bm 337647

UNIVERSITY OF STRATHCLYDE
DEPARTMENT OF CIVIL ENGINEERING

THE CHANGING RIVER BASIN: A PROCESS APPROACH TO RESERVOIR MANAGEMENT

VICTOR ONOSAGHOGHO ORITSEBEMIGHEYI OKE

Phd. 1991

A thesis submitted in fulfilment of the requirement for the degree of Doctor of Philosophy in Water and Environmental Engineering in the Civil Engineering Department of the University of Strathclyde.

LIST OF CONTENTS

		PAGE
LIST OF FIGURES		
LIST OF TABLES		
LIST OF PLATES		
SYNOPSIS		1
ACKNOWLEDGEMENTS		6
CHAPTER 1.	INTRODUCTION	8
1.1	The Definition of the Problem	10
1.2	Scope	11
1.3	The Reservoir Management Problem in River Basins.	31
1.3.1	The Spatial and Temporal Distribution of Water and Allocation	31
1.3.2	Impoundment Induced Climatic Changes	21
1.3.3	Landuse Impact on Climate and Sedimentation	24
1.3.4	Reservoir Sedimentation Patterns: Dead Storage Level	26
1.3.5	Global Climatic Changes	32
1.3.6	Problem Summary	36
1.4	Research Strategy	36
1.5	Case Study	42
1.6	Final Remarks	45

CHAPTER 2. RIVER BASIN WATER MANAGEMENT: PRACTICE FAILURES 46

2.1 The Management of Water Resources 47

2.1.1 General 47

2.1.2 Multipurpose Development

2.1.3 Multidisciplinary Management 50

2.1.4 Integrated Basin Management 51

2.1.5 The Inter-Basin and Multi-Basin Management Approach 53

2.1.6 Systems Approach 54

2.2 Planning and Water Resources Management 57

2.2.1 Definition 57

2.2.2 Analysis and Appraisal 58

2.2.3 Other Planning Approaches 58

2.2.4 Project Choice and Implementation 58

2.2.5 Strategy 59

2.3 Reservoir Management 60

2.3.1 The Planning of Reservoirs 60

2.3.2 Assessing Yields 61

2.3.3 Reservoir Operations 63

2.3.3.1 The Role of Simulation Time Periods. 65

2.3.4	Reservoir Operation Rules and Rule Curves	67
2.4	Uncertainties in River Basin Management	69
2.4.1	Hydrological Uncertainties	69
2.4.2	Economic Uncertainties	70
2.4.3	Assessing Uncertainties	70
2.5	Managing Hydrological Extremes and Uncertainties	71
2.5.1	The Process of the Hydrological Cycle	71
2.5.2	Drought Management	73
2.5.3	Floods and Flood Management	74
2.5.4	The Role of Minimum Reservoir Operating Levels	75
2.6	Examples Highlighting The Failures of Current Management Strategies	75
2.6.1	The Aswan Case	75
2.6.2	The Akosombo Case	79
2.6.3	The Kainji Reservoir Case	84
2.7	The Role of Data Management	86
2.7.1	Hydrological Data	86
2.7.2	Other Data	87
2.7.3	Data Generation	88
2.8	Final Remarks	89

CHAPTER 3.		THE SEDIMENT PROBLEM PROCESS ASSESSMENT AND MANAGEMENT	92
	3.1	Definition of The Sediment Problem	94
	3.2	River Basin Erosion	94
	3.2.1	General	94
	3.2.2	Reviewing Sediment Assessment and Methods	95
	3.3	Sediment Erosion Assessment and Management	97
	3.3.1	Sheet Erosion	97
	3.3.2	Non-Sheet Erosion	101
	3.4	Sediment Transport Process and Sediment Yield Assessment	104
	3.5	Reservoir Sedimentation	106
	3.5.1	General	106
	3.5.2	Assessment and Management	108
	3.5.3	Examples and Economic Costs	110
	3.6	Managing Reservoir Sedimentation	111
	3.7	Final Remarks	112
CHAPTER 4.		MATHEMATICAL MODELLING IN RIVER BASIN MANAGEMENT	117
	4.1	Introduction - The Mathematical Modelling Techniques	119
	4.2	Classification of Mathematical Models	120

	4.3	Computer Simulation	134
	4.3.1	Definition	134
	4.3.2	Methodology	135
	4.3.3	Limitations	138
	4.3.4	Applications	139
	4.4	River Basin Hydrological Models	141
	4.4.1	River Basin Processes and Modelling	141
	4.4.2	Models and Modelling methodology review	144
	4.5	River Basin Sediment Models	154
	4.6	The Strathclyde River Basin Model	157
	4.7	Final Remarks	162
CHAPTER 5.		RIVER BASIN MANAGEMENT SYSTEM: WATER AND SEDIMENT COMPONENTS	168
	5.1	System Description Outline	169
	5.2	Modification of System Components	169
	5.3	The System as a User-Friendly Management System	179
	5.4	The Watershed Model	189
	5.5	The Routing Model	191
	5.6	The Watershed - Routing Model Modification	194
	5.7-5.8	The Erosion and Sediment Routing Model	200

	5.9	The Reservoir Delta Model	216
	5.10	Final Remarks	
CHAPTER 6.		ECONOMICS AND RIVER BASIN MANAGEMENT: THE ECONOMIC SIMULATION MODEL	168
	6.1	Introduction	234
	6.2	Water Resources Management and Third World Economics	236
	6.3	Economic Method in River Basin Management	243
	6.4	The Economic Simulation Model	247
	6.4.1	General	247
	6.4.2	Model Operations	250
	6.5	The Economic Simulation Model Structure	252
	6.5.1	The Evaluation of Costs and Data Requirements	252
	6.5.2	Gordon's Techniques for Estimating Hydropower Costs	
	6.5.3	Estimating Non-Power Costs	275
	6.5.4	Evaluation of Benefit	290
	6.6	Final Remarks	298
CHAPTER 7.		THE MAGNITUDE OF THE GLOBAL CLIMATE CHANGE	299
	7.1	Introduction	301
	7.2	Facts and Figures	302

	7.3	Quantifying the Global Climate Conditions	308
	7.4	Global Climatic Change: The Humid Tropical Regions	338
	7.5	The Donga Basin: Current Hydrological Conditions	340
	7.6	The Global Climatic Change: The Donga Basin Case	351
	7.7	Final Remarks	353
CHAPTER 8.		DONGA BASIN WATER BALANCE ANALYSIS	355
	8.1	Running the Watershed Model	358
	8.2	Running the Routing Model	360
	8.3	Watershed Model Calibration Techniques	362
	8.4	Watershed Model Calibration Runs and Water Discharge Simulation	365
	8.5	Testing the Impacts of Short Term Landuse Changes	378
	8.6	Testing the Impact of Reservoir Induced Changes	380
	8.7	Testing the Impact of Global Climate Changes	382
	8.8	Final Remarks	384

CHAPTER 9.		DONGA BASIN SEDIMENT ANALYSIS	445
	9.1	Introduction	447
	9.2	Sediment Yield of the Donga Basin	449
	9.3	Calibration of the Erosion Model - Donga Basin Case	460
	9.4	Testing Landuse Changes in the Short Term	466
	9.5	Testing the Impact of Impoundment Induced Climate Changes	468
	9.6	Testing the Impact of Global Climate Change	470
	9.7	Conclusion and Final Remarks	471
CHAPTER 10.		GEMBU RESERVOIR OPTIMISATION STUDY: STEADY STATE AND SHORT TERM SCENARIO: ANALYSIS AND DISCUSSION OF RESULTS	473
	10.1	Economic Tests Methodology	475
	10.2	Testing Water Allocation to Various Users for both Highead and Lowhead Cases	478
	10.3	Testing the Sensitivity of the Dead Storage Capacities Gembu Reservoir Scheme	483
	10.3.1	The High Head Case (960m)	

	10.3.2	Testing the Sensitivity of the Dead Storage Capacity by Lowering Minimum Operating Levels: Gembu Reservoir	485
	10.4	Testing the Effects of Changing Landuse Paramaters	486
	10.5	Testing the Influence of Local Climate Change Resulting from Reservoir Construction	501
	10.6	General Discussion of Results	502
	10.7	Summary of Research Finding and Conclusion	507
CHAPTER 11.		GEMBU RESERVOIR MANAGEMENT, DONGA RIVER BASIN TESTING LONG TERM ECONOMIC VIABILITY FROM CLIMATE CHANGES:ANALYSIS AND DISCUSSION	510
	11.1	Introduction	510
	11.2	Presentation of Gembu Reservoir Simulation Results - Case 1	515
	11.3	Economic Simulation Results for Gembu Reservoir - Case 2	525
	11.4	General Analysis of Results with Comments	527
	11.5	Analysing Case 1 Results	529
	11.6	Analysing Case 2 vis-a-vis Case 1 Results	531
	11.7	Conclusions	540

CHAPTER 12. GENERAL CONCLUSION AND RECOMMENDATIONS 543

12.1 Study Achievements and Contributions 544

12.2 Specific Conclusions for the Donga Basin Study 545

12.3 General Conclusions 548

12.4 Recommendations 550

REFERENCES

APPENDIX

LIST OF FIGURES

CHAPTER 1

Fig 1.1a Global Water surplus and Deficiency.

Fig 1.2/1.1 Long term hydrograph for Kainji Reservoir, Nigeria .

Fig 1.3 Kainji Town long term hydrographs.

Fig 1.4 The Sahel region of Africa : showing the areal extent of the drought.

Fig 1.5 The Reservoir environment.

Fig 1.6 Profile of typical reservoir delta and nomenclature for deposits.

Fig 1.7 Deposition pattern along the Colorado River through Lake Mead.

Fig 1.8 River Basin Management problem.

Fig 1.9(a) Defining the Process Approach to Reservoir Management nd comparison with Non-Process Approach.

Fig 1.9(b) River Basin Management system.

Fig 1.10 Donga River Basin proposed reservoir sites.

CHAPTER 2

Fig 2.1 Water resources assessment and requirement.

Fig 2.2 Reservoir storage zones.

Fig 2.3 The Nile Basin.

Fig 2.4 Volta River Basin.

CHAPTER 3

Fig 3.1/3.2 Soil Erosion Process.

CHAPTER 4

Fig 4.1 Mathematical Methods In Hydrology.

Fig 4.2 A typical flood frequency curve.

Fig 4.3 System Approach to Water resources planning.

Fig 4.4(a) Planning via Simulation versus Optimisation.

Fig 4.4(b) General Simulation procedure.

Fig 4.5 The Mathemathical Model concept (and the calibration loop)

Fig 4.6 Data requirements in Mathematical Model

Fig 4.7 Acres Model.

Fig 4.8 Erosion Model components.

Fig 4.9 River basin model components

Fig 4.10 Flow chart of Modified Stanford Watershed Model.

Fig 4.11 Logic chart of the Channel Network Flow Routing Model.

Fig 4.12 Linkage of the Watershed and Sediment models.

Fig 4.13 Flow Chart of Fleming model.

CHAPTER 5

Fig 5.1 The operations of the River Basin Management Sys tem.

Fig 5.2 Microcomputer File allocation network for River Basin management.

Fig (5.2 b) 5.3 a) File and Channel allocation network for the River Basin management system.

Fig 5.3(b) Sediment deposit in Reservoirs: Variation of density with age and submergence.

Fig (5.3b1) 5.4 River Basin Management System multiple menus

Fig 5.4(a) DR Graph (after Digital Research,1983).

Fig 5.5 System menu.

Fig 5.6 Cursor Menu

Fig 5.7(a) River Basin Management System.

Fig 5.7(b) Sketch of a Channel section with Overbank Flow.

Fig 5.8 Flow chart of the segmented land erosion model showing particle size treatment and the interaction with the channel phase.

Fig 5.9(a) Structure of Velocity field.

Fig 5.9(b) Orientation of Computational Grids at river mouth.

Fig 5.10 Typical simulated Gembu Reservoir Delta.

Fig 5.11 Flow chart indicating structure and direction of flow between Subroutines of DELTAISM.

CHAPTER 6

Fig 6.1 River Basin Management System.

Fig 6.2 Structure and Operation of the Economic Simulation Model.

Fig 6.3 Cost energy curves for Nigerian reservoirs.

Fig 6.4 Cost capacity curves for Nigerian reservoirs.

Fig 6.5 Cost of tunneling in hard rock.

Fig 6.6 Donga River Basin: Reservoir sites & Tunnel path

Fig 6.7 Screening guide for estimating the cost of convention hydroelectric power plants.

Fig 6.8 Total equipment cost from recent quotations from Manufacturers versus Plant Capacity and Rated Head, 1982 Base year.

Fig 6.9 Site Factor - Plant capacity Envelope.

Fig 6.10 Relationship between cost and MW/H for projects estimated or built between 1956 and 1971, and costs escalated at three per cent per annum to 1971.

Fig 6.11 Relationship between cost and MW/H for projects where head exceeds 350m.

Fig 6.12 Relationship between cost and MW/H for projects estimated or built between 1977 and 1982, and costs escalated at 12 percent per annum to1982.

Fig 6.13 Cost per installed unit of capacity for recent hydroprojects.

CHAPTER 7

Fig 7.1(a) The Global Temperature Change.

Fig 7.1(b) Sea level rise from Global Warming.

Fig 7.2(a) Current Mean Temperature -January.

Fig 7.2(b) Future Mean Temperature - January (Minimum Scenario).

Fig 7.2(c) Future Mean Temperature - January (Mild Scenario).

Fig 7.2(d) Future Mean Temperature - January (Moderate Scenario).

Fig 7.2(e) Future Mean Temperature - January (Severe Scenario).

Fig 7.2(f) Future Mean Temperature - January (Maximum Scenario).

Fig 7.2(1) Current Mean Temperature - July.

Fig 7.2(2) Future Mean Temperature - July (Minimum Scenario).

Fig 7.2(3) Future Mean Temperature - July (Mild Scenario).

Fig 7.2(4) Future Mean Temperature - July (Moderate Scenario).

Fig 7.2(5) Future Mean Temperture - July (Severe Scenario).

Fig 7.2(6) Future Mean Temperature - July (Maximum Scenario).

Fig 7.3 Global climate classification .

Fig 7.4 Global mean annual precipitation.

Fig 7.4(a) Global Mean Annual precipitation (mild minimum scenario) .

Fig 7.4(b) Global Mean Annual precipitation (moderate scenario).

Fig 7.4(c) Global Mean Annual precipitation(severe scenario of Global warming).

Fig 7.4(d) Global Mean Annual Precipitation (maximum scenario).

Fig 7.5 Thornthwaites Method used to calculate present and future evapotranspiration from temperature values.

Fig 7.6 Evapotranspiration verses temperature charts.

Fig 7.7(a) Global potential evaporation from land surface.

Fig7.7(b) Future Global potential evaporation from land surface minimum 5% increase scenario.

Fig 7.7(c) Future potential evaporation from land surface - Mild scenario 10% increase.

Fig 7.7(d) Future potential evaporation from land surface - Moderate scenario 15% increase.

Fig 7.7(e) Future potential evaporation from land surface - Severe scenario of Global warming + 20% increase.

Fig 7.7(f) Future potential evaporation from land surface - Very severe scenario (35% increase).

Fig 7.7(g) Future potential evaporation from land surface - Maximum scenario.

Fig 7.8 Map of Nigeria.

Fig 7.9 Donga River Basin : Proposed reservoir sites & relief.

Fig 7.10 Annual Rainfall for Donga catchment (1959-1980).

Fig 7.11(a) Donga Basin future precipitation : Mild scenario (5% inc).

Fig 7.11(b) Donga Basin future precipitation : Moderate scenario (10% increase).

Fig 7.11(c) Donga Basin future precipitation : Severe scenario. (15% increase)

Fig 7.11(d) Donga Basin future precipitation : Maximum scenario (20% increase).

CHAPTER 8

Fig 8.1 Donga River Basin :Proposed reservoir sites & sub-basins.

Fig 8.2 Donga River Basin :Proposed reservoir sites & reach division.

Fig 8.3 Simplified flowchart showing the Reservoir flow Routing processes.

Calibration, verification and validation for Donga Basin Water balance study.

Fig 8.4 Donga basin to Tepkwar/Gembu (1976-78 , 1981-82).

Fig 8.5 Donga Basin to Abong town (1976-78, 1981-82).

Fig 8.6 Donga Basin to Manya (1959-62).

Testing landuse changes,IMPV and EPXM.

Fig 8.7 Hydrograph based on Initial conditions.

Fig 8.7(a) 1. Hydrograph (IMPV=0.01); {Year 1976-77}

2. Hydrograph (IMPV=0.1); {Year 1976-77}.

3. Hydrograph (IMPV=0.0); {Year 1977-78}.

4. Hydrograph (IMPV=0.05); {Year 1977-78}.

5. Hydrograph (IMPV=0.10); {Year 1977-78}.

6. Hydrograph (IMPV=0.01); {Year 1981-82).

7. Hydrograph (IMPV=0.01); {Year 1981-82}.

Fig 8.7(b) 1. Hydrograph (EXPM=7.5/9.0); {Year 1976-77}.

2. Hydrograph (EXPM=0.8/1.0); {Year 1976-77}.

3. Hydrograph (EXPM=0.8/1.0); {Year 1981-82}.

4. Hydrograph (EXPM=7.5/9.0); {Year 1981-82}.

5. Hydrograph (EXPM=7.5/9.0); {Year 1977-78}

6. Hydrograph (EXPM=0.8/1.0); {year 1977-78}.

Testing Reservoir-Induced Climate Changes.

Fig 8.8 1. Hydrograph for current precipitation +0% changes. {Year 1976-77}

2. Hydrograph for current precipitation +5% changes. {Year 1976-77}

3. Hydrograph for current precipitation +10% changes. {Year 1976-77)

4. Hydrograph for current precipitation +0% changes.

{Year 1977-78}

5. Hydrograph for current precipitation +5% changes. {Year 1977-78}

6. Hydrograph for current precipitation +10% changes. {Year 1977-78}

7. Hydrograph for current precipitation PPT + 0% (1981-82)

8. Hydrograph for current precipitation PPT + 5% (1981-82)

9. Hydrograph for current precipitation PPT + 10% (1981-82)

Testing the combination of reservoir and globally induced climatic changes.

Fig 8.9

1. Hydrograph for current (precipitation +14%)and (evaporation +0%). {Year 1976-77}.

2.Hydrograph for current (precipitation +14%) and (evaporation +2.5%). {Year 1976-77}.

3. Hydrograph for current (precipitation +14%) and (evaporation +5%). {Year 1976-77}.

4.Hydrograph for current (precipitation +14%) and evaporation +7.5%). {Year 1976-77}.

5.Hydrograph for current (precipitation +14%) and (evaporation +10%). {Year 1976-77}.

6.Hydrograph for current (precipitation +14%) and (evaporation +12.5%). {Year 1976-77}.

7.Hydrograph for current (precipitation +16.5%) and (evaporation +0%). {Year 1976-77}.

8.Hydrograph for current (precipitation +16.5%) and (evaporation +2.5%). {Year 1976-77}.

9.Hydrograph for current (precipitation +16.5%) and (evaporation +5%). {Year 1976-77}.

10.Hydrograph for current (precipitation +16.5%) and (evaporation +7.5%). {Year 1976-77}.

11.Hydrograph for current (precipitation +16.5%) and (evaporation +10%). {Year 1976-77}.

12.Hydrograph for current (precipitation +16.5%) and (evaporation +12.5%). {Year 1976-77}.

13.Hydrograph for current (precipitation +20%) and (evaporation +0%).{Year 1976-77}.

14.Hydrograph for current (precipitation +20%) and

(evaporation +2.5%). (Year 1976-77).

15.Hydrograph for current (precipitation +20%) and (evaporation +5%). (Year 1976-77).

16.Hydrograph for current (precipitation +20%) and (evaporation +7.5%).(Year 1976-77).

17.Hydrograph for current (precipitation +20%) and (evaporation +10%). (Year 1976-77).

18.Hydrograph for current (precipitation +20%) and (evaporation +12.5%). (Year 1976-77).

19.Hydrograph for current (precipitation +24%) and (evaporation +0%). (Year 1976-77).

20.Hydrograph for current (precipitation +24%) and (evaporation +2.5%). (Year 1976-77).

21.Hydrograph for current (precipitation +24%) and (evaporation +5%). (Year 1976-77).

22. Hydrograph for current (precipitation +24%) and (evaporation +7.5%). (Year 1976-77).

23.Hydrograph for current (precipitation +24%) and (evaporation +10%). (Year 1976-77).

24.Hydrograph for current (precipitation +24%) and (evaporation +12.5%). (Year 1976-77).

CHAPTER 9

Fig 9.0 RBMS
Fig 9.1 River Donga at Nyivu; Sediment/Discharge relationship.

Fig 9.2 River Donga at Tepkwar; Discharge/suspended relationship.

Fig 9.3 Dimensionless sediment/discharge relationship for Donga Basin

Fig 9.4(a) R.Donga at Tepkwar: flow duration curve.

Fig 9.4(b) River Donga at Nyivu: flow duration curve.

CHAPTER 10

GEMBU RESERVOIR ECONOMIC ANALSIS.

Fig 10.0 RBMS
Fig 10.1(a) Effect of water allocation to users.

Fig 10.1(b) Effect of water allocation to users.

Fig 10.1(c) Effect of water allocation to users.

Fig 10.2(a) Testing the effect ofminimum levels.

Fig 10.2(b)Testing the effect of minimum levels.

Fig 10.2(c)Testing the effect of minimum levels.

Fig 10.2(d)Testing the effect of minimum levels.

Fig 10.2(e)Testing the effects of Minimum levels.

Fig 10.2(f)Testing the effect of Minimum levels.

Fig 10.2(g)Testing the effects of Minimum levels.

Fig 10.2(h)Testing the effects of minimum levels.

Fig 10.3(a)Short term effects of landuse changes. (Year 1976-77).

Fig 10.3(b)Short term effects of landuse changes. (Year 1977-78).

Fig 10.3(c)Short term effects of landuse changes. (Year 1981-82).

Fig 10.4 Effects of Reservoir induced climate.

CHAPTER 11

Fig 11.0 River Basin Management System.

Fig 11.1 Comparison of Hydrographs :Long term and 1976-77.

GEMBU RESERVOIR ECONOMIS ANALYSIS

Fig11.2(a) Effect of climate change on benefits.

Fig11.2(b) Effect of climate changes.

Fig11.2(c) Effects of climate changes.

Fig11.2(d) Effect of climate changes.

Fig11.2(e) Effect of climate changes.

Fig11.3(a) Effects of climate changes on benefits.

Fig11.3(b) Effects of climate changes.

Fig11.3(c) Effects of climate changes.

Fig11.3(d) Effects of climate changes.

Fig11.3(e) Effects of climate changes.

Fig 11.4 The Reservoir Environment :Economic Factors.

LIST OF TABLES

Table 1.1	Surface Area of Water in reservoirs by Region
Table 1.2	Volume of Water Stored in Reservoirs by Region
Table 2.1	Freshwater Availability and use by Country.
Table 3.1	Sediment Yield Assessment at Manya, Donga Basin.
Table 5.1	Typical computer Print of Reservoir Data.
Table 5.2	Sensitivity of Catchment Sediment Yield to Climate and Landuse Changes Donga,River Basin Yields at Gembu Reservoir and Abong.
Table 6.1	Debt Indicator by Country in Africa.
Table 6.2	External Debt Indicators by Selected Countries
Table 6.3	Ghana's Capital Expenditure for 1986-88.
Table 6.4	Computer Summary of Economic Model Results.
Table 6.5	Gembu Reservoir Project Cost Estimates by Model and by Consultants.
Table 6.6	Project Cost Estimates for Existing Nigerian Reservoirs.
Table 6.7	Typical Indian Crops and their Water Requirements.
Table 6.8	Water Demand by Countries.
Table 6.9	Electricity Tariffs in Some West African Countries.
Table 7.1	Regional Scenarios for Climate Change.
Table 7.2	Range of Expected Temperature Changes by Regions.
Table 7.3	Range of Expected Global Evapotranspiration Changes.
Table 7.4	Mean Monthly Flows, Donga Basin, Nigeria.
Table 7.5	Mean Annual Temperatures for Eight Stations in the Donga Basin.
Table 7.6	Annual Potential Evapotranspiration at Gembu, Donga Basin.
Table 8.1	Donga River to Tepkar, Final Calibration Parameters.
Table 8.2	Donga River to Abong, Final Calibration Parameters
Table 8.3	Donga River to Manya, Final Calibration Parameters.
Tables8.4	Donga to Gembu, Calibration (76-77), Verification (77-78), and Validation (81-82) Flow Summaries.
Tables8.5	Donga to Abong, ,,
Tables8.6	Donga to Manya, ,,

Tables8.7 Total Annual Simulated Runoff and Runoff Peaks at Abong due to landuse Changes.

Tables8.8 ditto, due to Impoundments Induced Climate Change

Table 8.9 ditto, due to Global Climate Changes.

Table 9.1 Sediment Loads in African Rivers.

Table 9.2 Results of Calibration Run of Erosion Model.

Table 9.3 Simulated Sediment Yields for the Donga Basin by Year.

Tables9.4 Simulated Annual Sediment Yields by Landuse Parameters.

Table 9.4(a) Gembu Reservoir Elevation-Capacity Curves as affected by Landuse Changes.

Table 9.5 Simulated Annual Sediment Yields as affected by Impoundments Induced Climate.

Table 9.5(c) Gembu Reservoir elevation-Capacity Curves as Affected by Climate Changes.

Tables9.6 Simulated Annual Sediment Yields as affected by Global Climatic Change.

Table 10.0 Typical Economic and Operation Output of River Basin Management System.

Tables 10.1 Economic Results of Water Allocation Tests for Gembu Reservoir.

Tables 10.2 Economic Results of Minimum Level Tests for Gembu Reservoir.

Table 10.3 Short Term Economic Results for Landuse Changes.

Table 10.4 Short Term Economic Results from Impoundment Induced Climate Change.

Table 11.0 Typical Long Term Economic-Operations Model output.

Tables 11.1 Long Term Economic Results for Gembu Reservoir Case 1 design for Global Climatic Changes.

Tables 11.2 Long Term Economic Results for Gembu Reservoir Case 2 design for Global Climatic Change.

LIST OF PLATES

Plate A Rill and Sheet Erosion, Donga Basin, Nigeria.

Plate B Mass and Gully Erosion, Donga Basin, Nigeria

Plate C Sheet and Rill Erosion, Donga Basin, Nigeria.

PLATE D Sheet and Rill Erosion, Donga Basin, Nigeria.

Plate E Gully and Rill Erosion, Donga Basin, Nigeria.

THE CHANGING RIVER BASIN: A PROCESS APPROACH TO RESERVOIR MANAGEMENT

SUMMARY

Surface Water Reservoirs are constructed to store and provide water for optimal use, on a long term basis. To achieve this, Water resources designers and managers must adequately address the problems of sedimentation and the spatial and temporal distribution of water (e.g droughts and floods). Sedimentation severely affects reservoir live storage capacities, thus affecting their economic performance. Further, the untold human suffering and the economic losses associated with droughts and floods cannot be overemphasized.

Clearly, the construction of a reservoir changes the entire river basin response and the affected population is now dependent on the new system created, which must be maintained or even extended to meet the change in growth and development stimulated by the premier impoundment.

Reservoir management is further complicated by climate and land use changes in river basins. Climate shift can be observed on a macro scale, e.g,

(a) the observed global climate changes caused by the 'Greenhouse effect'. Actually, global warming is already taking place as result of the accumulation of

greenhouse gases (GHGs) (CO2, CFC's. etc) and this may worsen. (UNEP/WMO/ICSU, 1985; 1987).

and on a micro scale, e.g,

(b) from the construction of reservoirs. The phenomenon of local or micro climate changes associated with or induced by natural or man-made lakes and other detained surface water such as swamps are well described in the literature (Mather, 1984).

Past hydrological data can be inappropriate for use in representing future conditions for the design and operation of water resource projects. This is because the traditional assumption that long term historical data is representative of the future trend in hydrological events is incorrect, if climate change is proved. Even, in cases where future hydrological data, such as rainfall and stream flows, are generated stochastically or deterministically, appropriate functions representing the changing trends in climate should be incorporated to make the results more valid. This is especially so as these methods tend to use historical data as an analytical base.

This research is focused primarily on the hydrological and economic impact of environmental changes on reservoir management policy. The impact analysis is based on a computer simulation approach. The mathematical models used herein are deterministic and incorporate the catchment sediment and hydrological processes draining to a reservoir system.

Recent reservoir studies have been directed mainly at reservoir operation and involve the optimisation of releases and storage capacity allocations while overlooking the economic impact of the interacting sediment and hydrological processes. For example, the black box approach has led to a policy of specifying excessive dead storages to take care of sediment deposits. Sometimes, the total reservoir sizes are made unnecessarily large to accommodate the large dead storage capacities. In reality and unfortunately, it is the live storages that are rapidly depleted by sedimentation in most reservoirs. This constitutes another aspect of the reservoir design and management problem which needs to be tackled. The process approach adopted herein addresses this by simulating the real time and real life sedimentation patterns and optimising the dead storage levels by simulation.

The technique utilised involves the use of a River Basin Management System built by interlinking a modified Stanford watershed model (Linsley and Crawford, 1966; Fleming and McKenzie, 1982); the Strathclyde erosion model; and a system operation and routing model, (Fleming and McKenzie, 1983); reservoir delta simulation model (Bonham-Carter and Sutherland, 1968) and an economic analysis simulation model (Gordon 1978; 1983) as shown in the figure overleaf. The routing and delta models used are extensive modifications of the original while the economic simulator was developed as required for economic analysis.

The System is available for use on both micro and mainframe computers.

In testing the system, the Donga River Basin in Nigeria was used as a case study with particular emphasis on the Gembu Reservoir Project. The impact of future climate and landuse changes on the hydrology of the catchment and the economic performance of the scheme were assessed for both short and long term scenarios.

Before the tests, the magnitude of global climate change, coupled with consequent changes expected in the Donga Basin were evaluated and are presented.

From the test results obtained, it was observed that the combined effect of increases in precipitation and evapotranspiration resulting from global warming and landuse practices will:

(a) Increase or decrease catchment runoffs and sediment yields depending on the scenario of global warming or landuse practices.

(b) Reduce the long term economic benefits from the Gembu reservoir scheme as designed. The extent of the losses in benefits depends on the scenario of warming. A slightly larger Gembu reservoir, on testing, gave higher benefits.

From the results, it is concluded that:

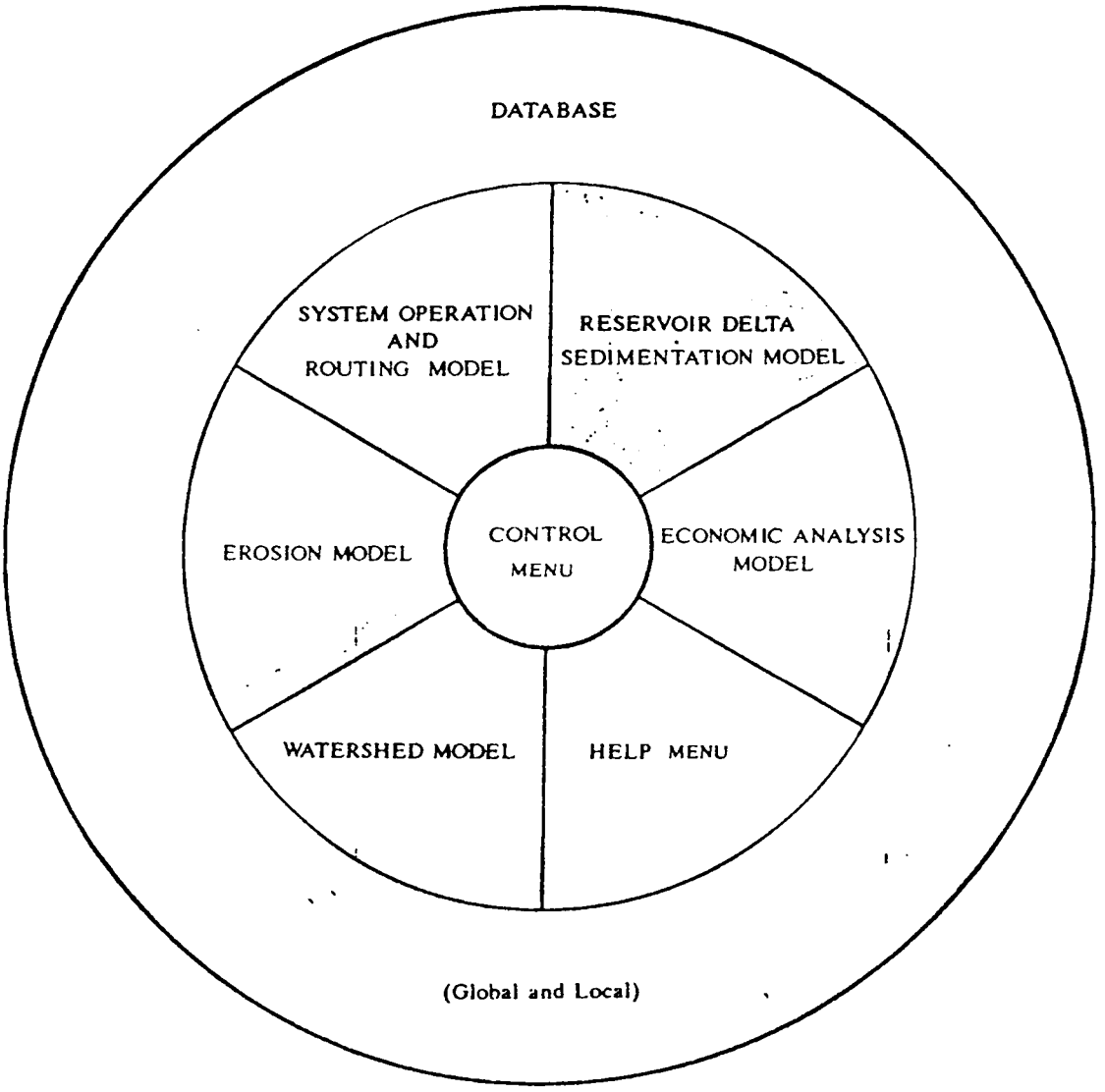
(a) The global climate change will have a significant effect on the hydrology of the Donga Basin and on the economics of the proposed Gembu Reservoir project. Other

catchments and reservoirs elsewhere may be significantly affected by the global climate shift and this aspect should be incorporated in their management.

(b) The process approach provides a step forward in the attainment of more optimal design and management of Water resources storage schemes in a changing environment.

(c) The working River Basin Management System has the potential for the optimal management of river basin/reservoir schemes.

(d) A major potential of this research is that the sediment and hydrological processes and changes can be monitored deterministically. This process approach can be adopted and incorporated into optimisation models for long term and real time reservoir operation. Alternatively, the optimisation of operations can be incorporated into process-based deterministic models, such as the River Basin Management System. These additions will provide better results in the reservoir management process.



RIVER BASIN MANAGEMENT SYSTEM

This system is available for use on both micro and mainframe(VAX) computers.

ACKNOWLEDGEMENTS

The Author wishes to thank his Supervisor, Professor G. Fleming, specially for his immense assistance and encouragement throughout the period of this research. Thanks are due also the members of staff and students of the Water and Environmental Management Group of the Civil Engineering Department of the University of Strathclyde for their friendliness and any assistance offered in the latter stage of the study.

Gratitude is extended to the EEC and NEPA for jointly sponsoring the River Basin Simulation Management Course here at the Civil Engineering Department thus enabling the write up of this research.

Thanks are due to Mr Tom Tennant for assisting with the drafting of some of the drawings. Thanks are also extended to my family; especially to Rukky my son, and Alero, my daughter for their patience during the entire length of the study. Special thanks are due to Alero in particular as she was delivered in my absence in Nigeria and did not lay her eyes on me until my return from the study.

My parents, late Grace. O. Oke and the late Mr. R.E.A. Oke, late school teacher and late Headmaster respectively of Christ African Church Primary School, Warri, Nigeria deserve untold gratitude for their care and prayers in the early part of my existence on this planet.

DEDICATION

This work is dedicated to my late mother, Grace Olumaigin Oke, whose death in the final days of this work (Aug. 12, 1990) caused me a lot of pain.

CHAPTER 1

CHAPTER 1: GENERAL

In Chapter 1 the problem addressed by this study will be defined and discussed. Prior to the problem discussion, a brief chapter by chapter outline of the thesis will be provided. In addition, the main objectives of the research will be presented. The approach and techniques used in assessing the stated problem will be described.

1.1 The Definition Of The Problem

Surface water reservoirs are constructed to store and provide water, on a continuous basis over long time periods. To achieve this, engineers must address the problem associated with the spatial and temporal distribution of water and sedimentation. This distribution of water is often physically disastrous and economically devastating, as for example, during severe droughts and floods. Sedimentation also reduces the economic performance of storage schemes, often leading to serious economic losses and disasters, especially, in relatively poor third world economies shouldered with a heavy debt burden.

The realistic prediction of the behaviour of the interacting processes associated with the temporal distribution of water vis-a-vis sedimentation, coupled with an accurate assessment of their impact on the economics of reservoir design and operations, constitute the problem of this thesis. This problem is further complicated by the effects of environmental changes, resulting mainly from the ongoing and future global climate shifts, landuse changes and local climatic change due to water impoundments. These factors must be adequately provided for if optimal designs and management of storage projects are to be obtained. This research is focused primarily on the assessment and prediction of the impact of these environmental changes on reservoir performance and management policy.

1.2 Scope of this Research

Chapter 1 defines the problem, Chapters 2-4 review the literature, Chapters 5-6 describe the study tool, Chapter 7 provides the database, Chapters 8-11 presents the analysis and discussion of test results, and 12 concludes the study.

The following structure will be utilised during the course of this research: A definition of the problem, the scope, the detailed discussion of the problem, and the objectives and strategy of this work is provided in this Chapter in that order.

2 A review of river basin water management practice with a focus on its failure is undertaken with appropriate examples in Chapter 2.

3 A description of the river basin sediment management problem is presented in Chapter 3 complete with assessment methods and failure examples.

4 The current state of the art in mathematical modelling techniques in relation to river basin and reservoir management, is described in Chapter 4.

5. A description of the proposed research tool, the river basin management system and its water and sediment balance components are provided in Chapter 5.

6. A detailed description of the general structure and composition of the economic simulation model, the final component the system, is provided in Chapter 6. Prior to this, the role of economics and a brief description of economic methods as applied to water resources management are given.

7. The magnitude of the expected future changes in global climate and the consequent impact on the Donga Basin case study are evaluated and presented in Chapter 7 for use in simulation studies.

8. Results of a water balance study for the Donga Basin involving the calibration and the general simulation of diverse scenarios of climate and landuse changes are presented, analysed, and discussed in Chapter 8.

9. A similar study as in (8) involving sediment yield assessment is undertaken and the results are presented, analysed, and discussed in Chapter 9.

10. Chapter 10 presents the analysis and discussion of the results of Gembu reservoir economic simulations for short term changes in landuse practices and climate.

11. Results of economic simulations involving the conditioning of the Donga Basin to long term scenarios of global climate changes are presented, analysed and discussed in Chapter 11. The Economic result presented is the final output of the system.

12. Chapter 12 summaries the research findings, presents the general conclusions and recommendations, and lists the scope for further research.

1.3 Problem Description and Discussion

This study is concerned with four major factors namely; the spatial and temporal distribution of water and its allocation; the climatic influence of impoundments; landuse impact on climate, catchment sediment yields and reservoir sedimentation; and the local impact of global climatic change.

1.3.1 The Spatial and Temporal Distribution of Water and Its Allocation

As World population increases rapidly, the World's water resources, like its land resources, are becoming its most important assets. Water is vital for human consumption and sanitation; for the production of industrial goods; for the production of food and fibre; for transportation; and plays an important role in recreation. Water resources management in general and reservoir management in particular, is concerned with satisfying this growing demand, efficiently, and economically.

By the end of the century, global consumption of fresh water will be two to three times the 1980 levels and at least thirty nations may be in real trouble with water supply (Meyers, 1985). This problem is compounded by the current inefficiency in the use of available water. Agriculture is the largest consumer of water, industry accounts for the fastest growing share, and domestic demand is also soaring. Irrigation, which, accounts for 70% of all available water in use and produces 30% of our food from 15% of our croplands, is only 30% efficient,

even in the U.S.A. The need to adopt efficient irrigation practices, worldwide, such as practiced in Israel and Carlifornia, where farmers make three times better use of scarce water cannot be overstated. Trickle-drip irrigation practices are extensively used in both places. (Meyers, 1985; WRI, 1987).

The world wide distribution of Water Resources is provided in the literature (Mather, 1984) and is illustrated in Chapter 2 (figure 2.1 (a)). The long drought experienced in the Sudan prior to August 1988 and the sudden outbursts of record rainfall and unprecedented floods in that month illustrate the time dependent nature of water resources distribution. The high frequency of floods in Bangladesh presents an example of the space dependent distribution of surplus water.

Droughts

The recent and extended drought sequence in Africa, north of the equator, was interrupted in August (1988), by one of the heaviest rainfall and floods in this century. Some of the most severe flood flows were recorded on the River Nile and in Northern Nigeria. In the Sudan, the 1988 floods were so severe that many settlements were washed away causing widespread human suffering. Reservoir gates had to be opened to release excess water. Large volumes of sediment were deposited in the Rossieres and Sennar reservoirs in the Sudan and in the Aswan reservoir in Egypt. In Kano, Nigeria, the Bagauda dam collapsed in the second week of August 1988, due to inflows from heavy

floods. Figures 1.1, 1.2 and 1.3 show the long term recorded hydrographs for Kainji dam inflow site (Nigeria), located in the same Sahel climatic zone as Kano City in Nigeria and Khartoum in the Sudan. In these illustrations, the drought, normal and flood sequences over the years are clearly visible.

Floods

Bangladesh, a country lying in the delta of the Ganges river system, is very susceptible to floods. Two recent floods, (1972 and 1988), were particularly severe and destructive. In the 1988 floods, three quarters of the country was inundated, one million homes were swept away, ten million other homes were damaged, 280 bridges were washed away or damaged and three million tonnes of rice were destroyed making thirty million people homeless and hungry. Surprisingly, the death figure were in the order of 2000 only. In Dacca, the capital city, almost a million people were displaced and the city was half a knee deep in dirty water. The daily death tolls from water borne tropical diseases, and other ailments such as meningitis and pneumonia were reported to be about 50 (BBCTV, Oct 3rd, 1988).

The Role of Reservoirs

To minimise the damage and suffering caused by floods and to maximise the benefits, thoughtful management is required. The construction of adequate reservoirs for multipurpose uses like irrigation, water supply

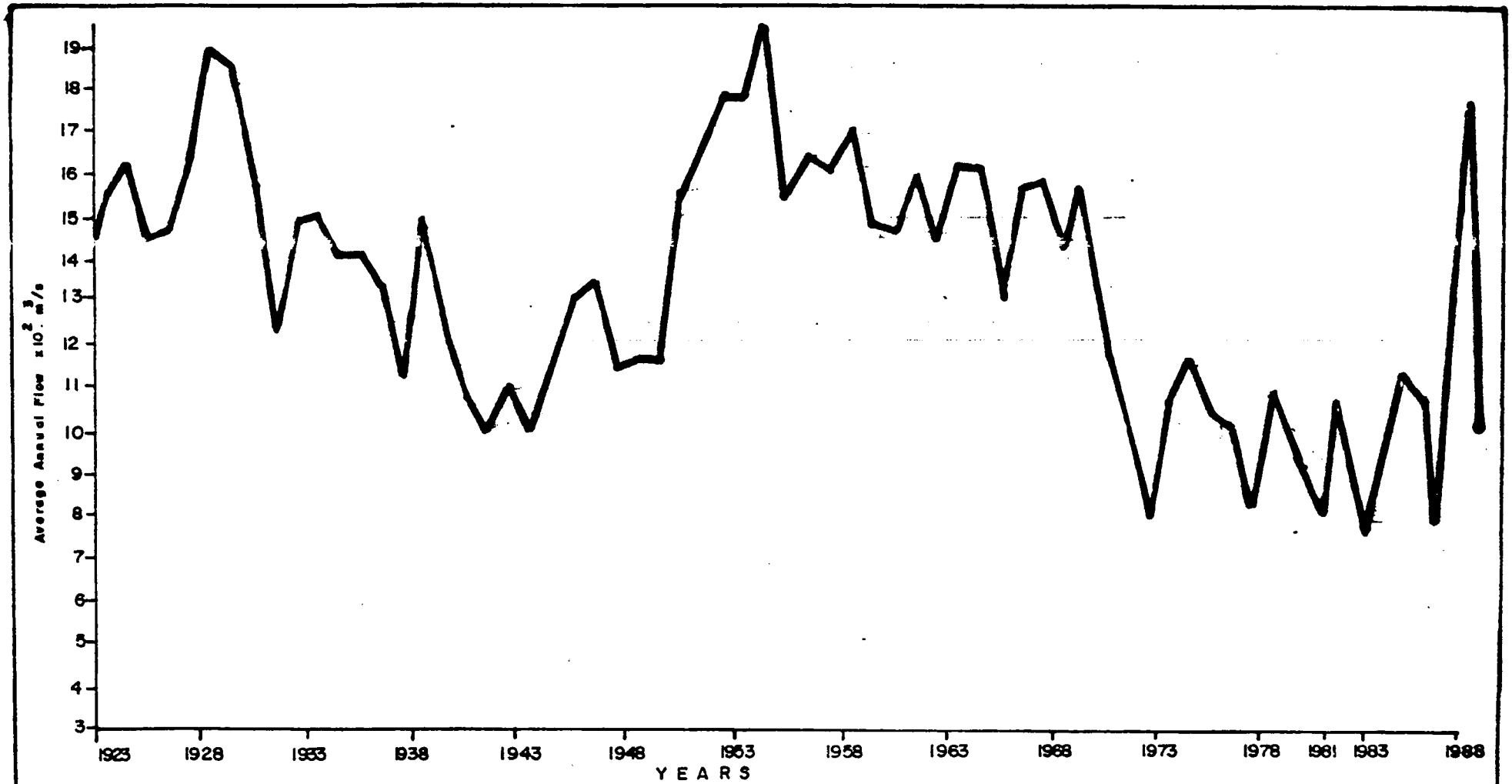


FIG. 1.1 LONG TERM HYDROGRAPH FOR KAINJI RESERVOIR, NIGERIA

This typical hydrograph shows the temporal distribution of water in the Sahel Regions of Africa



FIG 1.2

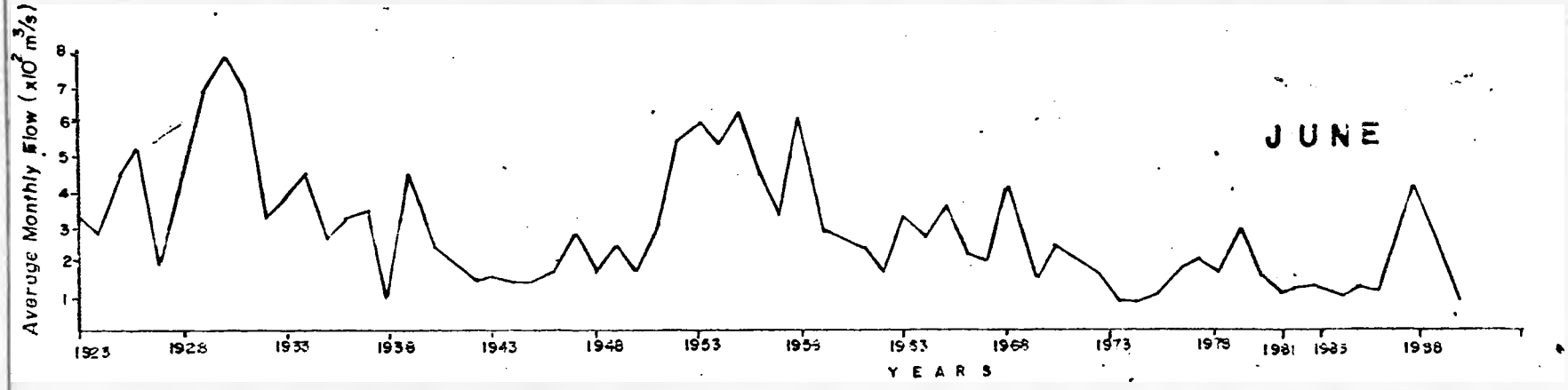


FIG 1.3 KAINJI TOWN LONG TERM HYDROGRAPHS

hydropower, drought amelioration, flood control and navigation is a major aspect of this management. Tables 1.1 and 1.2 show by region the volume and area of water stored in reservoirs around the world. To maximise the use of the stored water, optimal design and operating concepts are the 'key', especially, for multipurpose uses.

In the northern half of Africa, for example, large, high capital intensive reservoirs have been constructed to meet long term development needs. The recent extended Sahel drought cycle, has revealed the long term inadequacy but short term adequacy of such reservoirs as Aswan (Egypt), Akosombo (Ghana) and Kainji (Nigeria), among others, to meet national power and irrigation requirements. As the economies of these three countries carry heavy debt burdens, these shortfalls in reservoir yields have significantly disrupted development efforts. This is worsened by the fact that these countries respectively rely mainly on each lake for virtually all water needs. Figure 1.4 illustrates the areal extent of the Sahel drought. The problems associated with these Sahelian reservoirs include the extensive loss of water to evaporation, and inadequate long term water storage for drought mitigation and loss of storage capacity due to sedimentation.

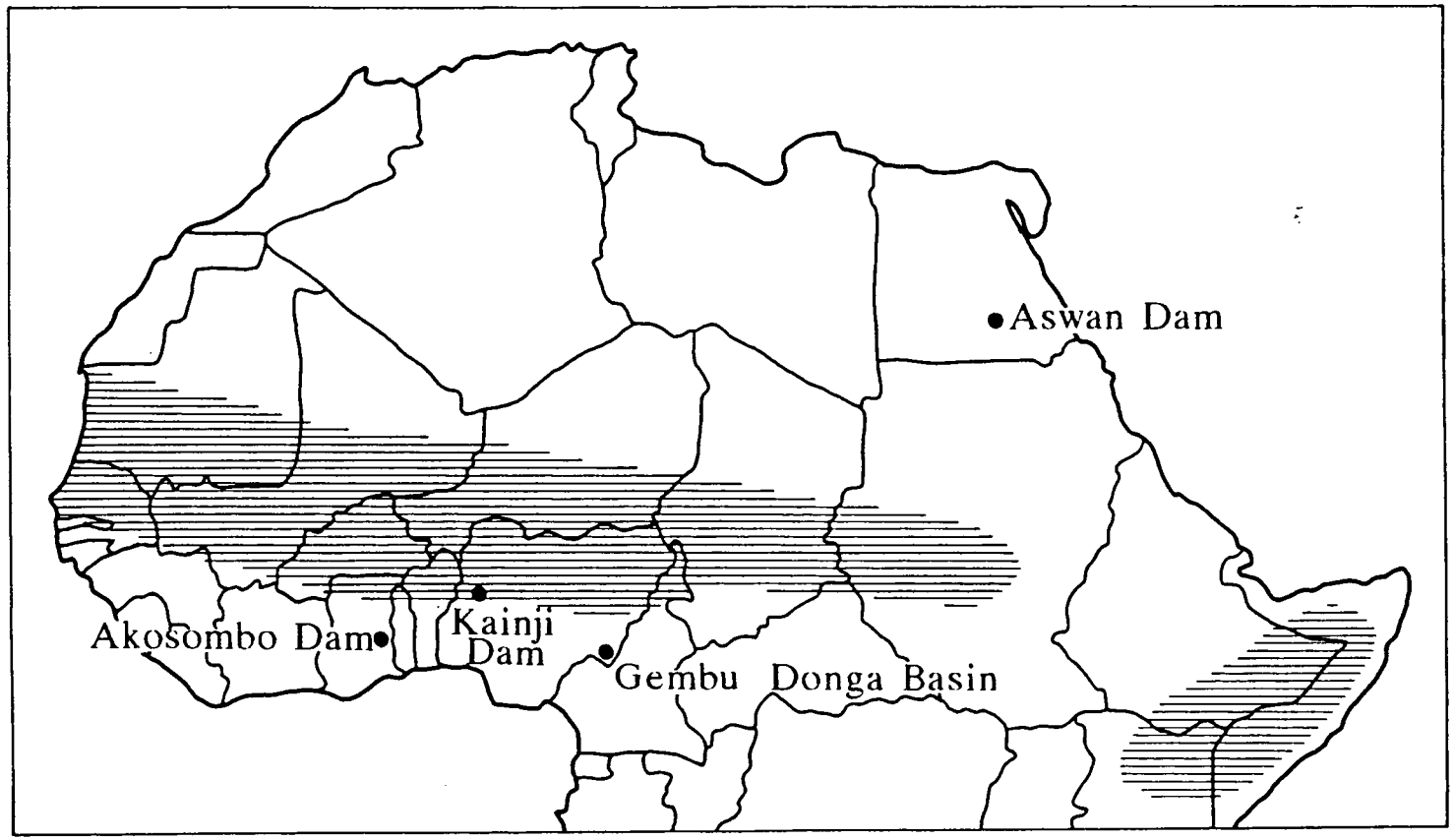


Fig. 1.4 THE SAHEL REGION OF AFRICA
Showing the areal extent of the drought

Source: Mather,1984

Table 1.1: **Reservoir areas by region**

Region	Reservoir Area (million) sq metres		
Period	Before 1961	1961-80	After 1981
World	130,784	177,639	32,753
Africa	7,316	16,990	3,271
Central America	16,836	11,091	1,287
South American	4,719	34,510	8,247
Asia	3,819	11,909	10,011
Europe	11,892	8,932	1,990
USSR	84,051	90,082	6,654
Oceania	2,150	4,124	1,294
USA	13,970	8,435	604

This table is extracted from World Resources, 1987 (IIED) Large reservoir areas lead to huge water losses by evaporation which is limiting long term mitigation of drought - often with devastating results.

Table 1.2: **Reservoir Gross Capacities and Runoff Volumes by Regions**

Geographical Area	Annual Runoff volume-cu. km		Gross Reservoir Capacity - cu. km		
	Total	Natural Base Flow	Volume	% of world	% of runoff
North America	5950	1900	975	20.0	16.4
Asia	13190	3440	1770	36.3	13.4
Africa	4225	1550	1280	26.2	30.3
South America	10380	3740	340	7.0	3.3
Europe	3100	1125	450	9.2	14.5
Australia	1965	465	65	1.3	3.3
World	38810	12170	4880	100.0	12.6

Table is extracted from Mahmood (1986)

1.3.2 Impoundment Induced Climate Changes

Changes in the climatological parameters occur following the construction of a large or a series of large reservoirs in a River Basin. This is especially true in hot climates. The change can be triggered mainly by the excessive lake evaporation caused by the large surface area of water and subsequent temperature effects. Large natural lakes, such as the great lakes of North America, are known to have a strong climatic influence on adjacent regions; a warming effect in winter and a cooling effect in summer. This is because water bodies require longer time to warm and retain heat much longer than land surfaces (Nace, 1974).

Mechanism of Climatic Change

Heat stored in lakes influences atmospheric temperatures, in their vicinities. In large water reservoirs with undirectional prevailing winds, both annual and daily temperature ranges are lowest on the "lee zones". In summer, the stored water has a relatively cold surface, and this has a stabilising effect on passing air masses, which in turn, reduces the amount of precipitation on the leeward shores. In winter, the airmass moving over the relatively warm lake surface picks up additional moisture charge and at the same time becomes more unstable from heating below. Consequently, the amount and frequency of precipitation, fog, and cloudiness are greater on the leeward areas (Nace, 1974).

In hot climates where temperature differences between seasons are small, the diurnal influence of impoundments are likely to be more profound than the seasonal. Thus, the usual winter and summer temperature conditions may be likened to that which obtains during night and day times respectively.

Magnitude of Climatic Change

Mather (1984) stated that "the creation of large reservoirs or the drainage of swamps modifies local evaporation and even to a small degree, local precipitation conditions". The significance of the effect of reservoirs on local precipitation depends mainly on their surface areas. The larger the reservoir area the larger the effect. Mather (1984) commenting on McDonald's work (1962) in Arizona reported that theorised increases in precipitation of up to 10% may result from the construction of a lake of about 50,000 sq km, that is 16% of the total area of Arizona! This is larger than the areas of lakes Erie and Ontario (North America) combined. This means that the impact of surface water areas on climate change is micro and local rather than macro and global, irrespective of the lake sizes. This work by McDonald (1962) provides a quantitative basis for estimating impoundment induced climatic effect.

Orographic Influence

The presence of smaller reservoirs in regions with high mountains or the location of such lakes on or near high plateaus, tends to increase precipitation appreciably through orographic cooling. Mather (1984) indicated that though precipitation is low around the Caspian sea in the U.S.S.R., there is appreciable precipitation just south of the lake where high mountains exist. This possibly due to orographic uplift cooling of the moisture laden air masses.

Assessing Local Climate Change

In planning a reservoir, a study of the long term meteorology of other catchments with existing lakes subject to similar conditions to the one envisaged may be helpful in deducing and estimating the likely effect of the impoundment on the local climate. Interestingly, an increase of catchment precipitation may be expected under the right wind conditions. In addition, catchment evaporation is expected to increase by a magnitude depending on prevailing wind conditions. The two expected conditions lie between two extremes, namely,

(a) no increase in rainfall coupled with a substantial to extreme increase in evaporation.

(b) maximum rainfall increase which may be considered equal in water content to the total reservoir evaporation.

Any of these two conditions or a mean can be used for analysis as appropriate.

1.3.3 Landuse Impact on Local Climate and Sedimentation

Land surface conditions are constantly changing with time from both human intervention and natural causes. This is aggravated by increases in population, agriculture, deforestation, irrigation, industrialisation amongst others. Man-induced changes are triggered by changing wealth and habits coupled with the need for more efficient harnessing of resources to meet prevailing and future economic needs and realities. It is necessitated by the need for an increase in agricultural and industrial export commodities and domestic food supply. The former helps nations in earning badly needed foreign exchange.

The Role of Human Landuse Cultural Practices

Mather (1984) reported that recent studies indicate that local cultural practices such as urbanisation, land use, and certain aspects of industrialisation can indeed modify local climates significantly. This depends on the geographical scales and the order of magnitudes of the practices concerned. There is no dispute that the construction of large urban complexes significantly alters aspects of local heat and water budgets. The construction of cooling towers to remove heat from industrial effluents and power stations tends to increase evaporation, the opportunities for fog, mist, or icing conditions and may also cause additional cloudiness or convective showers. The scale of their influence is usually limited to a few hundred square kilometres.

Power (1988) and Mahmood (1986) have indicated that human intervention such as deforestation and open cast mining have led to excessive erosion and siltation. Diyam and Binies (1985) reported that an increase in cattle population on the Mambilla Plateau (Donga Basin) in Nigeria by two and a half fold in twenty years has led to a fivefold increase in erosion rates and sediment yields (Hudson, 1982; MRT, 1987)

The Role of Reservoirs

Further, increases in land use activities are expected as a result of the planning and construction of new reservoirs, due to the presence of plentiful supply of water which stimulates agriculture and other activities. These changes in activities may well increase soil erosion rates, channel sediment transport rates and reservoir sedimentation.

Thus, beneficial changes in land use may have detrimental effects on existing water supply, irrigation and hydroelectric power storage schemes. These changes should be taken into consideration in reservoir and river basin planning, otherwise projects are unlikely to meet their objectives in the long term. Consequently, the engineer may be required to assess the changing trend in land use activities. With the trend established, mathematical models may be used to assess the effects of these changes on catchment runoff and sediment yields, parameters required for water resources design.

1.3.4 Reservoir Sedimentation Patterns: Dead Storage Levels

Sedimentation in reservoirs can be severe, especially in regions where inflow streams carry large sediment loads. This poses serious problems, particularly when such reservoirs are relatively small in capacity and hence have very small design lives. Power (1988) reported that World Bank specialists estimated the replacement costs of silt-eroded reservoirs on a Worldwide basis to be some $6000 million (£3,300 million). This is certainly a colossal waste.

Excessive Dead Storage Capacities

Current engineering practice, generally, specifies a dead storage by a minimum operating level. The dead storages are usually very large and are supposedly capable of accommodating a 100-200 year or more of sediment deposits, far longer than the actual capitalization time of such schemes. It is normally assumed, in the long term that the dead storage zone, normally nearest the dam, would accommodate all the sediment entering the lake. This is not always so. A delta is normally formed by the deposits at the inflow point, reducing the useful live storage capacity of the reservoir (figure 1.5).

Live Storage Capacity Losses

Mahmood (1986) stated that "the life of a reservoir is commonly, but erroneously, estimated as the volume of dead storage divided by the expected mean annual volume of sediment deposits". He contests this by stating that

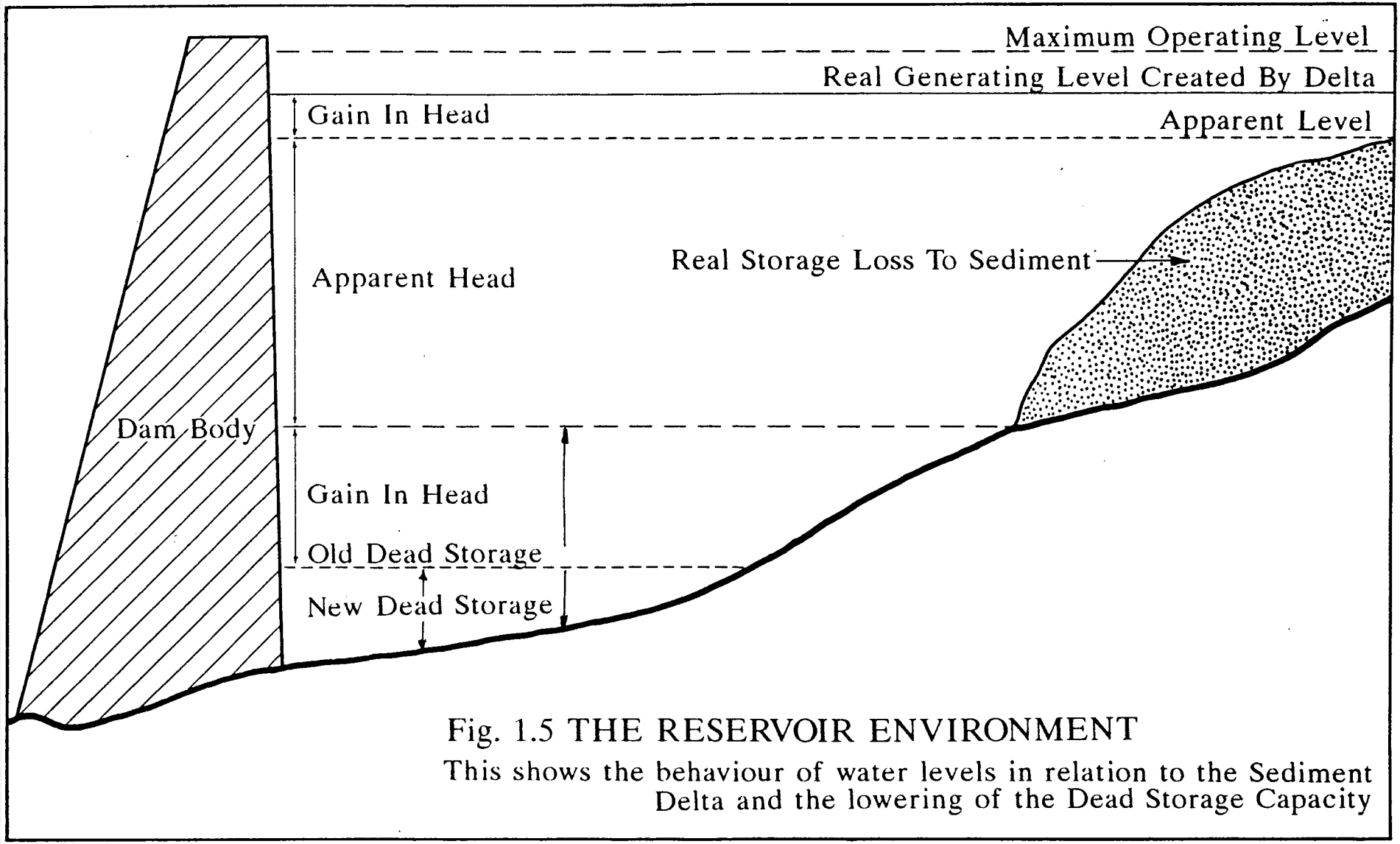


Fig. 1.5 THE RESERVOIR ENVIRONMENT

This shows the behaviour of water levels in relation to the Sediment Delta and the lowering of the Dead Storage Capacity

"such extrapolations are not valid" and that "reservoir sedimentation patterns are such that the usable capacity starts diminishing (long) before all of the non-usable component is filled up with sediments. This is illustrated in Figures 1.5 and 1.6. Figure 1.5 illustrates the response of a typical reservoir to sedimentation and the effect of lowering the minimum operating levels. Figure 1.7 shows clearly the siltation problem of the lake Mead (the Hoover Dam Reservoir) on the Colorado river in the United States. Sedimentation has been severe in the lake, especially during the first few years after construction. In general, the live storage in the lake is being lost much faster than the dead storage.

Delta Formation Studies

The mechanism of delta formation in reservoirs is still under study. The lack of adequate knowledge and understanding of these processes, is responsible for the over generalised dead storage specification policy (contested by Mahmood (1986)) associated with reservoir design and management in the past. Few reservoir sedimentation models suitable for practical design applications exist, and where they do, modifications are required to adapt them to solve specific design and management problems (Anandale, 1987).

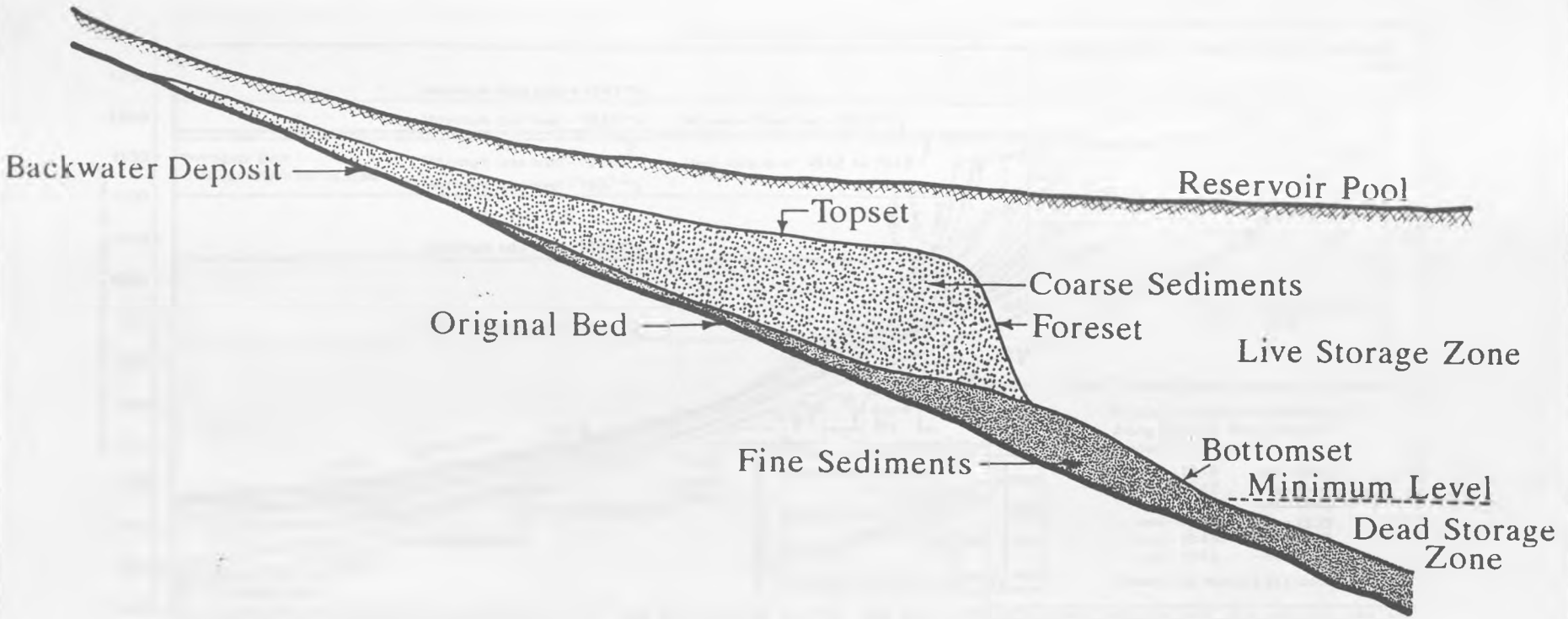


Fig. 1.6 PROFILE OF TYPICAL RESERVOIR DELTA AND NOMENCLATURE FOR DEPOSITS

This clearly shows that the live storage zone reduces much faster than the dead storage zone. The traditional approach of specifying excessive large dead storage to cater for sedimentation is unnecessary

Source: Mahmood,1987

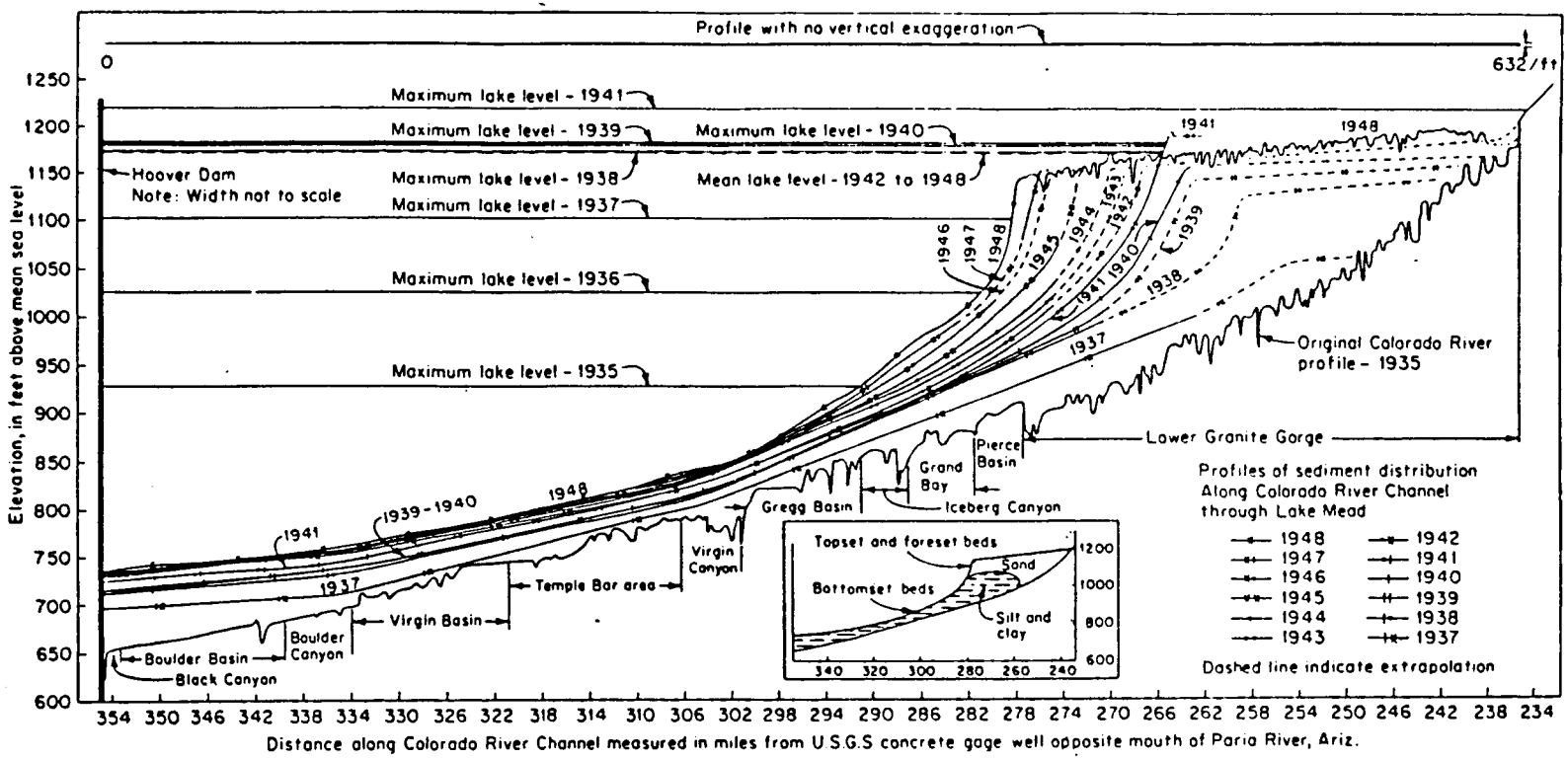


Fig. 1.7 DEPOSITION PATTERN ALONG THE COLORADO RIVER THROUGH LAKE MEAD

This is a real life reservoir situation illustrating the live storage reduction process.

Source: Smith et al,1954

Lowering Minimum Operating Levels

Excessive dead storage specification is an economic waste during critical periods such as droughts and floods. Minimizing the dead storage will be beneficial in such circumstances. For example, if a reservoir is lowered below its normal specified minimum operating level in contemplation of a large flood, then its capacity to absorb and attenuate the flood peaks are much increased. During drought, a lowered allowable operating level allows crucially needed water to be available for irrigation, water supply or power generation, as the case may be. As reported by Acres (1985) and Merz and McMillan (1986) the Akosombo reservoir in Ghana, West Africa, was overdrawn below specified minimum operating level by 2 metres during the Sahelian drought sequences. It was found that the energy yields improved by some twenty percent above that obtained by operating at the specified minimum level. Figure 1.5 shows the sediment depositional pattern in a reservoir and the effects of lowering the minimum operating levels.

Dead Storage Capacity Optimisation

In optimising dead storage spaces, reservoir managers must take into account:

(1) The question of debris. This consideration is confined to reservoirs that have not been properly cleaned, in which case, the dangers from rapid chemical and physical deterioration and destruction of equipment and the dam body must be expected during operation.

(2) The problem of cavitation (air entrainment) of hydropower turbines, as this effect tends to occur when minimum operating levels are too low.
(3) The stability of the dam structure
(4) The stability of the reservoir slopes

Computer simulation, mathematical modelling, empirical and correlation techniques have expanded the knowledge of sedimentation process in reservoirs (Annandale, 1987).

1.3.5 Global Climate Changes

It has been reported that the earth's climate is undergoing severe changes (UNEP/WMO/ICSU/DRCCPIW, Villach 1985; Ballagio, 1987; UNEP et al, 1988, 1989, 1990). At a given time, it is likely that at least one region of the earth is suffering from the disasters of a major climatic extreme. These are manifested mainly in the form of droughts and floods and appear to be increasing in intensity, duration, and frequency in many regions as the global climate changes. Noticable changes in local climates have been observed around the World. For example, in 1982-83, Meyers (1985) reported that extreme weather conditions were experienced around the World, especially in the less developed countries of the south, with widespread disruptions to agriculture.

It was estimated that economic recovery from the disasters would take at least five years. The damage was estimated at eight billion dollars. A possible cause of the 1982-83

weather disruptions was the El Nino phenomenom (Meyers, 1985), where a warm current was associated with a low pressure system originating from the Central Pacific Ocean. This phenomenon occurs once in every decade. The changing trend in the world climate however is caused mainly by global air/ocean heating due to air pollution, i.e the green-house effect (UNEP/WMO/ICSU/DPRCCPIW, Ballagio, 1987).

A Few Salient Examples

(1) The Sahel drought of the late sixties and seventies brought disaster to many African nations. A similar drought has also been experienced intermittently in the eighties. Widespread famine has been the effect.

(2) In 1972, a similar drought inflicted such damages on the Soviet wheat crop that the world market prices increased by 400% within two years.

(3) In 1974. a delayed monsoon in India wrought havoc for millions of people.

(4) In 1975, pulses of cold air ravaged Brazil's coffee crop, causing inflation of coffee prices around the World (Meyers, 1985).

The Greenhouse Effect

The greenhouse heating effect is reported to be mainly responsible for the Worldwide changes in climate, experienced in recent times. The gradual loss of the

ozone shield in the upper atmosphere and other factors are partly responsible but on a lesser degree. The greenhouse effect is caused by the accumulation of carbon dioxide, oxides of nitrogen, CFCs, etc which act as a heat shield in the lower atmosphere, leading to a warming of the lower atmospheric strata, the oceans and the earth's surface. There is also a general warming of the earth due to industrial activities. With the increase in industrial activities expected in the future, the greenhouse effect should accelerate even further climatic extremes, leading to consequent disasters.

These effects complicate water resources planning by introducing an added element of uncertainty in the use of past data. The policy of complete reliance on past hydrological data requires review. It can no longer be effective in the design and implementation of long term optimal water resources schemes. This view has evolved from worldwide observations of climatic parameters in the last two centuries (Holy, 1986) and the recommendations of the UNEP/WMO/ICSU conference on Climatic changes held at Villach, Austria in 1985 which stated that.....

"many economic and social decisions are being made today on long term projects.. such as irrigation and hydropower; drought relief; agricultural land use; structural designs and coastal engineering projects and energy planning - all based on the assumption that past climatic data, are a reliable guide to the future. This is no longer a good assumption since the increasing

concentration of GHGs are expected to cause a significant warming of the global climate in the next century. ...."

The UNEP/WMO/ICSU workshop at Bellagio, Italy in 1987 affirmed one of the above Villach recommendations thus.....

"It was recommended that governments and regional and intergovernmental organisations should take the results of the above assessment into account in their policies for social and economic development, environmental programmes and policies for the control of GHG emissions"

The Role of Mathematical Models

The use of process based mathematical models may be an answer to the data review problem in Water Resources management. The changing trend in local climate, in accordance with the global change, can be assessed by a careful study of past and current hydrological information. Mathematical models, taking cognisance of the changing trend, may be used in generating new time series data required for traditional hydrological analysis, design, and operation. An assessment of the magnitude and a discussion of the causes, impact, and implications of these climatic shifts are discussed in Chapter 7.

1.3.6 Summarising the Problem.

The reservoir problem highlighted in this section requires a design and management philosophy that will optimise performance in the long term to meet the challenges posed by the combined impact of:

(1) The hydrological and landuse conditions present in the basin prior to the construction of reservoirs. This is the base scenario which is continuous in both short and long time periods.

(2) The new hydrological and landuse conditions resulting from impoundments in or close to the catchment. This is a steady state scenario which has both short and long term effects.

(3) The effect of global climatic changes. This is an unsteady state scenario required in long term considerations.

(4) The interacting microprocesses in the river basin and reservoir environment.

The Figure 1.8 illustrates this summary of the reservoir management problem.

1.4 Research Philosophy and Strategy

The new strategy,

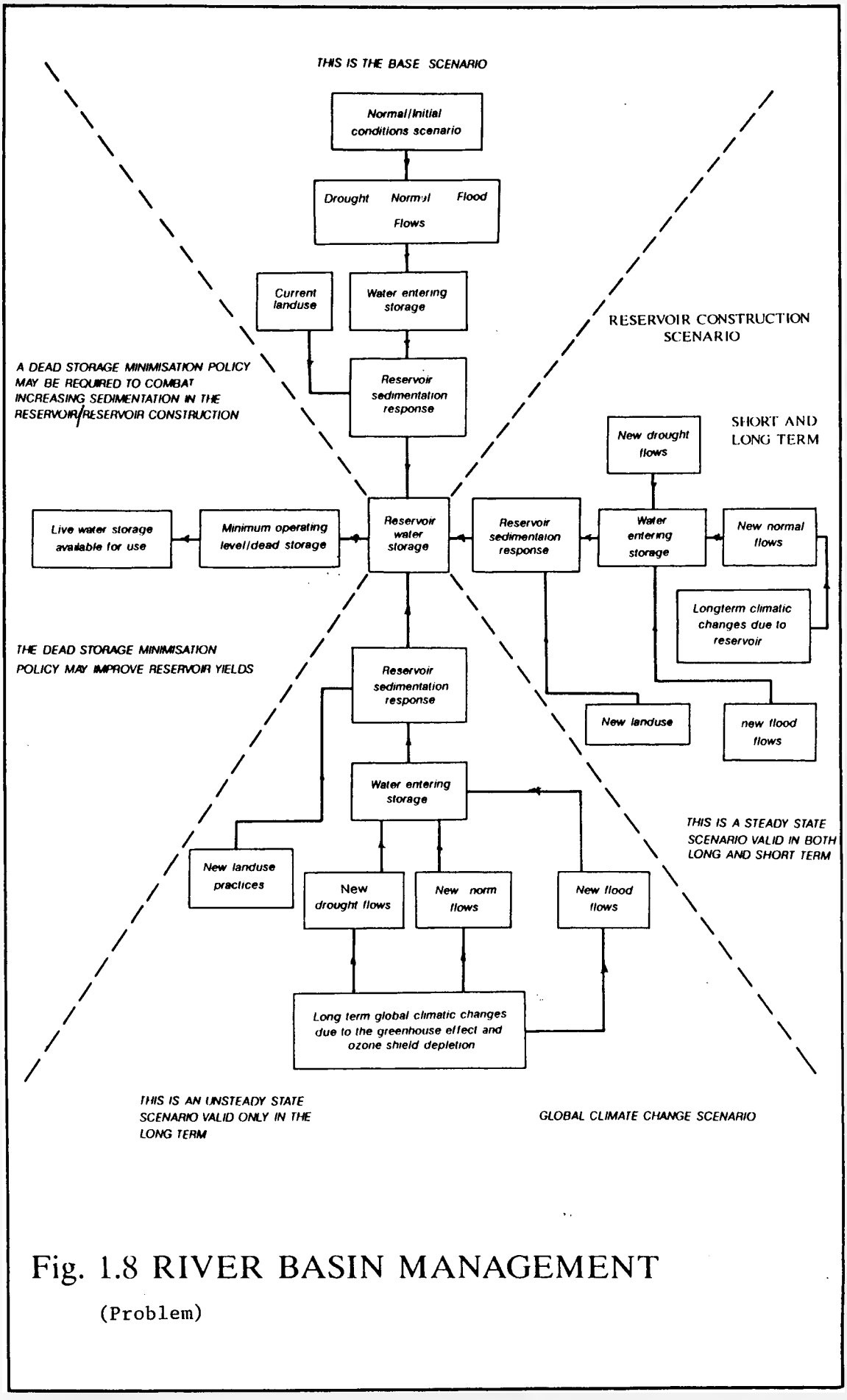


Fig. 1.8 RIVER BASIN MANAGEMENT

(Problem)

(a) accommodates future changes in dynamic catchment processes and conditions to assure an acceptable economic performance of water resource projects. These changes are mainly those experienced in climate and land use as discussed previously.

(b) involves the optimal allocation of the water stored, in new and existing reservoirs (or their appropriate combinations), for the maximisation of power, irrigation, water supply and other benefits.

(c) should involve establishing or confirming the optimal locations and sizes of dams and reservoirs in a river basin, through socio-economic analysis.

This proposal is dynamic and process oriented and therefore requires a process based analysis methodology for the realisation of accurate or realistic results for the optimal design of water schemes.

Comparing Current with Proposed Strategies

Until now, design and planning strategies and policies have evolved by the assumption of a fixed rather than dynamic conditions within a river basin. The latter, however, is the actual real-life and real-time situation.

The old strategy utilises a fixed database for estimating design parameters, and generalises the reservoir sedimentation patterns, leaving out the details of real-

time and real-life conditions. The new philosophy introduces and utilizes the effect of steady and unsteady changes in the catchment and its attendant sediment processes to optimise the design of the proposed storage schemes. This is clearly illustrated in Figure 1.9 (a).

The comparative effects of these two different possible strategies of development requires investigation so as to justify the need for the use of the new case as an alternative. This comparison should highlight the merits of one concept over the other. Lastly, it is important to assess possible improvement in socio-economic returns and to confirm that the extra effort involved in the use of the proposed strategy is justified.

Methodology

The methodology proposed for testing and implementing the new strategy involves the establishment of a reservoir and river basin management optimisation system, requiring the interlinking of several mathematical models, existing and new, with computer simulation as the main tool of study (Figure 1.9 (b)). This system will be used to assess the effect of dynamic scenarios on the economic efficiency and viability of storage projects.

There is a need, therefore, to apply, modify, calibrate, and interact the computer models and modelling techniques, required for the construction of the proposed system, to suit the conditions and problems involved.

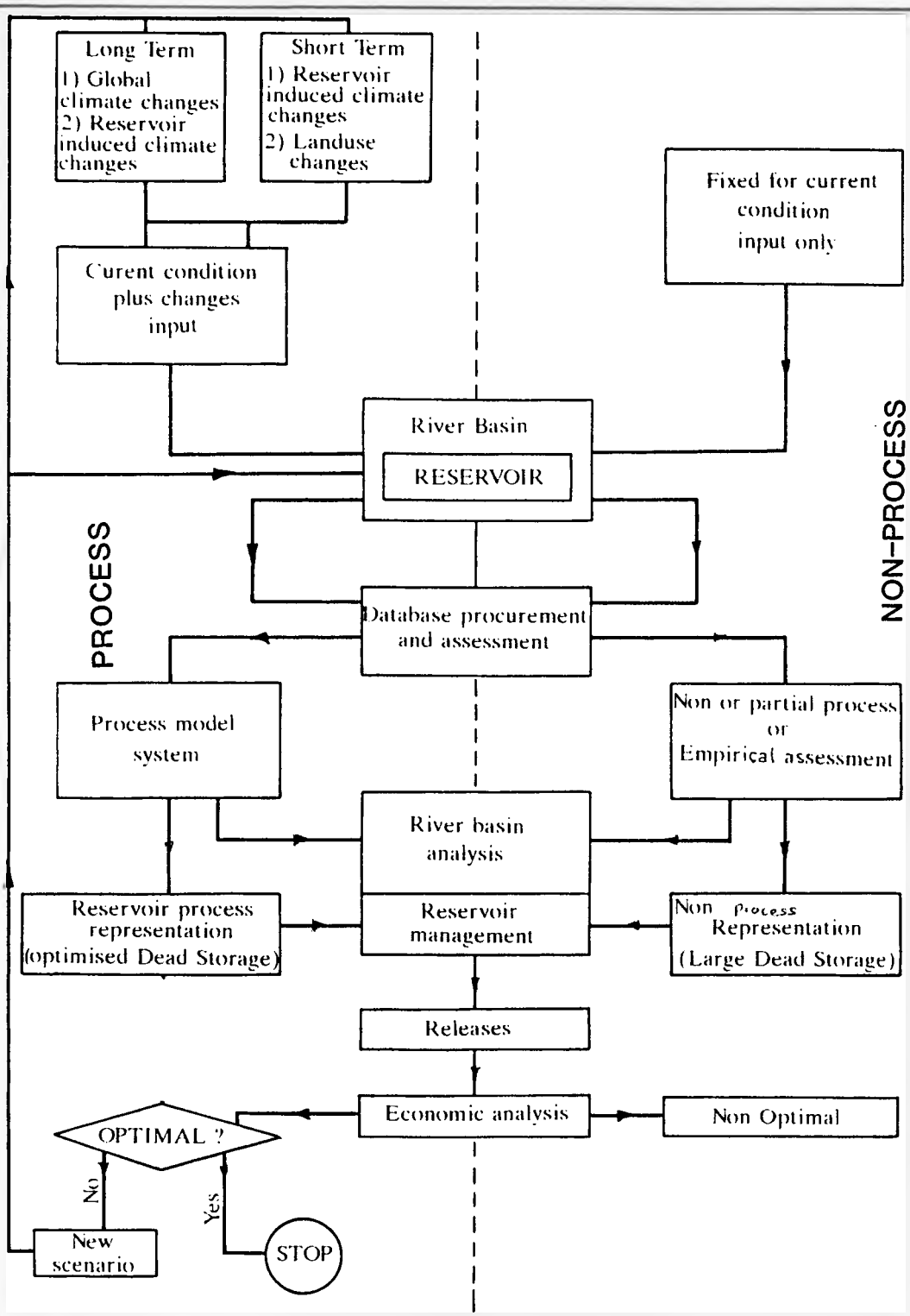


FIG 1.9 a DEFINING THE PROCESS APROACH TO RESERVOIR MANAGEMENT AND COMPARISON WITH NON-PROCESS APPROACH

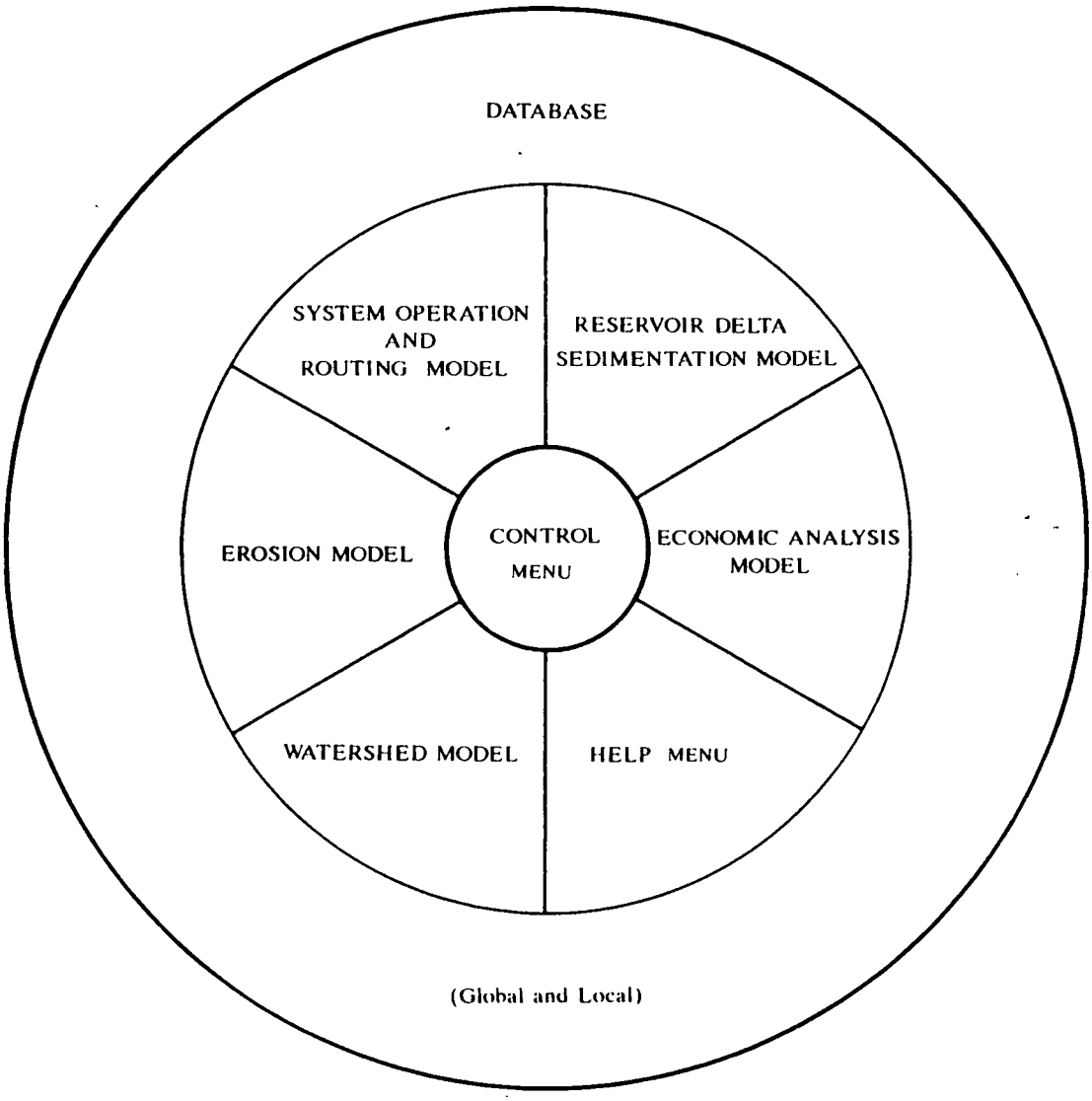


Fig.1.9bRIVER BASIN MANAGEMENT SYSTEM

This system is available for use on both micro and mainframe(VAX) computers.

