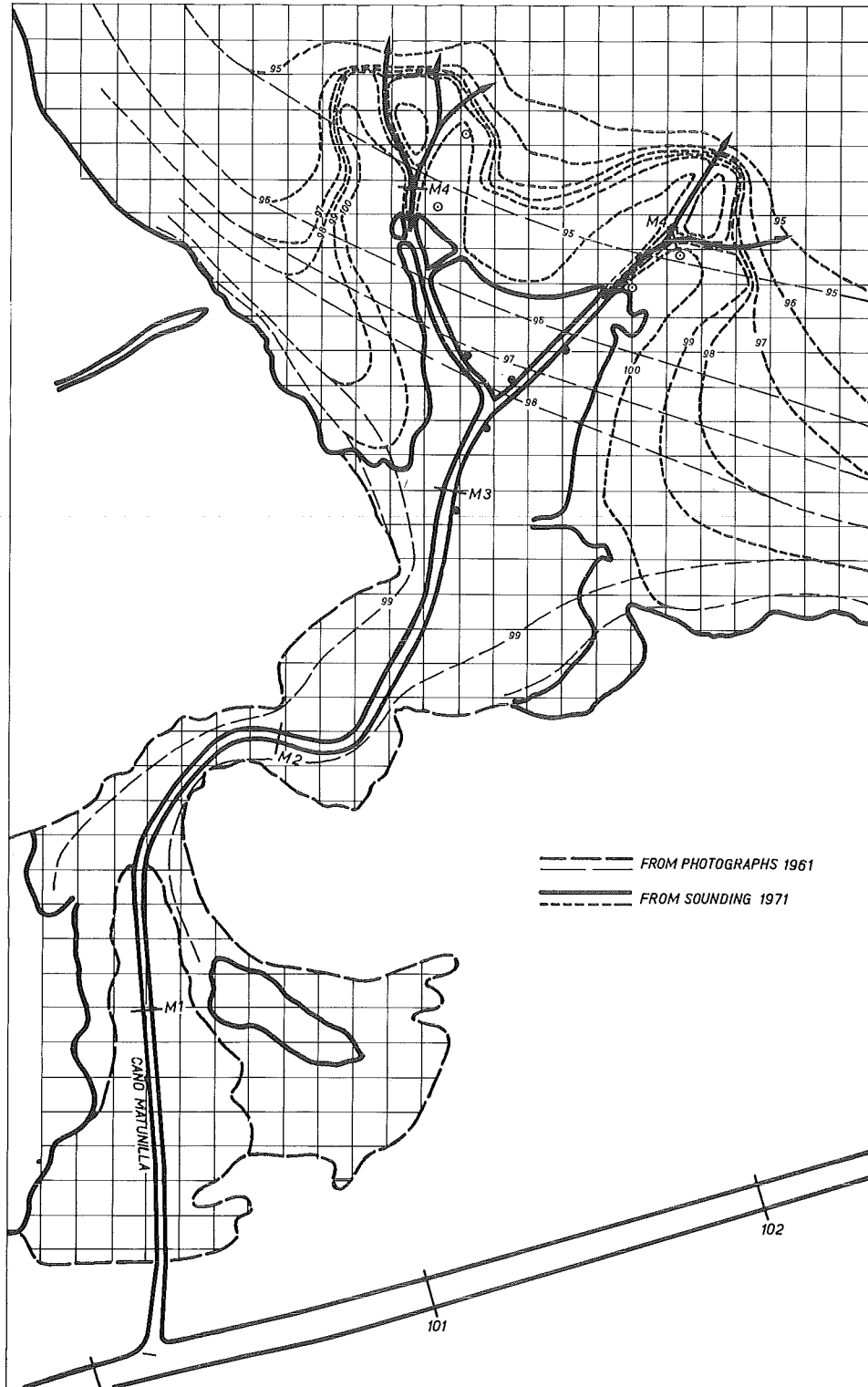


Figure 4.4.12 Sedimentation Caño Matunilla (from photographs 1961 and sounding 1971)

3) From soundings made by MITCH in 1971 and 1972, the amount of sedimentation at the outlets could be found in the same way as with the aerial photographs. For Matunilla and Lequerica also a net of squares was drawn, while for Pasacaballos a map of equal differences has been made (Figure 4.4.13).

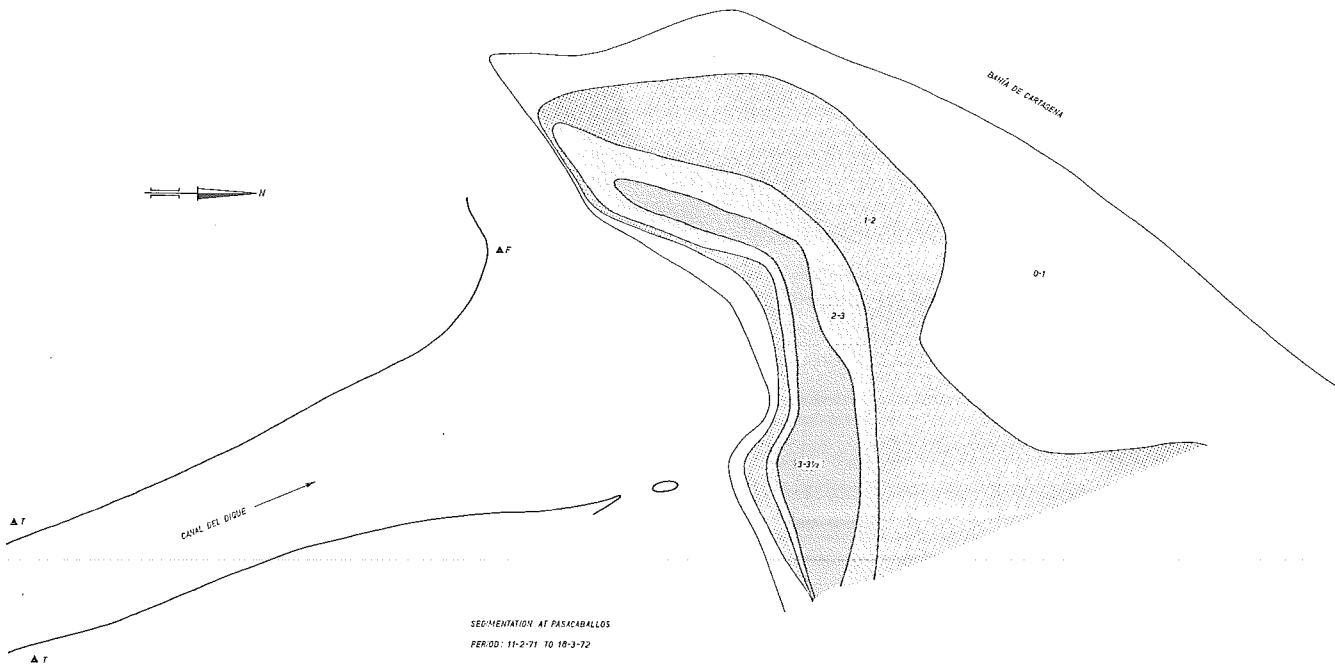


Figure 4.4.13 Sedimentation Pasacaballos (from soundings of 1971 and 1972)

The results have been compiled as follows:

Place	Computed sediment transport ( $\times 10^3 \text{ m}^3/\text{year}$ )	From photographs 1961 and sounding 1971 ( $\times 10^3 \text{ m}^3/\text{year}$ )	From soundings 1971-1972 ( $\times 10^3 \text{ m}^3/\text{year}$ )	From soundings 1971-1972 but reduced for a year with average discharges ( $\times 10^3 \text{ m}^3/\text{year}$ )
Pasacaballos	333	225	430	285
Lequerica	325	596	640	425
Matunilla	297	610	750	500
Correa bifurcation	58			
Correa-Matunilla	32			
Matunilla bifurcation	12			

Table 4.4.5 Yearly Sedimentation

Regarding these results the following observations can be made:

The caños Matunilla and Lequerica were opened in about 1958. During the beginning of their existence they were short and had slopes which were steeper than they have now and initially drew more water and sediment than at present. The first photographs were taken in 1961. It is likely that at that time still small quantities of water and sediment were supplied to the Canal del Dique downstream of the Caño Lequerica; this is in agreement with the fact that a smaller amount of sedimentation is indicated by the photographs for Pasacaballos

### III, 4.4

(compared with the computations). Nevertheless, the difference is not completely explained in this way. Even although the method by the photographs is not very accurate, it is possible that the computed values are somewhat low. (For that reason the reduction mentioned in Para. 4.4.2 has not been applied).

It should also be kept in mind that 1971-1972 was an exceptional year (see Figure 4.4.14). The amounts found from the 1971-1972 soundings have, therefore, been reduced by the same ratio as the discharges, and the results are given in the last column of Table 4.4.5 (As with the photographs the comparison of the soundings also was difficult, as no fixed points of the first sounding were available when carrying out the second sounding).

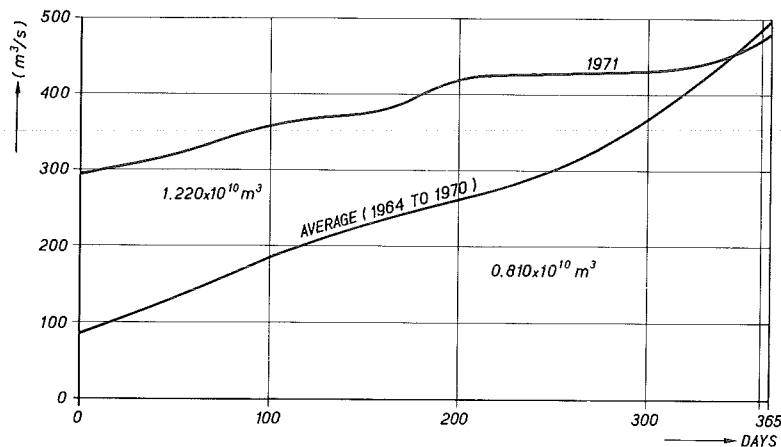


Figure 4.4.14 Comparison of Duration Curves of Discharges at Gambote

The following round figures have been taken as annual sedimentation (not all of this will have to be dredged; see Para. 4.4.4):

Pasacaballos	330,000 m <sup>3</sup>
Outlet of Caño Lequerica	330,000 m <sup>3</sup>
Outlet of Caño Matunilla	300,000 m <sup>3</sup>
Caño Correa Bifurcation	60,000 m <sup>3</sup>
In the Canal del Dique between the Caños Correa and Matunilla	30,000 m <sup>3</sup>
Caño Matunilla Bifurcation	10,000 m <sup>3</sup>

Considering the sounding of Figure 4.4.10 it may be seen that sedimentation takes place over a height of about 15 m and a width of about 1,000 m. The cross-sectional area is, therefore, about 15,000 m<sup>2</sup>, which means that the annual deposit in the bay is about  $330,000/15,000 = 22$  m/year.

As the channel from Mammonal to Boca Chica is at a distance of 2 km, no danger of appreciable silting of this channel may be expected for many years.

#### 4.4.4. Dredging of the Lower Canal del Dique

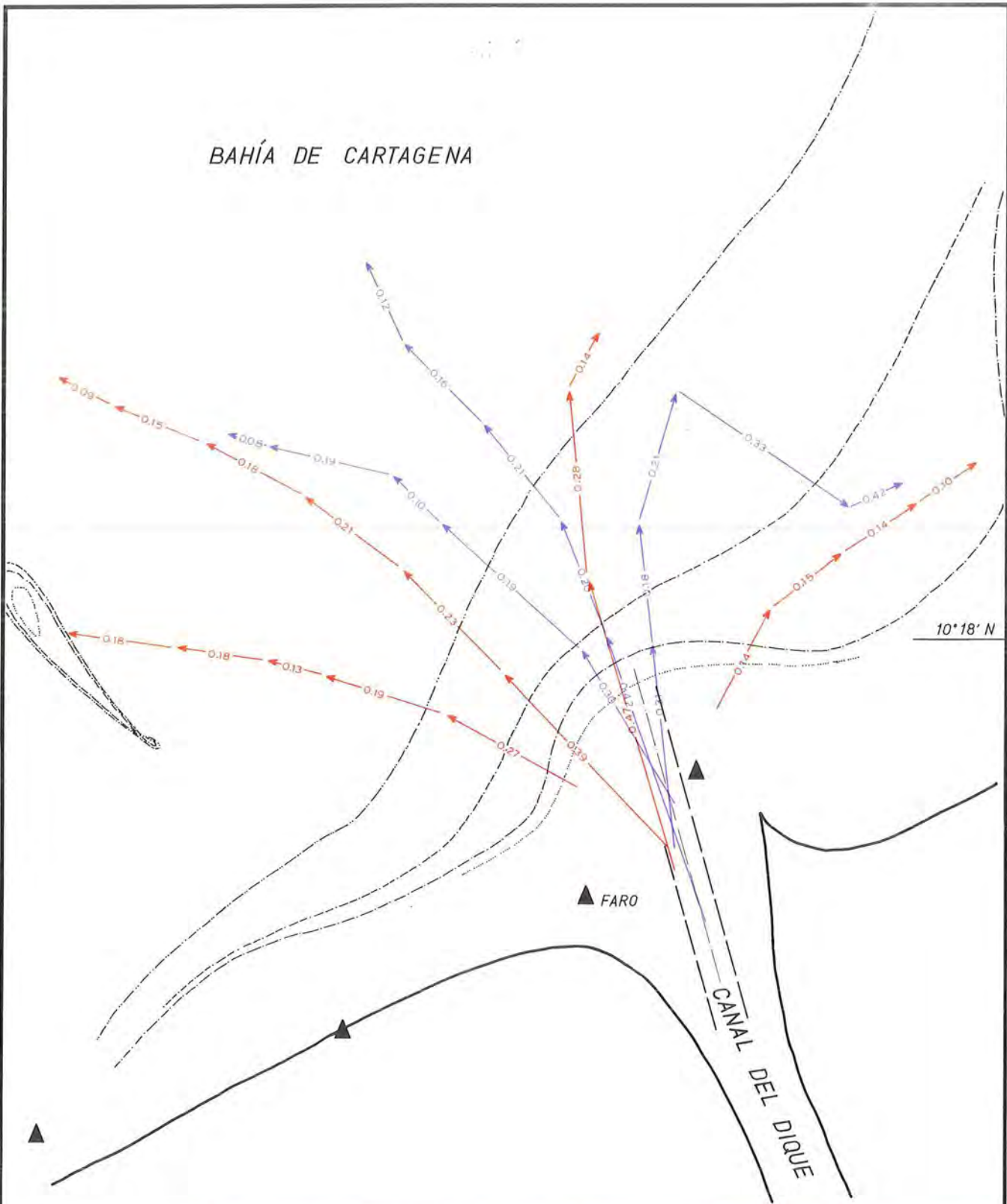
Fortunately not all of the sedimentation given in Para. 4.4.3 has to be dredged. Near Pasacaballos and the Caños Lequerica and Matunilla the amount to be dredged depends very much on the frequency of dredging. If the Caños are kept well open, the total amount will be less. It seems safe to estimate that about half of the total amount sedimented will have to be dredged (480,000 m<sup>3</sup>). Further, the amounts settling near the bifurcations will have to be dredged completely (100,000 m<sup>3</sup>). Including Calamar, the total amount to be dredged annually will be about 1.3 million m<sup>3</sup>.

In Figure 4.4.15 flow-lines are given for the outlet of the Canal del Dique in the Bahía de Cartagena. It will be seen that the flow spreads more or less evenly, the direction depending somewhat on the tide. A dredge cut has been given in this figure, but it is thought that the direction is not very critical.

Some guidance of the flow by dumping the spoil as much as possible as a ridge along the dredge cut may help a little to keep the channel open. Stimulating vegetation on nearly dry areas will also help. After some time a completely new channel may be selected in order to obtain a more evenly building out of the delta. A channel with a width at the bottom of 60 m and a depth of 2.60 m below L.L.W.S. is advised.

The channel dimensions are small, and in the large expanse of water of the Bahía de Cartagena the mouth is difficult to find. The channel should, therefore, be marked by (light) buoys, which are preferred to light beacons because they are more easily moved with a shifting channel and the risk of damage by ships is less.

# BAHÍA DE CARTAGENA



SONDEO  
SOUNDING

## BAHÍA DE CARTAGENA / LA BOCA del C.D.D.

ESCALA/SCALE 1:10.000

FECHA/DATE 18-III-1972

NIVEL DE REDUCCIÓN: 0.33 metros BAJO EL NIVEL MEDIO DEL MAR

CHART DATUM: 0.33 metres BELOW MEAN SEA LEVEL

FECHA/DATE: 17-IV-'72 / 9-VI-'72 NIVEL DE AGUA/WATERLEVEL: 0.27 m SOBRE EL DATUM / ABOVE DATUM

LÍNEAS DE CORRIENTE / FLOW LINES — 1.18 —> VELOCIDAD EN m/seg / VELOCITY IN m/s

	.....	5 m
CURVAS ISOBATAS	-----	10 m
DEPTH CONTOURS	-----	15 m
	-----	20 m

## Chapter 5

### DREDGING PROGRAMME AND PHASING OF RIVER-WORKS

#### 5.1. INTRODUCTION

From the foregoing Chapters it will have become clear that in the phasing of the different types of river-works the following general sequence should be kept:

- Aids to navigation (channel patrols);
- temporary river-works (dredging); and
- permanent river-works.

A start has already been made with the channel patrols and it is recommended to extend these services as indicated in Chapter 2.

As far as temporary river-works are concerned, it is thought that a very rigid system of recurrent dredging of crossings will be more promising than trying to close secondary branches (in a more permanent way), although the eventual closure of secondary branches must remain under consideration.

The present Report has, therefore, in the first place been based on the concept of such a dredging programme of crossings, besides a small number of permanent river-works, especially to be carried out near Barrancabermeja and the Río Regla Confluence. More about the dredging programme is said in the next paragraph, which will show that this programme also includes the Canal del Dique and a number of port entrances. As far as the permanent river-works are concerned, it is thought that works just upstream of Barrancabermeja, to ensure the access to the port, are the most urgently required. These works can then slowly be extended downstream to the Río Sogamoso Confluence, according to the alignment presented in Para. 3.5. River-works near the Río Regla Confluence are not so urgent, because at present there is not yet much traffic in the Barrancabermeja - Pto. Berrío section. As has been explained, the region near the Río Regla Confluence is very characteristic for the Río Magdalena, and as soon as warranted by increased transports, improvements should be commenced.

The works indicated near La Dorada - Pto. Salgar have other backgrounds besides navigation, and the urgency to carry out these works is mainly governed by these other aspects (Palanquero and La Dorada town). It is thought, however, that in any case a model study as indicated in Para 3.2.3 should be carried out as soon as possible. If it is decided to move part of the port facilities to Pto. Triunfo, these facilities should be ready before the opening of the Medellín - Pto. Triunfo road.

#### 5.2. DREDGING PROGRAMME

##### 5.2.1. The Organization

In various Chapters of this Part of the Report a concept has been developed about the recurrent dredging of crossings, still to be verified by means of test dredging. This will

### III, 5.2

require a very tight programme and a well-organized preparation, with the following activities being necessary as preparation for the actual dredging:

- i Measuring and elaboration of length profiles
- ii Determining from the length profiles all places along the river that have depths smaller than the required depths in relation to L.R.L
- iii Measuring of flow-lines at those crossings found under Point ii that cannot be schematized (for computation) by aerial photographs only
- iv Schematization of all crossings with the help of aerial photographs and flow-lines.
- v Computation of the scour that can be expected during a drop of the water-level to L.R.L.
- vi Determining those crossings that require dredging
- vii Determining the sequence in which the crossings will be dredged
- viii Sounding and measuring of flow-lines of the crossings to be dredged
- ix Determining the alignment of the dredge cut
- x Indicating by means of beacons the transit line of the dredge cut.

In addition to the actual preparation given in the foregoing ten points, the following activities will still be required during and after the dredging of the crossing(s):

- xi Instruction of the dredge master regarding the necessary depth of dredging in relation to the daily water-level
- xii Sounding of the dredged channel(s)
- xiii Beacons or buoying of the dredged channel(s).

Part of the above activities can be carried out by a properly organized channel patrol service (e.g., Points i, iii, viii and xiii), in close co-operation with a river conservancy department ("Unidad de Estudios Fluviales"). The other activities will have to be carried out by the river conservancy department itself.

The computation mentioned under Point v will have to be carried out by an agency that has computer facilities. In this case CETIH may prove to be of help, because of the experience recently gained by one of its staff members, with the computations carried out for the present Report. The schematization of the crossings will have to be carried out by the river conservancy department, in co-operation with the computer expert. In this respect, the experience gained by the "Unidad de Estudios Fluviales" will prove to be very important.

#### 5.2.2. Amounts to be dredged and required capacity of the dredge fleet

Although no actual dredging programme was prepared for the year 1972, the scour at a number of crossings was computed and the amounts to be dredged determined accordingly, for the low water season at the beginning of 1972. These amounts were determined for the Río Magdalena and the Canal del Dique, and are summarized in Table 5.2.1. The first four columns of this table concern recurrent (maintenance) dredging, and the last two columns dredging that has to be carried out only once. The 5,435,000 m<sup>3</sup> mentioned under backlog for the Canal del Dique, is required to enlarge the Canal del Dique cross-section in accordance with the adopted (MITCH) design cross-section.

The total amount to be dredged recurrently on crossings along the Río Magdalena (to obtain a depth of 7'6" below L.R.L. downstream of Pto. Berrío and 4'6" upstream of Pto. Berrío), including dredging of the entrances to river ports, is about 2,000,000 m<sup>3</sup> per low water season (months January to March) but, whether part or all of this dredging will have to be repeated in the subsequent low water (months June and July) cannot be said without practical experience. The time available for the amount to be dredged is between 2 and 3 months, say 75 days. The actual output of one new dredger (design capacity 1,200 m<sup>3</sup>/h) according to Figure 2.3.16 is about 730 m<sup>3</sup>/h when dredging crossings of 15,000 m<sup>3</sup>, and about 795 m<sup>3</sup>/h when dredging crossings of 25,000 m<sup>3</sup>. In 75 days the output is, therefore, between 1.3x10<sup>6</sup> m<sup>3</sup> and 1.4x10<sup>6</sup> m<sup>3</sup>. This means that for the execution of the dredging programme two dredgers are required.