Specifically, emphasis is placed on the assessment of sedimentation and evaporation in reservoirs and the significance of their effects. It must be noted, also that the optimisation of reservoir operation and design must involve the principles of cost benefit economic analysis. In this light, it may be interesting to include in the analysis such factors as reservoir placement costs, particularly regarding resettlement of population, relocation of infrastructure, and the impoundment of fertile land. This is basically a socio-economic aspect of water resources planning and is considered vital.

Finally, it is hoped that as investigations continue, findings which may be localised within the Donga Basin case study may be extended to the generality of water resources management practice elsewhere.

1.5 **Case Study: Specific Considerations**

A proposal, involving the construction of as many as five reservoirs (Figure 1, 10) as part of a major hydroelectric and irrigation scheme is contemplated for the Donga Basin in Nigeria. This will result in major changes in river basin response in the region of mideastern Nigeria and midwestern Cameroons. This presents an ideal case study for testing the approach proposed in this work.

One of the objectives of the Donga Basin study, lies in establishing the optimal basin development strategy and this involves assessing,

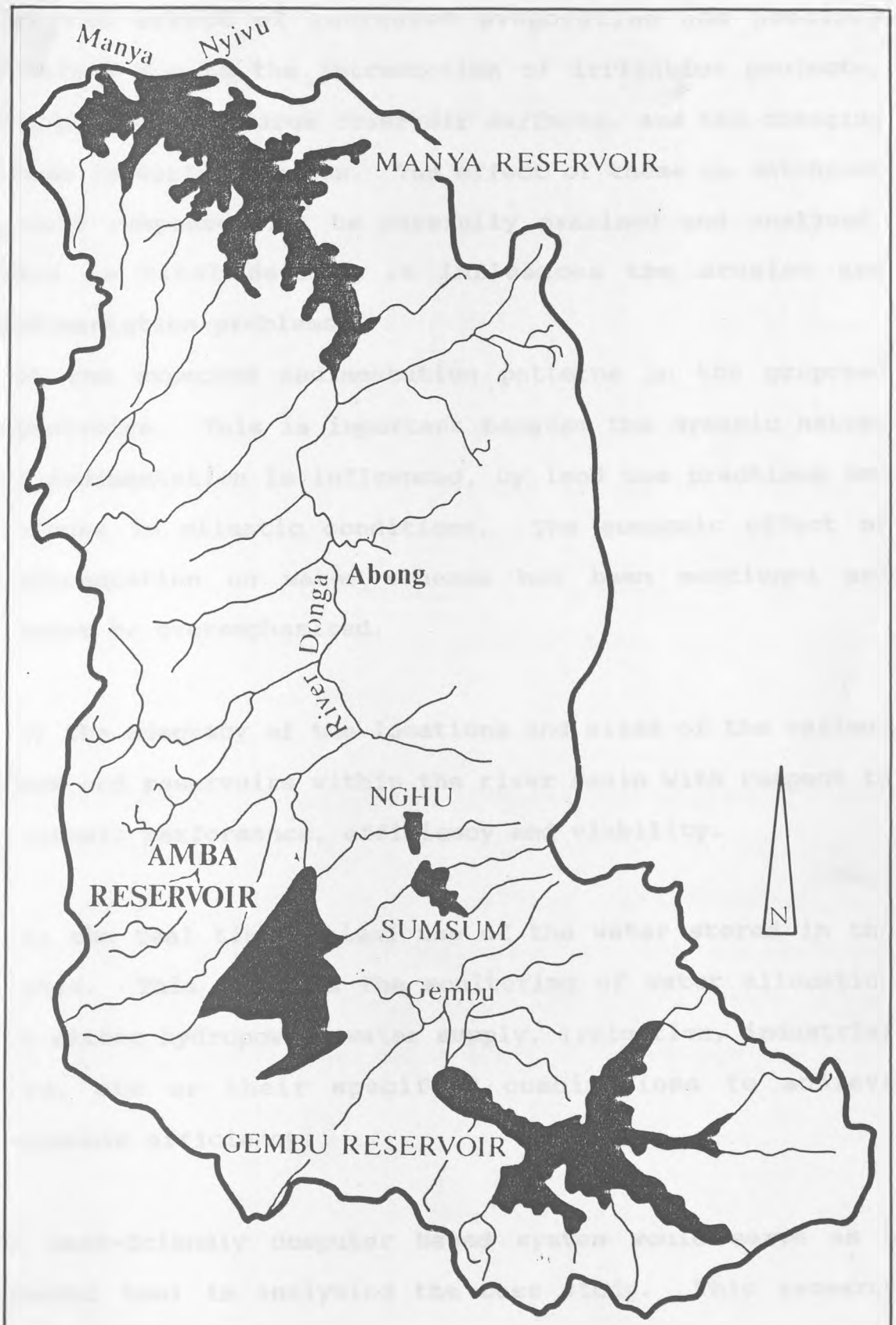


Fig. 1.10 DONGA RIVER BASIN
PROPOSED RESERVOIR SITES

(a) the effect of increased evaporation and possibly rainfall due to the introduction of irrigation projects, the presence of large reservoir surfaces, and the changing trend in world climates. The effect of these on catchment runoff response will be carefully examined and analysed. This is vital because it influences the erosion and sedimentation problems.

(b) the expected sedimentation patterns in the proposed reservoirs. This is important because the dynamic nature of sedimentation is influenced, by land use practices and changes in climatic conditions. The economic effect of sedimentation on water schemes has been mentioned and cannot be overemphasized.

(c) the adequacy of the locations and sizes of the various dams and reservoirs within the river basin with respect to economic performance, efficiency and viability.

(d) the real time optimal use of the water stored in the lakes. This includes the monitoring of water allocation to either hydropower, water supply, irrigation, industrial use, etc or their specified combinations to achieve economic efficiency.

A user-friendly computer based system would serve as a useful tool in analysing the case study. This research proposes to build such a system.

1.6 Final Remarks.

The proposed approach to develop a management system could provide a better understanding of the hydrological and hydraulic processes in river basins and their economic implications. To achieve this, the Donga basin is used as a case study to establish the water balance and sediment yield of the basin. The response to appropriate changes in climate and landuse activities is analysed and studied heuristically to assess the phenomenon of cause and effect. The impact of such changes on reservoir design, operation and economics will be better understood and quantified. The water balance response can be used to assess the long term availability of water in the river basin for future management.

Now that the problem aspects of reservoir management has been defined, the state of the art in river basin management is initially reviewed in chapter 2. Thereafter, attention is directed to the management problems involving flood and drought events with suggestions regarding possible areas of improvement.

CHAPTER 2

CHAPTER 2: RIVER BASIN WATER MANAGEMENT: PRACTICE AND FAILURES

In this chapter, the art of river basin management is considered with the focus on the reservoir management problem. Reservoir management is a vital part of river basin management hence any discussion of the latter automatically includes the former. The role of planning in water resources management is discussed in section 2.2.

A brief review of the practice of reservoir management is provided in section 2.3 and the numerous uncertainties encountered are outlined in section 2.4. Current attempts at managing extremes and uncertainties in hydrology are described briefly in section 2.5. In section 2.6, the failures of these management policies are illustrated by examples of Aswan reservoir (Egypt) on the Nile basin; the Akosombo reservoir (Ghana) on the Volta basin: and the Kainji reservoir (Nigeria) on the Niger basin.

In section 2.7, the role of data management in river basin management is outlined. A brief comparison in section 2.8 of the present and the proposed approach of this research concludes the chapter.

2.1 The Management of Water Resources

2.1.1 General

Water resources management basically involves the efficient provision of water and its optimal use. Mather (1984) described it as

" having the right amount of water available for a particular use at the right time and with the right quality. Having too much, too little, or the wrong quality of water is poor management...."

Table 2.1 provides information on the availability and use of freshwater by selected countries in Africa. (See fig. 2.1a)

TABLE 2.1 Freshwater Availability and Use, 1959-1985 (Annual)

This table is evaluated from a larger one given by World resources (1987).

COUNTRY	AVAILABILITY		WITHDRAWAL		
	total (cu km)	per capita (cu m*1000)	total (cu km)	per capita (cu m*1000)	% Avail ble
Algeria	16.9	0.73	3.38	0.18	80.7
Botswana	18.0	0.84	0.09	0.10	97.0
Cape Verde	0.2	0.59	0.04	0.15	80.0
Djibouti	0.3	0.77	0.01	0.03	77.0
Egypt	57.5	1.20	45.00	1.18	22.0
Gambia	22.0	31.50	0.02	0.03	99.9
Ghana	53.0	3.65	0.30	0.03	93.1
Libya	0.6	0.15	1.47	0.54	52.0
Madagascar	40.0	3.77	16.30	1.68	59.3
Mauritania	7.4	3.75	0.73	0.47	90.0
Mauritius	2.2	2.02	0.36	0.41	82.0
Morocco	30.0	1.30	10.50	0.50	65.0
S. Africa	50.0	1.47	9.20	0.40	84.0
Sudan	30.0	1.31	18.60	1.09	38.0
Togo	11.5	3.66	0.05	0.02	99.9
Tunisia	3.5	0.47	1.32	0.22	33.7

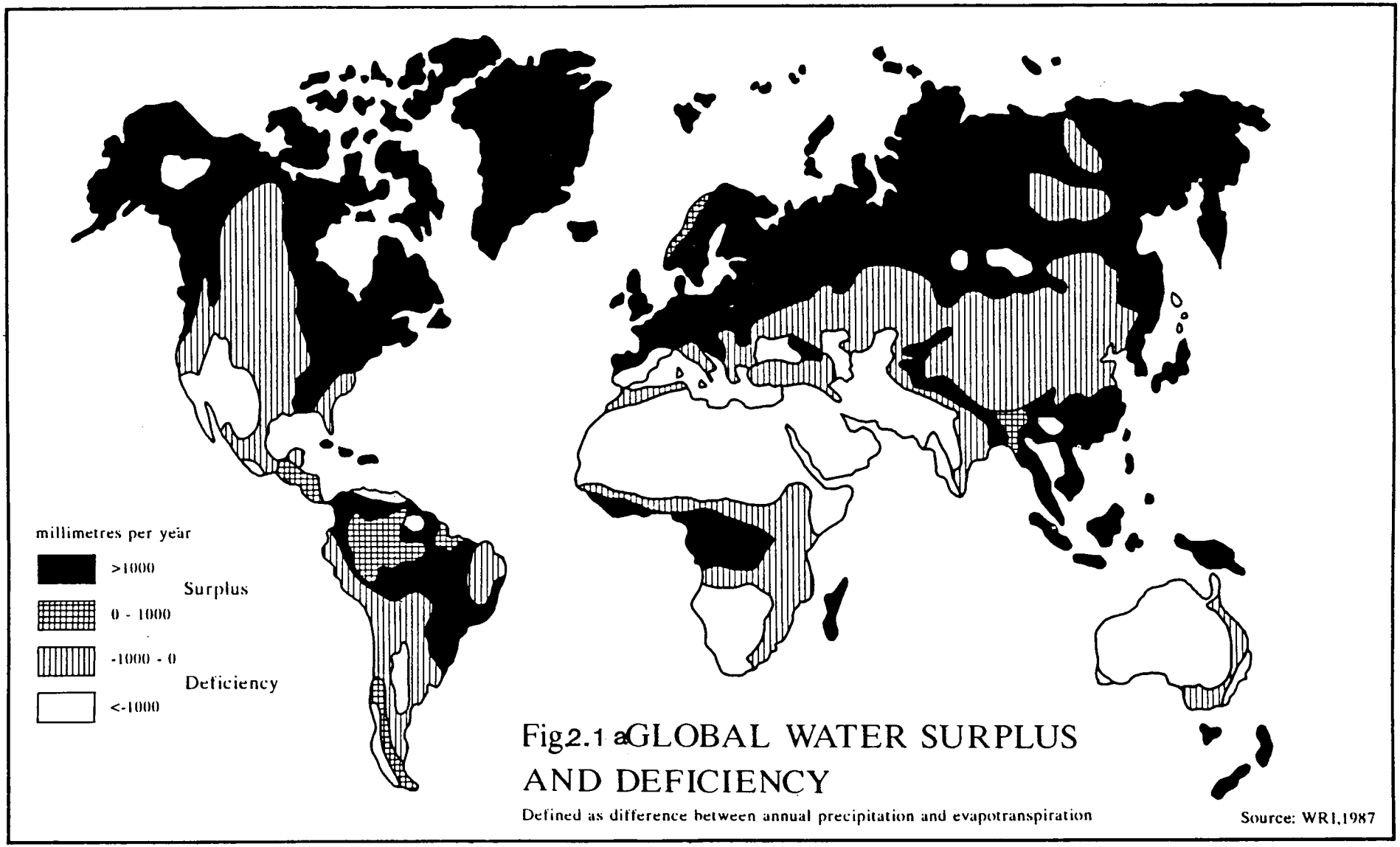


Fig2.1 aGLOBAL WATER SURPLUS AND DEFICIENCY

Defined as difference between annual precipitation and evapotranspiration

Source: WRI,1987

From the table 2.1, it is clear that the problem of water resources is not that of insufficiency but of management. The management of water in its basic form has been practiced by man since early times when water was stored in containers in caves for use at a later time. In recent years, however, water management has become imperative and sophisticated due to the increasing demand for the resource. The several important approaches now commonly used in water and river basin management (singly or in various combinations) include multipurpose, multidisciplinary, integrated, inter-basin or multi-basin and systems.

2.1.2 Multipurpose Development.

While some basins are developed for single purposes, such as flood control or power generation, more basins are being developed nowadays for multipurpose uses. The multipurpose development of water resources maximises benefits in the context of the spatial and temporal distribut ion of the resource. For example, water is usually required for irrigation during the drier season of the year and it is thus advantageous to include another compatible use in the wetter season to maximise accruable benefits. It is generally considered good practice, nowadays, to ensure that as many uses as possible, such as irrigation, water supply, power, navigation, recreation, and flood protection are included in the design and operation of a water resource project (Linsley and Frazini, 1979).

2.1.3 Multidisciplinary Management

This approach involves the expanded use of many disciplines or professions (e.g economics) to help provide better returns by avoiding the pitfalls caused by omissions created by a lack of complete knowledge of all processes or factors involved in the scheme by the main planning group. In addition, data requirement, procurement and utilisation for water projects concerns various disciplines ranging from hydrology to economics, sociology, amongst others.

2.1.4 Integrated Basin Management

Water resources management is closely linked to land management. Landuse and allocation, determine to a large measure the quantity and quality of water available for use. With land management in mind, the development of water resources must be closely tied to the technique of integrated basin development, now commonly employed in river basin management.

Initially, the well, spring or borehole was considered the basic planning unit of water resources development. As population, urbanisation, industrialisation, and economic growth increased, it became clear that water resources management needed a larger unit. This has led to the adoption of the watershed or river basin as the base unit for Water Management. The river basin, is itself, a conglomerate of small watersheds and includes the whole of the contributing area to a specified point on a river and this consists of the main channels and tributaries,

reservoirs, and the surrounding land mass.

The United Nations Commitee for Natural Resources (U.N.C.N.R, 1980) emphasised the need for the development of water resources in a river basin to take cognisance of the overall plan of the entire basin. This will assist in avoiding the dismal situation whereby activities in one part of a river basin adversely affects the other. It is also desirable that the sequence of implementation of the various projects should conform with an overall plan, aiming at the most beneficial use of available resources. This confirms the need for integrated river basin development.

Integrated basin development involves the practice of river basin activities like cultivation, soil conservation techniques, vegetation management, and water management (including reservoir developments) to achieve economic efficiency. The downstream development of a basin must be coordinated with upstream developments. Similarly, upland management must be coordinated with flood plain development. The aims of downstream development are, inter alia, flood protection, channel stability, navigation, irrigation, water quality, and environmental protection. Upstream development may be mainly directed towards river regulation, flood protection, irrigation and water power. Downstream channel erosion or sedimentation may result from upstream or upland development(UNWRS,1985). Climatic modification a part of a basin may be caused by large surface water development in another.

Integrated management also has a bearing on the various aspects and forms of water namely the quantities of the surface, ground, and atmospheric water and its quality.

Integrated basin development becomes even more complex in international, interprovincial or interstate river basins. International or interstate treaties or agreements are required with the cooperation of the countries or states concerned before optimalty in water management and use can be achieved.

2.1.5 The Inter-Basin and Multi-Basin Management Approach

Nowadays, even the River Basin is becoming too small a unit for planning, hence there is increasing adoption of Multibasin Development, which is practiced in the form of interbasin water transfers. Examples of such transfers include the California Valley Project (C.V.P) and the proposed Russian plan to divert rivers flowing northward to the arctic ocean southwards into the Caspian sea through dry regions (Mather, 1984). Even in Africa, there is a new proposal to divert part of the River Zaire, which has an enormously surplus water potential to lake Chad and the Nile basin. This proposal, if implemented, will help relieve the water scarcity problem of the arid and drought prone Sahel region of Africa and relieve the flood and poor drainage (swamps) problems in the Zaire Basin.(Umolu/Oke, 1987; I.R.I Italstat, 1983).

2.1.6 Systems Approach

This study considers the river basin as a system. The United Nations Water Resources Series 59 (1985) report stated that for a good understanding of the interacting processes involved in a system, detailed modelling is required. This may not be accomplished by utilising only one 'model' because of the different inter acting processes involved. Many parts of the system, will usually be analysed by the use of specific models/submodels or other techniques, whilst other models or techniques are utilised to interlink the various models into a viable management system package. This is the systems approach. Optimisation, prediction and assessment of outputs may be carried out as appropriate.

The UNWRS report (1985) also stated that the systems approach is a broader notion than the widely used system analysis term which is just a technique utilised in carrying out the approach. Systems analysis may be regarded as a vehicle of the systems approach or its mechanics. On the other hand 'policy or strategy analysis' may be considered as one of the aims of the systems approach. All three are therefore interrelated. . Policy analysis aims at providing decision makers with a limited number of attractive policies or strategies by the use of the systems approach (through systems analysis) or otherwise, with the objective of comparing them using defined criteria. Economics is the usual criteria for such comparisons in water resorces management (Biswas,1976).

2.1.7 The Process Approach of this Research

An approach that combines the outlined strategies are rare to come by. The approach of this research does just that. It involves the use of a system, integrated, dynamic, process, multipurpose, and multidisciplinary methodology in analysing the performance of reservoirs as part of a river basin. It is integrated because it covers all activities in the river basin ranging from landuse to climate over the surface of the basin and includes the impact of upstream developments on those downstream e.g upland forested development impact on low flood plain developments. This study excludes water quality at this stage but this can be a subject of future research.

The approach is also multipurpose, providing water releases to various users (power, irrigation, etc); and multidisciplinary, combining the techniques of economics, hydrology, and other disciplines. Finally, it is process based and dynamic because of its use of mathematical models which takes into consideration the various time and space variant processes, parameters, and changes. These include both the dynamics of microprocesses and that of large scale changes with time in both hydrology and micro-economics. The reliability of results must be verified, inter alia, by extensive tests and sensitivity analysis. The approach is different because it combines the merits of the different approaches defined. A proper implemention of it will enhance the benefits from river basins and their storage projects in both short and long time periods. Figure 2.1 illustrates the practical parameters

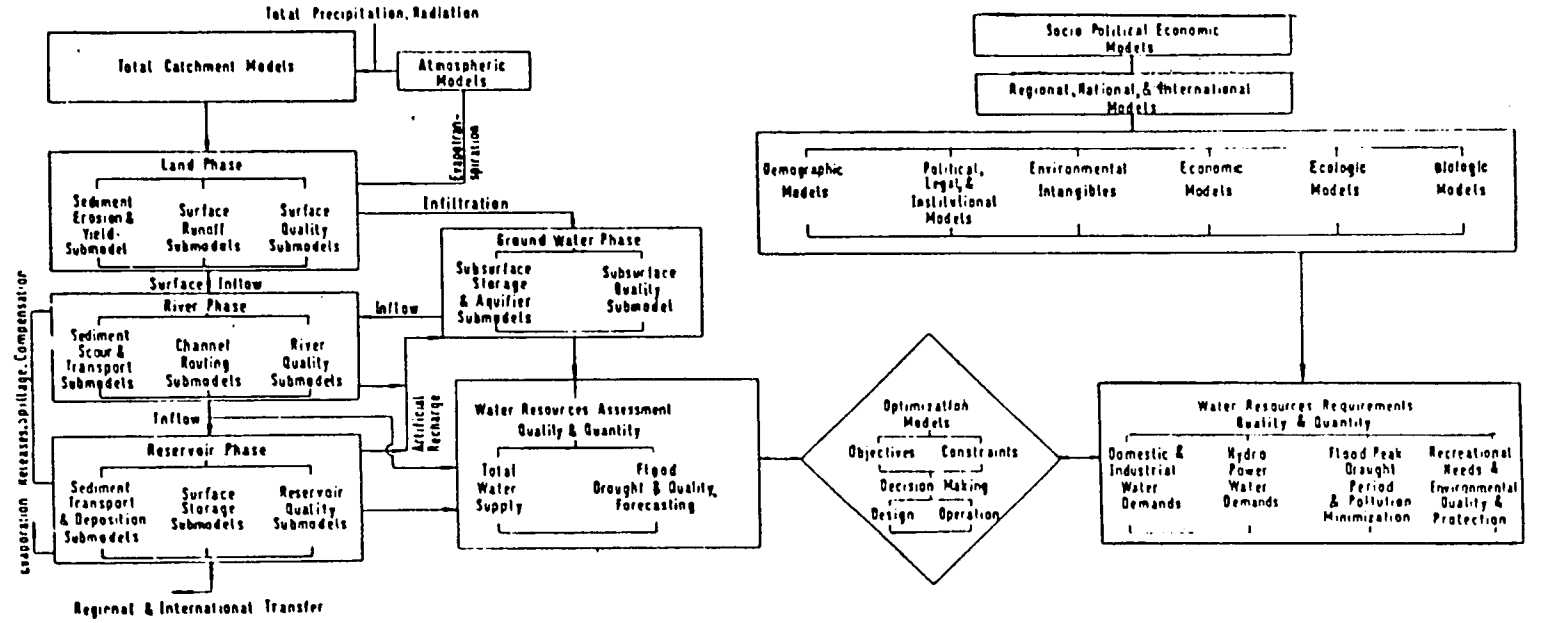


Fig. 2.1. Water resources assessment and requirement. (After Fleming. 1975)

utilised by the approach, including models, methods and policies.

As water resources management involves the rational planning of water uses and the evolution of ways to conserve available supplies, planning is therefore a prime aspect of water management. This is discussed in the next section.

2.2 Planning and Water Resources Management.

2.2.1 Definition

Linsley and Frazini (1979) defined planning as the orderly consideration of a project from its original conception through the evaluation of alternatives and the choice of the final design, using available and appropriate analysis techniques, to the point of a final decision on a course of action. This includes all work associated with the scheme from conceptualisation to final preliminary design, sometimes termed the feasibility report. The design of detailed engineering structures and other facilities can be undertaken after the feasibility stage. Each step, leading to a final decision, must be supported both by engineering judgement and quantitative economic analysis. This study, as part of the planning process reviews current practice in reservoir management and introduces new ideas on its optimisation with regard to prevailing changes in the global and local climatic regime and land use patterns (refer ch 1).

2.2.2 Analysis and Appraisal

Project economic analysis and appraisal, a part of the planning process, measures the inputs and outputs involved and relates them to each other by cost-benefit and, sometimes, by other economic techniques, using appropriate discounting factors. Sometimes, the benefits from a project are immeasurable or unquantifiable. Under such circumstances, the project objective clearly becomes one of defining the aims and trying to satisfy them at least costs. This is the cost effectiveness or least costs analysis policy (Goodman, 1984; Linsley and Frazini, 1979).

2.2.3 Other Planning Approaches

Other approaches to planning include the concept of regional masterplan, multi-objective, multi-disciplinary, etc are well discussed in the literature (Mather, 1984; James and Lee, 1971; UN/ESCAP/WRS 59, 1985, etc)

2.2.4 Project Choice and Implementation

The choice and implementation of a water project depends upon numerous engineering, environmental, economic, sociological and political factors, some of which are difficult to quantify. While the main guiding criteria for obtaining an optimal management policy may be economics and engineering, other considerations involving social and political factors may affect the final decision on project acceptance. In short, accurate application of engineering and economic analysis is not sufficient to guarantee the

implementation of economically sound projects. Nevertheless, the use of facts and figures from engineering and economic analysis methodology is vital for convincing clients and decision makers about long term returns from a project or alternatives.

2.2.5 Strategy

Linsley and Frazini (1979) stated that current strategies for general water resources planning tend to include the items summarised and seperately criticised below.

(a) Firstly, goals, objectives, criteria, and proposals are defined according to needs,prior to the planning exercise. After planning, implementation may require legislation or group consensus.

Sometimes, objectives are too complicated for specification by political, administrative and other associated means and usually without adequate feedback to ensure that their consequences are in the general will. Often, various pressures force the water manager to make new decisions before the feedback from previous considerations crystallise.

(b) Secondly, an optimal scheme is identified by subjecting the various alternative designs to socioeconomic and engineering analysis using all available information and data. Goodman (1984) and ASCE (1974) add that there is the need for a preliminary and detailed assessment of impacts.

(c) Thirdly, conflicts among project objectives and between project purposes do sometimes arise with and within the balance of the national economy. These are supposed to be sorted out by an integrated approach utilizing appropriate experts in various disciplines, such as hydrology, meteorology, economics, ecology, hydraulics, etc. In general, for now, no water resources plan can be considered perfect, as limitations, such as fragmentary data, limited analytical proceedures and tools, hazily defined objectives, difficulties in assessing social parameters, and so on, exist. This planning methodology is focused on reservoirs in the next section.

2.3 Reservoir Management.

2.3.1 The Planning of Reservoirs

The review of water resources and river basin management and planning would be incomplete without isolating and outlining the practice currently employed in reservoir management. This is because reservoirs, as vital water storages, remain one of the most important components for optimal water management in a river basin.The design and management of a surface water storage reservoir starts with the following proceedures (Mather, 1984).

(a) The identification of the project purpose

(b) The identification of the source of water

(c) The identification of suitable sites for the construction of the reservoir.

The feasibility study for evaluating a possible site would normally include (Cole, 1975)

(a) topographical surveys to determine the proposed water area and volume by elevation.

(b) geological survey of the dam and reservoir sites to identify their suitability for the most economical dam structure and reservoir respectively. Possible fault and potential leakage zones are identified and investigated for detailed design purposes.

(c) hydrological assessment of the catchment flows at the chosen site, including design floods for spillways.

(d) appraisal of the necessary landuse patterns and changes.

(e) estimating the cost of the scheme including the cost of the impounded land and the loss of production value of the use to which the impounded land could have been subjected. Costs must also include the cost of structures.

(f) the estimation of reservoir yields and expected benefits from the scheme with an assessment of the schemes viability

(g) the preliminary design of the final project choice among alternatives.

2.3.2 Assessing Yields

The economic viability of a reservoir is determined by its yield which is the volume of water regularly available over a unit period of the time (Shaw, 1987). Reservoir

yields can be determined from charts consisting of the plots of yields and effective storage capacities for various catchment areas for specific regions (for example, the Lapworth chart for the U.K), These are available in the literature (I.W.E, 1969). Specifically, the I.W.E (Shaw (1987)) defined reservoir yields as

" the uniform rate at which water can be drawn from the reservoir throughout a dry period of specified severity without depleting the contents to such an extent that withdrawal at that rate is no longer feasible."

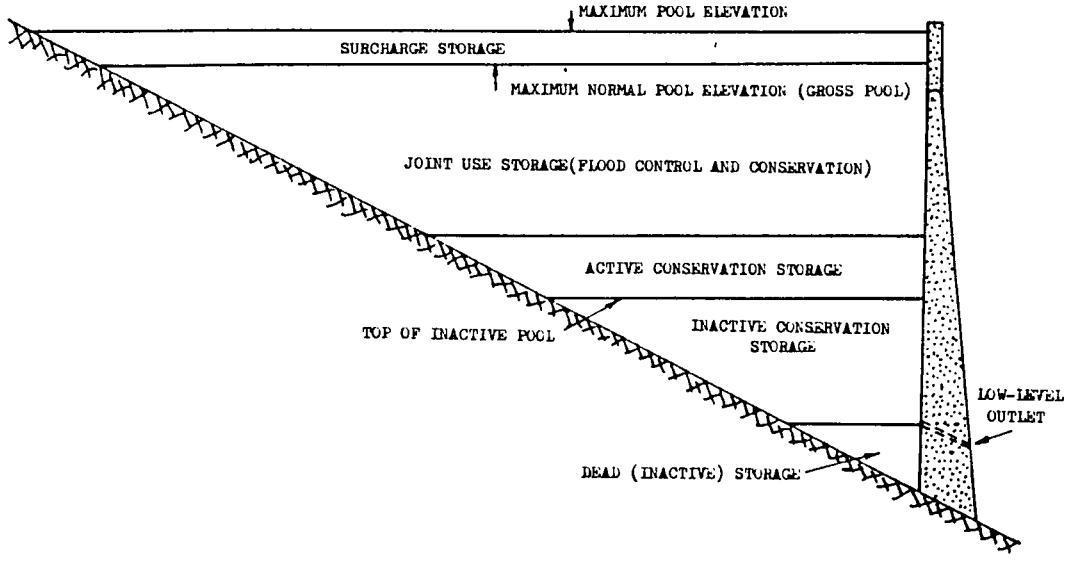
Shaw (1987) explained that for a fixed storage capacity of a reservoir over a drought period of greater severity than that of the design, the regular quantity of water available for supply (i.e the yield) would have to be reduced. In short, the occurence of any drought governs the yields from a given storage. Alternatively, if a higher yield than the design is required, then the design storage capacity would have to be increased to meet the demand. Thus, the evaluation of reservoir yields requires a thorough study of low flows and the phenomenom of drought. Research on the subject of reservoir yield analysis and drought studies have been extensive and interlinked and are well documented in the literature (McMahon and Mein, 1978; W.R.A, 1966, Mustafa, 1981).

The more common methods utilised for reservoir yield determination are the Mass curve or Rippl technique and the Residual Mass curve method. These use historical drought flows for their analyses and these are given in

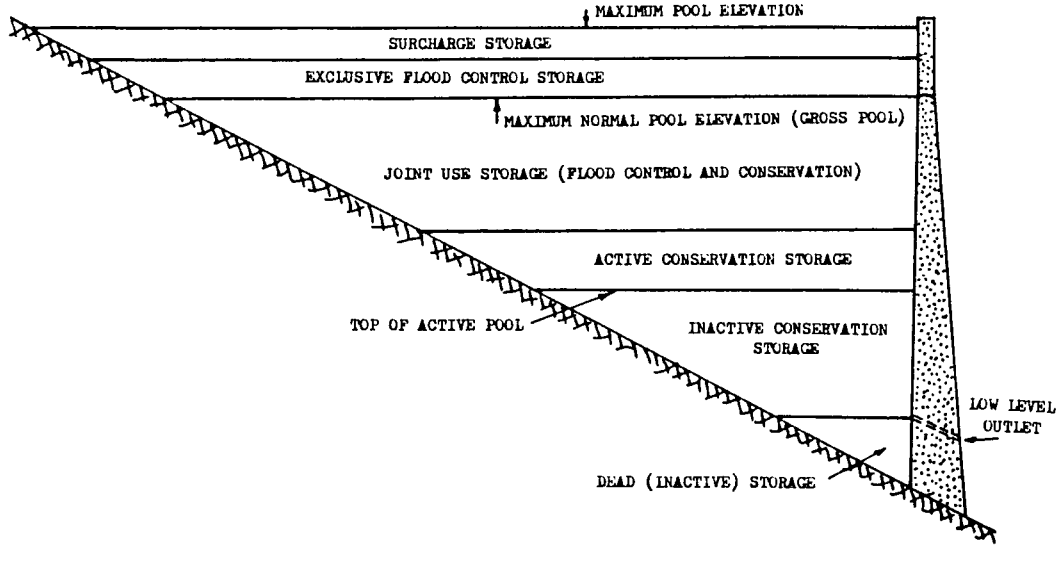
the literature (Shaw,1987; McMahon and Mein, 1978). A third method, the Hurst method uses synthetic drought flows to estimate reservoir yields (Hurst et al, 1965). The yield of a reservoir determines how it should be operated in the short or long term to maximise multipurpose or single purpose benefits. The operation of the reservoir is controlled by the use of an appropriate rule curve. The concept of reservoir operation and rule curves are discussed in the following subsections.

2.3.3 Reservoir Operations

A reservoir operation study involves routing streamflows for a period of years through the reservoir and making reservoir releases in accordance with schedules for various specified purposes and for flood control (Peterson, 1984; Acres, 1985). A range of conservation yield combinations and flood damage reductions are used to determine an array of storage space allocations which control the releases that serve as a basis for benefit costs evaluations. Some of these zone combinations are illustrated in figures 2.2. Reservoir operations are a vital part of River Basin Management because it controls the supply of water to various uses over time. In addition, the planning of reservoirs involves not only its design but the forecasting of operational releases over short and long time periods so as to maximise benefits on a continous basis.



a) At sites where annual 'wet' and 'dry' seasons can be clearly identified



b) At sites where major floods can occur at any time of the year

Fig.2.2 RESERVOIR STORAGE ZONES

Source Patterson, 1986

2.3.3.1 The Role of Simulation Time Periods

The time period and time step utilised in reservoir operations studies vary. For long term forecasting, both reservoir inflow assessment and operation models (i.e river basin models), are required to work on a monthly or annual basis. For a large scheme, the use of a minimum period of 50 years is the general practice and monthly time increments are considered appropriate.Peterson (1984) states that yearly intervals may also be used but only for preliminary analysis. For projects with small storage capacities, weekly or daily time intervals may be required to give satisfactory results over a considerable time period. On the other hand, for short term forecasts, the models are required to compute on a daily or hourly basis or less. For example, an operation study that may require short term analysis, involves the assessment of the firm yield and power output in a critical or extreme dry cycle.

This Research and Long Term Forecasting

In reality, not many models are able to perform the dual task of forecasting short and long term releases simultaneously and incorporating them into operation models is not easy. The approach adopted in this work overcomes this difficulty since the representation of processes and their interactions over time in the models utilised herein requires the use of small time intervals (15-30 minutes) in computations spread over a complete water year.

In addition, all years with available data can be simulated sequentially. Outputs of reservoir releases, inflows, reservoir levels, storages for specified 30 to 60 or more minute intervals covering a period ranging from a few hours to a whole year or more as required, can be obtained. The only limitation is that the size of data required for multi-annual simulations is enormous. But modern computers can cope.

One technique that has been utilised in this work to overcome the problem of a large database required for long term simulations involves the use of one year of data, i.e the year with a flow hydrograph closest to that of the long term mean. This will be particularly useful for long term simulations of future conditions. However, expected departures from the long term mean conditions must be included in the simulations if realistic results are to be obtained. This can be done by sensitivity analysis or otherwise. This is discussed further in Chapter 3.

Real Time Operation

Real time operation is concerned with the optimal operation of an existing reservoir system with decisions on releases for various purposes being made in short time intervals (for example, in the study of a flash flood). A review of the subject of the optimisation of reservoirs operations by Yeh (1985) reveals that no general algorithm exists for this (Chapter 3). The choice of methods depends on the characteristics of the reservoir system considered, the specified objective function and criteria, and the

availability of data (Chapter 3). The optimal operations of reservoirs, be it real time or long term is the primary concern of water resources managers, worldwide. This cannot be achieved without the specification of appropriate reservoir operating rules or rule curves.

2.3.4 Reservoir Operating Rules and Rule Curves

Loucks and Sigvaldason (1982) summarised the reservoir operating rules commonly adopted in River Basin Management practice. Some of these are outlined below.

(a) For multiple reservoirs in series, downstream reservoirs are depleted before utilising upstream reservoir water to meet downstream demands. This cuts down on spillages from downstream reservoirs.

(b) For reservoirs in parallel, water is released first from reservoirs with relatively larger catchment areas per unit storage volume capacity. This improves conservation policy.

(c) In general, spills will be minimised if water is drawn to equalise the probability of reservoir filling for each reservoir in a series or parallel combination. This requires the continuous monitoring of water-in-storage volumes and the estimation of future inflows.

Loucks and Sigvaldason (1982) suggested that operating policies may include:

(1) maintaining target storage volumes or levels as

closely as possible and at the same time satisfying various water needs downstream.

(2) meeting defined multiple storage allocation zones such as conservation, flood control, spill or surcharge, buffer, and inactive or dead storage zones..

(3) incorporating a flow range which describes the relationship between reservoir levels and channel flows. The release would depend on the defined subzone or zone in which the storage volume is in.

(4) the use of conditional rule curves such as described below.

Rule curves are often used in the actual system operation of reservoirs. For single or multipurpose reservoirs involving hydropower or recreation, operating policies and their rule curves usually define the desired storage volumes and discharges at any time of the year as a function of existing storage volumes, the time of the year, the demand for water or power and possibly the expected inflows (Loucks and Sigvaldason, 1982).

Any of the above policies of reservoir operation can be applied in testing the approach proposed in Chapter One. For simplicity, however, a water conservation policy which allows the reservoir to build up freely without spills will be used. An elevation/water-in-storage release rule curve will be used in implementing this policy. Figures 2.2 illustrate examples of reservoir zoning for multipurposes. The figures are self explanatory. Details

are available from the literature (Peterson, 1984).

Finally, many diverse uncertainties are encountered in River Basin Management in general and in reservoir planning and operations in particular and these are described in the next section. These uncertainties, if not properly assessed and managed, can lead to severe socio-economic losses.

2.4 Uncertainties in River Basin Management.

2.4.1 Hydrological Uncertainties

All projects are planned for the future and hence the precise conditions to which the works will be subjected are relatively unknown. This is particularly true in water resources engineering as parameters such as design flows, rainfall, evaporation, and floods are uncertain. This is because of combination of the stochastic nature of these events and the ongoing changes in world climate resulting from the greenhouse effect. In addition to the aforementioned hydrological parameters, other factors subject to some form of uncertainty include erosion and sedimentation rates, sediment yields, land use, water requirements for plants and crops , water supply for urban and rural populations, benefits and costs,etc. Errors in hydrological and hydraulic estimates are basic and may be devastating as it may lead to catastrophic effects on the economy of the whole scheme under consideration. This reinforces the need for comprehensive assessment of these parameters (Linsley and Frazini,1979). A process assessment approach may achieve this.

2.4.2 Economic Uncertainties

Uncertainties also occur in other external factors which affect the economics of any proposed scheme. These include inflation rates, interest rates, etc. This in turn leads to uncertainties in acquiring financing for the project, especially from external sources, as lending bodies tend to have drawbacks, as they are unsure of the expected economic returns of projects, a situation aggravated by uncertainties (James and Lee, 1979) .

2.4.3 Assessing Uncertainties

Rainfall, flow,sediment erosion, and deposition sequences for future years are unpredictable and cannot be assessed with reasonable certainty using current engineering methods involving statistical guesses. The use of such statistical predictions involve some calculated risk factor. More refined probabilistic and statistical methods have been developed for analysing these risks with the main objective of reducing them (chapter 3). However, the risks remain and may be substantial in certain cases. However, the use of process based models for simulating these sequences as adopted in this research may improve data and parameter estimates and consequently, minimise the risks considerably.

Some of the statistical methods, currently, employed in water resources management include frequency series and recurrence interval techniques for design against floods and droughts (chapter 3); others are, flood formulae,

partial duration series, P.M.F's, and stochastic modelling techniques. In addition, deterministic simulation is now increasingly utilized in the determination of some of these flow and flood parameters and sediment erosion/deposition rates, characteristics and yields. Descriptions and discussions of these methods are available in the literature (Fleming, 1975). For more realistic design and operational information generated by the use of these methods, the concept of the changing river basin must be introduced prior to their use or to their results. The management of hydrological uncertainties and extremes are discussed in the next section (2.5).

2.5 Managing Hydrological Extremes and Uncertainties

Prior to the efficient management of the water resources of a river basin, an assessment of the amount of water available and its variability must be made with an acceptable degree of accuracy. It is necessary therefore to review and evaluate the hydrological processes and concepts required to effect such an assessment. This leads one to the concept of the hydrological cycle.

2.5.1 The processes of the Hydrological Cycle

The study of hydrology naturally begins with the concept of the hydrological cycle, which is the temporal and spatial interaction and distribution of precipitation, evapotranspiration, physical watershed characteristics, and soil moisture conditions, with the hydrograph of

streamflow as the response (Mather, 1984; Fleming, 1975). The heating of the earth's land surface and oceans leads to evaporation, which is the transfer of water from the liquid to the gas eous state and its ultimate rise to the atmospheres, where it may remain for as many as ten days. This vapour changes back to the liquid form through condensation thus forming clouds, from which precipitation is produced. The precipitated water returns to the oceans directly or via the land surface.

Precipitation on the land surface is partially intercepted by vegetation and part may return immediately to the atmosphere as evaporation while the portion reaching the ground may collect to form surface runoff or infiltrate into groundwater storages, groundwater runoff or interflow. Vegetation absorbs water from the groundwater and soil moisture storages which is eventually transpired. Runoff in rivers may eventually end up in lakes, reservoirs, and the oceans from where water returns to the atmosphere as evaporation. This briefly, is the hydrological cycle concept.

The major problem facing engineers and hydrologists and other practioners concerned with water matters is the quantification of the amount of water in the different phases and the water transfer rate between the phases of the cycle. This information which is required for the design and management of water schemes is obtained by direct measurement or remotely by mathematical simulation.

The hydrological cycle is a macroscopic view of the water

balance of a large system such as the earth. This balance can be narrowed down to a smaller system such as a river basin. This balance is often characterised by extreme conditions as experienced during droughts and floods.

2.5.2 Drought Management

The drought phenomenom is basically the scarcity or lack of water as defined in chapter one. It tends to result when meteorological conditions do not supply adequate precipitation to satisfy plant and human water needs for an extended period of time. It is experienced commonly in many parts of the world, especially in semi-arid areas. In West Africa, many authorities have indicated that the recurrence of drought sequences is 30 years but this cannot be substantiated by available evidence because of the paucity of data and the recent trend in global climate change (NFMST, 1985; Thambyapillay, 1985; Lavender, 1984; Acres, 1985; Merz and McLellan, 1986). Mather (1984) states that the determination of the 50 to 100 year frequency of drought is very dificult, if not outrightly impossible because of its stochastic nature. There is very little in the way of available technology for the elimination of droughts though there are ways to lower the severity of its effects.

Two solutions, namely, the provision of drought resistant species of plants for agriculture and the cultivation of shelterbelts in areas subject to drought-induced desertification, have been mentioned and used (in West Africa and elsewhere) for managing drought effects but

these have been found to be ad oc and on implementation, have not been totally successful (NFMST, 1985). Inter basin water transfers and the recycling of water have also been proposed as long term drought management options. Interbasin water transfers have been successfully utilised in Carlifornia (Mather, 1984).

Droughts and their effect can be best minimised by the efficient allocation of water and a good management of available water resources, in the form of operation and maintenance of existing facilities and the design of new ones. Mather states that "the war against drought cannot be won with big engineering works". Nevertheless, the use of surface and underground water storages to retain water for use during drought periods would be helpful in such circumstances. The water surface may be protected by the use of available and cost effective technology to minimise losses to evaporation.

2.5.3 Floods and Flood Management

Floods, like droughts, can be ha zardous when severe but in a different way. They can lead to sudden disasters on flood plains and settlements. The flood disasters in Bangladesh and the Sudan have been mentioned in chapter one. The control of flood waters in the short and longterms by the use of storage reservoirs is common practice, nowadays. Interbasin water transfer is being used increasingly as a flood management technique. Flood protection structures are built in alluvial or flood plains to lessen or eliminate flood damages. Flood

forecasting techniques combined with appropriate reservoir operation proceedures are also utilised to manage floods in many places. Flood warning techniques are used nowadays to minimise damage and loss of life from floods. The lack of information on the above mentioned uncertainties can be worsened by poor data management practice and can lead to socioeconomic failures from water projects.

2.5.4 The Role Of Minimum Reservoir Operating Levels

Lowering the minimum dead storage levels in reservoirs during operations and specifying low but optimal dead storage capacities (proposed in chapter 1) is a possible technique for alleviating the flood and drought problem. Some examples of failures from drought , not unassociated with the inadequacy of current river basin management, design and operation policy are next presented. The examples from flood disaters have been discussed extensively in chapter 1, and a repeat is considered inappropriate.

2.6 Examples Highlighting the Failures of Current Management Strategies

Failures or the non performance of many river basin and reservoir projects during normal hydrological periods or during periods of stress as experienced during droughts or floods are common place. This is caused by the inadequacies of past and present management policies and criteria, especially due to the neglect of detailed process interaction and their dynamic mechanisms in project design in the catchment concerned. A few examples

of these failures relating to floods have been discussed in Chapter 1. This section focuses on the drought problems of Sahelian Africa. These include the Aswan reservoir project in Egypt, the Akosombo scheme in Ghana, and the Kainji dam project in Nigeria. By highlighting some of the problems encountered, it is hoped that future storage schemes, such as the Donga basin plan in Nigeria, may be spared some of the planning, design, and operational pitfalls common with such projects.

2.6.1 The Aswan Case

The Nile system whose most remote source is the Luvironza river in Tanzania flows 6,824 km across the African continent,through nine countries and three climatic zones to the Mediterranean sea. The river is actually two main rivers, the Blue and the White Nile, which converge near Khartoum in the Sudan, after following long and complex routes (figure 2.3). Smith (1986) reported that the White Nile is relatively free of sediments while the Blue Nile is laden with silts and sediments. The Aswan high dam is the latest and largest impoundment on the Nile to date and has a capacity of about 186,000 million cubic meters, which is directed mainly to irrigation, hydropower, flood control and low flow augmentation. The inflow in to the Aswan reservoir is about 55,488 million cubic meters in a normal year, twelve to fifteen percent of which is lost to evaporation and seepage leaving less than 49,320 million cubic meters in the live storage. In the last six years, however, average inflow into Aswan has been less than

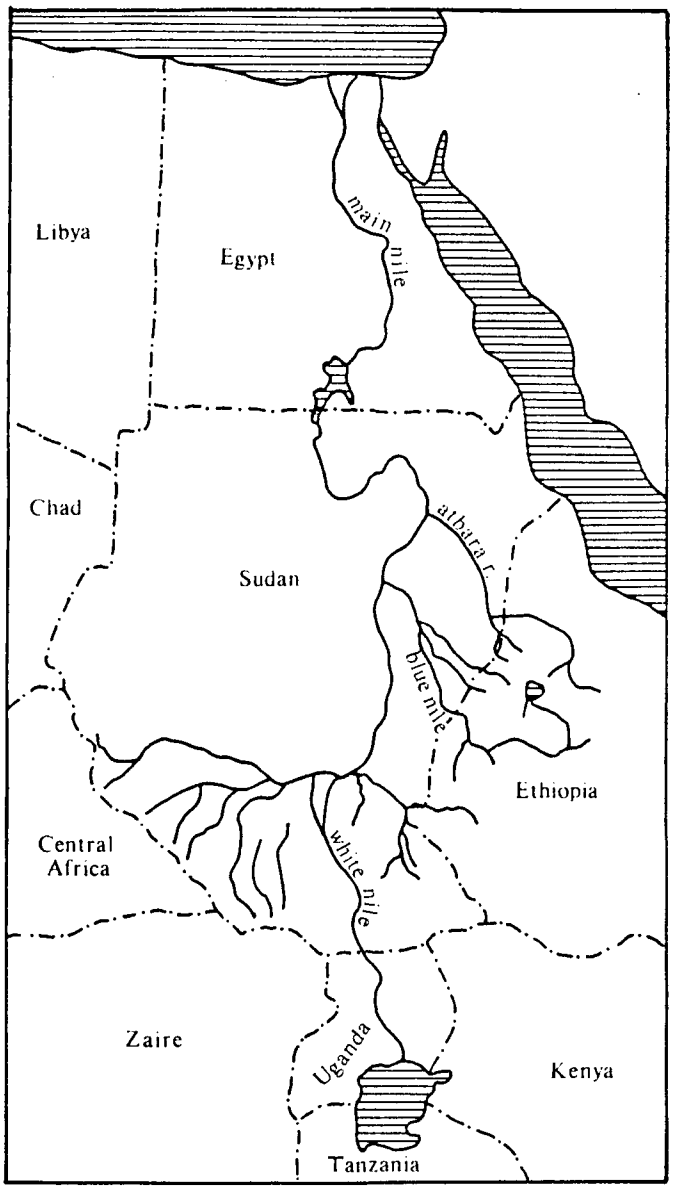


Fig. 2.3 THE NILE BASIN

Source Smith, 1986

43,157 million cubic meters and this is far exceeded by outflows and withdrawals. In fact, since 1978, there has been no net addition of water to the reservoir except during the 1988 Nile floods.

Falling Reservoir Levels from Drought

Egypt's demand for water in 1985 totalled 59,432 million cubic meters, of which, 49,568 million went to irrigation, 3,946million cubic meters went to power generation, 3,329 million cubic meters went to industrial users, and 2,466 million cubic meters for domestic consumption. The annual shortfall of 16,522.2 million cubic meters was compensated for by the enormous buffering capacity of the reservoir. The net amount of water available from the reservoir is dwindling in the long term, since water levels are so low and continue to fall. To meet demands, draft reductions (below normal minimum operating levels)have occured in the last few years to prevent catastrophic shortfalls. This is a technical failure. The reservoir is not likely in its present state (despite the 1988 flood) to withstand further years of drought. Thus, a complete failure may occur. The fact that the reservoir survived so many years of drought is due, not necessarily to the adequacy of the original design or operating policy, but to the commendable water conservation methods of the Egyptian Government (Smith, 1986).

The drought sequence was interrupted in August 1988 by heavy rainfall in the Blue Nile river basin resulting in heavy floods, thus replenishing the stored water at

Aswan.An immediate resumption of the drought sequence following this recent interruption or the contemplation of a future extended drought sequence may necessitate the review of existing operation policy by the lowering of the minimun or dead storage levels to meet an increasingly catastrophic situation. However, this may be difficult in this case, being an existing dam, as long established hydropower intakes may be impossible to operate during such failures or drawdowns.The criticality of the situation would determine the necessity of the construction of a supplementary intake and powerhouse to cater for badly needed energy and irrigation requirements for the 60 million or more people in Egypt.

Extra Water Management Options

In addition to the suggested review of the design and operation policy of the Aswan reservoir to meet an extreme drought, other alternative methods for meeting the critical situation are :

(a) The reuse or recycling of agricultural drainage water will assist in saving about 4,932 million cubic meters. Recycled water, however, contains a higher concentration of dissolved salts which may be injurious to plants and crops, thus reducing soil fertility.

(b) Farmers may be required to irrigate only at night time to reduce evapotranspiration losses.

(c) Farmers could be encouraged to cultivate more of low water-demand crops.

(d) The use of groundwater in the Delta aquifer should be encouraged. The wells should be used only in the event of shortages because of the potential problem of salt water intrusion from the medite. sea.

(e) Reduction of evaporation from the Aswan reservoir. Wind breaks on the northwestern side of the reservoir would significantly reduce evaporation which acounts for about 12 percent of the total inflow to the reservoir. Harmless evaporation reducing chemicals,such as certain alcohols, could be utilised also. This is done by spraying reservoir surfaces with the chemicals.

2.6.2 The Akosombo Case

The River Volta has the largest river system in Ghana. Its main tributaries are the Black Volta, the White Volta, and the Oti rivers. The White Volta and the Black Volta originate in Upper Volta and constitute the northwestern portion of the Volta basin. The Oti river, rising in Benin Republic, drains through Togo forming the northeastern portion of the Volta basin. These tributaries flow south in the Akosombo reservoir created in 1965. The Volta after passing the Akosombo and the smaller Kpong development, joins the Gulf of Guinea about 90km east of Accra, the Ghanain capital (figures 2.4 and 1.4).

Drought and Reservoir Failure

The main problem in Ghana, and indeed in the whole of West Africa and the Sahel region of Africa is climatic. A recent study by Acres international ltd (Lavender and

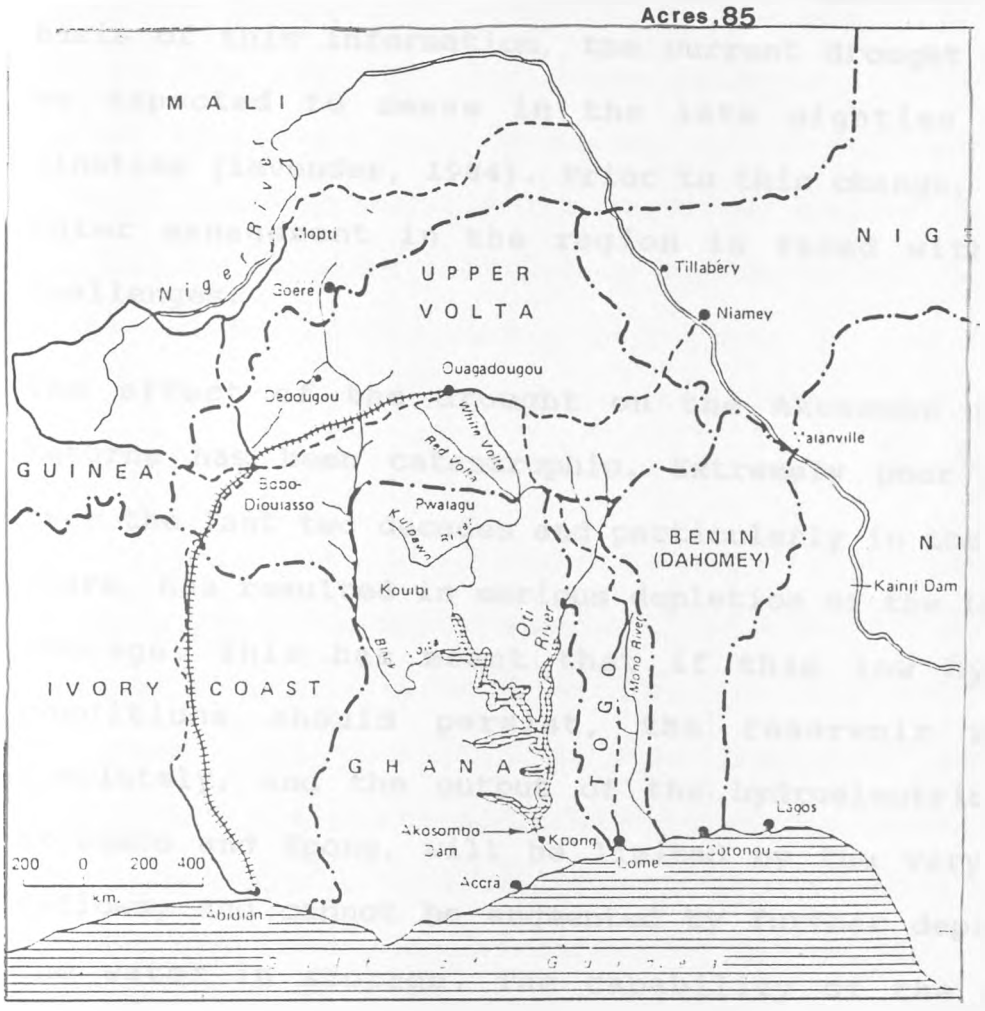


Figure 2.4 Volta River Basin. (After Futa, 1976)

Anderson, 1984a) which analysed long term flow and rainfall records from 18 gauges throughout Africa, including the Volta basin area, indicated that the recent drought experience in the African Sahel zone may be part of a normal 30 to 35 year cycle of wet and dry periods, in line with past African hydrological experience. On the basis of this information, the current drought cycle may be expected to cease in the late eighties or early nineties (lavender, 1984). Prior to this change, however, water management in the region is faced with serious challenges.

The effect of the drought on the Akosombo reservoir returns has been catastrophic. Extremely poor hydrology over the last two decades and particularly in the last few years, has resulted in serious depletion of the Lake Volta storage. This has meant that if this low hydrologic conditions should persist, the reservoir may fail completely, and the output of the hydroelectric plants, Akosombo and Kpong, will be limited by the very low nett inflows, and cannot be augmented by further depletion of the water in storage. The capability of the existing hydroelectric system without storage depletion is only 570 Gwh/a or 1.56Gwh/d. This is less than 45 percent of domestic demand and 15 percent of total demand. Hydropower is at present the only form of energy utilised in Ghana, though there are plans to resuscitate the Tema diesel plant, a direct result of the current drought problem. The alternative of importing power from neighbours such as the C'ote D'ivoire, is nonexistent as these countries are also

affected by the regional drought with their thermal energy capabilities being severely limited.

Load Curtailment

This adverse hydrological balance in the last 15 years has affected lake levels severely. At the end of the 1983 wet season and some of the years following, the Lake Volta water level was well below the normal minimum operating level. Load curtailment (or shedding) had to be practiced to prevent a total collapse of the power system, which in turn endangered the cash flow required by the Volta Power Authority to service its foreign debts and meet power operation costs.

Short term measures to avoid complete collapse of the power system, promote reservoir recovery, and maintain financial stability were initiated in 1984 and 1985. These include load shedding to the domestic and foreign customers; significant adjustment to both local and foreign tariff rates; maintenance of a minimum water level above elevation 73.15m within a 90 percent confidence limit, giving an energy draft rate less than 7.7 Gwh/d. For the reservoir to recover appreciably, to a level of about 75.60m, the energy draft must be less than 5.5 Gwh/d but to maintain a positive foreign cash balance within a 90 percent confidence limit, an energy draft rate greater than 7.3 Gwh/d will be necessary.

Evaporation Losses

This desperate situation at Akosombo is complicated further by the high evaporation rates because of the enormously large reservoir area and the aridity of the region. Annual reservoir evaporation rates, as obtained from studies by Halcrow (1970's),Penman(1960's), Kaiser Engineers(1965, 1966) on the Volta reservoir are 1.4m, 1.78m and 1.52m respectively. The range is therefore between 1.4 to 1.8m, depending on the year. This is considered extremely high averaging about 15 percent of inflows.A recent study by the Ghana Meteorological services yielded similar results.

Improved Reservoir Management by Operating with Lower Minimum Levels

Acres (1985) mentioned that a change in operating policy at Akosombo, involving a review of the minimum drawdown level from 75.59m to 73.15m results in higher energy generation, lower spill energy, early reservoir recovery, and better foreign exchange cash-in-bank projections. This policy adjustment, would return the reservoir conditions to normal in a few years, with at least 80 percent certainty. Spillage at Akosombo and Kpong under planned or designed operation is equivalent to about 900Gwh/a which is about 24 percent of potential energy. Reducing the minimum drawdown level could reduce spilled energy to about 5 percent, with total energy climbing from 5192 Gwh/a to just over 6000 Gwh/a. This gain of energy clearly offsets the losses to evaporation and spill, and would be very beneficial, especially during periods of prolonged droughts.

All recent reservoir operations levels at Akosombo vary between the full supply level of 84.13m and the newly recommended minimum drawdown level of 73.15m rather than the unoptimised minimum level of 75.59m, specified in the original design. This latter level is the lowest at which normal shipping operations on the reservoir are feasible, otherwise it may have been possible to operate the reservoir at slightly lower levels. In practice, however, energy has been generated at levels lower than 73.15m but this has been achieved only with difficulty since significant problems of vortex entrainment which cause cavitation have been experienced. Optimisation of reservoir levels to combat the effects of low inflows (due to drought) or excessive lake evaporation losses on power generation or otherwise, as experienced at Akosombo, must take these constraints of navigation and cavitation into consideration.

2.6.3 The Kainji Reservoir case

The River Niger which drains virtually the whole of West Africa, originates from the Futa Jalon mountains in the west of the subcontinent, flowing north-east for about 1000 km, beforeswinging southeast into Nigeria, from where it flows south into the Atlantic Ocean. The river, which is 4184 km long, has one of its major impoundments at Kainji in Nigeria, about 100 km into the country. The Kainji/Jebba lake complex (figure 2.4) has been a major power source in Nigeria for many years, after its completion in 1968.

2.6.3.1 Power Failures from Drought Impact

The problem at Kainji is similar to that of Akosombo for the same reason, that is, drought. The reservoir levels have been so low that the plant is run at less than half of its installed capacity (760 Mw). The severe drought of 1978 in the Sahel led to almost complete failure of the reservoir and most of the country had less than six hours of power supply daily. This power was mainly obtained from supplementary Thermal plants, still undergoing development at the time. The situation was so critical in the late seventies that three emergency and minaturised gas and diesel fired 20Mw power sets were installed in Lagos, the capital, to reduce the unbearable darkness experienced there and to reduce the near panic situation.

The national power situation only improved slightly after emergency construction of the new Ogorode/Sapele gas and steam turbine plant and the expansion and rehabilitation of the Afam and Delta plants. The situation is still far from normal as the Kainji reservoir was constructed as the main bulwark of the Nigerian power system. The problem is further complicated by the ever increasing loads in the country, rated as high as 20 percent in the late seventies and early eighties.

2.6.3.2 Evaporation and Sedimentation

Coupled with low inflows from drought are the twin reservoir problems of excessive evaporation and sedimentation. The latter is important because the Niger

is a major carrier of sediment. In addition, the reservoir storage is much smaller than at Akosombo. The Kainji reservoir has substantially silted, in the twenty years since its construction.

Despite the enormous and numerous problems at Kainji, no effort at improving yields has been initated unlike at Akosombo, yet Nigeria continues to suffer serious power shortfalls. As the drought continues, one hopes that some attempt at reservoir recovery and water yields' improvement will be undertaken in an economical manner. The same problem is now experienced at the newly completed Jebba reservoir, an afterbay of Kainji,just 60 km downstream of Kainji.

2.7 The Role of Data Management

2.7.1 Hydrological Data

The above mentioned uncertainties and failures could have been minimised if adequate data collection and management had been untertaken before and after the construction of the projects. Water Resources Development involves research, project conceptualization and formulation, design, construction, operation, and maintenance. All these aspects require the provision and use of adequate data. Data collection must therefore be regarded as very serious business and should be pursued vigorously and professionally. Many schemes have failed not only because of inadequate or inappropriate management concepts, strategies and criteria but because of the dearth of data.

For example, in drought and flood management, at least 50 to 100 years of rainfall, streamflow, and evaporation data may be required to provide the necessary confidence for the optimal performance of a water resource scheme.

2.7.2 Other Data

In reservoir design, many years of sediment erosion and transport data are required to assess and predict the influence of past, present and future land use patterns on reservoir lives. Population, land use, inflation rates, discount rates, costs, benefits, and other socioeconomic data are required for adequate economic development of a water resources scheme.In planning such a water resource project, a data collection programme must be initiated and implemented on a continous basis to supplement available information and to have an upto date knowledge of the behavior of the basin of consideration. The programme must not terminate with the construction of the scheme as the most recent information of the project or catchment may be required for reviews, improvements, new schemes, and the solution of problems that may arise from the original project. There is a tendency to abandon data collection programmes after the completion of a feasibility study, especially if project implementation is defered, delayed or cancelled, a situation particularly common in the third world. This must be avoided at all costs since conditions change with time. New conditions may make implementation possible or newer schemes viable in the area.

2.7.3 Data Generation

The dynamic changes in climate and land use practices necessitates a more active data collection and management programme with the main aim of assessing the magnitudes of the changes, especially as this will assist in water management decisions involving data application to solve water resource problems. Since it is impractical to avoid implementation of viable project on the account of the dearth of data, it may be wise to improve the available database by correlation and or computer simulation techniques using deterministic or stochastic models, as appriopriate. These methods may also be used to create a completely non existent database such as is the case of ungauged streams or catchments. The application of these techniques in the planning of water resources projects in ungauged areas, a situation common in developing countries, is vital for the development of such societies and examples abound in the literature (Fleming, 1975; Praisewater, 1974). Data management is so vital that in many countries, it is usual to set up separate data planning and collection agencies and while in others seperate departments within an organisation do the job. Data gathering is a continuous process and one seldom gets all the data one needs at any particular point in time. Finally, it is important to state that in many countries, the information and data needed for water schemes are standardised by appropriate regulations and criteria.

2.8 Final Remarks

The problems of hydrological extremes highlighted above are not limited to the Nile or West Africa but like the reservoir sedimentation problem discussed in chapter 3 are commonly experienced in other parts of the World. It is therefore a World-wide problem. The scale of the problem indicates that current planning concepts and strategies are now inadequate and require a comprehensive review to obtain better results in an increasingly hostile World.

As World population is increasing rapidly, so are the land use patterns. The World is experiencing serious climatic changes, resulting in new and more prolonged water deficits in certain parts and surpluses in others. Drought cycles seem longer in drought prone areas. The severity and frequency of floods in many parts of the world have increased. Population pressures, aerosol use, and the general drive to industrialise in certain countries, have subjected the weather to the greenhouse effect resulting in unprecedented extremeties. Excessive reservoir evaporation is a also severe problem in hot climates and losses can be substantial and economically wasteful.

This dynamics in the global hydrological balance makes the inclusion of real life process representation in design policy mandatory for the optimal development of river basins. Unfortunately, the past and current approach, which may be multidisciplinary, multipurpose, integrated or systems in strategy as indicated in the preceeding sections (2.1-2.3) tend to ignore the interaction of the

real-life and real-time interaction of processes and the possible impact of future environmental changes and uncertainties. These water management policies are directed mainly to a fixed or semi-fixed condition which is inadequate as natural situations are both dynamic and process based.

For example, many engineering practices utilise a simple empirical catchment rainfall-runoff relationship using daily or monthly rainfall data to assess reservoir inflows for design and operational purposes. This concept, though integrated, is discrete and unrealistic. It differs from the approach adopted herein which uses a small time step to monitor the dynamic interaction of water and soil particles. Further, it realistically traces them through the channel and reservoir system by the use of small time steps, and accounts for the deposition of individual sediment particles in the live storage of the reservoir. Time series data are passed from the land through the channnel to the reservoir phase by mathematical models.

In particular, the monitoring of the temporal and spatial behaviour of interacting parameters remain sadly absent in current policy due to the use of a black box methodology. The resultant failures mentioned and others indicated show that these inadequacies are significant. Mo dern management and planning of water resources, therefore, must avoid, as much as possible, these pitfalls.

To avoid these pitfalls, optimisation maonagement must be dynamically based. Thus, process and dynamic mathematical

models are expected to play a significant role in water management in the future, especially, in assessing the mechanics and impact of catchment processes and other external factors, such as climatic shifts and land use changes, because of their predictive and heuristic qualities.

The approach proposed herein, of linking process based River Basin Models into a River Basin Management system, for effective river basin assessment is considered a step in this direction. The designs obtained by the use of this approach should minimise project failures. Water resources schemes should therefore stand the test of time. With more detailed research, the system proposed can be improved to minimise computation times for easier use on desk computers. Alternatively, dynamically and process-based design and operation charts can be developed for quick use by design engineers and planners. The efforts involved may be necessary as it may be justified by the socio-economic returns available from the project considered. The next chapter is an expansion of the river basin management practice with a focus on the sediment problem complete with appropriate examples.

CHAPTER 3

CHAPTER 3 : THE SEDIMENT PROBLEM:PROCESS ASSESSMENT AND MANAGEMENT

In the preceeding chapter, the practice of river basin management has been reviewed. Its role in controlling variations and extremes in the hydrological cycle were discussed. In this chapter, however, the role and impact of the sediment problem in the management of river basin developments are presented. A brief general definition of the sediment problem is given in section 3.1. Section 3.2 is focussed on the erosion aspect of the problem. A brief review of the theory , assessment and management of erosion is provided in section 3.3. In section 3.4, sediment transport processes and their assessment techniques are briefly reviewed. A discussion of the problem and economic impact of reservoir sedimentation is undertaken in section 3.5 with appropriate examples. Section 3.6 outlines available sedimentation control methods. The chapter is finalised with a suggestion that an assessment of the sediment problem can be best carried out by the use of mathematical models coupled with appropriate physically based measurements.

3.1 The Sediment Problem

The sediment problem begins with erosion of the land surface within a river basin and ends with the problem of sedimentation at river channels, reservoirs, lakes, and oceans. Fleming (1974) defined the sediment problem as

" the detrimental depletion by erosion and transport of soil resources from the land surfaces and the subsequent accretion by deposition on other land surfaces, in river channels, irrigation systems, lakes, reservoirs, and coastal areas".

Intermediate between the erosion and the deposition phases is the transport phase. In this phase, water is associated with sediment pollution and colouration. Sediment movement in streams and lakes often obscure the light needed for aquatic plant growth and can smother fish eggs. Sediment ranks with industrial and domestic wastes as a major source of pollution..Examples of the damages caused by the sediment problem are discussed later in this section and is well documented in the literature (Fleming, 1977).

3.2 River Basin Erosion

3.2.1 General

Soil erosion is the mechanism of detachment of sediment particles from the land surface. It is affected by the action of wind, precipitation,gravity, glaciation, river flows, frost, ice, and man. In the river basin, erosion occurs in all three reaches, upland, middle, and alluvial. Erosion processes are dominant in the upland reaches and progressively reduce downstream and downslope.

Erosion rates are affected by two important factors, climate and the work of man. Higher precipitation increases erosion rates. Higher evaporation from the land surface prepares the ground for higher erosion. Increased human activities such as cultivation, deforestation and the like also leads to higher erosion. The insutu deterioration of the physical properties of the sediment mass by climate and human activities to a level where the sediment resources are unsuitable for agriculture, forestry, or other uses is termed soil degradation and forms a constituent part of the general sediment problem. The FAO (1974) defines soil degradation as follows :

"Soil degradation occurs when soils develop characteristics adverse to the maintenance or stability of the land use system, either in terms of the decrease of quantity or quality of the output of the system or by increase of the amount of inputs required by the system ".

3.2.2 Reviewing Sediment Assessment Methods

The universal soil loss equation, (Wischmeir, et al 1960); the Fleming relationships (Fleming,1969), the Fournier equations (Fournier,1969), Gill's(1974), Al Khadimi's(1982) are some of the empirical relations in use for predicting soil erosion. The use of these relationships are limited to specific regions. The results obtained by their use are generally treated with caution as they tend to over or under-estimate the erosion rates and sediment yields from catchments (Fleming, 1977). It is good practice to

utilize them in conjunction with other methods where the accuracy of the soil yields and erosion rates are required, especially, in intricate design problems. For example, Diyam (1985) utilised some of these relationships to estimate suspended sediment yield for the Donga Basin at Manya (catchment area 9,000 sq km). The results obtained are summarised in table 3.1. The results were very different when compared for the relationships used. Only Al Khadimi's gave results near what was obtained (or estimated) from field sample data, 8 million tonnes compared to 9 million tonnes per year.

Table 3.1

SEDIMENT YIELDS million tonnes/yr	VARIABLE USED	REFERENCE
0.8	catchment area	Fleming (1969)
11	mean annual dischg and veg cover	Fleming (1969)
8	soil erosion index	Al Khadimi (1982)
4	Mean annual runoff	Gill (1974)
9	Field sample data	Diyam (1983)

Mathematical simulation of the sediment problem can be used to compliment the results from empirical methods. This is necessary because of the caution and restraint attached to their use and is due to the lack of adequate process representation in their formulation. Many mathematical models, built to assess the problem are available. One model, that is available at Strathclyde University, is the Strathclyde River Basin Model (Fleming, 1986) and consists of a watershed, erosion, channel and

3.3 Sediment Erosion Assessment and Management

3.3.1 Sheet Erosion

Musgrave (1947) proposed one of the early techniques for sheet erosion by the use of the Musgrave equation

$$E = F * R * (S/10)^{(1.35)} (L/72.6)^{(0.35)} (P/1.25)^{(1.75)} \quad \ldots\ldots 3.1$$

where

E is the average topsoil loss (tons/acre/ft or in/acre/yr)

F is the erodibility factor for given soil (tons/acre/yr)

R is the dimensionless factor i.e the ratio of the erosion rate of surface under existing cover to the rate under continous fallow or raw crop condition.

S is the land slope (%)

L is the slope length in feet.

P is the maximum 30 minute, 2-yr frequency rainfall (inches)

The equation like most erosion and sediment yield functions was first obtained by the use of plot experiments and multiple regression techniques.

Ellison (1947) proposed a power function relationship called the Ellison splash equation during his studies of

sediment routing modules. These analyse the water balance, erosion, channel flows, and sediment movement and deposition. This approach is an improvement on the empirical-conceptual-deterministic methods and involves a more complex deterministic representation of the basin, and includes many vital parameters, to help predict the catchment water and sediment response. The technique also helps in understanding the interplay and behavior of various parameters in a heuristic manner. This model has been used in this study to predict sediment erosion in the Donga River Basin. Erosion models of this sort can be used to assess the damages resulting from soil loss.

For example, it has been recorded that more than fifty percent of cultivated land is susceptible to serious erosion and the loss of topsoil reduces the soil fertility and its ability to maintain a good cover.In addition, erosion removes the absorbent topsoil thereby exposing the less pervious subsoil, This increases storm runoff and aggravates floods in downstream areas. The level of soil erosion in a river basin determines the amount of sediment entering reservoirs in the basin and this in turn determines the level of reservoir sedimentation.

rain drop detachment. The equation is simply written as

Es=K*V**(4.33)*D**(1.07)*I**(0.65) 3.2

where

Es is the splash loss.

D is the raindrop diameter.

K is the soil factor

V is the raindrop velocity

These two expressions are significant developments in assessing erosion. The problem is that they are not universal but are confined to the specific regions where the original data used were obtained.

Wischeir et al (1960) developed a universal steady state technique using plot experiments and the equation obtained was called the universal soil loss equation which is given as

$$E = RKLS.C.P \qquad 3.3$$

where

E is the annual soil loss (tons/acre)

R is the rainfall factor

K is the soil erodibility factor

L is the length of slope factor

S is the steepness of slope factor

C is the crop management factor

P is the supporting conservation practice factor e.g terracing, strip cropping, contouring, etc.

The main limitation of this method is that it does not account for the dynamic or time variant nature of sediment detachment- erosion process. A second limitation is that its use is limited to regions where data exist for parameter determination, as in many parts of the U.S.A. Thirdly, the technique does not indicate whether the rainfall causing detachment is also available for transporting the detached material.

Dragoun (1967) incorporated a dynamic concept in his erosional process representation by using empirical methods to relate soil loss to specific storm events and to account for antecedent conditions in the catchment producing an equation of the form

$$Ls=C.E.I.\ (1+Pa-Qa) \qquad 3.4$$

where

Ls is the quantity of sediment transported in a particular storm (in tons)

Pa = antecedent precipitation for a five day period (inches)

Qa = antecedent runoff for a five day period (inches)

Finally, a simulated-conceptual approach using a space-time variant mathematical model is the modern and more complete approach of predicting erosion and sediment yields. The first attempt at this was made by Negev

(1967). This model attempts to simulate erosional processes of detachment by rainfall and pickup by overland flow. The model, therefore requires continous overland flow data which is easily provided by the Stanford Watershed Model, (Linsley and Crawford, 1968) which is a hydrologic, water balance, and runoff simulator. The main limitation of the Negev model lies in the use of its empirical steady state power functions to represent the link between processes.

A second attempt was made by the United States Department of Agricutlure (Agriculture Research Series) (Meyer and Wischmeier, 1969). This considered four basic erosion structure factors, namely, rainfall detachment (Dr), runoff detachment (Df), transport capacity of rain (Tr), and transport capacity of runoff (Tf). The model composition operation is dynamic and interactive. It is currently in a state of further development (figure 3.1).

Fahmy and Fleming (1973) also incorporated the above variant sediment erosion factors into the strathclyde sediment model which is capable of assessing erosion and sediment yield for all kinds of catchments. The structure and operational methodology utilised in formulating the model are outlined in a later section of this chapter (figure 3.1).

3.3.1 Non-Sheet Erosion

Apart from the sheet erosion discussed above, other forms of erosion that may occur in a catchment include gully,

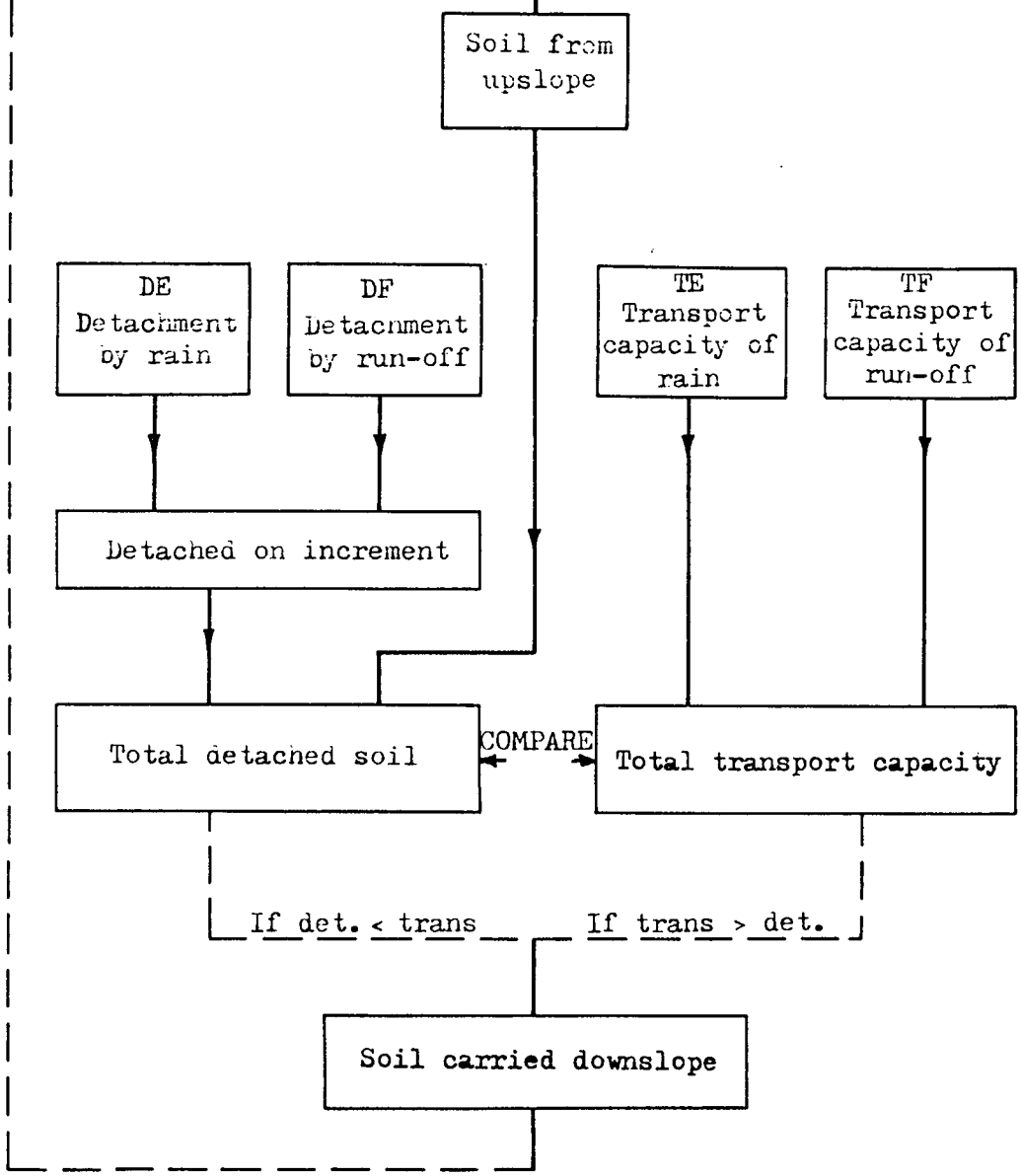


Fig. 3.1 SOIL EROSION PROCESS

Approach used to simulate the process of soil erosion by water. Four sub-processes (DE,DF,TE,TF) were evaluated for each successive slope length. Increment and soil movement was routed downslope as illustrated.

Source Meyer & Wischmeier, 1969

channel, and mass erosion. Gullies are normally associated with regions of extreme storm events, flash floods, and snowmelts. It is usually accelerated by poor and inappropriate land use practices. The mechanisms and causes of gully forms has been considerably studied in the U.S.A (Piest et al, 1972; Ireland et al,1939; Beer et al 1965; Thompson,1964; U.S.D.A,1966). There is a dearth of accurate methods for assessing the rate of gully erosion or gully advance, though a few statistical, regression and correlation techniques exist in the form of power relationships (see above refrences).

Channel erosion is the detachment or scour of sediment particles from the bed, banks, and flood plains of river channels. The problems caused or associated with this type of erosion are numerous, namely, the undermining of bridge piers, water table lowering, the lowering of stream waterlevels below intakes and sewage outfalls, etc. Many charts have been prepared for assessing this form of erosion (Fleming, 1977).

Mass erosion involves sediment detachment by gravity. This is a major erosion process common in very steep soil covered slopes. Important forms of mass erosion are landslides, and mudflows or soil creep. Examples of this form of erosion abound in the literature but detailed research into this processes is still required. Current knowledge has been obtained by research associated with the disciplines of soil mechanics, foundation engineering, and geology. Assessment techniques for mass erosion are

still in the rudimentary stages. Erosion as a sediment process is closely followed by sediment transport.

3.4 Sediment Transport Processes and Yield Assessment

Sediment transportation is obviously the connection between the erosion and deposition processes. The transportion of sediments have been widely studied and can be generally classified into wash, bed and saltation loads. The suspended load is the combination of the wash and saltation loads. The solution load is the sediment formed from chemical processes which is transported in solution. Suspended load assessment theories abound in the literature (Fleming, 1977, Walker, 1979). Most of the early theories of assessing sediment yields from conveyances have power relationships and involve the determination of the critical velocity, that at which silting is just prevented (Kennedy, Lindley, and Lacey from Fleming, 1977). Some of the later relationships use mean or average velocity values in place of the critical. Most of the mentioned theories and relationships are based on irrigation canal data. However, Leopolds and Maddock, and Fleming (1969, 1977) utilised natural river data in developing their relationships. These two approaches are significantly different and so are the respective relationships obtained by their use, especially, in terms of regimes, as indicated by the comparisons between canals and rivers provided below.

(1) A canal begins with the largest discharge at intake and reduces downstream because of smaller canal outlets. A

river is the reverse.

(2) Canals tend to lack significantly varying discharges and therefore cannot be compared to a river in flood or drought.

(3) The longitudinal profile of a canal is convex upward while that of a river is concave upward.

The Fleming equations, given below, are obtained in a study of suspended load data from 253 river catchments around the world. The results obtained show general trends between flow discharges, catchment area, vegetation, and suspended loads. The equations are not regime in the true sense but do show general relationships which provide a start point in analysis and design of sediment related water schemes.

$$L=3.82.5\ A_c^{0.9576} \qquad 3.5$$

where

L is annual suspended sediment load (tons/yr)

Ac is the catchment area (sq miles)

and

$$Lm=111.9Q_m^{1.0207} \qquad 3.6$$

$$Lc=2523Q_m^{0.8227} \qquad 3.7$$

$$Ls=1926Q_m^{0.6523} \qquad 3.8$$

$$Ld=3773Q_m^{0.7158} \qquad 3.9$$

where

Lm, Lc, Ls, and Ld are suspended loads for mixed broad leaf vegetation, coniferous and tall grass vegetation, short grass and sands, and desert sands respectively in (tons/hr). Qm is the mean annual discharge in c.f.s. The table 3.1 shows the results obtained by the use of the relationships as compared to other methods.

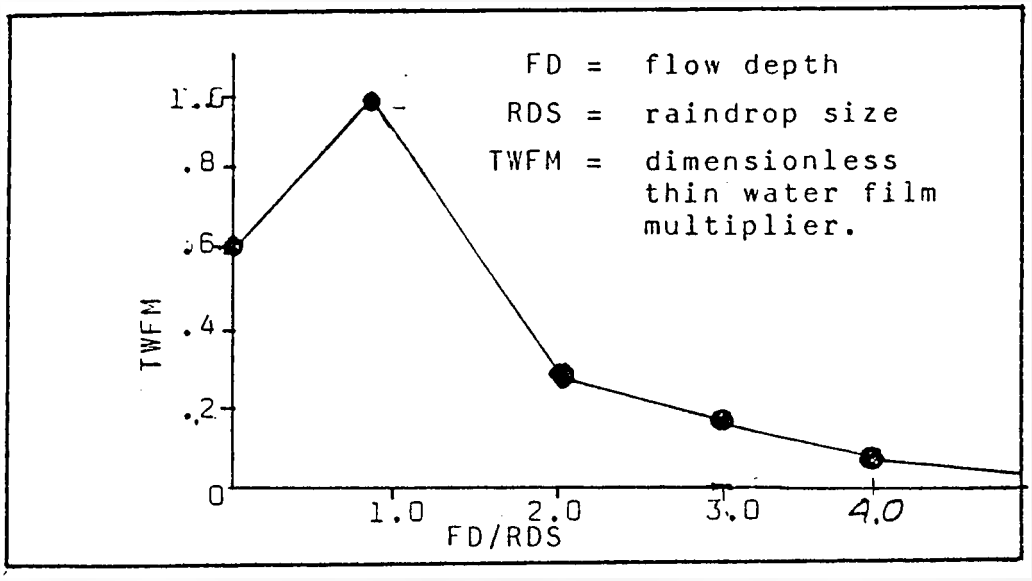


Fig. 3.2 DETACHMENT CAPACITY OF RAINDROPS ON A FILM OF WATER (AFTER PALMER, 1963)
(See fig 3.1 for reference

3.5 Sedimentation in Reservoirs

3.5.1 General

Sedimentation in channels, reservoirs, and river outlets such as harbours constitute a major part of the sediment problem. Sediment accretion is common in the lower

courses of rivers and creates obstacles for navigation with attendant economic losses. Sedimentation also increases water surface elevations in both reservoirs and rivers and this induces and amplifies floods. The deposition of infertile sediments damages crops and lessens the output of normally fertile flood plains. Eroded materials fill natural and artificial channels, waterways and harbours and this necessitates their removal by very expensive methods.

Sedimentation in reservoirs greatly reduces their useful capacities with time thus reducing water outputs for power, irrigation, urban, and industrial use. Sediment deposits also hinders the function of irrigation and drainage ditches and ruins recreational facilities. in rivers and reservoirs.

Most reservoirs have sediment pools (dead storages) filling rapidly while others remain stable and maintain adequate dead storage capacity for over a resonable period of time. The latter is characteristic of catchment with very little erosion. The lack of information on the capacity available in existing sediment pools, makes it impossible to differentiate exactly between the amount of sediments settling in the live and dead storages, though an estimate can be attempted. This dearth of vital data creates problems when estimating damages due to sedimentation (Crowder, 1987).

Reservoir sedimentation is severe in many parts of the World, especially, in areas experiencing severe erosion

and it consequently affects their ultimate lifespans and their economic feasibility (Power, 1988).

3.5.2 Assessment and Management

Research on reservoir sedimentation in recent years has been directed mainly at water resource projects in developing countries especially in Africa and Asia, which often have to cope with long droughts, flash floods, and severe erosion problems (Anandale, 1987). Large reservoir capacities are required to capture water and sediments provided by floods and flash floods so as to ensure the supply of water during droughts. These floods, due to their tremendous stream power, carry enormous volumes of sediments which, due to reservoir sizes and shapes are deposited in the reservoir bowl, leading to fast deterioration of such an expensive investment.

The vulnerability of the economic life of a water resource storage project to sedimentation emanates from the nonlinear relationship between water yield and reservoir capacity. Severe reductions in reservoir water yields can result from relatively small reductions in reservoir volumes. A conflict, therefore ,exists in areas where erosion is a problem. The balance between decreasing reservoir yield and increasing demand for water with time must be accurately predicted during the design and planning stages. Misjudgement can definately result in an unnecessary financial embarassment and possibly disaster for the client and communities concerned.

The influence of sediment deposition on reservoir design affects engineering and economic decisions regarding the placement of intakes and sluices in dam walls and the estimation of extra pressures due to deposited sediments on the dam body and intake towers. These decisions certainly depend on where the sediments, no matter how large, are deposited. The shape of the reservoir basin and the sediment transport characteristics may be such that most of the sediments are deposited in the upstream reaches, sometimes not even reaching the dam wall. In some cases, large amounts of sediments are deposited by the wall or on sluices, leading to forces extra to normal design hydrostatic pressures. In optimal reservoir design, there is the urgent need, therefore,for the use of accurate quantitative and distributive forecasts techniques for deposited sediments (Anandale, 1987).

Mordern computer simulation models tend to be more process based, nowadays, and thus provide solutions indicating fairly accurate prediction of sedimentation (Chang and Hill, 1977; Fleming and Walker, 1979). One such model, is the delta simulation programme, developed at Stanford university for the deposition of sediments at river mouths. It was originally intended for rivers discharging into the ocean but now modified in this work to fit reservoir basins (Bonham-Carter and Sutherland, 1968: Anandale, 1987).

3.5.3 Examples and Economic Costs

The ratio of usable to gross capacity of reservoirs varies between 38 and 59 percent depending on location with a global average of around 50 percent, according to Power(1987), citing Mahmood (1987). This usable volume is continuosly diminishing due to siltation. The rates varies widely depending on terrain, geomophorlogy, geology, land use, cimatic factors, etc. The Hoover dam, on the Colorado, loses its live storage by 0.3 percent annually (figure 1.7); the Tarbela reservoir in Parkistan diminishes in storage by 1.5 percent a year; the useful storage of the Sanmexia reservoir is lost by 1.7 percent annually; and worst of all, the Warsak dam reservoir in Pakistan lost 18 percent of its gross storage in its first operational year (Power, 1988).

In estimating the economic costs of reservoir sedimentation, Clark et al(1985) estimated current U.S costs of damages to in terms of current expenses. This ranged from 300 million to 1.6 billion U.S dollars, or a single value of $690 million per year on a 1980 basis, about 11 percent of the nations damages. Clark et al utilized four different estimation criteria namely dredging which is used mainly for small reservoirs costs about 550m$; replacement costs of lost capacity-$733m; construction of new sediment traps or pools costs $300m to $700m annually; and sediment outlets provision (used with little success with unknown estimates due to unavailable costs data). Crowder (1988), utilizing the replacement criteria and greater water storage capacity

obtained $819m as losses to sedimentation. Their results reveal that reservoir sedimentation costs to national economies are extraordinary when cumulated as above.