River section	Recurrent dredging of crossings and river ports (m <sup>3</sup> /year)				Dredging for permanent river-works (m <sup>3</sup> )	Backlog along the Canal del Dique (m <sup>3</sup> )	Remarks
	to 4'6" (below L.R.L.)	to 6' (below L.R.L.)	to 7'6" (below L.R.L.)	River ports			
La Dorada - Pto. Triunfo	217,000			10,000 <sup>1)</sup>	3,500,000 <sup>2)</sup> 500,000 <sup>3)</sup>		1) La Dorada - Pto. Salgar port 2) If port facilities remain in La Dorada - Pto. Salgar
Pto. Triunfo - Pto. Inmarco	53,000						3) To be dredged in La Dorada - Pto. Salgar port, if new port facilities be created near Pto. Triunfo
Pto. Inmarco - Pto. Berrío	157,000	547,000		10,000 <sup>4)</sup>			4) Possibly to be dredged in future in the Pto. Berrío port
Pto. Berrío - Barrancabermeja		500,000	1,000,000	PH <sup>5)</sup>	1,150,000 <sup>6)</sup>		5) If permanent river-works near Barrancabermeja will be carried out to safeguard the approaches to this river port
Barrancabermeja - Gamarra			498,000	25,000 <sup>7)</sup>			6) If permanent river-works near the Río Regla Confluence will be carried out
Downstream of Gamarra				PH <sup>8)</sup>			7) Pto. Wilches port
Calamar (Canal del Dique)			700,000				8) Downstream of Gamarra some crossings may require dredging
Bifurcating caños			100,000				9) To enlarge the Canal del Dique cross-section to the MITCH design cross-section
Bahfas de Cartagena and Barbaças			480,000			5,435,000 <sup>9)</sup>	

Table 5.2.1 Summary of Volumes to be Dredged

As can be seen in Table 5.2.1, half of the total amount (1x10<sup>6</sup> m<sup>3</sup>) has to be dredged between Barrancabermeja and Pto. Berrío, a river stretch of only 100 km. This indicates that the requirement of 7'6" below L.R.L. on this section is in fact too severe. If in this section the requirement would (at least temporarily) be lowered to a depth of 6' below L.R.L., one dredger could nearly do the complete stretch from La Dorada to Gamarra. This seems attractive, because it would mean that for the time being one new dredger would be sufficient, with any shortage in dredge capacity on the Río Magdalena being augmented by the converted dredger DH 6. The dredging programme may then be as indicated in Figure 5.2.1.

In general this dredging programme has been drawn up in agreement with the capacities and the properties of the available dredgers; for example, dredging of crossings, cannot be done by the DH 7. Some activities could, however, be interchanged: for example, part of the dredging in the Lower Canal del Dique could be carried out by the DH 7 instead of the DH 9.

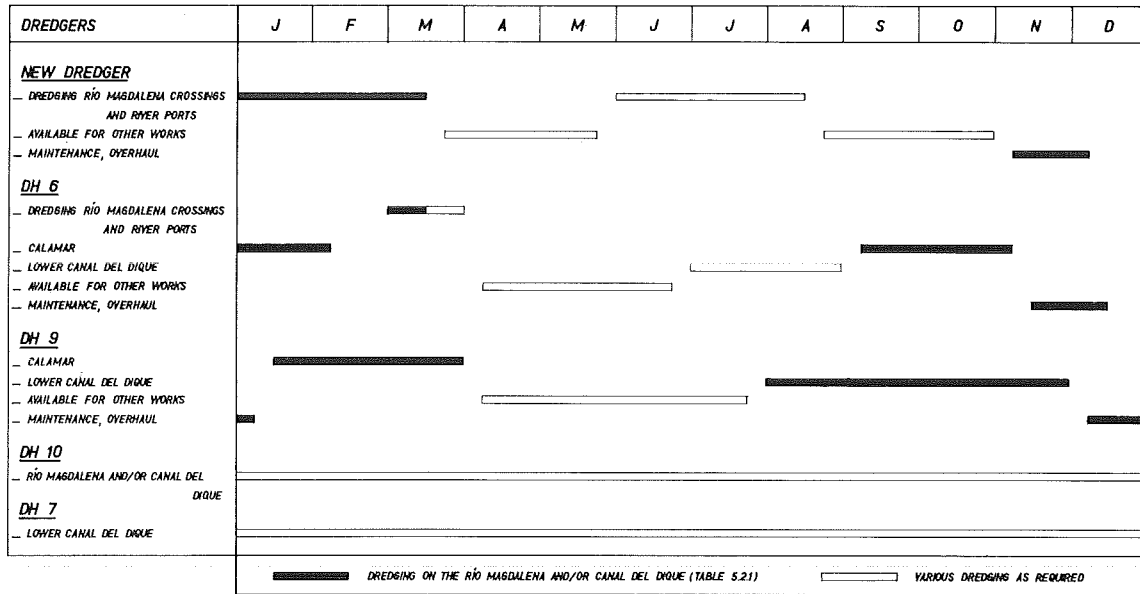


Figure 5.2.1 Dredging Programme

5.3. PHASING OF THE RIVER-WORKS

Only a rough outline of the phasing of the river-works can be given here because the phasing depends, apart from the required preparations and funds to be made available, strongly on the (awaited) results of "The Magdalena River Area Transport Study". The works and activities indicated below, must therefore only be considered as a guide.

1973: Test dredging.

Dredging of the Lower Canal del Dique.

Initiating the extension of the channel patrol service.

Besides general studies, a study of the development of dredged channels after test dredging.

Preliminary design of river-works upstream of Barrancabermeja, along the right bank of the Río Magdalena.

Improving the dredging organization and the output of dredgers.

1974: Model study for river-works in the La Dorada - Pto. Salgar area.

Further extension of the channel patrol service.

Dredging of crossings and river ports downstream of Barrancabermeja (with the converted DH 6).

Building of new river dredger.

Dredging of a sand-trap at Calamar and study of its development.

Dredging of the Lower Canal del Dique.

Executing river-works upstream of Barrancabermeja, along the right bank.

Building new port facilities near Pto. Triunfo.

1975: The carrying out of the complete dredging programme.

Further execution of river-works upstream of Barrancabermeja.

PART IV

HYDROGRAPHIC AND HYDROLOGICAL MANUAL



## CONTENTS

	Page
<b>PART IV HYDROGRAPHIC MANUAL</b>	
Chapter 1 GENERAL	337
Chapter 2 INSTRUMENTS	338
2.1. Introduction	338
2.2. Sextant	338
2.3. Theodolite	339
2.4. Levelling instrument	340
2.5. Rangefinder	341
2.6. Echo-sounder	342
2.7. Pendulum current meters	344
2.8. Propellor current meters	345
2.9. Water sampler	346
2.10. Delft bottle (DF)	347
2.11. Bed-load transport meter "Arnhem"	349
2.12. Bottom grab	349
2.13. Turbidity meter	350
2.14. Conductivity meter	351
2.15. Visual accumulation tube	352
Chapter 3 LOCATION FIXING	354
3.1. Triangulation	354
3.2. Local position fixing	356
3.2.1. With two sextants	356
3.2.2. With one sextant and a leading line	357
3.2.3. With one rangefinder and a leading line	358
3.2.4. With one sextant and a rangefinder	358
3.2.5. With one compass and a sextant	359
3.2.6. With one compass and a rangefinder	359
3.2.7. With one compass only	360
Chapter 4 MEASUREMENTS AND ELABORATIONS	362
4.1. Water-levels	362
4.1.1. Introduction	362
4.1.2. Gauges	362
4.1.3. Selection of gauge sites	364

	Page
4.2. Soundings and route maps	365
4.2.1. Introduction	365
4.2.2. Reduction-level	365
4.2.3. Preparations for soundings	366
4.2.4. Actual soundings	367
4.2.5. Elaborations	368
4.2.6. Charts involved	369
4.2.7. Route maps	371
4.2.8. Kilomatreage	371
4.2.9. Processing of route maps	372
4.3. Discharges	373
4.3.1. Discharge per unit width	373
4.3.2. Total discharge	375
4.3.3. Tidal effects	377
4.4. Sediment transport	381
4.4.1. Introduction	381
4.4.2. Sampling methods	382
4.4.3. Elaboration of samples	383
4.5. Salinity	384
4.6. Grain-sizes	385

## Chapter 1

### GENERAL

This Part consists of descriptions of the various survey routines and measurements that were developed by NEDECO in the course of the "Río Magdalena and Canal del Dique Survey Project".

The descriptions are shaped in the form of a Hydrographic and Hydrological Manual, giving the basic background and the detailed applications as far as they will be of use for the Unidad de Estudios Fluviales, MOP and ADENAVI.

This Manual starts with a description of the various instruments, that were used by the Mission (Chapter 2).

Secondly, descriptions are given of the positioning systems that were used (Chapter 3). Finally, the various measurements and their elaboration are described, as carried out in Colombia (Chapter 4). In this Chapter also the measuring techniques of the instruments dealt with in Chapter 2 is detailed, because in that chapter the instruments are only described in a general way.

## Chapter 2

### INSTRUMENTS

#### 2.1. INTRODUCTION

A description is given of the instruments used by the Mission. Most of them were selected and ordered in the Netherlands, but some (like the Kelvin Hughes and Berg & Berg propellor current meters) were borrowed from ADENAVI.

The description of the instruments contains some general information and technical data, as well as their application, and the advantages and disadvantages of each type.

For more detailed information about these instruments (required for repairs or adjustments), the manufacturer's manual of the relevant instrument should be consulted. Sometimes in this Chapter an additional instrument is mentioned or described, as such an instrument may be expected to be of use in future measurements.

The way in which the instruments should be used is dealt with in Chapter 4.

#### 2.2. SEXTANT

The sextant was originally designed for measuring vertical angles, such as the sun's or stars' altitude, from aboard ship. However, it also serves well for the measurement of horizontal angles, specially from aboard ship, where a theodolite cannot be used.

Measuring range is from  $0^{\circ}$  -  $115^{\circ}$ , while the graduated arc of the instrument holds  $60^{\circ}$  (see Figure 2.2.1) from which it got the name sextant.

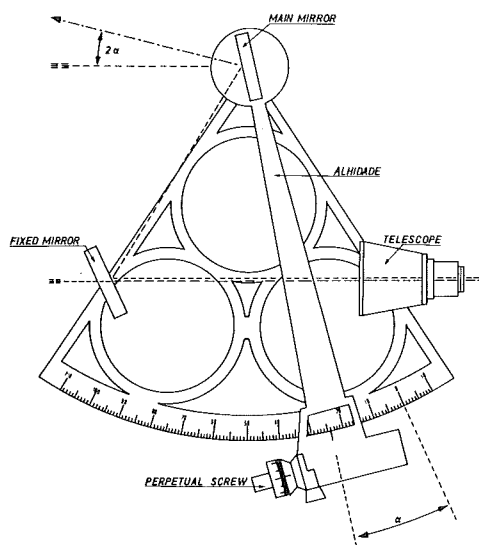


Figure 2.2.1 Sextant

Principle: Two beacons - of which the angle between them is to be measured - are brought to coincidence. The left-hand beacon is seen directly through the telescope, and the right-hand beacon is double-reflected by the main mirror on the alhidade, and the fixed mirror in front of the telescope.

Index-error: Almost every sextant has its specific error, because the non-plane parallelity of the two mirrors causes deviation from the exact zero situation. To determine the index-error, a sharply-edged object is made to coincide with itself, the object being farther than  $2\frac{1}{2}$  km away, preferably on the horizon, and the deviation from the exact zero reading indicates the index-error. Index-correction is the value of the index-error, but with changed sign (see Figure 2.2.2).

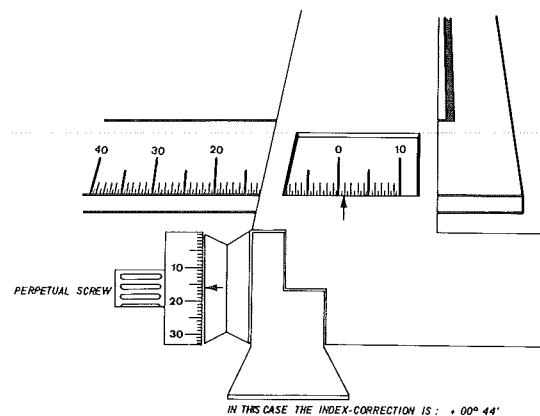


Figure 2.2.2 Example of Index-correction

Advantages: Angles can be measured with the instrument held in the hand, which is very useful aboard ship where a theodolite on its tripod cannot be used. In a minor triangulation in the bush or elsewhere on land, the sextant can be used for measuring angles from difficult places, such as trees and masts.

Disadvantages: The accuracy of the sextant is small compared with the theodolite. The Freiberg sextants used by the Mission were accurate to half an arc-minute. This gave sufficient accuracy for position-fixing during measurements inside an existing triangulation network, but little for building up a minor triangulation.

### 2.3. THEODOLITE

The theodolite is used to measure directions to two or more objects, together with the inclinations of these objects, all referred to the horizontal plane passing through the observation point. From these measurements horizontal and vertical angles are obtained.

The theodolite consists of the following main parts: a fixed base with tribrach, a movable upper part, and a telescope (see Figure 2.3.1). The upper part is movable about the vertical axis, and the two brackets for the horizontal axis are fixed, bearing the telescope. For rough levelling-up the base is fitted with a circular bubble, while more accurate adjusting is done with the alhidade tubular level. The telescope can be aimed in any direction, rotating about its horizontal axis and moving the alhidade about its vertical axis.

#### IV, 2.3

Advantage: The projection of an angle on the horizontal plane can be measured directly and irrespective of the difference in elevation between the observer and the objects.

Disadvantage: the theodolite has to be levelled-up accurately, and is therefore of no use aboard any moving vessel.

Use: Triangulation measurements and line of sight deviation measurements from a fixed point.

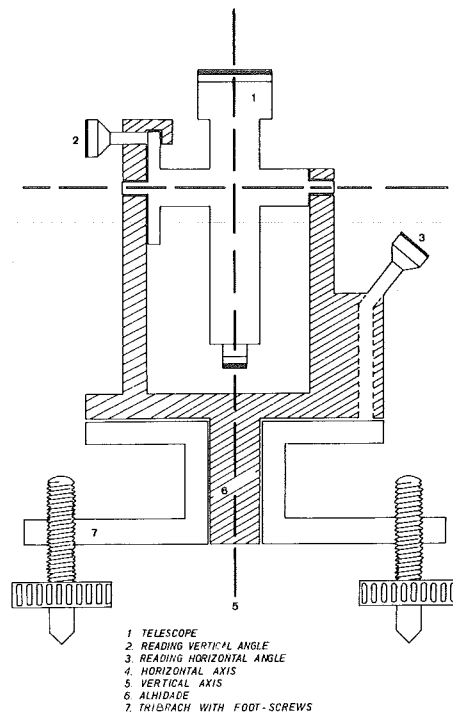


Figure 2.3.1 Principle of the Theodolite

#### 2.4. LEVELLING INSTRUMENT

The levelling instrument is used for determining the difference in elevation between two or more points. The instrument consists of a telescope of which the axis can be set horizontal by reference to a tubular level. The telescope and bubble-tube are fixed together to one rigid unit that rotates around the vertical axis by means of the alidade. The vertical adjustment of the alidade can be made by the three foot-screws on the tribrach that holds the alidade, and is fitted with a circular level. Final horizontal adjustment of the telescope is done with the tilting screw, in accordance with the bubble-tube. When an automatic levelling instrument is used, it works automatically by an optical-mechanical device acting under the influence of gravity or magnetism.

Accuracy: Depends, in principle, on the sensitivity of the level bubble, the magnitude of the telescope, and the climatological conditions. In general, the following directions should be carried out to secure a high accuracy:

- Levelling during the middle of the day should be avoided.
- The levelling instrument should be protected against the sun by a parasol.
- The three horizontal cross-hairs should always be read, and not the centre one only.
- Sighting distances should not exceed 50 metres.
- Special ground plates should be used to keep the levelling staff steady, specially when turned.

Use: Determining the difference in elevation between two or more points, such as the slope of a river, or connecting a gauge-zero with a benchmark with known elevation.

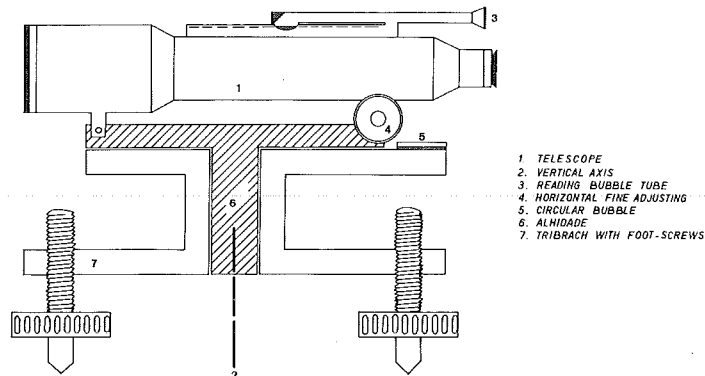


Figure 2.4.1 Principle of the Levelling Instrument

## 2.5. RANGEFINDER

This instrument determines the distance from observer to a certain object directly, and is based on the principle of the human eyes, only the base distance between the telescopes being 80 or 100 centimetres. One telescope is fixed under  $90^{\circ}$  to the base, and the other is movable and should be turned till the picture seen by this telescope coincides with that seen through the fixed telescope. The amount of turning indicates the distance to the seen object, and is directly shown on a dial.

The Barr & Stroud rangefinder used by the Mission had a base of 80 cms and showed both coinciding images in an upright position.

Advantages: The instrument gives a direct reading of the distance between the observer and a certain object; it can be used by hand without tripod; and adjustment is done easily for distance correction and image, while also a check on the accuracy of the instrument can easily be done. Finding the distance to a light at night is made easier by using the astigmatiser, showing the light in both images as a vertical light-ray.

Disadvantages: Because the system is based on the principle of the human eyes, it is clear that only up to a certain distance is the accuracy fair, and lessens with increasing distance.

Accuracy: Under reasonably practical conditions less than one metre up to distances of 250 metres,  $2\frac{1}{2}$  metres at 500 metres' distance, and so on (for Barr & Stroud rangefinder with a base of 80 cm), the error (according to the manufacturer) varying in proportion to the square of the distance.

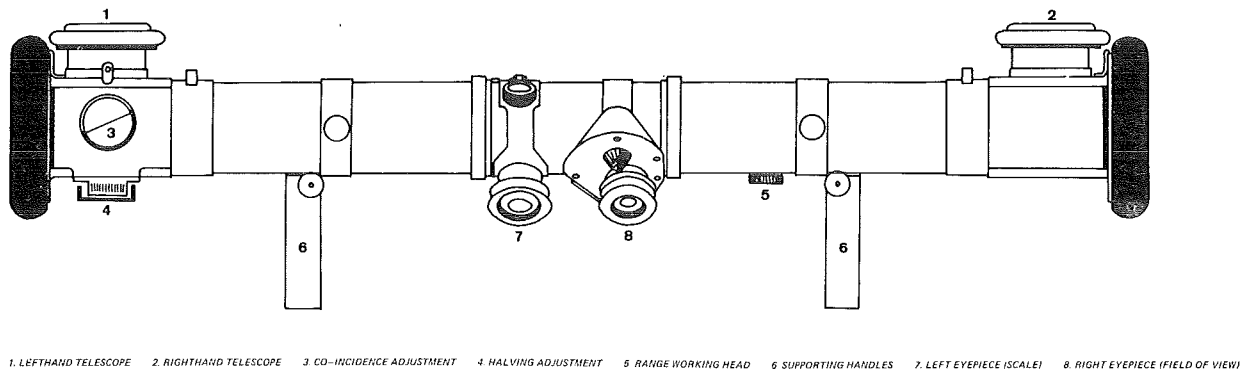


Figure 2.5.1 Barr & Stroud Rangefinder

Use: The rangefinder can be used for measuring a river-width, for position-fixing while sounding a river cross-section, and in combination with a sextant or theodolite for locating the position of floats from ashore (see Chapter 3.2).

The distance scale of the Barr & Stroud rangefinder starts only at 50 metres, but with the aid of a small board as beacon, the distances between 0 and 50 metres can also be determined. The board should be 80 centimetres long if the rangefinder's base is 80 cms, and divided into five parts of 16 cms each, as shown below. This board should be fixed on the beacon from which smaller distances than 50 metres are to be measured.

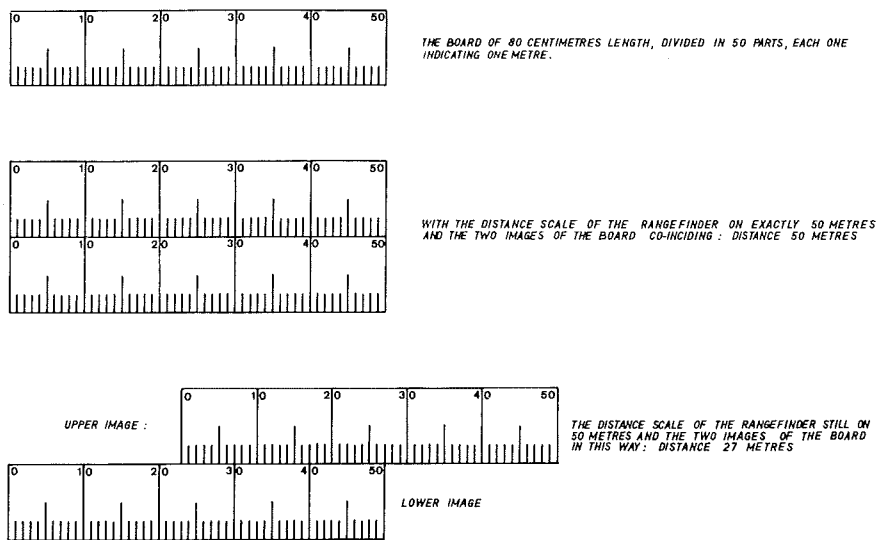


Figure 2.5.2 The Use of the Short-distance Board

2.6. ECHO-SOUNDER

The echo-sounder is an instrument to measure the water depth by means of sound waves. These are transmitted through and received by the transducer and recorded on the paper roll of the recording device, indicating the depth directly in metres or feet (depending on what system is used).

The transducer is located some feet below the water-level and as the distance between the bottom of the transducer and the reflecting surface of the river bottom is measured, the draught of the transducer should be added to the recorded water depth, thus giving the correct water depth. The transducer may be inboard, built in the hull of the vessel, or outboard, fixed to the ship with a bracket.

The echo-sounder used by the Mission was a DE-719 Raytheon Survey Fathometer, recording the depth in feet. Frequency of the transmitted sound waves was 200 kc and it operated on a 12 V battery. It had a single range of 205 feet in four phases (from 0 - 55, 50 - 105, 100 - 155 and 150 - 205 feet), and a double range of 410 feet also in four phases. (A similar, although older type of Raytheon echo-sounder was borrowed from ADENAVI and mostly used on the Canal del Dique.)

The instrument was fitted with adjustable zero recording, tide and draught correction, speed of sound control, phase indicator and calibration control.

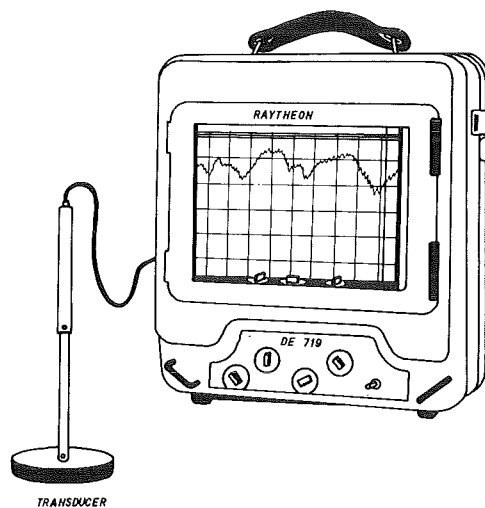


Figure 2.6.1 Recording Device and Controls of Echo-sounder

**Advantages:** The recorded water depth can be directly corrected, until the true water depth (from surface to bottom) is recorded. Also the battery condition can be controlled by means of the calibrating line. It also gives variable paper speed for the different types of soundings, with river cross-sections having the highest paper speed, and longitudinal soundings a lower speed.

**Disadvantages:** The echo-sounder has to be checked after each sounding, daily, and calibrated by means of a bar-check or a hand-lead and the echo-sounder corrected according to these data. The speed of sound is influenced by the water temperature and salinity, and also the condition of the battery.

**Use:** The echo-sounder is used for cross-sectional soundings of the river, longitudinal river-soundings, and local complete soundings of a part of the river or bay.

**Accuracy:** The Raytheon echo-sounder records on specially-graduated paper, indicating lines at 1 foot interval, thus making interpretation of half a foot possible.

#### IV, 2.7

Also used by the Mission was a small portable echo-sounder, "Seascribe", specially for small local soundings and shallow water investigations. This instrument is fitted with a flash-indicator as well as recording paper, and has a separated underwater transmitter and receiver. The unit operates on 8 to 24 V battery power or dry cells. The Seascribe portable echo-sounder records on small-scale paper, because the whole unit is small, and is therefore not very accurate. Still, for small and quick investigations it works accurately enough, making readings of one foot possible.

On both instruments a fix-marker is fitted, marking the paper with a thin vertical line whenever a position-fix is made. By giving these fix-marks the same running number as the location-fixes of the sounding, elaboration of the data afterwards is facilitated (see Chapter 4.2).

#### 2.7. PENDULUM CURRENT METERS

The pendulum current meter is used to measure the current velocities in a river or canal, and is based on the principle that a metal body, suspended by a thin wire from a measuring device, is moved by the current out of a position vertically below its point of suspension. With the help of calibration curves the angles read on the measuring device can be translated into velocities. Corrections have to be made for the bending of the wire. Several bodies of different shape and weight belong to the current meter set, each to be used in a matching range of velocities.

Two different types of pendulum current meters (based on the same principles) exist:

##### Planeta

In addition to the vertical angle read from the measuring cupola, a horizontal angle can be read, indicating the direction of the current. A total range of 0 - 3.5 m/s can be measured at various depths and with different bodies. This instrument, however, was not used by the Mission.

##### K.L.M.

This type of pendulum current meter weighs less than the Planeta, and can easily be held in the hand during the measurements. But since it is manually used, the heavier bodies of the Planeta for measuring velocities up to 3.5 m/s cannot be used, and only a total range of 0 - 1.9 m/s can be measured.

Advantages: Both instruments can be easily repaired and maintained in the field, and only a relatively cheap part (the body) can be lost. The instruments give a true recording of the water movement, and the observer is warned immediately if something is going wrong: for instance, if the vessel is moving due to wind (or if the body is lost). A change in the direction of the current in the vertical is measured easily with both instruments because of the change in wire-bending (important in tidal areas). The K.L.M. only measures the current velocity, but with the Planeta the current direction can also be read.

Disadvantages: The translation of vertical angles into current velocities is a rather elaborate process.

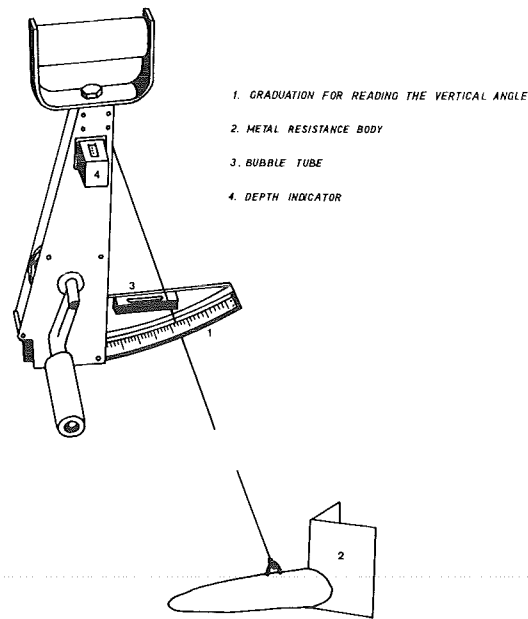


Figure 2.7.1 The K.L.M. Pendulum Current Meter.

## 2.8. PROPELLOR CURRENT METERS

The propellor current meter assesses the local velocity by counting during a certain interval of time the number of revolutions of a propellor driven by the current. The counting is done electrically, while a translation of the number of revolutions into velocities is made from calibration curves or formulas supplied with the instrument.

Three types of propellor current meters were used by the Mission: Ott, Kelvin-Hughes and Berg & Berg.

### Ott

The Mission used both the Ott-Arkansas and the Ott-Unstrutdag, the latter being equipped with a special bottom indicator which signals contact with the river bottom. Each type of current meter had a number of calibrated propellers for different velocities. Theoretically the Ott current meters are suitable for measuring velocities as high as 10 m/s and as low as 0.05 m/s. However, in practical use, with high velocities the instrument will shear too much, which may cause irregularities in the measurements.

**Advantages:** Propellor current meters are excellent tools for fast and accurate work, provided careful maintenance is done, great care is exercised when using the instruments, and the calibration is checked regularly. Translation from revolutions into velocities is done quickly by means of graphs.

**Disadvantages:** These types of meters do not provide information on the direction of the current. Due to its weight, a derrick or davit is required for handling the instrument.

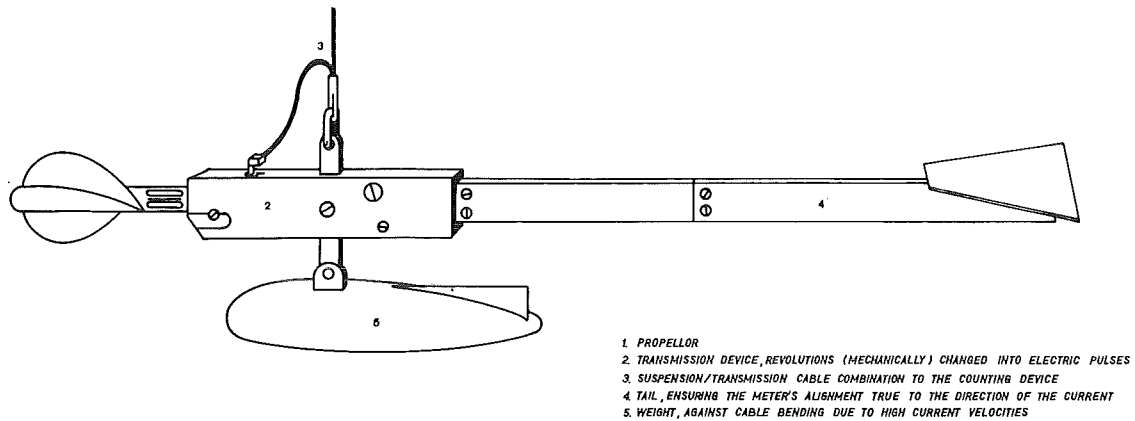


Figure 2.8.1 Ott-Arkansas Propellor Current Meter

Kelvin-Hughes

This direct-reading current meter measures both the velocity and direction of the current, and no special interpretation by curves or graphs is necessary. The instrument consists of an underwater body with propellor, an electricity and suspension cable, and the measuring device with dials for velocity and direction. The Kelvin-Hughes current meter has two velocity ranges: from 0.05 - 1.5 m/s and from 0.05 - 3.0 m/s respectively.

Despite trials and repairs the current meter was not in working order and some spare parts should be ordered from the manufacturers.

Berg & Berg

Although this may not be called a regular propellor current meter, the basic principle is similar. Instead of a propellor with a horizontal axis, the counter is driven by six cups on a vertical axis. Due to its light weight, the Berg & Berg current meter can easily be lowered by hand during the measurements and was often used by the Mission for discharge measurements in secondary branches of the Rio Magdalena and in affluents. Theoretically the Berg & Berg current meter can measure velocities as low as about 0.10 m/s and as high as about 2 m/s, though the heavy suspension cable causes a considerable drift of the instrument in water velocities exceeding 1 m/s.

2.9. WATER SAMPLER

The water sampler used to measure wash-load concentrations is a weighted device in which a bottle, closed by a rubber stopper, can be placed. This contraption suspended by a line is lowered to the required depth, and the rubber stopper pulled off by means of the thin line fixed to it. After sufficient time has elapsed for the bottle to fill, the instrument is hoisted and the bottle taken out, closed and labelled.

Disadvantage: The water sampler disturbs the flow pattern and consequently cannot be used to measure the total sediment-load transported by the river. The wash-load, however, consisting of very fine particles, is less affected by the distortion of the flow and may accurately be estimated by the elaboration of water samples collected in this way.

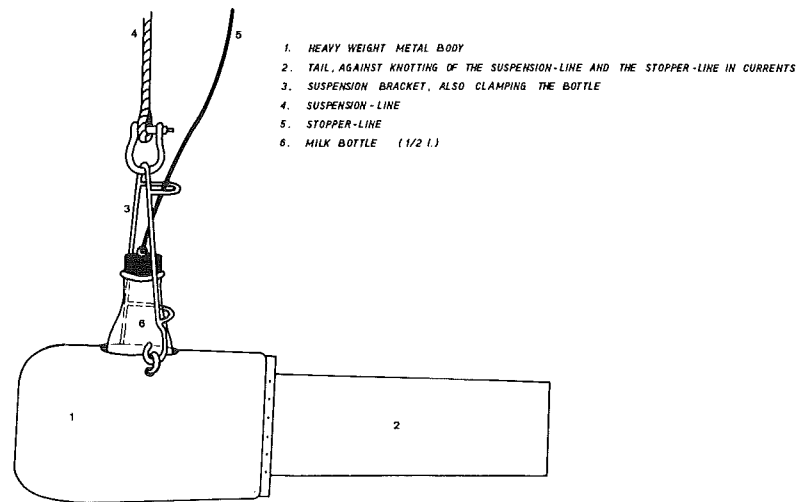


Figure 2.9.1 Metal Water Sampler

Another type of water sampler (based on the same principle) used by the Mission was a bottle-shaped perspex body of little weight, and with an opening on top which could be closed and opened by the above-mentioned rubber stopper on a line. With a locally-acquired weight connected to its bottom, the instrument could be used in the same way as the other sampler.

Advantage: The light weight of this instrument makes it possible to carry it around in hand-luggage, and to use it in preliminary investigations.

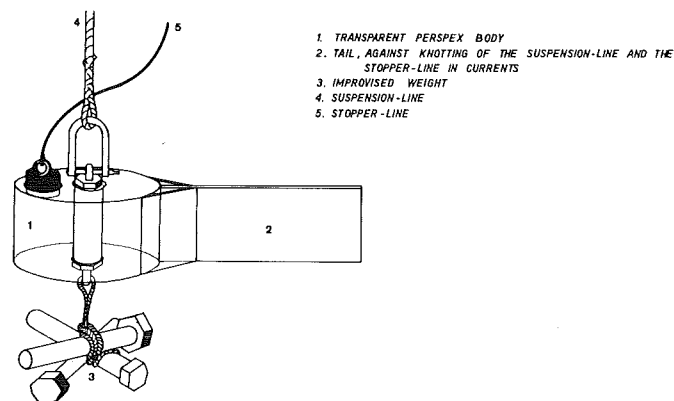


Figure 2.9.2 Perspex Water Sampler

## 2.10. DELFT BOTTLE

The Delft Bottle (D.F.) is an instrument to measure suspended-load in rivers. It measures from the surface down to 0.5 m above the river bottom when suspended by a wire, and from 0.5 to 0.05 m above the river bottom when fixed in a frame (sledge). In the frame the D.F. is tilted, and the use of bent nozzles is required for the depths of 10, 20 and 30 cms above the river bottom.

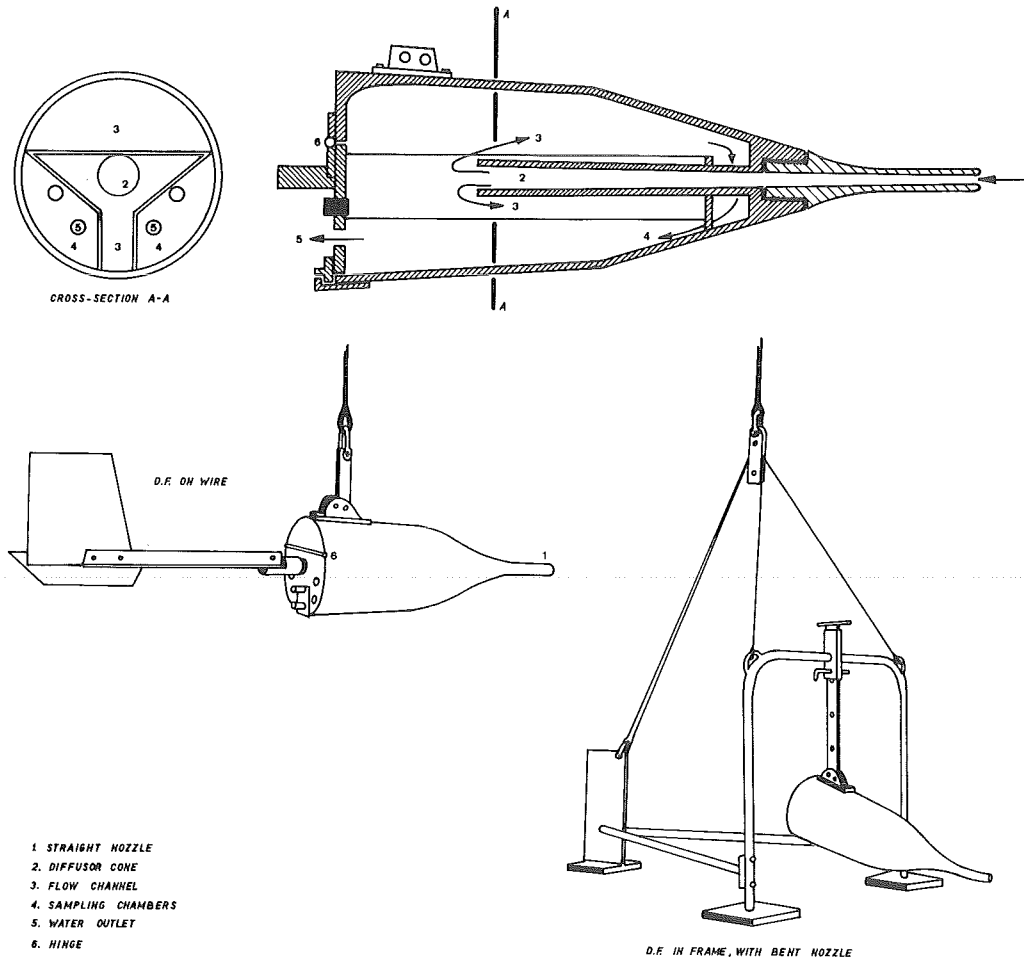


Figure 2.10.1 The Delft Bottle (D.F.)

Principle: The sediment-containing water flows through a bottle-shaped sampler, the shape of which induces a low pressure at the rear end (water outlet) in such a way that the water enters the mouth of the sampler with almost the same velocity as the undisturbed flow. The inside shape of the sampler and the sharp decrease of the velocity in the wide sampling chambers causes the sediment material to settle there. This settled material can be taken out and measured volumetrically after the D.F. is out of the water.

It is possible with the D.F. to measure sediment transport with velocities up to 2.5 m/s, although the correction factor increases considerably for such high velocities (see Chapter 4.4). The average grain size of the sediment must exceed 0.05 mm (50  $\mu$ m).

Advantages: Because of the flow-through principle, a large volume of water is sampled, and it is a direct transport measurement. The D.F. is of simple and sturdy construction, can easily be maintained in the field, and can be used at any required depth.

Disadvantage: Because of its weight a davit and winch are required for handling.

### 2.11. BED-LOAD TRANSPORT METER "ARNHEM"

The Bed-load Transport Meter "Arnhem" (B.T.M.A.) is an instrument to measure the bed-load of coarse sand and fine gravel just above the river bottom.

Principle: A frame-mounted sampler is pressed on the river bottom by a leaf spring. Behind the mouth of the sampler is a fine-meshed wire basket. The shape of the basket causes a low pressure behind the instrument in such a way that water and the transported bed material enter the mouth with the same velocity as that of the undisturbed flow. The bed-load particles which are too coarse to pass the meshing are caught. The B.T.M.A. catches material coarser than  $300\ \mu\text{m}$  (theoretical value of the meshes) and finer than  $5\ \text{mm}$  ( $5,000\ \mu\text{m}$ ).

Advantage: The instrument is of simple and sturdy construction, and can easily be repaired and maintained in the field.

Disadvantage: Because of its weight and dimensions, a davit and winch are required for handling it. The current velocity range in which it can be used is limited to  $2.5\ \text{m/s}$  due to the construction of the basket.

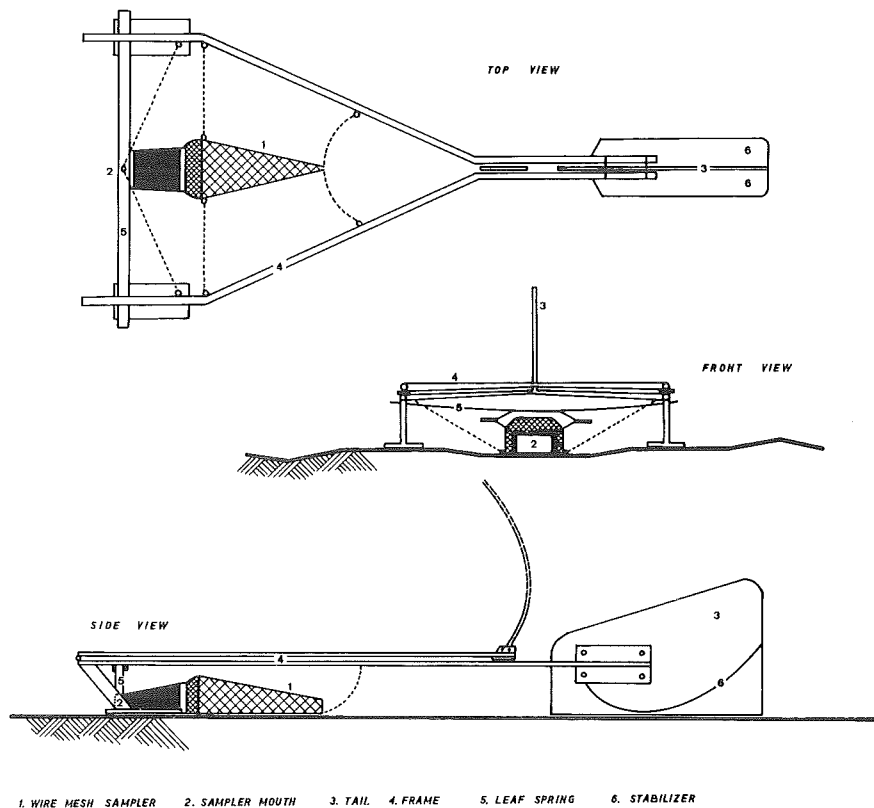


Figure 2.11.1 The B.T.M.A.

### 2.12. BOTTOM GRAB

Although not belonging to the range of instruments to measure the bed-material load transported in rivers, this device is useful to determine the material of which the river bed is composed.

#### IV, 2.12

The sampler consists of a grab which is lowered in an open position by a line. Contacting the bottom, the lever that keeps the grab in an open position disconnects and while hoisting, the grab is closed and holds the bottom sample.

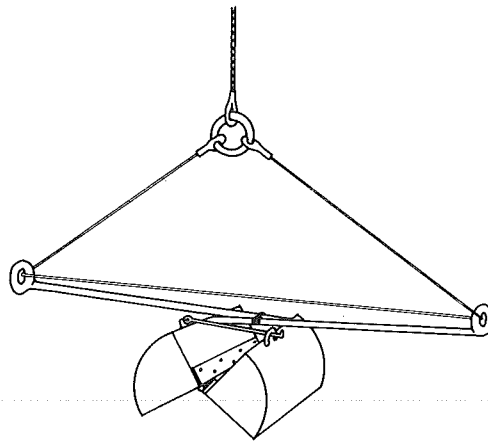


Figure 2.12.1 Bottom Grab

Advantages: The bottom grab is of simple and sturdy construction and can be easily maintained in the field, while no davit is required to lower the instrument.

Disadvantages: In strong currents it may be difficult to lower the sampler vertically, and if it lands on the river bottom on its side it will not grab a sample. It may then be easier to take a sample while the survey vessel is adrift.

#### 2.13. TURBIDITY METER

The turbidity meter is an optical instrument to measure the concentration of fine suspended sediment either in water samples or directly in rivers, according to the principle of determination of light-absorption by a sample of river-water compared with the absorption of a clear water sample. The latter should preferably be of the same river water, but cleared by filtering the silt out of it (the silt then being dried and weighed). A calibration curve must be made by plotting the concentrations found by filtering and weighing of the samples versus the extinction figures of the same samples found by the turbidity meter.