The global annual average of reservoir gross storage losses is estimated at 1 percent or 50 cubic km. In 1986, the estimated total usable capacity loss was around 540 sq km. Replacement of this loss will cost millions of dollars.

3.6 Managing Reservoir Sedimentation

The control of reservoir sedimentation can only be effectively achieved by controlling the rate and amount of sediment inflow to the impoundment. Methods involving physical removal of the sediment are difficult and expensive. Attempts have been made to facilitate sediment removal by the inclusion of drawoff towers or gates to allow the discharge of density currents carrying high sediment concentrations and the flushing out of deposits. Dredging and other diversion methods have been tried with limited success, as these control operations upset the daily routine of power generation, water supply, flood control and recreation (Fleming, 1977)).

Reservoirs, therefore, are designed and operated in close conjuction with overall watershed management. In river basin management, and indeed,in all water resources management policy, attention must be paid to and money invested in sedimentation problems as this is economically beneficial in the long term. Measures, normally, taken to

minimise the silting problem in reservoirs include:

(1) careful selection of reservoir sites.

(2) designs to facilitate sediment removal.

(3) control of sediment inflow by erosion control and adequate watershed management.

(4) control of deposition by the use of sediment traps, diversions, etc.

(5) increasing the size of the reservoir to cater for losses to evaporation, ground and especially to sedimentation in live storages.

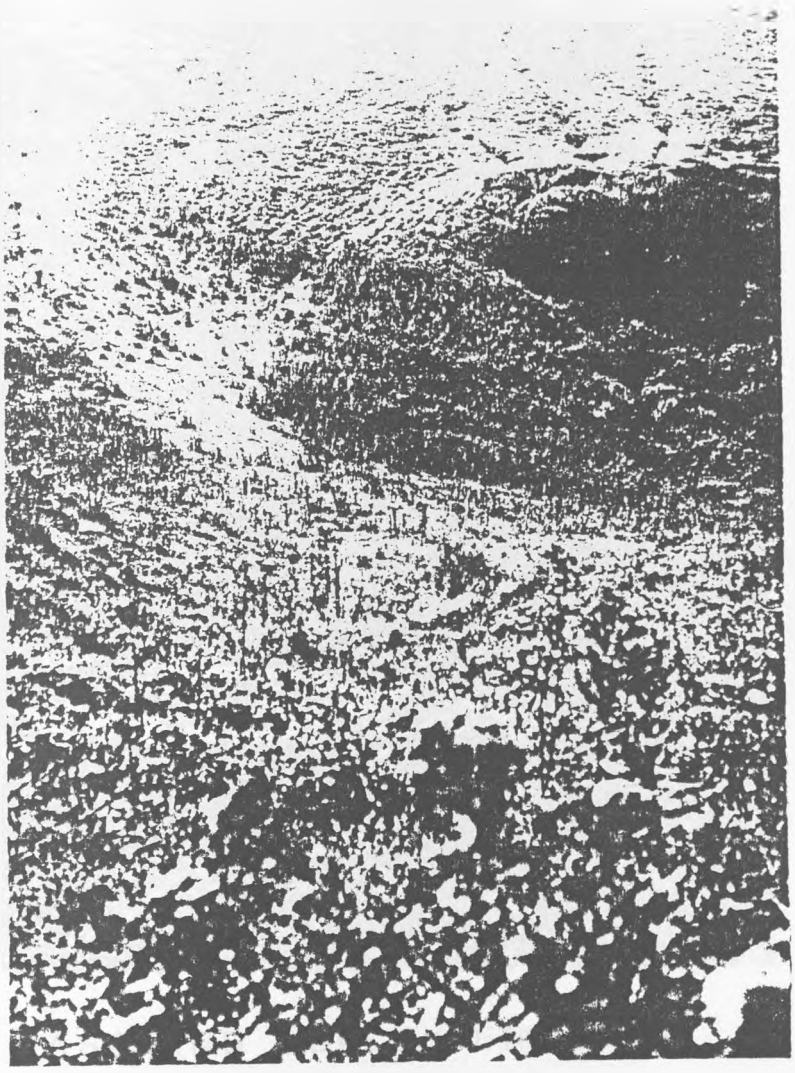
(6 minimising the dead storage capacity by specifying an optimal minimum operating level in both design and operation of reservoirs to increase live storage capacity and yields, especially during periods of hydrological stress, such as during severe floods and droughts was been proposed in Chapter One.

3.7 Final Remarks

The sediment problem existing both in the river basin-reservoir system is basically a nuisance to managers and planners. The interacting water and sediment processes are extremely complicated. For a realistic prediction and assessment of sediment behaviour and its socioeconomic impact, comprehensive techniques are required. The mathematical model approach presents a way of tackling the problem posed by this complexity.

The lack of adequate knowledge of the intricacies involved has led to the failures of past water resources projects subject to sedimentation. The current improvement in the understanding of the processes have led to the adoption of the process modelling approach of this research. The mathematical models and modelling techniques reviewed in chapter 4 should help provide more information on behaviour of the water-sediment interaction in river basins.

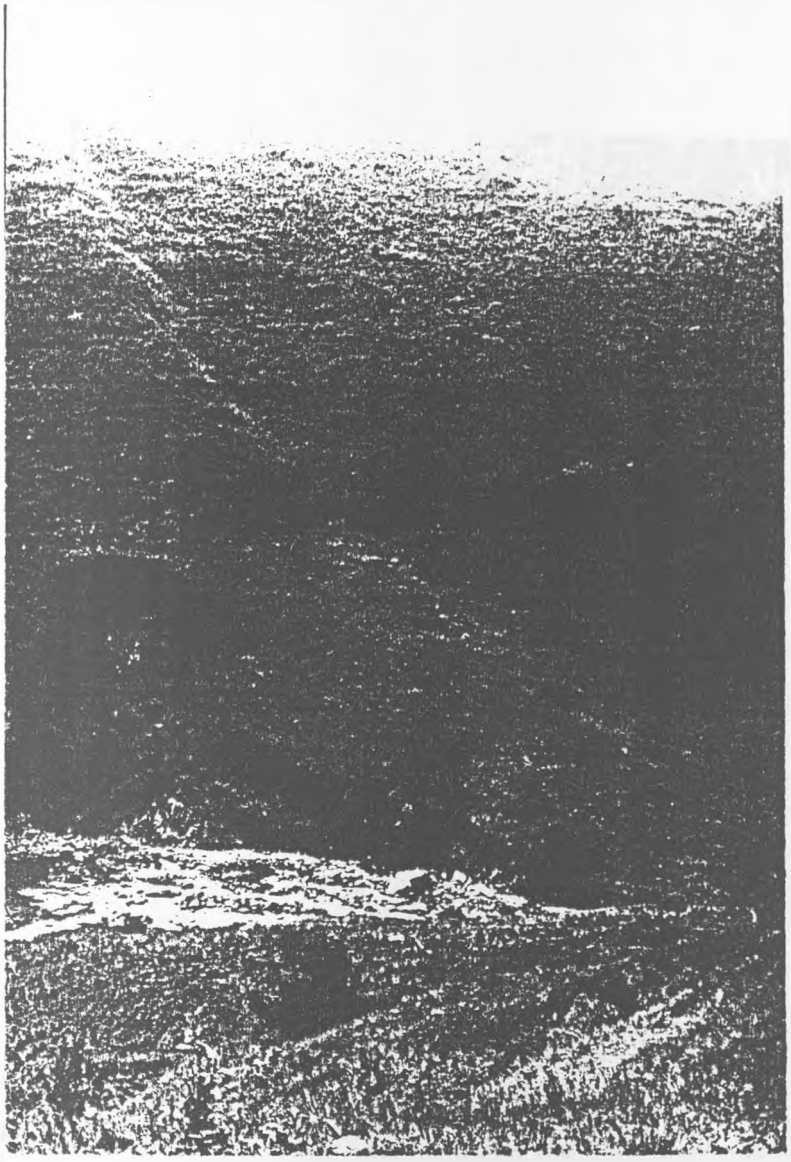
PLATE A



RILL AND SHEET EROSION

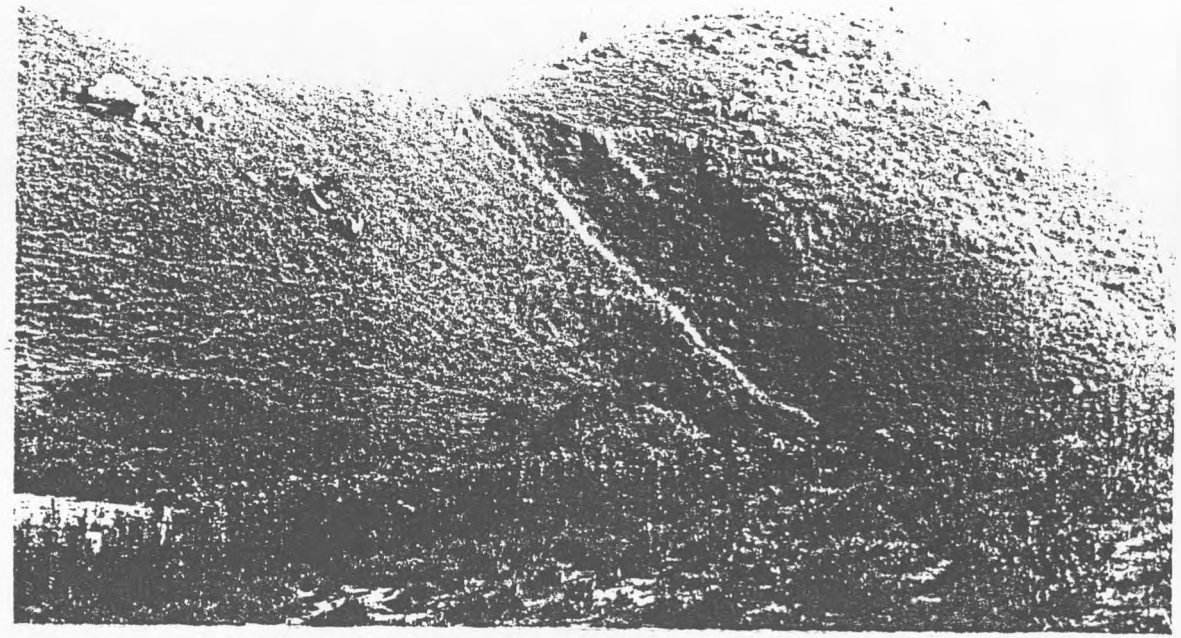
DONGA BASIN NIGERIA

PLATE B



MASS AND GULLEY EROSION
DONGA BASIN NIGERIA

PLATE C



SHEET AND RILL EROSION DONGA BASIN NIGERIA

CHAPTER 4

CHAPTER 4: MATHEMATICAL MODELLING IN RIVER BASIN MANAGEMENT

In Chapter 1, a process approach to river basin management using mathematical modelling and computer simulation techniques was proposed. In Chapters 2 and 3, the need for such an approach was highlighted after a description of the state of the art coupled with the problems commonly encountered in river basin management practice. The main objective of this chapter, however, is to review and analyse mathematical models and modelling techniques in relation to the interaction of hydrology and sediment processes in river basins and their impact on the economic performance of reservoirs.

A brief discussion of the modelling concept and a general classification of mathematical models is provided in sections 4.1 and 4.2 respectively. Section 4.3 describes the computer simulation concept and techniques as applied to river basin management. Section 4.4 reviews river basin hydrology models with emphasis on the modelling of reservoirs. A brief review of catchment sediment models is undertaken in section 4.5. Section 4.6 briefly introduces the Strathclyde River Basin Model. The River Basin Management System, the main tool and focus of this research, is briefly introduced as part of the final remarks provided in section 4.7. Firstly, the mathematical modelling and computer simulation concepts are now briefly outlined.

4.1 Introduction-The Mathematical Modelling Technique.

Modern science and technology has undergone a quiet revolution since the advent of the digital computer in the last few decades. These machines operate and solve mathematical problems at unimaginable speeds. This computational potential has continued to improve with time. This development has led to an interest in the mathematical modelling concept which basically is a mathematical representation of physical problems billed for comprehensive solutions. It is evident from the literature that much work has been undertaken in the application of mathematical modelling techniques to river basin management but much of it is unsuitable for the approach adopted herein and no single model is available to simulate complete river basin processes vis-a-vis the economics involved in the desired manner. Thus, this research has the aim of covering this gap by setting up a new, practical, water management modelling system. An important principle of the new system is that it must be functional on data and parameters which are readily measurable or estimatable. This implies that the model can be easily applied to real basins.

The mathematical modelling technique utilised is that commonly used in practice and involves the following basic steps namely:

1. An examination of the physical problem
2. Replacement of the physical problem by an equivalent mathematical problem

3. Solution of the mathematical problem with accepted techniques of mathematics.

4. Interpreting the mathematical results in terms of the physical problem.

This mathematical modelling technique has been applied to solve many water resources problems and a considerable body of knowledge exists in the management of hydroelectric power generation; water supply; sanitation (Hydrocomp, 1976); irrigation (FAO, 1986); inland waterways navigation; flow and flood forecasting (O'Connel et al. 1986) etc. Prior to a review of river basin models and the discussion of the simulation approach adopted in this research, a brief classification of mathematical models is next undertaken.

4.2 Classification of Mathematical Models.

Mathematical models can be best appreciated by classification and this is can be done by means of the mathematical methods involved in their formulation or solution. Figure 4.1 is a comprehensive representation of the mathematical methods and models commonly used in hydrology. Mathematical models are mainly classified as either, empirical or conceptual; deterministic, parametric, or stochastic; static or dynamic; and optimisation or simulation models. These major model classes will be defined briefly and discussed with regard to their use in river basin management. A detailed description is given in the literature (Fleming, 1975;

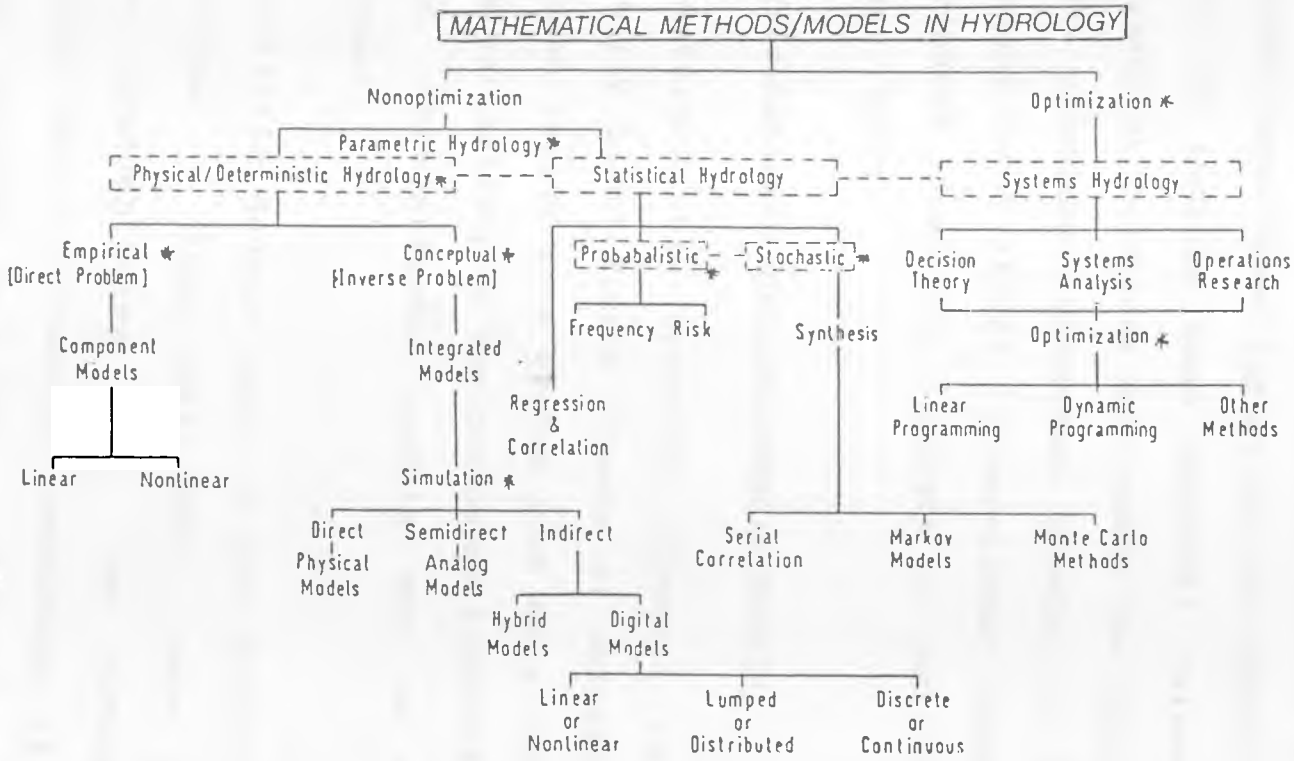


Fig.4.1 MATHEMATICAL METHODS IN HYDROLOGY

Source Fleming 1975

Hall, 1970).

(a) Empirical and Conceptual Models

Empirical models treat processes by the use of over-simplified mathematical functions, while conceptual models utilises established theory on the component processes in order to include some physical relevance into the mathematical functions and assess the integrated response of the processes. Conceptual models may treat component processes in a linear or non-linear fashion, lumped or distributed (spatial), discrete or continous (temporal). (Walker, 1979).

(b) Deterministic and Parametric Models.

The deterministic approach attempts to predict the physically based direct effects of exterior or interior processes acting in a system such as a river basin. The random variability of the processes involved are excluded, though. Deterministic models may be empirical or conceptual.

Deterministic Models such as the Stanford Model and Strathclyde River Basin Model, have a distributed System representation and can simulate internal processes such as transport mechanism of run-off and sediment to outlet from source through the overland and channel systems.

Parametric models are like deterministic models in some ways. For example, they tend to formulate the behaviour

of the processes in a direct physically based manner. The parametric approach in hydrology is generally defined as the development and analysis of relationships among physical parameters involved in hydrologic events. Parametric Modelling is less rigorously developed than the deterministic case and generally simpler in approach. Model parameters, here, are not necessarily defined as measurable physical entities although they are generally rational. Parameters may be determined by fitting the model to hydrological data using an optimisation technique.

Many Parametric Models are statistical or probabilistic in nature and these abound in the literature and are extensively used for practice. The Log Pearson Types 1, 11, 111, 1V, Log-normal, Gumbel, and Normal are examples of such models. They are used for the determination of extreme values for flood events, storm rainfall, flood volumes, low flows, peak river discharges required for design of the structures and other facilities utilised in river basin management.(Fleming, 1975)

This probabilistic approach assumes that the chance of an event occuring follows a fixed probability distribution. They are used in studying available data in order to attach a frequency to the size of a flood event, for example. The probability distribution is the plot of the probability of an event against the magnitude of the event. There are many types of distributions in use and they include the binomial; poisson; normal; gamma;

pearson; extrema 1, 11, 111; and the log normal.

Fleming (1975) stated that these distributions are an approximation of the actual relationship between data. This is seen from the observed scatter in the typical plots of a flood frequency data that uses the Gumbel (extrema type 1) distribution. This is shown in figure 4.2. The non linear shape of the curve is typical of the results obtained when these probabilistic methods are applied to most watersheds. This nonlinearity results from the assumption that the sample data is time-independent. In short, the probability distribution is not stationary as assumed but variable.

The problem of assessing extreme flow values by probability distribution is complicated further by the changing global climatic environment. Historical flood or drought events are unlikely to be representative of the future conditions and estimates from flood frequency analysis will be unrepresentative. In assessing design flows in River Basin Management, the need for a thorough appraisal and modification of historical flow records to incorporate the changing global climate cannot be overemphasized. This modification will assist in obtaining more realistic figures for the long term design and operation of water resources projects. Chapter Five provides the magnitude of expected changes in climate in different parts of the World.

after Fleming, 1975

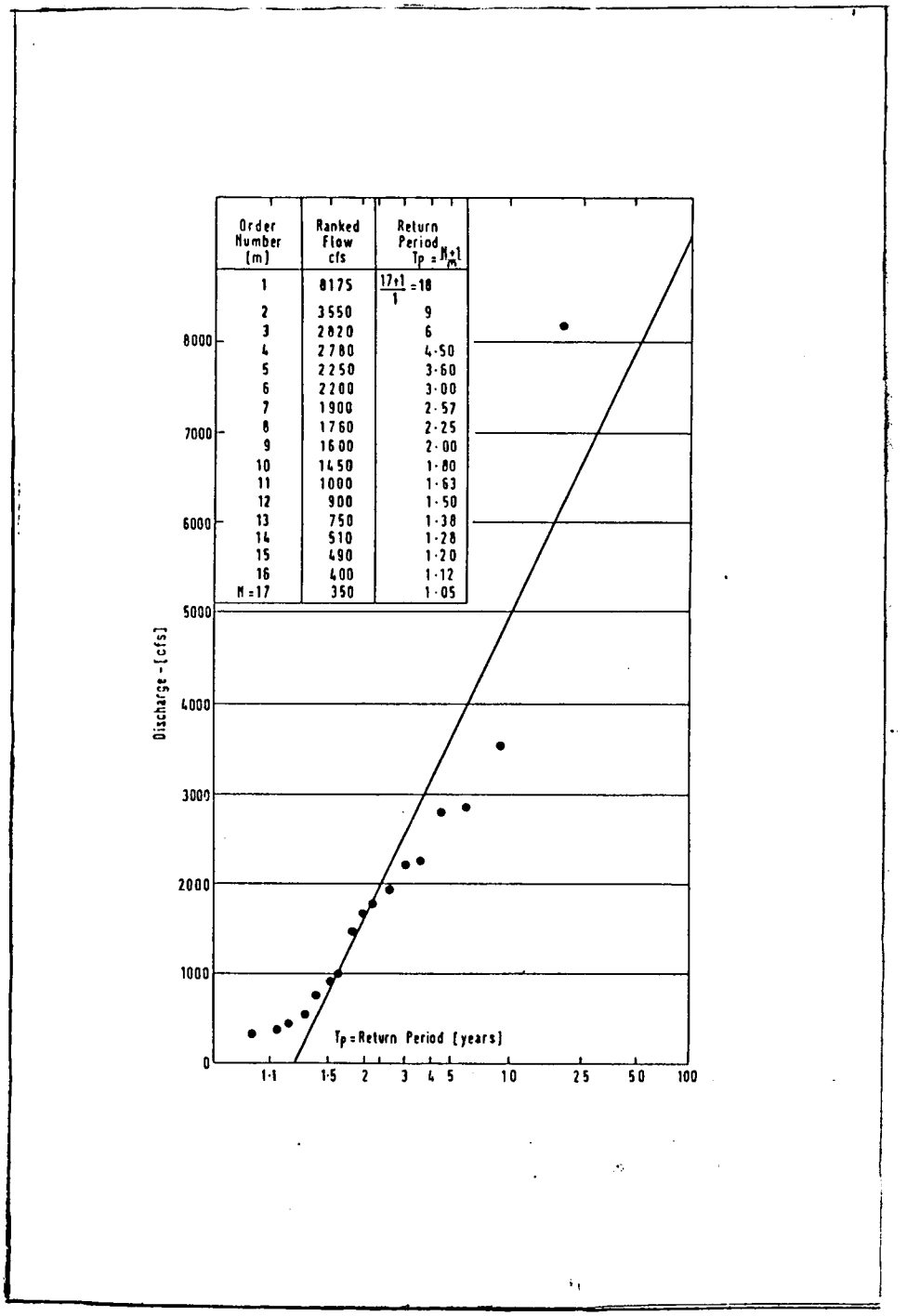


FIG 4.2 A TYPICAL FLOOD FREQUENCY CURVE

(c) Comparing Deterministic and Parametric Models

Deterministic models are verified by checking against real watershed data during application. The two modelling approaches are somewhat similar. Real differences lie only in the number of co-efficients or parameters involved which may be much more for the deterministic case. Rigorous verification is required for deterministic models to preclude suspicion which contrasts with Parametric modelling as parameters and coefficients are pre-determined by optimisation (Hall, 1970).

In deterministic modelling, data must be available at the point of application. In Parametric Modelling, data parameters can be obtained using data anywhere in the watershed, thereafter these are improved upon by correlation and other techniques using the physical characteristics of the catchment as a whole. This regionalisation makes data predictions for ungauged areas possible (Kirkby, 1978).

It is good engineering practice to use both forms of modelling on a complimentary basis. The TVA and the USGS have both developed many parametric regional models which are essentially lumped system representations and they have the capability of assessing the gross effects of land use on run-off but lack the capability of assessing the sensitivity of internal distributions of land use on run-off.

(d) Stochastic Models

Stochastic cases tend to monitor the random effects and statistically relate model input to output. Stochastic Models have variables that are time dependent and the input and outputs have distributions in probability. Clarke (1973) and Yevjevich (1970) both discuss these models in detail. The output in most cases is in form of a time-series and the parameters which are commonly used to characterise the series are the mean, variance, the auto-correlation function, the auto-covariance. These describe the time dependency and other features.

(e) Static and Dynamic Models

Static and Dynamic Models are further classifications of both simulation and optimisation models. The former does not involve significant time dependent changes. For example, while actual or predicted streamflows vary randomly on a daily, monthly or annual basis, the probability distributions of the flows at any time of the year is assummed static from one year to the next. In dynamic models, this assumption does not hold. Dynamic models allow the study of a system for a wide variety of physical configurations and over short and long time horizons depending on the time step used in their formulation.

(f) Optimisation Models

Optimisation models look implicitly at all possible

decision alternatives (Yeh, 1985). Biswas (1976) and Hall (1970) indicate that optimisation models cannot determine the exact solution to Water Resources Management problems because of several limitations namely, mathematical, computational, and criterial specification. Other limitations are hydrological, technological, and economic uncertainties, inaccurate and incomplete data, the complexities posed by the presence of a large number of parameters and constraints, etc. Some of these have already been mentioned in Chapter Two. These limitations restrict the use of optimisation models to preliminary screening where necessary. The alternatives that survive preliminary screening are further analysed, evaluated and improved by simulation techniques.

Optimisation methods in mathematical modelling include such techniques as systems analysis (used for making choices among competing alternative projects or development strategies), operations research and decision theory. These techniques may use the path of linear, non-linear, or dynamic programming to optimise solutions (Fleming, 1975). Systems analysis, which is more commonly utilised in Water Resources Management, especially in the area of the optimisation of reservoir operations, is now briefly described.

Biswas (1976) defined systems analysis as an analytical study or method which helps a decision maker to identify and select a preferred course of action among several feasible alternatives.Systems analysis, according to Hall(

1970), is undertaken in order to make rational decisions concerning the optimal design, selection and operation of a physical system. It consists of five phases, namely,

(1) The understanding and translating into quantitative terms, the objective and performance requirements sought for the system in relation to the environment in which it operates.

(2) Formulating quantitatively the structure and boundaries of the system by means of a descriptive flow chart.

(3) Constructing the mathematical model for the system that incorporates all the interelationships between quantifiable variables. All the quantifiable constraints and the functions yielding the input and output must be included.

(4) The coefficients of the model must be estimated and the desired input relations specified.

(5) The model should be validated in the light of the objectives established.

Fig 4.3 shows the cyclic nature of these variables and the systems approach as applied to water resources planning (Hall, 1970).

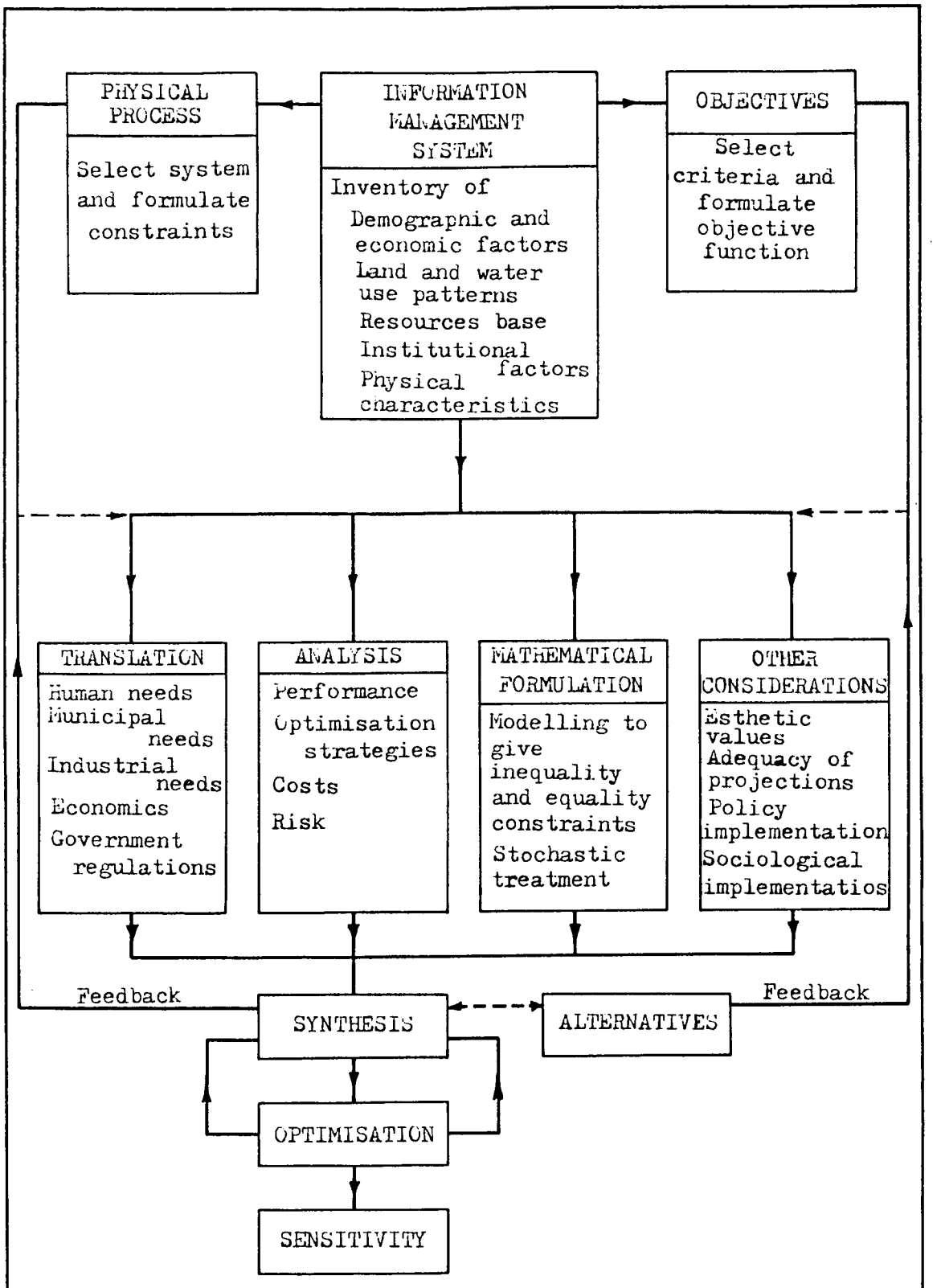


Fig. 4.3 SYSTEMS APPROACH TO WATER RESOURCES PLANNING

Source Hall, 1970

(g) Simulation Models

Simulation is commonly used in River Basin Management due to its simplicity in representing, assessing, and illuminating the interactions of the complex processes involved. Yeh (1985) states that simulation models are more flexible in simulating the response of a system than optimisation models. In simulation, an optimal solution may be approached by the use of a trial and error approach in the choice of parameters (Fleming, 1975) and in analysing alternatives. The simulation approach is adopted throughout this work and the general proceedure is illustrated in figures4.4. Simulation models as classified in practice are described below.

Biswas (1976) stated that river basin simulation models as consisting of a sequence of mathematical and logical statements describing the design and operation of a river basin system. This provides the means of simulating the operation of that system in order to predict and analyse its performance. Biswas indicated that such simulation models have proven to be effective tools for estimating the future hydrological and economic performance and for evaluating alternative configurations of reservoir and hydropower plant capacities, water use allocation targets, operation policies, etc. He contended, however, that simulation models may not be the best tools for choosing or defining the best combination of capacities, targets or operation policies. On the other hand, optimisation models are effective in approaching optimal solutions in such

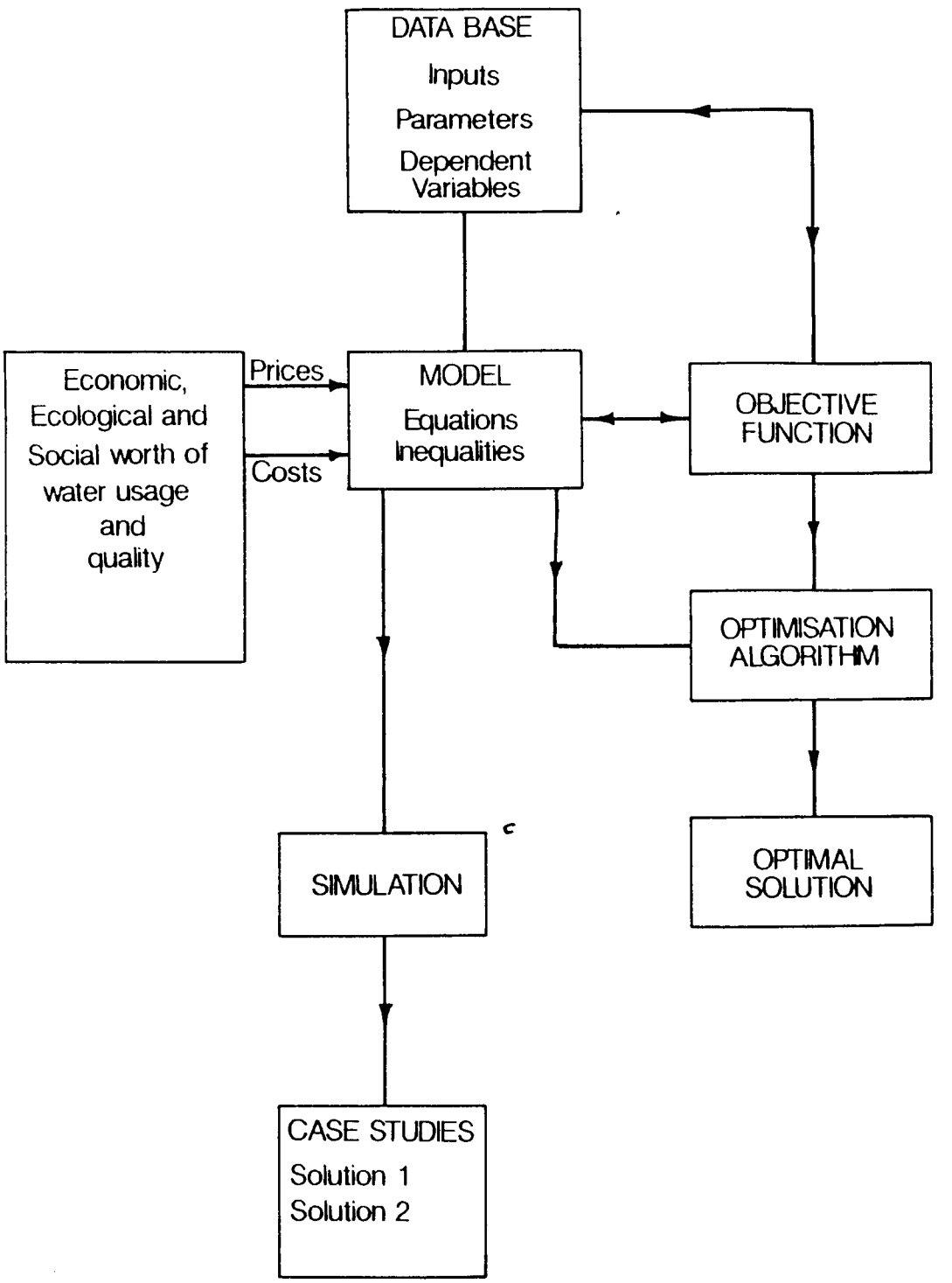


Fig.4.4 PLANNING VIA SIMULATION VERSUS OPTIMISATION

Source WRSE, 1970

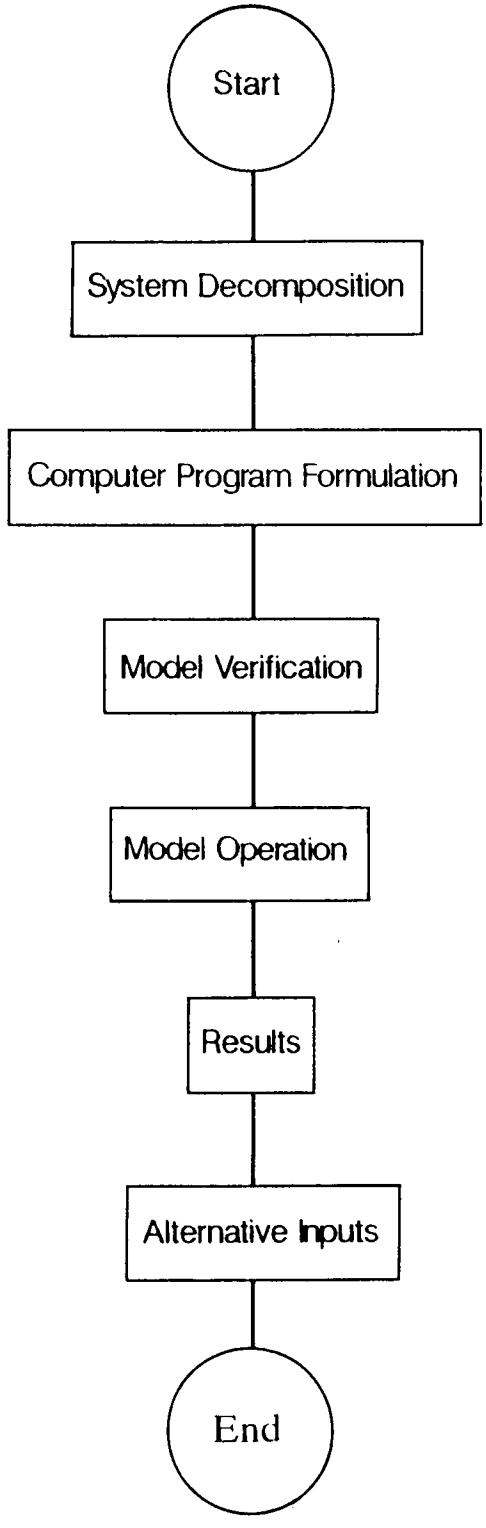


Fig.4.4(b) GENERAL SIMULATION PROCEDURE

Source WRCE,Hall,1970

circumstances. As this work confines itself to the use of deterministic simulation models, optimisation models are not discussed further. The figure 4.4 compares modelling by simulation and optimisation. From the figure, simulation models sometimes use an optimisation algorithm to obtain optimal solutions. A description of the basic methodology adopted in computer simulation and a review of river basin simulation models are provided in sections 4.3 and 4.4 respectively.

4.3 Computer Simulation.

4.3.1 Definition

The concept of computer simulation involves three very basic procedures. These are the definition of a system, the construction of a model and the use of the model to deduce the behaviour of the system under various practical and natural conditions.

A system may be defined as a set of dynamically interrelated components. If any part of the system is tampered with or changes, the repercussions are expectedly felt throughout or in parts of the system. Examples of the Systems include the entire universe, the solar system, the earth, the atmosphere and a river drainage system, amongst a host of others. The last few are lesser systems compared to the whole universe. A gross or mega system is considered to include a union of many smaller ones such as seas, beaches, river basins, deltas etc. The boundaries of a system are

very critical in its definition.

Computer simulation is the application of an existing or newly constructed model to predict the situation in a system, backward or forward in time and space. Input data are fed into the model and a corresponding output is

obtained. The effect of any change in any of the inputed physical process parameters on the entire or parts of the system are observed. This provides a better understanding of the behaviour of the physical processes within the system than would otherwise be possible. This is the heuristic advantage of simulation.

4.3.2 Methodology.

The Computer Simulation technique is especially valuable where there does not exist a complex knowledge of the physical processes in the mathematical terms. The real life system then acts as a guide for comparison with the model results. This comparison helps in calibrating the model for further use for the system under study.

Figure 4.5 shows steps in the model calibration and Figure 4.6 shows general steps in model usage inclusive of data requirements. Usually, there are disturbances or errors in the input supplied to the model such as missing or made up data or inconsistencies. The calculated output is compared with the measured output of the physical system which also has inherent disturbances due to

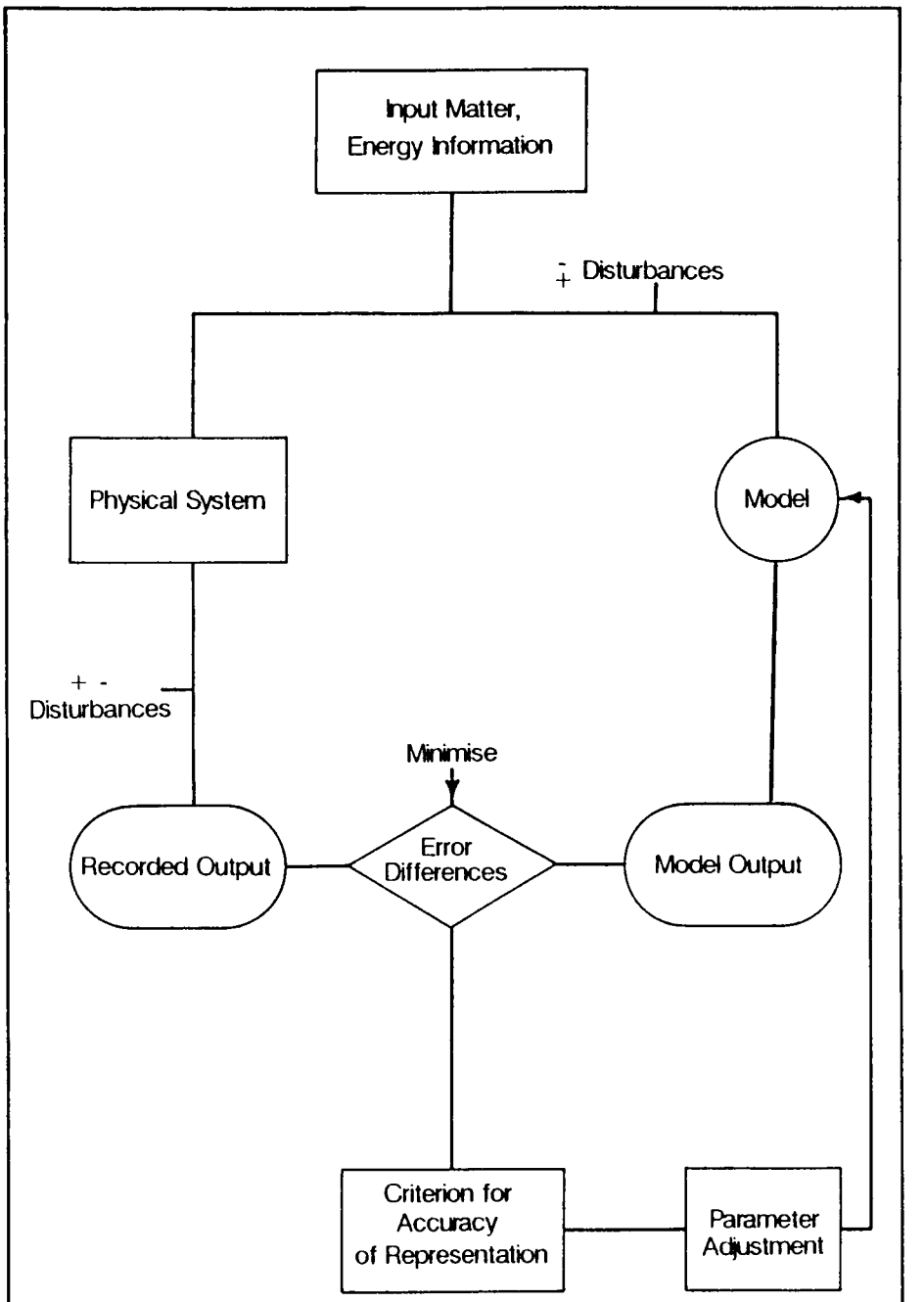


Fig4.5 THE MATHEMATICAL MODEL CONCEPT
(and the calibration loop)

Source Fleming, 1975

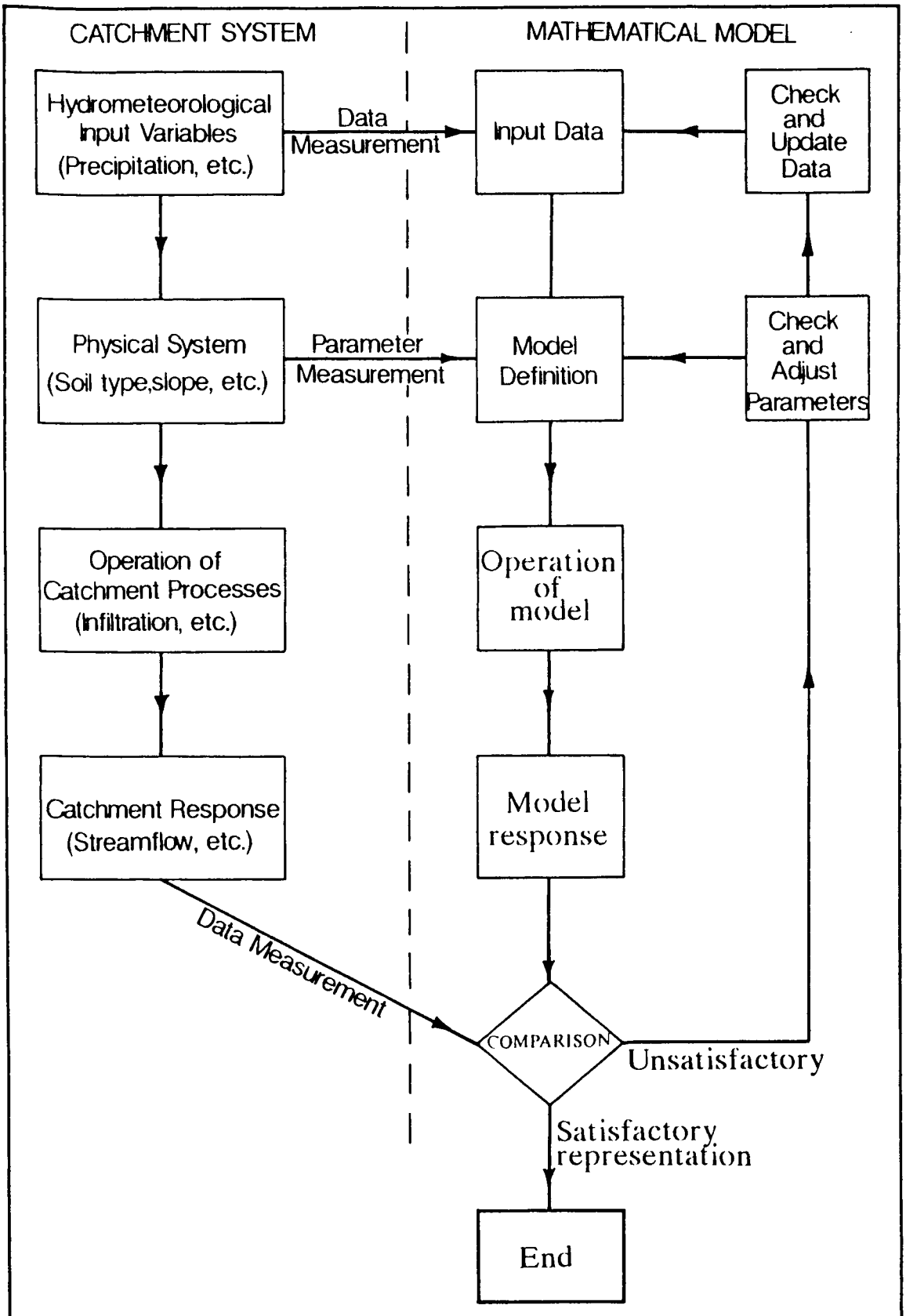


Fig.4.6 DATA REQUIREMENTS IN MATHEMATICAL MODELLING

After Hall 1970

measurement errors and the like. If upon comparison, no agreement of the simulated and recorded values occurs according to the selected accuracy criterion, then the input or process parameters are adjusted and the procedure is repeated until acceptance is found. Calibration is normally done by experience though many models have the ability for automatic calibration. Prediction and other applications follow after an appropriate calibration condition has been established (Fleming, 1975).

In general, simulation carries out 'experiments' on a model of a defined system to obtain data that can be evaluated to provide the best operating and design policies (Ackoff, 1961; Hilher and Liberman, 1967; Hufschmidt and Fiering, 1966).

4.3.3 Limitations

Computer simulation, like its alternatives of mathematical or systems analysis (optimisation), experimentation, and the reliance on experience and intuition has limitations, which depend on the specific problem under consideration. The first limitation is usually that of the validity. Many problems have a unique optimal solution which may be impossible to pinpoint exactly by simulation though this global optimum could be approached as much as possible. This problem has been generally by-passed by introducing the concept of local optimum which is determinable and is near the global as practically as

possible (Fleming, 1975; Hufschmidt, 1967).

Another limitation of simulation concerns changing the operation procedure of the system. A unique computer programme for the system must be written and verified for each particular set of operation rules.

Therefore, changing operation rules is expensive and time wasting. Other limitations are usually unique to the type of problem under consideration.

4.3.4 Applications

Basically, simulation models consist of five major components: State Variables, Exogenous Variables, Functional Relations, inputs and outputs. State Variables describe the state of activity within the physical system to be reproduced. These may be used to impose restraints or constraints on events generated by the model during the period. Exogenous Variables describe conditions or behaviours external to the area. Water Resources legalities or institutional constraints to control flows or aquifer recharge or usage are exogenous. Functional relations describe interrelations between internal components of the model and this assists in predicting the effects of exogenous factors acting on State Variables. The models output describes the activity state at the end of the simulation period.

Computer Simulation techniques are now widely used in a variety of disciplines in the physical and social sciences

for the study of complicated systems with variable time states. They have been used in analysing business systems, testing alternative production schedules, studying product flows in physical distribution systems, modelling the operation of airports and harbours and similar problems. The system operations and the efficient

planning of river basins and water resource projects in general can be achieved by its use.Here, the simulation approach may be more cost effective than the mathematical optimisation techniques involving dynamic, linear or nonlinear programming because of the presence of a large number of alternatives (Hufschmidt and Fiering, 1968). A trial and error simulation approach involving parameter changes can be very effective when combined with experience. In addition, the monitoring of the parameters and a better understanding of the processes involved, their sensitivities and overall effects can be clearly and heuristically obtained with the many stages of simulation (Fleming, 1975).

Biswas (1976) stated that " simulation models...have proven to be very effective tools for estimating the future hydrological and economic performance of any proposed surface-water-management policy".They have also been effective in evaluating alternative configurations of reservoir and hydroelectric power plant capacities, water-use allocation targets, operating policies and the like, but limited in locating the best combinations of these. However, simulation has been used in solving complex water

resources problems. These include the Nile valley irrigation plan (Morrice and Allan, 1959), and the Sydney, Australia water supply (1969) and in energy studies on the Columbia river (Lewis and Shoemaker, 1962). A review of the models utilised in addressing these complex river basin water problems are next described.

4.4 River Basin Hydrology Models.

4.4.1 River Basin Processes and Modelling

River Basins are large areas consisting of a series of hillslopes. Many hillslope models which incorporates the complex hillslope processes have been developed over the years (Kirkby, 1978). Kirkby (1978) stated that it is possible to model basin hydrology by integrating a hillslope model over an entire watershed but this is an arduous task, which is further complicated by the need to account for the spatial and temporal variability in soil-water interaction.

The soil-water processes of the river basin clearly interact in its land, river, reservoir and subsurface phases. The land phase consists of water which infiltrates into the soil or flows off as surface runoff, also called overland or sheet flow. Here, the processes of sediment erosion and yield, the surface and subsurface water dichothomy, and the entrainment of biological life forms and chemical particles by flows are represented. The river phase represents the process interaction of channels and tributaries which includes channel scour, sediment movement and deposition, water flow through the river

(routing), and the variations of the physical, biological and chemical processes. In the reservoir phase the interacting processes are sediment deposition; water inflows, outflows, circulation and storage changes; thermal stratification and density currents; and changes in the chemistry and biological processes of the

impoundment. The subsurface zone represents processes relating to water inflow and outflow, flow within the zone, biology, and the chemistry of the underground water.

Basin Models are sometimes modelled along different lines especially as under many conditions little input data exist across the hillslopes or the entire basin (e.g one or no rain or flow gauge may exist in the whole catchment). Under these circumstances, developing or utilizing a model detailing the complicated hillslope or basin processes is clearly time wasting and uneconomical. Further, the outputs of large hydrologic systems tend to be dampened relative to the input.

While extreme detail of a hillslope model may not be required or desired in a basin model, it may be necessary to use empirical relationships, algorithms, approximations and assumptions that are compatible with the processes and principles of hillslope hydrology. The incorporation of these validates the models during applications and facilitates their extrapolation to ungauged conditions thus, broadening their use.

In tackling the reservoir management problem, where the internal mechanisms of the hydrological cycle and

sediment processes are involved and interwoven, this white box approach of complete process representation , fully illuminative, is preferred to the black-box technique widely used for engineering design. The problem is inherently time dependent. Thus, steady state analysis has little value here. An integrated time dependent model covering the whole river basin will be necessary in dealing with the problem effectively.

A typical water resources system simulation model basically simulates the interval by interval operation of the system with defined characteristics, operating rules and specified inflows at all locations during each interval. The system may be a river basin, a reservoir, etc, depending on its defined limits.

Most simulation models assume that a particular sequence or combination of sequences of historical streamflows is representative of the entire hydrologic time series. This assumption is not strictly true due to the stochastic nature of hydrological events. Thomas and Fiering (1962) suggested the use of steamflow simulation to enhance model reliability. Burges (1979) reinforces this point. Yeh (1985) confirms this by stating that

"the philosophy behind using a synthetically generated streamflow is that if many sequences of flows are used and each is statistically likely to occur and has a length equal to the statistical period of interest, it is likely that an entire range of plausible future scenarios would have been explored".

Hufschmidt and Fiering (1965, 1966) used simulation in planning the multireservoir and multipurpose Lehigh River System. In their study of 35 different configurations, three different designs with higher benefits than the existing system were identified. Synthetic streamflow sequences obtained from a first-order linear Markov chain model were used as input to the system. Critical period hydrology has also been successfully used for design purposes in a reservoir simulation study (Askew et al, 1971).

In practical terms, though very many river basin models are available, the decision on the type and sophistication of the model required for tackling a particular problem is a major one. It depends on the complexity of the problem and the accuracy criteria required for the solution. Before addressing this problem of choice, a review of some existing river basin simulation models is first carried out.

4.4.2 Review of Existing Models and Modelling Methodologies.

From the literature, one of the earliest simulation models, associated with reservoirs in a river basin system, were used by the U.S Army Corps of Engineers in 1953 for an operation study of six reservoirs on the Missouri river (Hall and Dracup, 1970; Dracup, 1970). A similar study was performed on the Nile valley for problems concerning the size of the reservoirs and their

operations (Emergy and Meek, 1960; Yeh, 1985). In a similar model study, Maass et al (1962) demonstrated the application of simulation methods to assess the economic performance of a river basin (Yeh, 1985).

Yeh (1985) gives examples of recent development and applications of simulation models. These include :

(1) The HEC-3 model developed by the Hydrological Engineering Centre (1971)

(2) The SIM-1 and SIM-11 developed for the Texas Water System (Evanson and Mosely, 1970)

(3) The simulation of the Oswego Basin of New York using a modified version of the HEC-3 model (Liu et al, 1972).

(4) The Upper Wabash studies at Indiana [Toebes and Chang, 1972).

(5) The HEC-5 model by the Hydrological Engineering Centre (1979).

(6) The Arkansas River Basin Model (Coomes, 1979).

(7) The TVA Model (Shelton, 1979).

(8) The Hydro System Seasonal Simulator developed by the North Pacific Division, the Corps of Engineers (Jones, 1979).

(9) The Lower Colorado System Model (Freeney, 1979).

(10) The Duke Power Hydro System Model (Sledge, 1979).

(11) The Acres Model (Sigvaldason, 1976)

(1) The HEC-3 and HEC-5 Models

The Hec-3 model and the HEC-5 models are possibly the most documented of Reservoir and River Basin Simulation models utilised in Water Resources Management today. The HEC-3 was originally developed to assist in reservoir system analysis for water conservation purposes, while the HEC-5 was directed towards the simulation of flood and conservation systems. The models with continous improvement are able to simulate the response of water resource systems designed to satisfy simultaneously a variety of water based needs. They can accomodate any arbitrary configuration of reservoirs and channels.

The HEC-3 has been widely applied to the Arkansas-White-Red River System in the southern part of the U.S.A. (Fredrick and Beard, 1979); and to other river basins (Eichert and Bonner, 1979). The model has also been extended to the development of other models such as the Oswego Basin Model (Liu et al, 1972).

The HEC-5 like the HEC-3 performs monthly, daily or multihourly operations of multipurpose reservoir systems. The HEC-5 model has been used as an operation model for calculating flood damage and for the study of flood mitigation techniques. Specifically, reservoir system expansion for flood control purposes on the Susquehanna River System was assessed using the HEC-5. (Eichert and Davies, 1976)

The basic operational methodology of the HEC-3 and HEC-5 are listed below:

(a) They utilise a system search routine and therefore cannot simulate the operation of certain types of closed loops without special programming.

(b) They provide releases in accordance with fixed operation rules based on storage and use schedules.

(c) In exercising these rules, they specify a balance of desired storage, and provide this storage distribution subject to water availability in excess of demands at the various locations.

(d) They use a multireservoir zoning procedure.

(2) SIM-I and SIM-II Models

These models developed for the Texas Water Development Board use a procedure for minimising the costs of fluid transfer within a network using an out-of-kilter Linear Programming algorithm (Evanson and Moosely, 1970; Yeh, 1985). The algorithm is an optimisation technique. The SIM models perform reservoir operations by:

(a) The use of a network simulation and solution technique.

(b) The use of an Linear Programming algorithm for computing the optimum operation of the entire system for each time interval.

(c) The use of a system of delivery and storage

priorities, (reflecting the relative values of water) in relation to specified storage balancing levels, for controlling reservoir simulations.

(3) The Acres Model

The Acres Multipurpose, Multireservoir simulation Model (Sigvaldason, 1976; Sigdalvadason et al, 1975) :

(a) Includes the combined rule curve-zoning representation which is also available in the HEC-3 model. However, this combination was extended to include an additional "spill zone".

(b) As with the SIM models, it uses the out-of-kilter optimisation algorithm as a submodel for achieving optimal responses during individual time intervals. The out-of-kilter algorithm is a network flow optimisation technique which is used in simulating the reservoir rules of operation and to solve the most appropriate allocation of water required in satisfying various uses, e.g hydropower flow demand.

(c) has as its main design objective, the provision of the chief operator with an optimal decision and monitoring process for a particular operating policy.

(d) undertakes economic and financial analysis of various reservoir operation scenarios.

The model has been utilised on a day to day operational basis for defining reservoir releases in the Trent River System in Ontario, Canada (Sigvaldason et al, 1975). The

model has been used in simulations involving the existing Akosombo/Kpong hydroelectric system in Ghana (Acres, 1985). The model was applied to maximise the firm energy yield of the Akosombo/Kpong system by the use of a series of simulation runs. The purpose of the model in the Akosombo study is to forecast from reservoir utility operations the economic and financial outcomes. Clearly, this will be helpful in maximising benefits in both planning and real-time operation.In the Akosombo study, the model, which is in two parts, was utilised in simulating electric utility operations over a time horizon of up to 20 years.

Figure 4.7 illustrates the general model structure. The left-hand side of the figure lists the input subroutines, the right-hand side shows output subroutines, and the central portion contains the analysis subroutines.The first module of the model, GF1, simulates the despatch of generation facilities including a complete simulation of reservoir operations with a detailed output of results. The second module, GF2, conducts both a financial analysis and an economic evaluation for the period of simulation. The input routines read the data, echo checks them, and carries out minor data processing as required. The output routines do statistical analysis and outputs all results as required. The analysis routines are broken into three blocks as shown in figure 4.7 and are noted below as:

(1) system operations

(2) fixed financial operations

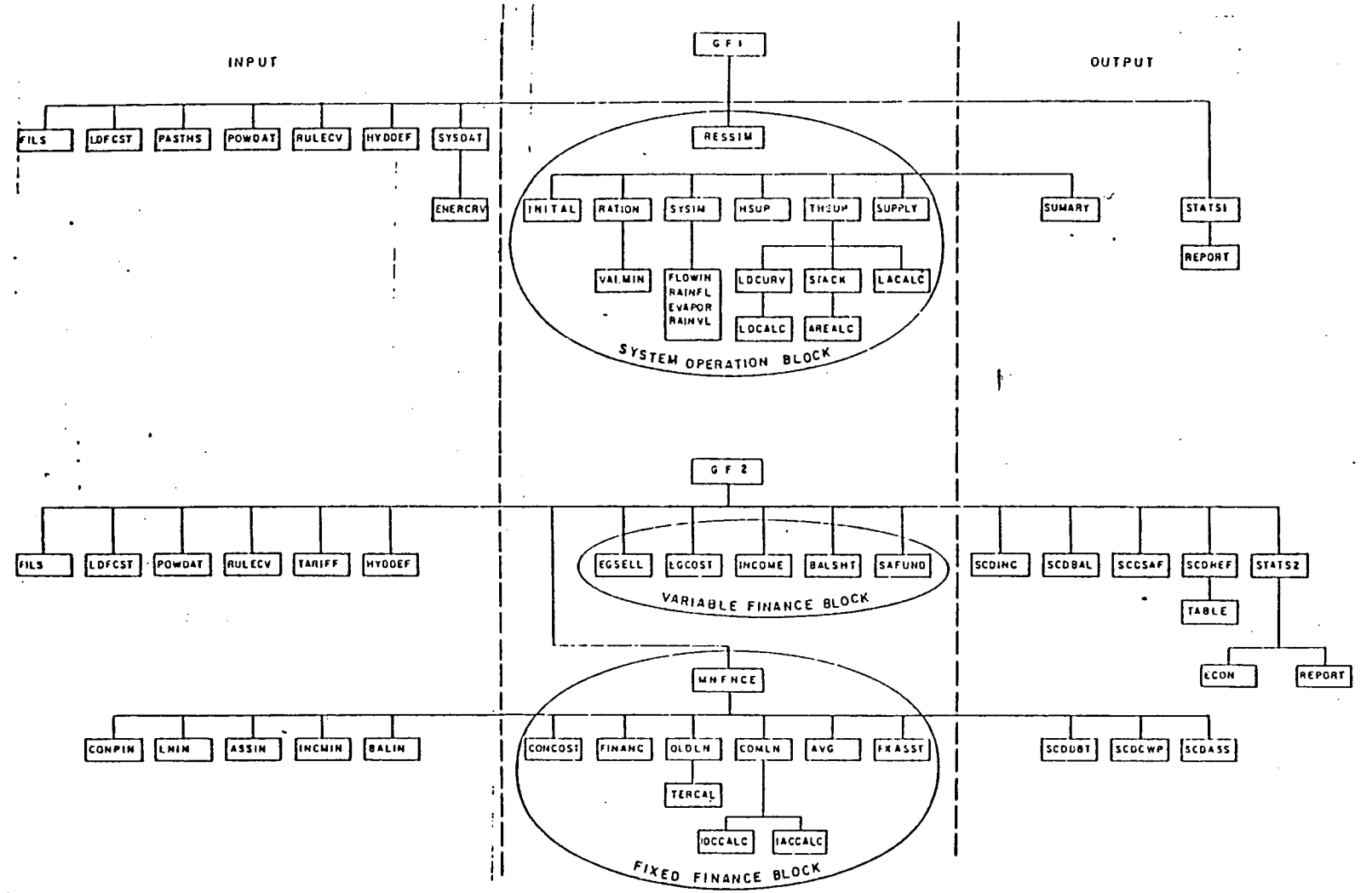


Fig. 4.7 ACRES MODEL

Source Acre, 1985

(3) variable finance operations

A description of the subroutines making up the three blocks of the model shown in figure 4.7 are given in appendix 3. The detailed description of the methodology of the model and its component subroutines are given in Acres (1986).

Acres (1986) stated that in order to investigates the effects of variable hydrological input to the reservoir system, the modules simulate the operational and economic/financial outcome for a number of scenarios as defined by the user. Each defined scenario uses a specified (e.g 10 or 20 year) hydrological sequence from a part of the recorded input data. For example, for a 40-year record, a scenario might use the 20-year portion, with the years in sequence. Thus, as many as 40 sequences could be evaluated by the use of this example.

(4) The Merz and Mcllelan ROSS model

The ROSS model is a reservoir operation simulation program developed in-house by Merz and McLellan, U.K (1986). The model produces a statement of the downstream flow, or energy, or both, which is available from any system of reservoirs and power generating stations in cascade (afterbay), using the basic data under the restraints provided in the input. An extended version of the well known out of kilter algorithm ,also utilised in the SIM and Acres models, is used by the model for the optimisation of reservoir operations.

The model prints out a month by month statement of operational outcomes but Merz and McLellan (1986) states that alterations to the program would allow variations in conditions and simulations over a shorter or longer period, if desired. The model has many optional features, for example, M & M stated that the inclusion of power stations is not necessary. In addition, only the selected options in input can affect model computations or appear in the output.

The reservoir input data required by the ROSS model, which are mostly similar to that utilised by the other reservoir operation models discussed, include month-by-month inflows to the first scheme, live storage-elevation-surface area relationship, initial upstream reservoir levels.

The other optional data which are input as required are evaporation rates, rainfall, evapotranspiration rates, monthly flows not available for power generation, flow rate-friction loss relation, and the flow rate-tailwater relation.

The generation data required are the number of sets, the overall efficiency of generation and its variation with both head and the rate of flow.

Many of the models discussed above are river basin hydrology models oriented specifically to reservoir operation planning. Other basin models are made up of relationships representing the physical and hydrological configurations of the specific basins and reservoirs under

consideration and are named as such. These models however use existing and established simulation or optimisation models as the basis of their development. Examples include the Upper Nile Basin Simulation Model (SMEC, 1977), the Rio Colorado Basin Optimisation Model (Major and Lenton, 1979), the Niger Basin Simulation Model (WMO/NBA, 1987) , amongst others. The limitations of these models in their initial stages of development is that catchment and reservoir sediment processes are not included in their configurations.The inclusion of these processes is necessary considering their interactions with water processes in a river basin. The Nile model is now outlined as an example.

The Upper Nile Basin Simulation Model consists of three components namely, a catchment model, a lake model, and a channel model (W.M.O, 1977). The catchment model estimates daily discharges into the lakes and channels. The lake model carries out a water balance accounting for inflows and outflows from the lakes. The channel model predicts the movement of water in the main river channels between the lakes. The W.M.O (1977) stated that the catchment model for the upper Nile Basin were adapted from the well known Sacramento model as presented by Barnesh et al (1973). The channel model was developed by SMEC (1977) from the original concept by by Amein and Fang (1969) but substantially different from it. The eight lake models were all specifically developed by SMEC (1977) for the study.

Clearly, from the above discussions, River Basin Simulation Models, whether broad-based or reservoir-oriented, are mainly hydrology models with very little consideration of time variant sediment processes. Comprehensive models that combine the processes of hyrology and sediments are advantageous and would provide more optimal designs and operation of water resources schemes.

One model that combines both the hydrology and sediment aspects of river basins and reservoirs is the Strathclyde River Basin Model. The model consists of three process models, a watershed model, a soil erosion model and a channel and reservoir routing model. Prior to the description of the Strathclyde River Basin Model and the River Basin Management System, a general review of sediment models is given in the next section.

4.5 River Basin Sediment Models

The Meyer and Wischmeier Model; the Negev Sediment Model (Negev, 1967) and Strathclyde Sediment Model are few examples, amongst other river basin model developments (Fleming, 1977)

The Meyer and Wischmeier (1969) erosion Model considers major components of the soil erosion process such as rainfall intensity, infiltration rates, slope lengths which are divided into short finite increments, with steady state conditions being initially assumed. Four mathematical sub-models compute values for each increment of detachment by rainfall;

detachment by run-off; transport capacity by rainfall and transport capacity by run-off. Figure 4.8 is the flow chart showing the erosive mechanism of these four sub-processes. The four sub-models representing them are combined to route the eroded soil downslope. Details are available from the literature.

The Negev Model (1967) represents erosion processes by mathematical functions on a digital computer. The functions have physical parameters as independent and dependent process variables which are interpreted by simple power relations with constants and exponents obtainable by calibration. This approach recognises the importance of rainfall intensify and overland flow as prime agents of erosion. Input are hourly precipitation and overland flow which were measured and generated (using the Stanford Model) respectively. This model also distinguishes between land surface erosion where rainfall and overland flow are primary movers and channels erosion where total flow is the agent.

The Strathclyde Soil Erosion and Sediment Model (SSEM) is the sediment and erosion module of the complete River Basin Model discussed in the next section. It basically utilizes four sub- systems of erosion; sheet, rill, gully and channel which are considered

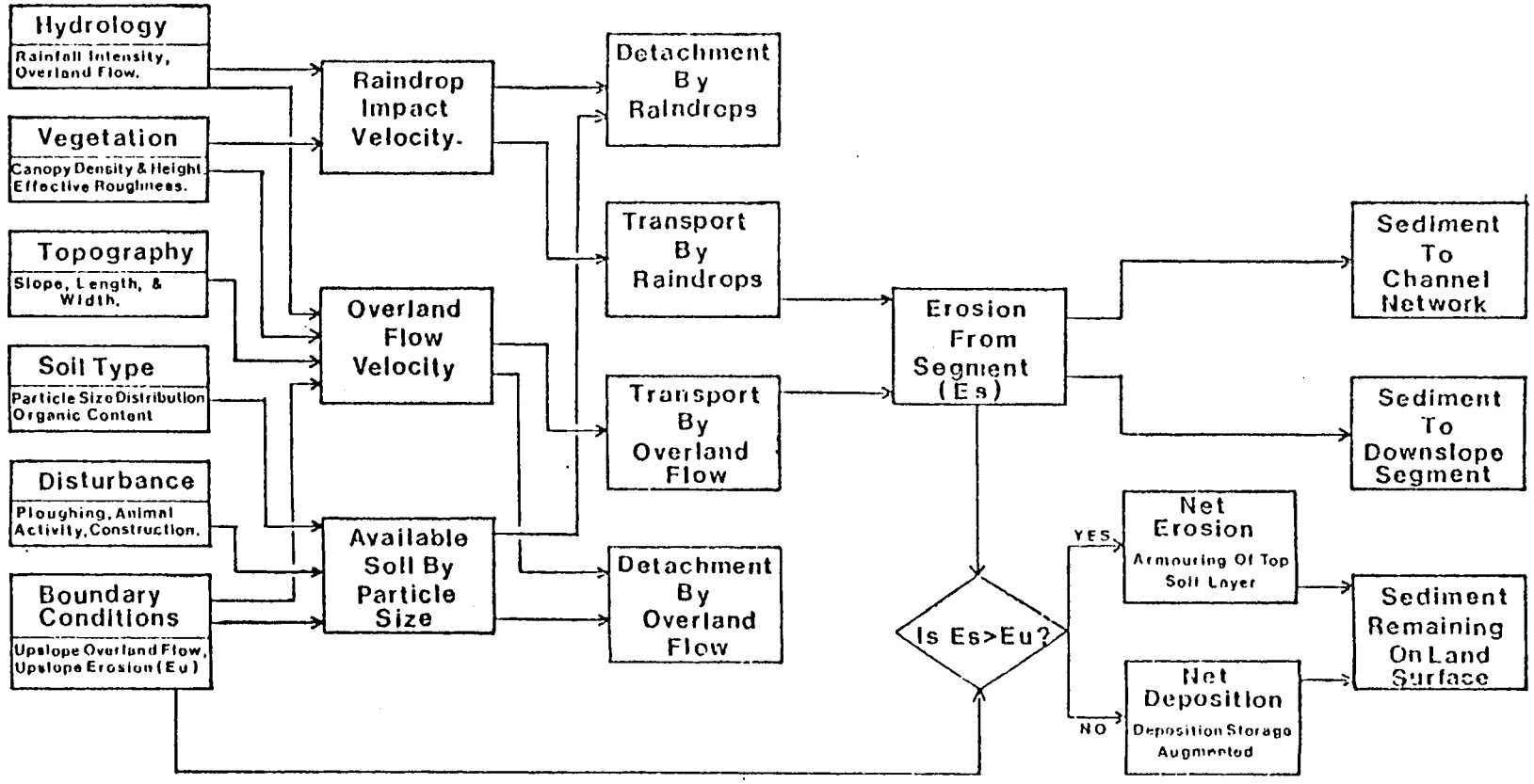


Fig. 4.8 EROSION MODEL COMPONENTS

Source Fleming, 1977

as storages and sediment is assumed transferred from one storage to another. The flow chart shown in Figure 4.9 illustrates the processes used in the formulation of the Model. The Model is discussed in greater detail in sections 4.6 and 4.7 as a part of the River Basin Model and the River Basin Management System respectively. The details of the process representation in these models are provided in chapter 5..

4.6 The Strathclyde River Basin Model

The River Basin Model simulates completely the combined watershed and channel system utilising the processes of the complete hydrologic cycle and soil-water interaction. The Model whose general working structure is shown in Figure 4.10. has three components; the Watershed, Erosion and Channel routing models. It simulates both the water and sediment response in River Basins. Data input is highly flexible and thus the programme can be used under a variety of input and output data conditions. The model is based on the Stanford Watershed Model, the hydrocomp simulation programme and the Strathclyde Sediment and Erosion model.

(1) The Watershed Model

The model, like its original Stanford model, (Crawford and

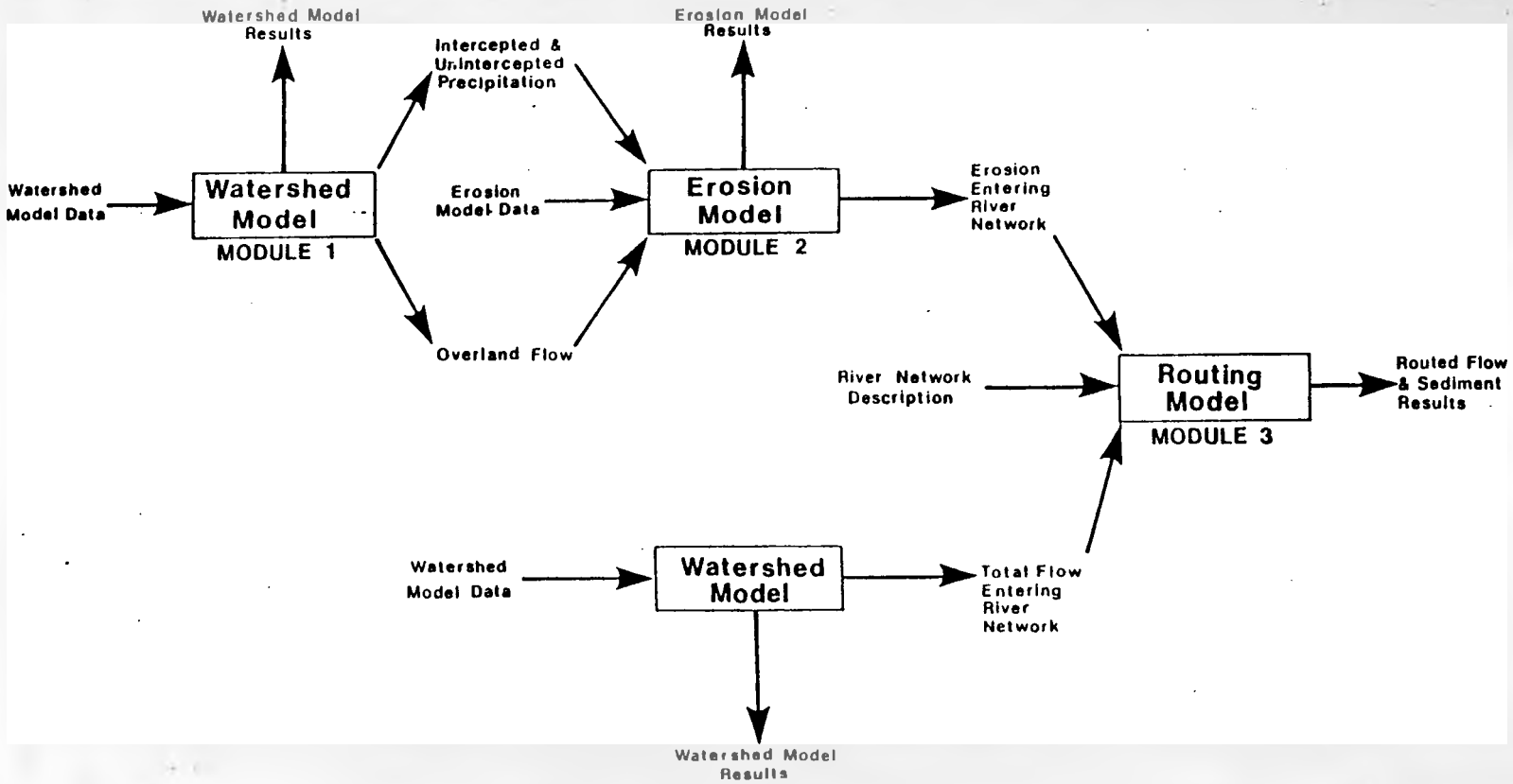


Fig.49 RIVER BASIN MODEL COMPONENTS

Source Fleming, 1986

Linsley, 1966) utilises an implicit partial area runoff methodology where infiltration is assumed to be variable across the watershed. For each storm, that portion of the watershed for which infiltration capacity is exceeded is determined. This in effect allows for partial area storm runoff contributions to be summed appropriately (Kirkby, 1985).

The Watershed component computes the hydrological response of land surfaces, taking into account rain and snow processes, interception, infiltration, soil storages, interflow, groundwater flow and overland flow. It also outputs time series information of overland flow and effective precipitation in separate computer files for use in the calculation of erosion and sediment yields by the erosion model component. Total surface run-off time series data are also computed and stored in computer files for use in the flow routing by the channel component.

Figure 4.10 illustrates the general structure of the Watershed Model component.

(2) The Land Erosion Model

The second component of the River Basin Model is the Erosion Model which takes as input the calculated overland flow and effective precipitation from the Watershed Model together with other physical land surface data to compute sediment yield from land to the channel network. Figure 4.11 shows the general structure of the erosion model.

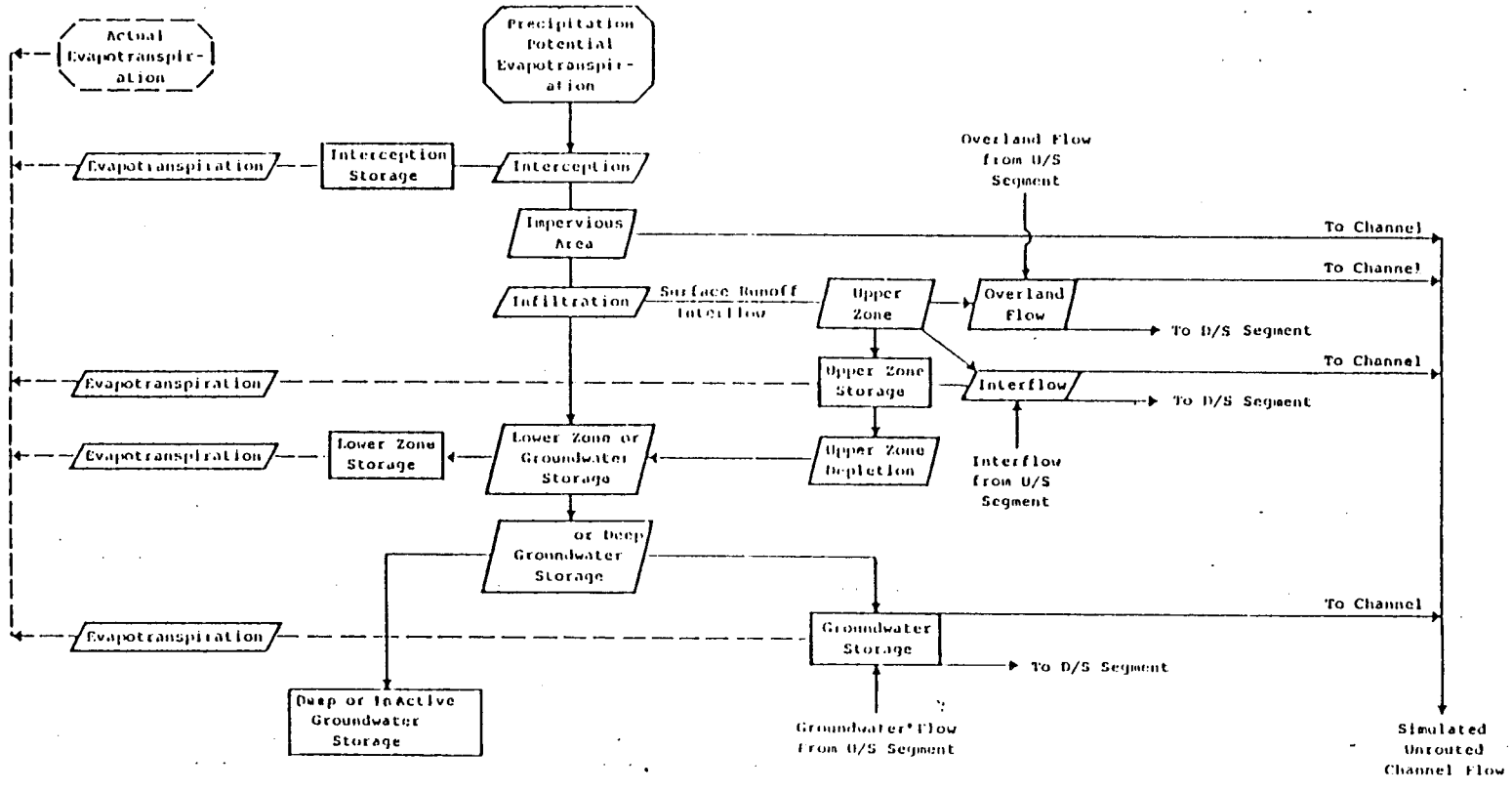


Fig. 4.10 : FLOW CHART OF MODIFIED STANFORD WATERSHED MODEL

Source Fleming, 1986

(3) The Channel Routing Model

The third component of the model is the channel routing module which incorporates river flow routing using the kinematic method. Sediment routing is combined with flow routing through river channels and reservoirs. Reservoir routing utilizes the level pool technique. Details of individual models of the River Basin Model as part of the river basin system are given in Chapters 6 and 7. Figure 4.11 illustrates the general structure of the routing model. The linkage of all three models are illustrated by figure 4.12. Figure 4.13 shows the process representation of the complete river basin model (Fleming's model) as developed by Fleming (1971, 1986).

4.7 Final Remarks

This chapter has reviewed the subject of mathematical models and computer simulation as applied to river basins mentioning the importance of incorporating the details of hillslope, watershed , channel and reservoir processes in model formulation.

From the review of models and modelling techniques, it is obvious that a lot of work has been done in the area of reservoir operational planning and in the design of the storage schemes. The interplay and interlink between river

LOGIC CHART OF THE CHANNEL NETWORK FLOW ROUTING MODEL Source Fleming, 1986

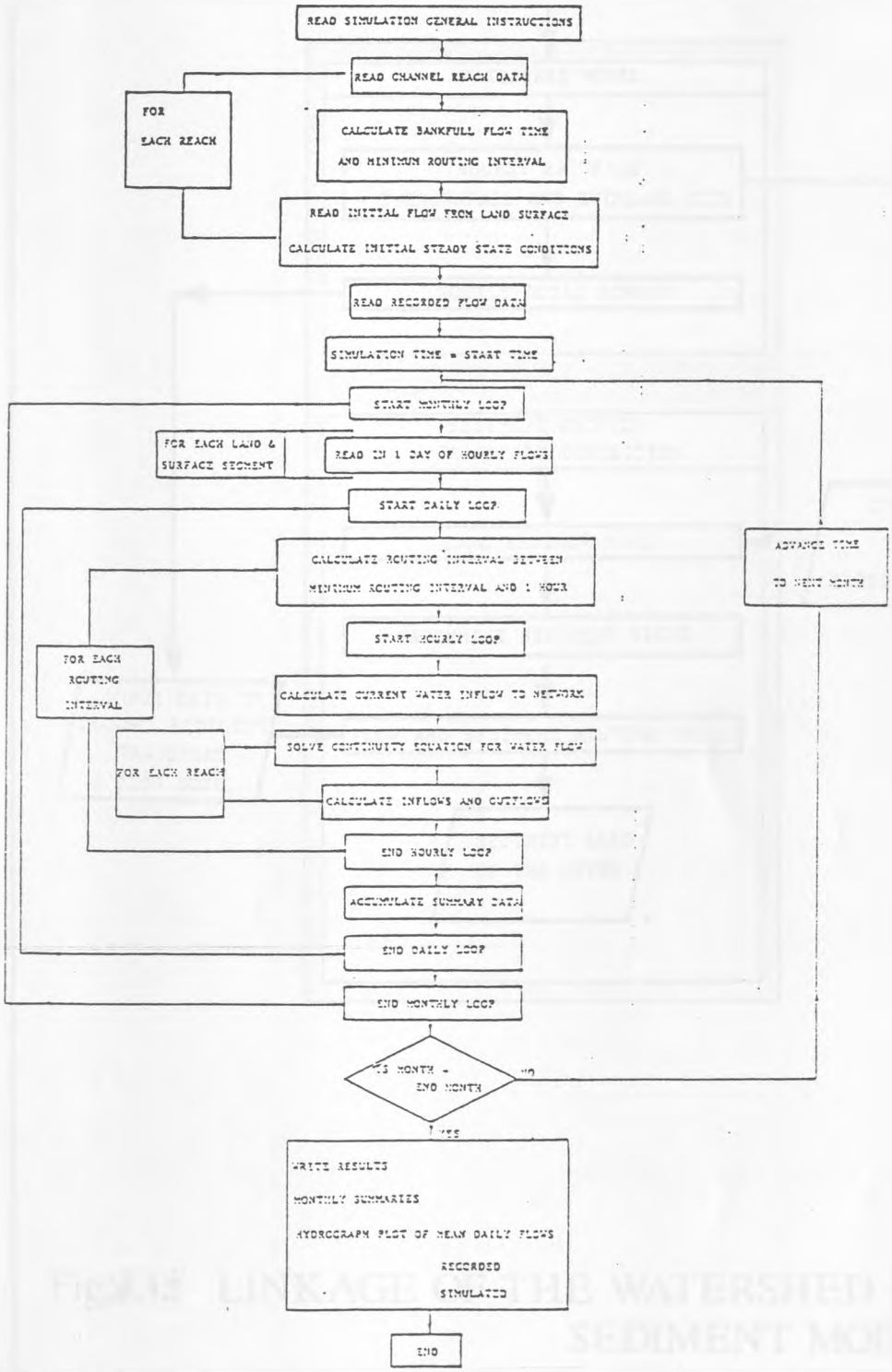


FIG 4.11

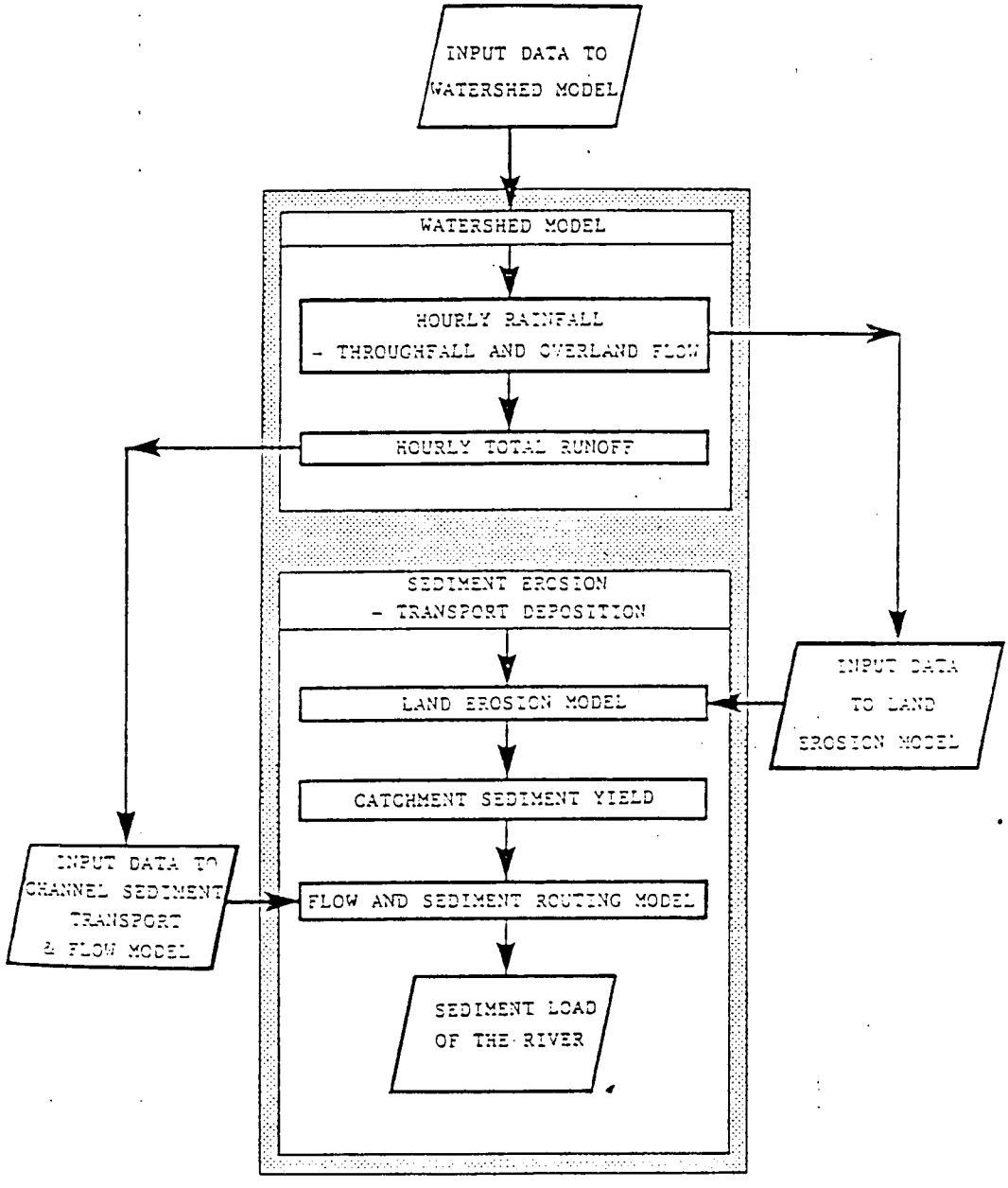


Fig 4.12 LINKAGE OF THE WATERSHED AND SEDIMENT MODELS

Source Fleming, 1986

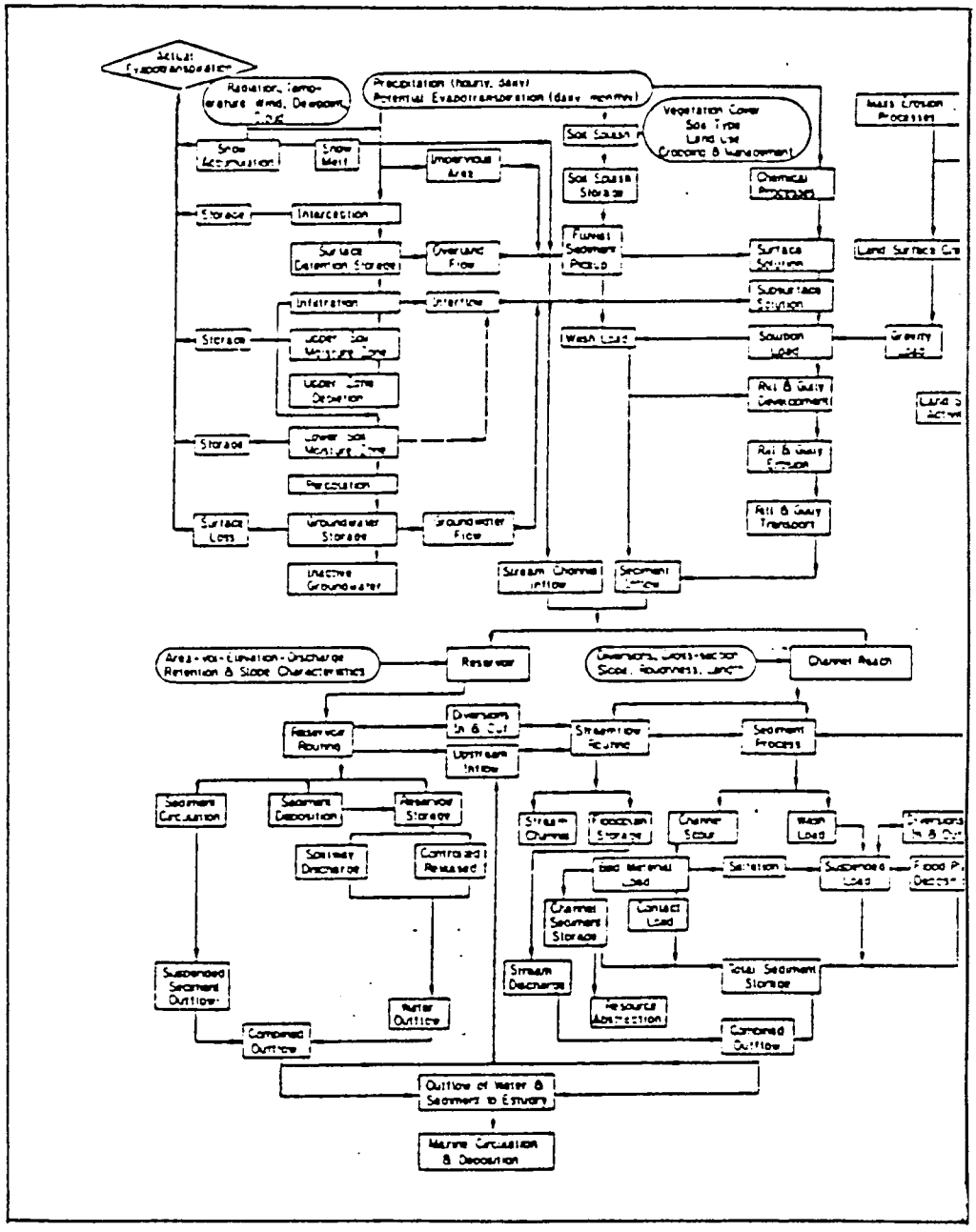


Fig.4.13 FLOW CHART OF FLEMING MODEL

(FLEMING, 1971)

basin processes (hydrology and sediment) and the reservoirs appears neglected in most of the available models except the strathclyde river basin models. Emphasis in models' structure has been directed mainly at the water balance of reservoirs. Their simulation timesteps of the long term planning models utilises monthly reservoir inflows, generally generated by monthly rainfall-runoff models. For short term operational purposes, extensive modifications are required in their conversions to real-time models.