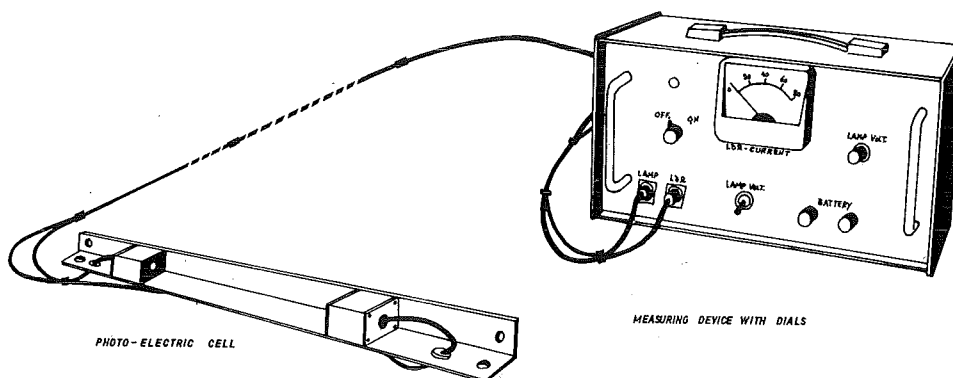


Figure 2.13.1 Turbidity Meter

The turbidity meter used by the Mission had a two-fold application: it could be used in the office laboratory for elaborating water samples, and it could be used in the field by lowering the photo-electric cell on a line to the required depth in the river, and the value of light-absorption then read on the measuring device aboard the vessel.

#### 2.14. CONDUCTIVITY METER

This instrument is used to measure the conductivity of the water, by means of which the salinity can be determined. The instrument consists of a measuring-cell on a combination suspension/electrical cable, which is lowered either in a water sample or the river itself. Readings are taken from the device aboard the survey vessel, which indicate the conductivity of the water in micro-siemens. By means of conversion tables these values can be translated into ppm salinity. When measuring in a sample, the sample must not be too small, to avoid influences of the container walls. The conductivity meter used by the Mission was an E.C.R. type P4E.

Advantage: The salinity in a measuring vertical at several depths can easily and quickly be determined.

Disadvantages: This type of conductivity meter was, according to the manufacturer's statement afterwards, not really suited for measurements in tropical waters, due to sensitivity to high temperatures of the electronic part of the instrument (see also Chapter 4.5).

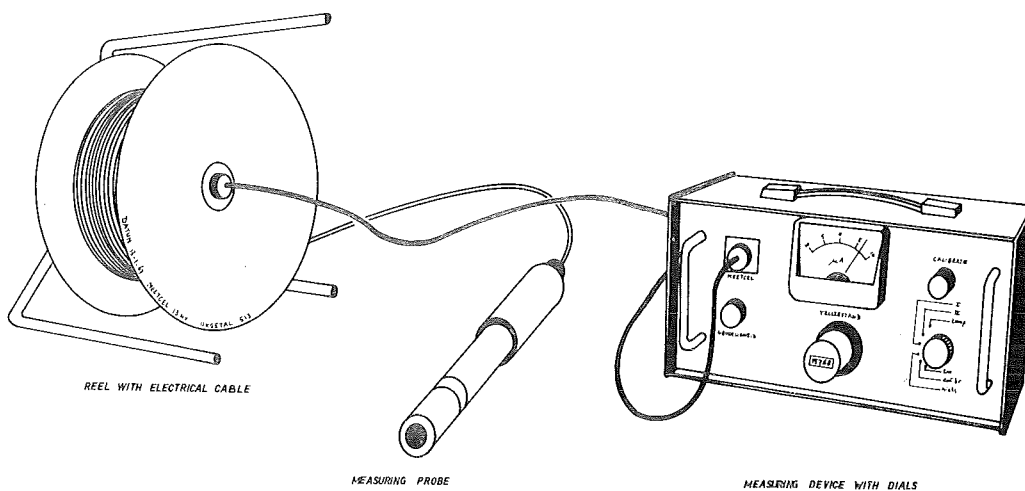


Figure 2.14.1 The E.C.R. Conductivity Meter

#### 2.15. VISUAL ACCUMULATION TUBE

The VAT can be used to determine the particle-size distribution of sand with a median particle-size of  $d = 0.05 - 0.4$  mm ( $50 - 400$   $\mu\text{m}$ ). The method is based on the determination of settling-time distribution of a small quantity of sand ( $0.2 - 1.0$  gram) in a settling tube. For this purpose the increase of the deposited quantity of sand is measured as a function of the time. By means of a calibration the grain-size of a certain percentage can be determined by the settling-time of that certain percentage (St. Anthony Falls (1957) [62]).

The VAT consists of a settling tube B (see Figure 2.15.1), inside diameter about 25 mm, with on top a cup A which is separated from the tube by a clamp-device on a rubber hose.

#### IV, 2.15

Under the settling tube is a capillary tube C in which the deposit can be measured with a measuring tape behind the capillary tube.

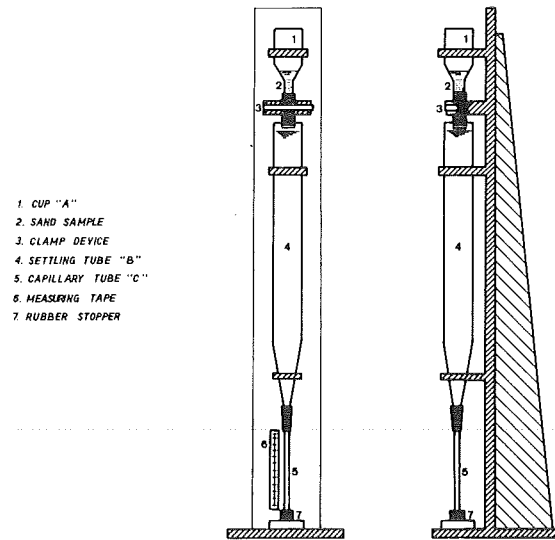


Figure 2.15.1 The V.A.T.

#### Procedure

1. Clean the cup A and settling tube B with water while the clamp device is open (after each measurement).
2. Close the capillary tube with a rubber cork in such a way that the top of the cork is level with a grid line on the measuring tape.
3. Fill the settling tube B via cup A with water up to 2 - 3 cm below the top edge of tube B, and measure the water temperature.
4. Close the clamp device.
5. Put the sample in cup A and pour some cc water on this sample so that all the sand is under water.
6. Open the clamp device and start the stop watch at the moment the sand begins to fall in tube B.
7. Note the time when the first sand grains reach the bottom (top of rubber cork) and subsequently the times when the height of the deposit reaches, for instance, 0.5, 1.0, 1.5 cm, etc.
8. Determine after 10 minutes the total height of the deposit and calculate the percentages corresponding with the height of 0.5, 1.0, 1.5 cm, etc.
9. Determine the grain-size from the settling-time corresponding with the percentages.

#### calibration

The VAT in the described form and with the described procedure is calibrated with a number of sand samples at a temperature of 29°C. The calibration is given in Figure 2.15.2.

This line is determined by using an effective length of 150 cm and the settling rate of sand (St. Anthony Falls, 1957 [63]). Any change in the instrument needs a re-calibration. The accuracy of the size determination is  $\pm 5 - 10 \%$ .

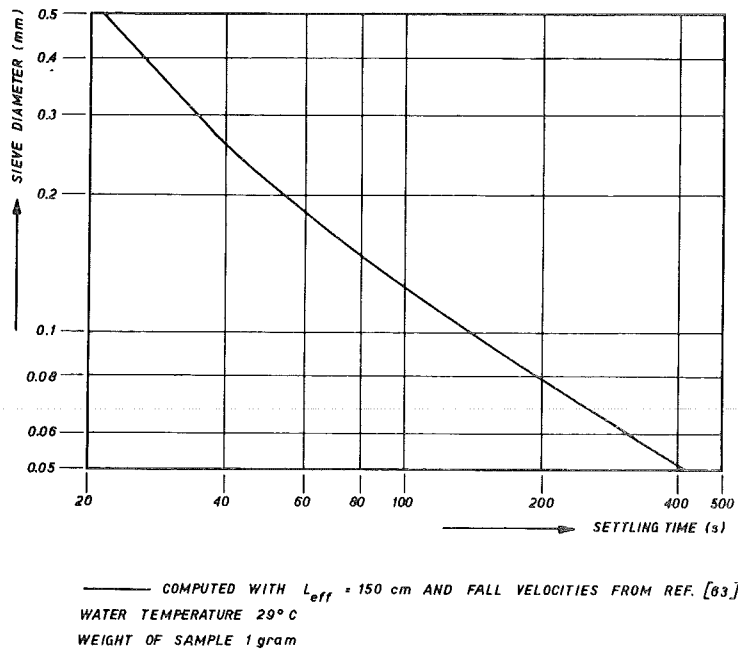


Figure 2.15.2 The V.A.T. Calibration Curve

The calibration is done at  $29^\circ \text{C}$ ; for deviations in temperature the following correction factors for the size should be used:

D	$20^\circ \text{C}$	$25^\circ \text{C}$	$29^\circ \text{C}$	$40^\circ \text{C}$
50-500 $\mu\text{m}$	1.095	1.045	1.00	0.91

thus:  $D_{20^\circ \text{C}} = D_{29^\circ \text{C}} \times 1.095$ .

## Chapter 3

### LOCATION FIXING

It will be clear that during any kind of measurement in a river the position or location fixing is very important. Two types of position fixing can be distinguished:

- First, the system in which several main beacons or conspicuous points as towers, chimneys etc. are related to each other by means of a triangulation network; and
- secondly, the local position fixing within this network involved in any measurement.

#### 3.1. TRIANGULATION

The first system consists of a network preferably of triangles formed by beacons or temporary beacons (Figure 3.1.1). The part of the river in which the measurements are to be done is divided by well-chosen points into triangles, and beacons or temporary beacons are placed at those points. The triangles should, if possible, be equilateral or similar, and the visibility from one point to another secured, as well as to the points of the two adjoining triangles.

All angles of all triangles - preferably in overlapping sets of two triangles, hence eight angles, together forming a quadrangle - are then measured by means of a theodolite, or if the use of a tripod is impossible, with a sextant.

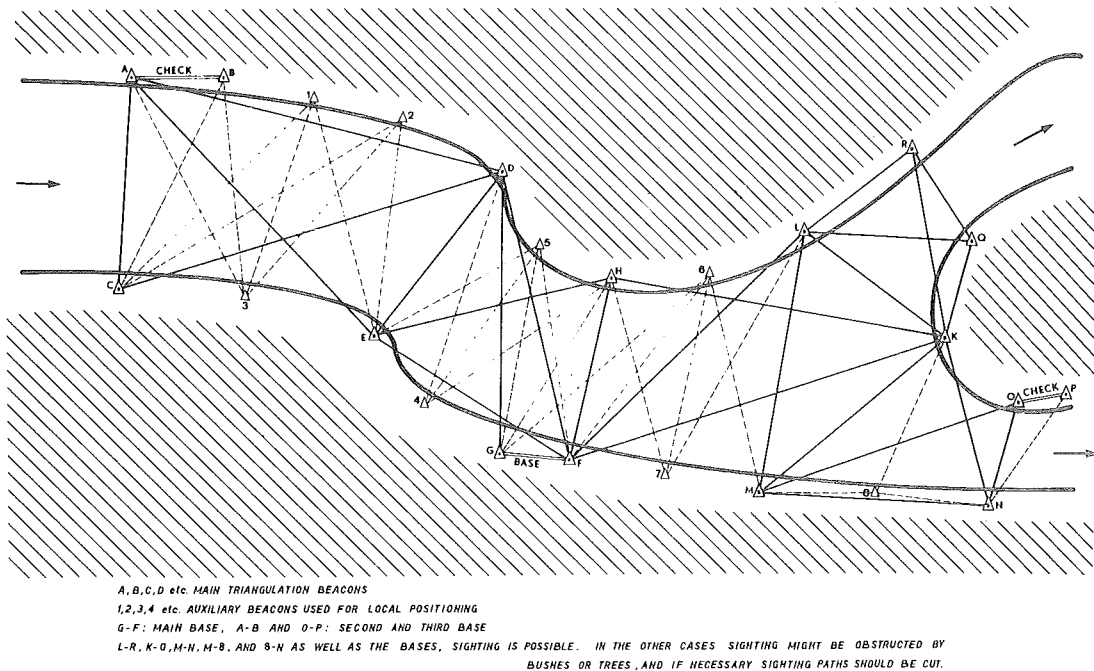


Figure 3.1.1 Example of a Triangulation Network along the Río Magdalena

Before the angles of a single triangle are used in the sine formula (main formula for elaborating a minor triangulation) they must be made to add up to exactly  $180^{\circ}$ . If the angles are assumed to have been equally well observed, the excess or deficiency should be distributed amongst them equally. If they have not been well observed equally, the excess or deficiency should be distributed in inverse proportion to the estimated accuracy of each angle. If a triangle forms part of a more complicated figure (preferably quadrangle), the adjustment of its angles is carried out as part of the adjustment of the whole figure (quadrilateral adjustment).

When every triangle is adjusted until its angles total  $180^{\circ}$ , as well as in other triangles or quadrangles integrated and each triangle still totalling its angles  $180^{\circ}$ , all sides can be calculated according to the sine law, starting from the centre base and elaborating both ways up to the auxiliary bases at the extreme triangles, which serves as check on the whole calculation.

Plotting all triangulation points graphically, a so-called basic chart is formed on which the local positions of measurements can afterwards be plotted, related to the main triangulation points.

It is clear that whenever in a minor triangulation network an existing triangulation point with known co-ordinates of a major network is met, the minor network should be transversed into the existing system of projection, thus given co-ordinates of the nationally used system. In all minor triangulation networks, however, set up during the Rio Magdalena survey, sufficient data could never be gathered to converse the co-ordinates to the nationally used system, U.T.M. Hence all maps and charts produced by the Mission have their own local co-ordinate grid.

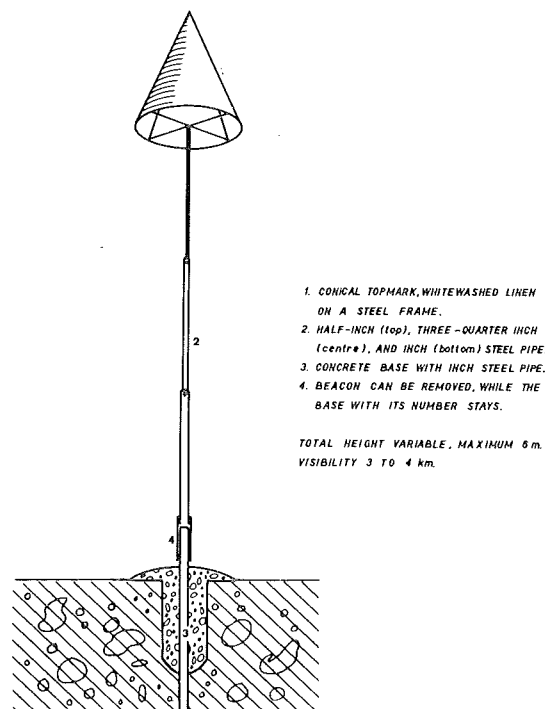


Figure 3.1.2 Triangulation Beacon Used by the Mission

#### IV, 3.2

For the base measurement (length between the two base points) a suitable stretch, preferably on open level ground, must be found as near as possible to the middle of the area to be triangulated. The length of the base should be between 0.5 and 0.3 of the mean side length of the triangles, as the triangles of this type of river triangulations often are very small.

In the two extreme triangles of the area a second and third base should be measured as a check on the accuracy with which the scale has been transferred throughout the work.

### 3.2. LOCAL POSITION FIXING

The following systems are used to determine the surveyor's position within an existing triangulation network, or related to known points along the river.

#### 3.2.1. With two sextants

Two horizontal angles between three beacons are measured simultaneously with two sextants from the survey vessel by two observers. Generally these angles should not be less than  $10^{\circ}$  to  $15^{\circ}$  and not exceed  $125^{\circ}$  (maximum capacity of a sextant). When the intersection of the arcs of the measured angles is still reasonable, however, smaller angles are allowed.

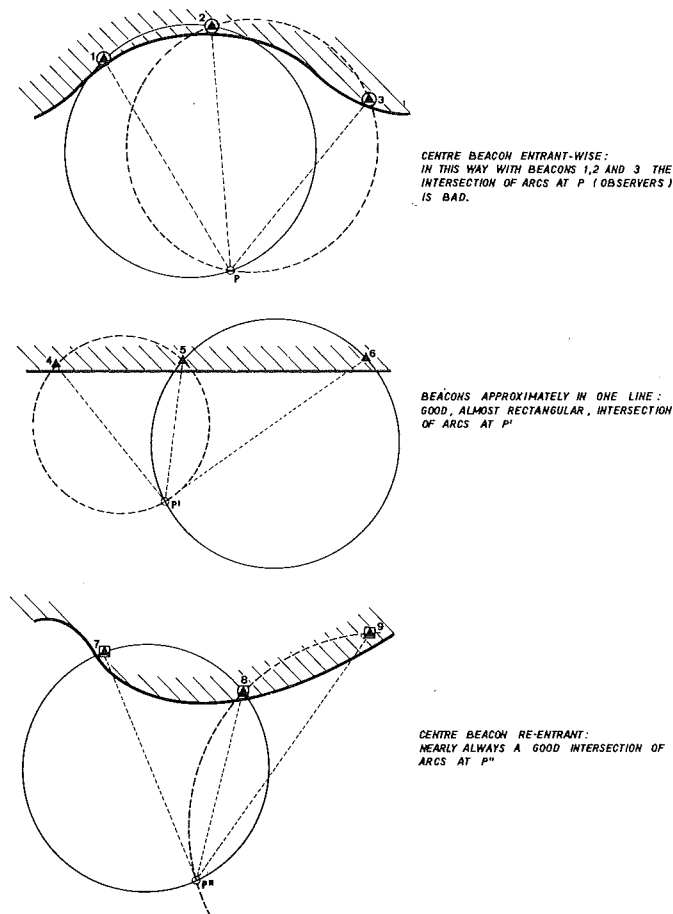


Figure 3.2.1 Examples of Intersection of Arcs

The principle of this system (Snellius problem) is that the locus of an observer who measures two beacons under a certain angle is the circle, passing through the two beacons and the observer, from which on any point the two beacons are measured with the same angle. The intersection of two circles passing through three beacons and the observer gives the exact position of this observer.

Preferably the three beacons should be approximately in line with each other or with the centre beacon re-entrant.

Advantages: As long as the intersection of arcs is good ( $30^\circ$  or more), the determined position is accurate. Measurements are done quickly with the sextants from a moving vessel.

Disadvantage: Subsequent plotting of the positions is rather elaborate by using the graphical method (construction of the arcs through the beacons), or the help of a station pointer is needed. This instrument consists of a protractor with three legs with which the two measured angles can be set. By putting the legs along the respective beacons on the map till they coincide with these beacons, the position is determined.

Use: This system of measuring two adjoining angles with two sextants can be used while sounding, as positions are frequently needed. Two experienced observers can simultaneously measure angles every minute or 30 secs. from aboard the sounding vessel, determining their position every 25 metres or so, depending, of course, on the speed of the vessel (see also Chapter 4.2).

### 3.2.2. With one sextant and a leading line

This system can be used by one observer only, who has to be exactly in direct line with two beacons ashore (leading line) and who measures the angle between the front one of those beacons and a third beacon. This angle should be between  $30^\circ$  and about  $115^\circ$ .

Advantage: This system can be used by one observer only, and is very accurate.

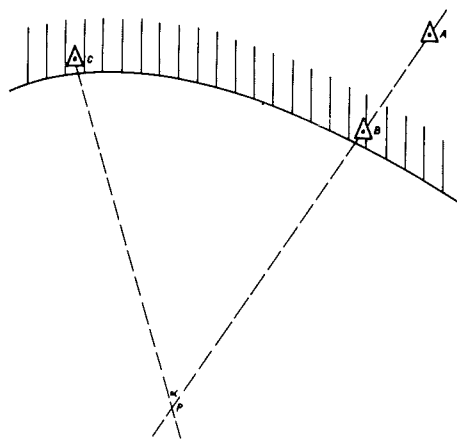


Figure 3.2.2 Positioning by One Sextant and a Leading Line

IV, 3.2

3.2.3. With one rangefinder and a leading line

The observer has to be in direct line with two beacons, and measures the distance to the front one.

Advantage: Only two beacons ashore are sufficient for this system.

Use: While sounding a single cross-section, or during measurements with other instruments at different points in a single cross-section.

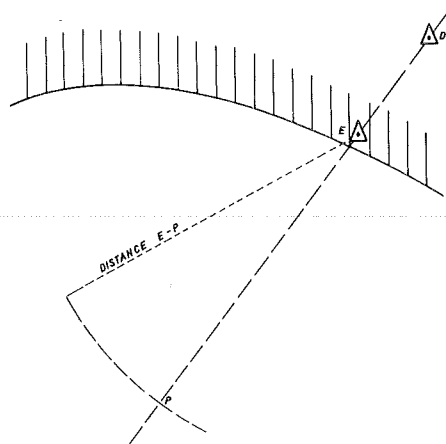


Figure 3.2.3 Positioning by One Rangefinder and a Leading Line

3.2.4. With one sextant and a rangefinder

The observer measures the horizontal angle  $\alpha$  between the two beacons F and G. A second observer measures simultaneously the distance to one of the beacons, choosing the beacon that is in best position for making a good intersection of distance and arc-angle possible.

Use: Only when two beacons ashore are available for measuring.

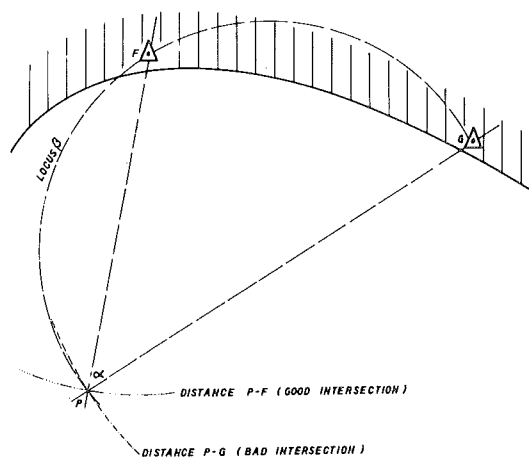


Figure 3.2.4 Positioning by One Sextant and a Rangefinder

3.2.5. With one compass and a sextant

The observer measures the horizontal angle between two beacons ashore and the second observer measures the magnetic direction to one of those beacons in such a way that a good intersection is obtained.

Disadvantages: This method is less accurate because the accuracy of a compass is limited and because the deviation of the compass has to be known (this deviation may vary with the position of the observer aboard the survey vessel). Also the magnetic North has to be determined by taking a compass bearing from the first to the second beacon.

Use: This system should only be used when no rangefinder is available and only two beacons ashore sighted.