To meet the main objective of this research, clearly, a comprehensive conceptual deterministic model which includes, the hydrological and sediment processes involved in river basin-reservoir systems and which is capable of assessing and monitoring their impact is required. The model must be capable of simulating,

(1) multi-annually for the study of projected long term changes in river basins

(2) annually for assessing the short term impact of catchment changes and for real-time operations.

(3) in short time steps for process interactions and impact to be properly and realistically simulated.

(4) watershed hydrology processes inclusive of all interacting hillslope processes.

(5) watershed sediment processes consisting of their hillslope behaviour.

(6) channel routing

(7) reservoir routing and system operations.

(8) reservoir sedimentation.

(9) economic analysis and impact of reservoir releases to single or multipurpose uses.

The River Basin Model described in section 4.5 above is capable of handling items (1) to (6) completely and (7) partially. To achieve the simulation of the above listed processes and items, which is required for implementing and testing the proposed process approach, it became necessary to update the Strathclyde River Basin Model into a River Basin Management System. This system is described complete with its merits and demerits, clearly delineated, in chapters 5 and 6.

CHAPTER 5

CHAPTER 5:RIVER BASIN MANAGEMENT SYSTEM: WATER AND SEDEMENT COMPONENTS

As a follow up on the review of river basin models discussed in chapter 4, the river basin management system, the main tool of this research is described and discussed herein. Section 5.1 provides a brief outline of the composition of the system and the links between models. Section 5.2 lists the modifications undertaken in various components of the system. Section 5.3 describes the facilities of the system which allows for easy use of the system, especially in micro computers. Sections 5.4 - 5.7 describe in reasonable detail the separate components of the system, namely, the watershed, routing operations, erosion, and reservoir delta models, in that order. The economic analysis model is discussed separately in chapter 6. Section 5.8 provides a few final remarks on the capabilities and limitations of the system.

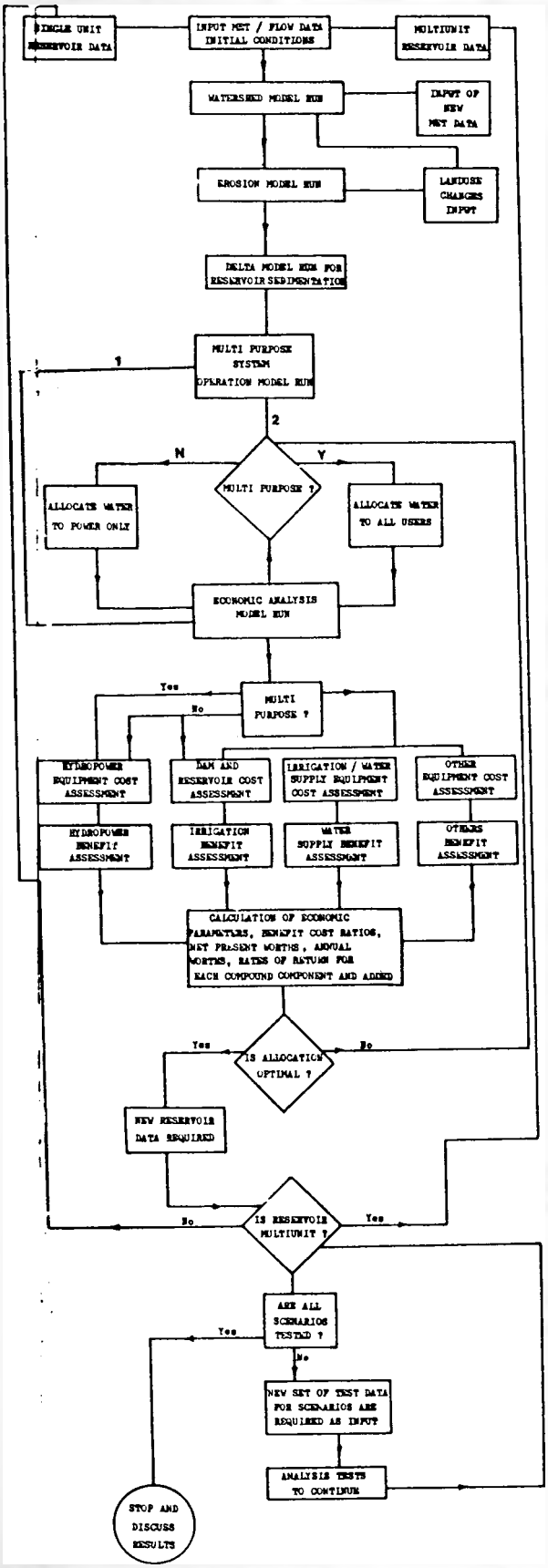
5.1 System Description Outline

The River Basin Management System assembled herein consists of the three components of the Strathclyde River Basin Model (the watershed, erosion, and routing models), a Reservoir Delta Simulation Model, and an Economic Analysis Model. These are all interlinked (figures 1.8 and 5.1). The choice of these models for the study conforms with the process representation criteria defined for this work as much as possible. A brief description of the watershed, routing, and erosion models has been provided as part of the Strathclyde river basin model in chapter 4.

The linkages of the model components are illustrated in figures 5.3 and 5.2 for main frame and micro computers respectively. The figures also illustrate the physical link of the models. Basically, the connections of the first three components are as in the river basin model as described in chapter 4. The delta model accepts annual values of sediment yields passed from the erosion model.The routing operation model passes annual or time series water releases to the economic component which sums them up and utilises the sum for analysis. A detailed description of each of the models used in the system is given seperately in sections 5.4-5.7 and chapter 6.

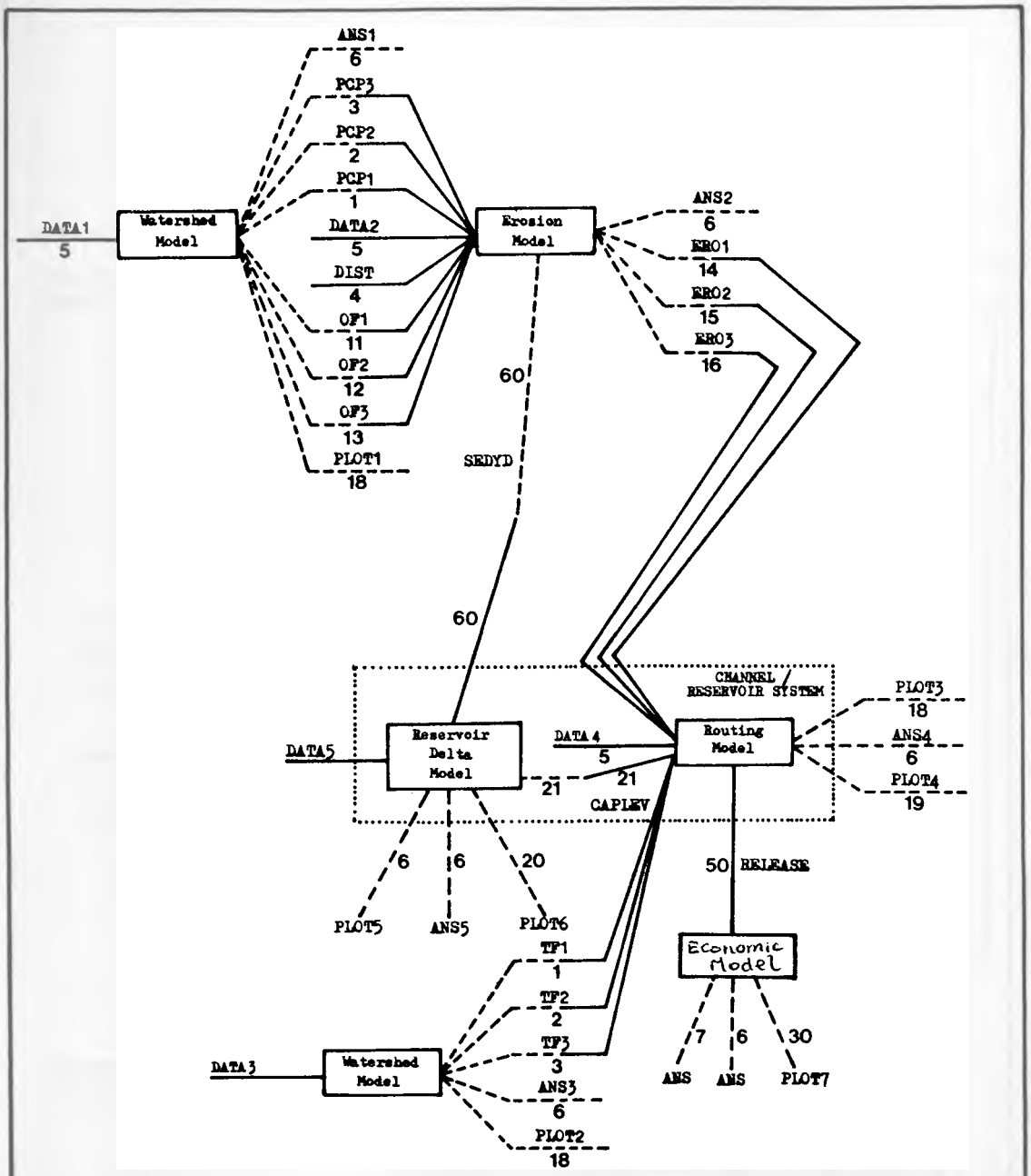
5.2 Listing the Modifications of System Components

The modifications effected by this writer in the existing models utilised and a brief outline of the potentials of the new economic simulation model developed by him, are



From the above algorithm ,optimisation of varios parameters such as water allocation, reservoir sizes, costs, benefits, etc., can be carried out for different scenarios of climate, land use, minimum operating levels, etc., for single or multiple reservoir systems.

Fig. 5 OPERATION OF THE RIVER BASIN MANAGEMENT SYSTEM



——— Input Channel

- - - - - Output Channel

5 Channel Number

TF1 File Name

FILES

PCP - - Total Precipitation
OF - Overland Flow
ERO - Erosion
RELEASE - Release
TF1 - Total Flow

FILES

SEDYD - Annual Sediment Yield
CAPLEV - Capacity / Elevation Relationship
RELEASE - Annual Reservoir Release

Fig. 5.2b FILE AND CHANNEL ALLOCATION NETWORK FOR THE RIVER BASIN MANAGEMENT

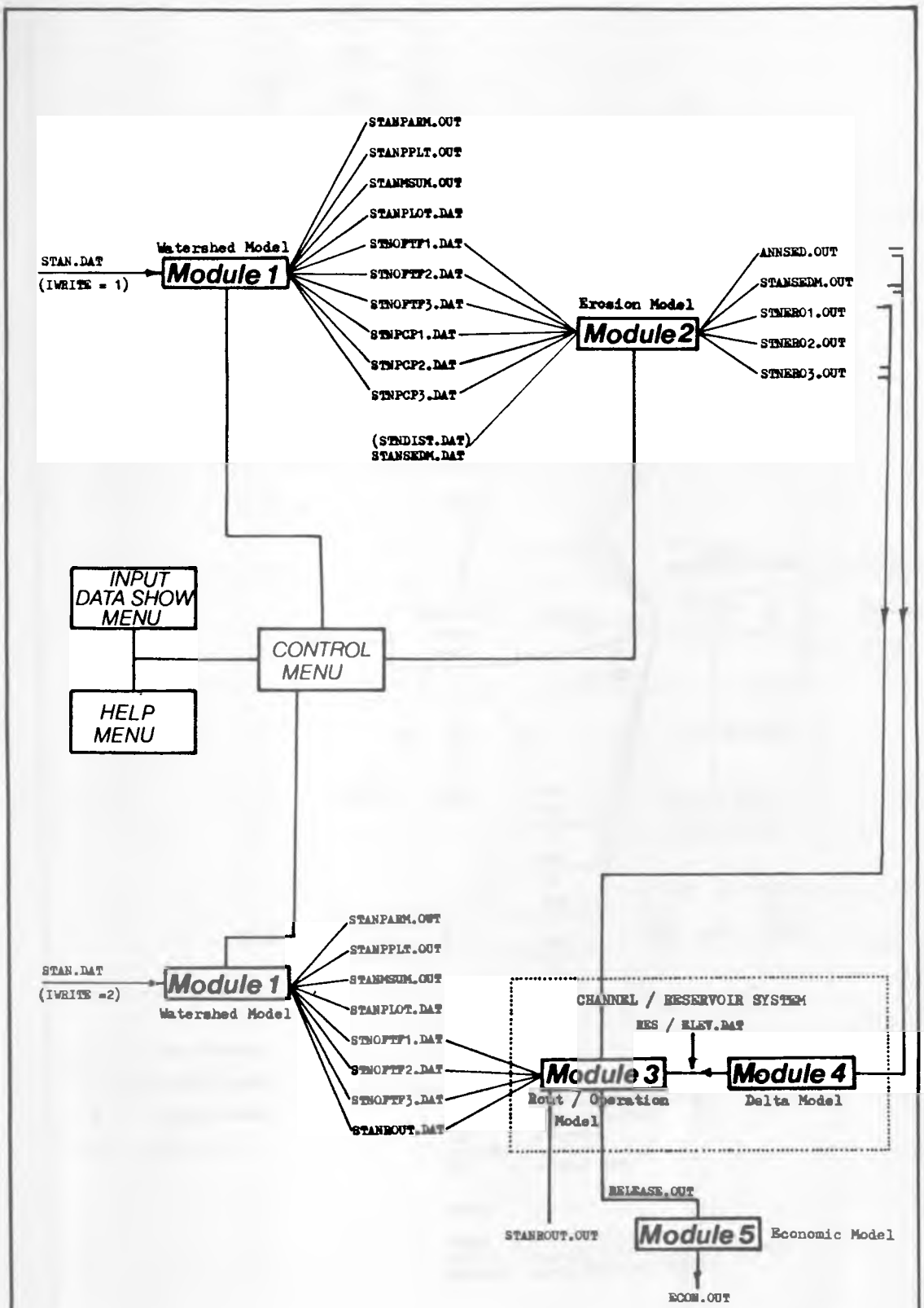
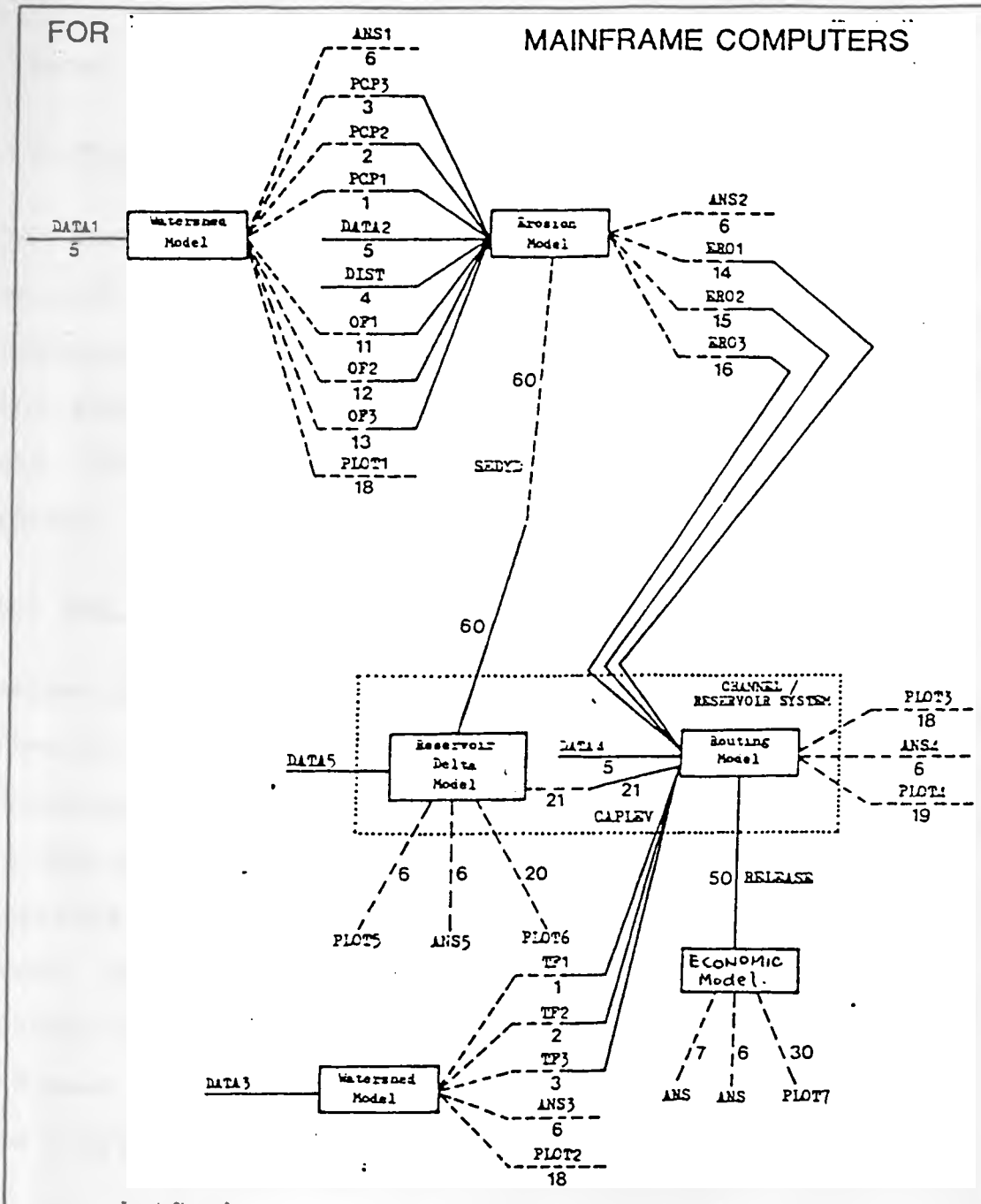


Fig. 5.2 MICROCOMPUTER FILE ALLOCATION NETWORK FOR RIVER BASIN MANAGEMENT



——— Input Channel

- - - - - - Output Channel

5 Channel Number

TF1 File Name

FILES

PCP - - Total Precipitation
OF - Overland Flow
ERO - Erosion
RELEASE - Release
TF1 - Total Flow

FILES

SEDYD - Annual Sediment Yield
CAPLEV - Capacity / Elevation Relationship
RELEASE - Annual Reservoir Release

Fig. 5.3 FILE AND CHANNEL ALLOCATION NETWORK FOR THE RIVER BASIN MANAGEMENT SYSTEM

listed in the rest of this section as follows;

(a) The Watershed Component

The model has been described in section 4.6.of chapter 4 as part of the River Basin Model and no modifications were necessary as it simulates the river basin interaction with the hydrological cycle comprehensively. Results from its use (the original) around the world have been acceptable (Black, 1973 ; Kirkby, 1985).

(b) The Land Erosion Component

Before testing the system, modifications were made to the erosion components. Fleming's empirical relationships for assessing sediment yields in a river basin were included in the erosion model for comparisons with the yields obtained from the original process formulations of the model. This is carried out as an investigation of the suitability of the use of the relationships for simulating sediment yields in the Donga Basin and for comparison with the original sediment assessing algorithms.

(c) The Routing and System Operation Component

Modifications were made to the routing model component for the improved handling of multipurpose reservoir system operations. This involves the provision of an algorithm for accepting an elevation-storage-release rule curve option as an addition to the daily release alternative already in use. Further, the model is able to sum up instantaneous releases annually or otherwise for use by the economic analysis model.

Modifications have been made to the model to account for reservoir evaporation losses in the water balance of the reservoir. The losses are output both in flow units (cumecs) and power and energy loss units (Mw and Gwh/a).

(d) The Reservoir Delta Simulation Component

The reservoir delta simulation model used is an extensive modification of the original sea-river delta generation model, developed at Stanford University by Bonham-Carter and Sutherland (1968). Modifications include,

(1) converting the unlimited sea depositional area to a confined reservoir depositional area with algorithms for calculating reservoir storage-elevation and area-elevation relationships for time periods as required or specified.

(2) the introduction of a time duration adjustment factor. This introduces flexibility as it allows the model to operate in any number and duration of time increments depending on the requirements of the user and the capability of the computer. This modification allows the model to be operated over a wide time horizon, ranging from a few seconds to hundreds of years.

(3) the introduction of an accounting factor for compaction. This is used to compress the vertical depths of reservoir deposits over time as would occur in real life. This is estimated from sediment compaction curves published by Graf (1971) and reproduced by Mahmod (1987).

The operation, graphics, and outputs of the model are

user-friendly, especially on the mainframe. This model component of the system is described in detail in a later part of this chapter.

The main limitation of the model is its inability to accept time- series data of flows and sediment yields. Any modification correcting this anomaly would require a complete overhaul of the computer program. Originally, the model receives water inflows and sediment load data on a mean annual basis. This is distributed statistically covering the whole simulation period.

(e) The Economic Simulation Component.

As the river basin model, lacks an economic, social and environmental cost benefit assessment component (a useful tool in river basin management), the system was developed to include an economic analysis program. Models, such as the Trent Economic Model developed for the River Trent in Canada, an adaptation of the Acres model discussed in chapter 4, include economic assessment modules but tend to lack in their formulation the details of the interplaying catchment processes (Sigvaldason et al, 1975). Thus, the Trent approach is different from that of this research.

This model component accepts annual or time series reservoir releases from the system operation model. Prior to economic analysis, time series release data, if supplied, are summed up appropriately. The model is thus able to analyse reservoir operations on a multi-annual,

annual, monthly, weekly or daily basis as required. The large database required for multi-annual (e.g 50 yr) simulations has necessitated the use of the annual release for the year whose mean hydrograph and annual flow values are closest to the long term mean.

The releases are utilised in carrying out economic analysis using benefit cost methods, namely, present worth, annual worth, benefit cost ratios, and the rate of return techniques. An added potential of the simulator lies in its capability of evaluating cost and benefits by use of appropriate functions obtained from the literature. The model is capable of escalating all costs and benefits into the future by means of an input (inflation rates) and discounts them back to the present for realistic economic comparisons.

The model can be run for single or multipurposes. The costs and benefits for hydropower energy generation, irrigation, water supply are the main aspects evaluated. Fish and recreation benefits (or costs) are evaluated by means of appropriately estimated benefit (or cost) factors related to the capacity and area of the reservoir respectively. Flood control and navigational benefits (and costs) are evaluated by means of assessed factors related to the releases from the reservoir.The factors are estimated by the use of data from reservoirs existing in the vicinity of the planned scheme. This was necessitated by the writers inability to procure appropriate cost and benefit functions from the literature. The multipurpose

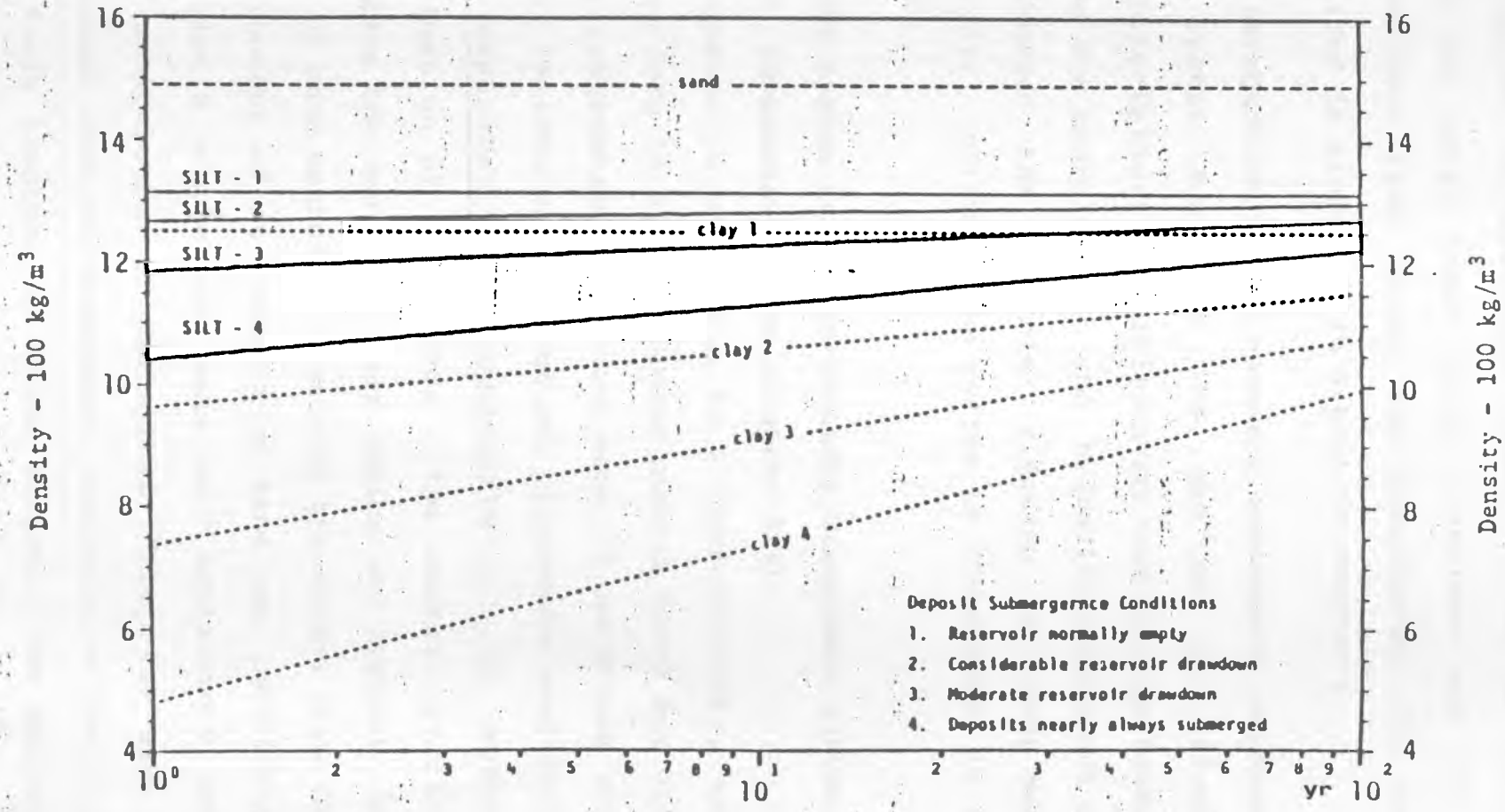


FIG 5.3b SEDIMENT DEPOSIT IN RESERVOIRS: VARIATION OF DENSITY WITH AGE AND SUBMERGENCE

a Graf 1971

orientation of the model is illustrated in model structure flowchart provided in chapter 6. The use of the model is not limited to reservoirs alone. It can be applied to intra and inter basin water transfers and for water abstractions from rivers and groundwater. The economic simulator is discussed in detail in chapter 6.

The introduction of the economic assessment component into the system broadens its use as an integrated, multidisciplinary, multipurpose and system management system for hydrological and hydraulic assessment as well as economic and financial planning of river basin and reservoir systems. The system is illustrated in Figures 1.8.

5.3 The System as a User-Friendly Management System.

5.3.1 Interactive Menus (figures 5.4)

The System is available in a user-friendly, and menu driven form in microcomputers(Amstrad 1640) but also runs in a programme command batch mode in mainframe computers (Vax). In both cases a central programme is used in driving the seperate model components or an appropriate combination of them. Thus, the central program also provides the source of user choice and control. As there must be some method of receiving the choice from the user, the concept of an interactive menu was introduced. This provides a simple and basic self explanatory method of user manipulation of the system. Thus, the user is not alienated from the management, analysis, or design process and feels involved in system control. The philosophy of

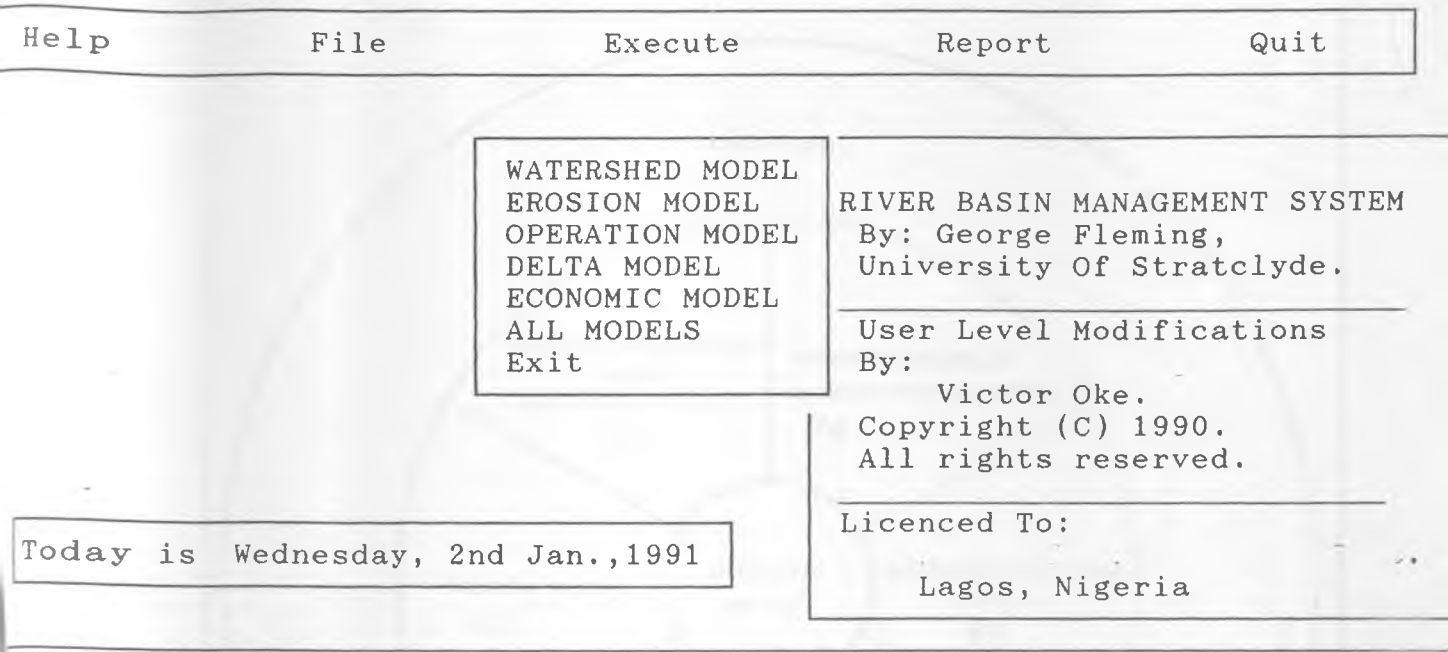
user-friendly or user controlled interactive environments is a beneficial step forward in the use of computer technology (Loucks, 1986; Dickson,1987).

The River Basin Management system is menu-driven in micro computers both in individual model or collective system runs and in understanding the use and methodology of the models and programme composition (through a programme help menu). This describes the intricacies of the models of the system including the areas of links. Thus, the help menu helps as computer-aided user guide.

5.3.2 Multiple Menus

It is possible to have several focal programme sections. Indeed, regarding each of the five models of the system, sub-menus can be provided to link each subroutine to enhance localised control and user understanding of model working. This has been partially accomplished due to time (Fig. 5.3b) constraints but remains an open, interesting and necessary option for future researchers. Advances in this direction would improve the marketability of the model by broadening its use.

Other sub-menus are provided by the writer, this includes a separate help menu for each model which is accessed from the main help menu, and a datashow main menu which accesses datashow menus for each model. The datashow menu for each model has three options namely, shows the data for the model and a help menu that describes the data. The figures 5.5 illustrates the menu options as they operate.


Help
File
Execute
Report
Quit
WATERSHED MODEL
EROSION MODEL
OPERATION MODEL
DELTA MODEL
ECONOMIC MODEL
ALL MODELS
Exit
RIVER BASIN MANAGEMENT SYSTEM
By: George Fleming,
University Of Stratclyde.
User Level Modifications
By:
Victor Oke.
Copyright (C) 1990.
All rights reserved.
Licenced To:
Lagos, Nigeria
Today is Wednesday, 2nd Jan.,1991
Carry Out a Study or an Analysis

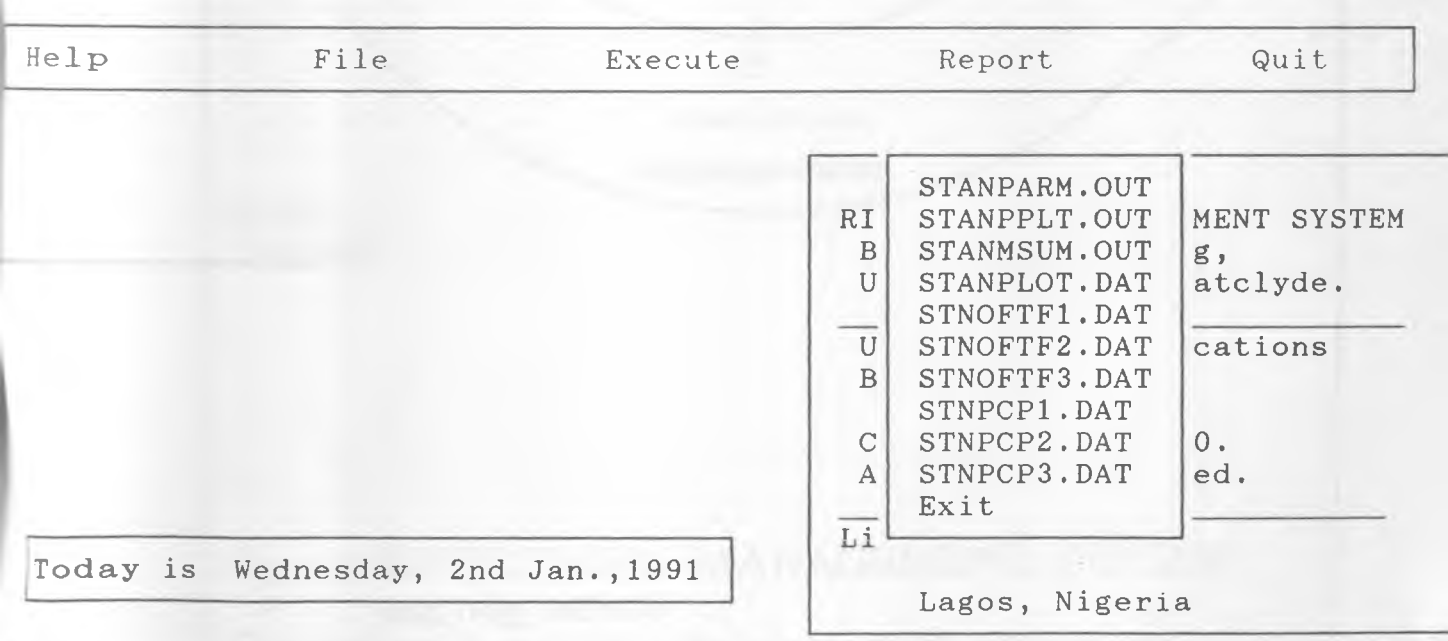

Help
File
Execute
Report
Quit
STANPARM.OUT
STANPPLT.OUT
STANMSUM.OUT
STANPLOT.DAT
STNOFTF1.DAT
STNOFTF2.DAT
STNOFTF3.DAT
STNPCP1.DAT
STNPCP2.DAT
STNPCP3.DAT
Exit
MENT SYSTEM
g,
atclyde.
cations
0.
ed.
Lagos, Nigeria
Today is Wednesday, 2nd Jan.,1991
Print Reports From one or more Studies

Fig. 5.3b1

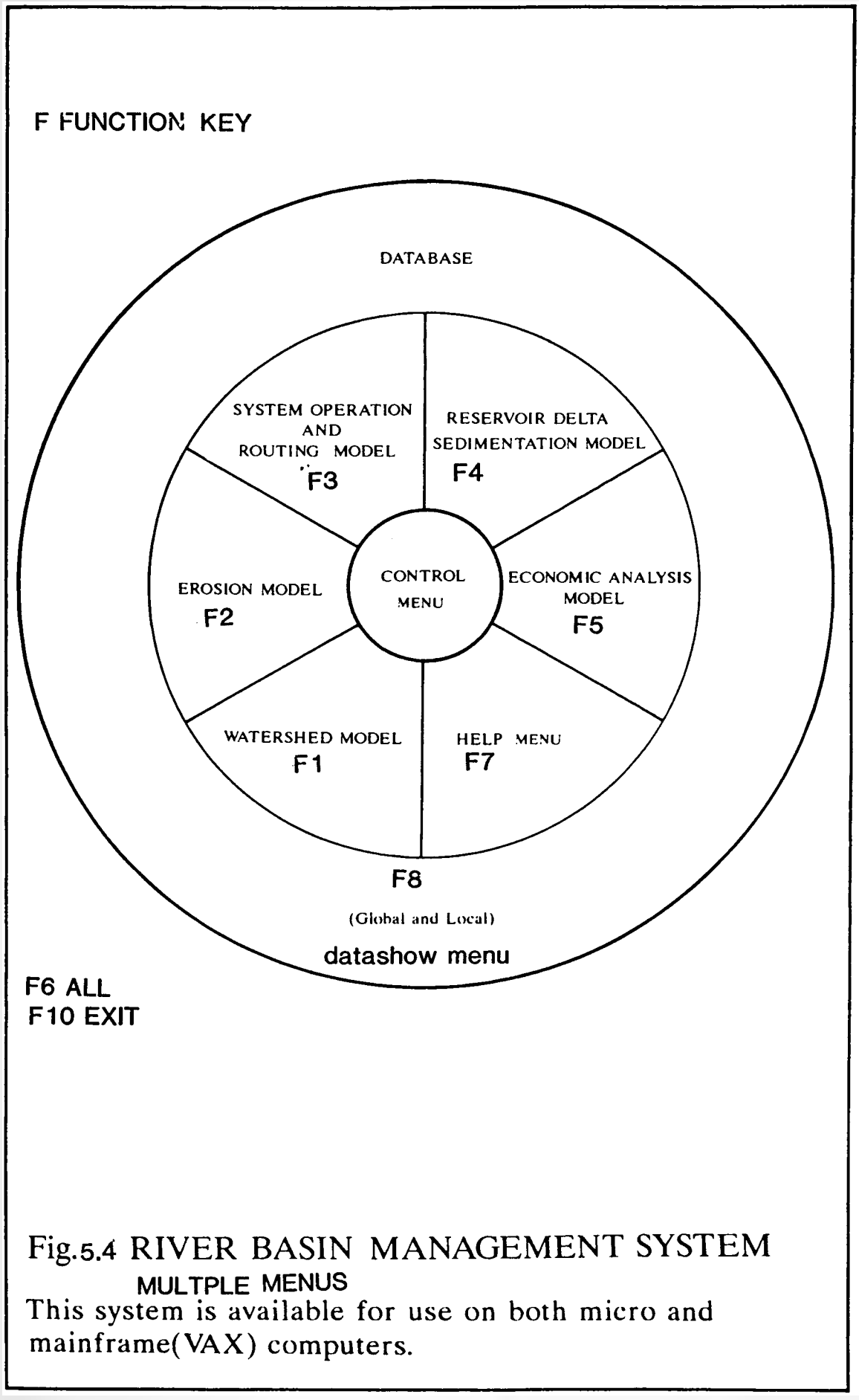


Fig.5.4 RIVER BASIN MANAGEMENT SYSTEM

MULTPLE MENUS

This system is available for use on both micro and mainframe(VAX) computers.

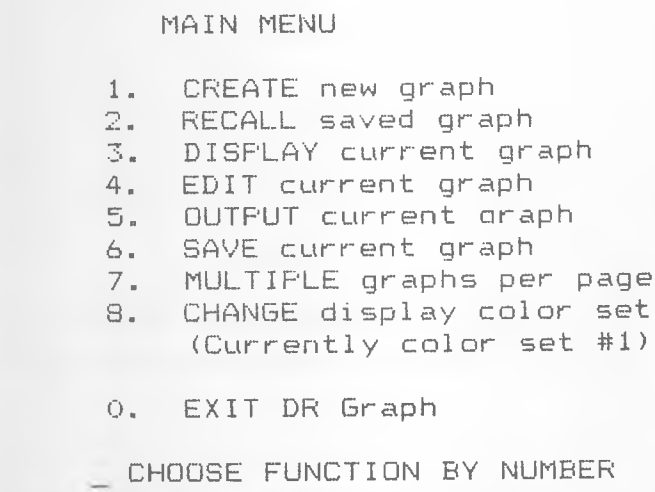


FIG 5.4a DR GRAPH (after Digital Research ,1983)

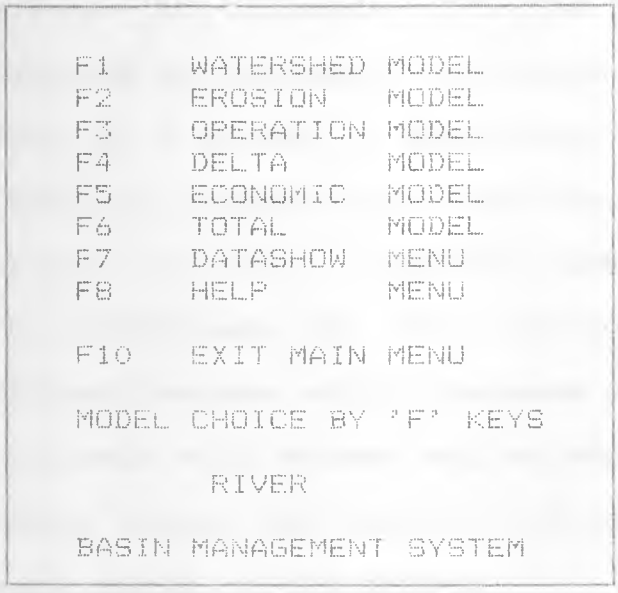


FIG 5.5 SYSTEM MENU

5.3.3 Menu Types

(a) Cursor Menus

Dickson (1987) compares interactive menus and their role in computer aided management. There are two common menu types widely used for computer-aided management practice, namely the cursor driven and the list representation menus. In a cursor pointing menu, choices are displayed in columns and rows using one or two word description of the choice. Selection of choices here is made by moving the pointer to coincide with the desired option and then followed by depressing the return key. This movement can be made by a mouse, a digitising tablet, key arrows or a light pen, depending on the available computer hardware. Figure 5.6 shows a typical format of this type of menu. The advantage of this cursor type menu is in its attractiveness which becomes even more pronounced in terminals with screen and colour resolution and adressing modes where the current choices on the menu can be highlighted on the screen.

(b) List Menus

In a list representation menus, choice is usually made by the use of a single key or a single number, followed by a return. This type of menu limits the menu size to a maximum of ten items. This was however found to be adequate for the River Basin Management system as the models comprising the system are five at this stage. Other options which include the help, datashow, and control menus etc can be easily accomodated by the other five

M A I N M E N U

Move through the Menu by pressing Function or Arrow Keys.
and select the Highlighted item by pressing SELECT or Home

Selecting a "Group" Name will cause a more detailed
Menu for that Group to be displayed.

Group	Define one of four types of design storm or a historic storm				
Hydrology	STORM	IMPERVIOUS	CATCHMENT	BASE FLOW	
Design	PIPE	ROUTE	POND	CHANNEL	DIVERSION
Hydrograph	START	ADD OVHYD	NEXT LINK	COMBINE	CONFLUENCE
Files	MANUAL	AUTOMATIC	FILE HYDRO	FILE RAIN	REWIND
Display	DISP QPEAK	DISP HYDRO	PLOT HYDRO	PLOT RAIN	
General	QUIT	MENUPROMPT /LONG	REVIEW	COMMENT	

UP 2 DOWN 3 LEFT 4 RIGHT 5 SELECT 7 HELP

FIG5.6 CURSOR MENU

function keys. The use of the sophistacated cursor menu was therefore not necessary.

5.3.4 The System's Chosen Menu

Specifically, there is a control menu with a list representation by number and where choice is made by the use of a single function key and followed by a return. An example of such a menu type shown in figure 5.4 (a) is the commonly used graph plotting DR graph menu (Digital research, 1983). The watershed model run is activated by the choice of the function key F1. Similarly, the erosion, routing and system operation, the reservoir delta, and the economic analysis models are activated by the functions F2, F3, F4, F5 respectively. On the choice of a model run by function number and prior to its activation, all the preceeding models supplying it with data are first activated and run sequentially. The use of the number F6 activates all the models sequentially and produces a complete river basin and reservoir analysis producing outputs of a water balance, sediment yield, reservoir sedimentation, channel routing, reservoir operation, and the economic analysis of reservoir releases.

5.3.5 Graphs

Graph plotting has been well developed for part of the system on microcomputers. This facility is available only for the watershed model, erosion, and routing model but not for the delta and economic analysis model. The delta model program has many calcomp graph plotting subroutines

which are not readily adaptable to the microcomputer. New microcomputer oriented plotting programs are required but these were not available at the time of this research. Moreover, this was not considered the main focus of the study. The main objective of the study, however, was to

develop a working management process based system that can study the effect of long and short term process changes on river basin response for use in both mainframe and micro computers. However, the graph plotting capability of the system is well advanced on the mainframe-the strathclyde university vax computer using the ginograf and calcomp systems.

5.3.6 The Computer Environment

In strathclyde university, previous work in hydrological simulation has been mainly undertaken on mainframe computers. These had included regional multi-access systems such as Numac (Newcastle-Upon-Tyne) (Wardlaw, 1978), and ERCC (Endinburgh) (McKenzie, 1983). The River Basin Management system was initially constructed on the Strathclyde Vax computer system. The rapid growth and popularity in the use of micro computers and the increase in their memory size and capabilities necessitated the transfer of the system to the widely used IBM compatible Amstrad 1640 with a coprocessor or hard disk. Success was achieved in compiling and running the system on this micro computer though run time was much longer than on the

mainframe for all runs, monthly, annual, or multi-annual. An annual run could take as long as 10 minutes compared to 2 minutes on the mainframe. The realistic computer run time achieved for short term simulation periods, makes the micro computer based management system ideal for real time operation studies and forecasting. Its potential for long

term planning cannot be overemphasized considering the finacial commitments and risks involved. Thus, the use of more powerful micro computers is justified for long term simulation studies.

Details of the methods of formulation, structure, and workings of the water and sediment model components of the system are provided next.in this chapter while the economic component is discussed in chapter 6 .
As economic and financial matters play an important role in River Basin Management, their role is discussed in the next chapter, prior to the description of the economic simulation model.In addition, a review of economic methods used in water management and a description of the methodology utilised in incorporating these into the economic simulation model of the system are also provided.

5.4 The Watershed Model

The watershed model's representation of the hydrological cycle is conceptual and complete. The model is continous in its representation of the time variant interaction of processes. A total of 34 parameters are included in the no-snow version of model or a no-snow run of the complete model. These are mostly assessed from topographical maps and existing hydrological data. Others, which are inassessible or unmeasured are obtained by calibration. The main parameters obtained by calibration are the nominal upper zone storage (UZSN), the nominal lower zone storage (LZSN), the infiltration rate (CB), and the interflow component (CC) (Fleming, 1975).

The watershed model forms the initial base of the Strathclyde River Basin Model and the River Basin Management System assembled herein. The shaded section of figure 6.1 shows the watershed model in relation to the River Basin Management system. Outputs from the model include simulated flows, water yields, overland flows, natural ground water recharge in any desired time step greater than or equal to 15 minutes. Hydrographs of daily flows for recorded and simulated cases are produced by the model for visual comparison, calibration and sensitivity studies. The model has been widely applied in areas of widely differing climatic, hydrological, and geological conditions with much success (Fleming, 1970; Carr, 1973; Al Khadimi, 1977) and has been acclaimed by many authors (Bell, 1966; Ibbit and O'Donnell, 1971). Black (1973)

observes that the model has reached a state of completeness achieved by few others. Apart from its main output, the model creates time series input files for subsequent use by the erosion and routing models. These include throughfall and overland flow for the erosion and total flows for the routing models. These computer files act as the link between the models in the management system.

The model satisfies the following criteria:

(1) It represents the hydrological regimes of a wide variety of streams and rivers with a high degree of accuracy.

(2) It is easily applicable to different watersheds with existing hydrological data and its use can be extended to ungauged catchments.

(3) It is physically and relevant, since it estimates other useful data such as overland flow, actual evapotranspiration, and ground water recharge.

(4) It is process based..

(5) It use a small time interval, 15 minutes as its base time for calculating water balances and this accounts for a realistic interaction of soil-water processes.

The main processes represented by the model are extensively described, discussed and illustrated in the literature (Crawford and Linsley, 1966; Walker, 1978)

The criteria makes the model an ideal choice for a realistic representation of the behavior of hydrogeological processes in the River Basin Management System proposed in this work and conforms with the strategy of this work.

5.5 The Channel Routing Model

This model, which forms an important component of the river basin management system can be used for general flow routing, flood and low flow simulation and forecasting and other associated applications in channels and reservoirs. Sediment can also be routed through channels and reservoirs. This discussed seperately in chapter 7

The complete model structure is given figure 4.12 of chapter 4. The model utilises the approach of the solution of the continuity and momentum equations, representing unsteady and non-uniform flow in open channels, thus:

$$d(Q)/d(X) + B * [d(Y)/d(t)] = q$$

and

$$Sf = So-[d(Y)/d(X)]-[d(V)/d(t)]*[V/X]-(1/g)*[d(V)/d(t)]$$

where Q is the discharge

X is the distance in flow direction

B is the surface width and Y is the flow depth

q is the lateral inflow and Sf and So are the friction and channel slopes respectively.

V is the flow velocity and g and t are the gravitational acceleration and time interval respectively.

These equations are conveniently expressed in terms of the Chezy or Manning forms of the resistance equations as follows,

Chezy:

Q = C*A * sqrt[(So-[dY/dX]-[dV/dg]*[dV/dx]-(1/g)*[dV/dt]]

Manning:

Q=(1/n)*R**(.67)*A*sqrt[(So-(dY/dX)-(V/g)*(dV/dX)-
(1/g)*(dV/dt)]

where C is the Chezy coefficient, R is the hydraulic radius, n is the Manning coefficient and A is the channel area..

Henderson (1963), classified flood waves in natural prismatic channels according to the number of slope terms in the momentum equation. Where So is large in relation to the other three terms, the flood wave is classified as kinematic, and the momentum equation reduces to the Manning's equation:

Q = (1/n) * R**(2/3) * A * sqrt(So)

The classification of So assumes that it is not in the range of a mountain torrent, and that on gentle slopes dY/dX is not significant, though the use of variable reach lengths can overcome this. Application of the kinematic

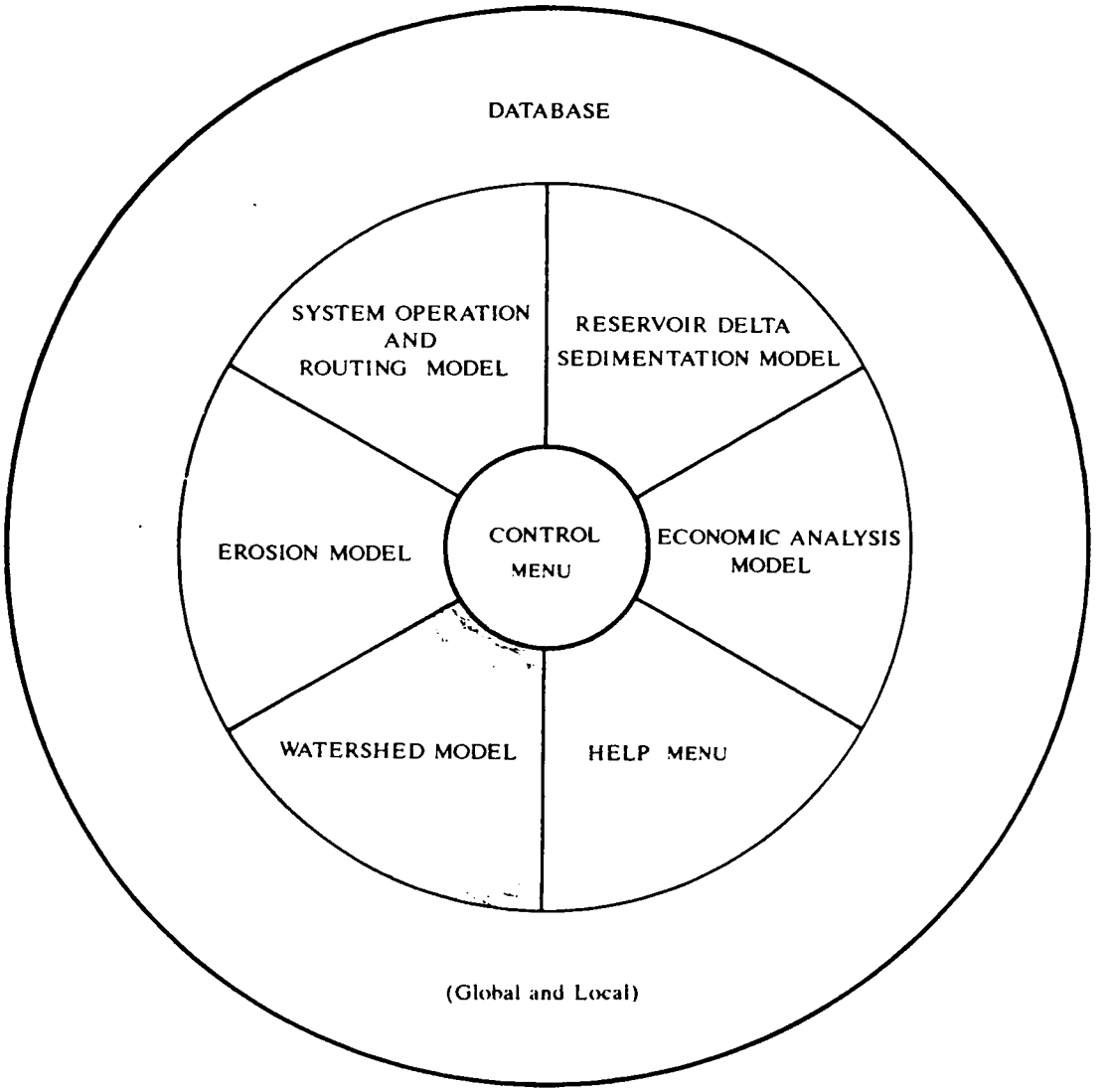


Fig 5.1a RIVER BASIN MANAGEMENT SYSTEM

This system is available for use on both micro and mainframe(VAX) computers.

routing technique requires data on slope, roughness and cross-sectional dimensions of channel reaches used in an analysis.

The model incorporates an algorithm for varying the routing time interval. During overbank flow, two flow areas are considered as shown in fig 5.5. In-bank flow is governed by channel roughness, reach slope and bankfull hydraulic radius while overbank flows are governed by the flood plain roughness, reach slope and the bankfull depth.

Reservoir routing is accomplished by the use of the level pool method. The hydraulic computations of the routing model are used directly in the monitoring of sediment transport (routing) and deposition in both channels and reservoirs. Although the model may be run without sediment computations, the sediment routines form an intrinsic part of the model. The details of parameter computation are given in the appendix and in the appropriate user guide available at strathclyde university.

5.6 Modifications to the Watershed and Routing Models

A more detailed discussion of the modifications made to existing models in the river basin management system outlined in chapter 3 are undertaken herein for the watershed and routing components.

This writer has made one modification to the watershed model. A factor for varying daily evaporation has been included to assist in investigating the sensitivity of

model response to evaporation.

The modifications to the routing model include, the introduction of elevation-capacity releases facility required for multipurpose reservoir operation. This is in addition to the daily diversion option which allows water withdrawals from both channels and reservoirs, already available in the model (McKenzie, 1983). Both diversion facilities broadens the model's spatial use and makes it more functional and flexible for operational purposes. These diversions are summed up as required, annually or otherwise, for economic analysis.

A modified algorithm for evaluating reservoir evaporation losses included in the the model has been outlined in section 5.1. This is useful in the study and assessment of reservoir evaporation problems for design purposes. The model computes lake evaporation by multiplying the instantaneous values of measured (or computed) open water evaporation with instantaneous reservoir surface area. All the computed values are cummulated for the whole year of analysis. Both instaneous and annual values are output as requested. The instantaneous reservoir surface area is computed from the reservoir area - elevation curve relationship which is a model input. The methodology adopted by the model for carrying out the above modifications involving releases and reservoir evaporation are described next.

The model calculates both the evaporation loss and the flow over the spillway by the use of a subroutine called rescalc. The subroutine stores data applicable to the reservoir which must first be calculated. The data takes the form of a table which has a maximum of twenty selected volumes. For each capacity value, an appropriate water depth or elevation, pre-set spillway flow volume, surface area, and pre-set release (in accordance with the chosen rule) are all given.

For any known volume of water in the reservoir, the spillway flow, reservoir evaporation, and the release can all be calculated by interpolating between the appropriate values continously for each time increment through out the specified simulation period. The same subroutine could also be used to calculate the length of the reservoir which at present is calculated by means of a mathematical relationship between the length and the depth. Table 5.1 is a typical computer print out of the reservoir table used for the stated evaluations.

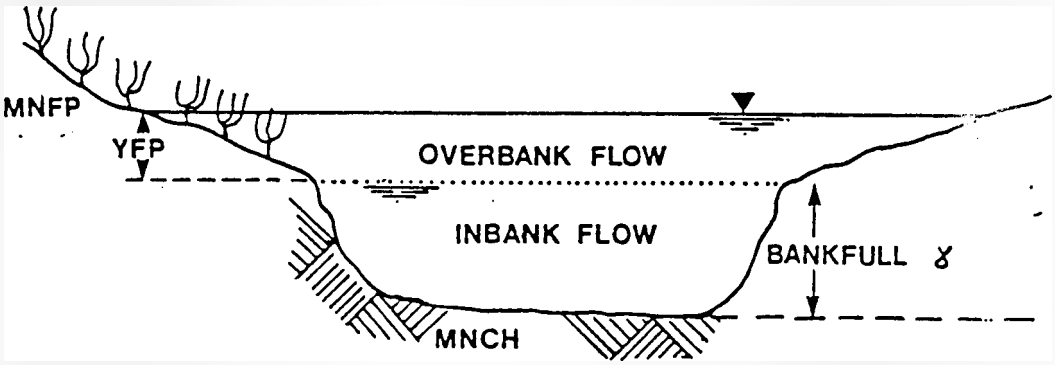
The annual releases, are used by the model to compute the annual power and energy generated, according to a specified fraction of the diverted water assigned to hydropower generation. The reservoir evaporation losses (cumecs), having been cumulated for the year, is converted to power and energy losses (Mw) for comparison with the actual values generated. The model, originally, outputs spillway discharges (cumecs) from each reservoir reach. The monitoring of evaporation and spillway loss occuring in the

TABLE 5.1 TYPICAL RESERVOIR DATA ARRAY

```
DIVERSION INDEX (1=YES 0=NO)     0
CHANNEL CONSTANTS                LENGTH(M)        SLOPE(-)         MANN             WMAX
                                 0.1000E+05       0.1000E-03       0.2000E+00       0.2500E+01
TRIBUTARY AREAS TO REACH         SR01(KM*KM)      SR02(KM*KM)      SR03(KM*KM)
                                 0.1140E+02       0.2000E+01
CURRENT REACH TRIBUTARY TO       REACH
                                   9
HYDRAULIC RADIUS PARAMETER       CR1              CR2(M**M)
                                 0.1050E+00       0.2000E+00
FLOW DEPTH PARAMETER             CY1              CY2              CY3
                                 0.2500E-01       0.2000E+00       0.1100E-01


REACH BREACH TYPE                 RES


DIVERSION INDEX (1=YES 0=NO)     1

DIVERSION INDEX (1=YES 0=NO)     0
ENTERED SPACER RECORD
REACH CONSTANTS                  HDAM(M)          WID1(M)          WID2(M)          SS
                                 2.7600E+00       0.2000E+03       0.3000E+03       0.2500E+02
TRIBUTARY AREAS TO REACH         SR01(KM*KM)      SR02(KM*KM)      SR03(KM*KM)
                                 0.4000E+02       0.2000E+01
CURRENT REACH TRIBUTARY TO       REACH
                                   9
RESERVOIR PARAMETER(1)           C1LEN            C2LEN            VELUP            VELMIN
                                 0.3333E+00       0.2000E+01       0.1000E+01       0.1000E-03
                  0.1770E+000.1540E+03
                  0.0000E+000.1150E+040.0000E+000.0000E+000.0000E+00
                  0.1000E+070.1200E+040.0000E+000.0000E+000.2500E+01
                  0.1770E+000.1020E+00+0.0000E+000.1500E+070.2500E+01
                  0.2000E+000.1040E+040.2400E+040.3300E+070.2500E+01
                  0.1175E+000.1050E+040.3470E+040.5500E+070.2500E+01
                  0.1040E+000.1056E+040.3470E+040.6600E+070.2500E+01
                  0.0000E+000.3000E+000.0000E+000.0000E+000.0000E+00
                  0.0000E+000.0
```

Figure 5.7b

SKETCH OF A CHANNEL SECTION WITH OVERBANK FLOW



MNCH CHANNEL ROUGHNESS
MNFP FLOOD PLAIN ROUGHNESS

system is useful in obtaining the efficient operation and management of the reservoir.

These modifications have effectively broadened the scope of the model from a flow routing program into a complete system operation model suitable for power generation and multipurposes. The model operates both annually and multi-annually.

5.7 The Strathclyde Erosion Model Structure

5.7.1 General

The use of mathematical models to represent complex sediment processes is increasing, especially as more powerful computers have become readily available and the understanding of sediment processes improves. One such sediment model as indicated in a previous section of this chapter is the Strathclyde Sediment Model. The model, which is able to assess, erosion rates and sediment yields from watersheds has been incorporated by the writer into the River Basin Management System (figure 1.8; 5.1).

The structure and theory of the model, previously called the land sediment erosion-transport-deposition model has been outlined earlier, as part of the description of the River Basin Management System. This model component utilises hourly time series rainfall and runoff generated by the watershed component of the system and stored in computer files. When simulating the soil eroded from a land surface due to the actions of rainfall and runoff, the model (like the Watershed Model) represents up to three segments by a relief or altitude criterion defined by the user. Each segment is characterised by a width, length and slope, the width being apportioned between that which borders the channel network, and is evaluated from topographical maps. The amount of soil in each particle size fraction, available to the eroding agents is represented by a top soil storage. This storage is enhanced or depleted by the input or output rates of sediment gain or loss. The model constrains this by

maintaining the total mass of the top soil storage at a constant value, i.e the same amount of total soil is made available to the agents of erosion. This is achieved in the model by the use of a top soil exposure function (TSE). The distribution of the particles within the topsoil store, is variable.

The erosion output rate depletes the storage of the most easily erodible particle sizes. In the absence of disturbance, therefore, the particle sizes of the top soil becomes progressively coarser with time and the layer becomes armoured. The erosion from an upper slope fills in sediments to the lower slopes. A net deposition occurs, if the rate of sediment supply to a lower slope exceeds its erosion rate. Sometimes, in very gentle slopes, deposition is the dominant process. Extreme events, however, do wash out such sediment accumulations into the channel system.

Apart from erosion, disturbances from natural or man induced activities, feeds the topsoil store, thus increasing the fines to the armoured surface layer. The model accounts for the influence of vegetation by representing the measurable physical characteristics of vegetation cover, and height, thus accounting for the interception process as it affects the energy of rainfall impact on soil particles.

The model is parametric and utilises the concept of dynamic elements described by Fahmy (Fahmy, 1976). Fleming (1970) concludes that sediment processes are extremely time and space dependent and that a sediment assessment

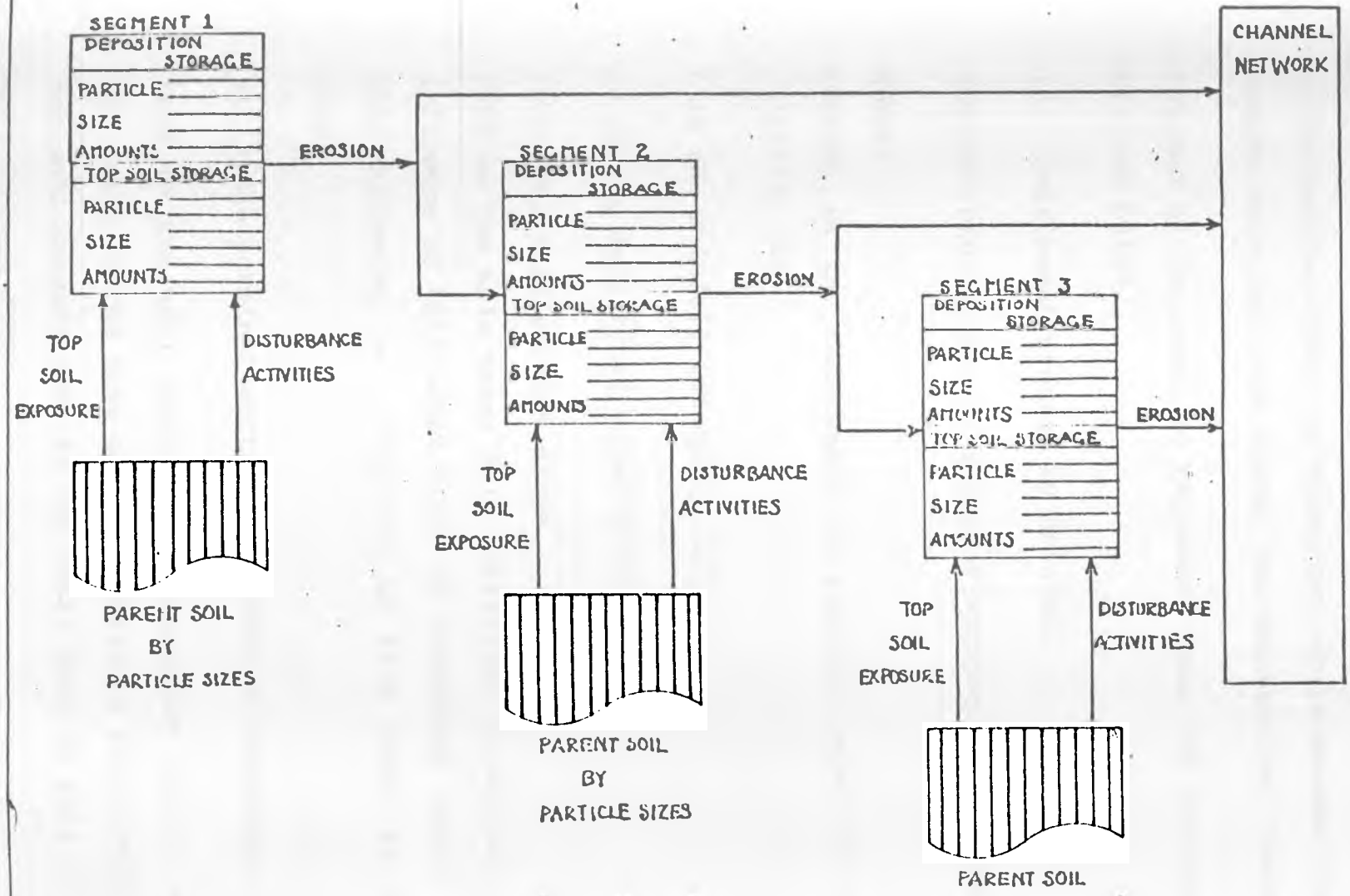


Fig. 5.6 (a) Flow chart of the segmented land erosion model showing particle size treatment and the interaction with the channel phase

model must include dynamic interaction. The dynamic erosive agents considered by the model are rainfall and runoff and the erosive process is subdiveded into two sub-processes, sediment detachment and transport. The multiplicative effect of these two sub-processes gives the erosion rates per unit width. The mathematical function

utilised by the model to represent these sub processes are outlined below.

(a) Detachment process by Rainfall

DETACHP=(CAPL*I*Pw*cos SS*TWFM*Vt*SAVP*OCAF)/(CDTCHP*Vs)

where:
DETACHP is the detachment of particle size by rainfall (M/(L**2) /T)

I is the rainfall intensity (L/T)

Pw is the density of water M/(L**3)

SS is the soil surface slope
TWFM is the thin water film multiplier to account for the influence of thin water layer on waterdrop impact forces. This parameter is a function of flow depth to raindrop size.

Vt is the terminal settling velocity of raindrops.

SAVP is the soil availability parameter and is equal to the ratio of the mass of soil available for erosion in the top soil storage (TS) to the total mass of soil in the top soil storage (TTS).

OCAF is the organic content adjustment factor to account for the rainfall energy dissipated in dislodging organic material on the soil surface.

CDTACHP is the coeficient of detachability to account for the detachment of small size particles (250 um by Farmer, 1973). where upon detachability is governed by interparticle bonding. For particle sizes greater than (250 μm) this coefficient is set to unity.

Vs is the terminal settling velocity of soil particle.

CSPL is a coefficient equal 1.52.

(b) Sediment Transport by Runoff

The mathematical function used in the model describing the transport length of particlews by rainsplash is based on the experimental work by Mutchler and Hansen (1970) and Mutchler and Young (1975). The mathematical function utilised is a function of the maximum width of crater resulting from raindrop impact and land surface slope as follows:

S=CEROP*REPDS*Sin SS

where:

S is the transport distance (L)

REPDS is the representative drop size of rainfall,

CEROP is a dimensionless coefficient.

(c) Detachment by Runoff

Overland flow occurs before runoff has appreciable detaching capacity due to insufficient flow depth. Thus

this type of erosion was set to zero. Detachment, therefore, is mainly by rill flow which is modelled using Young's unit stream power relation (Young, 1972, 1973, 1976) as follows:

DETACHQ=CSCR*V*S*Pw*d*SAVP*OCAF/(D*CDTCHQ)

where

DETACHQ is the detachment of a particle size by runoff (M/(L**2)* T)
V is the velocity of flow (L/T)
SS is the land surface slope,
Pw is the density of water
d is the flow depth,
SAVP is the soil availability parameter and is equal to the ratio of the available soil in the top soil storage to the total mass of soil in the top soil storage (TTS),

OCAF is the organic content adjustment factor to account for the detachment of small size particles (200-400) mm (Farmer, 1973) whereupon detachability is governed by interparticle bond.

BCSCR is a dimensionless coefficient,

D is the particle size,

CDTCHQ is a coefficient of detachability.

(d) Sediment Transport by Runoff

The equations used to model the transport distance of a sediment particle due to runoff, both overland and rill,

are based on the forces acting after detachment are as follows:

(1) For Overland flow erosion,

S={(FD + W sin SS)/(W cos SS)} Y * CEROQ

(2) For Rill erosion,

S={(FD + W sin SS)/(W cos SS - FL)} Y * CEROQ

S is the transport distance,

FD is the drag force on particle (M/LT**2)

W is the bouyant weight of soil particle (M/LT**2)

SS is the soil surface slope,

FL is the lift force on soil particle (M/LT**2),

FL = CLIFT * W

CLIFT is the coefficient of lift,

Y is the uplift distance of a particle (L),

Y = D * (Vq/Vd)**(EXPY)

D is the particle size,

Vq is the flow velocity,

Vd is the terminal velocity of soil particle in water,

EXPY is an exponent,

CEROQ is the coefficient of runoff erosion.

(e) Top Soil Disturbance

The model accounts for this process and simulates the rate of soil generation from the bottom sublayer and for each particle size faction. The mathematical functions used is as follows:

DIS (I) = CDIS * FSBL (I),

where:

DIS (I) is the disturbance of soil particle size (I),

CDIS is the top soil disturbance rate,

FSBL (I) is the fraction of particle size (i) in soil sub-layer.

The discussion above has been focussed mainly at sediment processes in the land phase of the river basin.Thus, this discussion of the sediment catchment processes would be incomplete without an accompanying assessment of sediment routing in channels as illustrated by the description of the Strathclyde sediment channel routing model undertaken below.

5.8 The Sediment Channel Routing Model

In addition to the erosion model, there is a channel sediment transport module which is linked to results from the sediment erosion model but is actually an integral component of the channel and reservoir flow routing model. This structure is inevitable as flows are a requirement for the sediment routing process. This transport sub-model utilises the time series outputs of sediment yields emanating from the land surface into the channel system as generated from the land erosion model. This model utilises Bagnolds sediment transport relation (Bagnold, 1966). Bagnold's relation, unlike others, is exclusively valid for situations where the volume of sediment is limitless. It is specifically selected to account for sediment eroded from the land surface to the channel system.

The relation is based on defining the total energy which the stream of flowing water possessed. From physics, the total amount of transported sediment particles of a specified diameter is evaluated by the determination of

the portion of total stream flow energy which is available to them. The relation gives a total sediment load which consists of two parts, bed and suspended. The relation defines bed loads as the total load wholly supported by solid transmitted stresses and suspended load as that wholly supported by fluid transmitted stresses. This model also routs sediment through channels and reservoirs and accounts for sediment deposition as well.

The sediment routing is carried out through each defined reach of a channel network, inclusive of reservoirs. A reach may receive sediment required for erosion by three sediment sources, namely, the land surface area directly linked to the reach, the reach immediately upstream, sediment on the top layer of the channel bed. Each of the above group of sediments is described by a particle size and mass. Thus a total particle size distribution is formed of available sediment for each reach and for each routing interval.

A transformed shields diagram is used to define the critical particle size less than which there is no motion. The critical particle size less than which the motion is by suspension is calculated using the Bagnold approach (Bagnold, 1966). This effectively divides the sediments into three types under a particular or current flow conditions. These are sediments exhibiting suspended load and bed load movements as well as those at rest. The actual transport amounts are obtained by the comparison of the amount that the flow is capable of carrying as

determined by Bagnold's relation, with the sediment availability. Thus the process of channel bed armouring, scouring, and deposition can be simulated in a physically based fashion from the flow parameters in a reach (i.e bed shear, flow depth, etc). The complete erosion-transport-deposition chain are simulated throughout each reach in the channel network. The relationships utilised in the model are outlined below.

(a) Critical Particle Size for Bed Load Movement

A modified Shields diagram is utilised. The relationship used is

Dcrit = (Y * V)/U*

where:

D is the particle size of sediment, Dcrit is the particle critical size

U* = sq rt (To/Po) , U* is the shear velocity,

V is the kinematic viscosity,

Y is the critical value of ((U*)D/U),

To and Po are shear stress in bed and density of water respectively.

(b) Critical Shear Stress for Suspended Load

Bagnold suggested that some upward velocity exists which equals the fall velocity of the particle, during the critical condition for suspension. This velocity exists as a spectrum of upward velocity eddies with a mean, Vup. This upward velocity resists the fall velocity suspended particles in flow. The critical point for suspension occurs when the mean upward velocity of eddies near the

bed is equal to the particle terminal fall velocity. Bagnold relates the mean upward velocity to the mean velocity at the bed by:

Vup =1.56 V where V = 0.8 U*

Therefore, Vup = 1.25 U* = 1.25 sq rt (To/P)

The critical condition for sustained suspension occurs when Vup = V

Hence, V = 1.25 sq rt (To/P)

(c) Flow Capacity for Sediment Transportation

The Bagnold approach for sediment load transportation is used by the model to compute the bed load and suspended load transport rates of individual particle sizes from the already defined the particle sizes available for transportation as bed and suspended loads. THe concept defines suspended load as that portion of solids

which is supported wholly by a random succession of upward impulses imparted by eddie current of fluid turbulence. Bed load occurs as rolling, sliding, or saltation and upward impulses are due only to contact with stationary grains in the bed. that is bouncing. Saltation is the natural form of bed load movement, the limiting case being particle sliding along the bed. The equations utilised for computing sediment transport rate by the model are given below.

(1) Bed Load Transport Rate

Ib = Eb * W/tan a

where:

Ib is the bed transport rate of solids by immersed weight

per unit width,

Eb is the efficiency of bed load transport

Eb = w/P, w is the work rate of bed load and P is the available power supply per unit area.

Eb = To * U/(To * V) = U/V

U is the mean velocity of bed,

V is the mean velocity of flow,

W is available power per unit bed area equal pgQS/b where Q, S, and b are flow, energy slope, and width of flow respectively.

tan a is the coefficient of solid friction which is the ratio of tractive stress needed to maintain transport to that of normal stress due to the immersed weight of the load.

(2) Suspended Load Transport Rate

$$IS = w.(1-Eb) * Es * (Us/Vs)$$

where:

Is is the suspended transport rate by immersed weight per unit width,

Es is the efficiency of suspended load transport,

Us is the mean velocity of the suspended particles approximately equal to mean flow velocity, V,

Vs is the particle settling velocity,

W*(1-Eb) is the available power supply for suspension after the dissipation of energy used by bed load transport, with W and Eb as defined previously,

The term (1-Eb)*Es was estimated to be 0.01 and the

equation for suspended load transport is thus reduced to:

$$Is = 0.01 * W * V/Vs$$

(c) Total Transport Load

$$I = Ib + Is$$

$$I = W*((Eb /\tan a) + 0.01 * V/Vs)$$

where:

I is the total transport load in immersed weight per unit width

(d) Calculation of Sediment Transport Rate

The rate of sediment transport through a channel reach is calculated by comparing sediment available for transport and the capacity of the flowing water to transport available sediment, the latter is calculated as above. The available sediment is that contributed from the land surface, upstream reach or channel bed. If the available sediment is greater than the transport capacity of the flow, the transport capacity limits the amount of sediment transported and deposition occurs. The reverse causes net scouring of the channel bed. To account for the sediment available at the bed, the principle of conservation of mass is applied to each reach for each sediment size, as follows:

$$d(BEDS)/d(t) = SDS + UPS + TSE - DEPS - TTR$$

where:

BEDS is the top bed storage,

SDS is the land surface sediment inflow rate,

UPS is the upstream and tributary reaches sediment inflow rate,

TSE is the exposure rate of sub-soil, equal to zero in new deposition case.

DEPS is the deposition storage amount, equal to zero in new scour case

TTR is the total transport rate = bed load + suspended load.

Prior to the presentation of test results of the sediment erosion and routing models, whose processes are described above, their potentials are first assessed.

5.8 The Potential and Limitations of the Erosion and Sediment Routing Models

The sediment erosion model has many potentials which include the estimation of gross and net erosion rates and sediment yields on a monthly and annual basis, updating or extending sediment records, estimating channel and reservoir sedimentation rates, bed sediment distribution, scouring, and estimating the effect of changes in land use or runoff on sediment yields.

The effect of physical changes to the land surface and upstream channel network on reservoir deposition and the distribution of sediment sizes along the reservoir can be assessed. Such an assessment would be important in a quality study of the reservoir bed. Toxic chemicals from fertilisers, pesticides, and fishing as well as plant nutrients tend to adhere to finer soil particles and would

thus concentrate in places where these fines are deposited. The sediment deposition in reservoirs helps in any proposed dredging operation to recover lost storage volume.

The input requirements for all components of the combined model are in general easily measurable and so its use and application is practical. Walker tested the erosion model on many catchments and obtained realistic results. The results of his test of the reservoir sedimentation model on the Esk Reservoir near Edinburgh, Scotland is also realistic and are presented in Walker (1979).

Fleming's relationships have been incorporated into the sediment erosion model by this writer as an alternative to the universal soil loss equation for estimating soil erosion rates and sediment yields. Some results were obtained when these were applied to the test Donga catchment. The reason for the incorporation is to make the model more universally applicable, as different catchments tend to have differing erosion behaviour. The results are indicated in table 7.4.

The main limitation of the sediment routing model which concerns reservoir sedimentation lies in its inability in readily providing results and plots of the profile of the delta formed by the deposited sediments. Thus, it is not possible to use the model in its present state to assess the volume of sediment deposits or to produce the elevation-capacity curves for reservoirs.

TYPICAL EROSION

METHOD	FIELD WORK	EROSION MODEL	FLEMING AREA	FLEMING S.GRASS	FLEMING T.VEGTN	FLEMING MIXED
units	(ton/sq km/yr)	(ton/sq km/yr)	(ton/sq km/yr)	(ton/sq km/yr)	(ton/sq km/yr)	(ton/sq km/yr)
ORIGINAL	2000	2200	307.4	2677	2168	405.4
RAIN+10%		26850	307.4	3152	2665	523.7
RAIN-10%		113.62	307.4	2215	1707	301.4
EVAP+10%		62.8	307.4	2468	1957	357.0
EVAP-10%		15600	307.4	2884	2382	455.6
KPV*5.0		2200	307.4	2789	2283	423.3
KPV=0.0		2200	307.4	2648	2139	398.7
EPIM*3.0		11890	307.4	2853	2144	400.0
EPIM*0.3		181.1	307.4	2694	2166	409.6

Table 5.2 SENSITIVITY OF CATCHMENT SEDIMENT YIELD TO CLIMATE AND LAND USE CHANGES DONGA RIVER BASIN YIELDS AT GEMBU RESERVOIR AND ABONG

NOTES ON TABLE 5.2

The Donga Basin ,from source to Abong town is the Mambilla Plateau with an elevation of about 2000m above m.s.l. This area consists mainly of grassland with taller vegetation and woods in valleys as shown in Plates (A to E),. The results shown in Table 5.2 indicate close simulation for the Erosion Model as compared to Fleming's relationship for short grass/ tall vegetation for moderate runoff condition (current conditions). Departures are observed for very high and very low runoffs. This is probably due to the non-process empirical nature of Fleming's equations. Thus ,the relationships are not utlized further in this work since they do not conform to the process approach. In addition, the Fleming's area and mixed vegetation equations do not fit the Donga Basin conditions.

Another limitation lies in the dynamic constraint of time. This actual means that only one year of sedimentation output is realisable. An improvement that is necessary for an economic study involves extending its capability to the simulation of average values of long term sedimetation using a mean annual sediment inflow or time series data. These limitations have led to the choice of an appropriate delta simulation model for simulating sedimentation. in reservoirs in the management system. This model is next discussed.

5.9 The Reservoir Delta Simulation Model- Deltasim

5.9.1 General Assumptions and Base Theory

This model was originally developed by Bonham-Carter. G. and Sutherland. A. J as a base tool in the study and understanding of the delta formation process in river mouths, especially those entering the sea. The non consideration of ocean currents, tides, off-shore wave energy factors, and the tectonic behaviour of coastal environment has made its application more appropriate to rivers entering reservoirs. The only limitation is that the velocity fields utilised were developed using plane jet rather than an axial jet theory.

Plane jet theory tends to reflect the marine deposition case more realistically due to the density and salinity difference between the river and the sea. Bates (1953) emphasized the importance of river jet mixing in the delta formation process. He pointed out that a river discharging into water of equal density and salinity, such as a lake

or reservoir, would mix both vertically and horizontally, which would be similar to an axial jet. River water entering the sea, however, would be bouyed up due to the density difference between fresh and saline water, hence free vertical mixing between fresh and salt layers would be curtailed (or inhibited), and the system would be similar to a horizontal plane jet.

Further, Bates (1953) suggested that the position of submerged levees and distributary mouth bars are governed by the structure of the jet velocity field. In this model, Bates plane jet hypothesis is adopted and modified to include the depth-velocity variation in an open channel. This modification is extended by the consideration of the diffusion of sediments in this velocity field.

It can be argued that the plane jet theory is applicable to rivers discharging into reservoirs since reservoirs act as storages for salts and nutrients washed down from all parts of river basins. These salts and nutrients build up fairly quickly and hence the salinity and density of impoundment may become slightly higher than the river's in the long term. This assumption is likely to be more relevant in catchments susceptible to excessive erosion, especially nutrient and salt rock erosion. The Donga basin has a lot of erosion problems due mainly to cattle overgrazing and poor land use practices. Preliminary water quality assessment in the basin by Diyam (1985) indicated the presence of high concentration of nutrients and salts, especially ammonium salts and nitrates , in the river.

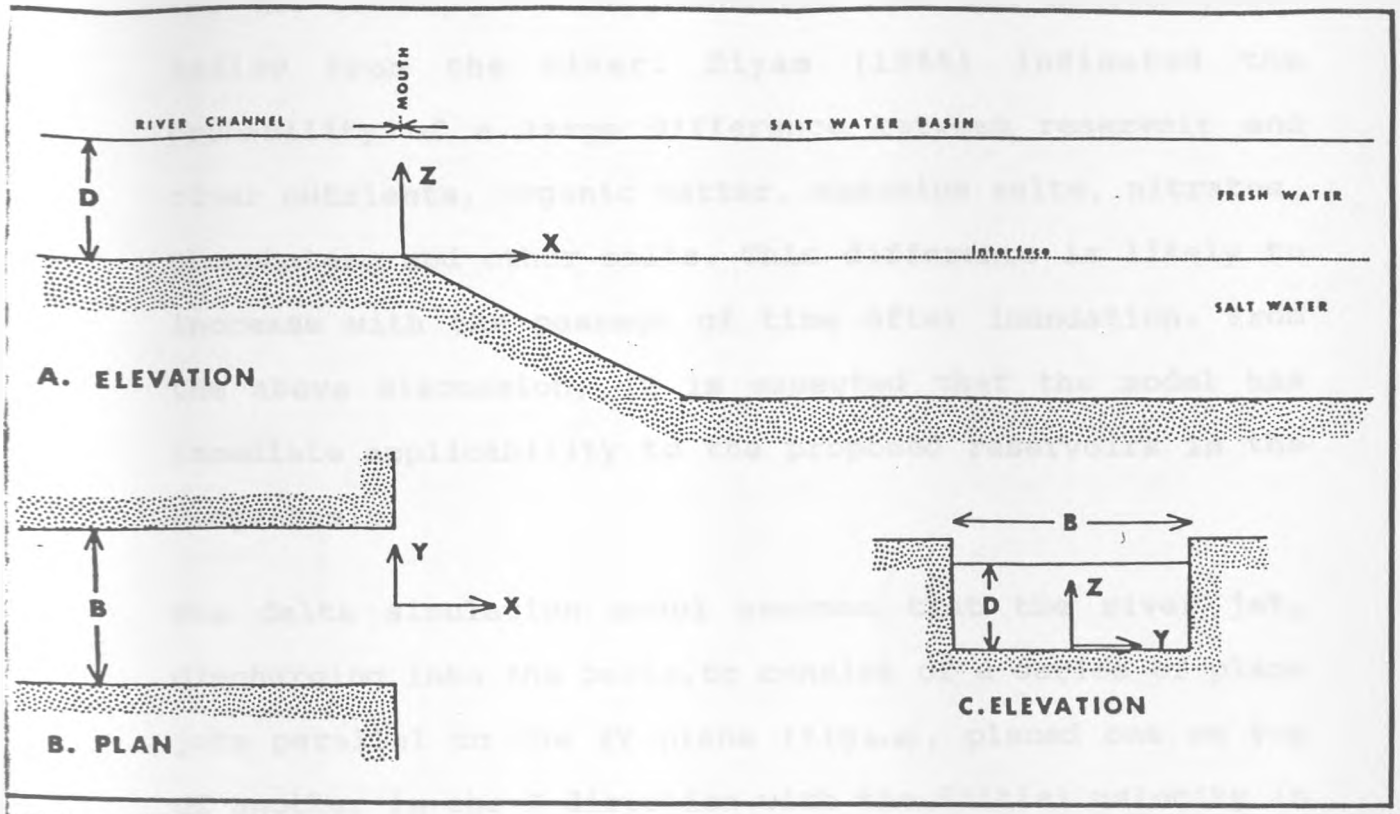
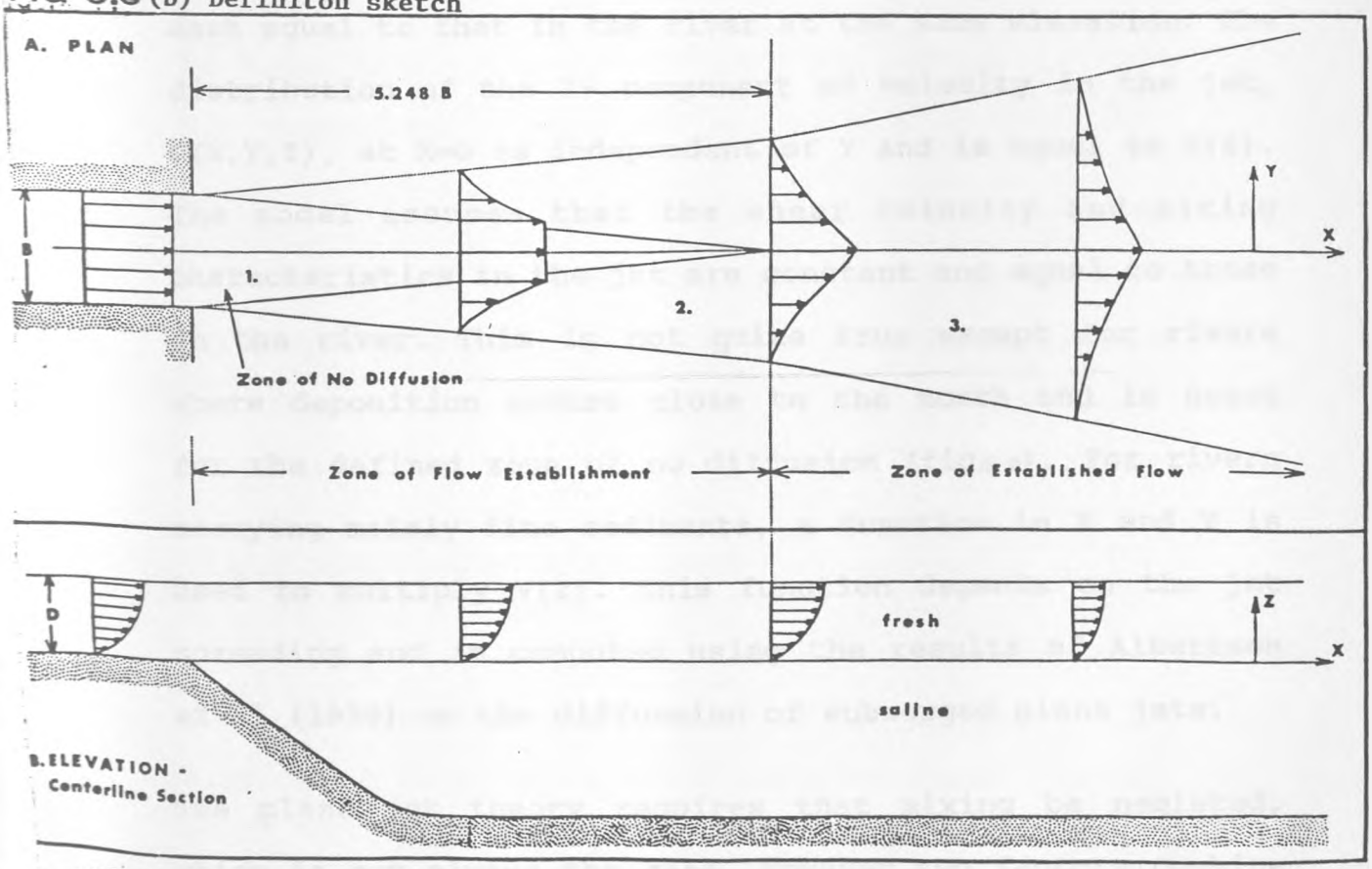


FIG 5.8 (b) Definiton sketch



5.9 (c) Structure of velocity field. Plane-jet velocity field is based on results of, and uses the terminology from, Albertson and others (1950).

The two factors that will probably be responsible for this are the leaching of the inundated land and nutrient/salt inflow from the river. Diyam (1985) indicated the probability of a large difference between reservoir and river nutrients, organic matter, ammonium salts, nitrates, phosphates, and other salts. This difference is likely to increase with the passage of time after inundation. From the above discussion, it is expected that the model has immediate applicability to the proposed reservoirs in the Donga Basin.

The delta simulation model assumes that the river jet, discharging into the basin,to consist of a series of plane jets parallel to the XY plane (fig5.9), placed one on top of another in the Z direction with the initial velocity in each equal to that in the river at the same elevation. The distribution of the X- component of velocity in the jet, U(X,Y,Z), at X=0 is independent of Y and is equal to V(Z). The model assumes that the shear velocity and mixing characteristics in the jet are constant and equal to those in the river. This is not quite true except for rivers where deposition occurs close to the mouth and is exact for the defined zone of no diffusion (fig5.9). For rivers carrying mainly fine sediments, a function in X and Y is used to multiply V(Z). This function depends on the jet spreading and is computed using the results of Albertson et al (1950) on the diffussion of submerged plane jets.

The plane jet theory requires that mixing be negleted, which is not always the case. However,two factors combine

to ensure that interfacial mixing is minimal. The velocities in the jet near the interface is small due to the velocity profile in the river. The density difference at the interface tends to inhibit mixing in that region. The mixing occuring at the edges of the jet is not affected by the density difference because mixing occurs mainly in an horizontal plane.

Finally, the sediment that reaches the interface is assumed to settle vertically to the bottom of the basin where it remains. Particles tend to settle vertically because their horizontal velocity which is small at the interface is rapidly reduced to zero, due to the viscous drag in the lower fluid

5.9.2 Model Structure

In the river, the velocity profile assumed is that of a two dimensional channel defined by the expression:

$V(Z) = \bar{V} + U*/K(1 + LOGeZ/D)$ where K is Von Karman's constant which is taken as 0.4 when running the model, evidence for smaller values of K in sediment laden streams has been presented (Vanoni, 1946). If dimensionless variables are used,

$$\frac{V(Z)}{\bar{V}} = f(Z) = 1 + U*/K(1+\ln Z/d)$$

Albertson et al (1950) predicted the streamwise velocity on plane jets in the Ocean Basin or reservoir. They distinguished three distinct velocity zones (fig 5.9).

(a) The zone of no diffusion, denoted (1) in fig 5.9 Here,

$U(X,Y,Z) = F1(Z)$

(b) In the zone of flow establishment (2), for constant Z,

$U(X,Y,Z) = f1(Z)\exp(-42.3*[0.096 + (Y-0.5)/X]^2) = f1(Z)*f2(X,Y)$

The multiplying function, f2(X,Y) depends on the jet spreading.

(c) In the zone of established flow (3), Albertson et al (1950) showed that for a fixed value of Z,

$U(X,Y,Z) = f1(Z)*X\exp(-1/2) \exp[0.828 - 42.3Y\exp(2)/X\exp(2)]$

$= f1(Z)*f2(X,Y)$

U(X,Y,Z) is the streamwise component of velocity.

Individual sediment behaviour during the transport and deposition phases is random, though the gross behaviour of total sediment load can be modelled deterministically. In the model, the river is subdiveded into a number of stream tubes and the total sediment travelling along any tube is represented by the nominal or statistical particle at the centre of the tube. The model concerns itself with the calculation of the proportion of the total sediment load represented by each nominal particle and then tracks the movement of the particle as they settle in front of the river mouth. The sediment transported in a channel is distributed unevenly with depths, so the actual quantity of sediment represented by a nominal particle will depend on the height of the particle above river bottom.

The concentration Cz of sediment at any elevation z from

the river bed with a settling velocity Ws is given by the formula,

$Cz/Ca = [((d - z)/z) * (a/(d - a)]^{exp\ P}$

where $P = Ws/(B\ K\ U^*)$

(after Vanoni, Brooks and Kennedy, 1961)

Cs is the concentration at elevation z

d is the ch annel depth

Ca is the reference concentration at elevation a

a is the reference elevation

Ws is the fall velocity of sediment in still water. B is defined as the ratio between diffusion coefficients of sediment and momentum respectively. Studies have shown that the value of B is close to 1, though Singamsetti (1966) reported values up to 1.4 for coarser sediments..

U* is the shear velocity, a measure of the frictional characteristics of the channel floor. K is Von karman's constant., a numerical factor that varies with the amount of sediment in suspension and is taken as 0.4.

Also, $S(Zi,Zj) = \text{Intergral } [f1(Z) * C_a[(d-z)/z * (a/(d - a)]^{exp\ P}\ dZ$ from limits Zj to Zi.

S is the suspended sediment discharge per unit width in the river between elevation Zi and Zj. If we define F(Zi, Zj) as a fraction of the total sediment discharge passing between elevations Zi and Zj then

$F(Zi,Zj) = S(Zi, Zj)/S(E,d)$.

the evaluation of this equation follows directly from the

last. The elevation Z = E is introduced because the velocity profile f1(Z) is not valid at the boundary layer near the river bed (i.e, below Z = E). E is taken as four times the maximum particle size.

Particle trajectories are calculated from expressions developed by the use of the double grid accounting and coordinate system, that of the river mouth which is vertical and that of the receiving basin which is horizontal. The methods of probability and statistics are extensively employed in defining the trajectories. Typical trajectories are given in fig 5.9 while details of trajectories' expressions are given in BCarter(68).

The model can be run both statically by the use of one time increment and dynamically with several time increments. The model output is impressive. An artistic impression of deposited sediments according to facies is output in the basin grid system. A contour of deposition according to sediment types are also shown with the basin grid as background. Statigraphic crossections of the prograding delta deposits are given in both the X and Y directions, with clearly defined lines for each time increment. Details of the model program structure, complete with subroutine and input data description, are clearly described in BCarter(68).

Finally, the main limitation of the model for use in reservoirs is the non or minimal mixing concept provided by the use of plane jet methodology. However ,its use may yet be practical enough for reservoirs with large sediment

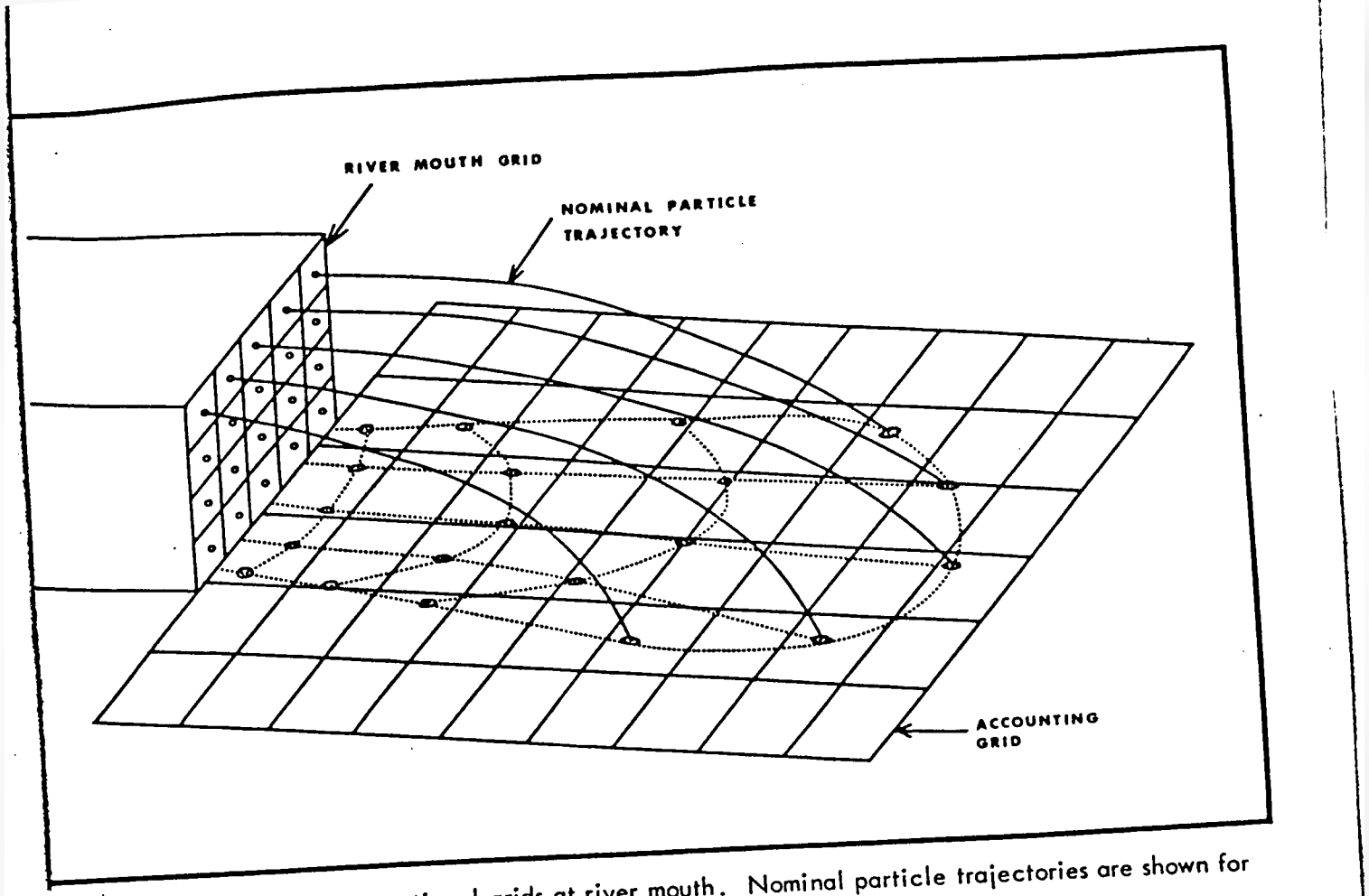


FIG 5.9 Orientation of computational grids at river mouth. Nominal particle trajectories are shown for uppermost row of cells in river-mouth grid.

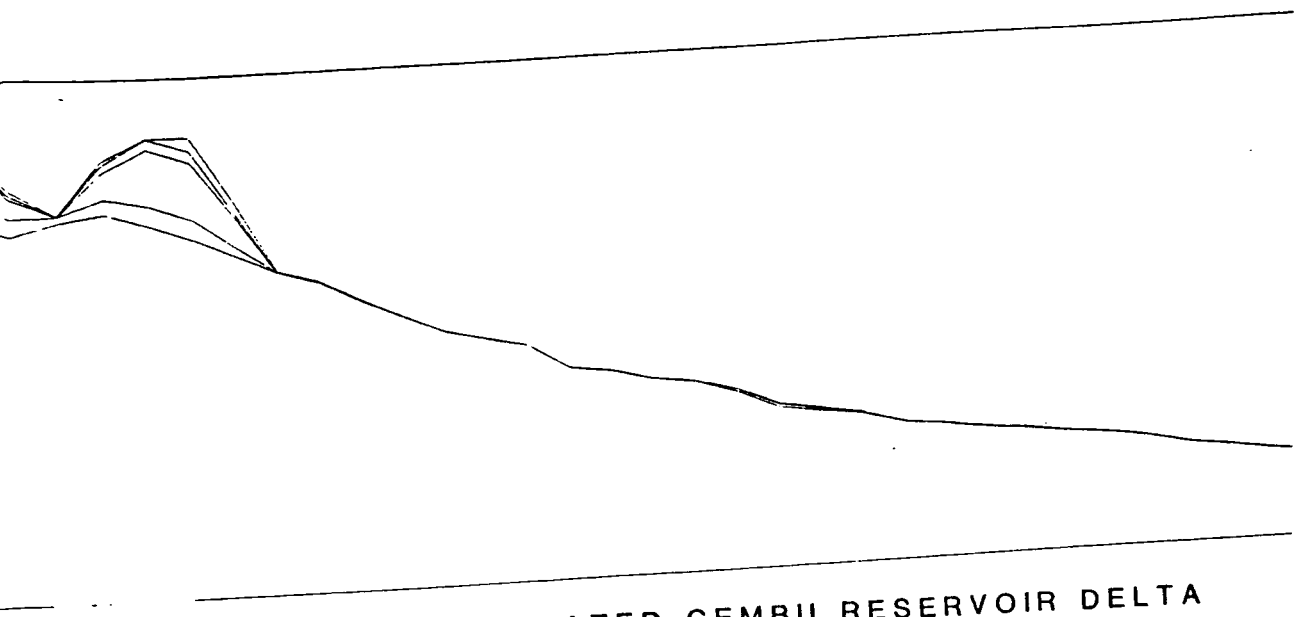
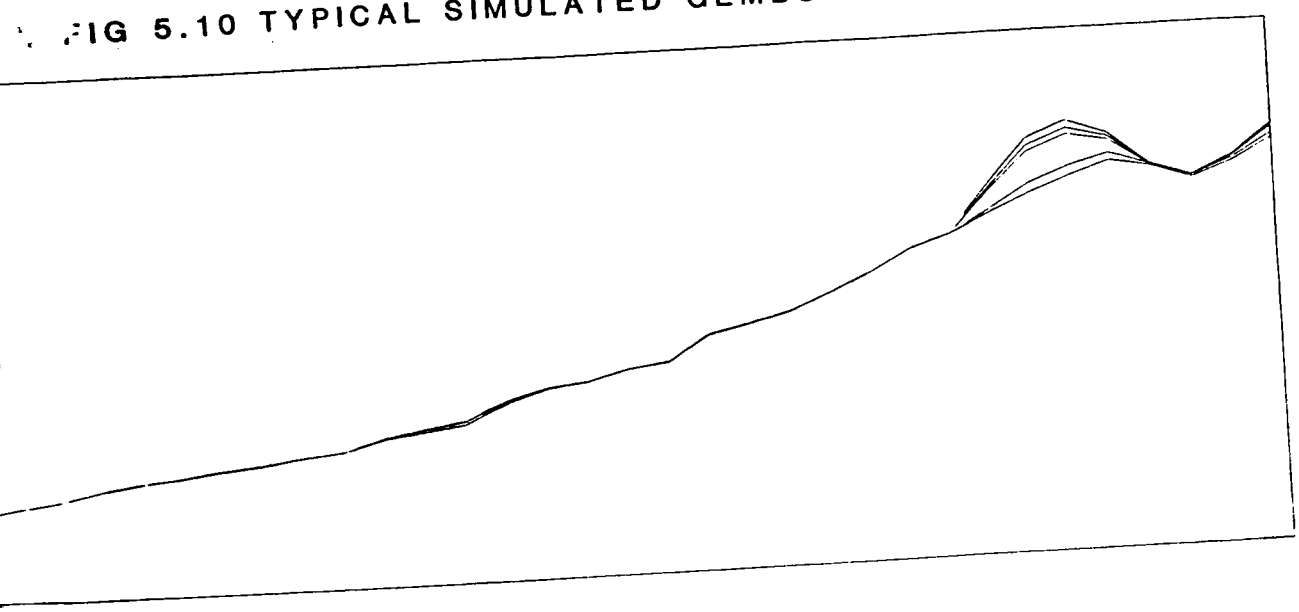


FIG 5.10 TYPICAL SIMULATED GEMBU RESERVOIR DELTA



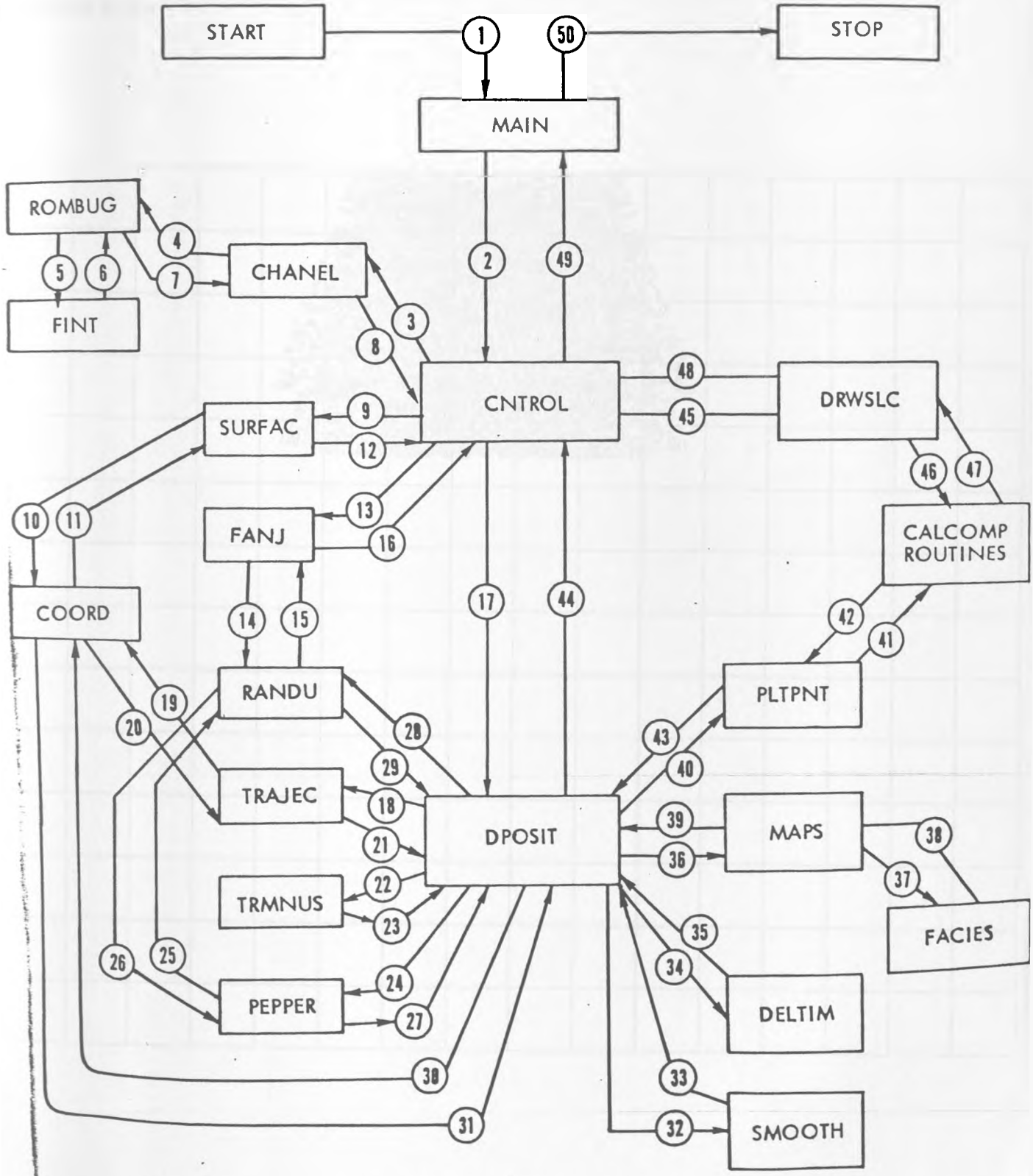


Fig. 5.11 a B.CARTER etal, 1968

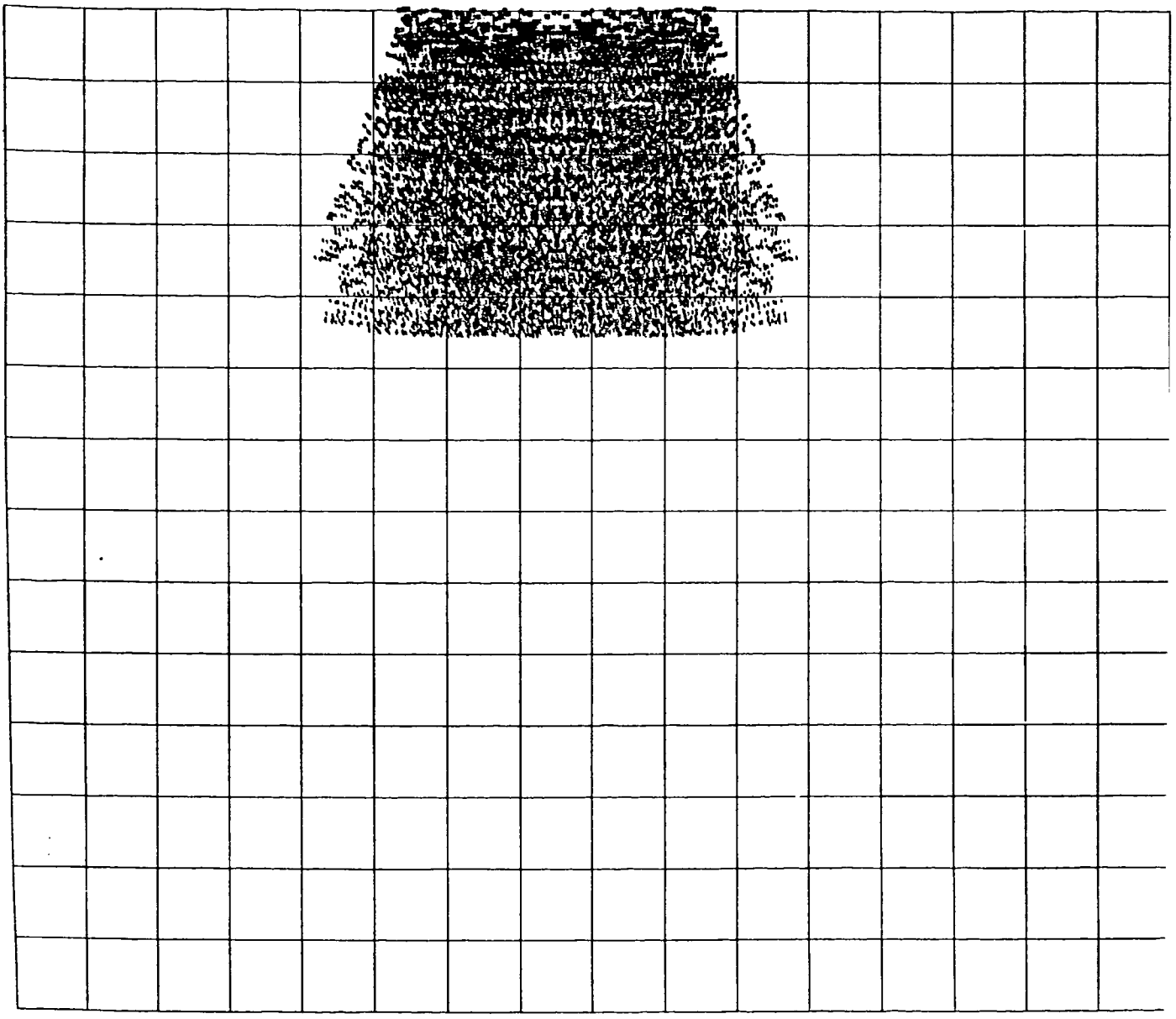
Flow chart indicating structure and direction of flow between subroutines of DELTASIM.

TEST DECK FOR DELTA SIMULATION

SCALE - 1 INCH = 10.658

PARTICLE DIAMETER= MMS. 1.000

INCREMENT NUMBER 1

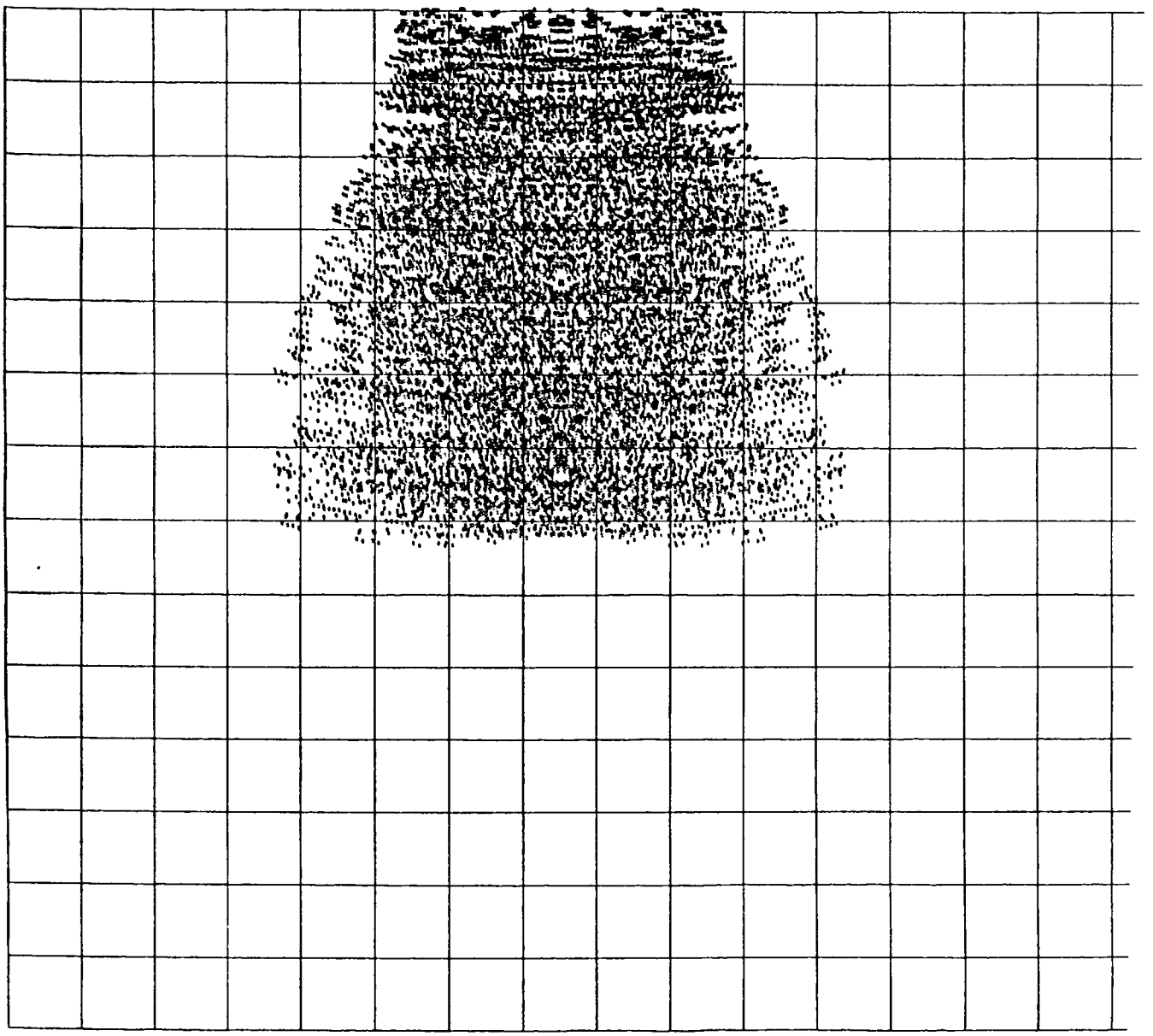


TEST DECK FOR DELTA SIMULATION

SCALE - 1 INCH = 10.658

PARTICLE DIAMETER= MMS. 0.500

INCREMENT NUMBER 1

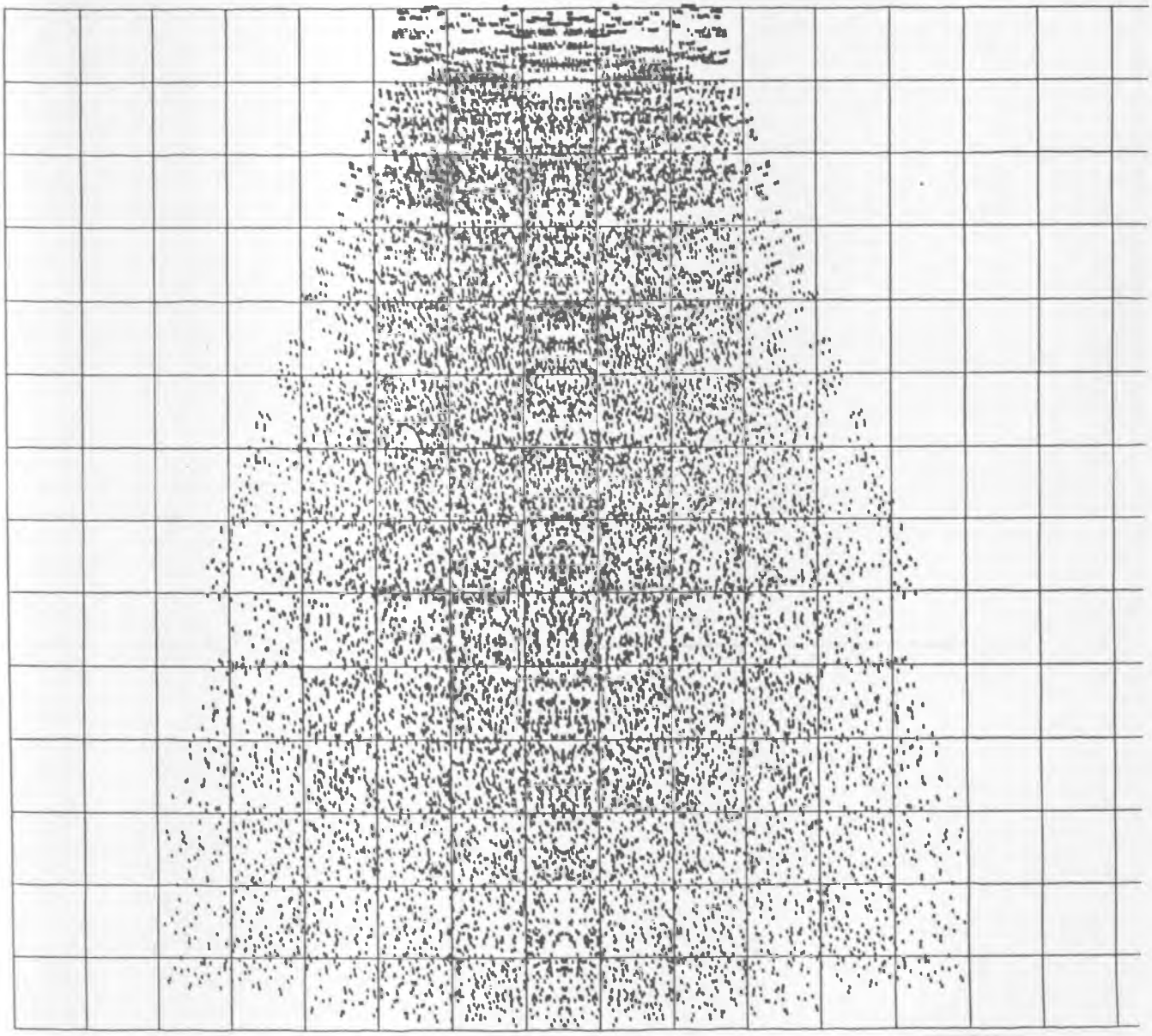


TEST DECK FOR DELTA SIMULATION

SCALE - 1 INCH = 10.658

PARTICLE DIAMETER= MMS. 0.250

INCREMENT NUMBER 1



inflows and a high proportion of dissolved salt or chemical load. On testing the model with a real life reservoir, unsatisfactory results, should lead to the incorporation of the axial jet mixing theory by future researchers.

FIGZ5.11 DELTSIM SUBROUTINES

and calls CNTROL.

Subroutine CNTROL

1. reads input parameters (see input instructions)
2. calls CHANEL
3. prints channel characteristics (see Table 4)
4. calls SURFAC
5. calls FANJ
6. calls DPOSIT
7. plots X-sections by calling DRWSLC

Subroutine CHANEL

1. computes channel parameters
2. finds maximum distances of travel along jet centerline for each particle size
3. scales a/c grid to fit depositional area if GRID.GT.0 (see input instructions)
4. calculates total sediment discharge for each grain size, if not already supplied as data, storing the result as (SEDLOD(L), where L is an index denoting grain size number)
5. calculates sediment discharge for each row of river grid for each grain size storing the result as (DISC(L,KK)), using ROMBUG and FINT
6. solves left-hand side of equation 11 for each row of river grid storing the result as HYT(KK)

Subroutine DPOSIT

1. sets up major DO-loop which is entered once per time increment
2. calls TRAJEC
3. sets up DO-loop which is entered once for each cell in river grid
4. calls TRMNUS
5. returns to (3) until all cells complete
6. sets up new loop which is again passed through for each cell in river grid
7. calls PEPPER
8. enters loop which is completed for each fan position
9. calls COORD
10. increments temporary a/c grid cell by appropriate quantity of sediment.
11. repeats 9 and 10 for the mirror image in the other half of river grid
12. returns to 8 until all fan positions complete
13. returns to 6 until all river grid cells complete
14. calls SMOOTH if NSM.GT.0 (where NSM is described in the input instructions)
15. calls DELTIM
16. updates water depth array, DEPTH (I,J)
17. writes out sediment budget (see Table 12)
18. calls MAPS
19. returns to (1) for each time increment
20. calls DELTIM

Subroutine TRAJEC

1. if this is the first time increment, calculates x_d (equation 6) y_d, y_0 (equation 7) for each column of the river mouth grid.
2. calculates position of delta lip for particles issuing from each column of river grid storing the results in (XLIP(K), YLIP(K))
3. calculates maximum values of the first two integrals on right side of equation 11; note that the lower bounds change as the delta lip advances; values for each column stored in SUMZND(K), SUMZFE(K)
4. writes out a table of trajectory constants, one row of constants for each column in the river mouth grid (see Table 11)

Subroutine SURFAC

1. calculates a value of LDS which is stored in (DEPLIM(I,J)) for each cell in the a/c grid
2. if KSRF.LT.1, the stream depth is set in each cell of the a/c grid
3. if KSRF.GE.1, the LDS value is made a function of the local velocity; a total of KSRF points are calculated (on a line normal to jet axis) and averaged for each a/c grid cell

Subroutine TRMNUS

1. calculates x_t, y_t, the terminal coordinates of a nominal particle, given the cell inaices in the river grid (see Appendix B)

Subroutine COORD

1. if KON = 0, jet coordinates (x,y) are transformed to a/c grid coordinates (I,J)
2. if KON.GT.1, a/c grid coordinates are transformed to jet coordinates

Subroutine DELTIM

1. finds indices of a/c grid cell that will fill to its LDS value more rapidly than any other cell
2. calculates the length of time required to fill this cell which becomes the length of the time increment (DT)
3. stores indices of cell calculated in 1, (ISAVE ( ), JSAVE( ); this cell now is part of delta platform
4. if this is the last time increment, the history of delta growth is printed, using the ISAVE ( ) and JSAVE( ) values

Subroutine PLTPNT

The CALCOMP subroutines called by PLTPNT

and DRWSLC are described in a Stanford Computation Center library program write-up, April 1967, obtainable by writing to the Systems Documentation office, Stanford Computation Center at a charge of 50 cents. The calls are similar to those in use at other computing centers.

1. if L = 1, labels and draws grid, and places an X in the delta platform cells
2. if L = 2 plots a point on this grid, given the coordinates (U,V); these are scaled automatically to floating point inches (see Fig. 6, 7, 8 and 9)
3. if L = 3, terminates this picture by moving plotting origin forward

Subroutine FACIES

If an accounting grid cell contains a sediment thickness comprising two or more size fractions, it is desirable to classify the sediment according to its composition. In order to do this, several standard facies may be set up, whose compositions are read in as data. Sediment in an a/c grid cell is then matched compositionally to the standard facies, classified according to which standard is 'closest' and assigned the appropriate alphameric symbol for plotting. Distance coefficients are used as measures of similarity (Fig. 15).

Subroutine SMOOTH

1. smooths by column or by row, whichever is parallel to the axis of jet flow
2. smoothing along a row or column is done by simple running averages, in groups whose size is specified by NSM (see input instructions)
3. cells containing zero sediment are treated as though they were the end of a row or column; thus the depositional area is not 'smeared' at the edges

Subroutine FANJ

1. enters loop; once through per fan position
2. calculates position of fan (FANANG) by drawing random number from Gaussian distribution, $\mu = 0^{\circ}$, σ = FANDEV°; if FANANG. GT.FANLIM, draws another number
3. calculates duration spent in this fan position in years; made proportional to height of Gaussian curve - (EXP(FANANG/2.0*FANDEV))

Subroutine DRWSLC

1. determines scale factors, vertical exaggeration and size of plot, according to input options
2. plots headings and frame of X-section
3. plots sea level
4. plots original bottom profile
5. enters loop once for each time increment
6. calculates position of bottom profile for that time increment
7. plots bottom profile
8. returns to (5) until all time increments exhausted

Subroutine MAPS

1. prints sediment thickness maps for current time increment, for each size fraction (see Fig. 14a,b,c)
2. prints facies map (Fig. 15)
3. prints depth of water map (Fig. 16)

Subroutine PEPPER

The most lengthy calculations made by this program involve the solution of the equation describing each particle trajectory. However, unless a large number of particles are used, the resulting sediment 'surface' in the accounting grid becomes very uneven (subroutine SMOOTH helps to improve this). In order to minimize the number of calculated trajectories, yet maintain a large number of points, each nominal particle is used like a 'pepper pot' to generate a number (IEXTRA) of points that are positioned randomly in a rectangular area around the original point. The four margins of this rectangular area are placed midway between the original point and the four neighboring 'original' points, one on each side.

Subroutine ROMBUG

Romberg integration method translated from a Stanford ALGOL procedure by R.S. Dobson; uses an external function (DUMF), lower and upper limits and a tolerance.

Subroutine FINT

Function specifying a sediment concentration for any height above the bed of an alluvial channel. This function used by CHANEL and ROMBUG for calculating discharge between given channel elevations.

Subroutine RANDU

IBM 360 Scientific Subroutine Package routine for generating uniformly distributed random numbers between 0 and 1. Under OS/360 this routine is automatically available in the library.

Subroutines

STRTP1

CHAPTER 6

CHAPTER 6: ECONOMICS AND RIVER BASIN MANAGEMENT: THE ECONOMIC SIMULATION MODEL

This chapter is devoted mainly to the economic simulation model, the last part of the management system described in chapter 5. Prior to the description of the model, a discussion on the role of economics in water resources management with particular reference to less developed countries (LDC s) is briefly presented in section 6.1. This role is illustrated by the examples of Egypt, Nigeria, and Ghana in sections 6.2, in line with the examples provided in the first three chapters of this thesis. In sections 6.3, the economic cost benefits methods are discussed briefly. The structure, operational methodology and data requirements of this model and its multi-purpose costs and benefit functions are presented and discussed in sections 6.4 and 6.5. The interrelationship between water management and economics is first presented.

6.1 Introduction

Water resources development plays a significant role in the economics of the committee of nations due to its use in commerce, industries, power generation, agriculture etc. Consequently, economics plays an important role in the management of water resources as the optimal development of schemes are desired within the limits of scarce financial resources. The development of water related projects involves a lot of money and the basic

economic principles of demand (public need or want) and supply (by governments/private utilities) come to play. In addition, the evaluation of alternatives, financing, satisfaction, benefits and costs are all utilised to obtain a plan optimal in socio-economic returns and acceptable environmentally. This is not readily attained in practice as priority is usually given to short term gains rather than long term considerations in water management, especially in the third world.

In particular, many planners tend to overlook the economic importance of long term environmental problems and tend to concentrate on the short term returns. The enormous economic losses and huge human suffering associated with flood and drought disasters (chapter one and two), may be caused by a situation whereby short term benefits takes priority over long term returns. If this trend in the economic approach to management should continue, with changing world climate, things may worsen. For example, the construction of a large multipurpose reservoir by a third world country may appear expensive and economically unattractive in the short term but may provide bigger long term benefits for minimising the suffering and damages resulting from floods and droughts. This research is focused on the long term economics of water projects, in LDC s.

6.2 Water Resources Management and Third World Economics

As many third world countries are confronted with the problem of scarcity in foreign exchange, a long term strategy in water management is required. This lack of exchange mainly emanates from the frequent drops in the prices of exportable raw materials and minerals, their main foreign exchange earners. This has led to heavy borrowing and a resultant accumulation in international debts which is severely hampering their development programmes. Table 6.1 shows the debt burden of LDC's by country. The first, second, and third columns provide the public debt in millions of United States dollars, debt as a percentage of G.N.P, and the debt service ratio values as at 1985 respectively. The debt service ratio is the ratio of the external debt annual payments to exports of goods and services in percent.

Table 6.1: Debt Indicator by Country in Africa-from W. Resources '87

Country	Debt $m	Debt % GNP	Debt Service Ratio	year
Algeria	12025	24	32	1984
Cameroon	1975	25	8	1985
Congo	1396	76	21	1984
Egypt*	15808	50	34	1984
Ethiopia	1384	30	14	1984
Gabon	725	24	9	1984
Ghana*	1122	23	13	1984

Table 6.1 (Ctd)

Country	Debt $m	Debt % GNP	Debt Service Ratio	Year
Guinea	1361	59	-	1984
Kenya	2633	46	22	1984
Liberia	874	80+	4+	1985
Madagascar	1636	73	24	1984
Mali	1327	115+	9+	1985
Mauritania	1171	171	10	1984
Nigeria*	13061	15	31	1985
Senegal	1555	69	7	1984
Sudan*	5659	77	14	1984
Tanzania	2594	68	14	1984
Zaire	4825	137+	8+	1985

*** Examples frequently referred to in this thesis.**

Table 6.2: External Debt Indicators by Selected Countries.

(World Resources, 1987)

Country	Debt $m	Debt % GNP	Debt Service Ratio	Year
Egypt	15808	50	34	1984
Ghana	1122	23	13	1984
Nigeria	13061	15	31	1985
Indonesia	22883	30	15	1984
Malasia	11846	39	8	1984
Thailand	9800	25	15	1985
Argentina	28671	41	29	1984
Brazil	66502	34	27	1984
Chile	10893	63	26	1984
Mexico	69007	42	34	1984
Peru	9825	59	15	1984
Venezuela	17247	38	13	1984
India	22403	12	10	1984
Bangladesh	5966	39	17	1985*
Pakistan	10681	32	32	1985
Turkey	15774	32	23	1984

As a result of this problem , which has now attained a crisis status, the management of water resources becomes even more significant. This is particularly so as many LDC s have agro-based economies. LDC s facing problems include Egypt, Ghana, and Nigeria. Ghana and Nigeria have monocultural economies with respective exports of cocoa and petroleum. All three depend mainly on water storage projects for their agriculture and energy needs.

The Aswan reservoir in Egypt supplies most of her electricity, irrigation and navigation needs. The kainji-Jebba reservoir system provide a bulk of the power and energy supplies required for servicing the Nigerian economy. The Akosomobo reservoir supplies virtually all the electricity needs of Ghana and part of the needs of the Cote d' Ivoire, Burkina Faso, Togo, and the Republic of Benin. The recent Sahelian drought sequence has severely hampered the performance of these reservoirs and strained the debt-burdened economies of these countries. This adverse interaction between climate and economics could be complicated further by the ongoing and future changes in world climates due to GHG s and this could worsen the economic plight of these LDC s. Developed countries may also be affected. This possibility has led to the consideration of the changing river basin approach throughout this work.

(a) Egypt

In Egypt, the economic failure of the Aswan reservoir was only averted because of its enormous storage capacity and good water conservation programmes by the authorities (Smith, 1986). Nevertheless, the large Egyptian national debt and the high debt-service ratios coupled with the problems of a large population and the possibility of more severe drought, makes the adoption a more optimal water management strategy necessary (Table 6.1).

(b) Nigeria

In Nigeria, the frequency of unbearable power failures resulting from drought effects has led to the importation of portable household and industrial generators costing billions of dollars to the economy. The economic problem was compounded by the resultant fall in industrial and commercial production in such years. The economic situation is further complicated by a combination of low oil prices (in the last few years), a large national debt, high debt services ratios, high rate of population increase, and low agricultural production (from drought) Table 6.1 and 6.2).

(c) Ghana

A similar situation in Ghana caused severe losses in foreign exchange revenues from power sales to neighbours. Acres (1985) stated that:

"an extremely poor hydrological cycle in the last decade has resulted in an adverse situation for the VRA (Volta River Authority of Ghana). At the end of the 1983 wet season, the lake Volta water level was well below the minimum operating level. Load curtailment was necessary to prevent a complete collapse of th VRA power system, which inturn endangered the cash flow required to service its foreign debts and foreign operation costs".

To avoid a repeat of this unacceptable situation, the government of Ghana has recently considered rehabilitating the Tema thermal power station, near Accra, to supplement

Akosombo generation (Merz and Mclellan, 1986). In addition to the power problems associated with drought, the Ghanian economy faces other problems, some of which are related to water resources. These are listed as follows:

(1) A 25% fall in GDP in the last 15 years.

(2) A high inflation rate (128% in 1983).

(3) A sharp decline in mineral production in the last 15 years.

(4) A severe drop in foreign exchange reserves by high oil prices in the seventies and early eighties.

(5) A huge population increase in the last 15 years.

(6) Fall in agricultural production from drought, disease, fire, and low prices.

(7) a huge national debt compared to GNP (Table 6.1 and 6.2).

With these economic problems in mind, maximising the benefits from water related schemes is imperative , especially in third world countries. This is one aim of the approach proposed in this work.

(d) The Economic Role of the Water Sector

The economic problem of a water scheme lies in the consideration of several alternative development sequences and the selection of a sequence which allocates a country's limited resources (capital, labour, etc) so that the net benefits to society will be maximised.

The energy and water sectors in any economy, particularly in LDC's represent a sizeable proportion of the expenditure. For example, table 6.3 represents the capital expenditure for the Ghanaian economy for 1986-88. The energy (hydropower) and water (includes agriculture) related sectors account for about 40% of the total national expenditure. Thus, these sectors are 'key', both in the process of structural adjustment and as a direct export earner, conditions necessary for the survival of third world countries. Structural adjustment programmes are conditions pursued by economically strained third world countries to enhance economic recovery. The adjustments are carried out on the advice and recommendations of international lending bodies such as the IMF and IBRD.

It is the interaction between economics and water resources as exhibited by the problems and examples defined above that has necessitated the inclusion of an economic simulation model (a type of economic monitor) in the proposed management system. This will help in the economical assessment of water schemes. The structure and construction methodology of the economic simulator is described in sections 6.4-6.5. Prior to this the economic methods used as applied to water resources management are next discussed.

Table 6.3: Ghana's Capital Expenditure for 1986-88

(Constant 1985 prices in cedi M)

Sector	Expenditure Proposed	% share	Foreign Financing Commited/ Under Neg'
1. Agriculture	9,511	12.4	2,834
2. Mining	10,549	13.7	4,788
3. Energy*	11,023	14.3	11,235
4. Industry	2,572	3.3	848
5. Roads/Highways	15,576	20.3	5,712
6. Trans/Commu'	15,057	19.6	7,964
7. Social	6,662	8.7	704
(water)*	(2,162)		(236)
(education	(2,500)		(228)
(health)	(2,000)		(240)
8. Others	5,805	7.6	-
Total	76,800	100.0	34,085

*Water-related sectors.

Exchange Rate used is 90C=$1 or 125C=£1

Source: Merz and Mclellan, 1986

6.3 Economic Methods in River Basin Management (James and Lee, 1971)

The mastery of the basic planning and operations techniques described in chapters 2 and 3 combined with a basal knowledge and use of economic theories and methods characterise a good water resources management programme. An important and quantifiable concept for plan evaluation are the economics of benefits and costs that a proposed scheme would entail on implementation. In the

mathematical models used in the representation of a water resources system, the important design and operating variables affect economics parameters (benefits and costs). These include for example, water releases (instantaneous or annual), the time-variant volume of water in reservoirs, pipelines, reservoir location and the size of impounded area, land costs, etc. These must be carefully assessed if water projects are to meet their socio-economic goals.

After obtaining the economic variables outlined, the economic methods utilised in assessing and comparing alternative schemes (or different sizes of one scheme) are the benefit cost discounting techniques and the least cost method. Each method involves the use of an appropriate discounting factor and a specified discount rate. These factors are described in the literature in any standard text on engineering economics. The benefit cost methods include:

(a) The present worth method

The benefit costs ratio is the present worth of the benefits divided by the present worth of the costs. Annual values can be used without affecting the ratio. James and Lee (1971) indicated that the best project is the one with the greatest nett benefits, not necessarily that with the highest benefit cost ratios. Many authors suggest that the technique yields different results when compared to other methods but this only occurs in the

cases of mutually exclusive alternatives, when an increasing cost ranking of the ratios is calculated without the use of the recommended incremental costs and benefits techniques. The use of the BCR method without a supporting incremental BCR analysis or an alternative comparative technique may lead to serious error of judgement. Interdependent schemes cannot be ranked according to their benefit cost ratios because each 'enlargement' must pass through the incremental benefit cost ratio test.

(c) The annual cost or annual worth method

This method converts benefits and costs into equivalent uniform annual figures by multiplying by a cost recovery factor and these are compared for different alternatives. The annual cost is the equivalent first cost considering salvage and it is called the capital recovery cost. The salvage value estimate identifies the future value of the original investment at a specified future date. The annual worth is the annual benefits less the annual costs. The annual cost method uses constraint multiples of the present worth method and hence have the same merits and demerits. As many people tend to think of annual cost or worth, it is sometimes preferred to the present worth method. Further, where benefits cannot be evaluated, this method is the last resort.

(d) The rate of return method

The rate of return is the discount rate at which the present worth of a scheme is zero. In other words, it is the interest rate which makes the sums of the present worths for expenditures and payments equal to zero. That is, the interest rate at which benefits equal costs or the net benefits equal zero. A trial and error approach is used in estimating the rate of return. The advantage of this method is that it does not require a pre-selected discount rate and it is intuitively meaningful to many investors and the estimated rates can be compared with those of

many other types of investment. The drawbacks of the method are

(a) that it provides ambiguous answers due to dual solutions (James and Lee, 1971)

(b) the necessity of calculating incremental return rates for interdependent projects.

(c) the dangers of accepting overall rates as against incremental return rates as indicators of rank.

(d) the complexity of the required trial and error solution.

Computational time on a computer will be large, if programmed.

Details of the rules utilised in applying any of these methods are available in the literature. The benefit cost methods, if correctly used, leads to the same evaluation

and results of relative merit. The benefit cost ratio and the present worth methods are the most widely used on the list. The BCR method is considered consistent with economic theory. These methods have all been included in the economic simulation model because of their advantages and disadvantages. This is to lessen the probability of error in economic comparisons of alternatives. The simulation model is described next.

6.4 The Economic Simulation

6.4.1 General

The economic simulation model of the river basin management system (shaded in figure 6.1) has been developed during the course of this research to assist in

reservoir optimisation studies by computer simulation techniques. The economic simulation package, utilizes the economic concepts of benefit-cost analysis described in section 6.3.

Basically, the model uses annual releases from reservoirs or channels obtained by a continuous summation of daily or instantaneous diversions (depending on the chosen rule curve) from the modified channel and reservoir routing model. The annual releases are allocated by the economic model, to competing interests, such as power generation, irrigation, water supply etc. Navigation and flood control releases are co-ordinated with power generation.

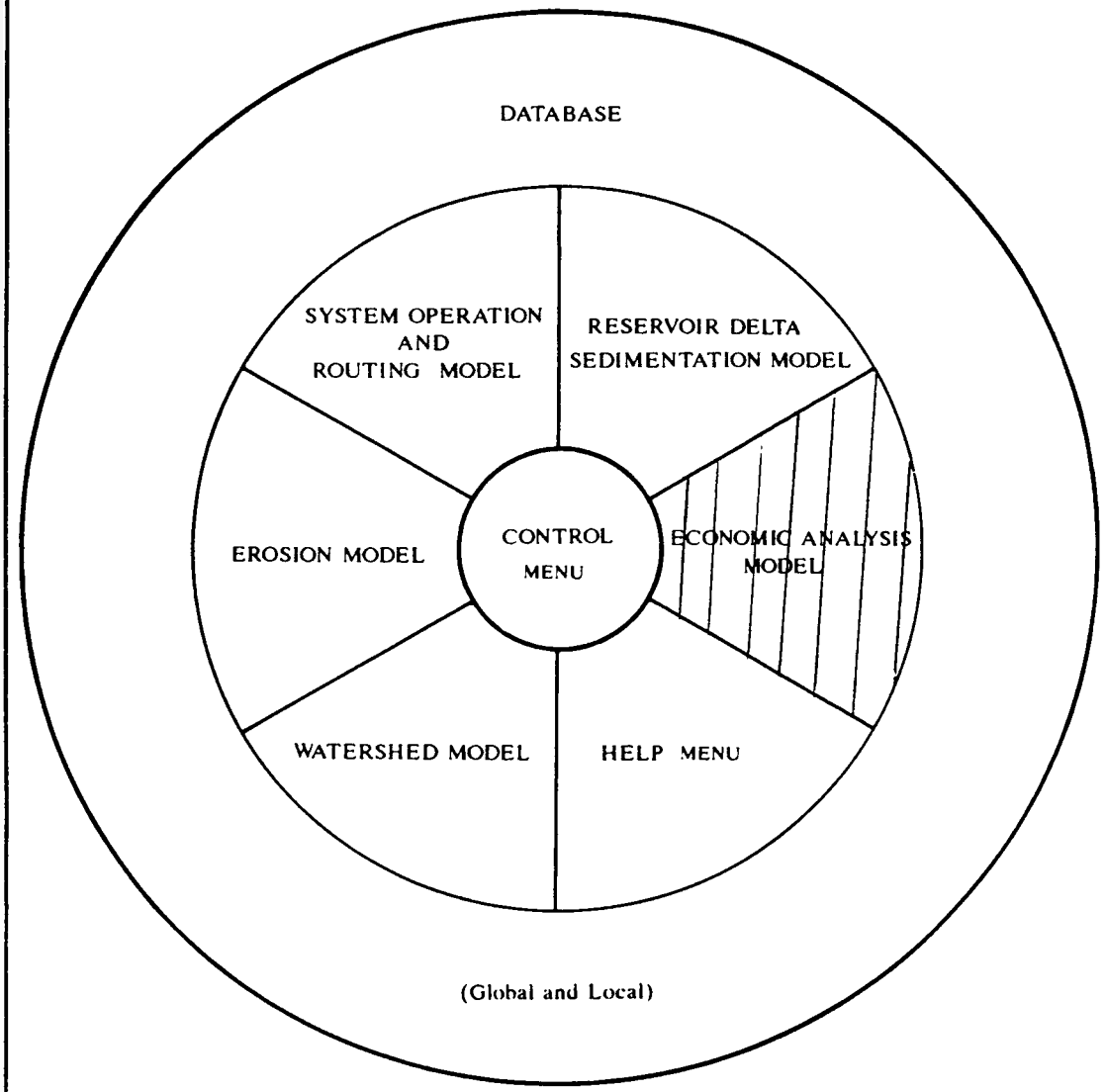


Fig. 6.1 RIVER BASIN MANAGEMENT SYSTEM

This system is available for use on both micro and mainframe(VAX) computers.

Benefits and costs are calculated in $U.S for power, irrigation, urban and rural water supply using available local and international economic information such as exchange rates, unit prices of water and energy, population, rate of population increase, inflation rates, discount and interest rates, amortisations, etc. Costing techniques and specific costs of reservoirs, dams, and equipment are obtained from the literature and from consultants reports and cross checked with current international costing standards from such bodies as the World Bank, W.M.O, U.N.D.P, etc. (UNWRS 42, 1972). Sometimes, specific unit costs and unit benefits are carefully evaluated as required and these are input as data in $U S. currency, being a hard international exchange. Both costs and benefit functions, where utilised, are sometimes related to the quantity of water released, the amount of water in storage, the capacity or size of the reservoir, and the results obtained are cross-checked from costs incurred from actual existing reservoirs in Nigeria or from consultants estimates.

Finally, all costs and benefits are brought to their present and annual values, as they are required for comparative analysis, by multiplying by an appropriate economic discounting factor. The model accomplishes this automatically without resorting to use of standard tables as employed in practice. Comparisons of costs and benefits are made from tabulated computer outputs and plots of present values, annual values, annual and present

worths, and benefit-costs ratios. A typical computer summary of the model results is provided in table 6.4 for the information of the reader.

6.4.2 Model Operations

The items listed below illustrate the sequences involved in a typical run of the economic simulation programme (ESP).

(1) After the runs of the watershed and erosion models, annual diversion flows (releases) are computed and sent to a computer file by the channel and reservoir routing model using a specified rule curve.

(2) The computer then interactively prompts the user for the run of the economic model by a function key code. The provision of the appropriate function key automatically activates the model (chapter 5).

(3) the model utilises the economic data file and the water release value stored in a computer file.

(4) The model automatically allocates water as required by the use of a data prompt to competing users, such as hydropower, irrigation, water supply, by means of input data fractions, thus providing some flexibility.

A typical model run, which utilizes both the economic input data and the filed annual diversions as expected is much faster on the main frame vax computer system than on the microcomputer. The durations of the runs depend,

	HYDROPOWER	EVAP LOSS	SPILLAGES
DISCHARGE CMS	0.9256E+00	0.7884E+00	0.0000E+00
MEAN POWER MW	0.6493E+01	0.6145E+01	0.0000E+00
MEAN ENERGY KWH	0.5688E+08	0.5383E+02	0.0000E+00

CAPITA 0.8677E+03 HECT 0.2879E+01

$

FIRST COSTS	0.2357E+09	0.2357E+08
ALTNT COSTS	0.4292E+09	0.4292E+08

NONPOWR GR.TOTAL	POWER	IRRIGATION	W.SUPPLY	OTHERS	TOT.N PW	TOTAL
PRESENT VALUES-COSTS	0.3061E+09	0.6710E+05	0.3777E+05	0.6253E+08	0.6263E+08	0.4412E+09
PRESENT VALUES-BENFTS	0.3200E+08	0.8098E+03	0.7322E+05	0.7306E+07	0.7380E+07	0.3938E+08
PRESENT WORTHS VALUES	-.2741E+09	-.6629E+05	0.3545E+05	-.5522E+08	-.5525E+08	-.4018E+09
BENEFIT COST RATIO	0.1045E+00	0.1207E-01	0.1939E+01	0.1169E+00	0.1178E+00	0.8926E-01
ANNUAL COSTS VALUES	0.2473E+08	0.5422E+04	0.3052E+04	0.5052E+07	0.5060E+07	0.3365E+08
ANNUAL BENEFIT VALUES	0.2585E+07	0.6543E+02	0.5916E+04	0.5903E+06	0.5963E+06	0.3182E+07
ANNUAL WORTH VALUES	-.2214E+08	-.3797E+04	0.2865E+04	-.4462E+07	-.4464E+07	-.3246E+08
ANNUAL BEN.COST RATIO	0.1045E+00	0.1207E-01	0.1939E+01	0.1169E+00	0.1178E+00	0.1132E+00
UNDISCOUNTED COSTS	0.8030E+09	0.1016E+06	0.7016E+06	0.2223E+09	0.1467E+10	0.2270E+10
UNDISCOUNTED BENFTS	0.6606E+09	0.1768E+05	0.1598E+07	0.4061E+09	0.4077E+09	0.6679E+09
BEN.CST RATIO-UNDISCOUNTED	0.8227E+00	0.1519E+01	0.2278E+01	0.1827E+01	0.2778E+00	0.2942E+00

TABLE 6.4

however, on the number of reservoirs being analysed. As many as ten or more reservoirs can be analysed in a single run. In addition, multi-annual simulations are possible.

6.5 The Economic Simulation Model Structure and Data Requirements

The flowchart in figure 6.2 completely describes the structure of the model shown in the sequence of costs and benefits calculated and the source code for the model is provided in the appendix and in the appropriate user guide.

6.5.1 The Evaluation of Costs and Data Requirements

Apart from cost-release and cost-capacity relations, the model also carefully relate cost when appropriate to other outputs such as energy, agricultural hectarage, agricultural yields and population by appropriate cost functions. As many cost items tend to be site specific and time varying, unit costs are used in such circumstances and these are estimated for the reservoir under study and read in as data. Some costs, such as flood control, navigation, fishery, and recreation, which are difficult to access, or assess, have been specifically related to the parameters of the reservoir, such as the total capacity, annual mean reservoir outflows, etc. The figure 6.2 shows the sequence of these costs.

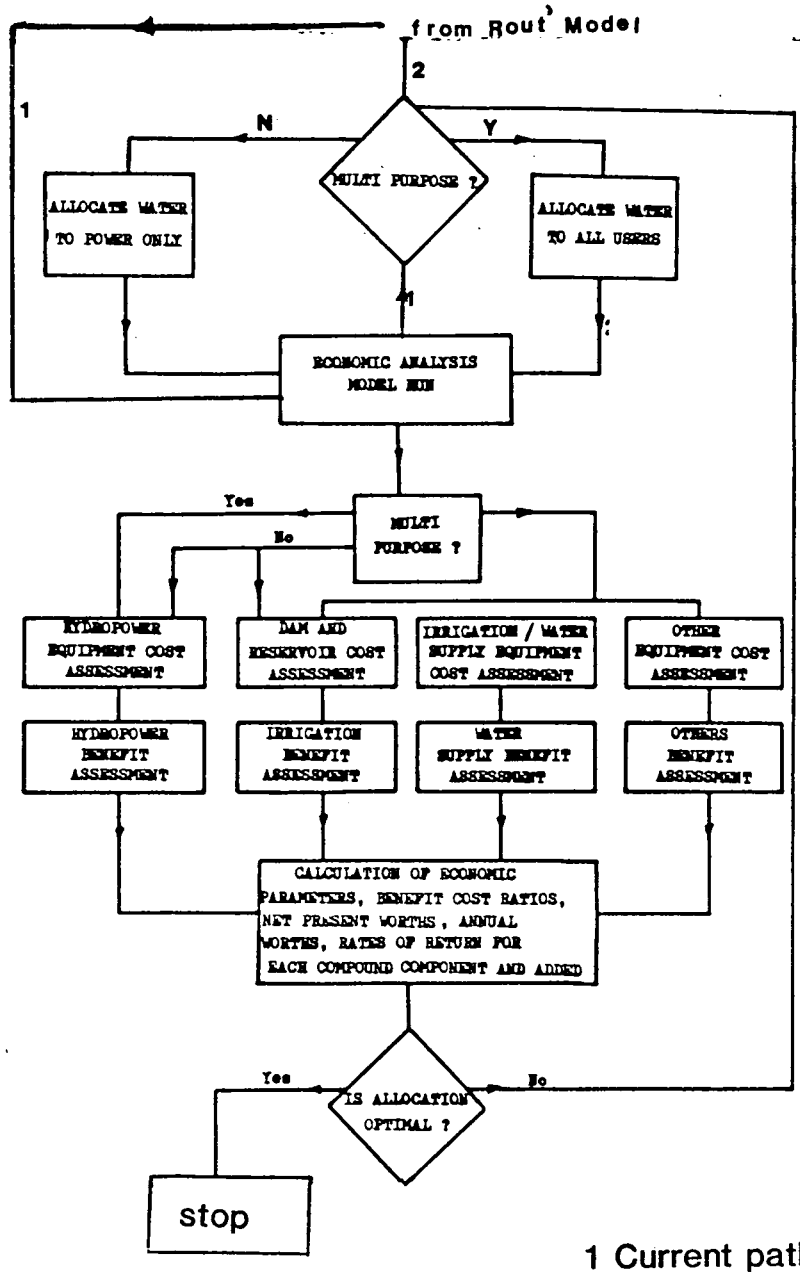


Fig.6.2 STRUCTURE AND OPERATION OF THE ECONOMIC SIMULATION MODEL

(a) Hydroelectric Generation Costs

These costs are estimated by two methods and their results are compared and the more realistic one is used. The first is a local approach which strictly utilises costs incurred for existing reservoirs in Nigeria. The second approach involves the use of costs functions originally developed by Gordon (1978). The former technique is treated in this section and the later is discussed in section 6.5.2.

(1) Local Approach

Local hydropower costs are estimated by the combined use of a unit equipment cost of total long term or capitalisation energy in cents per kilowatt hour and a unit reservoir infrastructure cost in cents per cubic metre of reservoir storage. For Gembu reservoir in the Donga basin, respective local values of 0.027 cents per kilowatt hour of expected gross generated energy (5,010 Gwh per annum for the 50 year simulation period) and 0.2 cents per cubic metre of reservoir capacity (6 McM), are deduced from slopes of cost curves plotted from existing Nigerian reservoirs given in figures 6.3 and 6.4 respectively. An appropriate current foreign exchange rate factor (5N=$1) has been utilised when and where necessary. These cost values are compared with and found to be consistent with the values estimated by various consultants such as Binnies/Diyam (1983), M.R.T (1978), and A.L.T.S (1978) in their seperate feasibility reports on Donga River Basin-wide hydropower and multipurpose

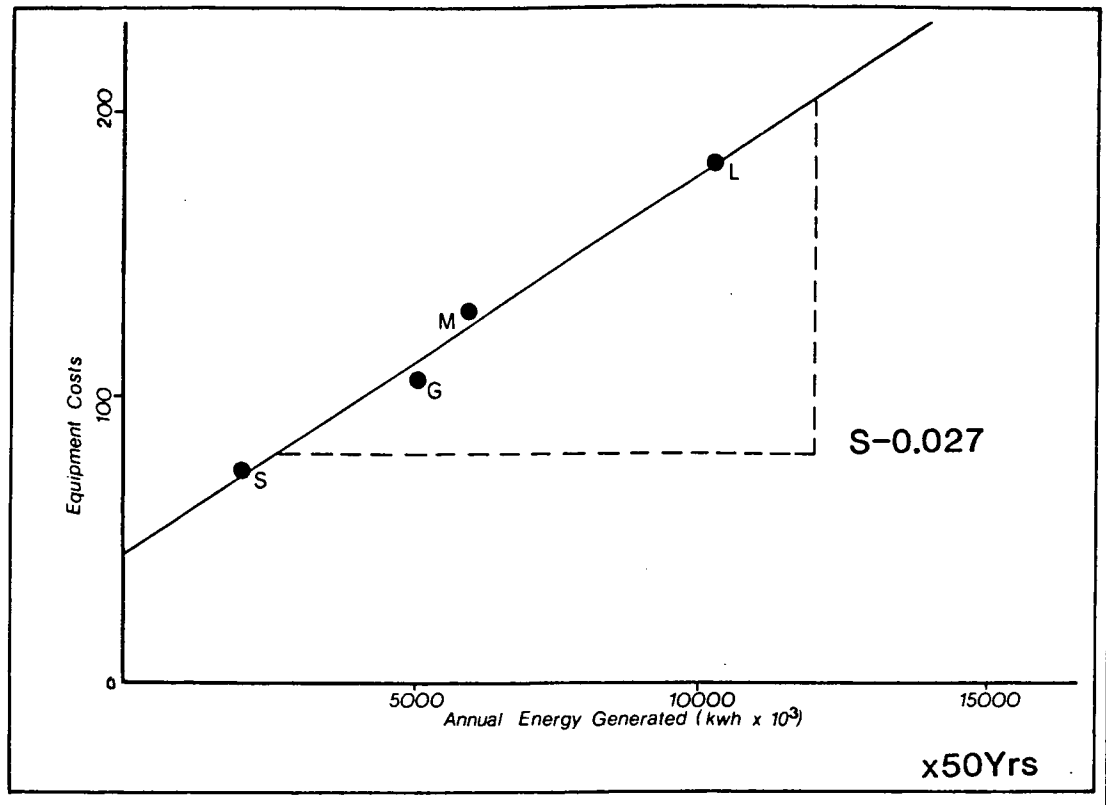


Fig. 6.3

S - Slope

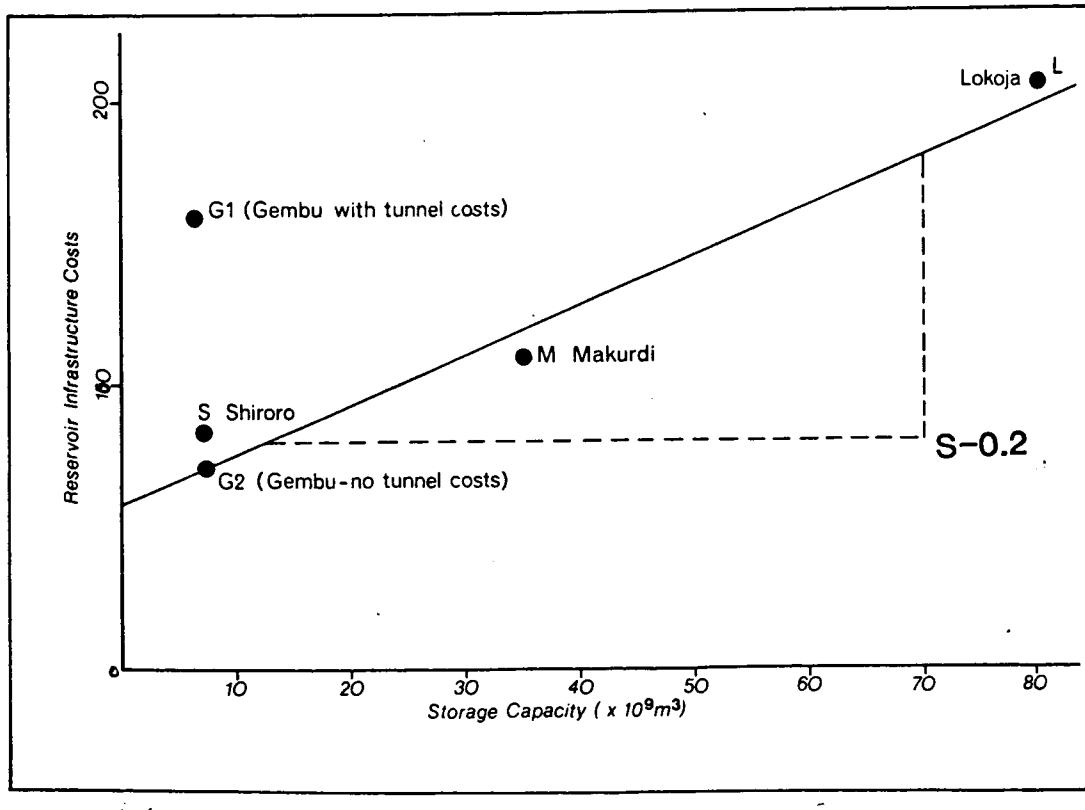


Fig.6.4

development plans. They also compare with that of an alternative costing approach given in section 6.5.2 (see * in table 6.5)

The method utilised involves the use of two different equations. The first equation, 6.1, is a simple total power cost equation which relates total costs as a sum of site and reservoir costs, equipment costs is obtained by dividing the total costs in equation 6.1 (a) by the construction period in years as indicated in equation 6.1 (b).

$$PCT=[(LOP+Q)*ANENG*PCOST+((TOTST*DMCM*PRFRA)]/100) + TNL*TNNC \quad 6.1\ (a)$$

$$PC=PCT/CP \quad \ldots\ldots\ldots\ldots 6.1\ (B)$$

where the product

TOTST*DMCM*PRFRA

is the reservoir development cost allocated to power and DMCM is the cost per unit storage capacity of the reservoir. A value of 0.2 cents per cubic metre was obtained by estimation utilising cost data from other Nigerian reservoirs (Fig 6.4).

TNL*TNNC gives the water conveyance tunnel costs. TNNC, the mean tunnel cost per unit length is obtained from standard tunnel cost curves obtained from international practice (figure 6.5) (U.N.W.R.S, 1972; M.R.T, 178, Diyam/Binies, 1985). In the case of the three Donga Basin reservoirs, Gembu, Sumsum, and Nghu, the high head power

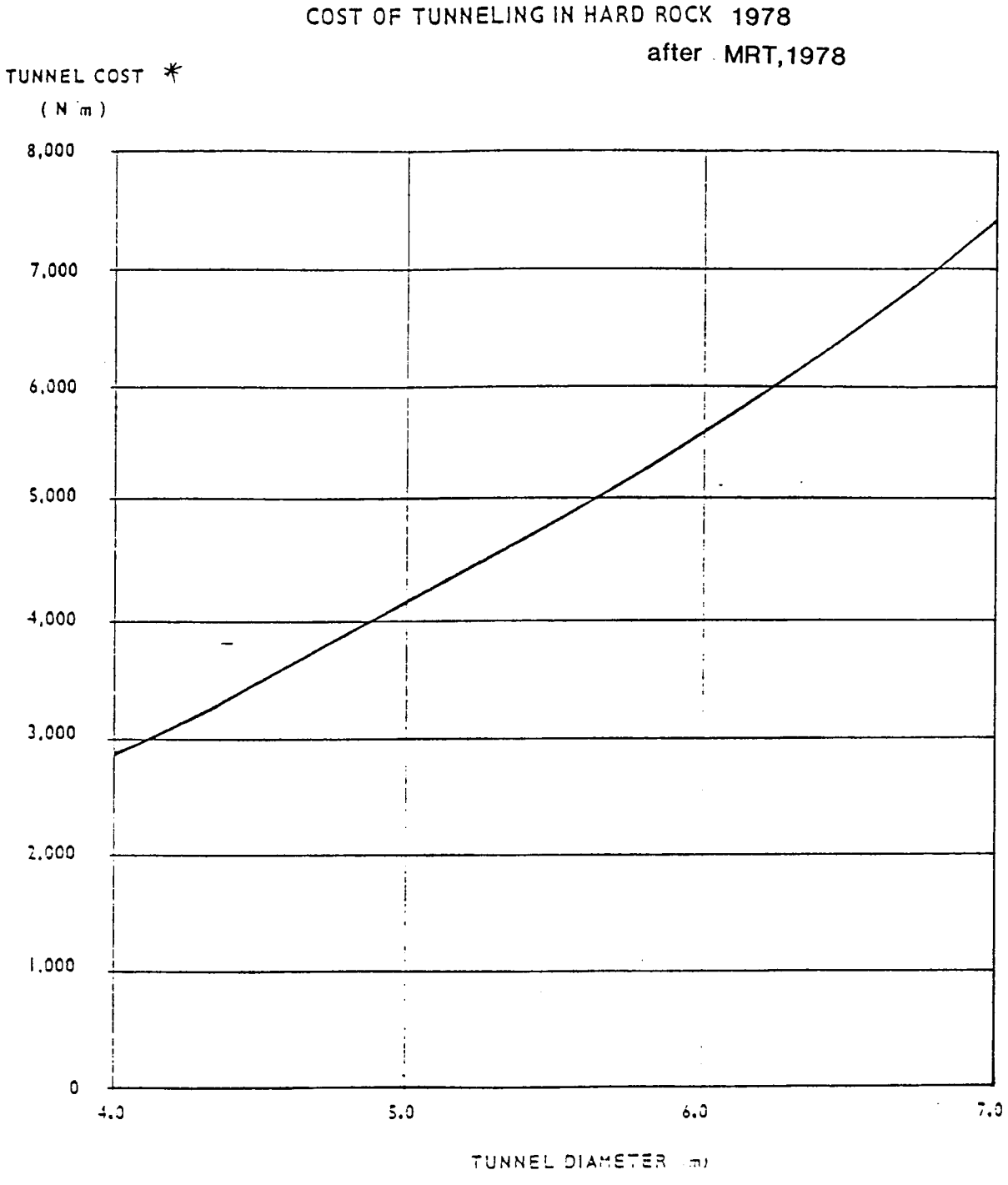
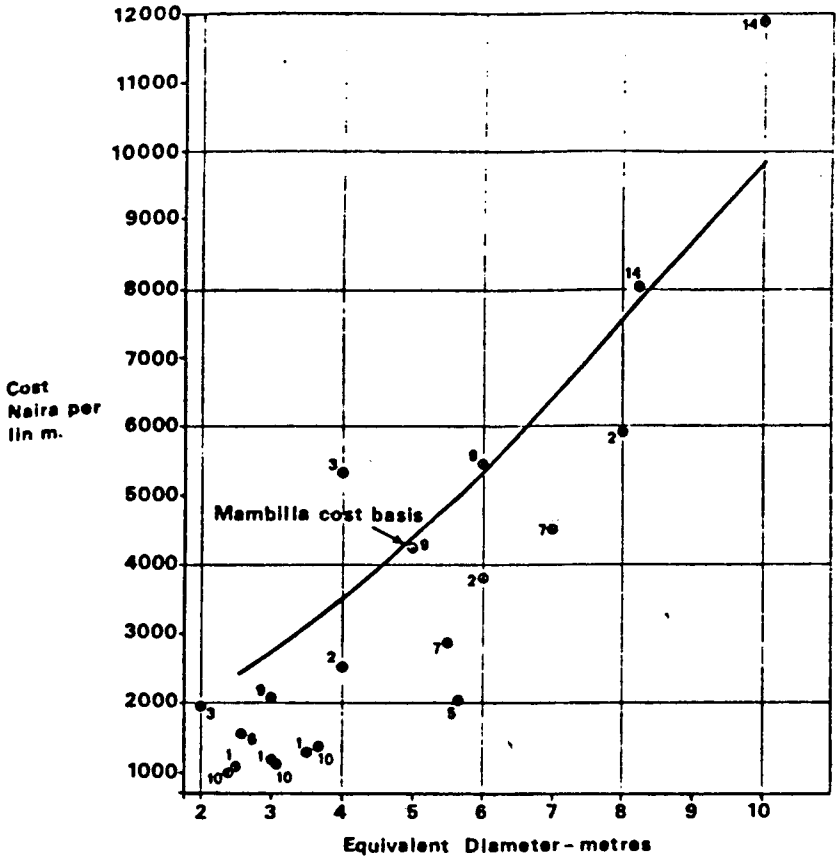
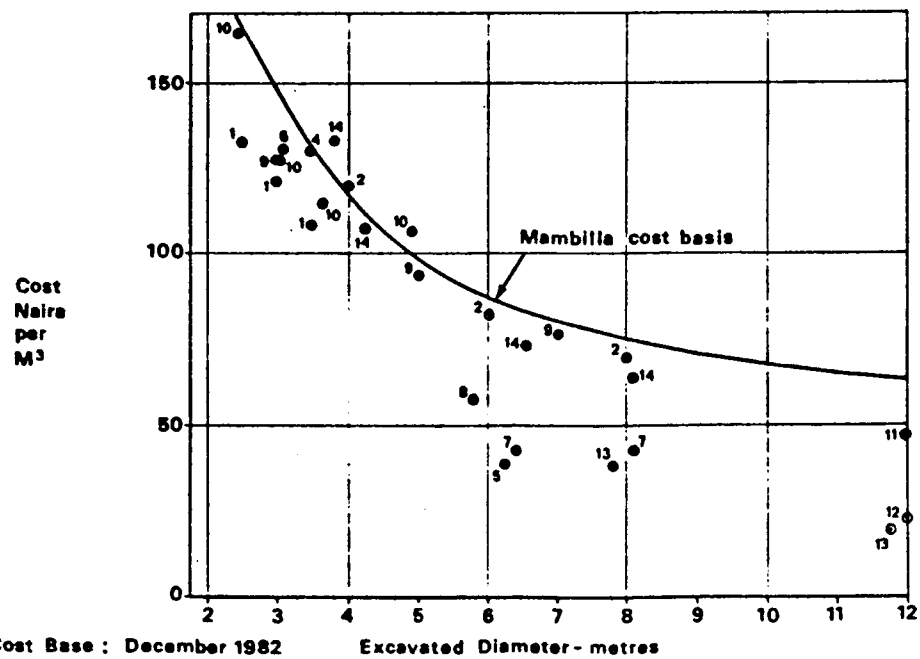

COST OF TUNNELING IN HARD ROCK 1978
after MRT,1978
TUNNEL COST *
(N m)
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
4.0
5.0
6.0
7.0
TUNNEL DIAMETER (m)

FIG 6.5 a



COSTS OF CONCRETE LINED
LOW PRESSURE TUNNELS



Cost Base : December 1982
£1 : 1·17 Naira : US $1·60

FIG 6.5b COSTS OF TUNNEL EXCAVATION : PTO

a Diyam 1985

TUNNEL COSTS

Figure

REFERENCES SHOWN ON FIGURE 6.5b

1. Harbour Island Project, B & P (Hong Kong) 1983. Average costs of four Water Supplies Department contracts involving eight tunnels - diameters between 2.5 and 3.4 metres.

2.* Salto Bajo Draft Prefeasibility Report, 1983, Motor Columbus, Binnie & Partners, Motlima, line graph produced from 10 references, including 5 Peruvian (studies) 2 UN reports, 1 Colombian. Range 2.0 to 8.0 m in "sound rock", requiring no temporary support.

3.* Mantaro Transfer Study, 1980 (Joint Study for optimising the Mantaro Transfer Project for Water and Power supplies - Vol. 2 (Draft) - Lahmeyer Saltzgitter B&P, August 1980). Based on eight Peruvian references 1975-78. Fully and partly lined tunnels 2.0 - 4.5 m diameter.

4. Kielder, 1974. As reported in Tunnelling 1979. 3.5 m dia. bored tunnel.

5. Camlough, 1971. Lowest tendered prices.

6. Lune Wyre-Wyresdale tunnels, mid 1975. Target cost contract 5.6 km in ground 60% requiring support 2.6 m finished diameter.

7.* Arakthos Hydro-electric project, Greece, B&P and associates 1982. Detailed estimates based on joint venture associates (Electrowatt, Stucky) prices. 5.5 m and 7.0 m dia. tunnels, concrete lined. 7 km headings.

8. Selby coalfield UK, 1983. As reported in Tunnels and Tunnelling March 1983 pp 57-58. Comparison of TBM (Robbins) + drill/blast for 5.8 m dia on 13 km drive.

9.* Lesotho Highlands Water Study, B & P, 1978. Estimates base[d] driving through basalt (mass concrete invert), sandstone (concrete) and mudstones and shales. Long drives.

10. High Island Water Scheme, Hong Kong, B & P 1971. Anal[ysis] lowest tender rates for tunnels predominantly 2.5 to and a short length of 6 m tunnel.

11. Estimating Underground Power station Costs - R. [illegible]n HE Energy Division, March 1982. Typical US costs.

12. Norwegian Hard Rock tunnelling - typical data and stations.

13. Sima hydroelectric project, Norway. 22 m[illegible]artners, 110 m^2 tailrace tunnels.

14. Dinorwic pumped storage scheme, James 1974. Short drives in competent ro[ck]

*Estimates

house location serving the three is far off from the dams and hence require long water transfer tunnels (25km) in very rugged and wild terrain (Figure 6.6). This is expected to be extremely expensive, especially as the terrain is mountainous, steep, and thickly forested in places. The Amba reservoir site in the upper Donga river also exhibits these same characteristics. A mean tunnel cost per metre value of 9000$ (1988 cost using an exchange rate of 1.25N = $1) was used in the Donga (Mambilla/Gembu) simulation runs.

PCT is the total estimated costs incurred for the hydropwer scheme.

PRFRA is the fraction of water diversion allocated to power generation.

PC is the total annual power costs and PCOST is unit equipment cost in dollars, a value of 0.027 dollars per kWh is deduced and used.

ANENG is the total annual energy in GWh which is calculated from the equation:

$$ANENG = YDIVQ * PK * GHEAD * 9.81 * \frac{0.836 * TIME}{1000} \quad \ldots\ldots \quad 6.2\ (a)$$

YDIVQ is the total annual diversion which is received from the system operation and routing model, and PK is an efficiency factor which is turbine dependent.

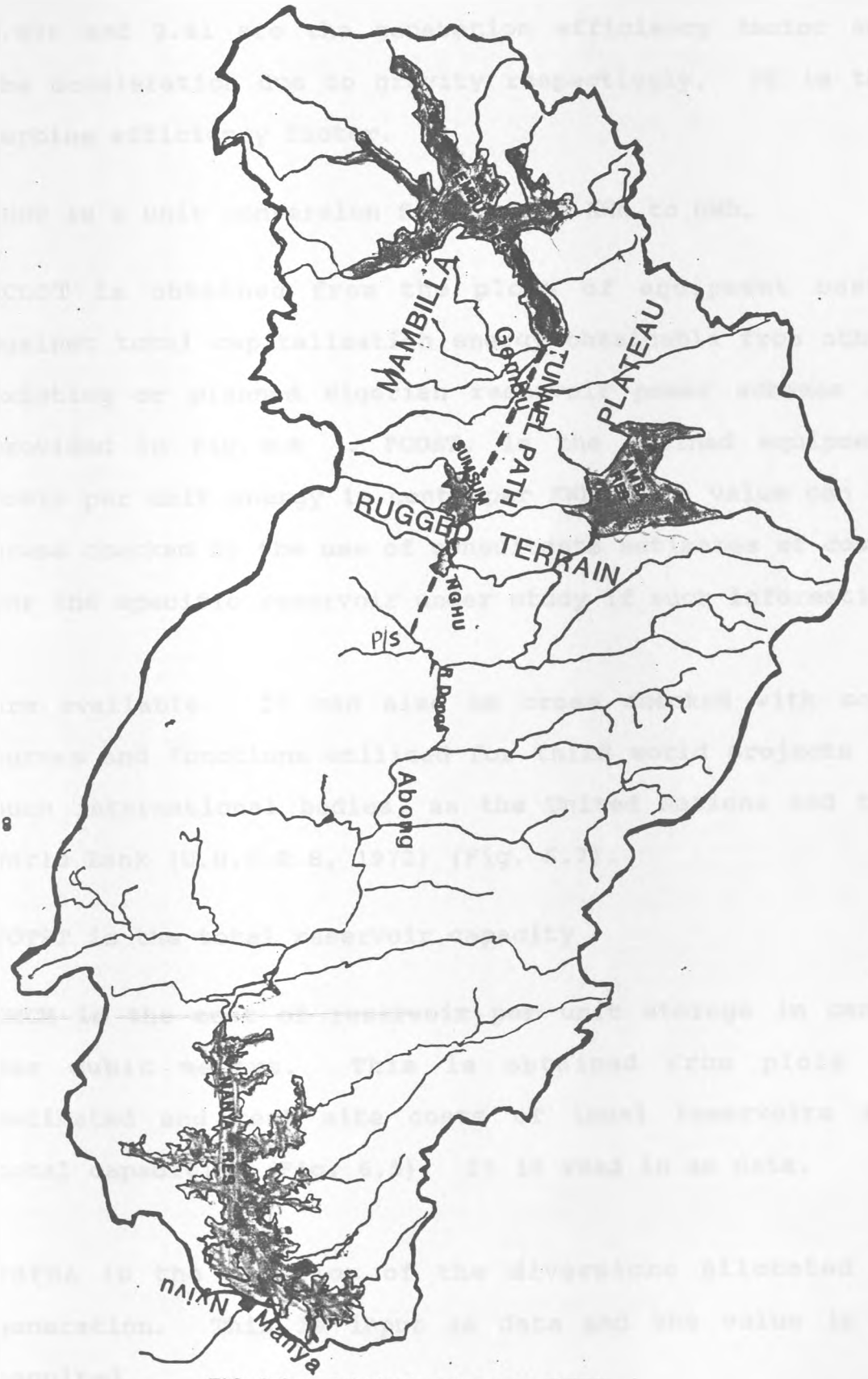


FIG 6.6 DONGA RIVER BASIN
Reservoir sites & Tunnel path

GHEAD is the generation head,

and TIME is the number of hours in a ear.

0.836 and 9.81 are the generation efficiency factor and the acceleration due to gravity respectively. PK is the turbine efficiency factor.

1000 is a unit conversion figure from KWh to GWh.

PCOST is obtained from the plots of equipment costs against total capitalisation energy obtainable from other existing or planned Nigerian reservoir power schemes as provided in Fig. 6.3 . PCOST, is the defined equipment costs per unit energy in cents per KWh. Its value can be cross checked by the use of consultants estimates of costs for the specific reservoir under study if such information

are available. It can also be cross checked with cost curves and functions utilised for third world projects by such international bodies, as the United Nations and the World Bank (U.N.W.R.S, 1972) (Fig. 6.7).

TOTST is the total reservoir capacity

DMCM is the cost of reservoir per unit storage in cents per cubic meters. This is obtained from plots of estimated and real site costs of local reservoirs and total capacities (Fig. 6.5). It is read in as data.