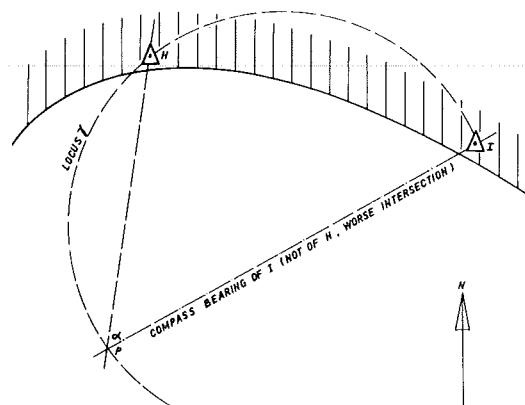


Figure 3.2.5 Positioning by One Compass and a Sextant

3.2.6. With one compass and a rangefinder

The first observer measures the distance to the beacon while the second observer takes a compass bearing on it.

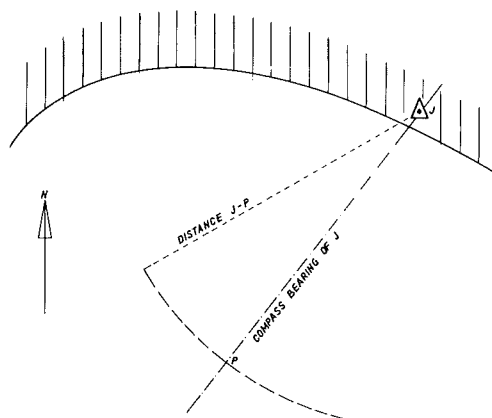


Figure 3.2.6 Positioning by One Compass and a Rangefinder

IV, 3.2

Advantages: Always rectangular intersection, and only one beacon is needed.

Disadvantages: Again the deviation of the compass should be known as well as the magnetic North in relation to the beacon.

Use: When only one beacon is available.

3.2.7. With one compass only

This system can only be used when the observer remains immobile for some time, because he has to take two compass bearings of two beacons (Figure 3.2.7). Doing this with two observers and two compasses (if there should be available two compasses and nothing else) is very inaccurate, because of the differing variations of the compasses.

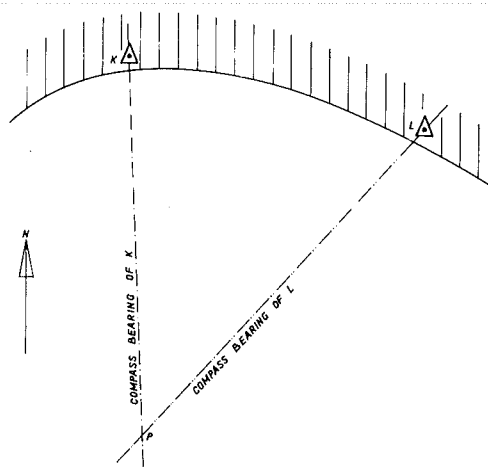


Figure 3.2.7 Positioning by One Compass

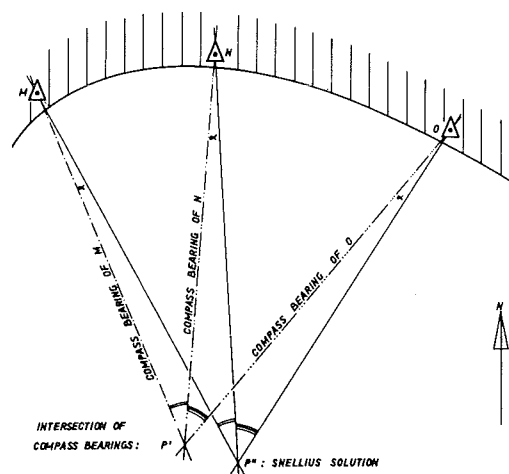


Figure 3.2.8 Positioning by One Compass, and the Determination of the Compass Deviation ( $\alpha$ )

By three compass bearings of three beacons, however, being taken by the same observer with only one compass, the deviation may be eliminated. (see Figure 3.2.8). By subtracting the bearings from each other, two angles remain, and position plotting can be done with the Snellius method (see Para. 3.2.1). (As the variation in deviation in this system is very small, the deviation is almost eliminated). By plotting the position according to compass bearings against the determined magnetic North direction and according to the Snellius solution, the deviation of the used compass can be determined.

Advantages: Determination of the compass deviation, and the possibility that the problem can be solved by the Snellius method.

Disadvantages: The determination of the position is rather elaborate, and this system can only be used when the position of the observer does not change.

Use: When only one compass is available and no other instruments (or for the determination of the deviation of the compass and the magnetic North direction).

## Chapter 4

### MEASUREMENTS AND ELABORATIONS

This Chapter describes the various measurements carried out by the Mission and their method of elaboration. For more detailed information about the instruments used with these measurements, reference is made to Chapter 2.

#### 4.1. WATER-LEVELS

##### 4.1.1. Introduction

Water-levels may be considered to be the basis for any river study. Most kinds of measurements have to be related to river stages in one way or another to make judgment of these measurements possible. It should be kept in mind, however, that in reality the discharge of a river is a better basic information than the water-level, and if it would be possible to measure the discharge daily or even several times a day at many places, this would be preferable (Part II, Para. 2.3.1).

Water-levels are obtained from gauges, either recording gauges or staff gauges, and serve several purposes. Water-levels plotted versus time, during a year, form the hydrograph (Part II, Para. 2.3.2). Each hydrograph is valid only for a particular gauge station and a particular year. Hydrographs of a series of consecutive years are used to form duration-curves (Part II, Para. 2.3.3). Water-levels versus discharges form the stage-discharge curve for a particular station and year, and are used in combination with duration-curves to indicate the probability of the occurrence of discharges. Apart from use for hydrological studies, the water-level data can be of direct profit for navigation.

##### 4.1.2. Gauges

In general, two types of gauges can be distinguished: staff gauges (directly read) and recording gauges.

Staff gauges (Figure 4.1.1) should preferably consist of a metal strip divided into meters, decimeters and centimeters. The strip should be fixed on a structure in such a way that height corrections can easily be made in order to facilitate gauge-zero corrections. The zero of the gauge should be determined by levelling to the nearest benchmark.

If wind or ship waves occur, the accuracy of reading the staff gauge can be considerably improved by installing a glass or plastic tube on the gauge plate. In this tube, which has some small perforations at the bottom, fluctuations caused by waves are considerably damped.

Attention should be paid in case of sloped banks, as it may then be necessary to use a series of staff gauges in order to cover the complete range of water-level variations.

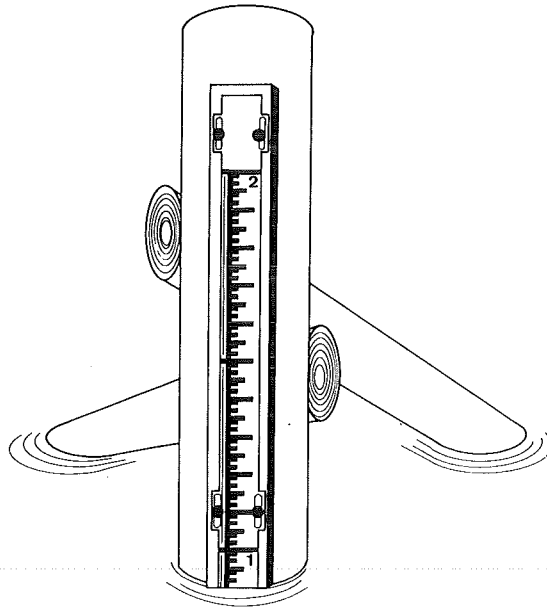


Figure 4.1.1 Staff Gauge

The principle of the automatic recording gauge is as follows (Figure 4.1.2). A float inside a pipe, which is perforated at the bottom, is moved up or down by the water-level. (Fluctuations caused by waves are almost eliminated). The movement of the float is transmitted by a thin wire to a mechanism which records these movements on paper, the method of recording depending on the type of gauge.

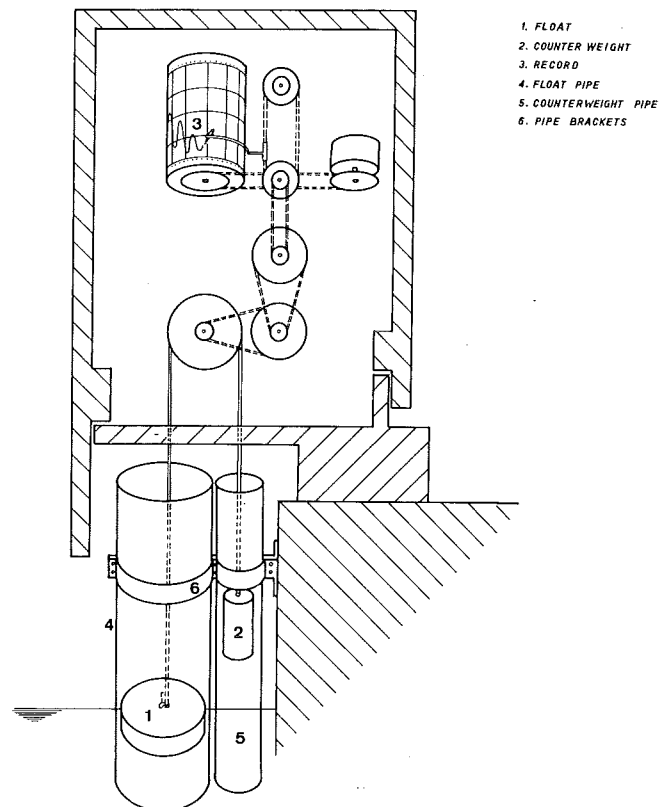


Figure 4.1.2 Automatic Recording Gauge

#### IV, 4.1

Normally a recording gauge is automotive during a week; then the paper should be changed and the clockwork wound. The recording gauges used along the Canal del Dique area were of this type ("Stevens"), as well as the gauge in Pto. Berrfo ("Seba").

##### 4.1.3. Selection of gauge sites

The network of gauges along a river should be so arranged that water-level information at any place along that river can be gathered by means of interpolation of the gauge records. Gauges should be placed where a change of water-level gradient, discharge, or in general the character of the river occurs, while the river sections in between two gauges should not be so long that a difference in discharge due to storage might become excessive. The selection of a gauge site and the installation of a gauge requires thorough knowledge about the hydraulic and morphological phenomena in a river, as well as knowledge about ship movements, etc. For the gauge site and installation of the gauge, the following requirements should be met:

- The site should be accessible to a gauge reader, enable easy reading of the gauge, and (unless an automatic recording gauge is used) a gauge reader should be available close at hand.
- Even during extreme low water-levels the gauge should still be in open connection with the river and not be dried.
- Even during extreme high water-levels the gauge should not be overflowed.
- Damaging of the gauge by ships, floating debris or slides in the river bank should not be possible.
- The location of the gauge site must be such that no influence is felt of backwater effects due to tributaries, etc., and preferably should be chosen just upstream of a control section in order to avoid the influence of local scour and sedimentation.
- A levelled benchmark should be near, for a regular check on the zero position of the gauge (change in datum of the gauge or accidental loss of the gauge can never be completely prevented). Control levellings should frequently be carried out (but at least twice a year as a routine) and the results properly filed for future reference.

Another problem is presented by the frequency with which gauge readings are required. This, of course, depends on the fluctuation of the levels. When these fluctuations are small, one reading a day can be sufficient, but if great fluctuations occur, 3 readings a day are required. At places with very rapid changes in water-levels, hourly readings should be taken, but it is preferable that continuous readings by an automatic gauge be taken.

It is often necessary to establish a temporary gauge, for example, during discharge measurements and soundings, in order to know the fluctuations of the water-level during these measurements. At places where these measurements are done regularly, the establishment of a benchmark is required.

## 4.2. SOUNDINGS AND ROUTE MAPS

### 4.2.1. Introduction

Local soundings of a part of the river are made to give general insight of the bottom profile, to determine the exact position of the best navigation channel as well as obstacles, and for the design of river-works. When regular detailed soundings of the same part of the river are made, indication is obtained of the shifts of navigation channel, banks, etc.

Route maps consist of a longitudinal sounding of part of the river, and specially for navigational purposes, information about shallows, banks and beacons.

### 4.2.2. Reduction-level

For the comparison of hydrographic measurements such as longitudinal soundings of a river or detailed soundings of a certain river section, the assumption of a reduction-level is required. In Colombia it is the practice to relate all water-levels to M.S.L. However, especially for navigation purposes, such a reduction-level is not very suitable. A river operator needs to know the Least Available Depth (L.A.D.) in a certain river stretch in relation to the pertaining water stages. Therefore, the reduction-level should have a relation to the daily gauge-readings.

It will be clear that for navigation purposes the reduction-level should be related to low water-levels and, moreover, that anywhere along the river the probability of the occurrence of a still lower water-level for a certain duration is equal; in other words, that on an average the reduction-level at each place along the river is exceeded during the same number of days per year.

The reduction-level, the so-called Low River Level (L.R.L.), which has been adopted by the Mission is defined as the level with an exceedance frequency of 95% of the year. This means that on an average only during 18 days of a year will the actual water-level be lower than L.R.L.

The line connecting the reduction-levels of the successive stations is a curve which more or less follows the average longitudinal profile of the river. However, it cannot be considered as an actually occurring water stage. The L.R.L. values (as read on the gauge) as they were adopted by the Mission along the Rfo Magdalena and the Canal del Dique are given in Table 4.2.1. For more detailed information regarding the reduction-level reference is made to Part II, Para. 2.3.5.

Rfo Magdalena			Canal del Dique		
Station		L.R.L. (m)	Station		L.R.L. (m)
Pto. Salgar	(km 887)	1.68	Calamar	(km 0)	2.13
Pto. Inmarco	(km 773)	0.20	Sta. Lucfa	(km 10)	1.95
Pto. Berrfo	(km 730)	0.60	Soplaviento	(km 33)	1.44
Barrancabermeja	(km 631)	0.99	Gambote	(km 66)	0.70
Pto. Wilches	(km 597)	0.75	Correa	(km 82.5)	0.51
Calamar	(km 91)	2.13	Matunilla	(km 100)	0.30
			Lequerica	(km 108)	0.21
Gamarra	(km 473)	36.70 (above M.S.L.)	Bahfa de Barbacoas		0.12 (L.L.V.S.)
			Bahfa de Cartagena		0.12 (L.L.W.S.)

Table 4.2.1 L.R.L. Values as Read on the Gauge

4.2.3. Preparations for soundings

In the area to be sounded either a staff gauge or an automatic recording gauge must be read the zero of which should be connected to an existing benchmark. Depending on the occurring water-level fluctuations and the required accuracy, the staff gauge must be read during the soundings either every hour or three times a day in order to know exactly the difference between the actual water-level and the reduction-level. This amount must be later subtracted so as to relate the soundings to the reduction-level.

The triangulation beacons and auxiliary beacons with which the position of the soundings can be elaborated must be established in such a way that at each position within the boundaries of the area to be sounded a good location fix can be obtained (see Para. 3.2.1). Attention must be paid to the fact that under certain conditions a white conical beacon should be short if, for example, it is to be in front of low brushwoods, but high in different circumstances, such as in front of tall trees without any brushwood at all.

Soundings in canals and rivers, that are not too wide, can be made on dead reckoning. This means that the survey launch sails cross-sections at estimated intervals, with the cross-sections being straight-lined by estimation. For wide rivers and bays like the Bahfa de Cartagena, sounding on dead reckoning is inefficient because of the distances. In such cases a map of arcs should be used to determine the position of each sounding immediately while sounding. This map of arcs serves a dual purpose:

- To be used as a plotting chart during the actual soundings; and
- to be used afterwards for quick elaboration of the sounding data.

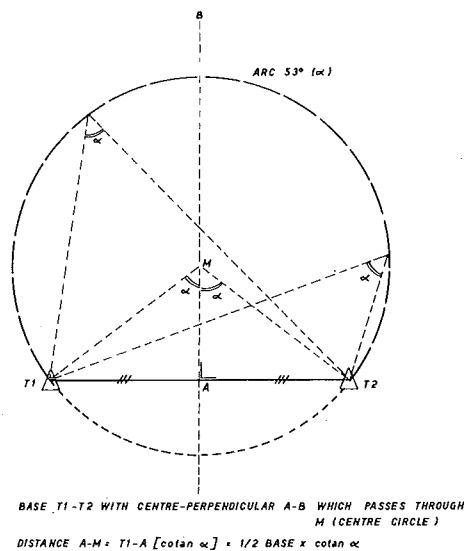


Figure 4.2.2 Chart of Arcs

The principle of the chart of arcs is the existence of all possible loci (arcs) derived from the existing triangulation and auxiliary beacons. In this way the surveyor has only to combine his two loci (arcs), (the arcs of the two angles with which the particular beacons are measured with the sextants) and then the intersection of the two arcs (either interpolated or not) is the exact position. If this is done during the actual soundings, the

course of the survey vessel can immediately be corrected up to the next position plot, and practically straight cross-sections can be sailed. The construction of a chart of arcs needs some preparatory office work, as also does the use of the basic chart, on which the beacons are plotted. For the construction of the arcs and an example of a chart of arcs see Figures 4.2.1 and 4.2.2 respectively.

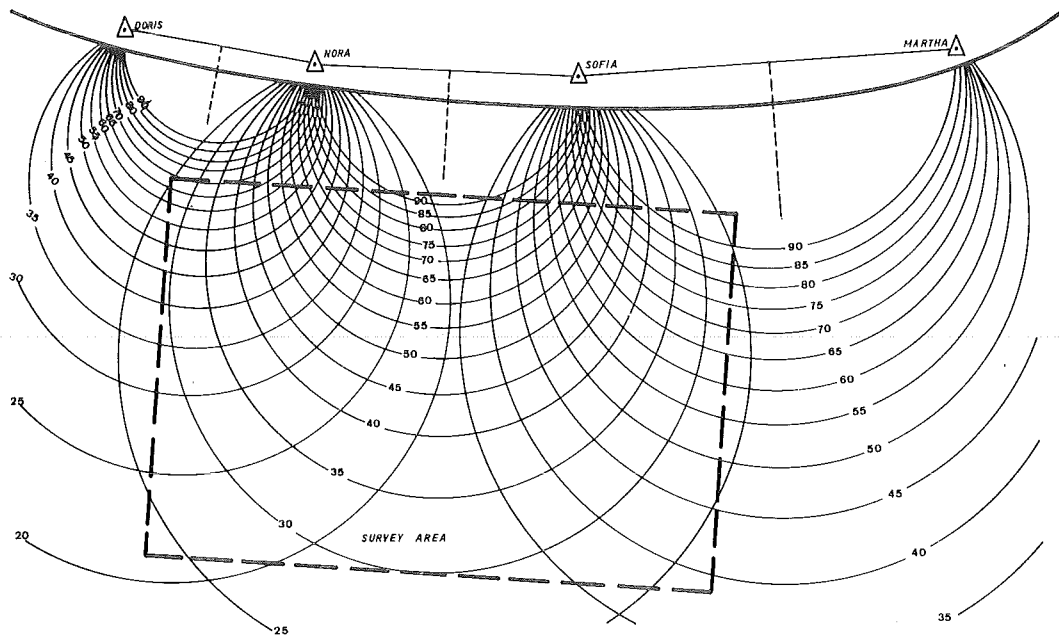


Figure 4.2.1 Construction of Arcs

Before the actual soundings, the echo-sounder should be checked for accuracy (see Para. 2.6) and calibrated. The method of directly adjusting and calibrating the echo-sounder is only valid when the water temperature and density are nearly uniform from surface to bottom. If there is stratification in temperature and density, the echo-sounder should be adjusted to the bar-check reading at the deepest position, and a correction-graph be made for the depths in between. This, however, only occurs in waters like the Bahía de Cartagena.

#### 4.2.4. Actual soundings

One small survey launch with a coxswain and three surveyors can carry out the actual soundings. Two surveyors simultaneously measure sextant-angles between the chosen beacons, and at exactly the same time the third surveyor fixes the recorded depth on the echo-sounder. He then notes the measured angles with their corresponding beacons in the sounding book (Figure 4.2.3) giving these angles and the simultaneously marked depth the same running number (fix). These numbers, together with the angles and beacons, the time and any remark made by the surveyor in command, are written in the sounding book.

The surveyor in command instructs the coxswain on the steering course, drift corrections, speed of the survey launch, and plots the fixes if a chart of arcs is available. If there is one observer at the echo-sounder and a second for keeping the sounding book, there should be a regular check-up between these two about the running fix numbers.

Posición No	HORA	Primer ángulo	Segundo ángulo	Profundidad sondeada	Corrección	Reducción	Profundidad a datum
		<i>Doris-Nora</i>	<i>Nora-Martha</i>				
642	12.15	95° 12'	67° 26'	5.70	+0.50	-3.20	3.00
643		94 01	73 20	6.25			3.55
644		92 45	79 06	6.35			3.65
645		91 13	97 56	6.10			3.40
646		90 00	109 25	5.70			3.00
			<i>Nora-Sofia</i>				
647	12.21	87°45'	19°17'	4.75	+0.50	-3.15	2.10
648		85 59	21 00	3.60			0.95
649		79 56	23 23	1.75			- 0.90

Figure 4.2.3 Extract of Sounding Book

Time intervals and distances between sounding tracks (cross-sections) are dependent on the required accuracy and chart scale. For example, when the chart to be made will have a scale of 1 : 5,000 the distances between the tracks as well as the distances between the fixes should not exceed 50 meters (gives distances of about 1 cm on the future chart).

The sounding tracks must be made rectangular to the depth contours, in order to have them properly located. Reefs and shoals should be separately approached on convergent sounding tracks, and be sounded cross-ways. The first and last fix of a sounding track (section) must be marked by a double marker line, as well as in the sounding book.

After the final bar-check of the echo-sounder each day, all relevant papers, the recording paper and the sounding book are dated and properly filed.

#### 4.2.5. Elaborations

The easiest and safest way to note the data is in the sounding book. Each page has columns for: fix-number (position), time, first (always left-hand) angle, second (always right-hand) angle, with their corresponding beacons, sounded depth, correction, reduction, and overall corrected depth (see Figure 4.2.3). By using such a book all data are together, and a subsequent check on doubtful figures is easily and quickly made. The sounded depth is taken from the recording paper of the echo-sounder, and either noted in feet or converted to decimeters. The echo-sounders used by the Mission recorded the depth in feet, and recordings were read by means of a conversion scale, thus directly noting in the sounding book the depth in decimeters. The next column for correction gives the depth or draught of the underwater transducer, and is always positive - because the draught of the transducer has to be added to the sounded water depth (Figure 4.2.4).