PRFRA is the fraction of the diversions allocated to generation. This is input as data and the value is as required.

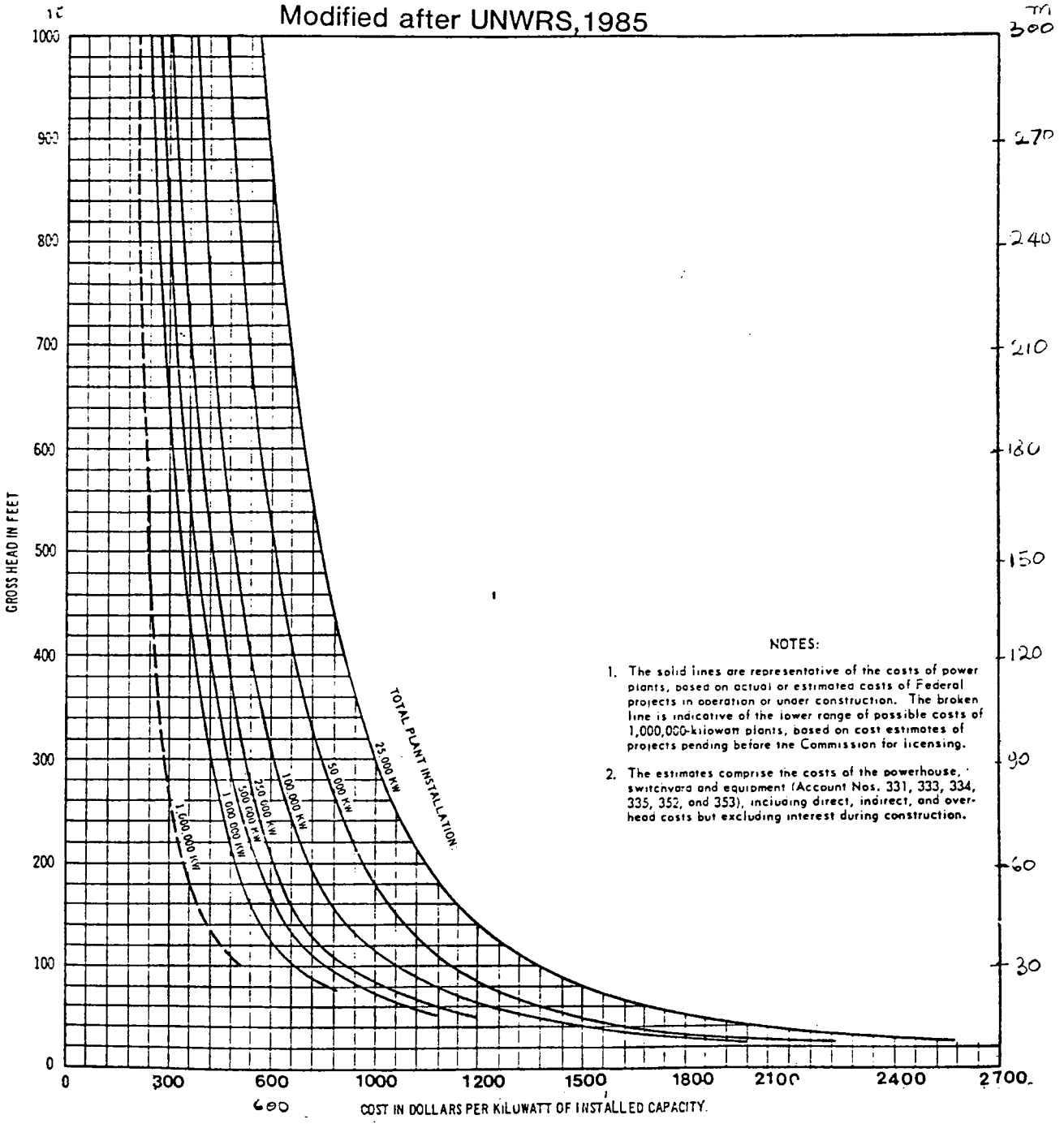


FIG 6.7 Screening guide for estimating the cost of conventional hydroelectric power plants

LOP and CP are the specified project life and the construction period in years respectively. For this study, a value of 50 and 10 are utilised respectively.

(2) The total long term power costs are evaluated from the equation 6.3 which is just an accumulated of annually assigned operation and maintenance costs and annual costs evaluated above:

$$TLTC = PC * (CP) + OMC * \frac{[(1.0+YL)^{(LOP+CP)} - 1]}{YL} \qquad 6.2(b)$$

where

OMC is the annual operation and maintenance costs in the first full year of operation. This is estimated at 1 percent of total project costs. This is based on the U.S. Bureau of reclamation standard of 1% for operation and maintenance for a manually operated hydropower plant in the range of 200 to 1000 MW installed capacity. The general worldwide range is usually between 1 and 5 percent. A percentage value of 3% has been utilised for this simulation study (U.N.W.R.S, 1972).

YL is an average realistic rate of inflation or escalation evaluation from experience and from local and international data. An average rate of 6 percent has been used throughout being the weighted mean of the combination of local and international rates for the construction period.

Q is the number of years of partial or full operation of plant/reservoir before the actual commissioning of the project.

(3) The present value of the total power generation costs, required for economic analysis is obtained by multiplying the total long term power costs by an appropriate series present worth factor. This is given by the equation:

PWI=OMC*FACT+(PCT/CP)*FACT2 ..... 6.3 (a)

where

PWI is the present value of investment when discounted at a given discount or negative interest rate throughout the whole life of the project.

FACT, FFACT, and FACT2 are the requisite series discounting factors which are calculated thus:

$$FACT=\frac{(1+RY)^{**}(LOP+Q)-(1+YL)^{**}(LOP+Q)}{(RY-YL)*(1+RY)^{**}(LOP+Q)} \quad ...6.3\ (B)$$

FACT is a nonuniform series present worth factor covering the whole project life period. It is used for discounting both operation and maintenance costs and benefits from the end of project life back to the start of operation.

$$FFACT=(1/(1+RY)^{(CP-Q)}) \quad ..... 6.3\ (d)$$

FFACT is single payment present worth factor used for discounting the present values of benefits and operation

and maintenance costs at the start of operation to the very beginning of construction.

$$FACT2 = \frac{((1+RY)^{(CP)} - 1)}{RY*(1+RY)} \qquad \text{.....6.3 (d)}$$

FACT2 is a uniform series factor used for discounting construction costs from the end of construction to the start of construction. For simplicity, the assumption here is that the total construction expenditure is uniformly dis-bused throughout the construction period.

where

RY is the discount rate, a kind of negative interest rate which is input as data and obtained from local and international water resources financing practice. A value in the range 6-10 percent is recommended for third world projects by the World Bank and other lending bodies (UNWRS 42,1972). A value of 8 percent is used in this work.

(4) The annual cost is evaluated by multiplying the long term costs by a capital recovery factor, which is obtained from the appropriate discount rate. The following equation describes the technique;

$$ANC = CRF*PC(1+TAXINS) + \frac{OMC*(1.0+YL)^{(LOP+Q)} - 1.)}{(YL)} \qquad \text{..... 6.4}$$

where

CFR is the cost recovery factor which is calculated by the equation:

$$CFR = RY*\frac{(1.0+RY)^{(LOP+CP)}}{[(1.0+RY)^{(LOP+CP)} - 1.0]} \qquad \text{.....6.4 (a)}$$

TAXINS is a fraction of total costs devoted to taxes, amortisation, and insurance and is inputed as data. A value of 3 percent has been used throughout. This figure consists of 2% for taxes, and 1% for insurance. The low figure for taxation is not unconnected with the fact that most third world projects are publicly financed. This is again obtained from international financial standards published by leading lending bodies such as the World Bank and from some Nigerian banks as well.

Table 6.5 Gembu Reservoir Project Cost Estimates by Model and by Consultants.

Method/ Consultant	Model (local	* Model (Gordon)	Diyam	MRT	ALTS	UNWRS Curves
Year of est'	1988	1988	1985	1978	1977	1988
Cost m$	455.0	429.0	382.0	302.0	270.0	390.0
Ins' Cap'MW	1300.0	1300.0	1300.0	1282.0	517+	1300.0
Energy Gwh/a	5010.0	5010.0	5010.0	5616.0	4100.0	5010.0

* Gordon's technique for estimating the costs of hydropower costs as used in this model is described and discussed below. These tabulated costs compare closely when they are brought to the present and range between 390 and 460m$. The United Nations Water Resources Series value is obtained from the curves shown in figure 6.7

6.5.2 Gordon's Technique for Estimating Hydropower Costs

This alternative involves the use of hydropower cost functions. The first function, given in equation 6.5 and plotted in figures 6.8 (a) relates the total equipment cost of a hydropower scheme to its installed capacity and head by separate exponential indices. The function is a modification by Gulliver, J.S (1984) of one originally developed by Gordon (1978). Figure 6.8 also illustrates the curve of the equation 6.5.

EC=9600*((kw)**(0.82))*(GHEAD)**(-0.35) 6.5

Site factor, SF=9.8*(KW)**(-0.14) 6.6

The site specific nature of site civil works costs necessitated the introduction of a site factor (equation 6.6) by Gordon and Penman, (1979) and modified by Gulliver, (1984). The total project cost is obtained by multiplying the equipment cost by the site factor. For plants larger than 5MW, a site factor of 3 is recommended. For plants below 5MW, the site factor is limited by the equation of the envelope given in fig. 6.9.

Generally, site factors for locations with nonexisting powerhouses lie between the two envelopes provided in fig 6.9. Gulliver warned that the site factors utilised herein do include all site costs except penstocks, diversion works, transmission lines, remote access facilities, and difficult terrains, and long tunnel

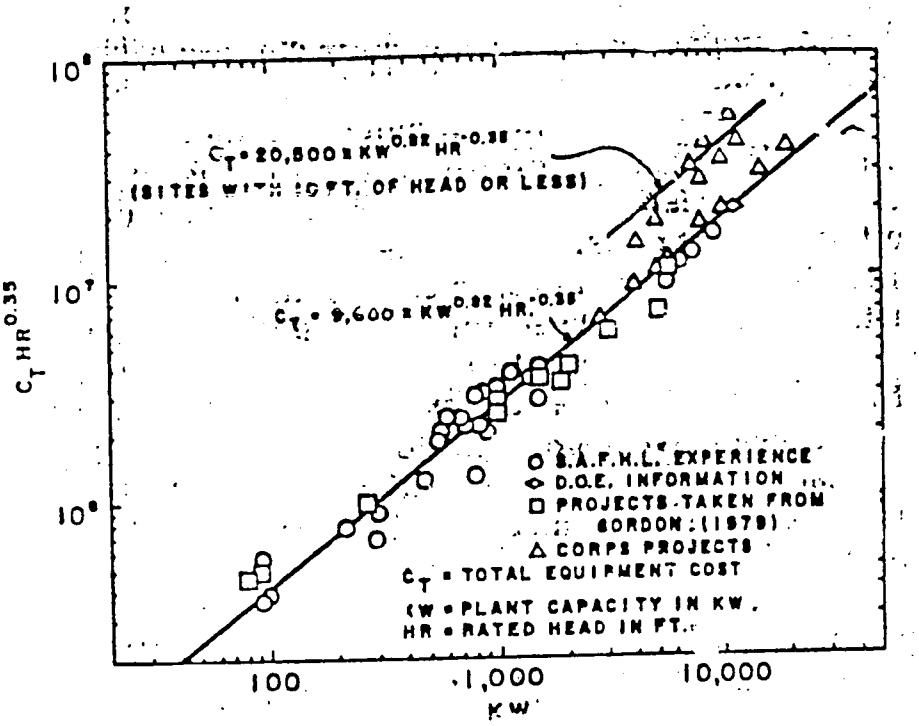
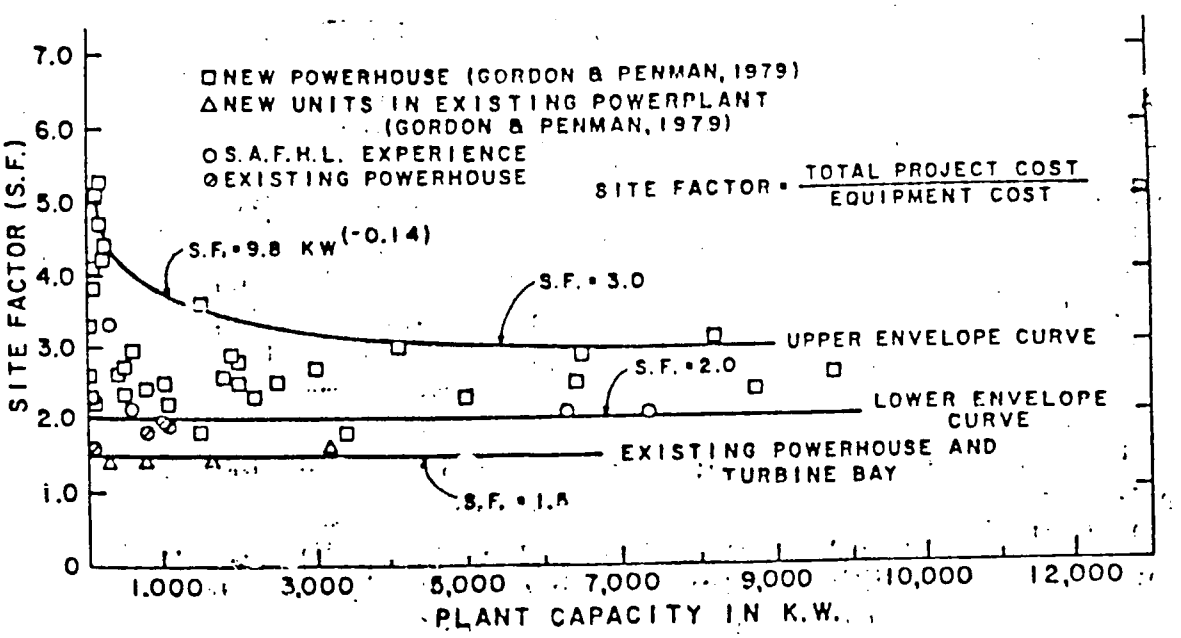


FIG.6.8—Total Equipment Cost from Recent Quotations from Manufacturers versus Plant Capacity and Rated Head, 1982 Base Year

Fig 6.9

AFTER GORDON,1978 MODIFIED BY GULLIVER,1984



transfers of water to power stations. When these additional items are involved, the site factor must be reviewed upwards by a predetermined coefficient. In accordance with the discussion above, the total cost of a power plant in Nigeria can be determined by the equation.

PCTA=SF*EC+TNL*TNNC 6.7

where

PCTA is the total costs for incurred for power generation facilities.

SF is the site factor which is normally 3. For the Donga basin reservoirs, a higher value of 3.5 has been used to reflect the extra costs from additional items such as a difficult terrain which lacks access facilities and the recommended use of an underground power station location.

EC is the equipment cost.

TNL is the transfer tunnel length in metres

TNNC is the unit cost of tunnel per meter

Table 6.6 Project Costs Estimates for Some Existing Nigeria Reservoirs using Gulliver's and Gordon's Equations as Compared to Actual Costs or Consultant Estimates.

	MW	HEAD (M)	GORD'/GULL'	ACTUAL	CONSUL'	S.E	SOURCE
KAINJI'72	320	30	220.0	255.0	255.0	1.5	GORDON'84
KAINJI'78	760	30	315.0	-	345.0	1.5	NEPA '76
JEBBA'84	540	27	248.0	300.0	280.0	2.0	NEPA '84
SHIRORO'88	600	110	264.0	450.0	305.0	2/5	NEPA '86
GEMBU(proposed)	450 /1300	960	-		382.0	5.0	DIYAM'85

S.E is the shadow exchange rate used in Nigeria in the stated year. From Table 6.6 the cost values obtained by the use of Gordon/Gulliver technique is of the same order as that obtained from consultants estimate or actual expenditure. The small differences may be due to contigencis and other unaccountable factors.

For example, the actual cost of the Shiroro project is high because of all kinds of delays and escalation due to bad project financing. Shadow exchange rates for the respective years have been used to convert actual costs from the Nigerian currency to U.S $. For Jebba which was completed in 1984, a shadow exchange rate of 2N to $1 was utilised. For Shiroro, whose completion has been delayed to 1988/89 since it began in 1979, a series of shadow exchange rates ranging from 2N to $1 to 5N to $1 have been used.

Values of 9,600 and 15,000 were used as the constants in equation 6.5 for Jebba and Shiroro respectively and a site factor of 2.5 was used for both stations. The higher value of the constant was used for Shiroro because the Shiroro site is a more rugged and difficult and less accessible terrain than that at Jebba. The locations of these reservoirs are shown in figure 6.6

Gordon and Gulliver (1984) confirmed that the constant in equation 6.5. varies from 20,500 to 9,600 for heads varying from 2.4m to 3.7m. On further investigation, the lower to mean values (9600-1500) appear to given acceptable results for higher heads, when tested on Nigeria reservoirs by the writer (Table 6.6). The equation was developed originally for small hydroplants up to 40-50 MW. Gordon (1983) confirmed the applicability of the equations to a wide variety of medium to large hydropower plants. Figures 6.10 to 6.14 illustrate and summarise the results of Gordon's work for all kinds of hydropower plants built around the world. Preliminary desk application of the relations (eq 6.5-6.7) and the curves (6.10-6.13) to the three existing hydropower plants in Nigeria, Kainji, Jebba and Shiroro met with some degree of success (Table 6.6). The results obtained from both the use of equations 6.5-6.7 were found to be of the same order as actual or estimated values. The results and the actual costs are consistent with Gordons cost curves of figure 6.10-6.13.

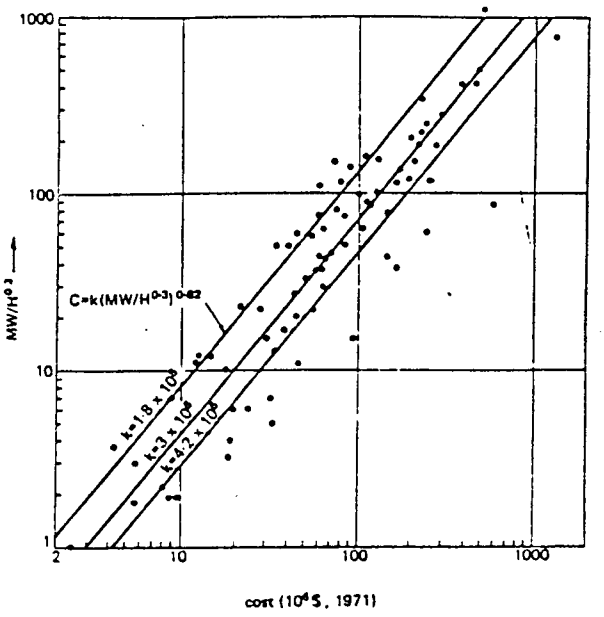


Fig.6 10 Relationship between cost and $MW/H^{0.3}$ for projects estimated or built between 1956 and 1971, and costs escalated at three per cent per annum to 1971.

AFTER GORDON, 1983

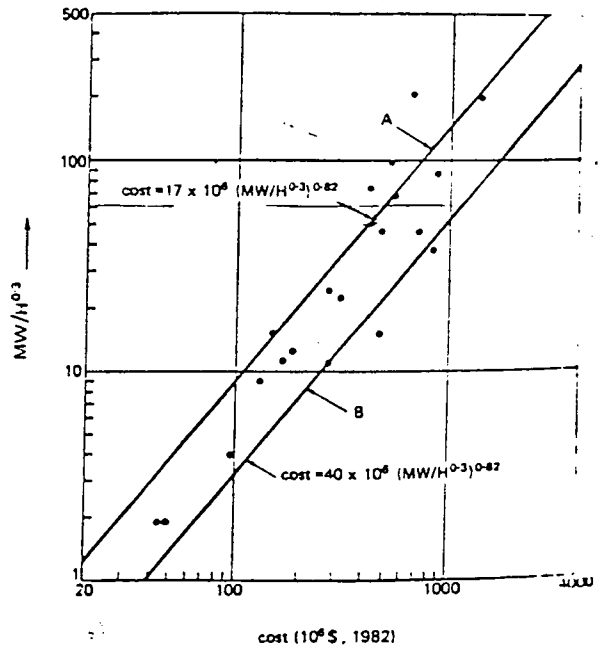


Fig.6.11 Relationship between cost and $MW/H^{0.3}$ for projects where head exceeds 350 m.

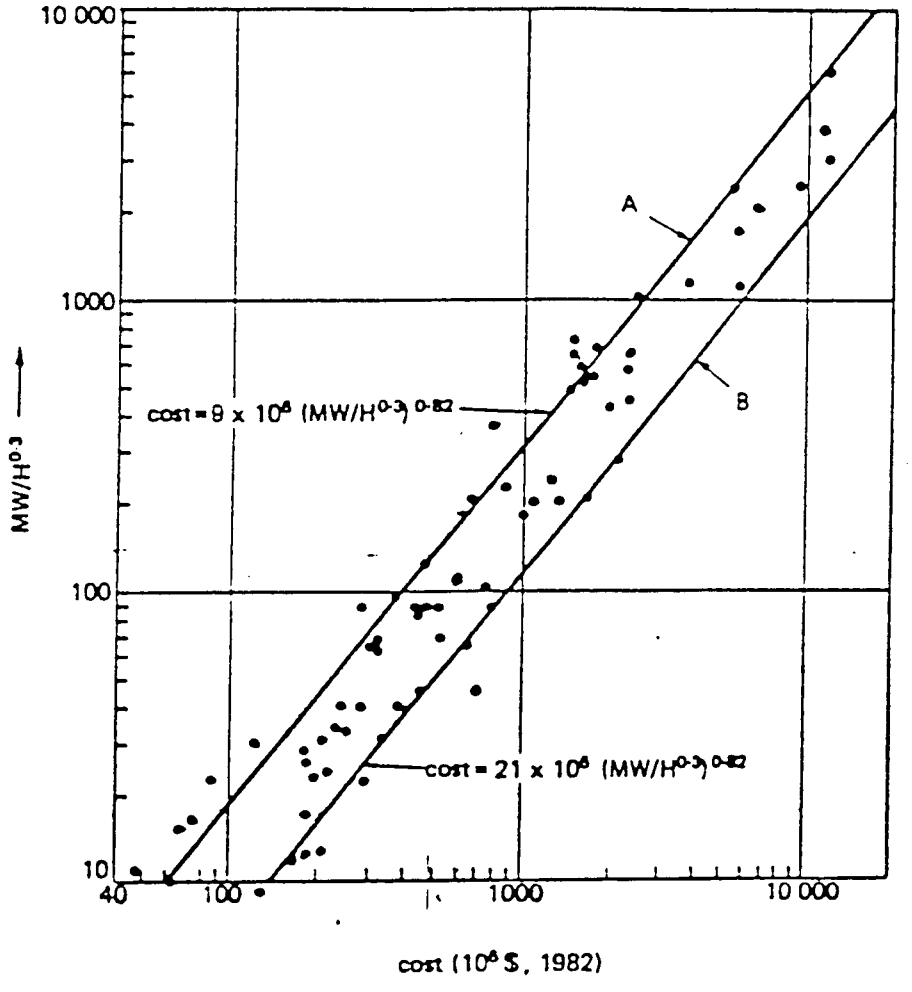


Fig.6.12 Relationship between cost and $MW/H^{0.3}$ for projects estimated or built between 1977 and 1982, and costs escalated at 12 per cent per annum to 1982.

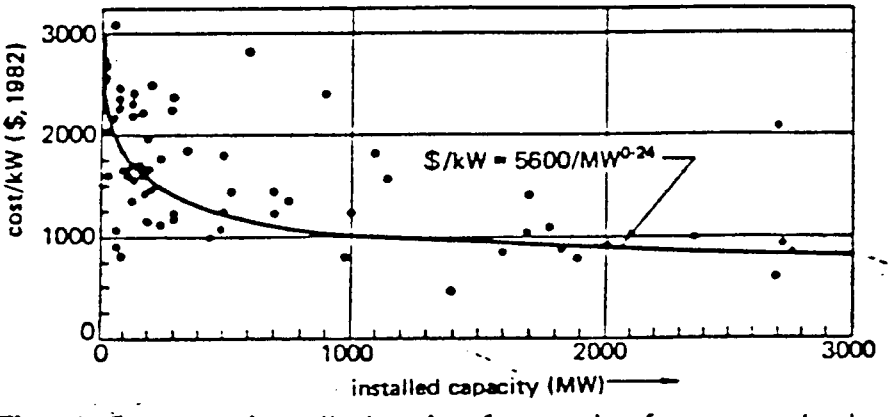


Fig.6.13 Cost per installed unit of capacity for recent hydro projects.

Due to their estimating potential, the equations 6.5, 6.6 and 6.7 have been incorporated into the economic simulator and they have been used for calculating all power related costs used in benefit cost analysis. The premier cost estimation approach from basic cost data of existing Nigerian reservoirs discussed above (section 6.3.1) provides an alternative for comparison with the Gordon-Gulliver technique.

The model distributes the total construction cost uniformly through the construction period to obtain an average annual construction cost which is used as a base for discounting.

PCA=PCTA/CP ..... 6.7 (a)

6.5.3 Estimating Non-Power Costs

These costs include irrigation investments, water supply costs, flood control, and navigation. These are discussed separately below and the reader is reminded that the sequence is illustrated in figure 6.2.

(a) Irrigation Development

The model estimates irrigation costs in components. First, there is the cost incurred as result of the reservoir and dam structure construction. The share of the total reservoir costs, allocated to irrigation is made proportional to the amount of water used or supplied. The

other shares go to other uses. This computed as follows:

$$IDC=GFRACT*TOTST*DMCM*\frac{(1.0-PRFRA)}{100} \quad \ldots\ldots\ldots 6.8$$

where:

IDC is development costs assigned to irrigation in U.S$

(1.0-PRFRA) is the fraction of water devoted to non-power uses, specifically, irrigation and water supply. This is inputed as data.

GFRACT is the fraction of non-power water allocated specifically to irrigation only. This is input as required.

DMCM value is as provided for the power case. This is the cost of the reservoir system per unit capacity.

(b) Estimating Irrigation Infrastructure Costs

The annual irrigation infrastructure costs has been related to the hectarage under cultivation by an estimated unit cost. This given by the equation;

$$CI=HECT*IIC*\frac{(1.0+YL)^{(LOP+Q)}}{(LOP+Q)YL} \quad \ldots\ldots\ldots 6.9$$

where

HECT is the hecterage under irrigation which is limited by the amount of water supplied. HECT is obtained by the equation,

HECT=WI/KA where;

WI is the water supplied per day in cubic metres to irrigation which is obtained from the equation;

WI=GHEAD*TOTQD*(1.0-PRFRA)*86400 .....6.10

TOTQD is the total annual diversion in cumecs for the case where a daily diversion set of rules are operated. For a storage-elevation set of rules for release, an accumulated total annual release, YDIVQ is cumecs is used instead. In short, if TOTQD is not utilized, it is assigned a value zero and it is automatically set equal to YDIVQ and vice versa. The same condition holds for the power generation case, already described above.

KA is the quantity of water in cubic metres, required for irrigating hectare of a specified crop or a defined combination of crops in one growth day. Rice has been chosen in this case study. Its irrigation water requirement value of 16,000 cubic metres for 128 growth days in deduced from table 6.7 as obtained from the literature (Mazumder, 1983). A KA value of 125 cubic metre for rice has been used as basic model data using the Indian case, since data for the Nigerian case was not available. India has a similar monsoon climate like the

Table 6·7 Some of the Principal Indian Crops and Their Water Requirements (Typical)

Name of Crop	Variety	Sowing Time	Period of Growth (Days)	Expected Yield (Quintals/ hectare)	Water Requirement FIR (cm)	No. of Irrigation	Remarks
Rice (kharif)	IR-8 Dwarf Varuna T-21 ⎫ U.P. T-3 ⎭ variety	Mid June to Mid July	115–140	60 (Dwarf) 40 (U.P.)	160	10 to 15: If no rain, apply irrigation once a week	Tillering and flowering most critical time
Maize (kharif)	Hyb. Ganga-2 Hyb. Ganga-5 Sona, Kisan	June to July	90–100	40–45	62	Two to three: Irrigation of 15 cm prior to monsoon. One irrigation of 6 cm whenever rain gap is long	Flowering most critical stage
Jawar (kharif)	H.Y. CSH-1	June to July	120–125	30–40	50	Irrigation at 25% AMC to bring soil at field capacity	Preflowering most critical stage
Wheat (rabi)	One gene—Sonalika Two genes— Kalyan Sona Three genes— Hira, Moti	Nov. (can be ext. up to mid-Jan.)	125–135	60–70	40	Six to seven	Crown root initially tillering most critical period.
Gram (rabi)	Kabuli ⎧K-47 ⎩K-57 T-3	First two weeks of October	165–175	Kabuli—10 to 12 T-3—25 to 30	16	Two: First watering 65 to 70 days after sowing and second at flowering if no winter rain.	
Mustard (rabi)	T-9 (Toria) T-35 (Lahi)	Early Sept.	90–100	6–9	16	Two: First at sowing and second at fruiting if no winter rain	
	T-15 (Yellow Sarson) D.S-17 (Surphali)	Oct. & Nov.	125–150	15–20			
Sugarcane (perennial)	Co. 1148, 1158 Co. 6425, 1007	1. Spring: Feb. March	270–300	1000	120-130	5 to 6 before monsoon 1 to 2 after monsoon	Premonsoon: First irrigation 6 weeks after germination; then at 15 days interval Post monsoon: At 30 days interval
	Co. 971, 561	2. Autumn: Sept.-Oct.	375–400	1250	150	7 to 8 premonsoon 1 to 2 postmonsoon	

a Matzunder 1983

case study area in Nigeria. This justifies the use of the available Indain value.

IIC us the unit cost of irrigation infrastructure in $U>S per hectare. This is dependent on the type of irrigation and the technology employed in its construction. The value is carefully worked out and read in as data. A value of 1000 has bee estimated for the Donga basin using a cheap type of infrastructure (e.g Furrow or Border types).

(c) Estimating Irrigation Water Costs

Sometimes water is sold to farmers, usually at a small enough rate in third world countries. The annual cost of this water to the farmer is included as a component of the irrigation cost. This cost is incurred as a result of the direct use of water and it is evaluated from the equation;

WC=WC*HECT*T 6.11

where;

HECT = Hectarage

T = Fraction of Year devoted to Irrigation

WC is the unit cost of the water supplied in cents per cubic metre. This is site, region, or country specific. A value of 0.1 cent per cubic metre is used in accordance with Nigeria rates. In the U.S. the rate ranges between 0.14 and 0.2 cents per cubic metres (Mutreja, 1986; Goodman,1984; James and Lee, 1971; Lee, 1978)

(d) Evaluating Total Irrigation Costs

This is the summation of all the component costs plus the overall operation and maintenance costs. This is an annual figure which is accumulated throughout the life of the scheme. The summation is expressed in form of the equation given below:

$$CIRG=WCA+CI+OMCI+\frac{IDC}{CP} \quad \text{.....6.12}$$

CIRG is the total annual irrigation costs

OMCI is the annual operation and maintenance costs, taken between 1 and 5 percent of the total costs. A value of 3% is used.

(e) Estimating Present and Annual Values of Total Irrigation Costs

The present value of the estimated total annual irrigation costs when accumulated over the project life using the appropriate inflation or escalation rates, is obtained by multiplying two seperate components, the long term annual cost component and the reservoir construction costs component with their required series present worth factors as follows;

$$TCI=(WCA+OMCI)*FACT1+\frac{(IDC*FACT2)}{CP} \quad \text{.....6.13}$$

where:

$$FACT2=((1.0+RY)^{CP}-1.0)/RY*(1.0+RY)^{CP} \quad \text{.....6.13 (a)}$$

The long term annual costs of irrigation is obtained by multiplying the undiscounted total irrigation costs by the

cost recovery factor as shown below:

ARGC=CRF*CIRG .....6.13 (b)

(f) Estimating Water Supply Cost

The water supply costs are obtained by multiplying the quantity of water supplied per capita in cubic metres by a unit cost. The equation below describes the method.

$$CAP=(1.0-GFRACT)*(1.0-PRFRA)*\frac{TOTQD}{C} \quad .....6.14$$

WSCN=CAP*KC+ALCK*(1.0-GFRACT)*IDC .....6.14 (a)

where CAP is the population supplied. CAP is computed by dividing the quantity of water required or supplied by the specified mean consumption rate per capita, C in cubic metres. A C value of 70 litres (0.7E-06 cubic metres) per capita per day is recommended by the World Bank for the whole of Nigeria (rural and urban) and this value has been utilised (NFMWR, 1986). CAP is assumed to increase annually by 1 percent. Tables 6.8 indicate typical water demand figures for the World by countries. The Nigerian demand value for large cities is about two times the national average figure of 70L/head/day, and this is comparable to those of other third world countries shown in table 6.8.

Table 6.8 Water Demand by Countries.

PLACE	COUNTRY	DOMESTIC/CAPITA CONSUMPTION (l/head/day)	YEAR
MALE	MALVIDES	40-100	1981
KATHMANDU	NEPAL	96	1973
CAIRO	EGYPT	157	1966
		181	1967
PORT SAID	EGYPT	127	1966
PROVINCIAL TOWNS	EGYPT	39-149	1977
PROVINCIAL TOWNS	LESOTHO	107-15	1977
ISTANBUL	TURKEY	119	1976
LIMA	PERU	212	1980
SANTA CRUZ	BOLIVIA	124	1981
SUCRE	BOLIVIA	98	1981
CAMIRI	BOLIVIA	137	1982
PARIS	FRANCE	143	1948
		159	1978
CAPE TOWN	S.AFRICA	140	1978
AMSTERDAM	NETHERLANDS	91	1948
		123	1978

SOURCE - WATER PRACTICE MANUALS (IWES, 1983).

PK is a fraction of the reservoir development cost allocated to water supply only. The remaining cost is collectively allocated to other beneficiaries such as navigation, flood control and fishery etc.

KC is the unit annual cost of supplied per capita in cents. A value of 0.025 has been evaluated and used from a water rate of 0.1 cents per cubic metre specified previously. Local information on the price of water supply in urban and rural Nigeria has been utilized.

The present value of this cost is obtained in the usual way by multiplying by the appropriate discount factor, thus

WSC=WSCN*FACT1 .....6.14 (b)

The annual value is obtained as usual by multiplying by the total undiscounted costs by a cost recovery factor as shown below.

AWSC=CRF*WSCN*(LOP+Q) .....6.14 (c)

The present value of the total investment costs for consumptive use of water is the sum of the present value of the total costs of irrigation and that of water supply. It is given by the equation

TWSC=TCI+WSC .....6.14 (d)

(g) Evaluation of other Costs-Fishery and Flood Control

In costing many reservoirs, the cost of fishing infrastructure (e.g. hatcheries) and flood control facilities are regarded as small and are input as zero. When it is significant however, then some pre-estimated

value of unit cost is evaluated. The cost per unit storage for fisheries and that per unit outflow (release) for flood control are estimated and supplied as model input. The final cost values are obtained by the multiplication of these unit costs with the storage capacity and annual reservoir outflow respectively. The results are annual and are accumulated accordingly over the project life and discounted appropriately to the present. The equations utilized are

$$OTH=(FC*TOTST+FFC*AQOUT)*FACT1+((1-ALCK)*(1-GFRACT)*\frac{IDC}{CP}*FACT2 \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.15$$

where OTHC is the present value of total non power costs
FC is the fish cost per unit capacity of reservoir

FFC is the flood control costs per unit volume of annual reservoir outflow

These two unit costs are set to zero or some minimum values if the fish flood control costs are considered negligible, otherwise they are carefully estimated. These costs including that of recreation and navigation have been assigned a value zero as they are considered minimum in the test Donga Basin.

FACT1 is a discounting factor already used

In a similar, way, tourism costs are related to reservoir area by the use of a unit cost- cost per unit area. These are included by the model, details are available in the users guide.

(h) Evaluation Pumping Cost for Irrigation and Non-Irrigation Uses

These are also computed by the model if required by evaluating the product of available water with the appropriate heads and efficiencies according to the equations.

PMPPI=((1.0-PRFRA)*GFRACT*TOTQD*PHEAD&EFFI)*.001 ....6.16

PMPWS=((1.0-PRFRA)*(1.0-GFRACT)*PHDWS*TOTQD*EFWS)/1000 .6.17

PMPPG=(FRACT*TOTQD*PHEADG*EFFG)/1000 .....6.18

PMPPI, PMPWS, PMPPG are pumping power required for irrigation, water supply and general transfers respectively.

PHEAD, PHDWS, PHEADG are their corresponding heads and FRACT is a fraction of total diversion devoted to intrabasin or interbasin transfers, if applicable.

RPW=ALPHA*(PMPPG+PMPWS+PMPPI) where RPW is the recoverable or recovered power from the water en route to various destinations and ALPHA is an inputed fraction of recoverable power compared with pumping power.

CRPW, the cost of harnessing recovered power is evaluated thus

$$CRPW=(RPW)*\frac{TCOST}{TRAT} \quad \ldots\ldots\ldots\ldots 6.19$$

where TRAT is the turbine rating in MW and TCOST is the turbine unit cost.

BPW, the balance power required for pumping is calculated from the equation

BPW=(PMPPG+PMPWS+PMPPI)-RPW 6.20

The cost of this balance power, CSPW, is obtained by multiplying this balance power by the same unit cost as input for the hydropower generation costs, but this is read in separately as the value may be different (chapter or more expensive). The product is

CPWB=BPW*CSPW 6.21

where CSPW is the unit production cost.

The pumping station is also costed by the equation

PSCT=CUD*TOTQD*PSCF*3.0

where PSCF is an inputed pumping station cost factor obtainable from tables and graphs in the literature. CUD is factor that combines the fraction of the total diversions passing through the pumping station and pump efficiencies factors. CUD is computed and input as data.

Pipe, tunnel, and canal costs, whichever is used, are also provided as indicated below.

PIPC=TOTAL VOL*DENST*CST 6.22

The dimensions and density of the material are input as data. CST is the cost of the materials utilised per unit mass.

TNC, the tunnel costs is simply the product of the total tunnel lengths and the inputed unit costs. This is obtained from information in the literature and from consultants.

TNC= TC*TLEN. 6.23

TC is the unit costs and TLEN is the total lengths.

CLC=A*LENC*CCHR+(1.0-A)*LENC*CCNR 6.24

where

LENC is the total canal length, A is a factor indicating the part of canal passing through hard rock. CCHR and CCNR are unit costs in hard and soft rocks respectively.

All the last four costs incurred above are site specific. In most sites, they are assigned zero. Their total sum is evaluated below

TPCP=CPBW+PSCT+PIPC+CLC+TNC 6.25

Their associated operation and maintenance costs are obtained by multiplying by the relevant factor as shown below,

OMPCPC=OMF*TPCP 6.26

where OMF is the operation and maintenance factor, which may be set at 3 percent or less.

The grand sum of the above costs is simply,

TTPCP=OMPCPC+TPCPC 6.27

The total annual cost of these site specific costs, ANPCPC, is obtained by the use of a cost recovery factor. This is stated in the equation below.

ANPCPC=CRF*TPCPC+OMPCPC+TAXINS*TPCPC 6.28

'Taxins' is the tax and insurance factor.

The present value of the sum of these costs is calculated in the usual way from the equation

WPCPC=OMPCPC*FACT1+TPCPC*FACT2 .....6.29

by the use of a cost recovery factor. This is stated in the equation below.

ANPCPC=CRF*TPCPC+OMPCPC+TAXINS*TPCPC 6.30

'Taxins' is the tax, depreciation, and insurance factor which has already been specified as 3% of the costs involved.

The present value of the sum of these costs is calculated in the usual way from the equation

WPCPC=OMPCPC*FFACT*FACT1+TPCPC*FACT2 .....6.31

where FFACT, FACT1 and FACT2 are the usual discounting factors already described.

The costs and unit costs for pumping, and canals have been assigned a value zero as none of the two are utilized in the Donga basin. There is also no case of interbasin transfer. Costs involving intrabasin transfer of water have been included in dam and lake costs.

(i) Grand Total Costs

The model sums all nonpower and all costs, inclusive of their present and annual values for comparative analysis, if required. These are given below as follows,

FTC, the final undiscounted total costs is given by the sum

FTC=TLTC+TNPCN+TPCPC 6.32

The present value of the final total costs is summed up as follows

FTCW=PWI+TNPC+WPCPC 6.33

The final total annual cost is a summation of the component annual costs, thus

FAC=ANC+ANCOST+ANPCPC 6.34

Alternately, the final annual cost can be obtained by the equation,

FATC=CRF*FTCW 6.35

Note that both FAC and FATC are the final annual costs.

6.5.4 Evaluation of Benefits

The model computes benefits of hydropower, irrigation, water supply, flood control, navigation, fisheries, and tourism, seperately. Each of the benefits are related to the features of the reservoir by a unit benefit factor which is evaluated and input as data. The sequence of benefit calculations is illustrated in figure 6.4

(a) Estimating Hydropower Benefits

Power benefits are calculated by multiplying the unit energy sales rate by the total amount of energy generated. This is on the assumption that all energy generated are sold. This benefit is given by the equation

ER=ANENG*ECOST/100 .....6.36

where ER is the total cost of annual energy supplied which includes both primary and secondary energy in dollars.

ECOST is the unit price of energy in cents per KWh. The current Nigerian mean power charge or 4 cents per KWh has been used. Table 6.9 gives the comparative values of the current electricity tariffs in some West African Countries.

Table 6.9: Electricity Tariffs in some West African Countries

COUNTRIES	TARIF US cents/Kwh	Nigerian Kobo/Kwh
BENIN	16	64
BURKNA FASO	22	88
CAMEROON	10	40
CORTE DE IVOIRE	14	56
GHANA	3	12
GUINEA	15	60
NIGERIA	1.75 (3)*	8 (30*
SENEGAL	16	48
TOGO	16	48

The official rate utilised here is the 1987 rate of N4=$1
Source - NEPA (1987) * New Value

The tariff value of 4 US cents per Kwh has been utilised in this work because a two and a half fold increase in tariffs from 8k to 20 has just been approved and announced in 1988. This four cent tariff is still too low compared with the tariffs of other third world countries (Table 6.9). The tariffs in the developed world are even higher,

the monetary benefits from hydropower generation from the Gembu scheme therefore are expected to be relatively low, especially when compared to costs.

This annual energy returns evaluated in equation 4.36 are accumulated, using an appropriate mean inflation rate (of 6%) over the whole project life by use of the quation

$$TPB=ER*\left(\frac{(1+YL)^{(LOP+Q)}-1}{YL}\right) \quad \ldots\ldots\ldots\ldots\ldots 6.37$$

The present value of the total returns from accumulated energey, PWB, is given by the expression

$$PWB=ER*FACT*FFACT \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.38$$

where

FACT and FFACT are the non-uniform series and single payment discounting factors respectively.

The annual benefits are obtained by multiplying the present values by the capital recovery factor as indicated by the expression;

$$APB=CRF*PWB \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.39$$

(B) Evaluation of Irrigation Benefits

The irrigation benefit values are obtained, simply, by noting and inputting the irrigation yield from the maximum hectarage obtainable with the water supplied.

The yield per hectare of the crop is obtained from experience and from the literature (Table 6.8). The current market price of the crop is obtained and input as data.

The annual irrigation benefit, AIBA, is expressed by the equation

$$AIBA=HECT*Y*MP \qquad \ldots\ldots\ldots\ldots\ldots\ldots 6.40$$

where

HECT is the maximum calculated cultiviated hectarage that supplied water can irrigate (already discussed).

Y is the mean crop yield per hectare. For rice a value of 4 tonnes per hectare is used.

MP is the average market price of crop or group of crops in dollars per tonne. The value of 400$ per tonne is the current market price of rice in Nigeria.

The accumulated irrigation benefit, IBA, is evaluated from the expression;

$$IBA=HECT*Y*MP*\left(\frac{(1+YL)^{(LOP+Q)}-1}{YL}\right) \qquad \ldots\ldots\ldots 6.41$$

The present value of the total irrigation benefit in dollars is given by the expression

$$MBA=IBA*FACT/100 \qquad \ldots\ldots\ldots\ldots\ldots\ldots 6.42$$

The annual values of the irrigation benefits are obtained in the usual way i.e.

$$AIRB=CRF*MBA \qquad \ldots\ldots\ldots\ldots\ldots\ldots 6.43$$

Estimating Water Supply Benefits

Water supply benefits are obtained by the use of a unit price per person and the number of persons supplied is calculated by dividing the amount of water supplied by the consumption rate per capita. The equation below describes the proceedure completely.

$$WSBAN=CAP*ZQ*\underbrace{\frac{(1+YL)^{(LOP+Q)}-1}{YL}}_{AC} \qquad \ldots\ldots\ldots\ldots\ldots\ldots 6.44$$

where WSBAN is the accumulated water supply benefits in U.S. dollars at YL inflation rate.

CAP is the population evaluated from the expression

$$CAP=(1-GFRACT)*\frac{(1-PRFRA)}{C}*TOTQD \qquad \ldots\ldots\ldots\ldots\ldots\ldots 6.45$$

C is the mean consumption rate per capita per day (70). The expression in the numerator is the fraction of total diversion devoted specifically to urban and rural water supply. The value of C has already been discussed.

The present and annual values of water supply costs are given by the expressions

$$WSBA=CAP*ZQ*FACT1*^{AC} \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.46$$

$$AWSBA=CRF*WSBA \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.47$$

respectively where ZQ is the mean water charge per capita supplied from local information and cross checked with United Nations and World Bank data (UNWRS42, 1972).

CRF and FACT1 are the capital recovery and non uniform series present worth factors respectively.

Other Benefits

(a) Estimating Navigation Benefits

The model relates annual navigation benefits to the mean annual outflows from the reservoir by a benefit factor and this is expressed by the equation

$$NAVBEN=BN*AQOUT \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.48$$

NAVBEN, NB, and AQOUT are the annual navigation benefit, the benefit factor and the mean annual reservoir outflow respectively. The navigation benefits are accumulated over the project life of the reservoir by the expression

$$NVB = NB * AQOUT * ((1+YL)^{(LOP+Q)} - 1)/YL \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.49$$

The present and annual values of the navigation benefits are given by the respectively expressions

$$NVBW = NB * AQOUT * FACT1 * AC \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.51$$

$$ANVB = CRF * NVBW \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.51$$

(b) Evaluating Fish Benefits

The model evaluates fish benefits by relating fish yields to the capacity of the reservoir using a fish yield factor, as shown below.

$$FYY = FY * TOTST \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.52$$

$$FB = FYY * FMP \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.53$$

Considering the experience from the existing Kainji reservoir (Nigeria), the annual fish yield FYY is expected to reach 3000 tonnes annually (NEPA), Motor Columbus, 1981). Consequently, the value of FY used is 0.06E-06 per cubic metres. FMP, the fish market price is the current price of fish in dollars per tonne in Nigeria. A mean value of 1000$ per tonne has been used.

$$FBC = FYY * FMP * ((1+YL)^{(LOP+Q)} - 1)/YL \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.54$$

$$FBA = FYY * FMP * FACT1 * AC \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.55$$

$$AFB = CRF * FBA \quad \ldots\ldots\ldots\ldots\ldots\ldots 6.56$$

FYY, FY, FB, FMP are the mean annual fish yields, fish yield factor, annual fish benefits, fish market price respectively.

FBC, FBA, and AFB are the accumulated long term value, the present value and the annual values of fish benefits respectively.

(c) Estimating Flood Control Benefits

The model evaluates this by relating flood control benefits to the reservoir storage capacity by an appropriate factor which is estimated and input as data. This is similar to the navigation case. The equation involved is

FLDCTB=FCB*TOTST 6.57

where FLDCTB, FCB, and TOTST are the annual flood control benefits, a flood control benefit factor and the reservoir capacity.

The accumulated benefit is given by the expression

$$FLDCTBC=FCB*AQOUT*\frac{((1+YL)^{(LOP+Q)}-1)}{YL} \quad .....6.58$$

AFCB=CRF*FCBA 6.60

FCBA= FLDCTBC * FACT1

The flood control and navigation benefits are first assessed and their respective benefit coefficient worked out before being fe d to the model.

Tourism and recreational benefits are related to the surface area by the use of an estimated unit benefit factor, in a way similar to that mentioned for tourism infrastructure costs.

6.6 Final Remarks

Rockfill embankment dams have been proposed for all five reservoir sites in the Donga Basin (Figure 6.6) by the various consultants who carried out preliminary development studies in the catchment (Diyam/Binnies 1983, Mrt 1978, and A.L.T.S/Lideco, 1978). This has been included in the cost functions of the model.

Typical results and plots of a trial economic analysis model run for Gembu reservoir have been presented in Table 6.4.

The economic simulation model described herein has a great potential for analysing river basin projects involving the release of water. The model, with slight modifications, can be applied to all aspects of river basin operations consisting of reservoirs, channel or groundwater withdrawals. Prior to its use in the river basin management system tests carried out in chapters 8 to 11, an assessment of the magnitude of the global climatic change and its impact on the Donga basin case study is undertaken in the next chapter (7). The magnitude of the reservoir change is also assessed. These are specifically used for testing the system in Chapters 8 - 11.

CHAPTER 7

CHAPTER 7 : THE MAGNITUDE OF THE GLOBAL CLIMATE CHANGE

The main objective of this chapter is to assess the global climatic changes and evaluate the order of magnitudes involved. The magnitude of the impact of this change on the Donga Basin is also deduced. This information is vital in testing the River Basin Management system described and discussed in chapters 5 and 6 in order to assess the posssible future impact of the changes on the water balance and sediment yields of the catchment and the economics of the reservoirs proposed therein. Section 7.1 highlights the causes of global climatic changes. Section 7.2 discusses the extent of the expected changes. Section 7.3 is devoted to a quantification of the global climate changes. Section 7.4 focusses on the magnitude of the change in the humid tropics, the main concern of this thesis. Section 7.5 describes the current hydrological and physical conditions in the basin. In section 7.6 the magnitudes of the climate changes expected in the Donga Basin due to the global effect are provided. Section 7.7 concludes the chapter.

7.1 Introduction

Man has become a force causing enormous and rapid environmental changes on this planet. His presence is strongly felt from the north to the south pole and from the great depths of the ocean to the highest zones of the stratosphere. The main challenge facing him today is to understand and to predict the consequences of these actions and to bring this knowledge to bear on policies that will preserve the viability of life on the planet to ensure the future survival. At the same time, he should be able to maximise the benefits offered by the earth's resources which are continually diminishing.

As a result of human intervention, the atmosphere, an extension of the biosphere, is reported to be changing rapidly. This change has been linked to an expanding human poputation and man's aspiration for a better life. This atmospheric change is most readily reflected in climatic changes now being experienced worldwide as reported earlier in Chapter One. In this chapter, the order of magnitude of the expected change is assessed and presented.

A UNEP/WMO/ICSU conference held in Villach, Austria in Oct 1985 and a workshop by the same group held in Bellagio in Nov 1987 stated and confirmed that certain "greenhouse" gases, notably carbon dioxide (CO_2), methane (CH_4), nitrous oxide, ozone (O_3), and chlorofluorocarbons (CFCs) are emitted to the atmosphere from natural and or manmade sources. These, though, present in small quantities have an important effect in the trapping of energy originating from

the Sun, in the form of heat , near the Earths surface. The presence of carbon dioxide and other GHGs leads to increased absorption of longwave solar radiation and a heating of the atmosphere. This phenomenom is termed the "greenhouse effect". The amounts of these gases in the atmosphere are increasing significantly due to human activities, notably industrialisation, and so is the "greenhouse effect"

Consequently, it has been confirmed by a concensus of scientists prior to and at Villach (1985) and Bellagio (1987) that during the next one hundred years, a rise of global mean temperatures larger than any previously experienced in human history could occur.

7.2 Facts and Figures

At Villach (1985), some of the points that were generally accepted are :

(1) possibly, that, if present trends continue, the combined concentrations of all the GHG's would be equivalent to a doubling of the carbon dioxide concentrations from pre-industrial levels, as early as the 2030's.

(2) that computer modelling techniques have indicated that this carbon dioxide doubling will lead to 1.5 to 4.5 degree C (or more) increase in average Global surface temperatures. (figure 7.1)

(3) that apart from the GHG's which has the main influence on climate, other factors such as aerosol concentrations (which also affects the Earth's ozone layer), changes in vegetation, and changes in solar energy input may also be involved.

(4) that computer model predictions indicate that warming will be greater in the high latitudes during late autumn and winter than in the tropics; annual mean runoff may increase in high latitudes; summer dryness may become more frequent than over the continents at middle latitudes in the Northern Hemisphere.

(5) that tropical regions will have smaller temperature increases than the average global increase; and that potential evapotranspiration will probably increase throughout the tropics, whereas in moist tropical regions, convective rainfall will increase

(6) that a Global warming of 1.5 to 4.5 degrees C would lead to a sea level rise of 20-140 cm.

(7) that observations show that global mean temperatures have increased by between 0.3 and 0.7 degrees C during the last 100 years. This is found to be consistent with modelled results.

(8) that the effects of the climate changes resulting from all GHG's which is equivalent to a doubling of carbon dioxide concentrations, could have profound effects on Global Ecosystems, Agriculture, Water Resources and Sea Ice conditions.

From the proceedings of the Bellagio workshops (1987), a summary of the magnitudes of some of the more specific Global climate changes which are expected and their probable effects between now and the end of the next century are presented in table 7.1 and figure 7.1. The historical record of actual climate changes since 1850 is also included in perspective in the illustrations. Further, a few important facts with comments on table 7.1 and figure 7.1 have been extracted and these are outlined as follows :

(a) Higher temperatures and sealevel rises are expected in the next one hundred years.

(b) Worldwide precipitation could be expected to increase by 2 - 3% for each degree of global warming. The new climate can be expected to be generally wetter than that of the recent past.

(c) The rate of climatic change in this century significantly exceeds the average rate for the last one.

(d) In figure 7.1, the sealevel changes provided would be the same throughout the world, although this may be affected by local geological (tectonic) phenomena.

(e) The annual temperature changes given reflect the annual average condition of the earth as a whole, not any specified place. Region specific temperature changes and precipitation are given in table 7.1.

TABLE 7.1
REGIONAL SCENARIOS FOR CLIMATE CHANGE

Region (a)	Temperature change (b) (as a multiple of global average) Summer	Winter	Precipitation Change (c)
High latitudes (60–90 deg)	0.5x to 0.7x	2.0x to 2.4x	Enhanced in winter
Mid latitudes (30–60 deg)	0.8x to 1.0x	1.2x to 1.4x	Possibly reduced in summer
Low latitudes (0–30 deg)	0.9x to 0.7x	0.9x to 0.7x	Enhanced in places with heavy rainfall today

a) The figures in this table are taken from computer modelling results for the Northern Hemisphere. Low latitude trends are likely to be similar for the Southern Hemisphere. The mid latitudes of the Southern Hemisphere may differ somewhat due to their relatively small landmass. The high latitudes of the Southern Hemisphere could respond much differently from Arctic areas because of the high altitude and land-based nature of the Antarctic.

b) The regional temperature changes are presented here as multiples of the globally and annually averaged temperature changes shown in Figure 7.1. Thus, for example, if from Figure 7.1 one supposes that the globally averaged annual temperature change for the year 2040 could be 2.0 degrees C, then Table 7.1 says that high latitude winter temperatures in the same year might have risen by 4.0 to 4.8 degrees. Similarly, a rate of change in the global mean temperature of 0.3 degrees per decade from Figure 7.1 would translate via Table 7.1 to mean a rate of change in the high latitude winter temperatures of 0.6 to 0.7 degrees per decade.

The particular temperature figures cited in this table are taken from two general circulation models of the earth's atmosphere, selected to reflect the range of results being obtained from today's most advanced scientific studies. In particular, for each range of temperature multiples given in the table, the first value is drawn from the models of James Hansen and his colleagues at NASA. The second value in each range comes from the work of S. Manabe and his colleagues at the US National Oceanographic and Atmospheric Administration laboratories in Princeton.

c) Precipitation is among the most difficult properties of climate to model or predict. The qualitative assessments quoted here are nonetheless supported by a number of studies. In particular, they draw from a recent review by Mitchell and his colleagues at the British Meteorological Office.

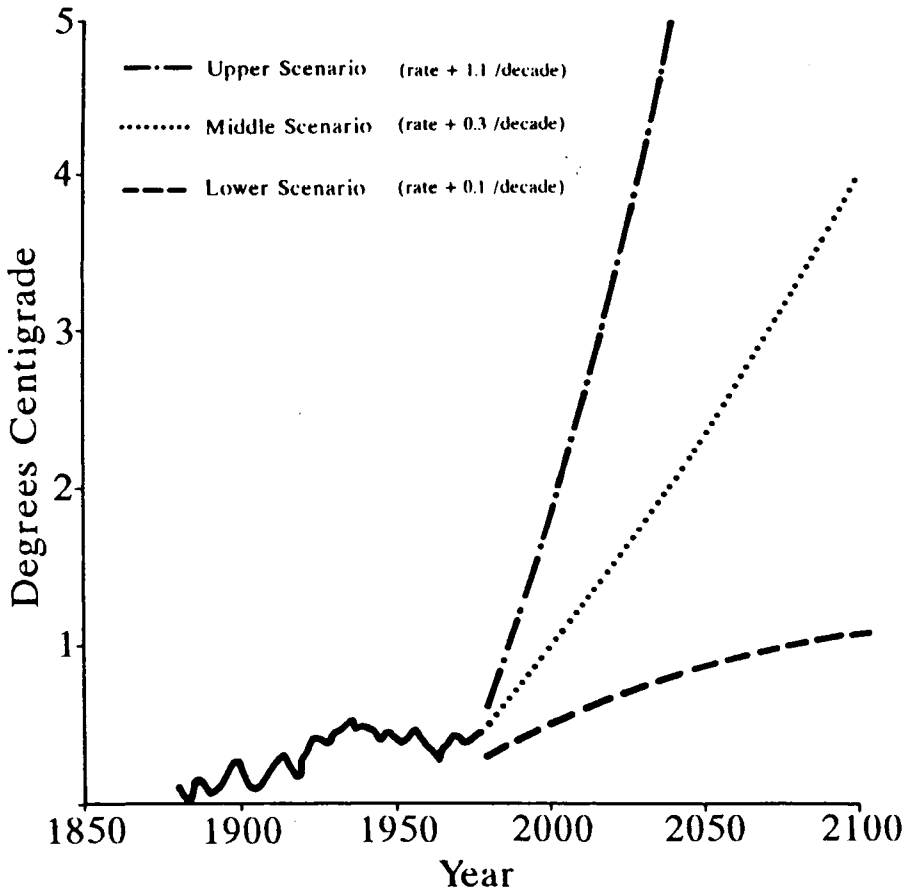


Fig. 7.1(a) THE GLOBAL TEMPERATURE CHANGE

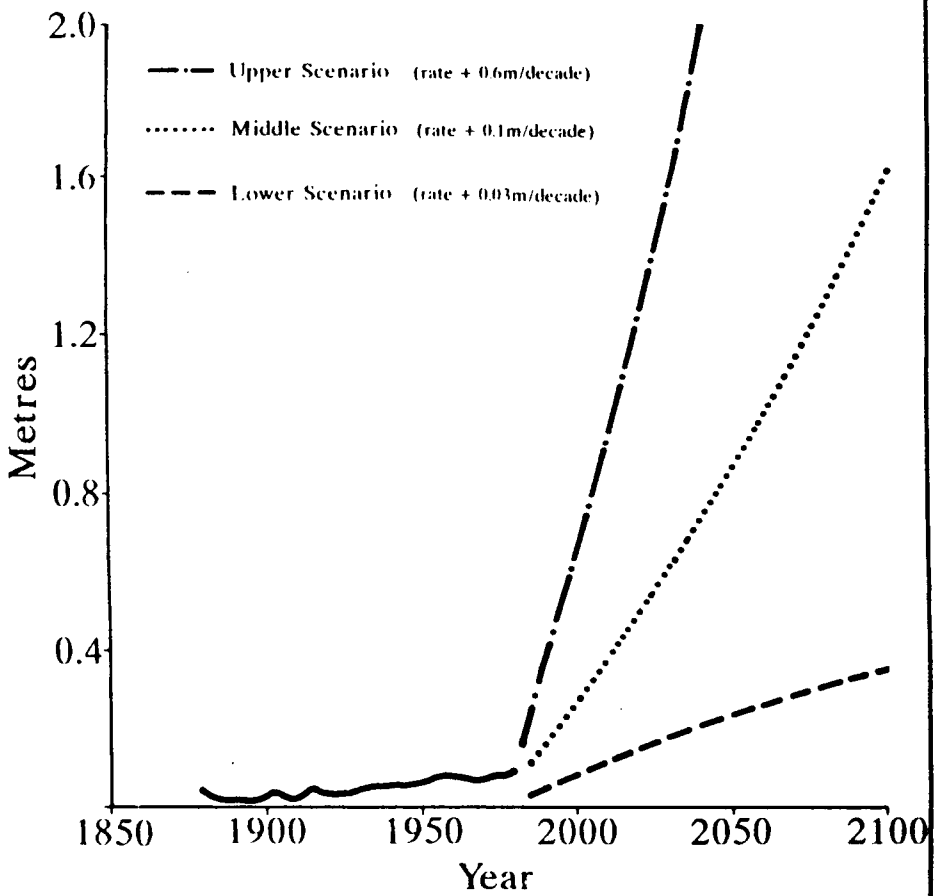


Fig. 7.1(b) SEA LEVEL RISE FROM GLOBAL WARMING

Source: UNEP/WMO/ICSU,1987

Flg 7.1 SCENARIOS OF CLIMATE CHANGE

NOTES The figure shows scenarios of globally averaged climate change that might develop in response to continued emissions of atmospheric GHGs. The middle curve of each panel reflects a scenario of continued present trends of emissions and a moderate climate sensitivity. There is a chance of 5:10 that the actual path of climate change could lie below the middle curve. The upper curve of each panel reflects a scenario of accelerated greenhouse emissions and a relatively high climate sensitivity. There is a chance of 9:10 that the actual path of climate change could lie below the upper curve. The lower curve of each panel reflects a scenario of curtailed GHG emissions and a relatively low climate sensitivity. There is a chance of 1:10 that the actual path of climate change could lie below the lower curve. The ceiling of 5 degrees on the temperature graph has been imposed because of the dubious relevance of present climate models under conditions that would prevail in response to a global warming higher than around 5 degrees.

(f) The middle curves in figure 7.1 for present GHG emissions is calculated with the assumption that the recent international protocol on protecting the ozone layer is successfully implemented, thus reducing the emissions of CFC's significantly below their recent rates of increase.

(g) Table 7.1 indicates that warming in the higher latitudes of the Northern Hemisphere could be two to two and half times greater and faster than the globally averaged annual values. In contrast, temperature changes in the low latitudes will probably be somewhat smaller and slower than the globally averaged values.

(h) There is likely to be enhanced winter snowfall in the high latitudes, intenstified rains in the presently rainy zones of the low latitudes and perhaps a decrease in the summer rainfall in the middle latitudes.

7.3 Quantifying the Future Global Climate Conditions

The figure 7.2 shows the worlds current precipitation patterns and isohyets. Figure 7.3 shows the climatic divisions of the world.The figures 7.4 (a)-(d) illustrates the projected precipitation patterns during the early part of the 21st century A.D.for the mildest, mean, and worst scenarios.In these figures the changes in precipitations envisagedin the humid tropics are obtained from the UNEP et al (1987) . The magnitude of possible changes in the other regions, the middle and high latitudes have been assumed to be similar to precipitation changes predicted for the U.K (+-20%) obtained from the U.K Department of the Environment