The reduction in the next column is negative when the actual water-level during the soundings is above the reduction-level. If the water-level is below the reduction-level, the reduction is positive. Reduction is the difference between the actual water-level (hence the necessity of gauge-readings) and the reduction-level.

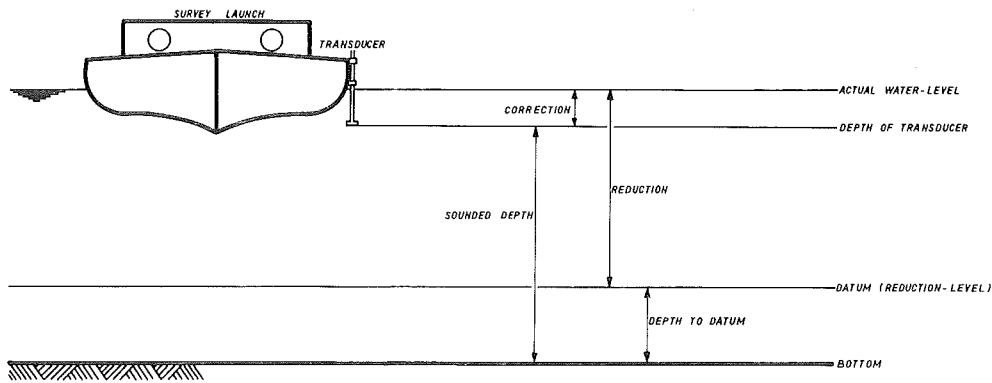


Figure 4.2.4 Actual Water-level during Soundings Related to Reduction-level

The last column gives the depth to datum, which is the algebraic sum of the columns of sounded depth, correction and reduction. These depths (to datum) are used to produce the chart with depth contours, as all depths in any hydrographic chart are related to a certain reduction-level.

#### 4.2.6. Charts involved (see Figure 4.2.5)

##### Basic chart

On the basic chart the triangulation beacons are plotted, as well as any other beacon used during the soundings, and if a co-ordinate grid system is used, this grid also appears on this chart. The basic chart is mother sheet for all following charts, and should preferably be drawn on a thermal and hygroscopic stable (true-scale) drawing film.

##### First chart

The first chart is copied from the basic chart, and also shows the beacons used during the soundings. On this chart the positions are plotted and given their running fix numbers. If between two soundings the interval is too large, additional points are plotted and simultaneously marked on the recordings of the echo-sounder. The first chart should also be drawn on true-scale transparent material.

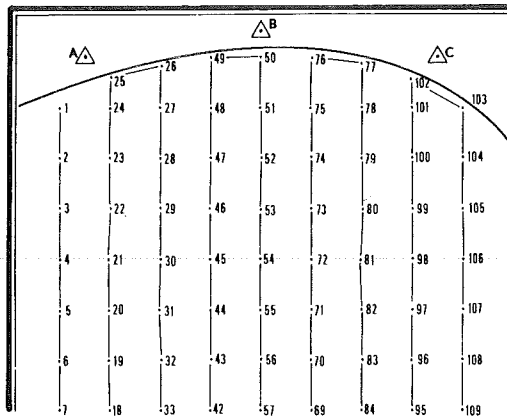
##### Second chart

A sheet of true-scale transparency is put over the first chart in such a way that the already plotted beacons coincide completely. The running fix numbers of the first chart are then covered on the second chart by their depths-to-datum values, the centre of these numbers being exactly over the centre of the fix numbers. The additional points on the first chart are also given their newly-elaborated interpolated values of depth-to-datum on the second chart. Each sounding track should now be completely covered by depth figures, and depth contours can be drawn. Doubtful depth figures are now detected, and a check can easily be made from the sounding book and the echo-sounder recordings.

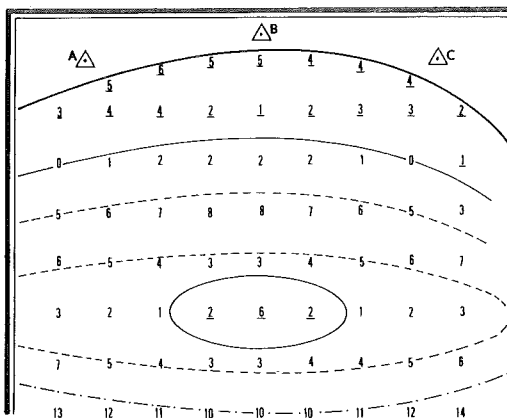
IV. 4.2

Final chart

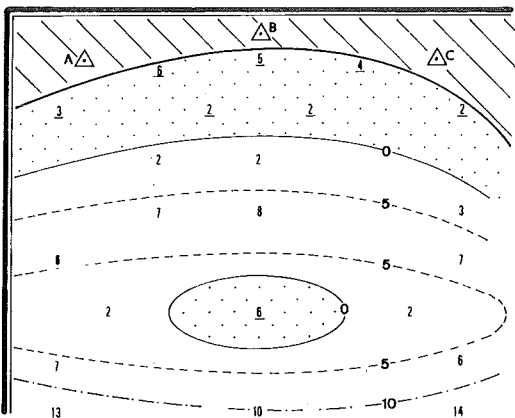
A sheet of true-scale transparency is put over the second chart and the depth contours are copied, as well as the coast lines or banks, and depth figures at regular intervals and at important positions. In this way on the final chart a more overall impression of the sounded area is obtained, and all necessary data such as definition of the reduction-level, depth units, area, date, etc. are mentioned.



PART OF A FIRST CHART, EXAMPLE.  
FIX NUMBERS OF THE SAME TRACK ARE CONNECTED BY A LINE TO MAKE INTERPOLATION FOR ADDITIONAL POINTS EASIER.



PART OF A SECOND CHART, EXAMPLE.  
NEGATIVE DEPTHS (PARTS OF THE BOTTOM EXCEEDING THE REDUCTION-LEVEL) ARE INDICATED BY UNDERLINED FIGURES. THESE PARTS ARE DRY WHEN THE WATER-LEVEL IS ON OR BELOW THE REDUCTION-LEVEL.



PART OF A FINAL CHART, EXAMPLE.  
UNDERLINED FIGURES INDICATE HEIGHTS ABOVE DATUM. DEPTHS AS WELL AS HEIGHTS IN DECIMETRES.

Figure 4.2.5 Examples of Charts Involved

#### 4.2.7. Route maps

One of the main reasons for the introduction of route maps was to collect the minimum of essential up-to-date information of the river. The route maps prepared by the Mission cover the Río Magdalena from Pto. Salgar to Gamarra. They show a recent longitudinal sounding of the navigation channel reduced to L.R.L., as well as some typical cross-sections and the topography of the same part of the river on each page. The latest positions of navigational beacons should also be shown on future route maps.

#### 4.2.8. Kilometrage

The original kilometrage of the Julius Berger Konsortium was considered to be valid for some topographic features of a permanent nature, such as old ranches and towns, along the river. Table 4.2.2 shows a list of points for which the Berger kilometrage was assumed to be still valid.

Location	Kilometer	Location	Kilometer
Pto. Salgar Bridge	886.5	Barriga	671.7
Hda. San Cayetano	866.7	Chucurf	659.6
Pto. Triunfo	824.9	Barrancabermeja	631.2
Pto. Miño	812.8	Rfo Sogamoso Confluence	612.2
Hda. El Rebozo	804.7	Pto. Wilches	597.1
Hda. Calmital	799.7	San Pablo	582.0
Hda. La Plata	792.5	Paturia	564.5
Pto. Inmarco	772.7	Bocas del Rosario	547.5
Pto. Berrfo Bridge	730.3	Vijaqual	534.6
Murillo	716.2	Badillo	520.0
Rfo Huevo	706.6	Bodega Central	494.5
Hda. Mosquitera	693.4	Gamarra	472.7
Presidio	680.7		

Table 4.2.2 Kilometers Copied from the Julius Berger Konsortium

Due to meandering of the Río Magdalena, the kilometrage between these points had to be determined again, which resulted in shorter kilometer distances at places where the river had straightened, and in longer distances where the river had started meandering. Derived from the new kilometrage, the following points were determined for the use of route mapping, and are given in Table 4.2.3.

The kilometrage system not only facilitates the determination of distances along the river but is also used for the exact indication of crossings, shallow waters, wrecks and bottle-necks.

The decimal system has been used throughout this Report because this system is gradually gaining ground all over the world (even in hydrographic work). Hence the longitudinal soundings of the route maps indicate depths in meters and decimeters. (See Para. 4.2.4, also metric units on the hydrographic charts).

The origin of the Río Magdalena kilometrage lies at the terminal of Barranquilla, kilometer 0; in this way the kilometrage always is positive. The origin of the Canal del Dique kilometrage lies at Calamar (kilometre 0 of the Canal del Dique and kilometre 90.7 of the Río Magdalena).

#### IV, 4.2

Name	Km	Name	Km
Hda. La Guaira	873.0	Hda. San Miguel	741.7
Hda. Las Brisas	864.2	Hda. Magdalena	740.1
Hda. La Cecilia	863.5	Hda. Sta Inés	738.5
Hda. El Conchal	861.7	Hda. Piemonte	735.8
Hda. Rfo Grande	858.8	Villa Carlina	735.7
Hda. Vernon	844.1	Hda. Sebastopol	731.6
Buena Vista	837.2	Sta. Cruz	721.5
Hda. Guadalcanal	834.9	Hda. Alicante	718.2
Hda. La Florida	830.7	Rfo Regla Confluence	710.9
Hda. Sta. Isabel	822.5	Hda. La Hadero	710.5
Hda. San Fernando	821.0	Hda. Las Brisas	705.6
Pto. Boyacá	806.0	Hda. Los Hangos	704.2
Hda. Miraflores	795.2	Hda. Sta. Clara	703.4
Hda. La Tigrera	793.2	Hda. Los Horros	691.6
Hda. El Deseo	790.6	El Caballo	686.9
Hda. La Posada	790.1	Carare	673.7
Hda. La Cabana	787.4	Hda. Berlin	624.1
Hda. Argelia	783.8	Rabon	617.7
Mpo. Nare	776.2	Cantagallo	594.0
Hda. Guadalagara	766.6	Yarirf	591.0
Hda. La Suiza	766.3	Villa Marquesa	583.6
Hda. Holanda	755.5	Roca de Limón	561.0
Hda. Alicia	755.3	Chingalé	554.5
Hda. Nápoles	753.2	El Guyabo	522.5
Hda. El Paraiso	746.8	Loma de Corredor	495.2
Hda. Piedralinda	745.8	Pto. Mosquito	490.5
Hda. Guamal	745.1	El Contento	485.2
Hda. Bogotá	742.7	Acapulco	477.7

Table 4.2.3 New Kilometrage along the Rfo Magdalena

#### 4.2.9. Processing of route maps

##### Water-levels

All depths shown on the route maps are reduced to L.R.L., so the longitudinal soundings as well as the cross-sectional soundings have to be corrected with reference to the gauge-readings. These are gathered primarily during the survey itself, and secondly afterwards, collecting all relevant water-level data of the time and area covered by the survey. Exact reduction of the soundings is then calculated in the office (see Para. 4.2.5).

##### Navigation channel

The main feature on the route maps is the longitudinal profile of the deepest channel of the river. Consequently, the main purpose of the survey is to determine the deepest channel by means of soundings, either cross-sectional or longitudinal. That attention must be paid to the fact that the difference between the talweg (deepest channel) and the navigation channel is rather obvious, as the talweg follows the deepest points of the river bed and often shows a great sinuosity which is almost impossible to sail from point to point. The actual course of the survey vessel when trying to follow the talweg is a smooth line, indicating the navigation channel.

Cross-sections of the river at short distances from each other obviously are the most accurate means of locating the alignment of the channel, and these in fact are the only alternatives in generally shallow rivers. Normally, however, the channel alignment has a regular course and the experienced surveyor will follow it with the help of the topographic

configuration of the river and the condition of its banks. Eroding banks or high banks are usually an indication that the main channel is nearby, while sedimentation points to the opposite (for the deepest channel to run on the inside of a bend is quite exceptional).

To locate the deepest channel and to record a continuous sounding of its depths, either of the following two procedures can be applied:

- The deepest channel is traced with the help of cross-sections at short distances apart, and at places of special significance to navigation like crossings, shoals, junctions, etc. The places of the consecutive cross-sections are marked on the map according to the kilometreage, and also their deepest points. Having covered a river stretch of a certain length (so that it is still possible to recognize the cross-section locations on return) a longitudinal sounding is made of the deepest channel as indicated by the cross-sections.
- At the starting point a cross-section is taken to locate the deepest channel. From there the channel is followed and continuous soundings of the channel are recorded. Slight divergencies of the correct course may be checked by zig-zag sailing. However, as soon as the surveyor is in doubt whether he is following the channel or not, the sounding must be interrupted to sound a cross-section. When sufficient cross-sections have been taken again to mark the deepest channel, the longitudinal sounding of the channel is continued by starting from a well-defined point in the previous recording, preferably a kilometreage mark. The time-interval spent for cross-sectional investigation is accordingly marked on the echo-recordings.

The first procedure leaves less room for subjective interpretations than the latter; however, higher accuracy is achieved at the cost of a lengthy and time-consuming sailing programme.

The decision which procedure be applied largely depends on the character of the river and the skill of the surveyor. A shoaly and predominantly shallow river with indistinct banks calls for the first method, while deeper rivers and a well-defined river bed are easily surveyed by the second method. The surveyor should always bear in mind that the longitudinal section shown on the route map is of direct value for navigation, and from this point of view his attention should be drawn to those places, like crossings and shoals, which might obstruct navigation. Slight course deviations from the talweg on the deeper stretches of the river are immaterial as long as the recorded depth minus a safe reduction is deeper than the draught of the largest vessels expected to sail the river.

#### 4.3. DISCHARGES

##### 4.3.1. Discharge per unit width

###### Selection of cross-section

The usual requirements for the selection of a cross-section for discharge measurements are:

The cross-section should be clear, without islands or sand-banks, thus permitting easy sailing from bank to bank. The direction of the current should be perpendicular to the cross-section, so river bends should be avoided.

IV, 4.3

Of course, it will not always be possible to fulfill these requirements for all the cross-sections needed. Generally this results in more detailed elaboration of the data, and possibly less accuracy.

Measuring velocities at regular depth intervals

The accuracy of the discharge calculation depends on the depth intervals with which the velocities have been measured in the vertical. Generally in each vertical the velocities have to be observed at depths with 1-meter intervals, starting at 0.5 m below the water surface, and the last measurement being made 0.5 m above the river bottom. (Only when the river is very deep, can 2-meter intervals starting at 1 meter below the water surface be considered).

These velocity data should be entered in duplicate on special forms on which, besides the position of the survey vessel, also the depth recorded by echo-sounder or measured by hand-lead is inserted.

The discharge per unit width ( $q$  in  $m^3/sec/m^1$ ) of the cross-section can be calculated, either using the graphical method (Figure 4.3.1) or the non-graphical method (Figure 4.3.2).

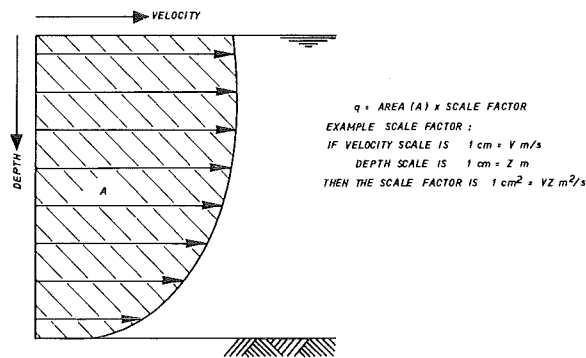


Figure 4.3.1 Discharge per Unit Width, Graphical Computation

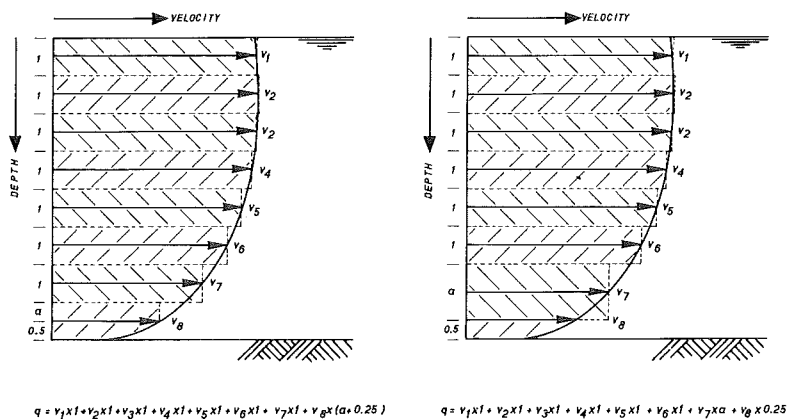


Figure 4.3.2 Discharge per Unit Width, Non-graphical Computation

Measuring velocities at one or two depths

A fairly good estimate of the discharge per unit width is still possible by measuring the velocities only at one or two depths of the vertical, assuming that the average velocity of the vertical occurs at  $z = 0.4h$  (One measurement at height  $z$  above the river bottom). This is based on the assumption that the velocity vertical can be described as a parabolic or a logarithmic function. For example:

$$v_z = \frac{v_*}{\kappa} \ln \frac{z}{z_0}$$

In which:

$v$  = shear velocity ( $=\sqrt{gRI}$ ),

$\kappa$  = Karman's parameter ( $= 0.4$  for clear water),

$z$  = distance from the river bottom, and

$z_0$  = distance from the river bottom where the velocity is zero.

The same is valid for velocity measurements at two depths, being  $z = 0.2 h$  and  $z = 0.8 h$ , assuming that the arithmetic mean of these velocities coincides with the average velocity of the vertical.

From these measurements the derivation of the discharge per unit width ( $q$ ) is, of course, less accurate than measuring the velocities at regular depth intervals. It may be an advantage that such measurements at one or two depths require less time. However, compared with the anchoring of the survey vessel or the time-consuming procedure of sediment-transport measurements, measuring in detail the velocity distribution in the vertical is a relatively short procedure.

Measuring the surface velocity

A way of estimating the average velocity ( $\bar{v}$ ) of a vertical is by measuring the surface velocity only, for example, by means of a float. In order to find  $\bar{v}$  it is necessary to reduce ( $\psi$ ) the surface velocity;  $\psi$  usually being 0.85 or 0.9. This reduction factor depends on the roughness of the bottom, which in its turn determines  $C$ . This means that a relation exists between  $\psi$  and  $C$ :

$C$ ( $m^{1/2}/s$ )	80	70	60	50	45	40	35	30	25
$\psi$	0.91	0.90	0.89	0.87	0.85	0.84	0.81	0.78	0.76

When multiplying the average velocity ( $\bar{v}$ ) by the depth of the vertical,  $q$  is found.

4.3.2. Total discharge

The accuracy of the discharge calculation depends not only on the accuracy of the velocity measurements but also on the distance between the velocity verticals. For practical purposes a minimum distance of 50 m is generally considered acceptable. Verticals too near a river bank should be avoided, to prevent disturbances.

#### IV, 4.3

In a non-tidal river where the flow is more or less permanent, one survey vessel can be used to measure all verticals consecutively, although of course, each time the vessel has to be anchored. The total discharge ( $Q$  in  $\text{m}^3/\text{s}$ ) can be determined by plotting the discharges per unit width ( $q$ ) against the positions of the relevant verticals (Figure 4.3.3).

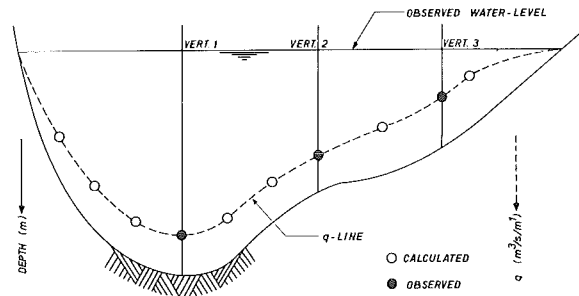


Figure 4.3.3 Discharges per Unit Width Plotted in the Cross-section

The area encompassed by the q-line represents the total discharge of the particular cross-section. If necessary, due to an insufficient number of velocity verticals or a too irregular-shaped cross-section, interpolation between the observed verticals can be done on the basis of the formula of Chézy:  $q = C I^{1/2} h^{3/2}$ , assuming that the roughness coefficient ( $C$ ) and the water-level gradient ( $I$ ) have the same value over the complete cross-section. The discharge per unit width ( $q$ ) can be calculated for any vertical with a depth  $h_i$  near the observed vertical with a depth  $h_m$ , according to the equation:

$$q_i : q_m = h_i^{3/2} : h_m^{3/2}$$

The discharge cross-section may not always consist of one clear main channel only. For example, an alluvial channel subject to seasonal floods generally consists of a main channel and one or two side channels (Figure 4.3.4).

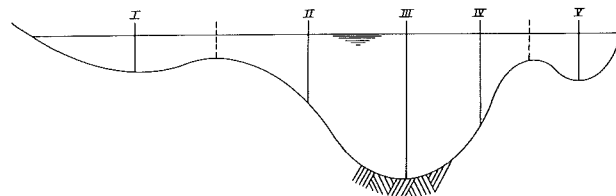


Figure 4.3.4 Cross-section with Flood Plains

In general, a flood plain is rougher than the main channel due to vegetation, its depth (and hydraulic radius) being smaller, and so the mean velocity of the main channel is higher than the mean velocities of the flood plain. The discharge for each section is determined separately from the measured velocity verticals, using the dashed lines (Figure 4.3.4) as the separation of the different sections. The separate discharges are added to determine the total discharge of the cross-section.

In such irregular-shaped cross-sections it will often be difficult to draw the  $q$ -line directly, because of the difference between the discharges per unit width. Hence it is better first to draw the line of mean velocity of the cross-section ( $q/h = \bar{v}$ ) which has a smoother course. Multiplication of the  $\bar{v}$ -line and the  $h$ -line gives the total discharge. An example of the elaboration of a discharge measurement is given in Figure 4.3.5.

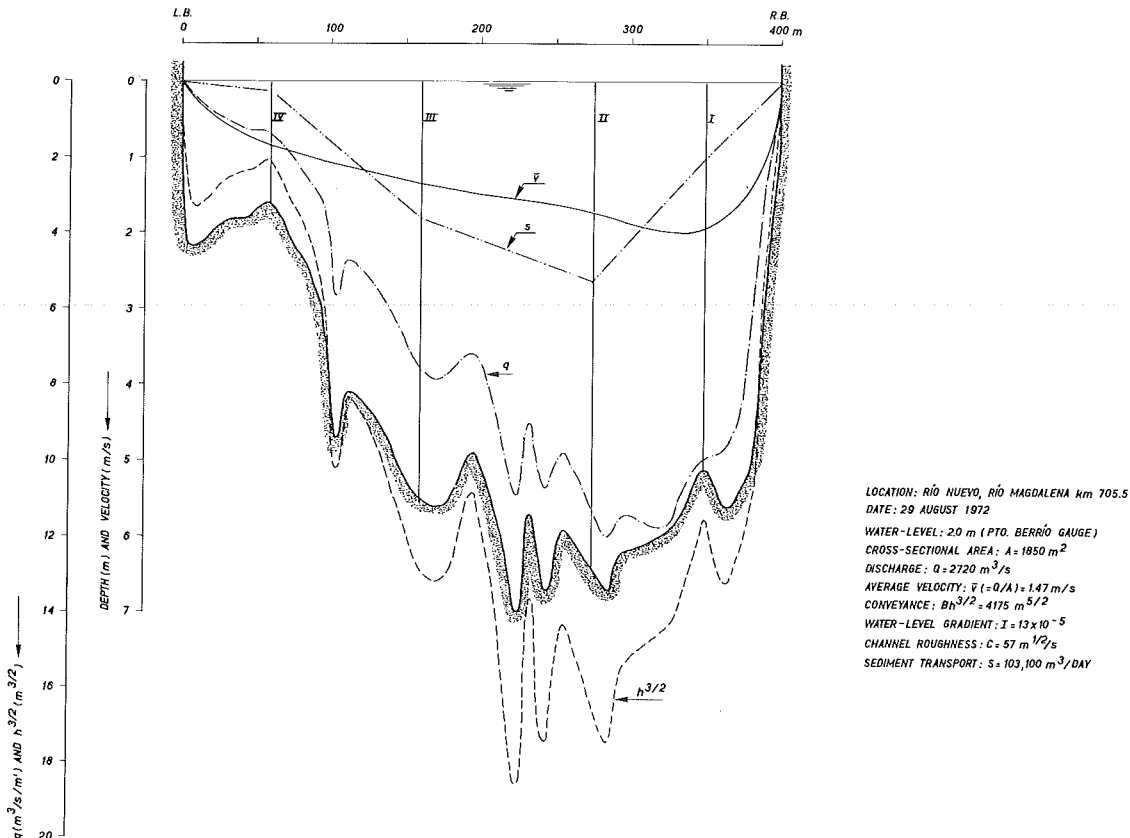


Figure 4.3.5 Elaboration of Discharge Measurement in the Río Nuevo

#### 4.3.3. Tidal effects

In the foregoing paragraphs it has been assumed that the flow in a non-tidal river is more or less permanent during the discharge measurements. In tidal regions, however, this assumption is no longer valid and the discharge measurements require a more elaborate procedure.

To evaluate the behaviour of the tidal currents it is necessary to select a number of cross-sections at significant locations, and to carry out the observations under comparable conditions. The ideal situation is when all measurements are done in the same period, and thus under exactly the same tidal conditions. If this is impossible, however, due to insufficient survey vessels, experienced observers or equipment, the measurements may be spread over a longer period covering several days. But then the tides have to be pre-selected according to the tide tables.

#### IV, 4.3

If the object of the measurements is to find the magnitude of the upland discharge only, the tide during which the measurements are done is immaterial. If, however, the object is to obtain data of the horizontal tide, the most complete information is obtained when observations are carried out during the extremes: spring tide and neap tide.

The observations in a cross-section should cover a complete tidal cycle of about 25 hours. If the cross-section is located in a river stretch with a not-too-strong tide compared to the upland discharge, the current direction will not change, and in that case it is better to observe a longer period than 25 hours (for example, 28 hours) to make certain that the tidal cycle has passed completely.

For the selection of the cross-section, the same requirements as for non-tidal flow observations are valid. Moreover, if there is no gauge-station located nearby, a temporary gauge should be installed at the cross-section and observed every half hour during the measurements.

The available survey vessels are then distributed across the section. When the cross-section shows a single channel, one vessel will be located in its deepest part and the remaining vessels evenly distributed across the remaining width. Where there is a two-channel situation, two vessels have to operate in the deepest verticals of each of the channels. (The greatest discharge per unit width normally flows through the deepest parts of the cross-section).

The measurements should consist of the following items:

- Half-hourly gauge-readings of a staff gauge located near the cross-section, or the records of an automatic gauge.
- Every hour the velocities are measured at intervals of one meter along the vertical, starting at 0.5 m below the water surface and finishing at 0.5 m above the river bed. The discharge per unit width ( $q$ ) has to be calculated immediately after the observation.
- Every time the velocity vertical is measured, the total depth should also be measured either by echo-sounder or by hand-lead. This depth can deviate from the depth found by the current meter, as the latter often records too great a depth in strong currents. The measured  $q$  can be corrected rectilinearly, according to:

$$q_{\text{corrected}} = \frac{h_{\text{recorded}}}{h_{\text{current meter}}} \times q_{\text{measured}}$$

- Not only the velocities but also the current direction should be noted.
- Each period of slack water should be noted, when flood changes into ebb tide or vice-versa.
- During the course of the observation period the position of the survey vessel should be checked repeatedly. Should it be dragging its anchor, re-positioning of the vessel is necessary.

If in the particular river section the tide predominates over the upland discharge, the survey vessel will sway, due to the difference in current direction at ebb tide and flood tide. The vessel will be located downstream of the cross-section during ebb tide, and upstream

of the cross-section during flood tide. However, observing the requirements for the selection of the cross-section, the changes of flow in its vicinity can be neglected, and the velocity distribution can be assumed to be observed actually in the cross-section. The depth of the vertical in the cross-section can be calculated from the recording of the echo-sounder and the gauge-readings. The discharge per unit width can be obtained by applying (see Figure 4.3.6):

$$q_{\text{cross-section}} = \frac{h_{\text{cross-section}}}{h_{\text{recorded}}} \times q_{\text{corrected}}$$

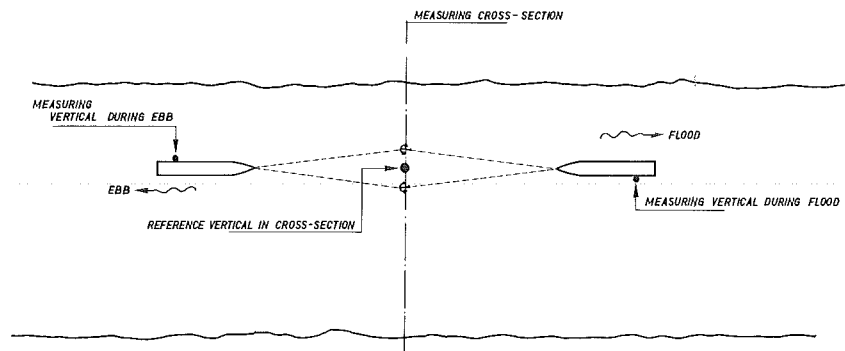


Figure 4.3.6 Measurement in Cross-section in Tidal Areas

For the calculation of the total discharge, three methods can be used:

- Computing the total discharge every hour, as described in Para. 4.3.2.
- A graphical method to avoid the many interpolations of each observation.
- A reference vertical in the cross-section.

#### Graphical method

This method should be used only when few survey vessels are available. The cross-section should be divided into as many parts as verticals, and for each part the discharge  $Q$  can be computed according to the following formula:

$$Q_i = \int_{B_{i-1}}^{B_i} qdB = \int_{B_{i-1}}^{B_i} q_{mi} \frac{h^{3/2}}{h_{mi}^{3/2}} dB = \frac{q_{mi}}{h_{mi}^{3/2}} \int_{B_{i-1}}^{B_i} h^{3/2} dB = \frac{q_{mi}}{h_{mi}^{3/2}} A_i$$

in which:

- $A_i$  = the area encompassed by the  $h^{3/2}$ -line, belonging to part  $i$ ,
- $q_{mi}$  = the measured discharge per unit width in vertical  $i$ , and
- $h_{mi}$  = the recorded depth in vertical  $i$ .

A graph is made between the water-level read from the gauge and the area  $A_i$ . This relation can be found by calculating the areas  $A$  only for the extreme and mean water-levels. The hourly total discharge can be calculated by means of a table.

IV, 4.3

Reference vertical

This method can be used in a tidal campaign where discharge observations have to be made simultaneously in a number of cross-sections, either in one river at different locations, or near bifurcations and confluences in different rivers. In each of the cross-sections one survey vessel should be anchored in the deepest vertical (the reference vertical) of the cross-section for the duration of the observations. Beforehand, or during the measurements a number of velocity verticals should be measured in rapid order in each of the cross-sections to enable the computation of the total discharge. A curve can then be made giving the relation between the discharge per unit width ( $q$ ) in the reference vertical, versus the total discharge ( $Q$ ) of the cross-section (Figure 4.3.7). As an example of the elaboration of the tidal measurements in the Lower Canal del Dique, reference is made to Part II, Figure 3.3.42.

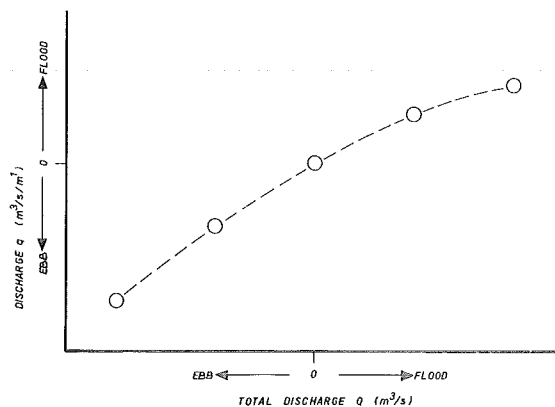


Figure 4.3.7  $Q_{\text{cross-section}}$  versus  $q_{\text{ref. vertical}}$

Finally, the tidal flow diagram can be drawn, in which the computed hourly cross-sectional discharge is plotted versus the time (see Figure 4.3.8).

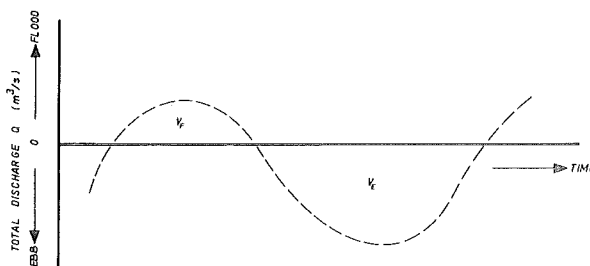


Figure 4.3.8  $Q_{\text{hourly}}$  versus Time

The ebb and flood volumes can be derived by the integration of the areas  $V_E$  and  $V_F$ . The upland discharge can be computed by subtracting the flood volume from the ebb volume and dividing this residual volume by the total time in seconds of the complete tidal cycle:

$$Q_{\text{upland}} = \frac{V_E - V_F}{\text{time in seconds}} \text{ (m}^3/\text{s)}.$$

#### 4.4. SEDIMENT TRANSPORT

##### 4.4.1. Introduction

It has been mentioned already that actually three different types of sediment transport can be distinguished; namely: bed-load, suspended-load and wash-load. For each type of transport to be measured, a special instrument is required. Before describing the elaboration of the sampling data, it is worthwhile to state clearly the differences between the three types of sediment transport mentioned.

##### Bed-load

Bed-load is the transport of sediment particles sliding, rolling or jumping over and near the river bed, generally in the form of moving bed forms such as dunes and ripples. When bed-load measurements are carried out, it is important to realise that this transport takes place as the propagation of bed forms, and that the transport intensity on the top of the dunes is large and in the troughs small or nil. Measurements should, therefore, cover at least the time required for several dunes to pass through the measuring section.

The integration time for a bed-load sampler is relatively small (2 minutes) for technical reasons, while the period of the fluctuations in the bed-load transport varies between several hours and even days. Consequently, an estimate of the actual average bed-load in a cross-section can only be obtained by taking a large series of measurements. Although after each single measurement the position of the sampler has to be changed, a random position of the sampler will automatically be obtained when the length of the bed-ripples is small compared to the variation in the position of the sampler due to its hoisting and lowering from a fixed point, the derrick or davit of the survey vessel. This will illustrate why, if possible, large differences in the average bed-level on both sides of the cross-section have to be avoided.

The sampler used by the Mission was the B.T.M.A. (Bed-load Transport Meter "Arnhem") (Para. 2.11).

##### Suspended-load

Suspended-load is the transport of bed particles when the gravity force is counter-balanced by upward forces due to the turbulence of the flowing water. This means that the particles make larger or smaller jumps, but return eventually to the bed. By that time, however, other particles from the bed will be in suspension and, consequently, the concentration of particles transported as suspended-load will not change rapidly in the various layers. A strict division between bed-load and suspended-load is not possible; in fact, the mechanisms are related. Bed-load and suspended-load together are often called bed-material load.

The instrument with which the Mission measured the suspended-load was the Delft Bottle (D.F.) (Para. 2.10).

Wash-load

Wash-load is the transport of small particles finer than the bulk of the bed material and rarely found in the bed. Transport quantities found from bed-load, suspended-load and total-load formulae do not include wash-load quantities. Normally there is no interchange with the bed particles, and the rate of wash-load is mainly determined by climatological characteristics and the erosion features of the whole catchment area.

The instrument with which the Mission determined the wash-load concentrations was the water sampler (Para. 2.9), the samples of which were filtered, dried and weighed.

4.4.2. Sampling methodsBed-load

Samples of the bed-load are taken with the B.T.M.A., which is lowered by means of the survey vessel's davit or derrick on to the river bottom. With a stopwatch the sampling time is measured, which is normally two minutes, after which the B.T.M.A. is hoisted aboard again and the basket with the caught bed-load sample is emptied. The sample is measured volumetrically. Generally 10 samples are taken and the results noted, averaged, and converted into a daily transport (in  $\text{m}^3/24 \text{ hrs}/\text{m}^1$ ); see also example of elaboration form for bed-load and suspended-load measurements (Figure 4.4.2).

Suspended-load

Samples of the suspended-load are taken with the D.F., which is lowered into the river to the required depth by means of the davit or derrick aboard the survey vessel. The depth of the instrument is determined by the quantity of paid-out cable and indicated on a counter block. This counter block, through which the suspension cable of the D.F. runs, is put on zero when the D.F. is exactly on the water-level. As soon as the D.F. is fully submerged it is kept there for a while and the instrument will incline backwards due to the air contents. The air will escape from the nozzle and a small opening at the top of the rear end. As soon as the instrument is filled with water, it is lowered quickly to the required depth. Sampling time then starts and is measured by a stop-watch. A sampling time of three minutes has proved to be sufficient in the Río Magdalena, the sample being always large enough to be measured. (It is recalled from Part II, Para. 3.3.3 that in future the hoisting of the D.F.-sampler be included in the total measuring time of the suspended-load measurements). The D.F. is then hoisted aboard again, and the contents of the sampling chambers emptied into the special D.F. glass and measured volumetrically. Generally samples are taken every  $1\frac{1}{2}$ -meter in the measuring vertical, as well as five samples at every decimeter in the half meter just above the river bottom (Para. 2.10).

The results of the caught volumes are then converted into a daily transport of suspended load (in  $\text{m}^3/24 \text{ hrs}/\text{m}^1$ ); see example of elaboration form for bed-load and suspended-load measurements in Figure 4.4.2.

Wash-load

Samples of the wash-load are taken with the water sampler, which is lowered into the river to the required depth by a hand-line. The rubber stopper is then pulled off the bottle by means of a thin line which allows the bottle to be filled up with river water. Care should be taken not to hoist the sampler too soon, but to allow time for the bottle to be completely filled in order to prevent exchange of water content. The bottle is then taken out of the sampler, corked and labelled.

4.4.3. Elaboration of samplesBed-load

The 10 catches of the B.T.M.A. are averaged and the volume of the average catch or the complete catch is converted into daily transport (in  $\text{m}^3/24 \text{ hours}/\text{m}^1$ ) by means of the B.T.M.A. calibration curve (Figure 4.4.1). This calibration curve is based on tests carried out by the Delft Hydraulics Laboratory [64].

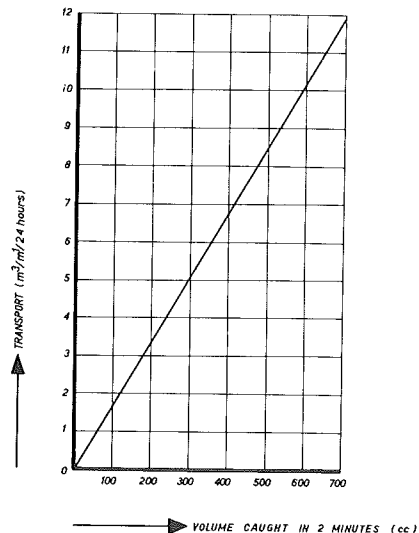


Figure 4.4.1 Calibration Curve B.T.M.A.

Suspended-load

The catches with the D.F. are volumetrically measured, and noted in cubic centimeters. The samples are caught either through the small nozzle (area  $1.9 \text{ cm}^2$ ) or through the big nozzle (area  $3.8 \text{ cm}^2$ ), which means that the daily transport per centimeter height of the vertical can only be derived by multiplying the catches with a correction factor, either 0.00063 for catches through the big nozzle, or 0.00126 for the small nozzle. The conversion from the caught sample (cc per 3 minutes) into a daily transport ( $\text{m}^3$  per day) is then made simply (see Figure 4.4.2).

Remark: Another correction factor, however, should be used, depending on the efficiency of the D.F. Sampler (see also Part II, Para. 3.3.3). This correction factor is shown in tables in the manufacturer's Manual of the D.F., its magnitude being determined by the flow-velocity, average grain-size of the caught sample, cross-sectional area of the nozzle, and whether a straight or bent nozzle was used. It is advised that this correction factor be applied in future suspended-load measurements.

MISION TECNICA COLOMBIA - HOLANDESA MEDECO - ADENAVI					TRANSPORTE DE SEDIMENTOS		Pag. 1 Total 5																																																							
Lancha Explorador					Sección: Río Magdalena, Ballena																																																									
Observador Guzman/H.J.O.					Localización																																																									
Fecha Oct. 25/71					Sección de aforo en Ballena, aguas arriba Río Regla.																																																									
Corriente					Vertical I 86m desde O.I. 712 m desde O.D.																																																									
Lectura mira					Boja Soba																																																									
D F Suspensión					OBSERVACIONES																																																									
Hora Inicial	Abura sobre el fondo	Profund. bajo nivel egua	tiempo en min.	Volúmen recogido en cm <sup>3</sup>	Nariz pequeño con los 2 tapones inferiores																																																									
10.40		0.70	3	23	<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
		2.10	3	12																																																										
		3.60	3	20																																																										
		5.10	3	30																																																										
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1</td> </tr> <tr> <td>0.30</td> <td></td> <td></td> </tr> <tr> <td>0.50</td> <td></td> <td></td> </tr> <tr> <td>0.76</td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.91</td> <td>9,1</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.92</td> <td>19,2</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>0.35</td> <td>3,5</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>1.34</td> <td>13,4</td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td>3.23</td> <td>32,3</td> <td></td> </tr> </table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1	0.30			0.50			0.76							0.91	9,1						1.92	19,2						0.35	3,5						1.34	13,4						3.23	32,3	
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1																																																												
0.30																																																														
0.50																																																														
0.76																																																														
				0.91	9,1																																																									
				1.92	19,2																																																									
				0.35	3,5																																																									
				1.34	13,4																																																									
				3.23	32,3																																																									
					<table border="1"> <tr> <th>TRANSPORTE</th> <th>X D (cm)</th> <th>TRANSPORTE</th> </tr> <tr> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por cm<sup>3</sup> abura</td> <td></td> <td>m<sup>3</sup>/m<sup>3</sup>/24 hrs. por abura.</td> </tr> <tr> <td>0.58</td> <td>Carga en suspensión</td> <td>307,1&lt;/</td></tr></table>					TRANSPORTE	X D (cm)	TRANSPORTE	m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.	0.58	Carga en suspensión	307,1</																																												
TRANSPORTE	X D (cm)	TRANSPORTE																																																												
m <sup>3</sup> /m <sup>3</sup> /24 hrs. por cm <sup>3</sup> abura		m <sup>3</sup> /m <sup>3</sup> /24 hrs. por abura.																																																												
0.58	Carga en suspensión	307,1</																																																												

conductivity meter (see Para. 2.14), of which the probe was lowered into the river to the required depths. The readings on the meter were translated into salinity or chlorinity by means of conversion tables.

Measurements were performed in verticals and at 1-meter intervals, the verticals being located along the longitudinal axis of the Canal del Dique.

With known values of salinity, depth and kilometrage, lines of equal salinity could be plotted (Figure 4.5.1) (See also Part II, Para. 3.3.9).

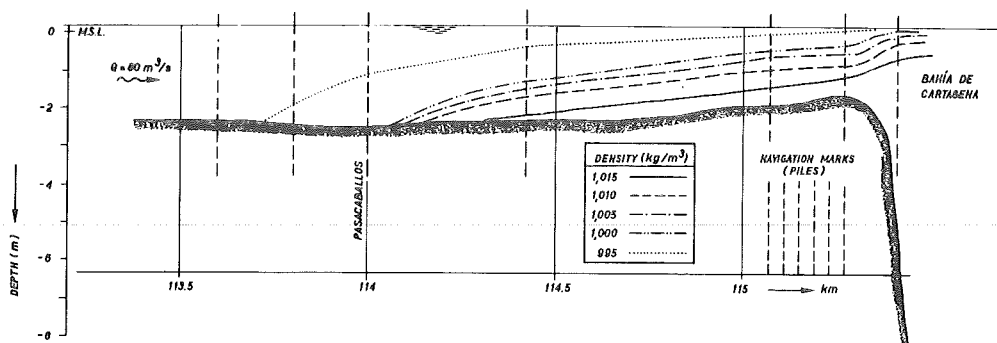


Figure 4.5.1 Salt-water Wedge near Pasacaballos

The E.C.R. conductivity meter used by the Mission was sent back to the Netherlands for repairs at the end of the study, and proved to have been completely damaged. According to the manufacturers, the measurements carried out with this instrument should be considered with suspicion.

#### 4.6. GRAIN-SIZES

The samples caught by the B.T.M.A. or bottom grab were analysed on grain-size distribution by sieving. A series of sieves gradually decreasing in sieve diameter (from 4.8 mm to 0.15 mm, according to the National Standard Norm N480) placed on top of each other and connected to a vibrator, left the amount of particles of each sample distributed over several sieves. The contents of each sieve were carefully measured (weighed) on an electronic balance with an accuracy of 0.1 milligram.