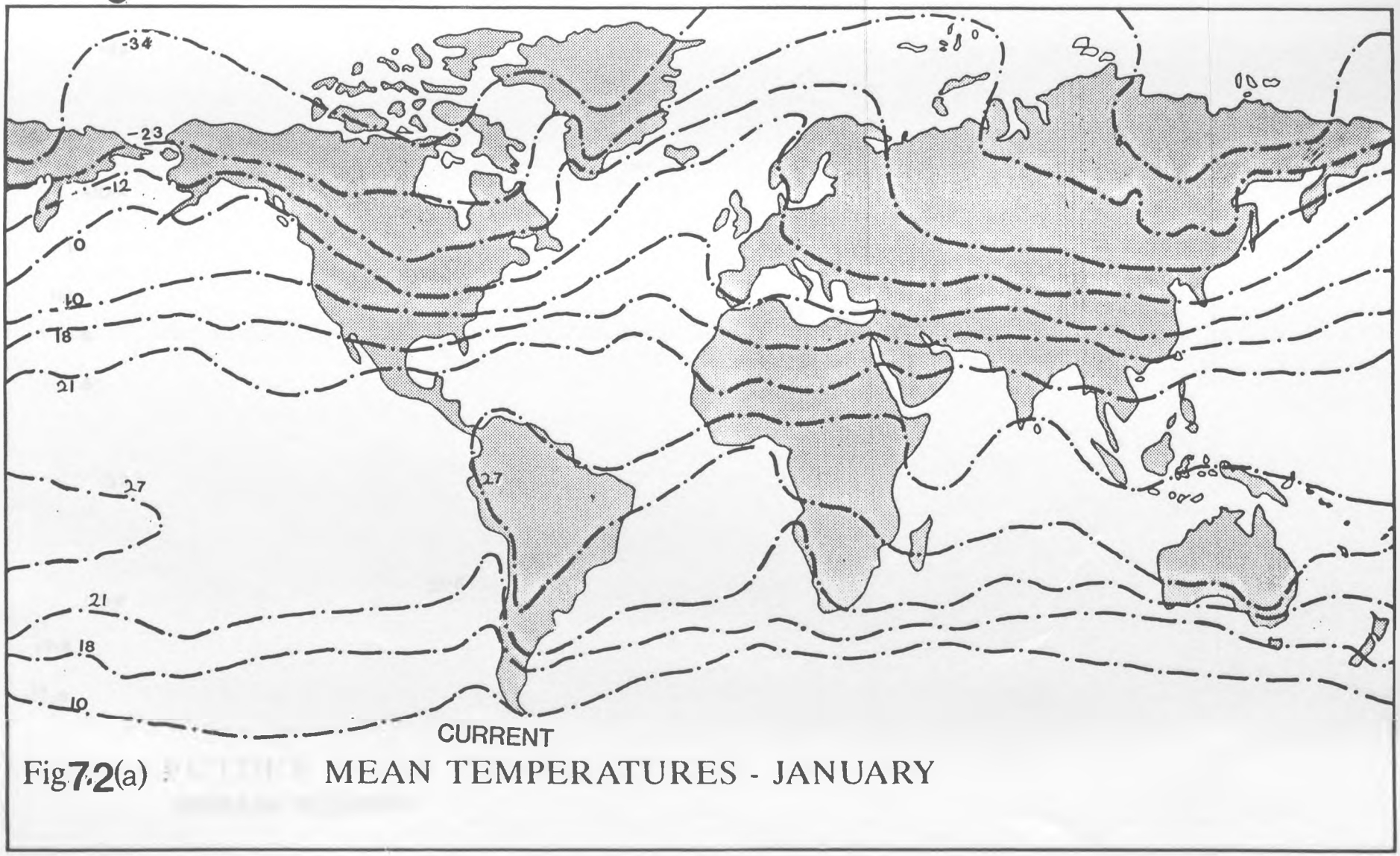


Fig 7.2(a) MEAN TEMPERATURES - JANUARY

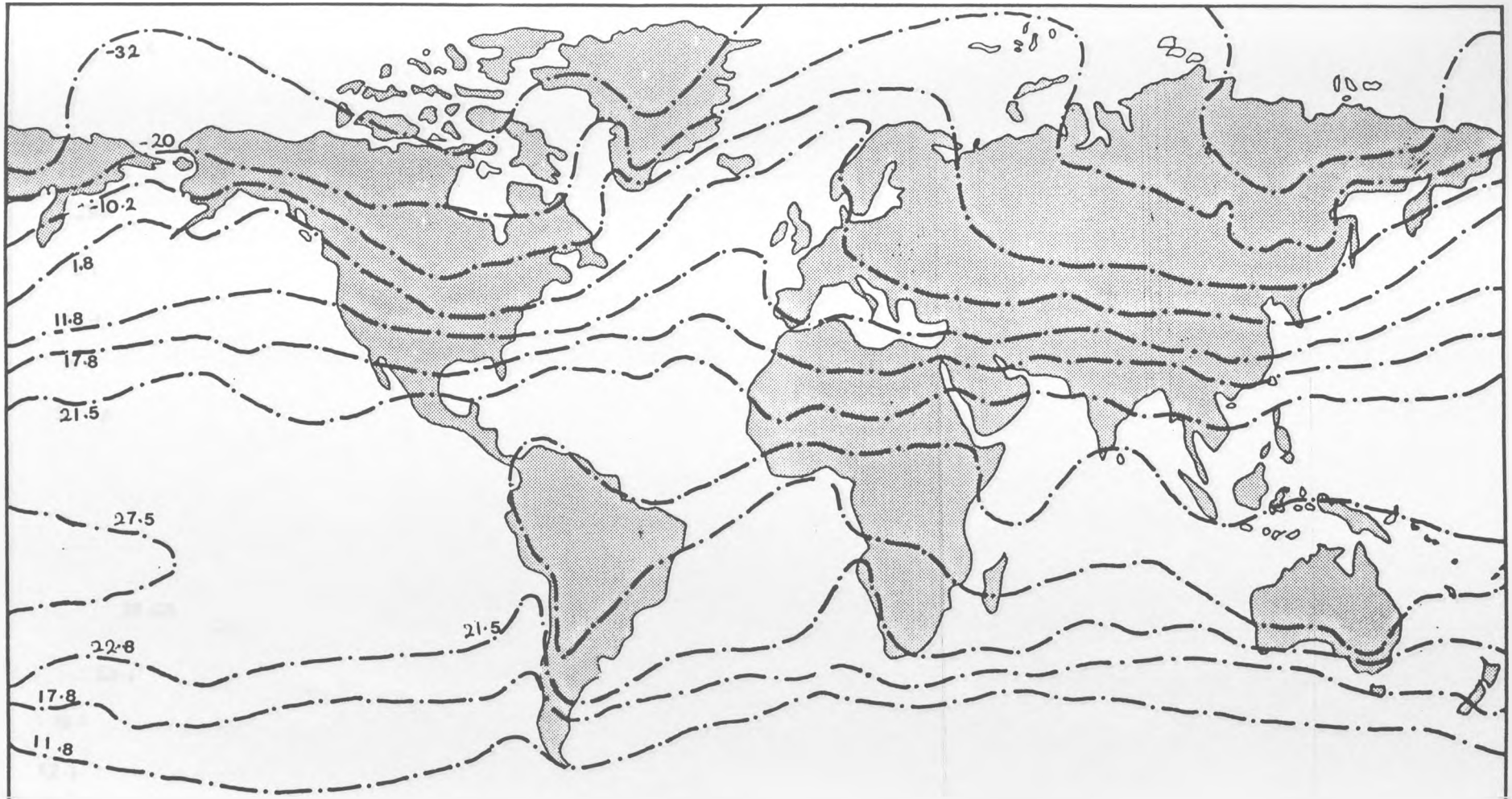


Fig. 72(b) FUTURE MEAN TEMPERATURES - JANUARY
MINIMUM SCENARIO

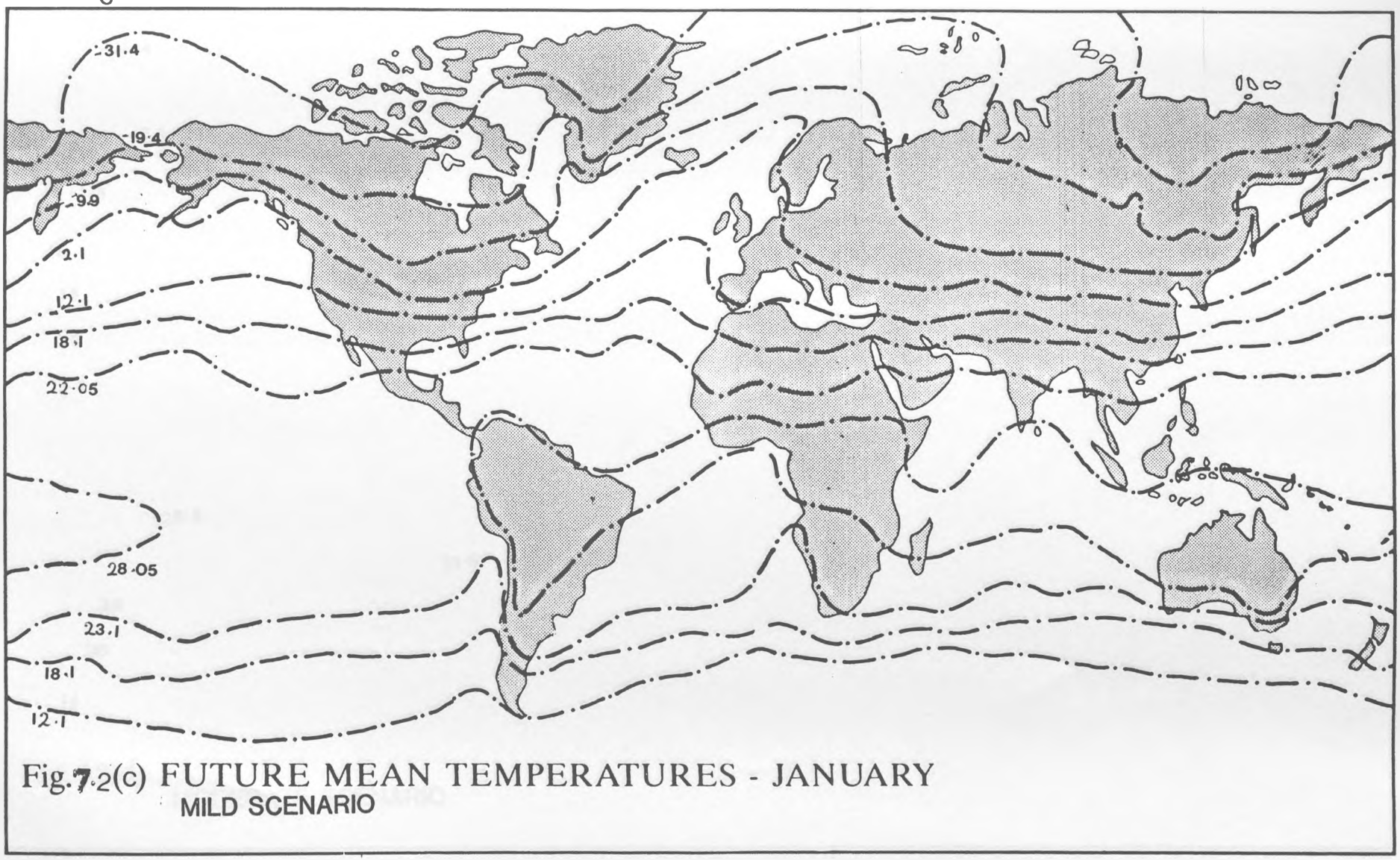


Fig.7.2(c) FUTURE MEAN TEMPERATURES - JANUARY
MILD SCENARIO

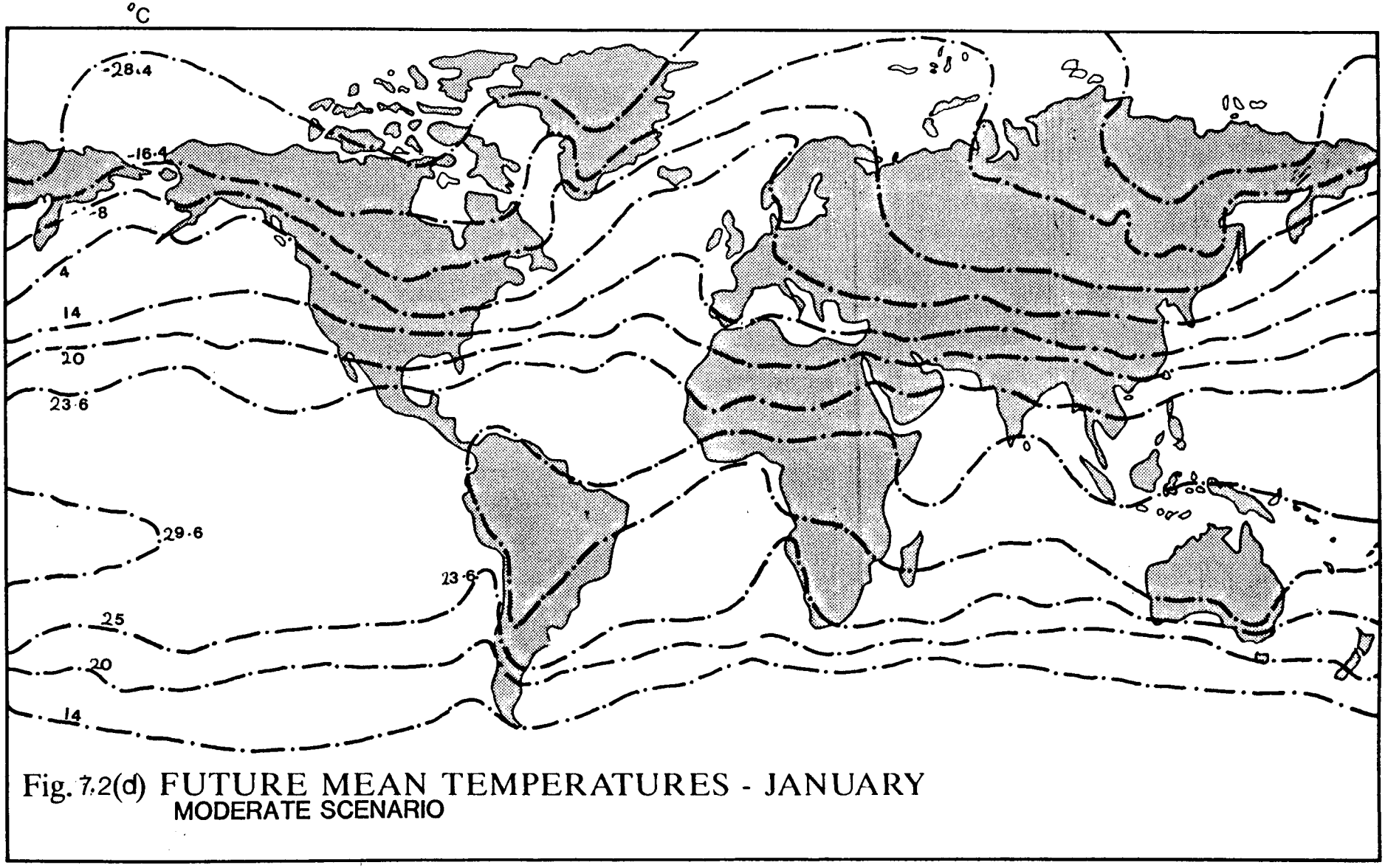


Fig. 7.2(d) FUTURE MEAN TEMPERATURES - JANUARY
MODERATE SCENARIO

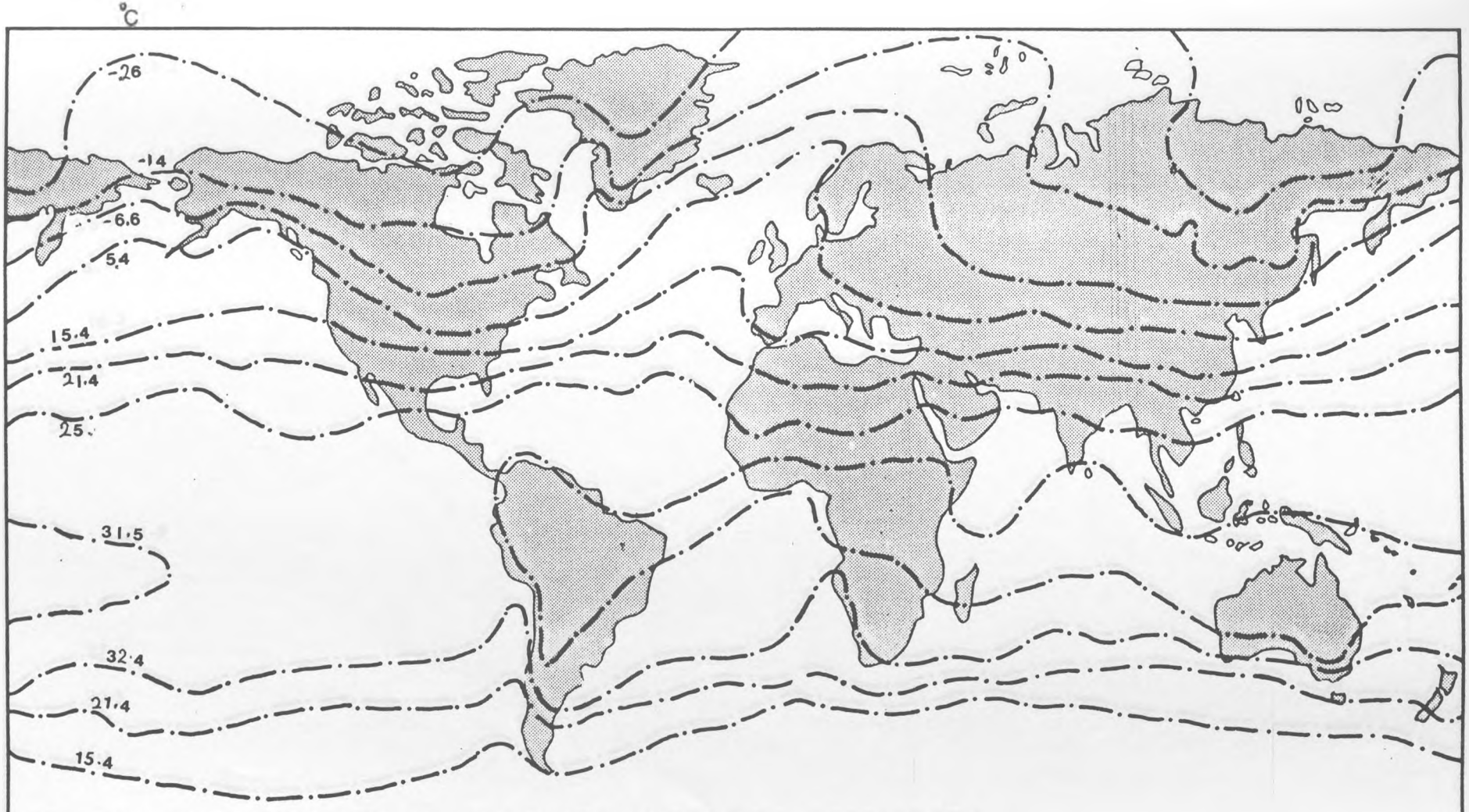


Fig. 7.2(e) FUTURE MEAN TEMPERATURES - JANUARY
SEVERE SCENARIO

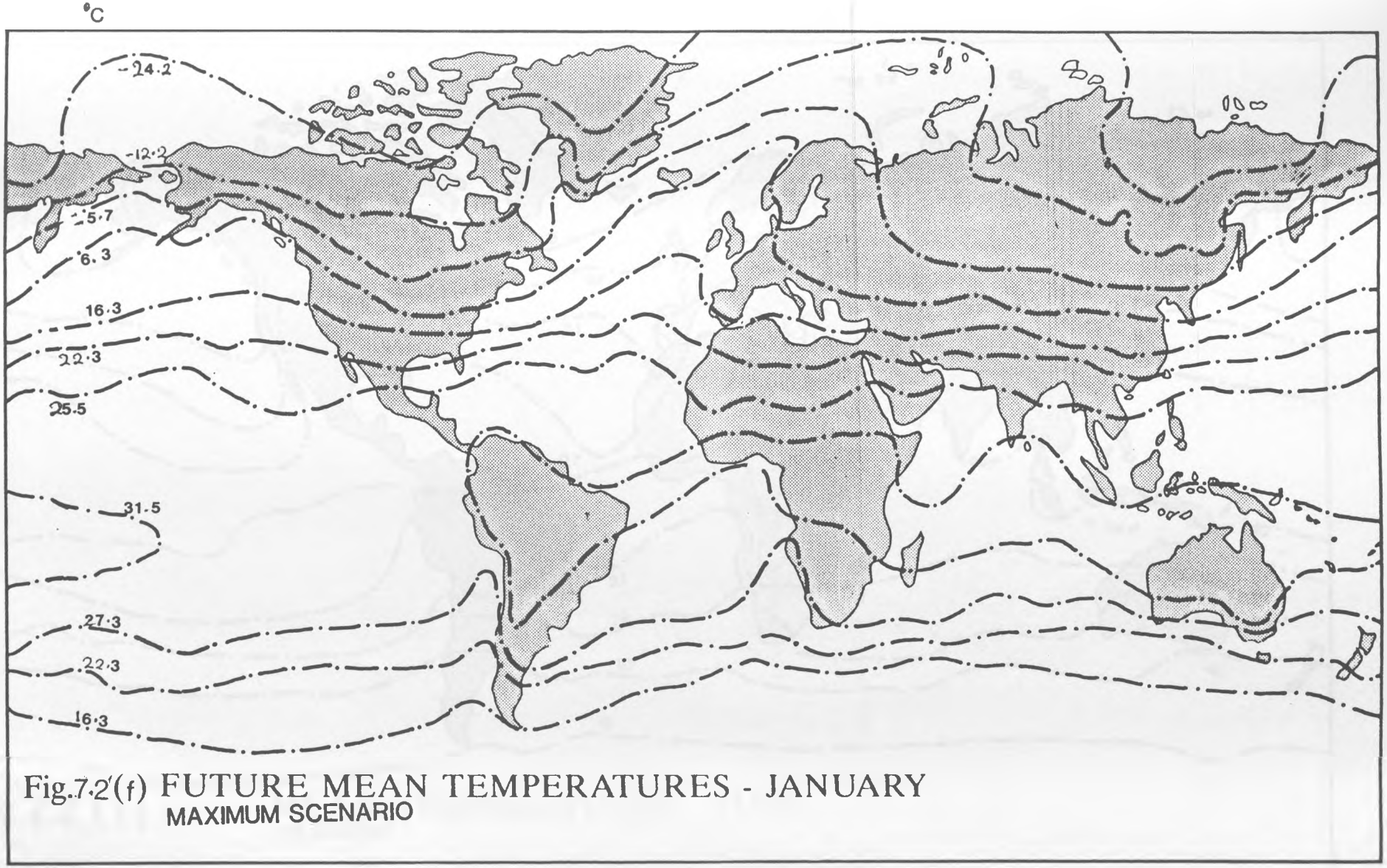


Fig.7.2(f) FUTURE MEAN TEMPERATURES - JANUARY
MAXIMUM SCENARIO

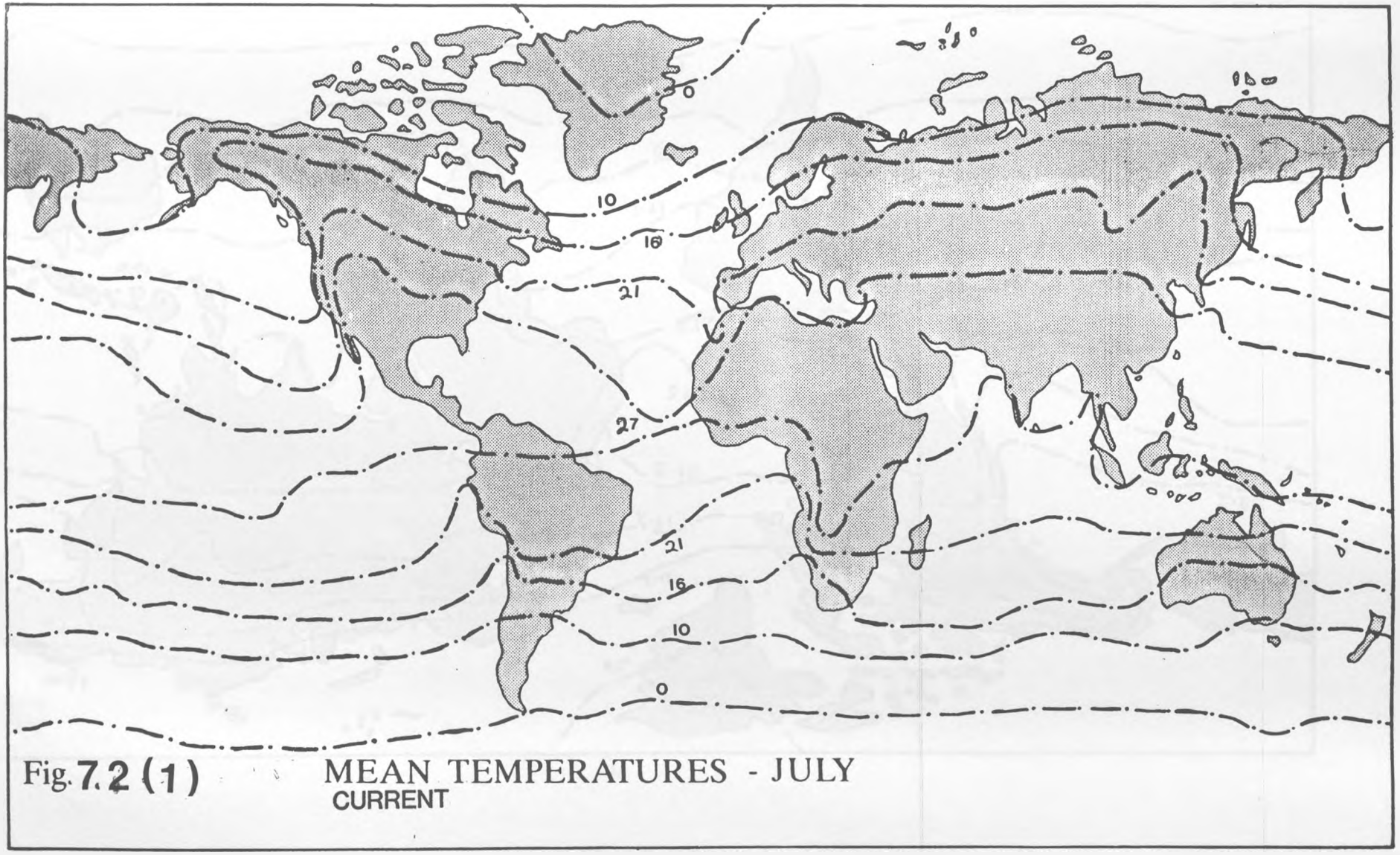


Fig. 7.2 (1) MEAN TEMPERATURES - JULY
CURRENT

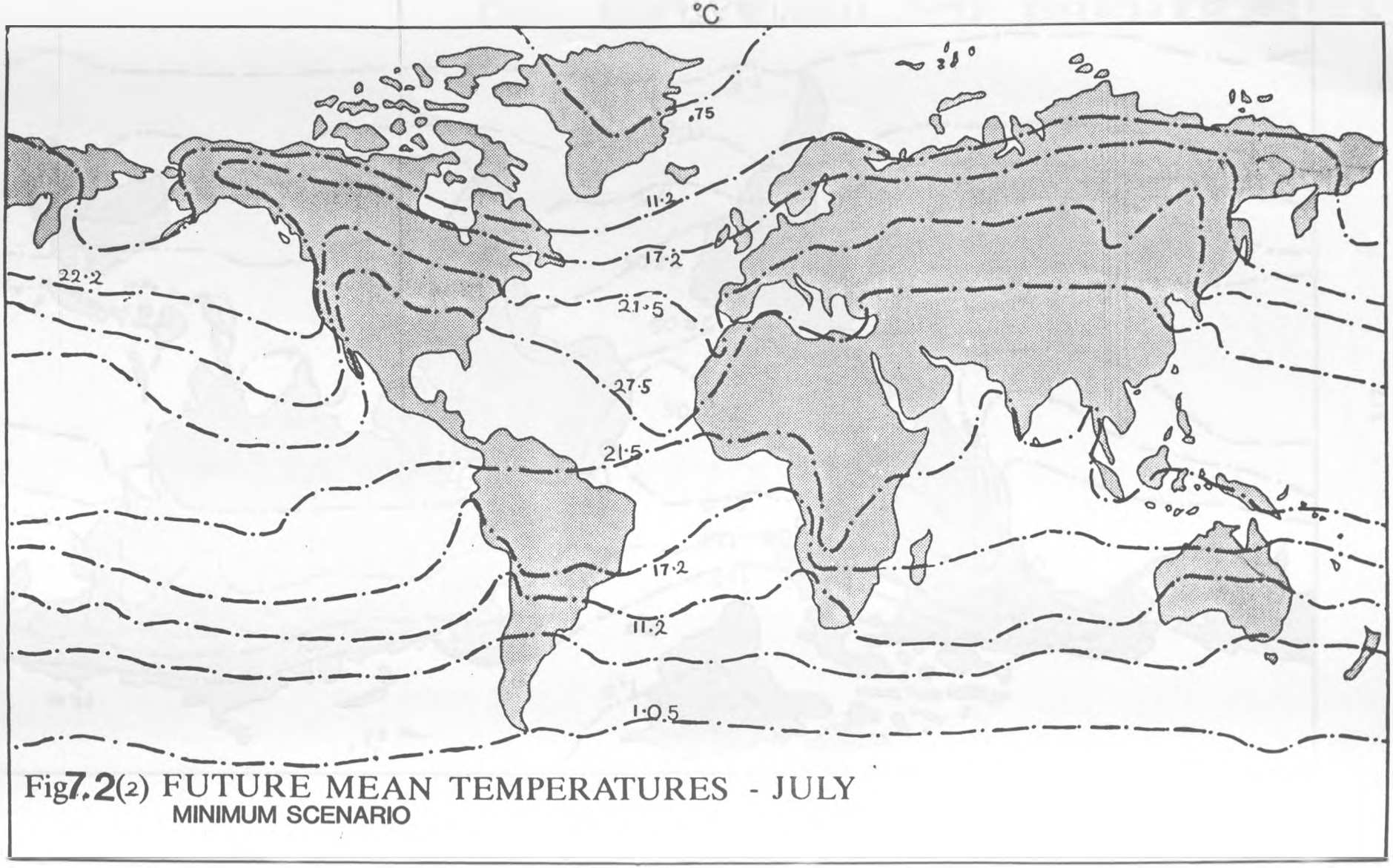


Fig 7.2(2) FUTURE MEAN TEMPERATURES - JULY
MINIMUM SCENARIO

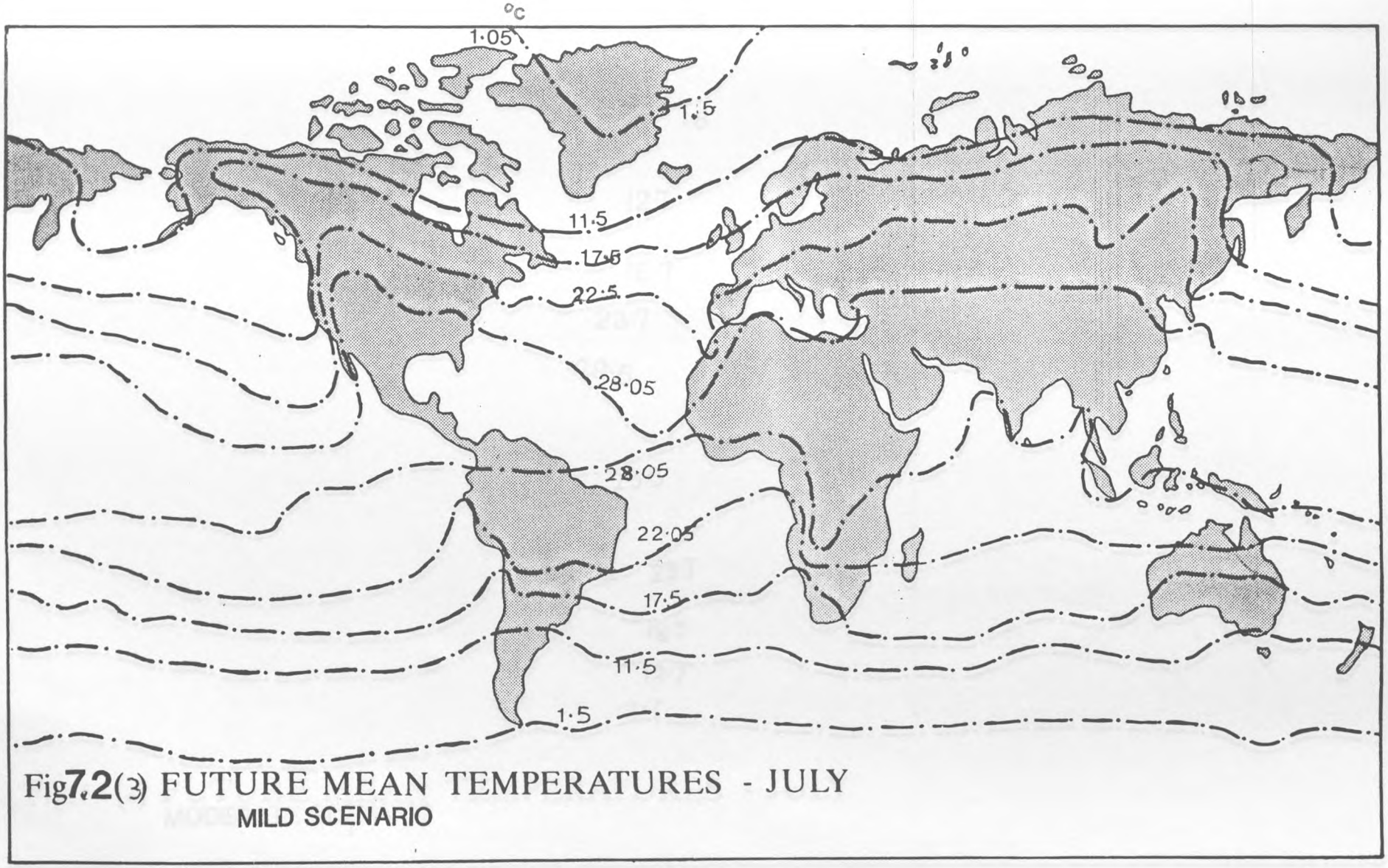


Fig 7.2(3) FUTURE MEAN TEMPERATURES - JULY
MILD SCENARIO

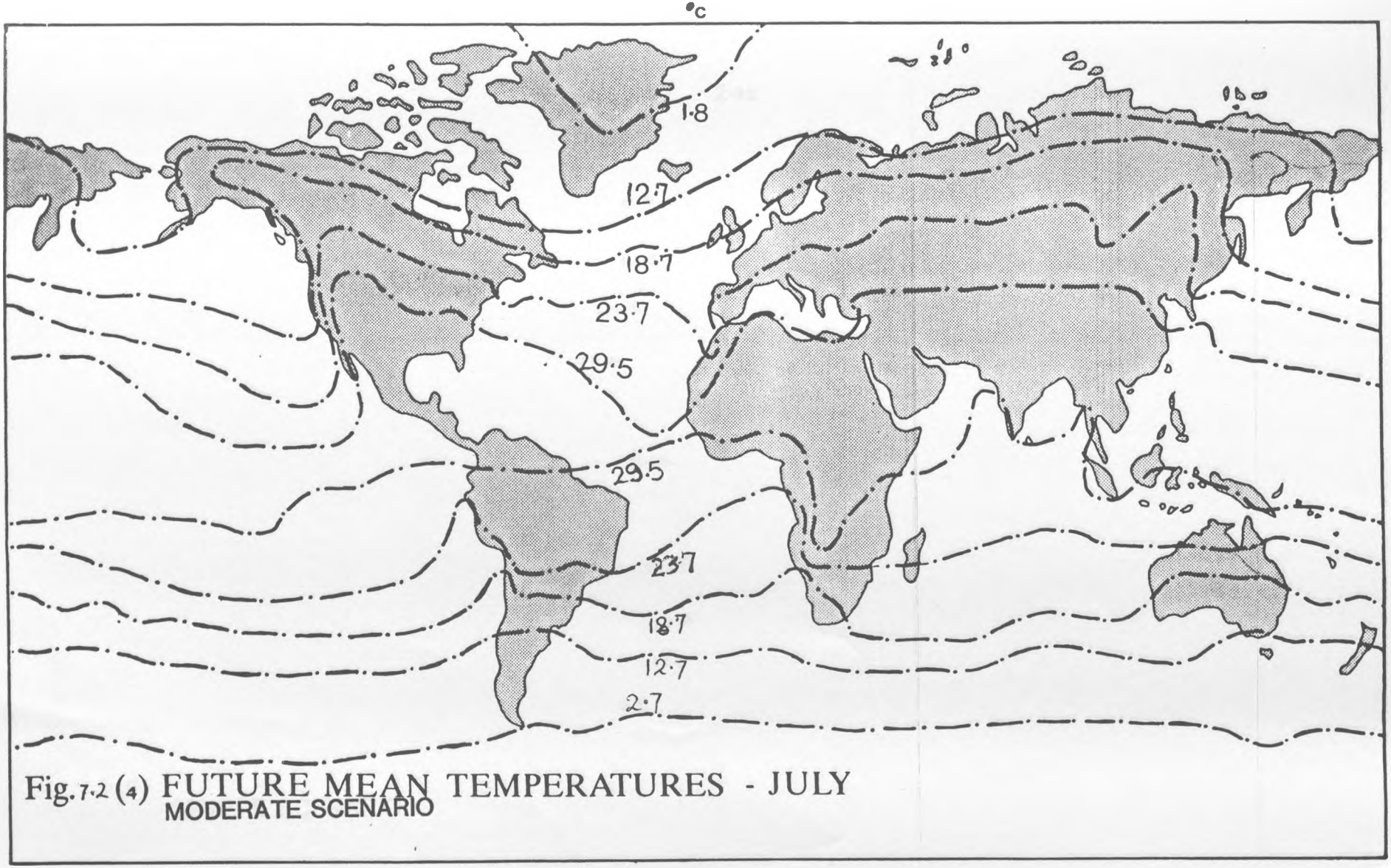


Fig. 7·2 (4) FUTURE MEAN TEMPERATURES - JULY
MODERATE SCENARIO

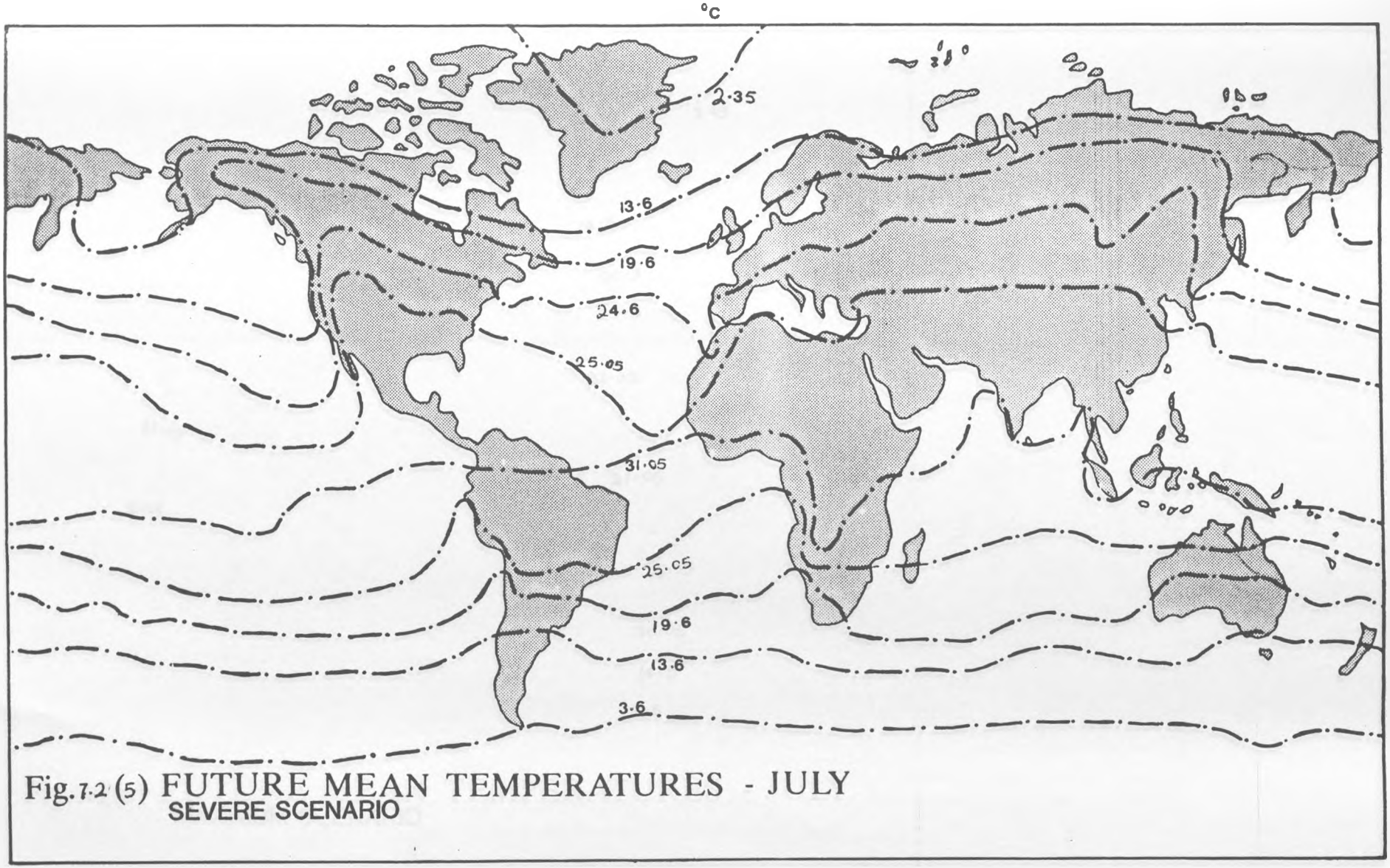


Fig. 7.2 (5) FUTURE MEAN TEMPERATURES - JULY
SEVERE SCENARIO

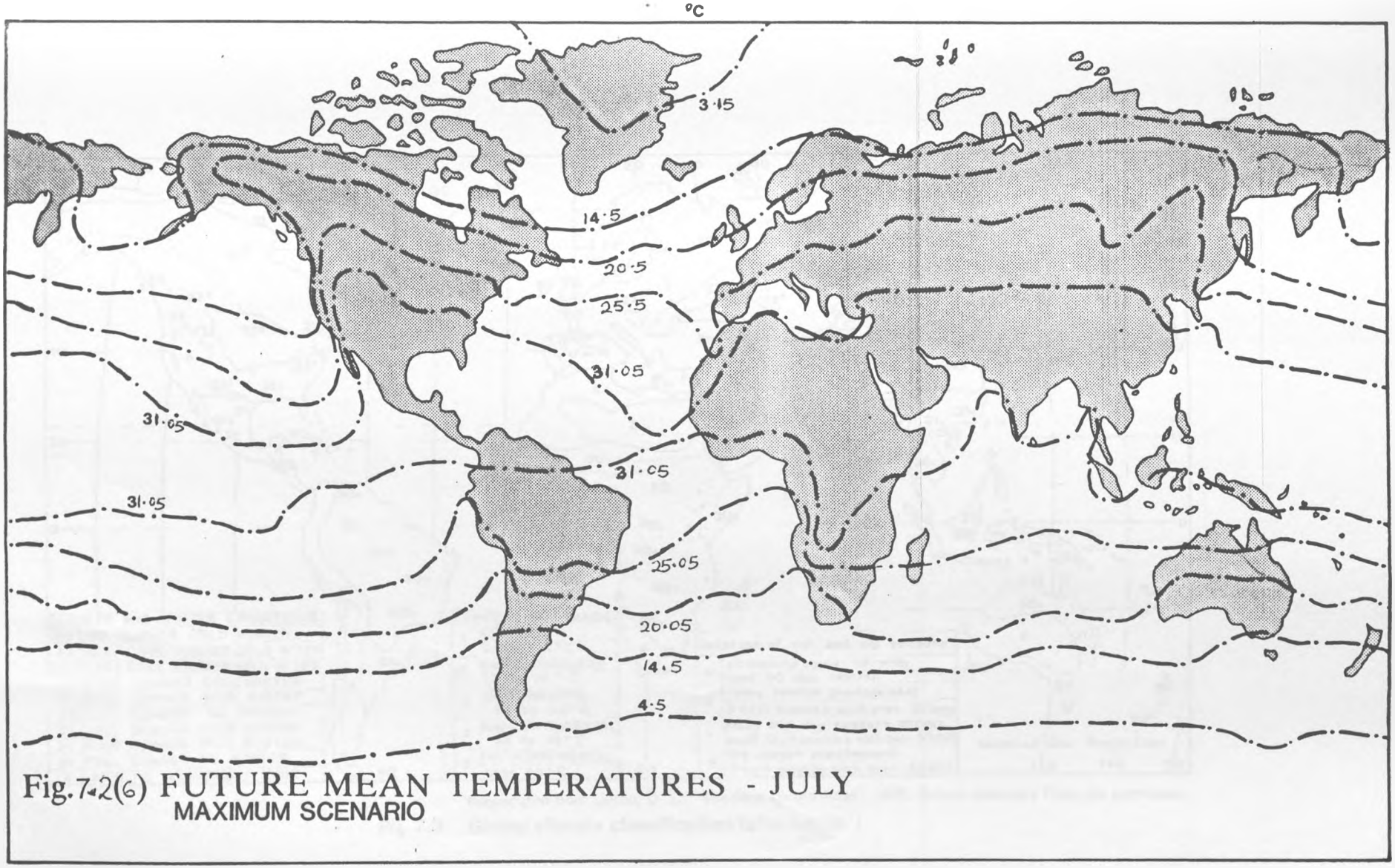
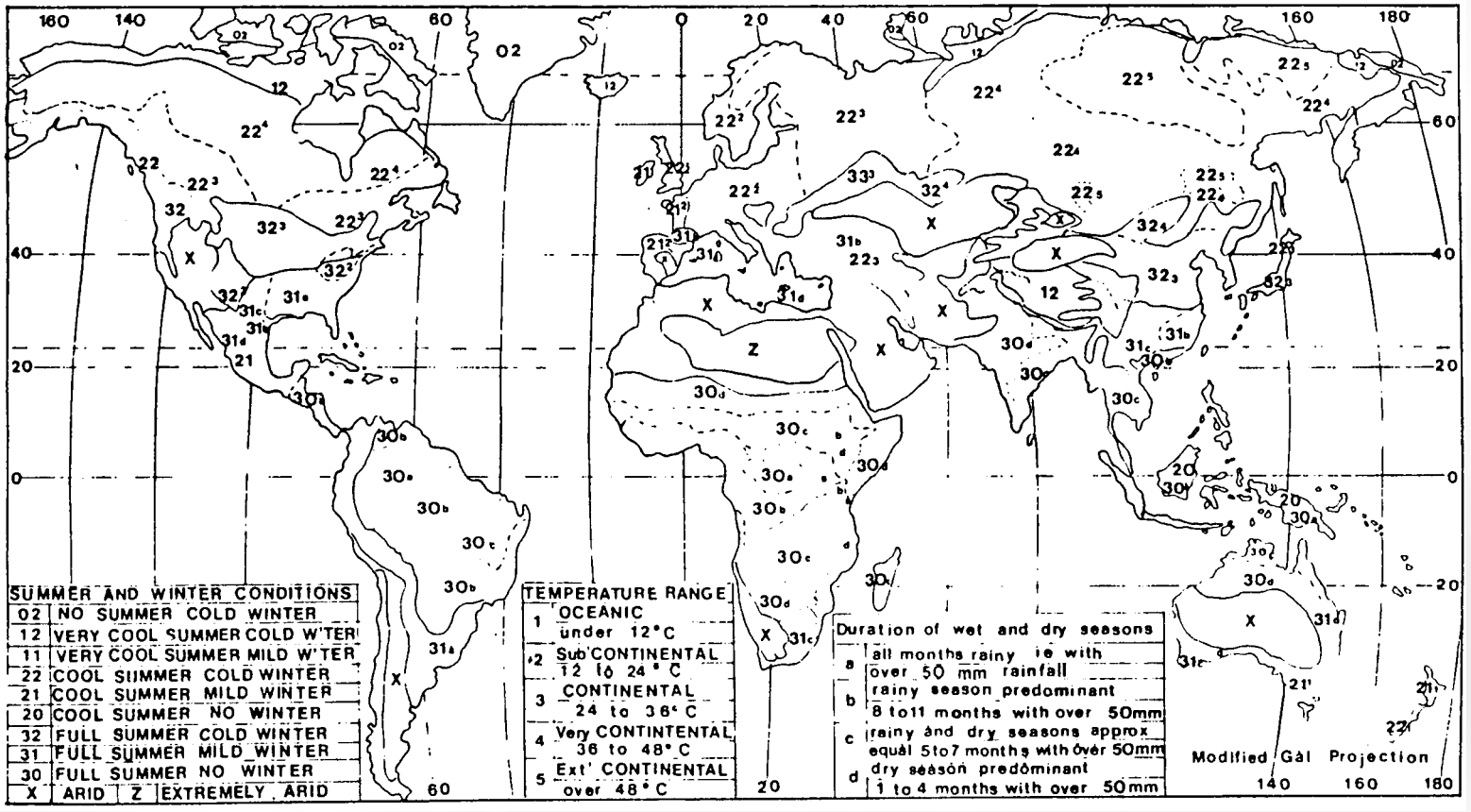
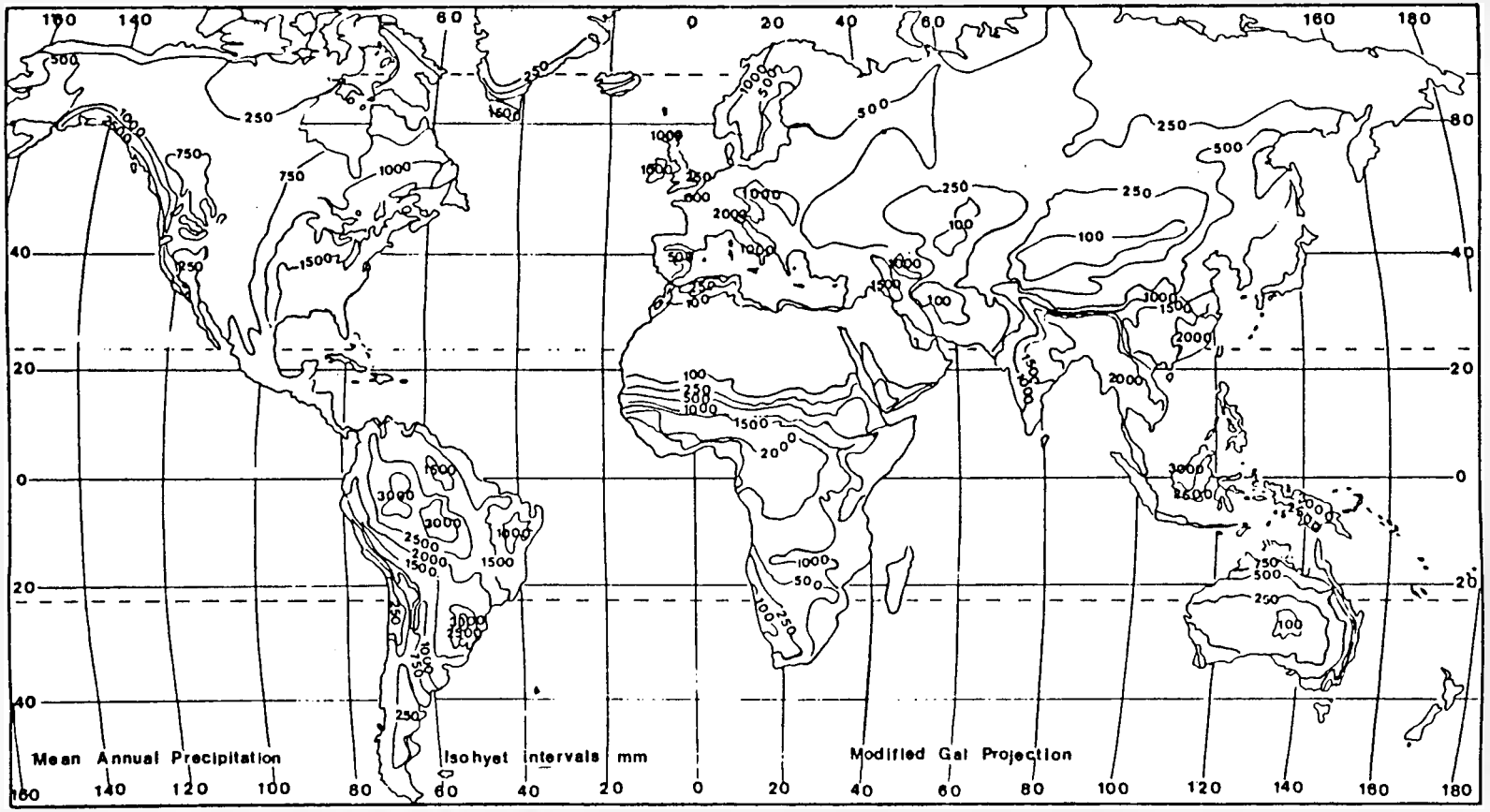


Fig.7.2(6) FUTURE MEAN TEMPERATURES - JULY
MAXIMUM SCENARIO



Reproduced from Linton, D. L., "The New Oxford Atlas", 1975, Oxford University Press, by permission.

Fig 7.3 **Global climate classification** (after Linton[1])



Reproduced from Linton, D. L., "The New Oxford Atlas", 1975, Oxford University Press, by permission.

Fig 7.4 Global mean annual precipitation (after Linton[1])

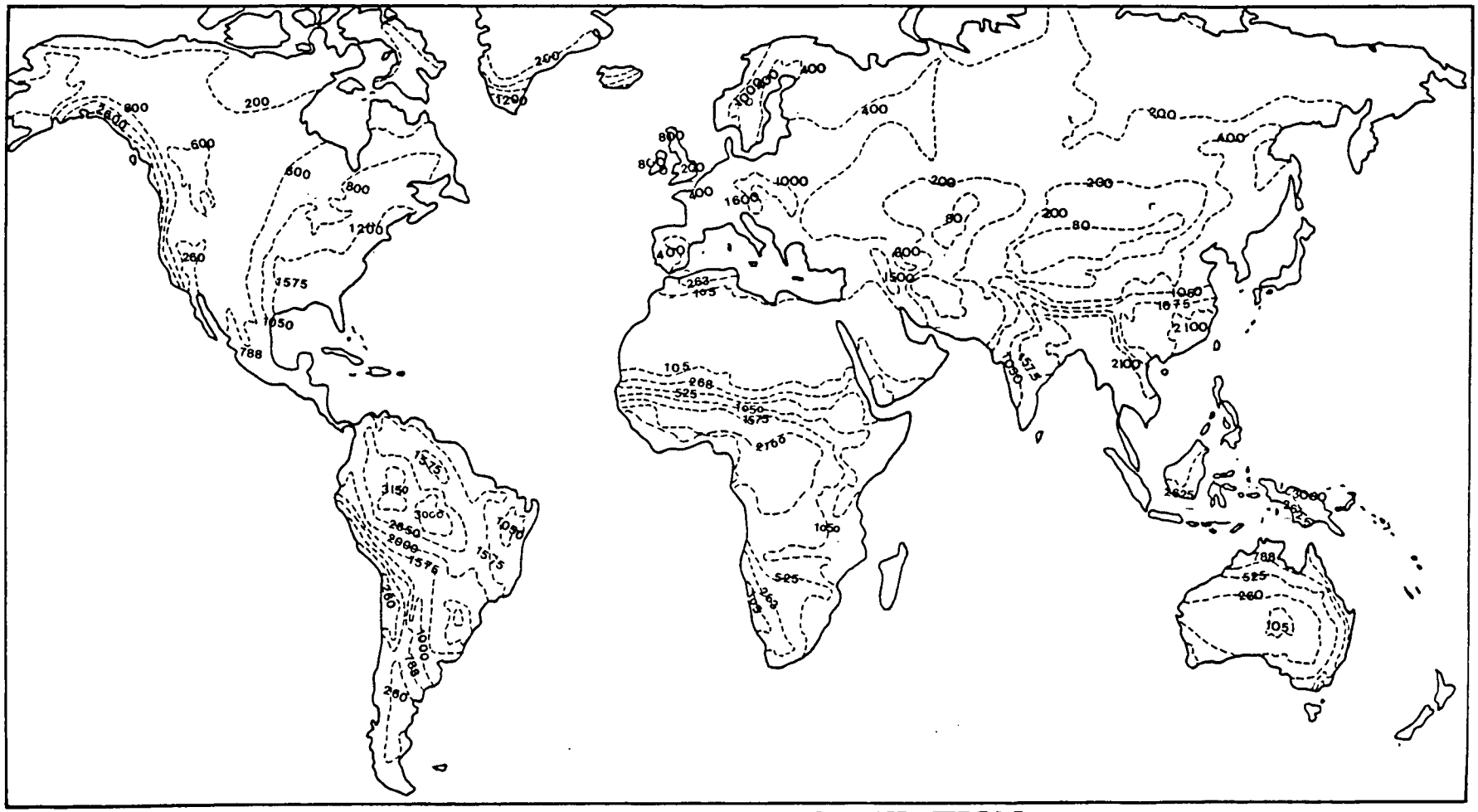


FIG. 7.4(A) GLOBAL MEAN ANNUAL PRECIPITATION

MILD MINIMUM SCENRIO
5% INCREASE-TROPICS - 20% DECREASE -HIGH/MID-LATITUDES

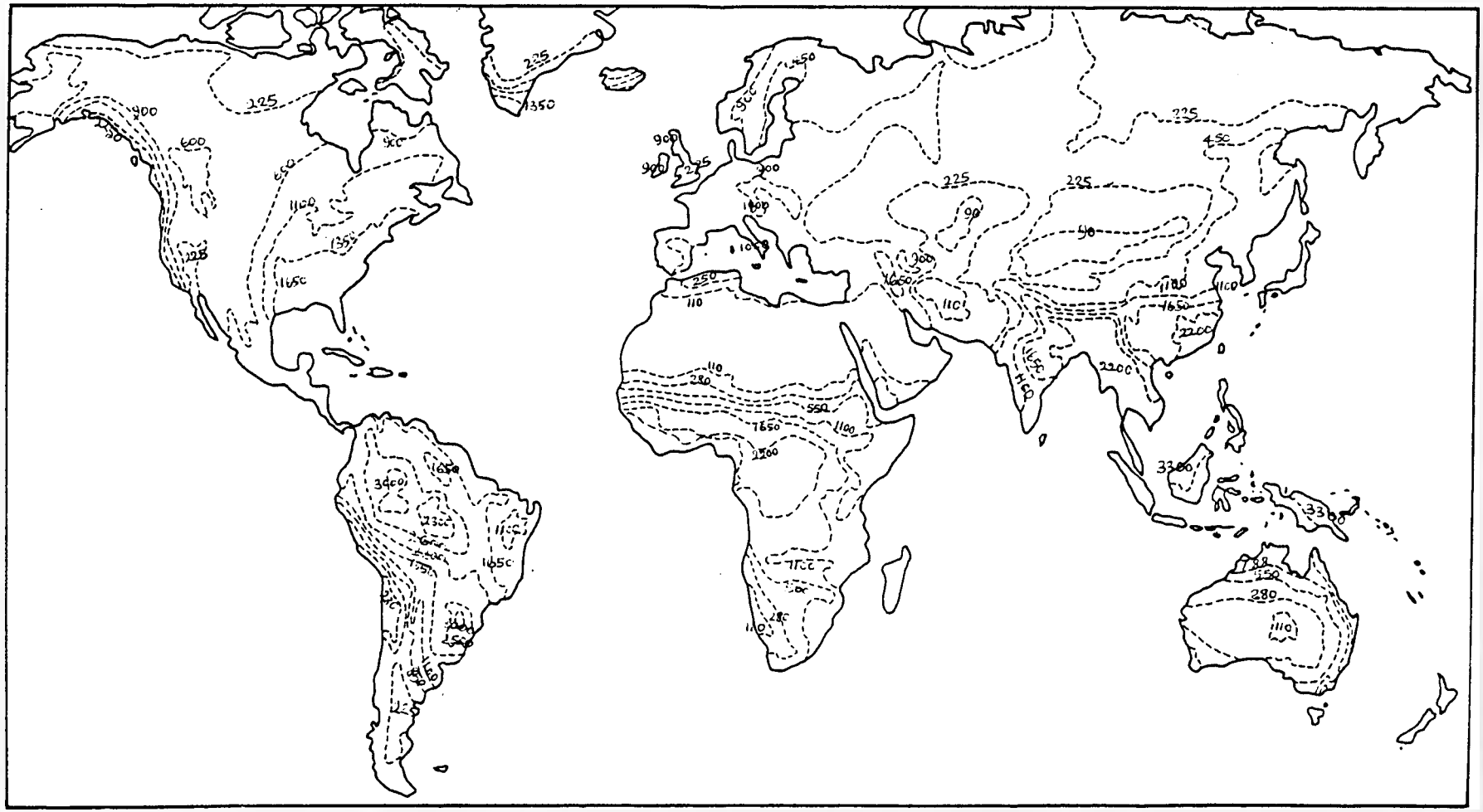


FIG. 7.4(B) GLOBAL MEAN ANNUAL PRECIPITATION

MODERATE SCENARIO
10% INC-TROPICS 10% DEC-HIGH/MID LAT

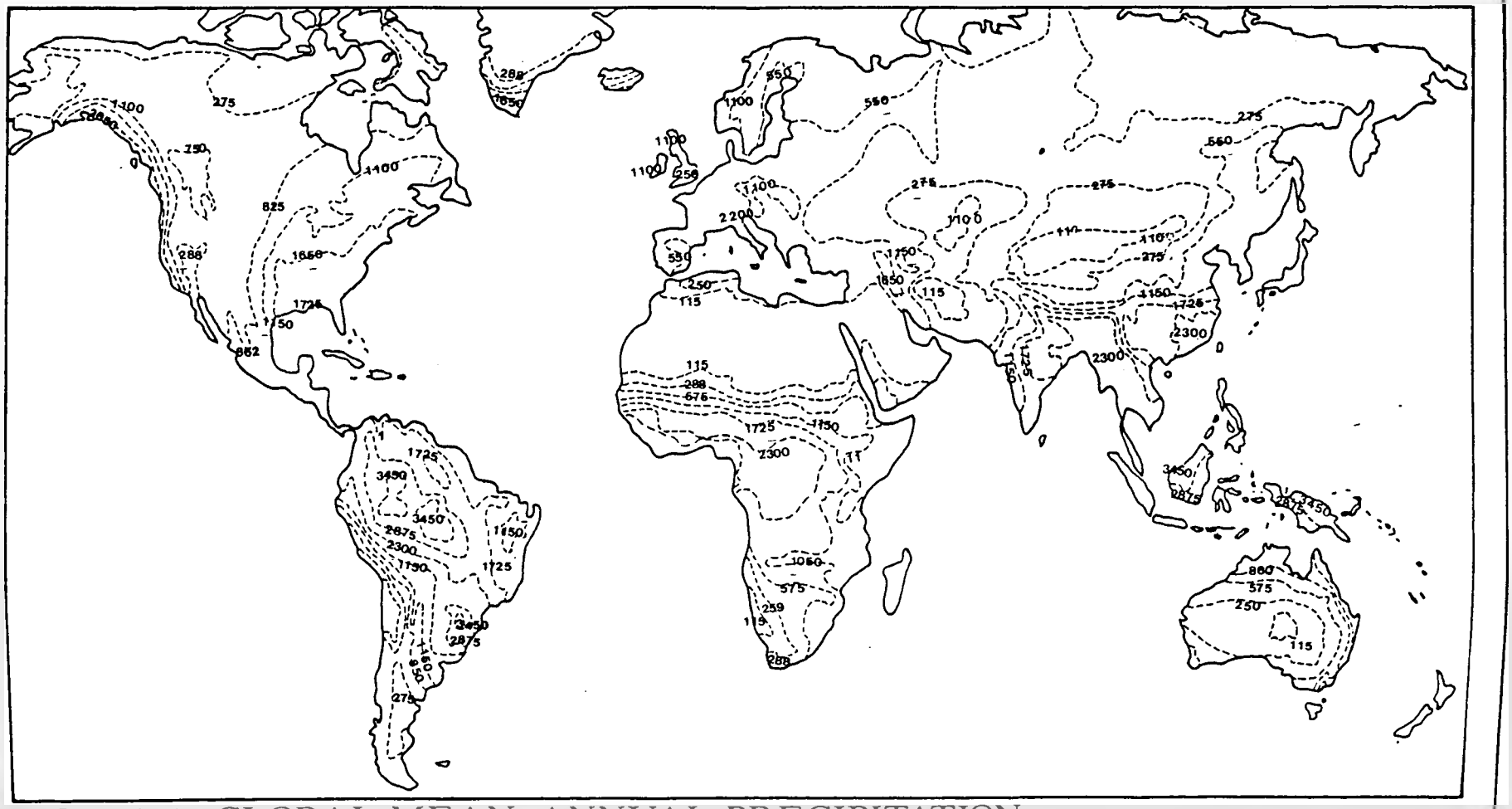


FIG. 7.4(c) GLOBAL MEAN ANNUAL PRECIPITATION FUTURE CONDITIONS

SEVERE SCENARIO OF GLOBAL WARMING

INCREASE : 15% TROPICS 10% HIGH MID-LATTITUDES

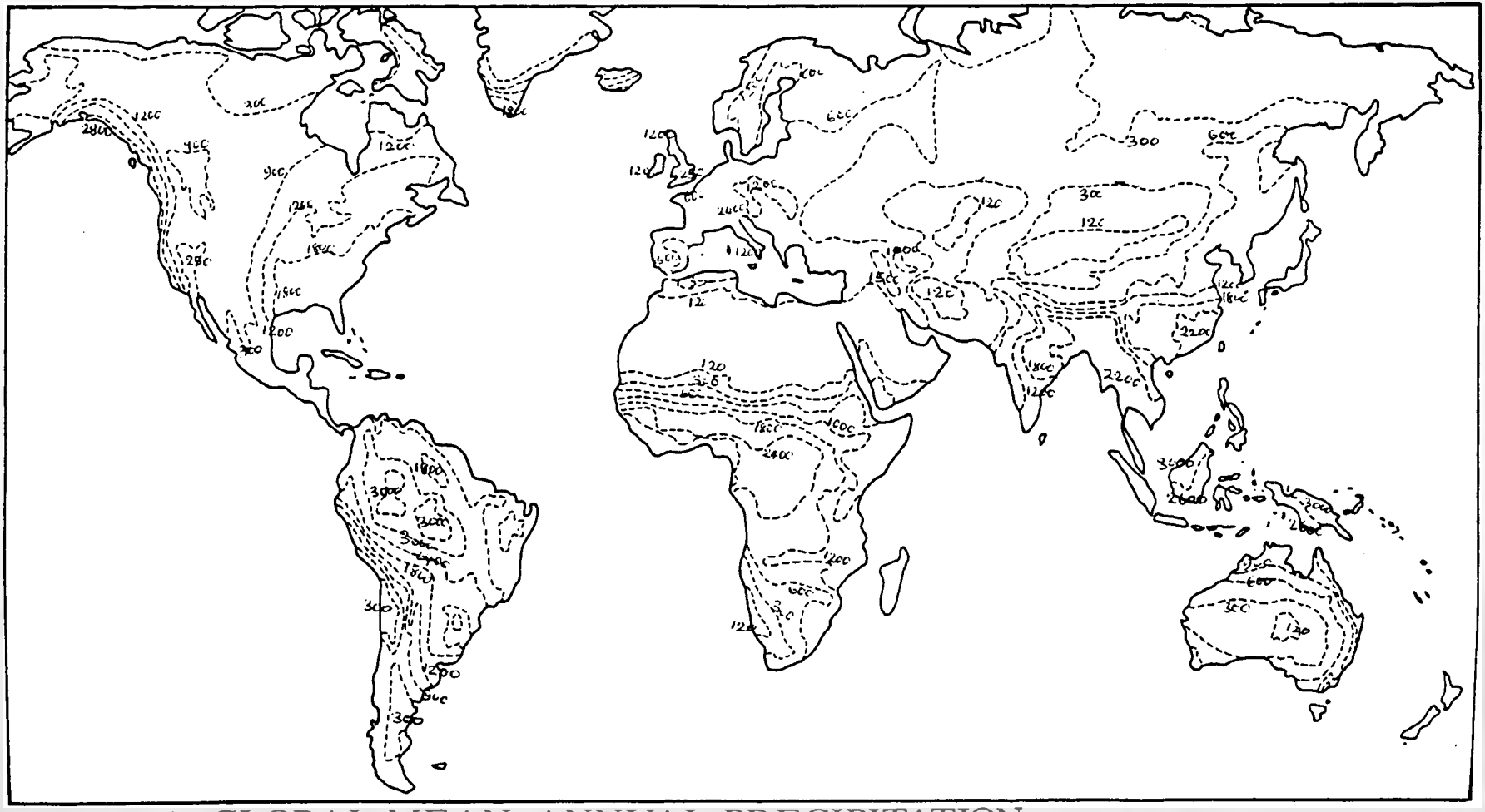


FIG. 7.4(D) GLOBAL MEAN ANNUAL PRECIPITATION

MAXIMUM SCENARIO
20% INCREASE

report (D.O.E, 1988).

The table 7.2 below provides the magnitudes in degrees centigrade by regions of expected changes in temperatures Worldwide, as worked out from table 7.1 .

TABLE 7.2 RANGE OF EXPECTED TEMPERATURE CHANGES BY REGIONS degree C.

REGION/ TEMPERATURE RANGE	SUMMER MONTHS		WINTER MONTHS TEMPERATURE RANGE	
	MIN	MAX	MIN	MAX
HIGH LATITUDES	0.75-1.05	2.35-3.15	3.00-3.60	9.00-10.80
MID LATITUDES	1.20-1.50	3.60-4.50	1.80-2.10	5.40- 6.30
TROPICS/LOW LAT	0.50-1.05	4.05-4.05	0.50-1.05	4.50- 4.50

The above temperature changes have been used by the author to project the spatial changes in mean annual potential evapotranspiration around the world using the Thornthwaite technique.Figures 7.7 (a)-(c) illustrates the projected potential evapotranspiration patterns expected during the early half of the 21st century A.D. for the mildest, mean and worst scenarios respectively.

The projected potential evapotranspiration values and patterns of figures 7.7 (a)-(e) were obtained from the percent changes by region given in table 7.3. These percentages were obtained from plots of mean annual temperature change and their corresponding potential evapotranspiration changes given in figures 7.5 and 7.6 . Figure 7.6 is a modification of figure 7.5. This modification involves the use of pan measured or Penman calculated evapotranspiration values as initial/current

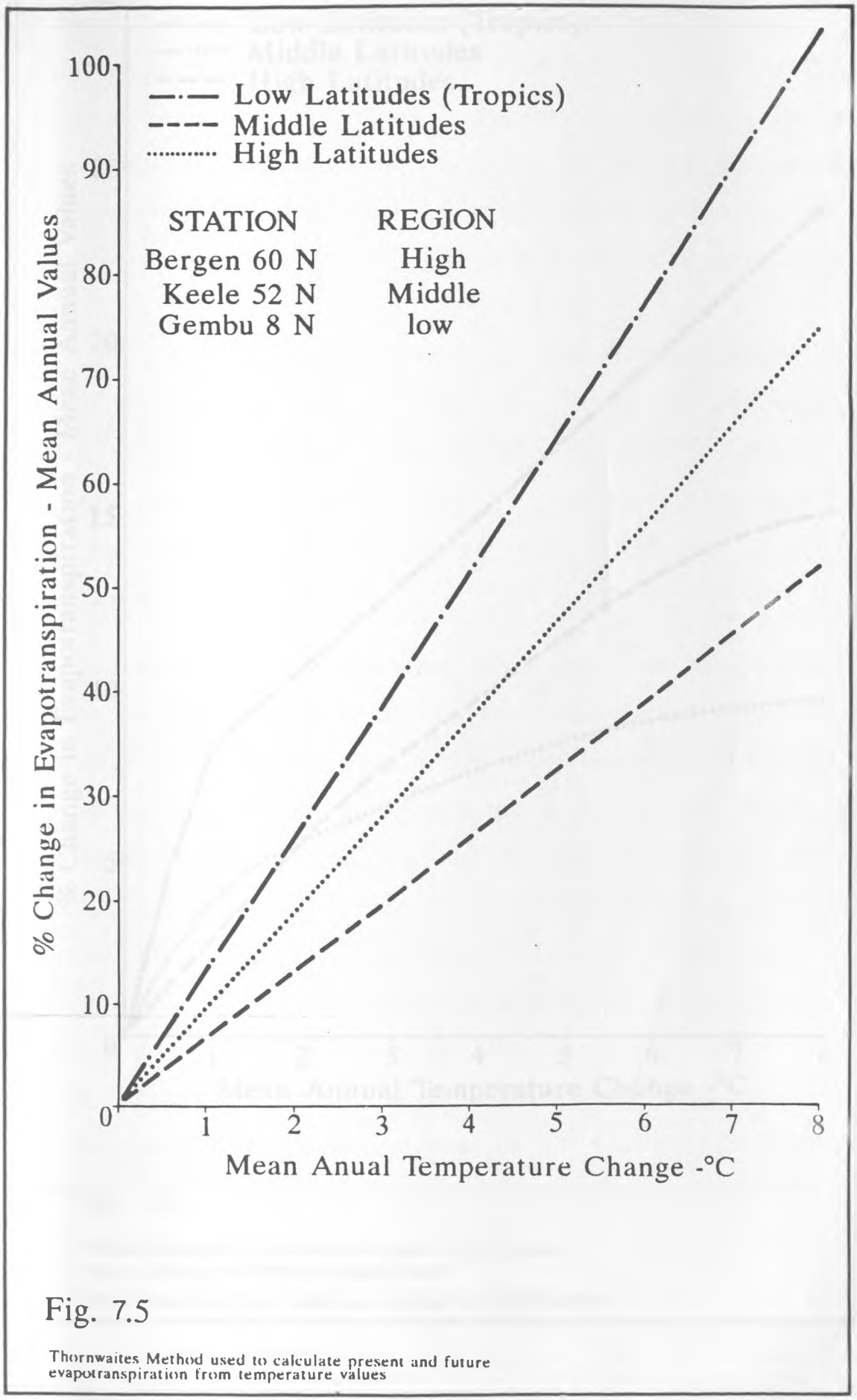


Fig. 7.5

Thornwaites Method used to calculate present and future evapotranspiration from temperature values

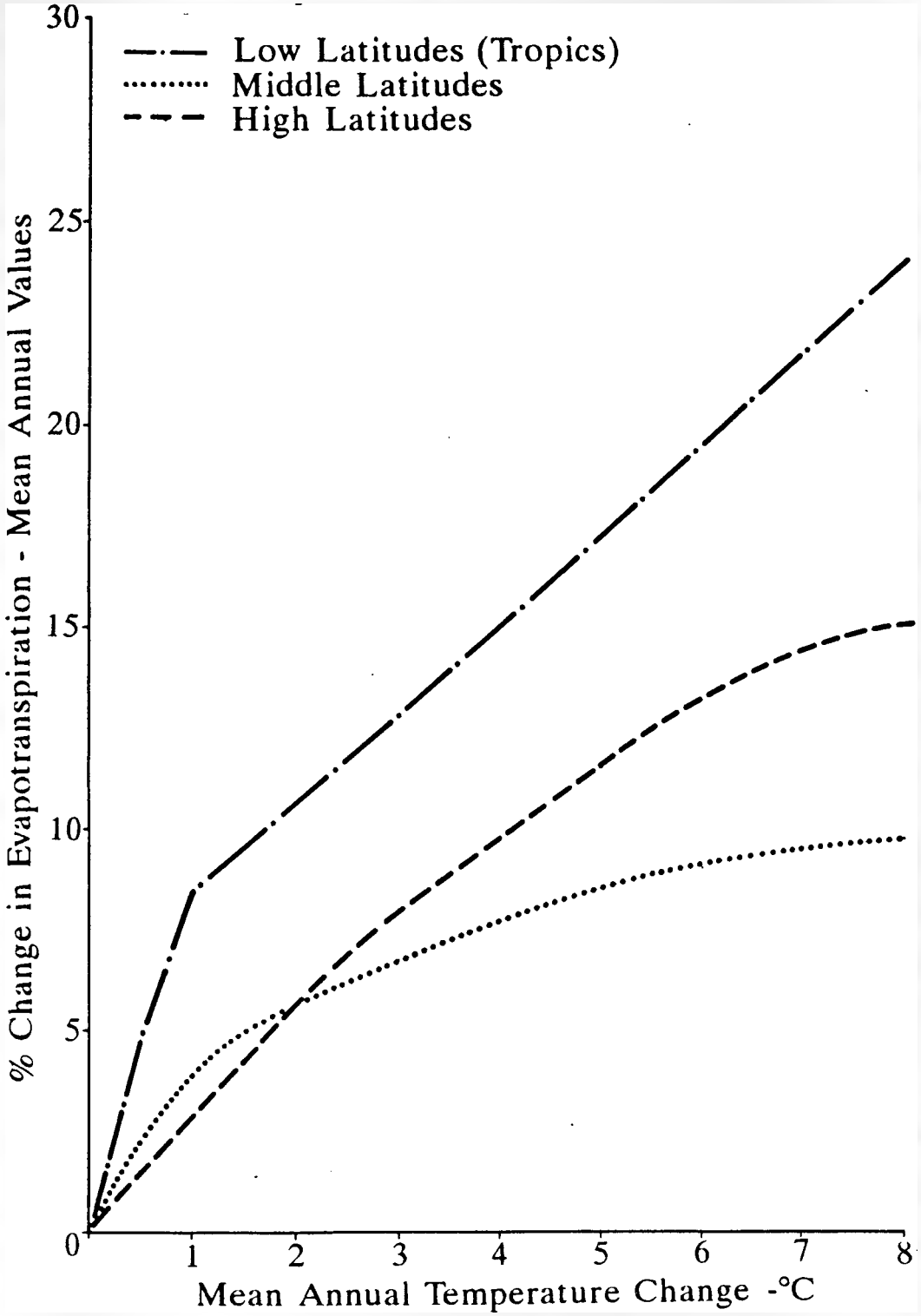


Fig. 7.6

1 Present documented evapotranspiraton values for the stations used are obtained by PENMANS/MEASURED

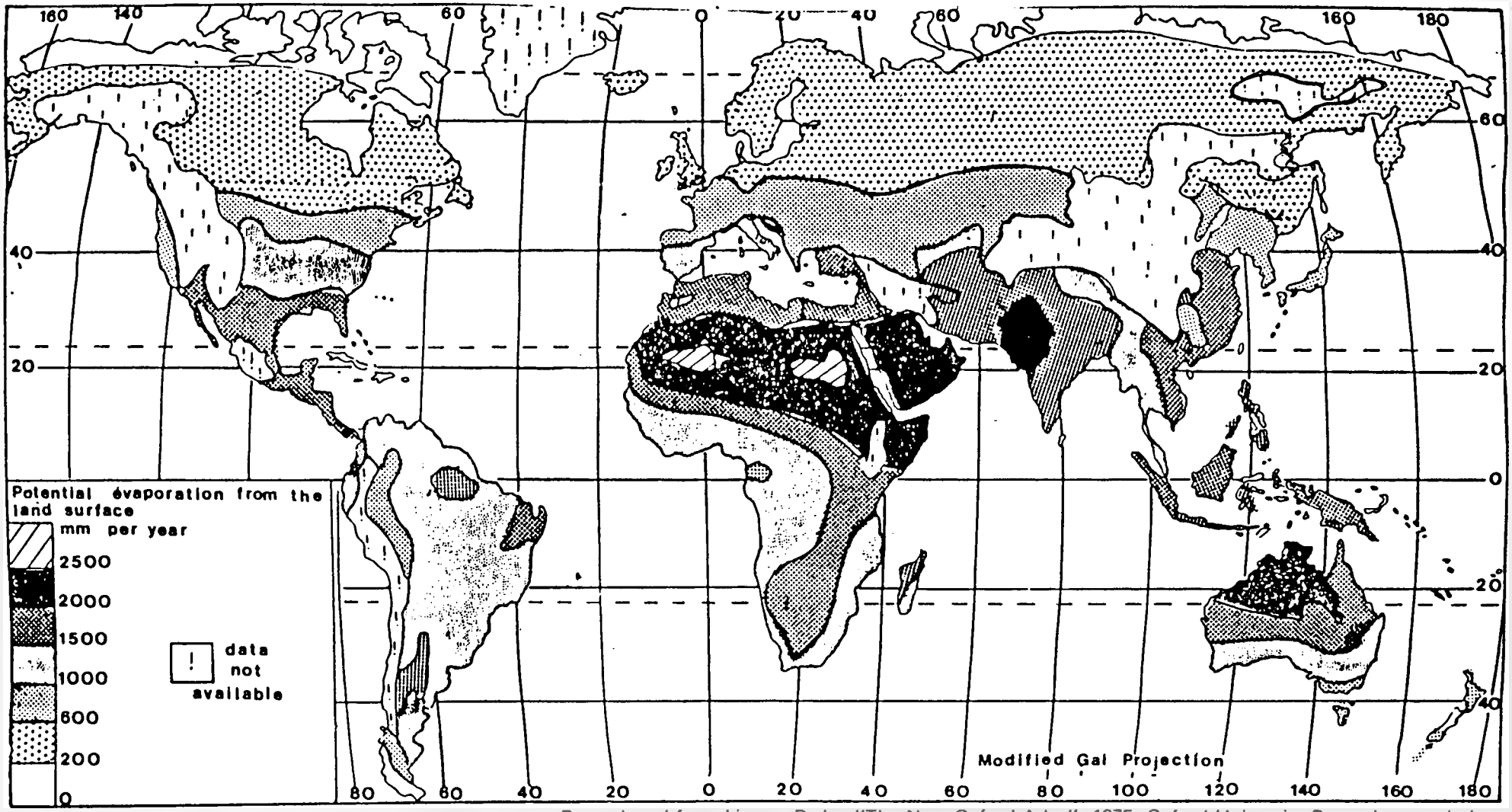
2 Future evapotranspiration values are calculated by THORNWAITES

values for each station representing each region. This was used to adjust the percentage changes expected in that particular region. This was found to be necessary because the Thornthwaite method utilised tends to exaggerate potential evaporation values, and complete reliance on it may give unrealisticly high percentage changes (Shaw,

1988). In preparing the potential evapotranspiration maps given in figures 7.7 (a)-(e), the range of percentages given in table 7.3 were used to specify probable scenarios of increase ranging from 5-65% for the high latitudes, 5-35% for the middle latitudes and 5-50% for the low latitudes or tropics.

The Thornthwaite technique has been used for this study as it is the only method that relates potential evaporation directly with temperature. The mean monthly temperature values used in the above projections are as obtained from the literature for the town of Bergen (61 degrees N) in Norway, Keele (52 degrees N) in England, and Gembu (8 degrees N) in the Donga Basin of Nigeria. These were used to estimate potential evapotranspiration values for the High, Middle, and Low Latitudes/Tropics respectively (Times Atlas, 1987; Shaw, 1988, Diyam and Binies, 1985). Table 7.3 illustrates the expected ranges of percentage changes (increase) in potential evapotranspiration by region around the world as calculated by the Thornthwaite and the adjusted Thornthwaite method.

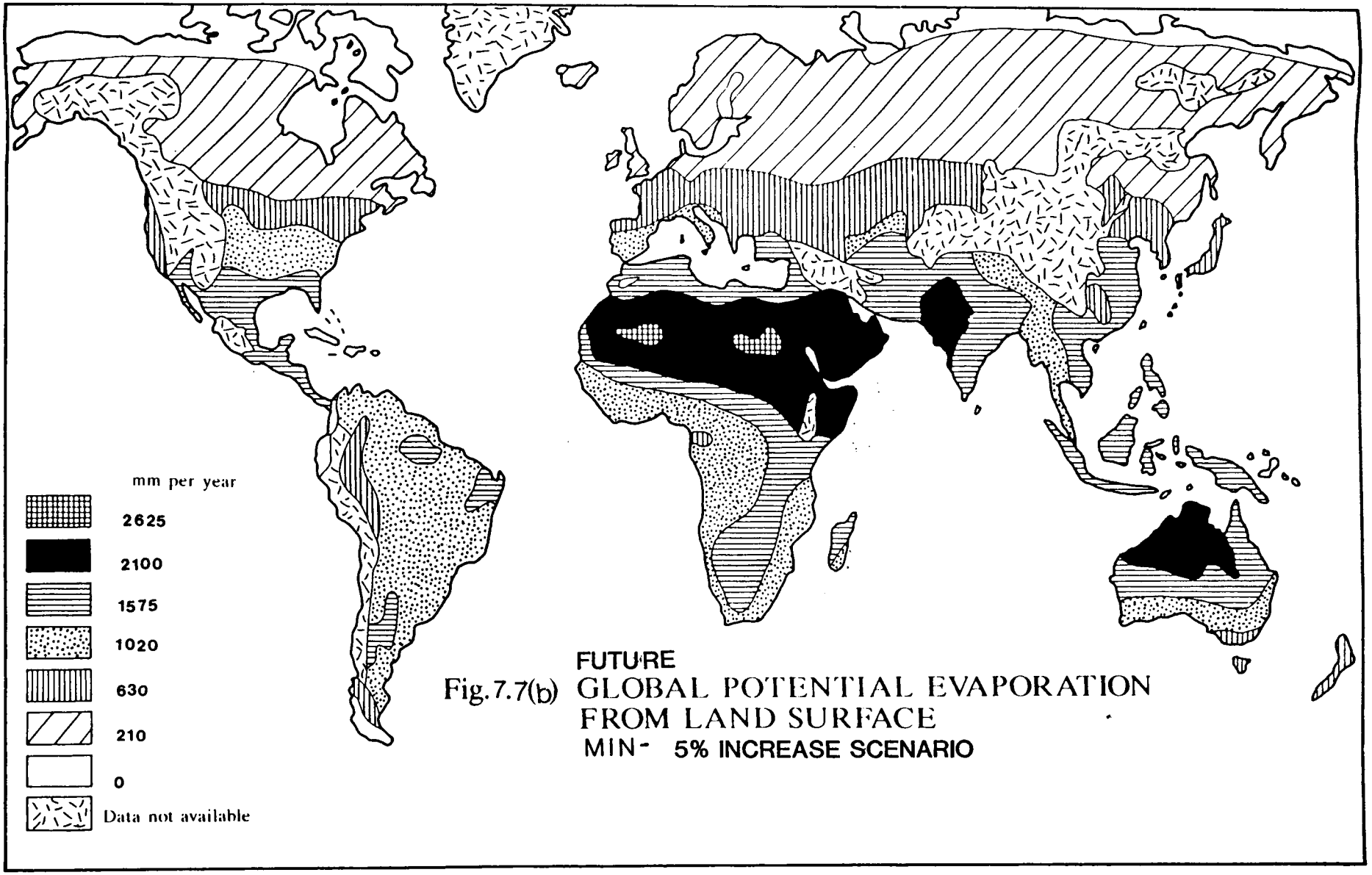
TABLE 7.3 RANGE OF EXPECTED GLOBAL POTENTIAL EVAPOTRANSPIRATION CHANGES AS DEDUCED BY THE USE OF TEMPERATURE RANGES IN TABLE 7.2



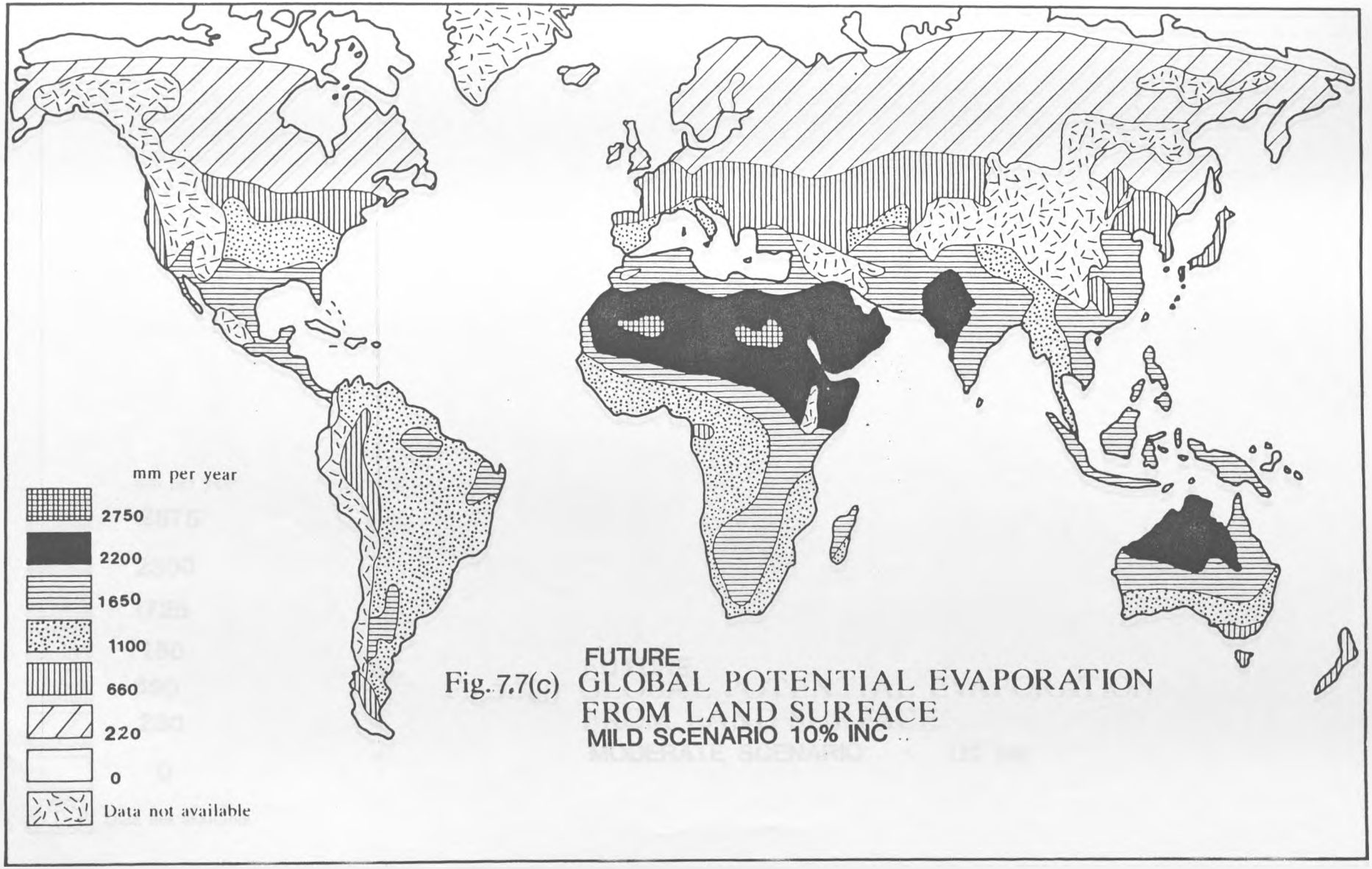
Reproduced from Linton, D. L., "The New Oxford Atlas". 1975. Oxford University Press, by permission.

Fig. 7.7a **Global potential evaporation from land surface** (after Linton[1])

TODAY



FUTURE
Fig. 7.7(b) GLOBAL POTENTIAL EVAPORATION FROM LAND SURFACE
MIN- 5% INCREASE SCENARIO



FUTURE

Fig. 7.7(c) GLOBAL POTENTIAL EVAPORATION FROM LAND SURFACE

MILD SCENARIO 10% INC ..

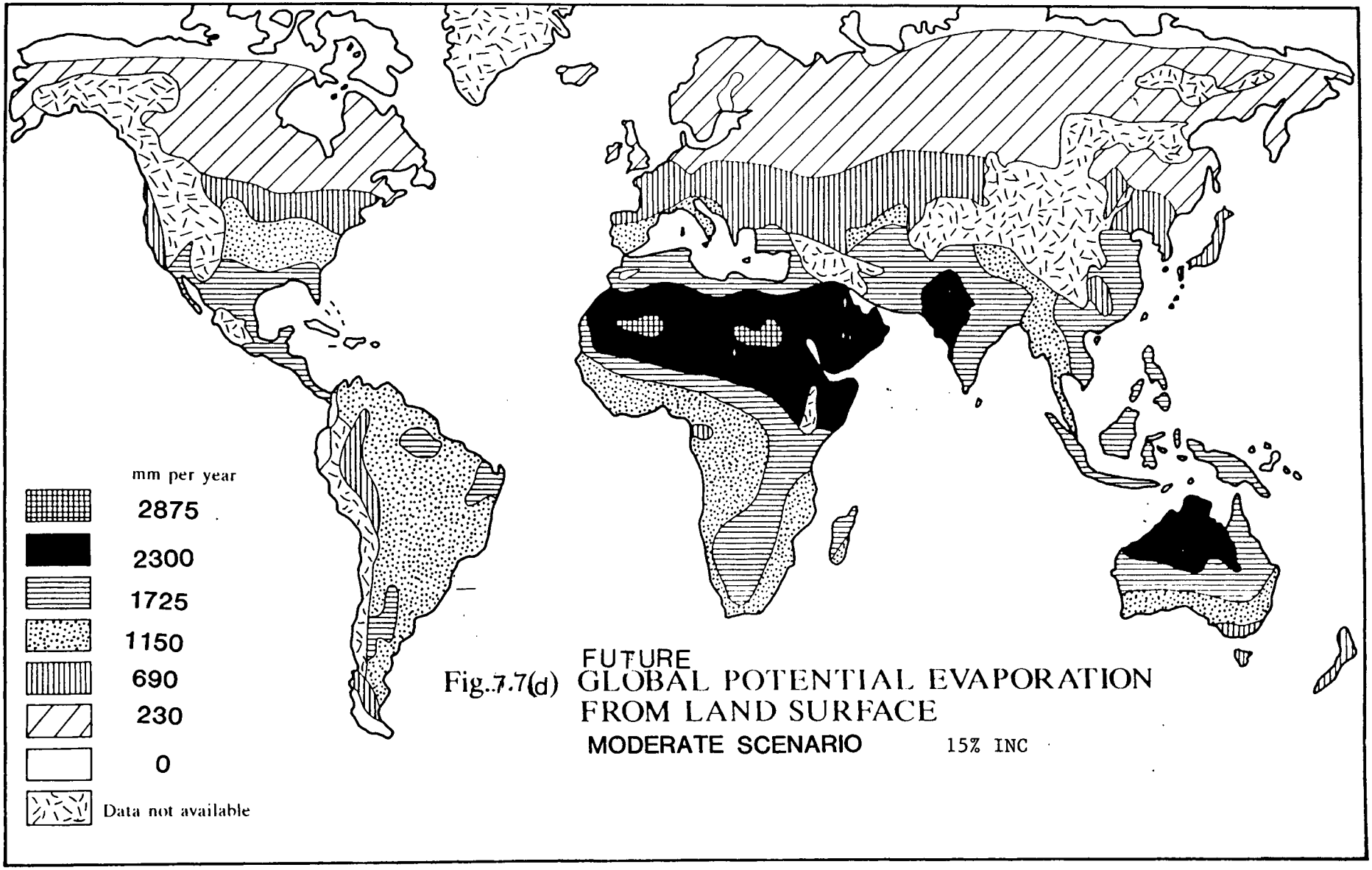


Fig. 7.7(d) FUTURE GLOBAL POTENTIAL EVAPORATION FROM LAND SURFACE
MODERATE SCENARIO 15% INC

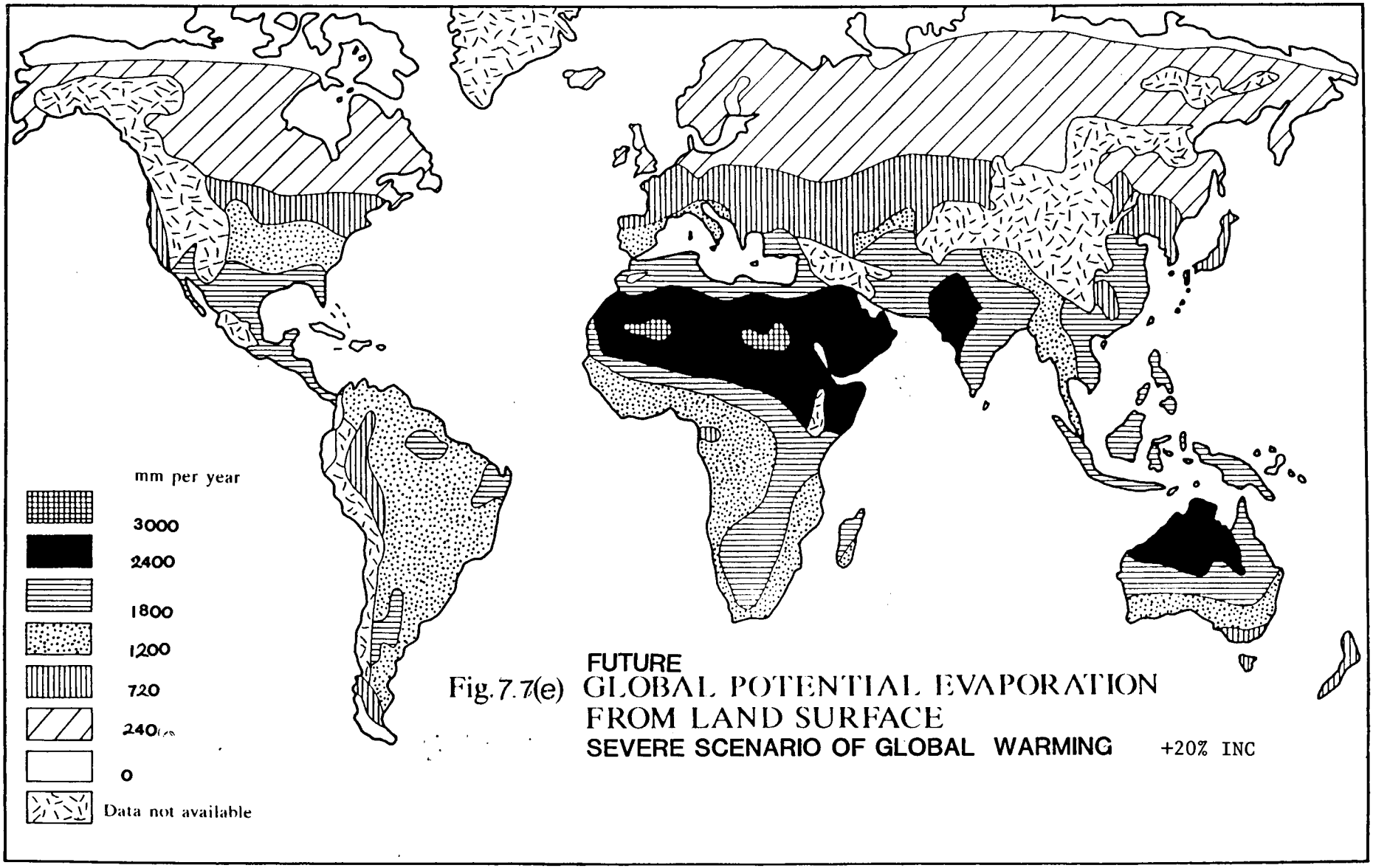


Fig. 7.7(e) FUTURE GLOBAL POTENTIAL EVAPORATION FROM LAND SURFACE
SEVERE SCENARIO OF GLOBAL WARMING +20% INC

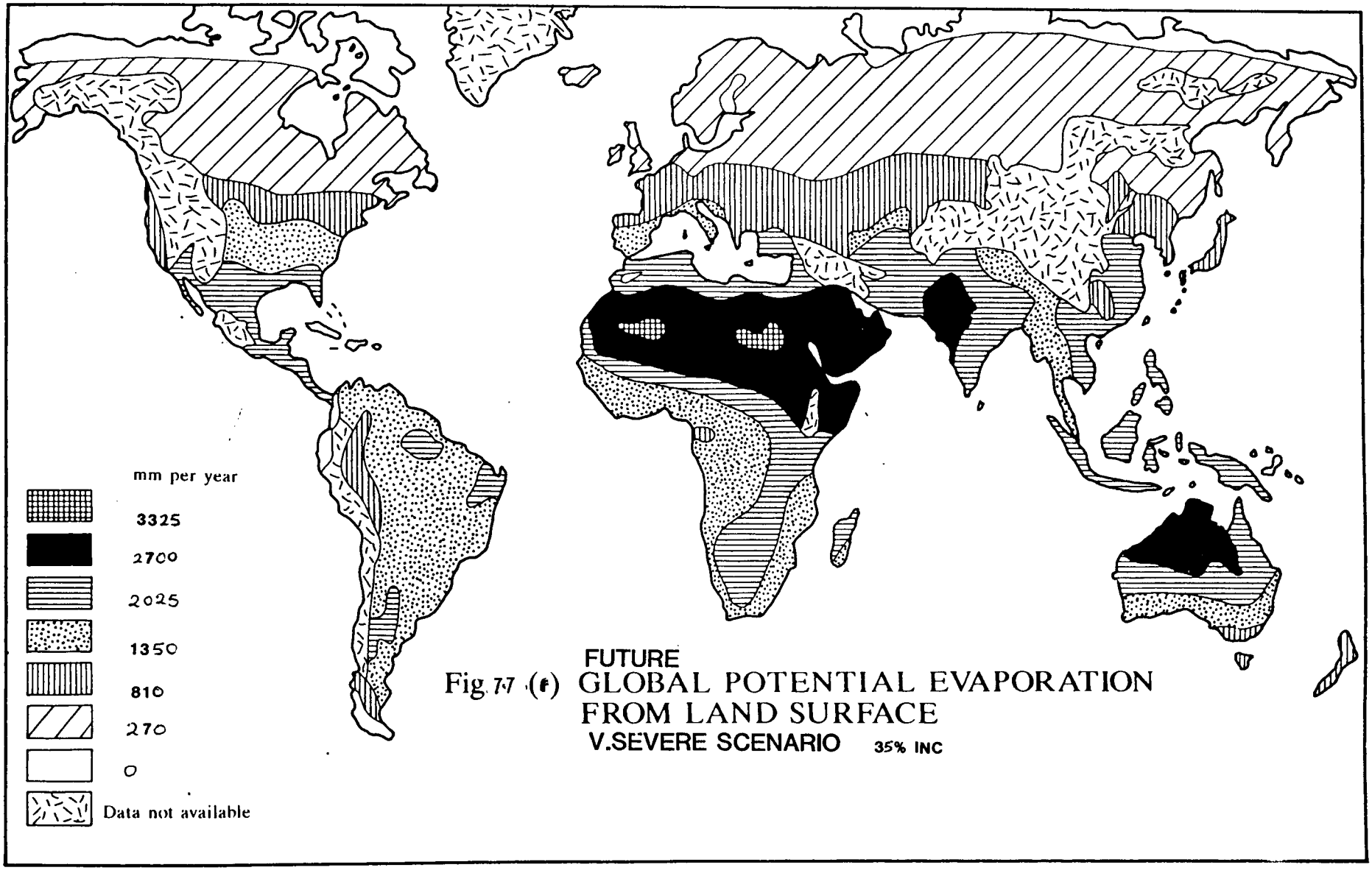
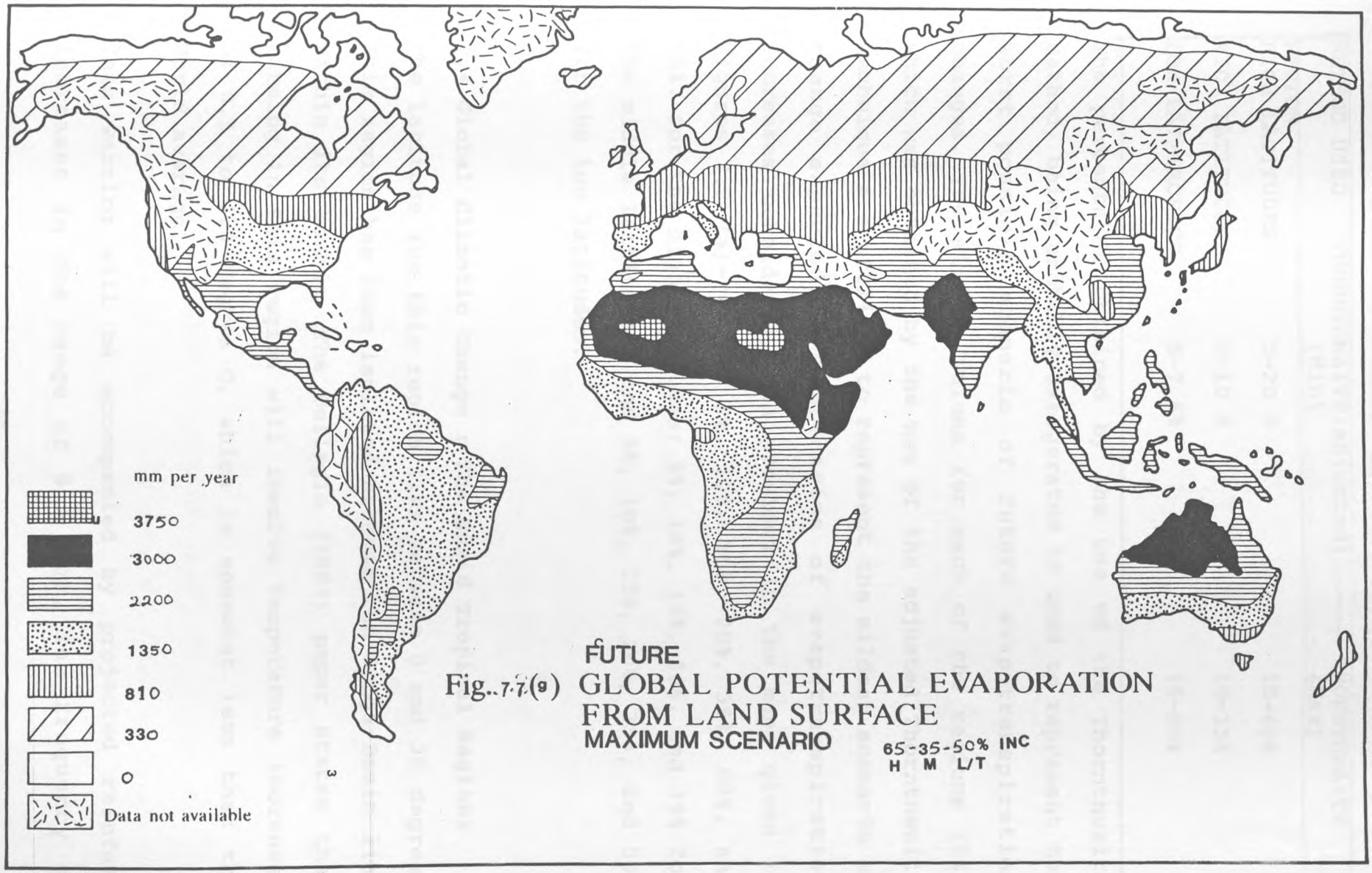


Fig. 7.7 (f) FUTURE GLOBAL POTENTIAL EVAPORATION FROM LAND SURFACE
V.SEVERE SCENARIO 35% INC



FUTURE

Fig. 7.7(g) GLOBAL POTENTIAL EVAPORATION FROM LAND SURFACE
MAXIMUM SCENARIO 65-35-50% INC
H M L/T

TABLE 7.3 RANGE OF EXPECTED GLOBAL POTENTIAL EVAPOTRANSPIRATION CHANGES AS DEDUCED BY THE USE OF TEMPERATURE RANGES IN TABLE 7.2

METHOD USED	THORNTHWAITE(adjusted) (Min)	THORNTHWAITE (Max)
REGION		
HIGH LATITUDES	5-20 %	15-65%
MID LATITUDES	5-10 %	10-35%
LOW LAT/TROPICS	5-7.5%	15-50%

The percentages obtained by the use of the Thornthwaite method, being probably exergerated is used to represent the worst possible scenario of future evapotranspiration changes. The minimum values for each of the regions (5%) which was obtained by the use of the adjusted Thornthwaite technique are utilised to represent the mildest scenario of change expected. The scenarios of evapotranspiration increases considered, and provided in the maps given in figures 7.7 (a)-(e) are 5%, 10%, 15%, 20%, 35%, 50%, and 65%. for the high latitudes; 5%, 10%, 15%, 20%, and 35% for the middle lattitudes; and 5%, 10%, 15%, 20%, 35%, and 50% for the low latitudes.

7.4 Global Climatic Change : The Humid Tropical Regions

The latitude for this region lies between 0 and 35 degrees. This region has been isolated because the Donga Basin lies within its bounds. The Bellagio (1987) paper states that warming in this region will involve temperature increases of 0.5 to 4.0 degree C, which is somewhat less than the World average.

This warming will be accompanied by projected rainfall increases in the range of 5 - 20%. The frequency of

rainfall and the length of the rainy season could decrease in places with possible shifts in cloudiness and geographical rainfall. Potential evapotranspiration will increase and this could give drought stresses in many local regions in these humid tropics.

With increased ocean temperatures, tropical storms may occur in areas where they are now uncommon. Where they already occur, increased wind intensity and rainfall would be expected.

The other major effects in this region resulting from climate changes from within and without this geographical limit are :

(1) the rise in water levels along coasts and rivers resulting from a combined effect of rising sealevels, greater charge of tropical storm surges and rising peak runoff. This will increase the occurrence of floods and salinity.

(2) the changing spatial and temporal distribution of temperature and precipitation and this has a significant impact on industry, settlement, agriculture, grazing lands, fisheries and forests

The Bellagio workshop concludes that the areas of this region most vulnerable to the effects of climate change over the next century are coastal and river zones which would be subjected to sealevel changes and storms and upland areas with infertile soils. Prior to an assessment of the

magnitude and effects of the future climate change in the Donga Basin with respect to the global situation, a brief description of the physiographic and present climatological characteristics of the catchment are now briefly described.

7.5 The Donga Basin : Current Conditions

The Donga river, a left bank tributary of the Benue, located in mid-eastern, Nigeria drains the Mambilla plateau and the northern slopes of the cameroon mountains in the upper third of its catchment. The basin has a total area of 20,000 sq km. Gembu (20,000), the main town on the plateau lies slightly north of the river in the central portion of the plateau. Below Gembu lies a good dam and a large reservoir site near Tepkwar village which drains an area of 2,000 sq km.

The Mambilla plateau terminates sharply at Abong town and has an area of about 6,000 sq km. A good reservoir site with a proposed dam at Manya lies in the middle Donga basin. Manya is about 125 km upstream of the Benue confluence. This reservoir, with a surface area of 426 sq km will impound runoff of some 9,000 sq km and lies on an undulating plain of low hills . Figures 1.10 shows the various reservoir sites and figures 7.8 and 7.9 show the location and relief of the basin respectively.

The main physiographic feature of the basin is the Mambilla plateau, which is a high peneplain reaching a maximum elevation of 1850 (masl). The plateau level is fairly

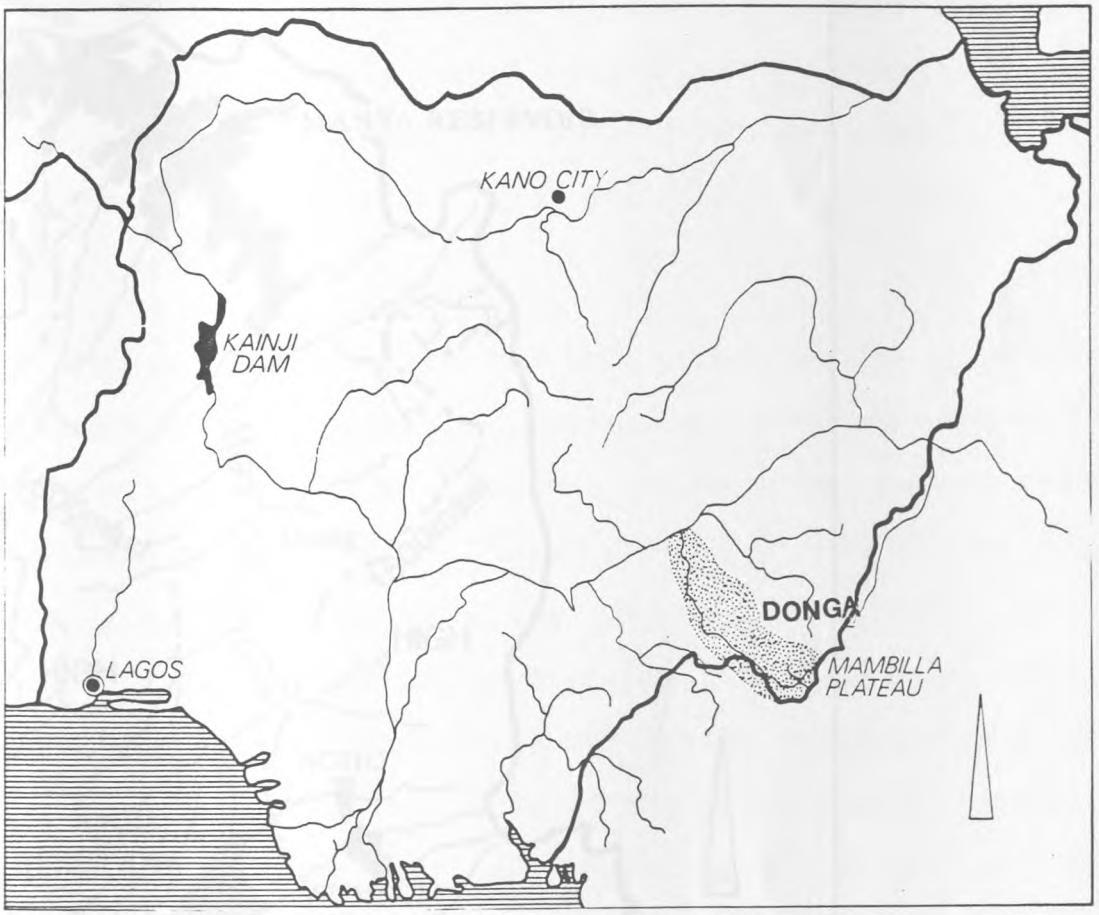


FIG 7.8 MAP OF NIGERIA

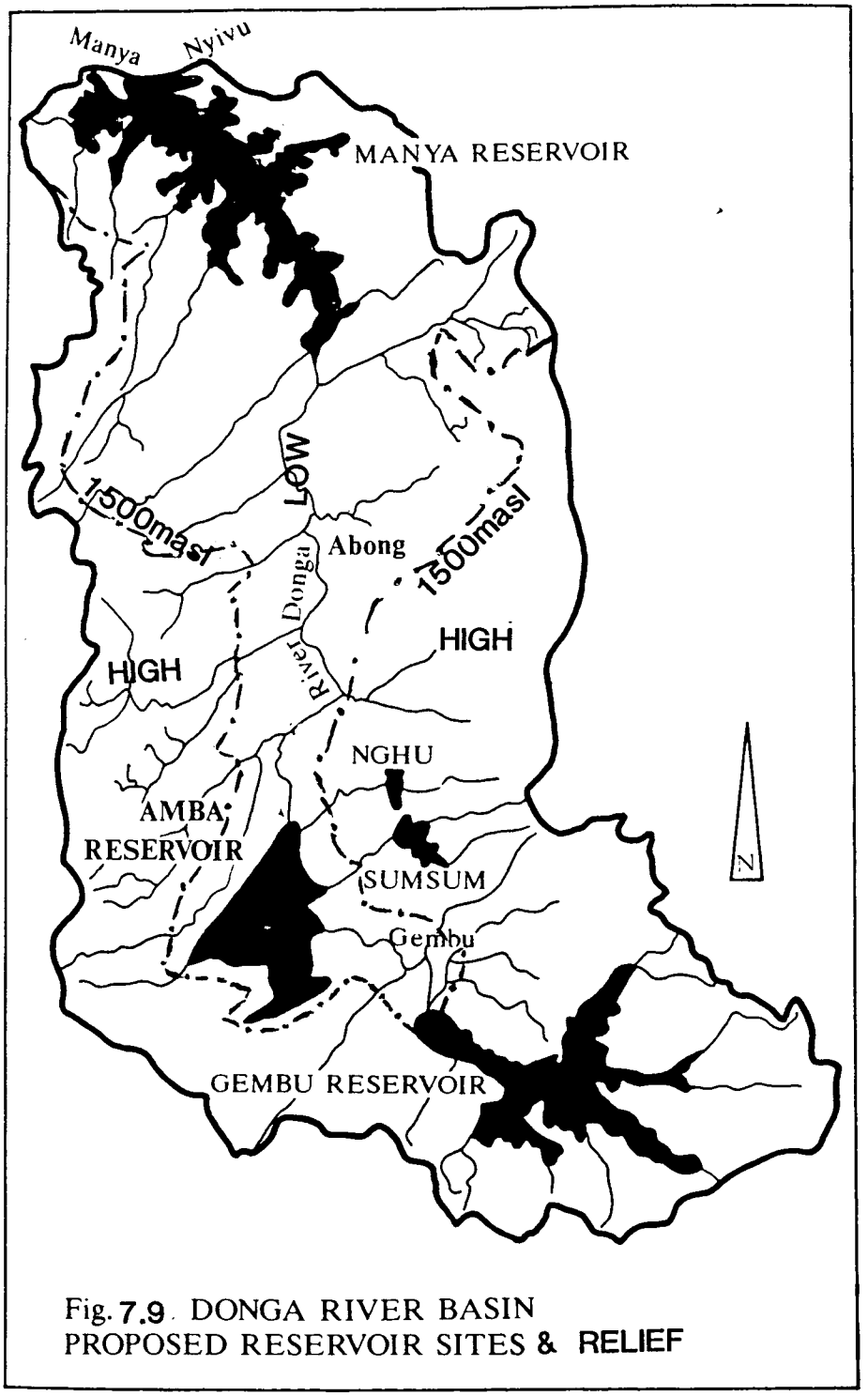


Fig. 7.9 DONGA RIVER BASIN
PROPOSED RESERVOIR SITES & RELIEF

consistent at about 1300-1400m. It is criss-crossed by deep incised valleys. Downstream of Gembu, near the town of Abong, the plateau falls nearly 1000m to the plain below which slopes gently to the Benue river. The undulating plain of the middle portion of the basin lies between 202 and 300 masl with isolated rugged hills rising a further 150m above this level and surrounded on three sides by higher ground. (Diyam and Binnies, 1985; MRT, 1978; ALTS/Lideco, 1978)

The geology of the basin consists of an ancient crystalline basement complex-granites, syenites, gneisses and a few remnant of tetiary basalt. Magmatites occur freely throughout the basin. Rivers tend to follow the major joints and fault lines of the granitic rocks which tend to have a NE-SW and NW-SE orientation. This gives a distinct pattern of parallel valleys with rivers turning sharp bends of 90 deg or more. Dam sites on the plateau mainly consists of granitic rocks with superficial deposits of weathered materials on the hillsides.