The sieve diameter and the weight of the caught particles, as well as their percentage of the complete sample, were used to form the sieve-curve (Figure 4.6.1). Many natural sediments have approximately a log-normal grain-size distribution, and therefore the sieve-curve is normally plotted on logarithmic paper.

A great number of samples were taken by the Mission in the course of the Study. A check was made whether the samples were big enough to allow an accurate determination of the representative diameters of the bed material, which were used in the elaborations on channel roughness and sediment transport (see Part II, Paras. 3.4 and 3.5). According to the design curve for the required mass of the samples (de Vries, 1971 [65]), a rough 80% of the samples taken proved to be of sufficient quantity to use with high accuracy in these elaborations (see Figure 4.6.2). A large variation, however, was found in the  $D_{50}$ -values of the

No. muestras: <i>53, de fondo</i>		MITCH	D10 = 520 $\mu\text{m}$	
Fecha: <i>18 Agosto 1972</i>			D65 = 340 $\mu\text{m}$	
Lugar: <i>Sección Ballena</i>			D50 = 290 $\mu\text{m}$	
Observaciones: <i>440 m. de orilla derecha</i>			D35 = 270 $\mu\text{m}$	
		ADENAVI NEDECO	Dm = 340 $\mu\text{m}$	
		ANALISIS GRANULOMETRICO	W = cm/s	

Datos del lab.		Curva Granulométrica			Dm		Velocidad de Caida	
Diámetro tamiz A [ $\mu\text{m}$ ]	Peso del material ret. [gr]	% ret. P <sub>i</sub>	Acumulada		M P. D. 100	Vel. Caida W <sub>i</sub> [cm/s]	$\sum P_i W_i$	$\sum \frac{P_i W_i}{100} = \bar{W}$
			% ret.	% pasa				
4.800	2.4411	0.38	0.38	99.62	18.24			
3.450	1.7454	0.97	0.65	99.35	9.18			
2.600	2.1369	0.33	0.98	99.02	7.92			
1.700	1.7756	0.38	1.96	98.74	4.76			
1.200	0.8152	0.13	1.39	98.61	1.56			
850	11.2605	1.75	3.14	96.86	14.88			
710	9.3621	1.45	4.59	95.41	10.30			
600	7.2602	1.13	5.72	94.28	6.28			
500	35.7903	5.59	11.31	93.69	22.95			
420	45.1928	7.01	18.32	91.69	29.44			
350	80.9600	12.57	30.89	62.11	44.00			
300	89.9032	13.80	44.69	55.31	41.40			
250	181.2226	28.85	73.54	26.46	72.13			
210	104.2215	16.19	89.73	10.27	34.00			
175	34.2966	5.42	95.16	4.34	8.50			
150	14.2413	2.29	92.45	2.55	3.43			
125	9.6416	1.49	88.94	1.66	1.96			
105	2.0862	0.45	89.39	0.61	0.47			
90	1.2222	0.30	89.69	0.31	0.27			
75	0.8877	0.15	89.34	0.16	0.11			
60	0.6152	0.10	89.34	0.06	0.05			

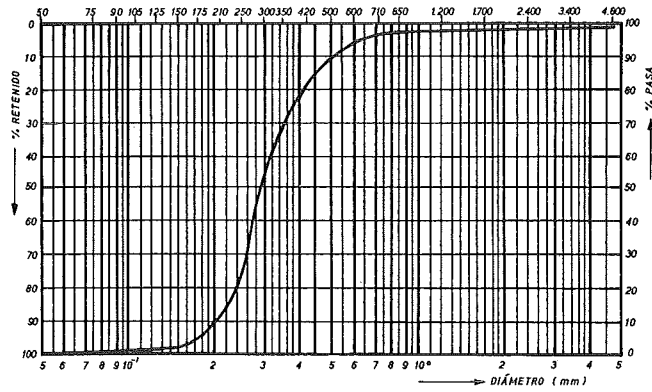


Figure 4.6.1 Sieve Analysis Form and Sieve Curve

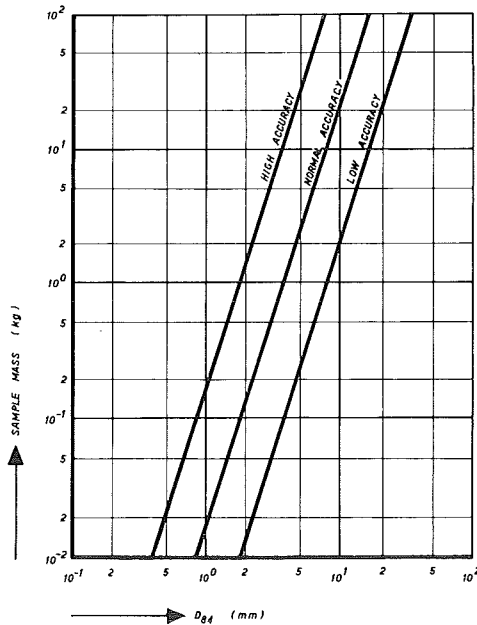


Figure 4.6.2 Design Curve for Sample Mass

samples due to the inhomogeneity of the bed material. For that reason also the standard deviation of the  $D_{50}$ -values was determined (see Part II, Tables 3.3.1 and 3.3.2). For example, the bed samples which were taken in the measuring cross-section in the Río Magdalena, upstream of the confluence with the Río Sogamoso are compiled in Table 4.6.1. The inhomogeneity of the bed material is clearly illustrated by the value of the standard deviation (compared with the  $\bar{D}_{50}$ ) and consequently a great number of samples is required.

Date	Sample mass (kg)	Accuracy	$D_{50}$ ( $\mu\text{m}$ )	$D_{84}$ ( $\mu\text{m}$ )
27 July 1971	0.30	Sufficient	440	700
28 July 1971	0.12	Too small	720	400
1 Febr. 1972	0.03	Sufficient	290	400
16 March 1972	0.11	"	270	600
9 August 1972	0.10	"	280	450
9 August 1972	0.19	"	550	950
9 August 1972	0.13	"	230	290
9 August 1972	0.13	"	250	320
			$\bar{D}_{50} = 375 \mu\text{m}$	
Standard deviation of $D_{50} = 110 \mu\text{m}$ , derived by the formula:				
$s = \sqrt{\frac{\sum (\bar{D}_{50} - D_{50})^2}{n-1}}$				

Table 4.6.1 Bed Samples taken Upstream of Río Sogamoso Confluence



REFERENCES  
AND  
LIST OF MAIN SYMBOLS



## REFERENCES

Throughout the study, the Mission frequently consulted the Julius Berger Konsortium Report (1922-1924) on their survey, and also that of the Apron y Duque Ltda., who carried out a survey in 1966. In view of the valuable information concerning the Río Magdalena contained in these Reports (particularly in the former), both are frequently referred to in this Report. For that reason, they have been listed separately.

Julius Berger Tiefbau A.G. Briske U. Prohl.: "Memoria detallada de los estudios del Río Magdalena, obras proyectadas para su arreglo y resumen del presupuesto". Bogotá, 1924. (Re-edited by the Servicio Colombiano de Meteorología e Hidrología, Bogotá, 1971).

Apron y Duque Ltda.: "Informe sobre mejoramiento de las condiciones de navegación y aprovechamiento integral de la hoya del Río Magdalena", Bogotá, 1967.

1. CHOW, V.T. (Ed.): "Handbook of applied hydrology". McGraw-Hill Book Cy, New York-Toronto-London, 1964.
2. GUMBEL, E.J.: "Statistics of extremes". New York, 1958.
3. THIESSEN, A.H.: "Precipitation averages for large areas". Monthly Weather Rev., Volume 39, pp. 1082-1084, 1911.
4. TURC, L.: "Le bilan d'eau des sols: relation entre les précipitations, l'évaporation et l'écoulement". Troisièmes Journées de l'Hydraulique: Pluie, Évaporation, Filtration et Écoulement. Alger, 12-14 avril, 1954.
5. LANGBEIN, W.B.: "Annual runoff in the United States". June 1949.
6. MANTILLA, G.E. et al.: "Memoria sobre los estudios de ampliación y rectificación del Canal del Dique". Ministerio de Obras Públicas, Colombia, 1941.
7. MANTILLA, G.E.: "Memoria sobre los estudios de ampliación y rectificación del Canal del Dique". Ministerio de Obras Públicas, Colombia, 1951.
8. DOODSON, A.T. and WARBURG, H.D.: "Admiralty manual of tides". London, H.M.S.O., 1941, 282 pp.
9. CHOW, V.T.: "Open-channel hydraulics". McGraw-Hill Book Cy. Civil Eng. Series, New York-Toronto-London, 1959.

10. BROWN, C.B.: "Sediment transportation". Proceedings of 4th Hydraulics Conference Iowa Institute of Hydraulic Research, June 12-15, 1949, (Edited by H. Rouse), John Wiley & Sons Inc., New York, 1950, pp. 796-797.
11. ENGELUND, F. and HANSEN, E.: "A monograph on sediment transport in alluvial streams". Teknisk Forlag, Copenhagen, 1967.
12. SCHULZ, E.F., WILDE, R.H. and ALBERTSON, M.L.: "Influence of shape on the fall velocity of sedimentary particles". Colorado Agric. and Mech. College, Fort Collins, Colorado, July 1954.
13. ROUSE, H.: "Modern conceptions of the mechanics of fluid turbulence". Trans. A.S.C.E., Volume 102, 1937.
14. EINSTEIN, H.A. and CHIEN, N.: "Second approximation to the solution of the suspended-load theory". Berkely Univ., Calif. Inst. Eng. Res., no. 3, 1954, 30 pp.
15. CHIEN, N.: "The present state of research on sediment transport". Proc. A.S.C.E., Volume 80, paper 565, 1954.
16. VANONI, V.A.: "Some experiments on the transportation of suspended loads". Trans. Am. Geophys. Union, 1941.
17. VANONI, V.A.: "Transportation of suspended sediment by water". Trans. A.S.C.E., Volume 111, 1946, pp. 67-133.
18. CARSTENS, M.R.: "Accelerated motion of spherical particles". Trans. Am. Geophys. Union, Volume 33, 1952.
19. BRUSH, L.M., HO, H.W. and SINGAMSETTI, S.R.: "A study of sediment in suspension", Bari, Intern. Assoc. Sci. Hydr., Commiss. Land Erosion, Pub. no. 59, 1962, pp. 293-310.
20. MATYUKHIN, V.J. and PROKOFYEV, O.N.: "Experimental determination of the coefficient of vertical turbulent diffusion in water for settling particles". Soviet Hydrol. (Am. Geophys. Union), no. 3, 1966.
21. MAJUMDAR, H. and CARSTENS, M.R.: "Diffusion of particles by turbulence: effect of particle size". Water Res. Center, Publ. no. 0967, Atlanta, Georgia Inst. of Techn., 1967.
22. SINGAMSETTI, S.R.: "Diffusion of sediment in a submerged jet". Journal of the Hydraulics Division, A.S.C.E., Volume 92, no. HY 2, 1966.

23. EINSTEIN, H.A.: "The bed-load function for sediment transportation in open channel flows". Washington, 1950, Dept. of Agric., Tech. Bull. no 1026; also in : "Sedimentation Symp. to honor prof. H.A. Einstein", Fort Collins, Colorado, 1972.
24. GRAF, W.H.: "Hydraulics of sediment transport". New York, McGraw-Hill, 1971.
25. LELIAVSKY, S.: "An introduction to fluvial hydraulics". New York, Dover Publications Inc., 1966.
26. STERNBERG, H.: "Untersuchungen über das Längen und Querprofil geschiebeführender Flüsse". Zeitschrift für Bauwesen, 1875.
27. FLAMANT, A.: "Hydraulique. Extraits des travaux de Mr. Lokhtine sur le mécanisme du lit fluvial". Paris, Libr. Béranger, 1909.
28. LOVERA, F. and KENNEDY, J.F.: "Friction factors for flat bed flows". Journal of the Hydraulics Division, A.S.C.E., Volume 95, No. HY 4, Paper 6678, July 1969.
29. EINSTEIN, H.A. and BARBAROSSA, N.L.: "River channel roughness". Transaction A.S.C.E., Paper no. 2528, 1951.
30. SIMONS, D.B. and RICHARDSON, E.V.: "Resistance to flow in alluvial channels". Journal of the Hydraulics Division, A.S.C.E., Volume 86, no. HY 5, paper 2485, 1960.
31. VANONI, V.A. and BROOKS, N.H.: "Laboratory studies of the roughness and suspended load of alluvial streams". Calif. Inst. of Technology, Pasadena, Report no. E-68, 1957.
32. MEIJER-PETER, E. and MÜLLER, R.: "Formulas for bed-load transport". Proc. Second meeting of the International Association for Hydraulic Structures Research, Stockholm, Volume 2, Paper 2, 1948.
33. SHEN, H.W.: "Development of bed roughness in alluvial channels". Journal of the Hydraulics Division, A.S.C.E., Volume 88, no. HY 3, Paper 3113, 1962.
34. ALAM, A.M.Z. and KENNEDY, J.F.: "Friction factors for flow in sand-bed channels". Journal of the Hydraulics Division, A.S.C.E., Volume 95, no. HY 6, Paper 6900, 1969.
35. ZWAARD van der, J.J.: "Roughness aspects of sand transport over a fixed bed". I.A.H.R., Istanbul, Paper A12, 1973.
36. COLBY, B.R.: "Discharge of sand-bed streams". U.S. Geological Survey, Prof. Paper 462-A, Washington, D.C., 1964.

37. TOFFALETI, F.B.: "Definitive computations of sand discharge in rivers". Journal of the Hydraulics Division, A.S.C.E., Volume 95, no. HY 1, Paper 6350, 1969.
38. TASK COMMITTEE FOR PREPARATION OF SEDIMENT MANUAL: "Sediment transportation mechanics: H. Sediment discharge formulas". Journal of the Hydraulics Division, A.S.C.E., Volume 97, no. HY 4, Paper 8076, 1971.
39. VRIES de, M.: "Hydraulic and mathematical models in hydraulic engineering practice". Univ. de Los Andes, Centro de Estudios Técnicos e Investigaciones Hidráulicas (CETIH), Seminar Bogotá, 1969.
40. VRIES de, M.: "General aspects of river morphology". Univ. de Los Andes, Centro de Estudios Técnicos e Investigaciones Hidráulicas (CETIH), Seminar Sobre Ingeniería Fluvial, Tomo 2, Bogotá, 1972.
41. VRIES de, M.: "River-bed variations-aggradation and degradation". International Seminar on Hydraulics of Alluvial Streams, I.A.H.R., New Delhi, 1973.
42. VREUGDENHIL, C.B. and VRIES de, M.: "Analytical approaches to non-steady bed-load transport". Delft Hydraulics Laboratory, Research Report S 78-IV, 1973 (to appear).
43. PRINS, A. and VRIES de, M.: "On dominant discharge concepts for rivers". Proceedings 14th Congress of the International Association for Hydraulic Research, Paris, Volume 3, Proc. Paper C 20, 1971.
44. LEOPOLD, L.B., WOLMAN, M.G. and MILLER, J.P.: "Fluvial processes in geomorphology". San Francisco, Freeman, W.H. and Co, 1964. Reviewed by: R.F. Peel in Science Progress, Volume 53, no. 209, p. 1489, 1965.
45. INGLIS, C.C.: "Meanders and their bearing on river training". Instn. of Civil Engrs., Maritime and Waterways Eng. Div. Meeting, 1947.
46. CHARLTON, F.G. and BENSON R.W.: "Effect of discharge and sediment charge on meandering of small streams in alluvium". Symposium Central Water and Power Research Station C.W.P.R.S., Volume 2, 1966.
47. ANDERSON, A.G.: "On the development of stream meanders". Proceedings of 12th Congress of the International Association for Hydraulic Research, Fort Collins, Volume 1, 1967.
48. HANSEN, E.: "On the formation of meanders as a stability problem". Basic Research Progress Rep. 13, Coastal Eng. Lab., Technical Univ. of Denmark, 1967.
49. ACKERS, P. and CHARLTON, F.G.: "Meander geometry arising from varying flows". Journal of Hydrology, Volume XI, no. 3, Amsterdam, 1970.

50. ZELLER, J.: "Flussmorphologische Studie zum Mäanderproblem". *Geographica Helvetica*, Bd. XXII, no.2, 1967.
51. LEOPOLD, L.B. and WOLMAN, M.G.: "River channel patterns: Braided, meandering and straight". U.S. Geological Survey Prof. Paper 282-B, 1957.
52. BAGNOLD, R.A.: "Some aspects of the shape of river meanders". *Geol. Survey Prof. Paper 282-E*, 1960.
53. BENDEGOM van, L., BOUWMEESTER, J., BOKHOVEN, W., BOER, P. and SPAN, H.J.T.: "Inland Navigation". Paris. 22nd Int. Nav. Congress, Section I, Subject 6, pp. 109-137, 1969.
54. BENDEGOM van, L.: "Enige beschouwingen over riviermorphologie en rivierverbetering". *De Ingenieur*, Volume 59, no. 4, pp. B1-B12, 1947.
55. SCOTT, R.F.: "Principles of soil mechanics". Reading, (Mass.), Addison-Wesley Publ. Cy., 1965.
56. WINKLEY, B.R.: "Practical aspects of river regulation and control". (Chapter 19, Volume I of "River mechanics" edited by H.W. Shen). Fort Collins, Colorado State Univ., 1971.
57. MADE van der, J.W.: "Nota betreffende de bestuurbaarheids eigenschappen van schepen, varende op kanalen". IJmuiden, RWS, Arr. Noordzeekanaal, 1955.
58. SCHÄLE, E.: "Begegnungen und Überholen von Schubverbänden in Neuen Main-Donau Kanal, sowie Berg- und Talfahrten auf dem Main". *Zeitschrift für Binnenschiffahrt*, 1969.
59. KUHN, R.: "Ergebnisse der Bamberger Kanal- und Schifffahrtsversuche und Folgerungen für die Verbreiterung von Kurven und für die Ausbildung der Dechwerke des Main-Donau Kanals". *Schiff und Hafen*, 20, no. 8, pp. 579-585, 1969.
60. Centro de Estudios Técnicos e Investigaciones Hidráulicas (CETIH): "Estudio en modelo hidráulico del Canal del Dique". Bogotá, Univ. de los Andes, Informe P2-M3, 1970.
61. Centro de Estudios Técnicos e Investigaciones Hidráulicas (CETIH): "Entrada del Canal del Dique, Estudio de sedimentación en modelo hidráulico". Bogotá, Univ. de los Andes, Informe P2-M7, 1973.
62. St. Anthony Falls Hydr. Laboratory: "The development and calibration of the visual accumulation tube". Minneapolis, Minn. (U.S.A.), Rep. no. 11, Inter Agency Committee on Water Resources, 1957.

63. St. Anthony Falls Hydr. Laboratory: "Some fundamentals on particle size analysis". Minneapolis, Minn. (U.S.A.), Rep. no. 12, 1957.
64. Delft Hydraulics Laboratory: "IJking bodemtransportmeters". Delft, D.H.L., Report M 601-III, 1969.
65. VRIES de, M.: "On the accuracy of bed-material sampling". Delft, D.H.L., Publ. no. 90, 1971.

## LIST OF MAIN SYMBOLS

A	cross-sectional area; a constant
a	$a = \frac{1}{2}k$ , where k is the Nikuradse roughness parameter; a constant
B	width of channel
C	Chézy coefficient, related to total roughness; concentration of suspended-load
C'	Chézy coefficient, related to the bed forms
C''	Chézy coefficient, related to the grains ( $C'' = 18 \log 12h/k$ )
$C_a$	reference concentration of suspended-load, at height a (= 0.1h) above the bed
c	celerity
$c_1$	$v + \sqrt{gh}$
$c_2$	$v - \sqrt{gh}$
$c_3$	celerity of discontinuity of $\partial z/\partial x$ and $\partial z/\partial t$
D	particle diameter
$D_{90}, D_{50}, D_{10}$	mesh of sieve which is passed by 90%, 50% and 10% respectively, of a grain-mixture
e	base of natural logarithm
Fr	Froude number
f	Darcy-Weisbach friction factor ( $= 8g/C^2$ )
f'	Darcy-Weisbach friction factor, related to the bed forms
f''	Darcy-Weisbach friction factor, related to the grains
G	weight
g	acceleration of gravity
H	energy head
h	depth of flow
h'	depth of flow, related to the bed forms
h''	depth of flow, related to the grains
I	water-level gradient; gradient of energy head
I'	gradient of energy head, related to the bed forms
I''	gradient of energy head, related to the grains
$I_o$	water-level gradient along the outer bend
i	indicates rank-number of interval, time or place step
k	Nikuradse roughness parameter
L	length
m	mass
n	Manning coefficient

P	wetted perimeter; probability
Q	total discharge
$Q_o$	dominant discharge
q	discharge per unit of width
R	hydraulic radius (= $A/P$ ); radius of river bend
$R_o$	radius of outer bend
Re	Reynolds number
S	sediment transport in volume per unit of time
$S_b$	bed-load
$S_s$	suspended-load
s	sediment transport per unit of width
T	sediment transport in solid volume per unit of time
t	time parameter
$\Delta t$	time interval
v	velocity
$\bar{v}$	mean velocity (= $q/h$ and = $Q/A$ )
$v_*$	shear velocity (= $\sqrt{ghI}$ )
w	fall velocity
X	transport parameter (= $T/\sqrt{\Delta g D^3}$ )
x	co-ordinate along the river axis
Y	flow parameter (= $\Delta D/\mu h I$ )
y	co-ordinate
z	bed-level, related to reference-level; exponent (= $w/(k v_*)$ ); co-ordinate
$\alpha$	a constant
$\beta$	$\epsilon_s/\epsilon$
$\gamma$	angle of side slope; specific weight
$\Delta$	relative density (= $(\rho_s - \rho)/\rho$ )
$\delta$	height of bed forms; thickness of viscous sub-layer ( $\approx 12\nu/v_*$ )
$\epsilon$	diffusion coefficient; porosity
$\epsilon_s$	diffusivity of solid particles
$\theta$	time interval
$\kappa$	von Karman's constant (for clear water, $\kappa = 0.4$ )
$\mu$	ripple factor (= $\tau/\tau'$ )
$\nu$	kinematic viscosity

$\pi$	3.1415
$\rho$	density of fluid
$\rho_s$	density of sediment
$\tau$	shear stress along the bed
$\tau'$	shear stress, related to the bed forms
$\tau''$	shear stress, related to the grains