The climate of the plateau, and indeed, the whole basin, is characterized by the prevailing dry north-east winds and moist south-west monsoons which dominate the whole of Nigeria for each half of the year. The junction of these two wind systems form the inter-tropical front (ITF or ITCZ or ITD). The climate of the basin and indeed the whole country is dependent on the north and southward movement of this front.

Variations in monthly temperatures are small but differences do occur due to local relief. Mean daily temperature differ typically by 10 deg C or so. Tables of mean monthly temperatures for the entire catchment are given in Diyam (85) . Relative humidity varies from 20% in the dry season (Nov-Feb) to 100% in the wettest months (Sept-Oct). Mean monthly humidity values are also provided in Diyam (85) . The number of sunshine hours per day varies with the seasons, maximum in the dry and minimum in the wet. Wind speeds and directions are directly related to the position of the 'ITCZ'. Diyam (85) also provides typical values of windspeeds and the length of sunshine hours.

Rainfall in the entire basin is strongly influence by relief, as it increases steadily with increasing elevation as one moves southwards higher up the basin and maximum falls occur along the edge of the plateau and on higher ground in the cameroon on the southwest boundary of the basin. The mean annual rainfall for the catchment downstream to Manya damsite is 2085 mm. Figure 7.10 shows a typical isohyet of the basin and the locations of the climate stations. Mean monthly flows for the Donga river at Gembu, Abong and Manya for the standard period 1959-81 are provided in table 7.4.

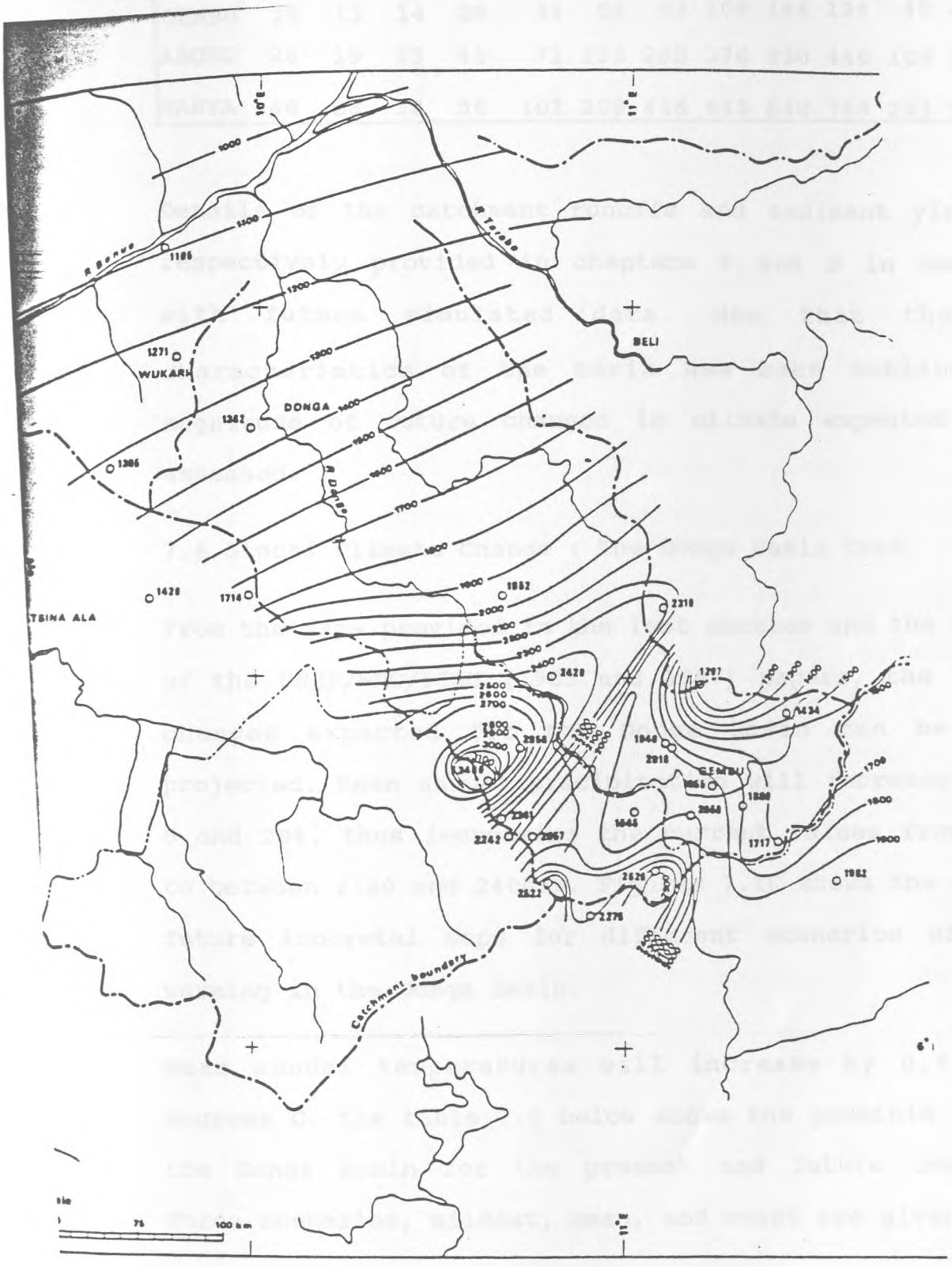


Fig 7.10
ANNUAL RAINFALL FOR DONGA CATCHMENT 1959-1980

TABLE 7.4 MEAN MONTHLY FLOWS (Cumecs)

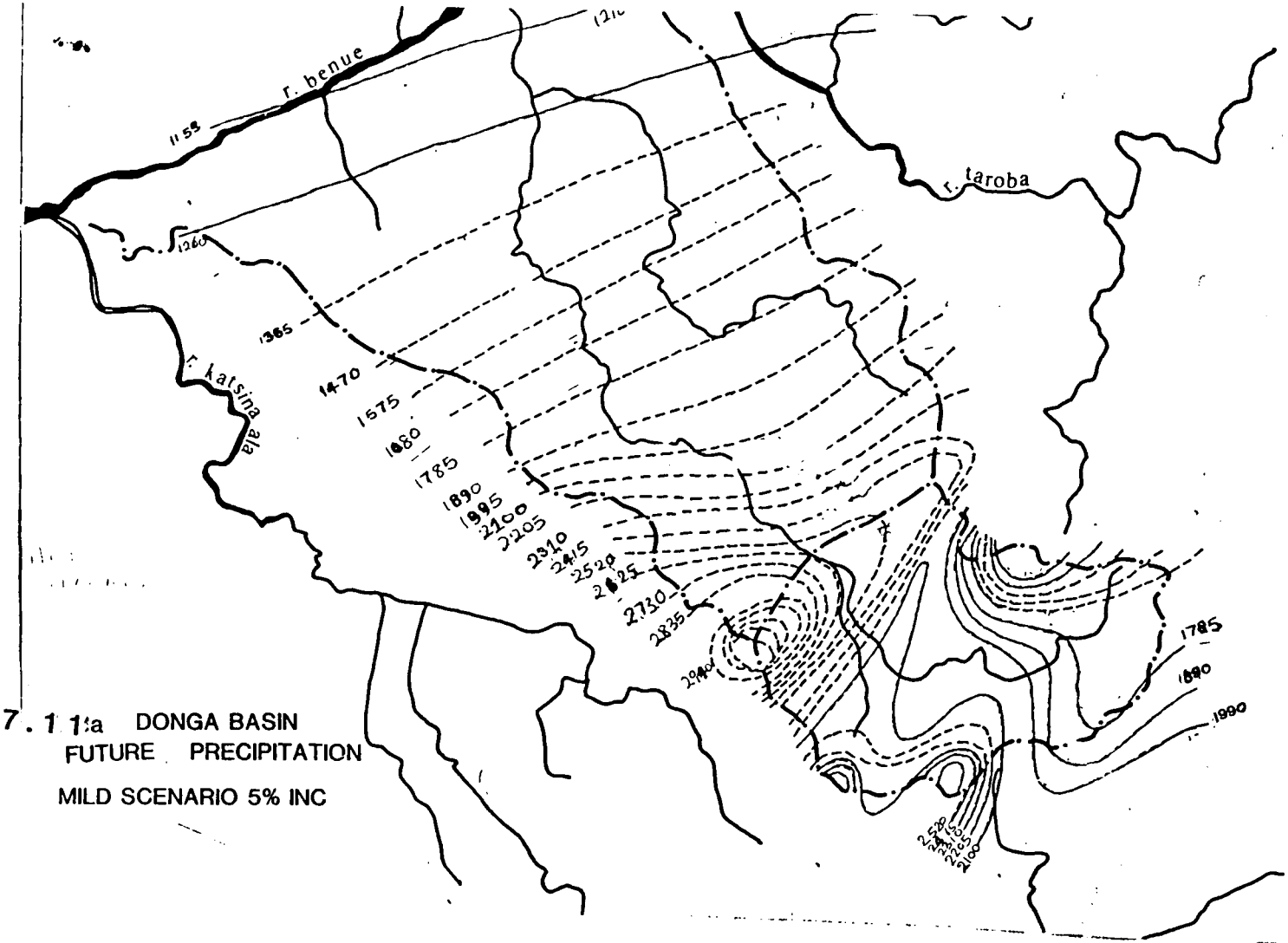
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Avg
GEMBU	15	13	14	28	41	58	83	106	144	124	45	21	58
ABONG	29	19	23	41	71	133	265	376	490	446	109	58	172
MANYA	46	28	32	56	102	208	446	645	840	768	293	95	298

Details of the catchment runoffs and sediment yields are respectively provided in chapters 8 and 9 in comparison with future simulated data. Now that the basic characteristics of the basin has been outlined, the magnitude of future changes in climate expected is now assessed.

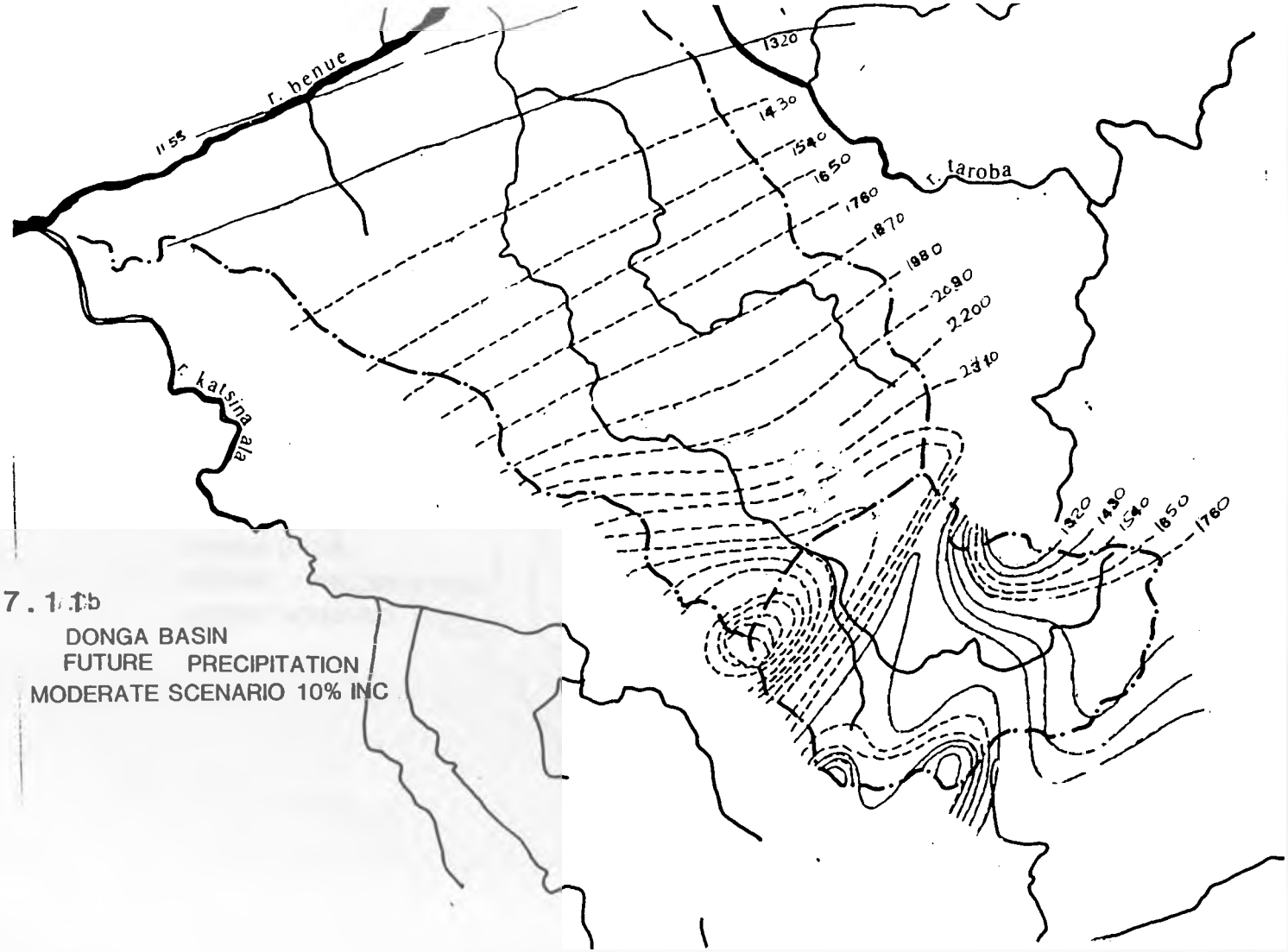
7.6 Global Climate Change : The Donga Basin Case

From the maps provided in the last section and the extracts of the UNEP/WMO/ICSU (1985 and 1987) papers, the climatic changes expected for the Donga Basin can be readily projected. Mean annual precipitation will increase between 5 and 20%, thus increasing the current values from 2000mm to between 2100 and 2400mm. Figures 7.11 shows the expected future isohyetal maps for different scenarios of global warming in the Donga Basin.

Mean annual temperatures will increase by 0.5 to 4.0 degrees C. the table 7.2 below shows the possible trend in the Donga Basin for the present and future conditions. Three scenarios, mildest, mean, and worst are given.



7.11a DONGA BASIN
FUTURE PRECIPITATION
MILD SCENARIO 5% INC



7.1.1b

DONGA BASIN
FUTURE PRECIPITATION
MODERATE SCENARIO 10% INC

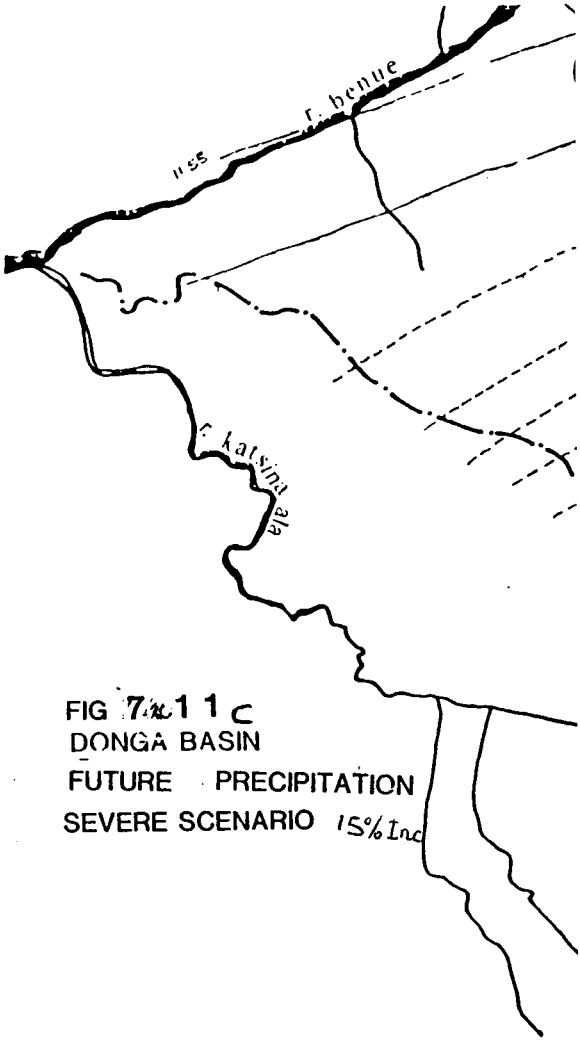
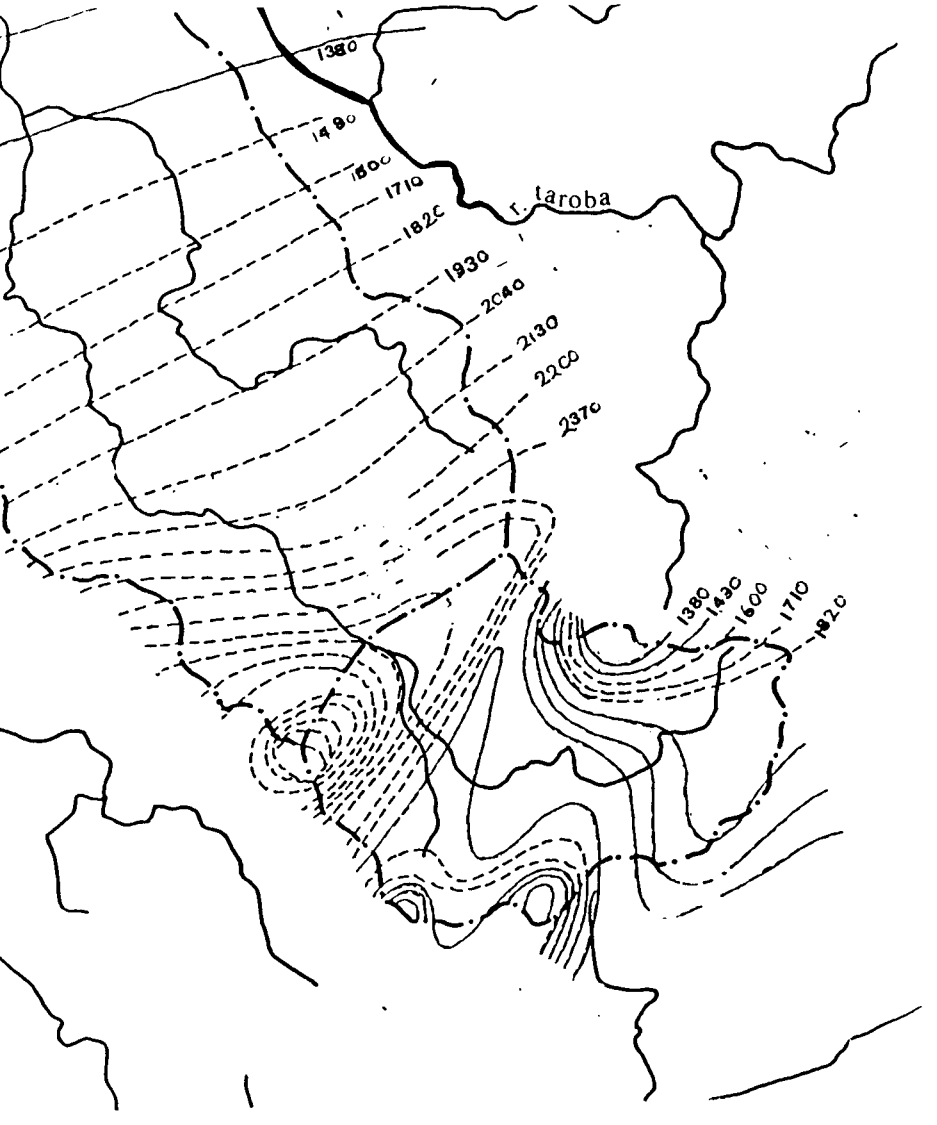
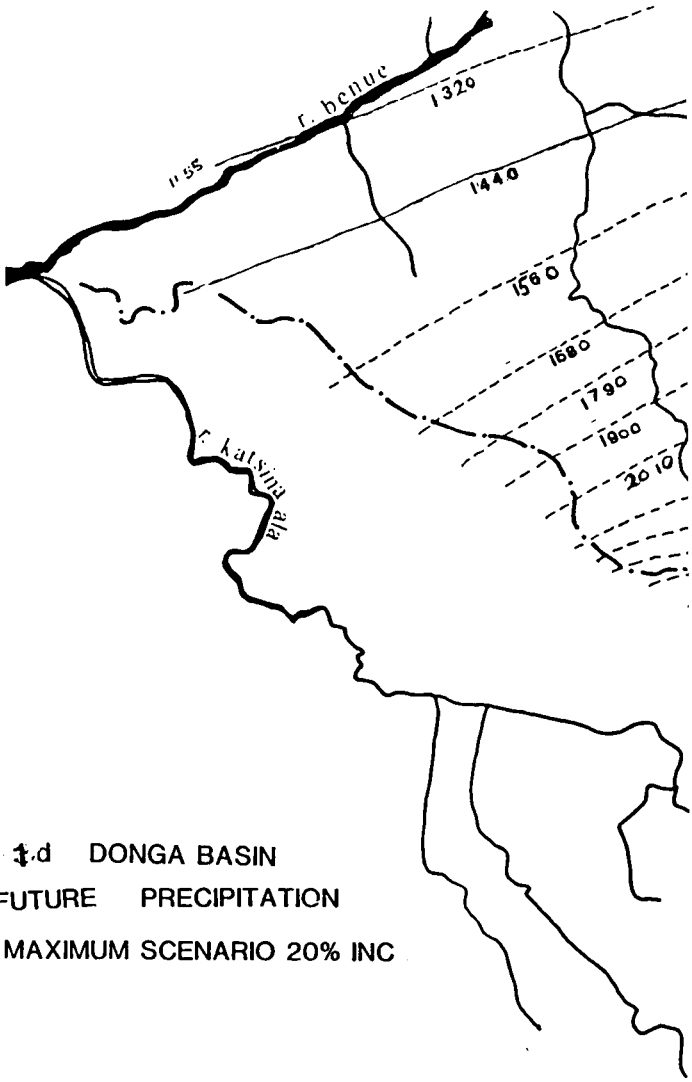


FIG 7.11c
DONGA BASIN
FUTURE PRECIPITATION
SEVERE SCENARIO 15% Inc


1380
1490
1600
1710
1820
1930
2040
2130
2200
2370
t. taroba
1380
1430
1600
1710
1820



7.11.d DONGA BASIN
FUTURE PRECIPITATION
MAXIMUM SCENARIO 20% INC

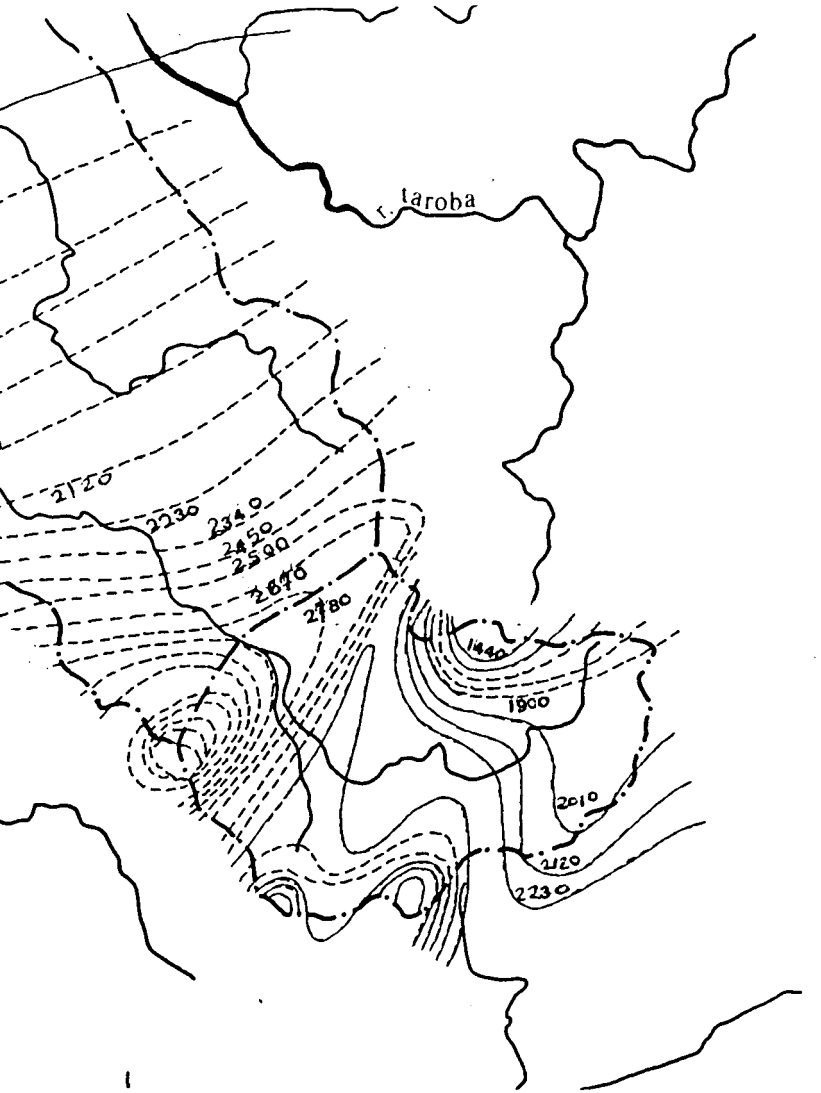

Taroba
2125
2230
2340
2450
2590
2670
2780
1440
1900
2010
2120
2230

TABLE 7.5 MEAN ANNUAL TEMPERATURES FOR EIGHT STATIONS IN THE DONGA BASIN IN DEGREE CENTIGRADE.

STATION	CURRENT	FUTURE MILD	FUTURE MEAN	FUTURE WORST
GEMBU	19.3	19.8	21.55	23.3
ARDO GORE	17.6	18.1	19.85	21.6
MBANGA	18.6	19.1	20.85	22.6
DOROFI	19.4	19.9	21.65	23.4
NGUROJE	18.0	18.5	20.25	22.0
MAI SAMARI	19.5	20.0	21.75	23.5
MAYO NDAGA	18.5	19.0	20.75	22.5
DONGA	27.0	27.5	29.25	31.0

From the temperature figures and changes above, future annual potential evapotranspiration values have been worked out specifically for the Donga Basin for the station at Gembu from appropriate equations and plots using the Thornthwaite equations (Shaw, 1988; Wilson,1983)

The estimated future annual potential evapotranspiration values in mm for Gembu on the catchment as compared to current values are given in table 7.5 below.The mildest, mean and the worst expected changes as obtained from figure 7.6, which is a modification of figure 7.5 are given in table 7.5 below.

TABLE 7.6 ANNUAL POTENTIAL EVAPOTRANSPIRATION AT GEMBU, DONGA BASIN

	MILDEST	MEAN	WORST
FUTURE (Thonthwaite)	1203	1400	1510
CURRENT ,,	1050	1050	1050
CURRENT (Penman)	1320	1320	1320

7.7 The Influence of Resevoir Construction

The effect of the construction of a reservoir on local climate has been mentioned in chapter 1. This effect may be significant and thus requires assessment. Thereafter the values obtained can be used in testing the river basin

management system.

Evaporation is expected to increase but precipitation may or may not depending on the local wind system. Evaporation losses in reservoirs are already accounted for in the routing operations model of the system (chapter 5). A mean annual reservoir evaporation of 0.80-0.98 cumecs or about 134mm, an increase of about 11.6% on a pre-construction catchment mean value of 983mm (Diyam, 1985) is anticipated, as output by the model.

Precipitation increases are not expected to be higher than 10%, considering the Arizona study undertaken by Mc'Donald (1962), and recounted by Matther (1984) (chapter 1). The study found that a theoretical lake of 50,000 sq km is required to increase mean July precipitation values in the state which has a total area of 313,000 sq km by 10%.

With this Arizona example in mind, the probable increase in precipitation projected for the Donga Basin by the use of the combined area of all proposed reservoirs in the catchment can be estimated. The combined area of these reservoirs from Gembu to Manya is 800 sq km (fig 1.8). The total catchment area of the basin at Manya is just over 9,000 sq km. The ratio of the proposed surface area to catchment area is about 9%. If wind conditions in the Donga Basin are assumed to be similar to Arizona's, then catchment precipitation in the Donga would be expected to

increase by no more than 5.6%, by simple proportion. As real conditions may be different from that assumed and projected, a precipitation rise ranging from a minimum of 0% to a maximum of 11.6% has been assumed. 11.6% is the expected lake evaporation in the basin.

7.8 Final Remarks

The magnitude of the expected global climate change in the next 50-100 years as evaluated in this chapter will be substantial, should the current trends in the production of GHG's continue. The magnitudes of the expected global changes in temperature, evapotranspiration and precipitation were calculated separately and presented. The values for the low lattitudes (or tropics), the region which contains the case study have also been presented. The level of change expected in the Donga Basin were similarly worked out and noted but prior to this, a description of the physico-geological characteristics and the present climatic conditions of the catchment were presented. The magnitudes of the expected changes obtained herein are used in running the River Basin Management System to assess their effects on

(1) the Donga Basin water balance. This is presented in Chapter 8.

(2) the sediment balance of the catchment. This is given in Chapter 9

(3) the reservoir sedimentation response for the Gembu

reservoir. This is also presented in Chapter 9.

(4) the economic performance of the Gembu reservoir. This is presented in Chapter 10 and 11 for short and long scenarios of the expected future changes in the climate of the Donga basin.

CHAPTER 8

CHAPTER 8 : DONGA BASIN WATER BALANCE ANALYSIS

In this chapter, the water balance response of the Donga Basin is assessed by tests involving the use of the River Basin Management system. These tests are performed in three parts, namely,

(1) Investigating the effect of short term changes in basic landuse parameters.

(2) Assessing the short term impact of reservoir induced climatic changes.

(3) Predicting the effect and impact of long term changes caused by the combined effect of estimated reservoir induced climate changes and the projected global climatic shift.

The magnitude of the global climatic change utilised in the tests has already been presented in chapter 7. Section 8.1 describes the simulation format and methodology for running the watershed model. In section 8.2 the run format and techniques for the operation of the routing model are described. Section 8.3 is devoted to the calibration exercise for the Donga Basin using the watershed model. The results of tests involving short term changes in landuse parameters are presented, analysed and discussed in section 8.4. The results of the short term impact of reservoir induced climate changes are presented in section 8.5. Section 8.6 is concerned primarily with the effect of long term climatic shifts resulting both from reservoir and globally induced changes, complete with presentation of

results, analysis and discussion. The chapter is concluded in section 8.7 with a general discussion of the implications of the findings with a focus on the problems envisaged for the Donga Basin.

8.1 Running the Watershed Model.

The watershed model is tested herein as part of the RBMS using the database of the Donga river. The figure 8.1 clearly indicates the three sub-basins seperately considered and utilised for water balance analysis. These are the Donga-Tepkwar sub-catchment with an area of 1890 sq km; the Donga-Abong one with an area of 6000 sq km and the Donga-Manya watershed with an area of 9000 sq km. The Donga-Benue portion which covers the whole Donga basin is not being considered due to the unavailability of flow data at the Benue-Donga confluence.

In all cases, the catchment has been subdivided into two segments by an altitude division of 1000 masl. The figure 8.1 also illustrates the segmental division in the Donga basin. Hydrometeorological data was available for a limited number of gauging stations. The existing gauge stations and catchment isohyets are shown in figure 7.10 (a) . In the Upper Donga Sub-basin, five of these raingauges were used to compute mean watershed precipitation by the Thiessen polygon method. These gauges are at Gembu, Mayo Ndaga, Dorofi, Nguroje and Nmbanga. In the second sub-basin, one additional gauge at Abong was included in the mean precipitation computation. In the third and largest sub-basin, an additional gauge at Baissa was included in the computation of mean rainfall. In all cases, the available isohyets, were utilised to check the gauge values and the computed mean basin precipitation. All rainfall data are daily while evaporation data are monthly.Daily evaporation

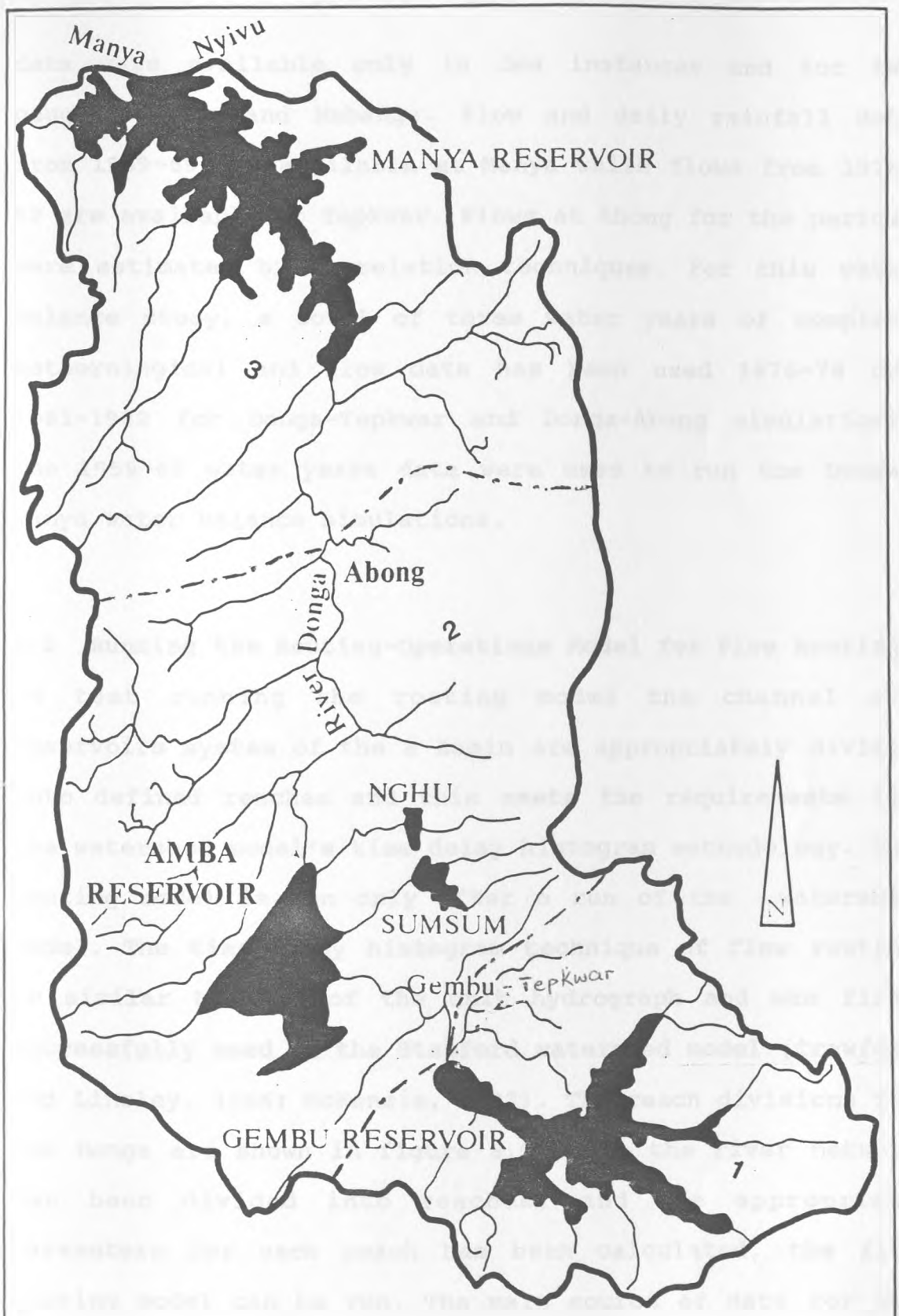


Fig. 8.1 DONGA RIVER BASIN
PROPOSED RESERVOIR SITES & SUB-BASINS

data were available only in few instances and for two gauges- Gembu and Nmbanga. Flow and daily rainfall data from 1959-69 are available at Manya while flows from 1976-82 are available at Tepkwar. Flows at Abong for the periods were estimated by correlation techniques. For this water balance study, a total of three water years of complete meteorological and flow data has been used 1976-78 and 1981-1982 for Donga-Tepkwar and Donga-Abong simulations. The 1959-62 water years data were used to run the Donga-Manya water balance simulations.

8.2 Running the Routing-Operations Model for Flow Routing

In test running the routing model the channel and reservoirs system of the a basin are appropriately divided into defined reaches and this meets the requirements for the watershed model's time delay histogram methodology. The routing model is run only after a run of the watershed model. The time delay histogram technique of flow routing is similar to that of the unit hydrograph and was first successfully used in the Stanford watershed model (Crawford and Linsley, 1966; McKenzie, 1983). The reach divisions for the Donga are shown in figure 8.2. Once the river network has been divided into reaches, and the appropriate parameters for each reach has been calculated, the flow routing model can be run. The main source of data for the model is produced by the watershed model, consisting of total hourly flow. The model converts this data to the flow entering each reach by multiplying the depth by the

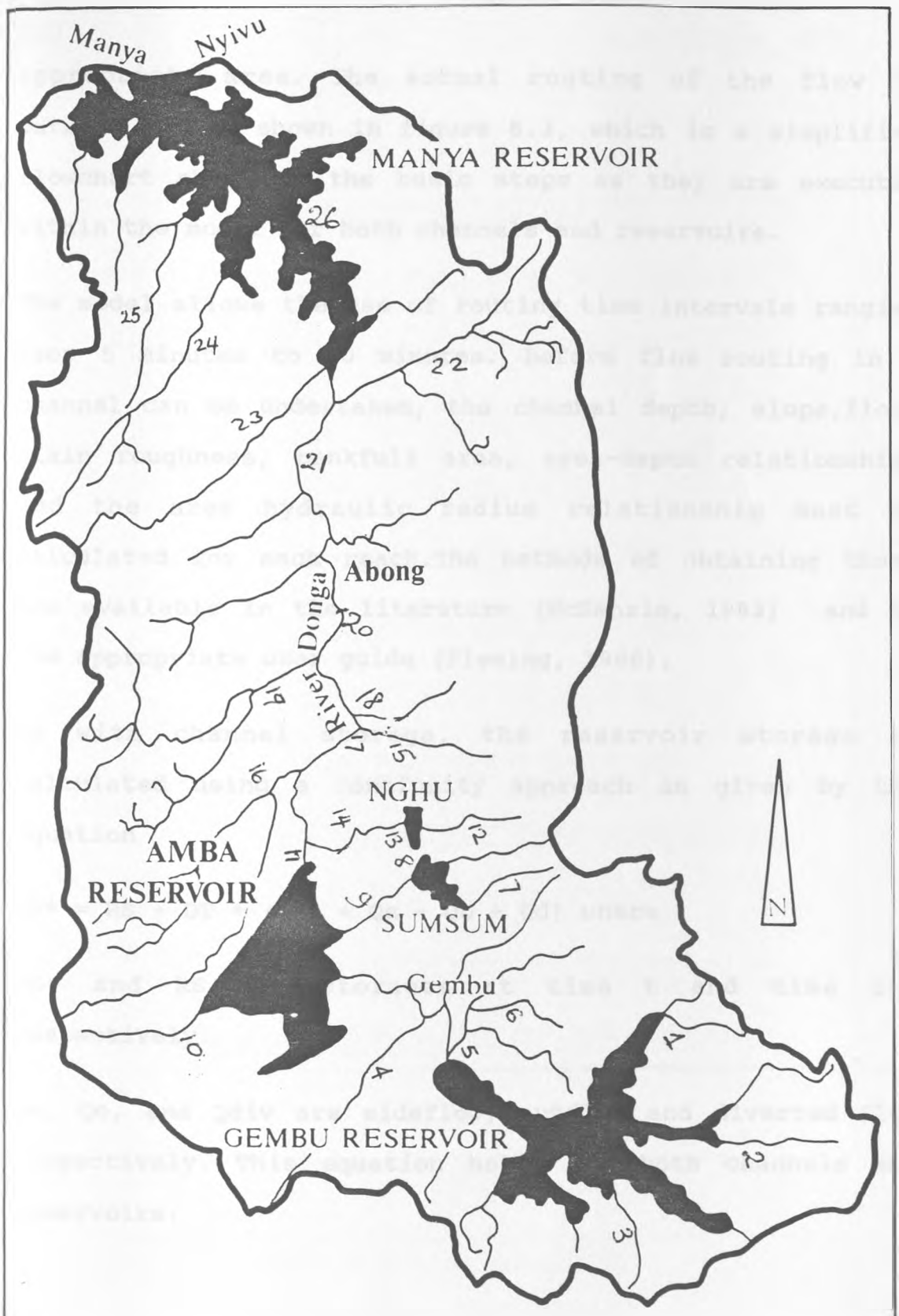


Fig. 8.2 DONGA RIVER BASIN
PROPOSED RESERVOIR SITES & REACH DIVISIONS

appropriate area. The actual routing of the flow is carried out as shown in figure 8.3, which is a simplified flowchart showing the basic steps as they are executed within the model for both channels and reservoirs.

The model allows the use of routing time intervals ranging from 5 minutes to 60 minutes. Before flow routing in a channel can be undertaken, the channel depth, slope,flood plain roughness, bankfull area, area-depth relationship, and the area hydraulic radius relationship must be calculated for each reach.The methods of obtaining these are available in the literature (McKenzie, 1983) and in the appropriate user guide (Fleming, 1986).

As with channel storage, the reservoir storage is calculated using a continuity approach as given by the equation

RS* = RS + DT * (Qin + Qs - Qo + Qd) where

RS* and RS are storages at time t and time t-1 respectively.

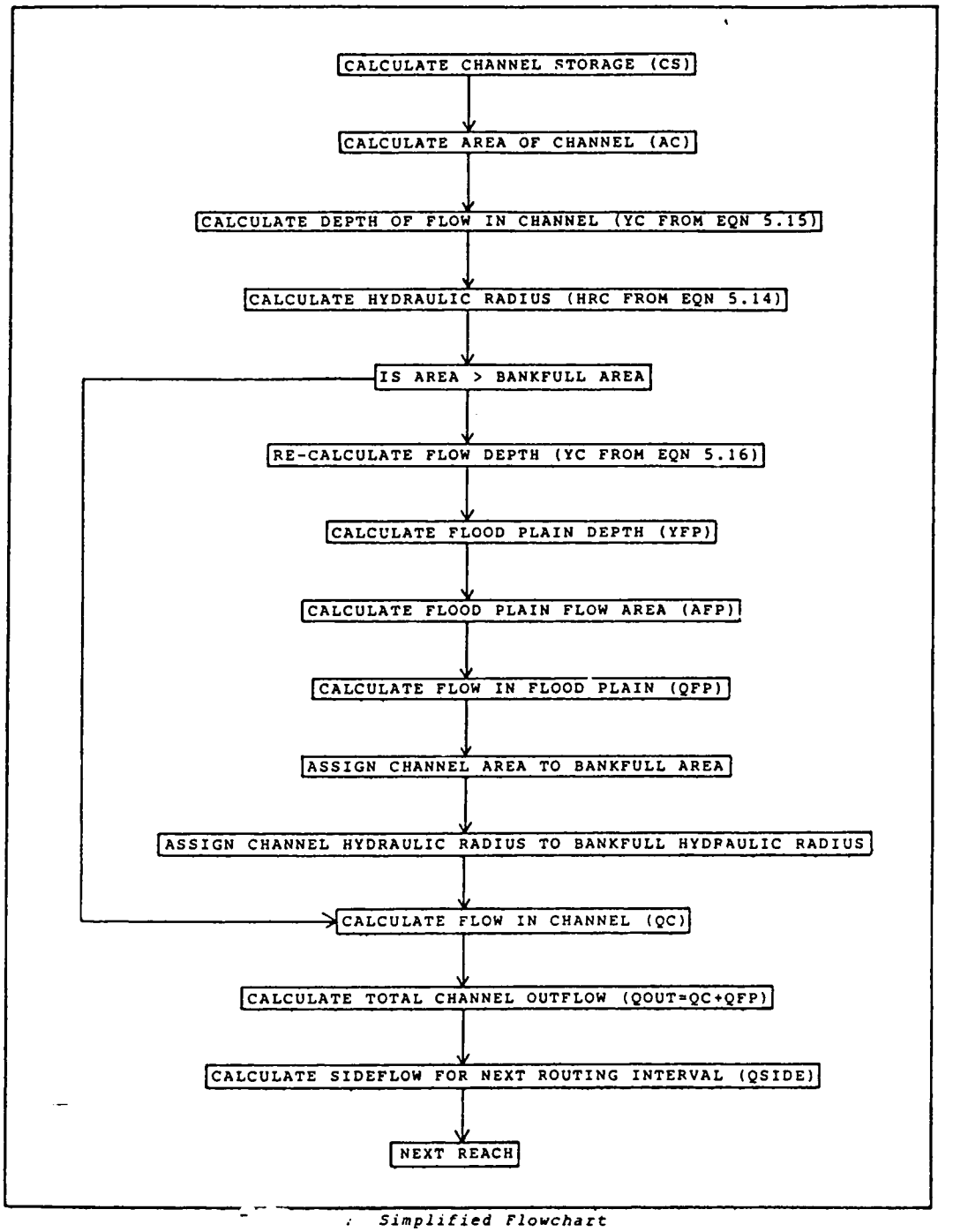
Qs, Qo, and Qdiv are sideflow, outflow and diverted flow respectively. This equation holds for both channels and reservoirs.

8.3 Watershed Model Calibration Technique

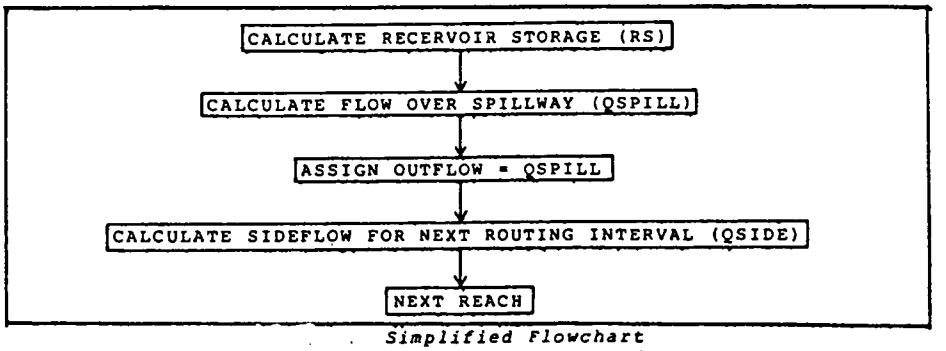
One of the first steps in any optimal water management programme is the establishment of the water or hydrological

FIG8.3

after McKENZIE, 1983



Simplified Flowchart Showing the Channel Flow Routing Processes



Simplified Flowchart Showing the Reservoir Flow Routing Processes

balance of the region, catchment or reservoir under consideration. This assists in assessing the quantity of water available and its real time distribution, thus ensuring efficient allocation of the resource to users. The establishment of this balance in catchment involving the use of the watershed model is best carried out by calibration.

Calibration of the model involves a number of steps whereby some parameters are adjusted continuously to obtain the best fit between the simulated and the recorded response hydrographs of the basin or sub-basin.

One water year period of record is first used to obtain the calibration for each of the sub-basins while the other years are used for verification and validation using the same set of calibrated parameters. Each period of record consists of 12 months of continuous daily precipitation data and daily or monthly potential evaporation data. Streamflow data is daily or hourly. For the Donga basin study, only daily precipitation data was available. The model is able to generate hourly rainfall randomly using an in-built hourly rainfall generator from the daily records for its use (McKenzie, 1983). The available streamflow data is also daily. The steps involved in calibration are discussed in chapter 4 and are well enumerated in the literature and in the appropriate user guide (Fleming, 1986,1975). The next section presents and discusses Donga basin runoff simulation results.

8.4 Watershed Model Calibration Runs and Water Discharge Simulations

The data for the 1976-77 water year was used for the calibration of the catchment to Tepkwar and Abong and the 1959-60 water year data were used for the Donga river to Manya dam calibration. Tables 8.1, 8.2, and 8.3 show the respective calibration parameters obtained. The response hydrograph of these calibration runs are shown in figure 8.4 (a), 8.5 (a), and 8.6 (a) and the summary of the monthly and annual simulation flows is given in tables 8.4 (a), 8.5 (a) and 8.6 (a). Verification of the Tepkwar, Abong, and Manya calibration runs is effected by the use of their respective calibrated parameters in simulating the 1977-78 (Tepkwar and Abong) and 1960-61 (Manya) water years response hydrographs. The respective response hydrograph are presented in figures 8.4 (b), 8.5 (b),and 8.6 (b) for and the respective simulation summaries are given in table 8.4 (b), 8.5 (b), and 8.6 (b). Validation was achieved by the use of the the same calibration parameters for the discharge simulation of the 1981-82 water year for Tepkwar and Abong and the 1962-63 water year for Manya.The respective hydrographs obtained are given in figures 8.4(c), 8.5 (c) and 8.6 (c) while the summaries of the simulations are given in tables 8.4(c), 8.5(c), and 8.6(c).

Discussion

In general, from the comparisons of the hydrograph shapes for each gauge, it is clear that good calibration were achieved in all cases and this is confirmed by the closeness of correlation coefficients, annual and monthly

TABLE 8.1 STDAT 1 DONGA - [illegible] BASIN

CURRENT FINAL CALIBRATION PARAMETERS

SEGMENT 1(UPPER)		SEGMENT 2 (LOWER)	
PARAMETER	VALUE	PARAMETER	VALUE
K1	1.65	K1	1.65
IMPV	0.01	IMPV	0.01
EXPM	2.50	EXPM	3.00
UZSN	36.00	UZSN	48.00
LZSN	600.00	LZSN	600.00
CB	9.00	CB	9.00
CC	1.50	CC	1.50
K3	0.15	K3	0.15
K24L	0.15	K24L	0.15
K24EL	0.00	K24EL	0.00
L	1780.00	L	1645.00
SS	0.75	SS	0.45
NN	0.44	NN	0.44
IRC	0.70	IRC	0.70
KK24	0.99	KK24	0.99
KV	0.25	KV	0.25
AREA	1507.00	AREA	383.00
USZ	180.00	UZS	180.00
LZS	600.00	LZS	580.00
SGW	20.00	SGW	20.00
RES	10.00	RES	10.00
SRGX	0.00	SRGX	0.00
SCEP	0.00	SCEP	0.00
AEPK	0.00	SCEP	0.00
POWER	2.00	POWER	2.00
GR	1.75, 0.20,0.00	GR	1.75,0.20,0.00
	2.00, 0.00,0.00		2.00,0.20,0.00
SD	3.00,0.00,0.00	SD	3.00,0.00,0.00
	3.00,0.00,0.00		3.00,0.00,0.00
	[illegible]		[illegible]
	0.00,1.00,4.00		0.00,1.00,4.00

TABLE 8.2 STDAT 1 DONGA -ABONG BASIN

CURRENT FINAL CALIBRATION PARAMETERS

SEGMENT 1(UPPER)		SEGMENT 2 (LOWER)	
PARAMETER	VALUE	PARAMETER	VALUE
K1	1.66	K1	1.66
IMPV	0.01	IMPV	0.01
EXPM	2.50	EXPM	3.00
UZSN	6.00	UZSN	78.00
LZSN	550.00	LZSN	550.00
CB	9.50	CB	9.00
CC	1.50	CC	1.50
K3	0.01	K3	0.01
K24L	0.01	K24L	0.01
K24EL	0.03	K24EL	0.03
L	1780.00	L	1645.00
SS	0.65	SS	0.45
NN	0.44	NN	0.44
IRC	0.70	IRC	0.70
KK24	0.99	KK24	0.99
KV	0.50	KV	0.50
AREA	4450.00	AREA	1751.00
USZ	250.00	UZS	250.00
LZS	300.00	LZS	300.00
SGW	20.00	SGW	20.00
RES	10.00	RES	10.00
SRGX	0.00	SRGX	0.00
SCEP	0.00	SCEP	0.00
AEPM	0.00	SCEP	0.00
POWER	2.00	POWER	2.00
GR	1.75, 0.20,0.00	GR	1.75,0.20,0.00
	2.00, 0.00,0.00		2.00,0.20,0.00
SD	3.00,0.00,0.00	SD	3.00,0.00,0.00
	3.00,0.00,0.00		3.00,0.00,0.00
	0.00,1.00,4.00		0.00,1.00,4.00

TABLE 8.3 STDAT 1 DONGA -MANYA BASIN

CURRENT FINAL CALIBRATION PARAMETERS

SEGMENT 1(UPPER)		SEGMENT 2 (LOWER)	
PARAMETER	VALUE	PARAMETER	VALUE
K1	1.13	K1	1.13
IMPV	0.01	IMPV	0.01
EXPM	2.50	EXPM	2.50
UZSN	6.00	UZSN	8.00
LZSN	90.0	LZSN	90.00
CB	7.00	CB	7.00
CC	0.90	CC	0.90
K3	0.01	K3	0.01
K24L	0.03	K24L	0.03
K24EL	0.01	K24EL	0.01
L	2000.00	L	2645.00
SS	0.65	SS	0.45
NN	0.44	NN	0.44
IRC	0.70	IRC	0.70
KK24	0.99	KK24	0.99
KV	0.40	KV	0.40
AREA	7450.00	AREA	1751.00
UZS	250.00	UZS	250.00
LZS	700.00	LZS	700.00
SGW	20.00	SGW	20.00
RES	0.00	RES	0.00
SRGX	0.00	SRGX	0.00
SCEP	0.00	SCEP	0.00
AEPK	0.00	SCEP	0.00
POWER	2.00	POWER	2.00
GR	1.75, 0.20,0.00	GR	1.75,0.20,0.00
	2.00, 0.00,0.00		2.00,0.20,0.00
SD	3.00,0.00,0.00	SD	3.00,0.00,0.00
	3.00,0.00,0.00		3.00,0.00,0.00
	0.00,1.00,4.00		0.00,1.00,4.00

	SIMULATED VOLUME		RECORDED VOLUME	
	CUBIC METRES	MM	CUBIC METRES	MM
OCT	3077.11	141.	3371.00	154.
NOV	1758.02	80.	1870.00	85.
DEC	660.54	30.	490.00	22.
JAN	383.01	18.	490.77	22.
FEB	233.44	11.	312.27	14.
MAR	185.22	8.	248.56	11.
APR	308.64	14.	396.91	18.
MAY	1397.56	64.	1013.00	46.
JUN	1267.69	58.	1474.43	67.
JUL	2718.42	124.	2423.88	111.
AUG	2235.54	102.	2309.81	106.
SEP	4690.09	214.	4295.18	196.
ANNUAL	18915.28	864.6	18695.81	854.6

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9889 VARIANCE OF RESIDUAL= 112.5352

EXPLAINED VARIANCE= 97.43 %

TAB8.4c DONGA-GEMBU VAL 81-82

	SIMULATED VOLUME		RECORDED VOLUME	
	CUBIC METRES	MM	CUBIC METRES	MM
OCT	2949.30	135.	2838.70	130.
NOV	1218.77	56.	1010.10	46.
DEC	488.49	22.	505.60	23.
JAN	515.73	24.	446.30	20.
FEB	399.64	18.	408.20	19.
MAR	493.60	23.	706.40	32.
APR	534.05	24.	759.20	35.
MAY	890.33	41.	1279.10	58.
JUN	1387.33	63.	1677.00	77.
JUL	4190.69	192.	3198.60	146.
AUG	3819.81	175.	3468.50	159.
SEP	3841.16	176.	4054.70	185.
ANNUAL	20728.89	947.5	20352.40	930.3

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9780 VARIANCE OF RESIDUAL= 291.2668

EXPLAINED VARIANCE= 93.35 %

TAB 8.4a DONGA-GEMBU CALIB 76-77

	SIMULATED VOLUME		RECORDED VOLUME	
	CUBIC METRES	MM	CUBIC METRES	MM
OCT	2828.63	129.	2685.60	123.
NOV	1003.80	46.	849.30	39.
DEC	523.53	24.	579.10	26.
JAN	322.25	15.	427.70	20.
FEB	201.10	9.	304.38	14.
MAR	161.39	7.	416.89	19.
APR	317.24	15.	1067.43	49.
MAY	1258.46	58.	1003.00	46.
JUN	1227.61	56.	1580.92	72.
JUL	2260.02	103.	2423.88	111.
AUG	1972.11	90.	2309.81	106.
SEP	3839.54	176.	4295.18	196.
ANNUAL	15915.68	727.5	17943.19	820.2

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9771 VARIANCE OF RESIDUAL= 169.6758

EXPLAINED VARIANCE= 95.29 %

TAB 8.4b DONGA-GEMBU VERF' 77-78

	SIMULATED VOLUME		RECORDED VOLUME	
	CUBIC METRES	MM	CUBIC METRES	MM
OCT	13195.03	184.	13340.00	186.
NOV	5409.75	75.	5810.00	81.
DEC	1706.98	24.	1286.00	18.
JAN	675.63	9.	699.00	10.
FEB	304.98	4.	302.50	4.
MAR	194.09	3.	350.00	5.
APR	403.71	6.	950.00	13.
MAY	1816.77	25.	3075.50	43.
JUN	4979.22	69.	4206.00	59.
JUL	10217.45	142.	7717.00	108.
AUG	6838.53	95.	7538.00	105.
SEP	16798.39	234.	17958.00	250.
ANNUAL	62540.54	871.3	63232.00	880.9

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9865 VARIANCE OF RESIDUAL= 192.5158

EXPLAINED VARIANCE= 97.08 %

TAB 8.5a DONGA-ABONG CALIB

	SIMULATED VOLUME		RECORDED VOLUME	
	CUBIC METRES	MM	CUBIC METRES	MM
OCT	7896.79	110.	7787.00	108.
NOV	1992.94	28.	2551.50	36.
DEC	1493.56	21.	1738.00	24.
JAN	1088.08	15.	1281.00	18.
FEB	728.67	10.	914.50	13.
MAR	1045.37	15.	1253.00	17.
APR	2719.90	38.	3205.50	45.
MAY	4790.84	67.	2995.50	42.
JUN	4552.05	63.	6262.00	87.
JUL	8362.06	116.	8347.80	116.
AUG	6117.47	85.	8085.00	113.
SEP	12104.38	169.	7649.00	107.
ANNUAL	52892.11	736.9	52069.80	725.4

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9119 VARIANCE OF RESIDUAL= 539.0107

EXPLAINED VARIANCE= 76.94 %

TAB8.5b DONA-ABONG VERIF 77-78

	SIMULATED VOLUME		RECORDED VOLUME	
	CUBIC METRES	MM	CUBIC METRES	MM
OCT	4751.98	66.	8486.40	118.
NOV	2791.16	39.	2968.50	41.
DEC	1112.24	15.	1493.60	21.
JAN	977.09	14.	1312.30	18.
FEB	984.33	14.	1202.20	17.
MAR	1260.93	18.	2095.00	29.
APR	1659.37	23.	2247.20	31.
MAY	2903.65	40.	3753.10	52.
JUN	4705.74	66.	4997.00	70.
JUL	11060.59	154.	9323.60	130.
AUG	13783.07	192.	10034.50	140.
SEP	12960.17	181.	12655.70	176.
ANNUAL	58950.34	821.3	60569.10	843.8

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9398 VARIANCE OF RESIDUAL= 583.5884

EXPLAINED VARIANCE= 84.10 %

FIG8.5c DONGA-ABONG VAL 81-82

	CUBIC METRES	MM	CUBIC METRES	MM
OCT	21935.14	206.	17945.60	168.
NOV	4276.89	40.	4623.40	43.
DEC	1586.23	15.	1859.70	17.
JAN	888.14	8.	971.80	9.
FEB	526.07	5.	538.50	5.
MAR	2685.37	25.	1701.30	16.
APR	2013.60	19.	2572.30	24.
MAY	2468.35	23.	3524.60	33.
JUN	10325.59	97.	8246.00	77.
JUL	14582.08	137.	12803.30	120.
AUG	14043.36	132.	19359.46	182.
SEP	30286.69	284.	27762.78	261.
ANNUAL	105617.51	991.7	101908.15	956.9

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9748 VARIANCE OF RESIDUAL= 476.8091

EXPLAINED VARIANCE= 93.52 %

TAB 8.6a DONGA - MANYA CALIB 59-60

	SIMULATED VOLUME		RECORDED VOLUME	
	CUBIC METRES	MM	CUBIC METRES	MM
OCT	22361.58	210.	20937.66	197.
NOV	9350.26	88.	9263.90	87.
DEC	3245.44	30.	3330.10	31.
JAN	2198.12	21.	1696.00	16.
FEB	1188.37	11.	1021.10	10.
MAR	790.04	7.	560.80	5.
APR	1371.90	13.	1188.90	11.
MAY	2105.33	20.	1280.70	12.
JUN	6210.83	58.	3844.60	36.
JUL	18542.19	174.	16583.20	156.
AUG	9443.03	89.	11498.10	108.
SEP	11129.74	105.	17178.50	161.
ANNUAL	87936.82	825.7	88383.56	829.9

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9637 VARIANCE OF RESIDUAL= 427.3317

EXPLAINED VARIANCE= 92.34 %

TAB 8.6 (b) DONGA-MANYA VERIF 60-61

	SIMULATED VOLUME		RECORDED VOLUME	
	CUBIC METRES	MM	CUBIC METRES	MM
OCT	22655.77	213.	17945.00	168.
NOV	4294.09	40.	4623.40	43.
DEC	1617.89	15.	1859.70	17.
JAN	892.60	8.	971.80	9.
FEB	524.75	5.	538.50	5.
MAR	2628.08	25.	1701.30	16.
APR	1985.81	19.	2572.30	24.
MAY	2421.92	23.	3524.60	33.
JUN	10308.56	97.	8246.00	77.
JUL	14532.79	136.	12803.30	120.
AUG	16884.06	159.	19359.46	182.
SEP	30837.19	290.	27762.78	261.
ANNUAL	109583.50	1028.9	101908.15	956.9

CORRELATION COEFFICIENT (MONTHLY VOLUME)=0.9864 VARIANCE OF RESIDUAL= 342.2167

EXPLAINED VARIANCE= 95.35 %

TAB8.6c DONGA-MANYA VAL 61-62

flow volumes between simulated and recorded information (tables 8.1-8.6). Other statistical parameters obtained such as the variance of residual and explained variance are satisfactory.

8.5 Testing the Impact of Landuse Changes in the Short Term

(1) IMPV

The sensitivity of runoff to changes in the impervious and urbanisation parameter, IMPV, is investigated by simulating the runoff responses when the value is changed from its original (0.01) to 0.10 and 0.00. The hydrographs of runoff response obtained are included in the figures 8.7 (a) respectively and their respective simulation result values are provided in summary form in tables 8.7.

From the table 8.7 (a) and (b) and figures 8.7 (a), , it is clear that the substantial increase in the IMPV values affect runoff volumes and peaks significantly in all the simulations. A change of IMPV from 0.01 to 0.10 increased annual runoffs by as much as 18% and peaks by as much as 30-100%. Also, the number of 'short' runoff peaks increased considerably. A slight decrease in IMPV from 0.01 to 0.00 reduced the runoff volumes marginally by about 2% (2-5)% in all the three water years. A decrease in IMPV also decreases peaks marginally. Simulating the decrease in IMPV is only a theoretical exercise as this is unlikely to occur in real life.

TABLE 8.7 (a) Total Simulated Annual Runoff by Landuse Parameter and by Year (in Cumecs or Cubic Metres*86400)

Parameter	INITIAL	EPXM		IMPV		DIST	
Value		.8/1	7.5/9	0.0	0.1	.09	.11
Year							
1976-77	**62,868**	**63,496**	**62,019**	**61,850**	**72,059**	**62,868**	
1977-78	52,892	53,830	52,270	51,858	62,201	52,892	
1981-82	58,950	62,720	55,108	57,825	69,100	58,950	

Initial Values EPXM=2.5/3.0; IMPV=0.01; and DIST=0.100

TABLE 8.7 (b) Total Simulated Annual Runoff by Percentage Increase over Initial Conditions by Landuse Parameter and by Year

Parameter	**INITIAL**	**EPXM**		**IMPV**		**DIST**	
Value		**.8/1**	**7.5/9**	**0.0**	**0.1**	**.09**	**.11**
Year							
1976-77	**0%**	**+2%**	**-1%**	**-1%**	**+16%**	**0%**	**0%**
1977-78	0%	+1%	-1%	-2%	+18%	0%	0%
1981-82	0%	+5%	-5%	-2%	+16%	0%	0%

Initial Parameter values as above.

Discussion

IMPV is currently relatively small due to the rural and virgin nature of the catchment. It is evident from the above simulations that only a substantial increase in urbanisation will affect the catchment runoff volumes or peaks. The experience of the present settlers in the area or government plans for the catchment currently indicate that such an increase is improbable in the short term. This lowers the practical significance of testing IMPV.

(2) EPXM

The impact of changes in the vegetation interception parameter, EPXM, (from its original values of 2.5/3.0 (two segments) to 7.5/9.0 (i.e x 3) and 0.8/1.0 (x 0.3)) on runoff were investigated from the hydrographs and flow values produced. These response hydrographs are respectively shown in figures 8.7 (b) while the results summary are presented in tables 8.7.

The stated increase in EPXM reduced annual runoff by 0-5% while high peaks are also substantially increased by between 28-36%. Conversely, the stated trial decrease did increase annual runoff marginally by 5% or less and peaks were not much affected. Peaks were increased or reduced only by about 5 % or less.

Discussion

From the hydrographs and tables, it is evident that the runoff sensitivity to changes in vegetation due to changes

TABLE 8.7 (c) Total Simulated Percentage Change in Runoff Peaks by Year and by Landuse Parameter. The Total number of Peaks Generated are given in Brackets.

WATER YEAR		1976-1977			1977-1978			1981-1982		
		Peaks			Peaks			Peaks		
Parameters		High	Med	Low	High	Med	Low	High	Med	Low
INITIAL		(2)	(1)	(5)	-	-	(6)	(1)	(4)	(2)
IMPV	0.10	+50	+100	+100	-	+17-100	+2	+20-30	30-50	100
		(4)	(6)	(6)	-	(16)	(10)	(11)	(11)	(8)
IMPV	0.00	-5	+5	-5	-	-	+1	-1	+1	-
		(2)	(1)	(5)	-	-	(6)	(1)	(4)	(2)
EPXM	7.5/9	+28	+14	+10-36	-	-	+5-28	-	+5-10	-
		(2)	(1)	(5)	-	-	(6)	(1)	(7)	(2)
EPXM	0.8/1	-1	-1	-5	-	-	-5	-	-	-5
		(2)	(1)	(5)	-	-	(6)	(1)	(7)	(2)

The Initial Values of EPXM=2.5/3 and IMPV=0.01. K3 was tested and found to be completely insensitive. Note that High Peaks are assumed to be greater than 800 cumecs, Medium Peaks lie between 500 and 800 cumecs and Low Peaks are less than 500 cumecs.

in land use activities such as afforestation deforestation, shifting cultivation, animal husbandary,and organised agriculture is profound when the vegetation parameter is substantially increased. Substantial decreases did affect annual runoffs or peaks sgnificantly. Marked changes in this parameter in the short term are not likely to occur, so the practical significance of testing these changes are again dampened.

(3) K3 and DIST

The K3 parameter was tested on the Donga Basin and found to be relatively insensitive for small short term changes. DIST , the land disturbance factor, is not a watershed model parameter and hence cannot affect the hydrograhs simulated by the watershed model.

8.6 Testing the Impact of Reservoir-Induced Climate Changes

In chapter one, the influence of large impoundments of water on a catchment climate was outlined from the literature. The magnitude of this impact has been deduced from McDonald's (1962) work in Arizona, and presented in detail in chapter 10. For the Donga Basin, an 8.6% increase in precipitation is expected due to this effect. In testing and assessing the climatic influence of the multiple reservoirs planned for the basin, values in the range of 5, 10, 11.6% were used in trial simulations and the runoff hydrographs obtained are provided in figures 8.8

Table 8.8 (a) summarises annual runoff volumes obtained while Table 8.8 (b) and 8.8 (c) provide the corresponding percentage increases in annual runoffs and peaks respectively. The 11.6% increase in rainfall utilised is equivalent to the quantity of water lost by the numerous reservoirs to evapotranspiration as evaluated by the routing model. This is equated to the maximum rainfall increase expected in the catchment, hence its use in the simulation for assessing the worst scenario of change (see chap 1). It is important to note that lake-induced evapotranspiration increases here are confined to the reservoir areas and their immediate vicinities and not the whole basin. Thus, evapotranspiration involving the whole basin was not included in the runoff simulations carried out in this section.

Discussion

Figures 8.8 and table 8.8 show clearly that the impact of reservoir construction on catchment runoff response will be significant. The results indicate a marked increase in annual runoff volumes ranging from 13 to 33%. Runoff peaks also substantially increased generally ranging from 5 to over 50% for all water years simulated. The results indicate that the impact of the reservoirs on the catchment runoff will be significant after construction. These results indicate the possibility of a severe flood problem in the catchment in the years following the construction of storage projects. This high probability of unusually high flood peaks will be accompanied by massive erosion,

TABLE 8.8 (a) Total Annual Simulated Runoff by Impoundment-Induced Precipitation Increase. Annual/Short term Scenarios

Values are in Cumecs or in Cubic Metres*86400. The percentage increase in Runoff are in brackets.

Water Year Precipitation	1976-1977	1977-1978	1981-1982	Adj %
+0%	**62,868 (0)**	**52,892 (0)**	**58,958 (0)**	
+5%	**71,234 (12)**	**59,602 (12)**	**66,310 (12)**	(+)
+10%	82,786 (25)	67,387 (23)	73,877 (21)	(+5)
+11.6	82,922 (26)	70,112 (26)	76,757 (24)	(+6)

TABLE 8.8 (b) Simulated Runoff Peak Increases by Percentage and by Year The Values in Brackets are the number of Peaks for Various Types Generated.

Water Year	**'976-1977**			**1977-1978**			**1981-1982**		
Precipitation	Peaks			Peaks			Peaks		
	High	**Med**	**Low**	**High**	**Med**	**Low**	**High**	**Med**	**Low**
+0%	0	0	0	0	0	0	0	0	0
	(3)	-	-	-	(3)	(1)	(4)	(3)	(2)
+5%	0-30	0	5-10	12-15	-	30	14-20	20	12
	(3)	**(2)**	**(2)**	**(3)**	**-**	**(1)**	**(4)**	**(3)**	**(2)**
+10-11.6%	**20-100**	**30**	**40-50**	**20-30**	**50**	**-**	**30**	**25**	**13**
	(3)	**(2)**	**(2)**	**(4)**	**(1)**	**-**	**(4)**	**(3)**	**(2)**

Note that High Peaks are greater than 800 cumecs. Medium Peaks lie between 500 and 800 cumecs. Low Peaks are less than 500 cumecs.

especially in a catchment already suffering from unacceptable erosion levels (plates in chap 3 and chap 9). Measures are therefore required in the design of schemes or otherwise to avert possible disasters, both human and economic, from the floods in the future.

8.7 Testing the Impact of Future Global and Reservoir-Induced Climate Change

The River Basin Management System was also utilised to assess the sensitivity of runoff volume and distribution to changes in climate parameters like precipitation and evaporation due to the greenhouse effect. The watershed model component of the system provided the required responses on testing. As evaluated in chapter 7, the stated magnitude of precipitation and evapotranspiration changes from various scenarios of global warming were tested.

Tests involving 14%, 16.5%, 20% and 24% increases in precipitation were carried out to represent mildest, mild, severe and worst scenarios of global warming. Evapotranspiration increases of 2.5%, 5.0%, 7.5%, 10.0%, and 12.5% representing the range of the mildest to the worst scenarios of global warming were combined with the precipitation changes for the analysis. The simulated hydrographs and annual runoff values are presented analysed, and discussed herein. These are presented in figures 8.9 - 8.12 and in tables 8.9.

TABLE 8.9 Simulated Annual Runoff for Various Scenarios of Global Warming. Long Term (50 year) Runs Values in Cumecs or Cubic metres*86400. Note that percent increases in Runoff are in Brackets.

EVAPTRN + PRECIPITATION+	0%	2.5%	5.0%	7.5%	10.0%	12.5%	ADJ
0%	62,868						
14.0%		**85,275**	**83,319**	**81,375**	**79,431**	**77,469**	
		(27)	(25)	(23)	(21)	(19)	(x1·3)
16.5%		89,589	87,625	85,678	83,724	81,773	
		(30)	(28)	(26.5)	(25)	(23.5)	(x1·4)
20.0%		97,160	95,192	93,233	91,270	89,315	
		(36)	(34)	(32)	(31)	(30)	(x1·5)
24.0%		**103,722**	**101,746**	**99,764**	**97,783**	**95,813**	
		(39)	**(38)**	**(37.5)**	**(36)**	**(34.5)**	(x1·6)

Discussion

From the table 8.9 (a), the annual runoff volume increases range from 2% to 65% depending on the scenario of global warming simulated. It is also observed that runoff reduces with increases in evapotranspiration but increases with rises in precipitation. For a 10% drop in evapotranspiration, annual runoff reductions of 12 to 8% are observed. Runoff peaks increase sharply with increases in precipitation but gradual drops occur as evapotranspiration is increased. Runoff peaks increase from a range of 5-70% to 60-240% depending on the scenario of global warming. The results above indicate that a future global warming, mild or severe will have a significant impact on the annual runoff volumes and runoff peaks in the Donga basin. Runoff peaks will be particularly affected possibly because of the highly mountainous nature of the basin from source to Abong town. The characteristic high slopes aid runoff peaking. Again the practical significance of the results of these tests is high probality (or even certainty) of extreme floods frequently occurring in the catchment. The resultant erosion is likely to be of the 'runaway' scale. Irrespective of the scenario of global warming that may occur, designs and management of water schemes must be directed now at the problem of extreme floods and erosion predicted.

8.8 Discussion, Final Remarks, and Conclusions

From the nature of the models and methodology adopted herein, it is clear that all processes involving the interaction of water in a dynamic catchment are incorporated in the simulations and the results obtained are therefore realistic. The analyses and discussions of the results indicate that significant changes in daily, monthly and annual runoff volumes will occur in the Donga Basin when changes in climate and land use patterns take place in both the short and long term.

From predictions, there appears to be a high possibility of severe to extreme floods resulting from proposed surface water schemes and the global shift in climate. Huge top soil loss from erosion are expected as will be shown in chapter 9. Moreover, reservoir schemes, if improperly designed, are likely to fail physically from flood events or economically from unacceptable sedimentation rates. It is important to note that if this analysis had not been carried out, the future problem likely to occur in the Donga Basin would have been unknown and the proposed schemes would have been built on a wrong foundation of assumptions with disastrous consequences.

The implications of this for a world with rapidly changing climate and land use patterns are far reaching. This presents a formidable challenge to engineers involved in the design and operation of water resource schemes.

The results confirm that the practice of the use of present and past hydrological data in designing water resource schemes needs review to ensure their long term performance in a continually changing environment. If past data are to be utilised in design, then the impact of catchment changes must be accomodated by accurate projection and testing. In this regard, engineers may well assess the changing conditions that are likely to affect a project and adjust runoff hydrographs accordingly by computer simulation methods or otherwise before applying the traditional design methods. The simulation approach adopted in this research has clearly shown the vulnerability of the Donga Basin to short and long term process changes in hydrology and landuse. The approach is recommended. The realibility of the results are ensured and assuring as both the system and the approach generally uses mathematical functions that are process and dynamically based to assess future responses heuristically. In continuation of the analysis and discussion of the results obtained by the application of the River Basin Management System on the Donga Basin, the effect of the runoff simulations generated and discussed above on catchment sediment yields and reservoir sedimentation are presented in chapter 9.

FIGURES 8.4 - 8.6 CALIBRATION, VERIFICATION, AND VALIDATION FOR DONGA BASIN WATER BALANCE STUDY

FIgures 8.4 Donga Basin to Tepkwar/Gembu (1976-78, 1981-82)

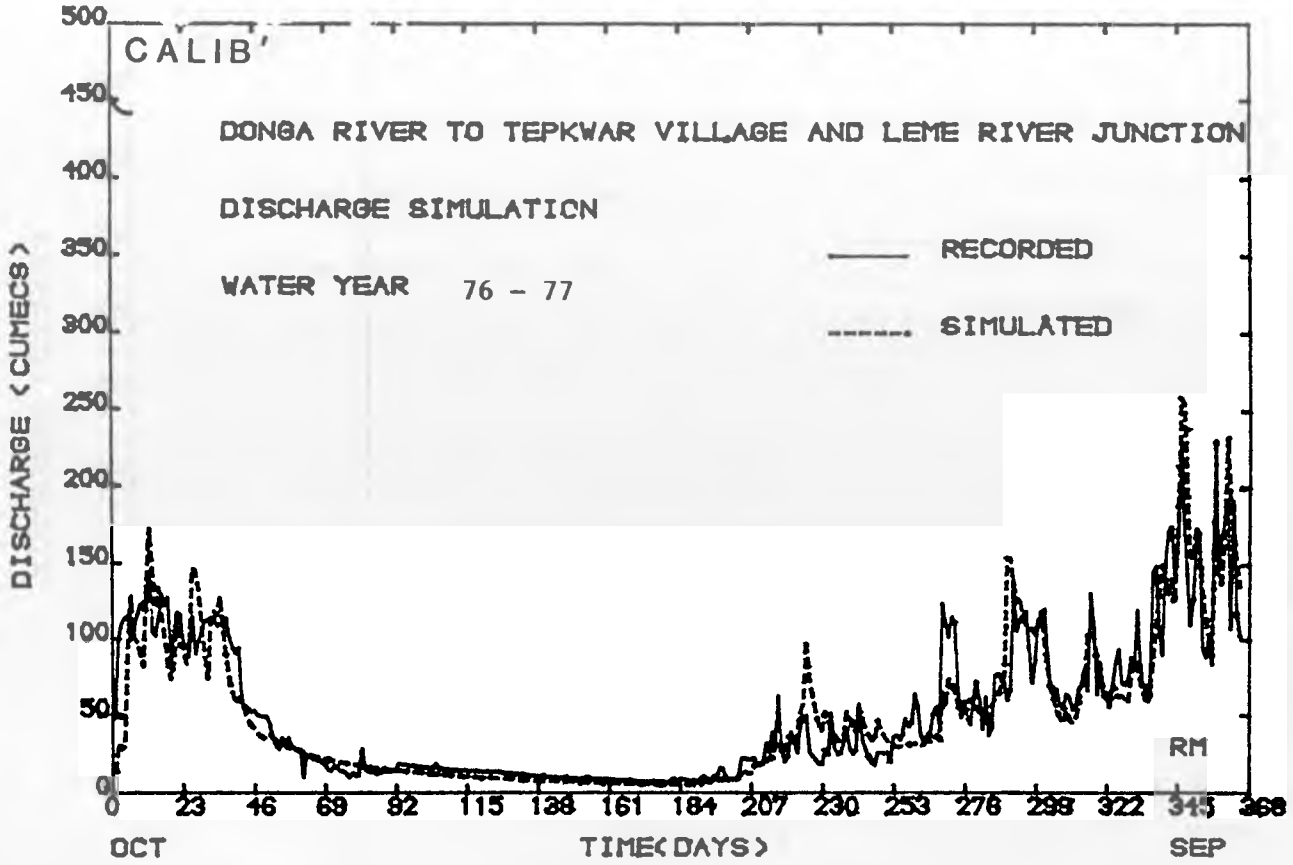
Figures 8.5 Donga Basin to Abong Town (1976-78, 1981-82)

Figures 8.6 Donga Basin to Manya (1959-62)

USER: CNAV61 PLOTFILE: PFILE ENTRY NO. 88861 QUEUED ON 22/07/88 AT 14:24 PLOTTED ON 22/07/88 AT 14:27

Fig. 8.4 (a)

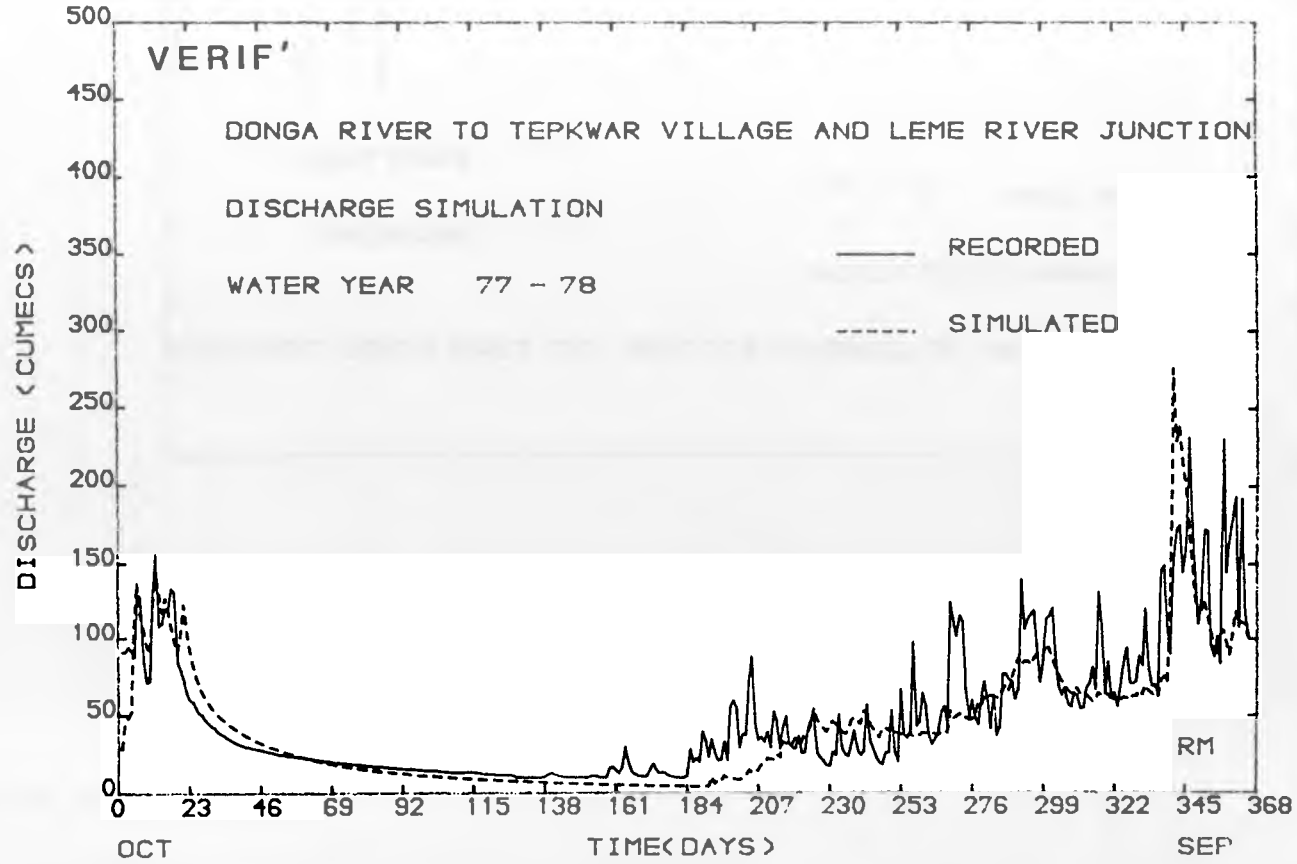
CALIBRATION



6.9 (a)

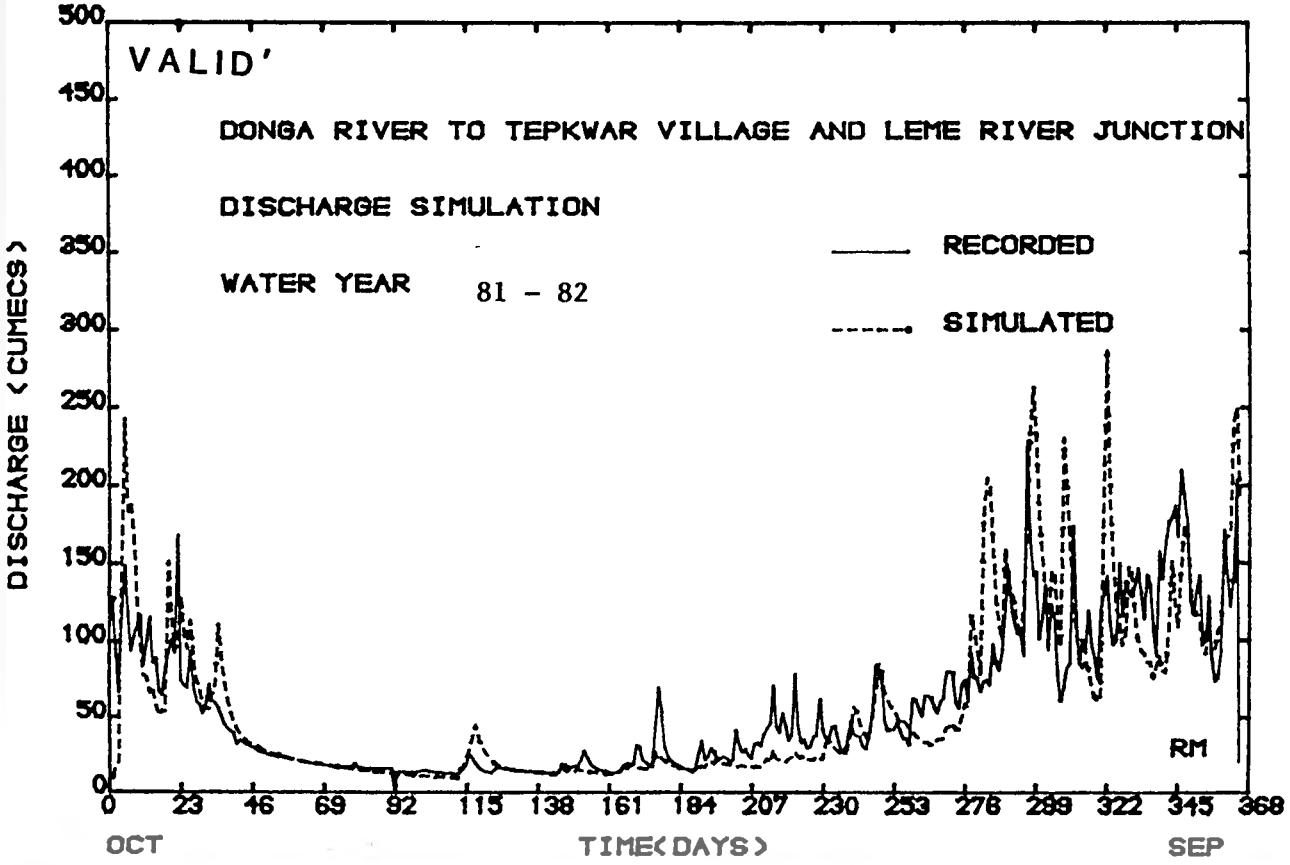
USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 81761 QUEUED ON 02/12/88 AT 20:11 PLOTTED ON 02/12/88 AT 21:09

Fig. 8.4 (b)



USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 89264 QUEUED ON 19/07/88 AT 17:19 PLOTTED ON 19/07/88 AT 17:20

Fig. 8.4 (c)



USER: CMAV81 PLOTFILE: PFILE ENTRY NO. 69770 QUEUED ON 14/09/88 AT 21:25 PLOTTED ON 20/09/88 AT 12:10

FINAL CALIBRATION

Fig. 8.5 (a)

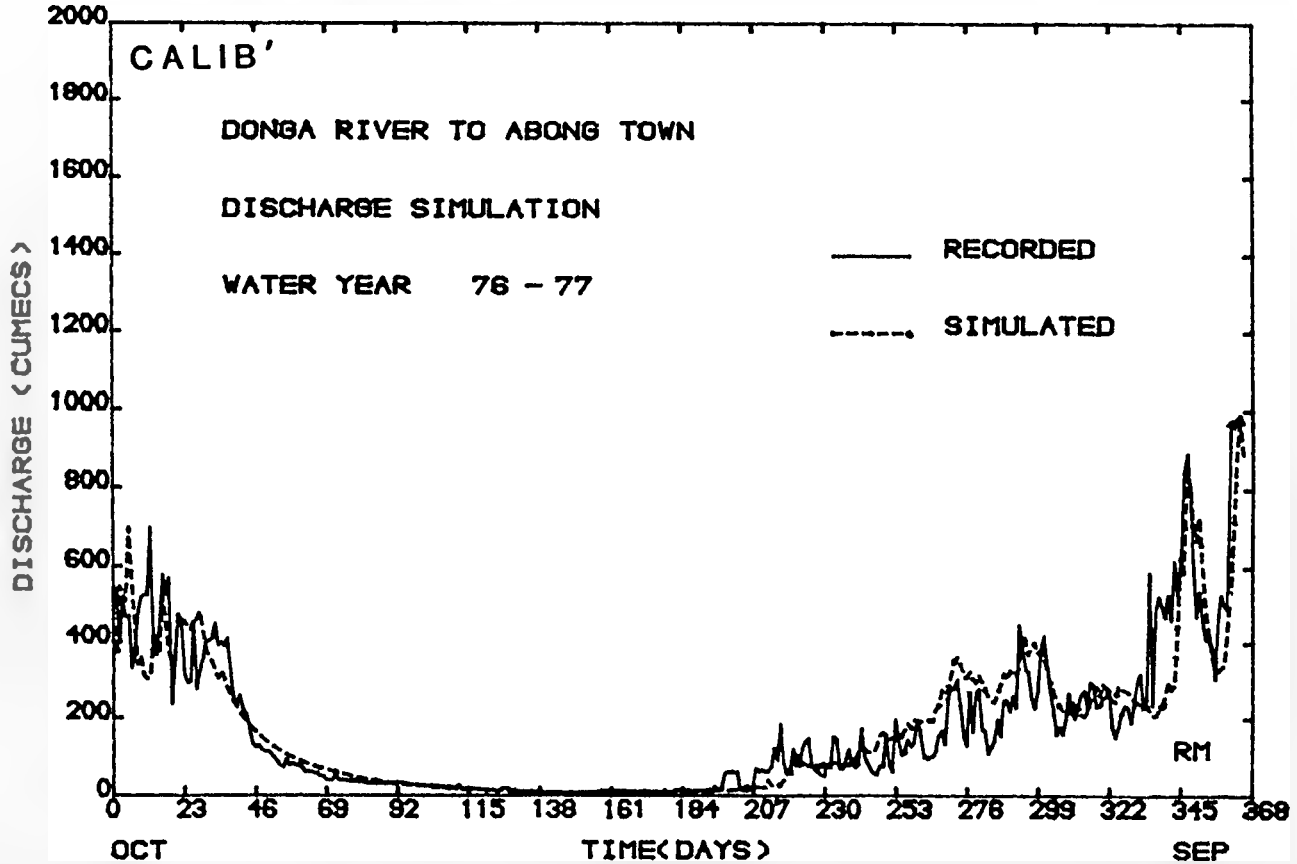
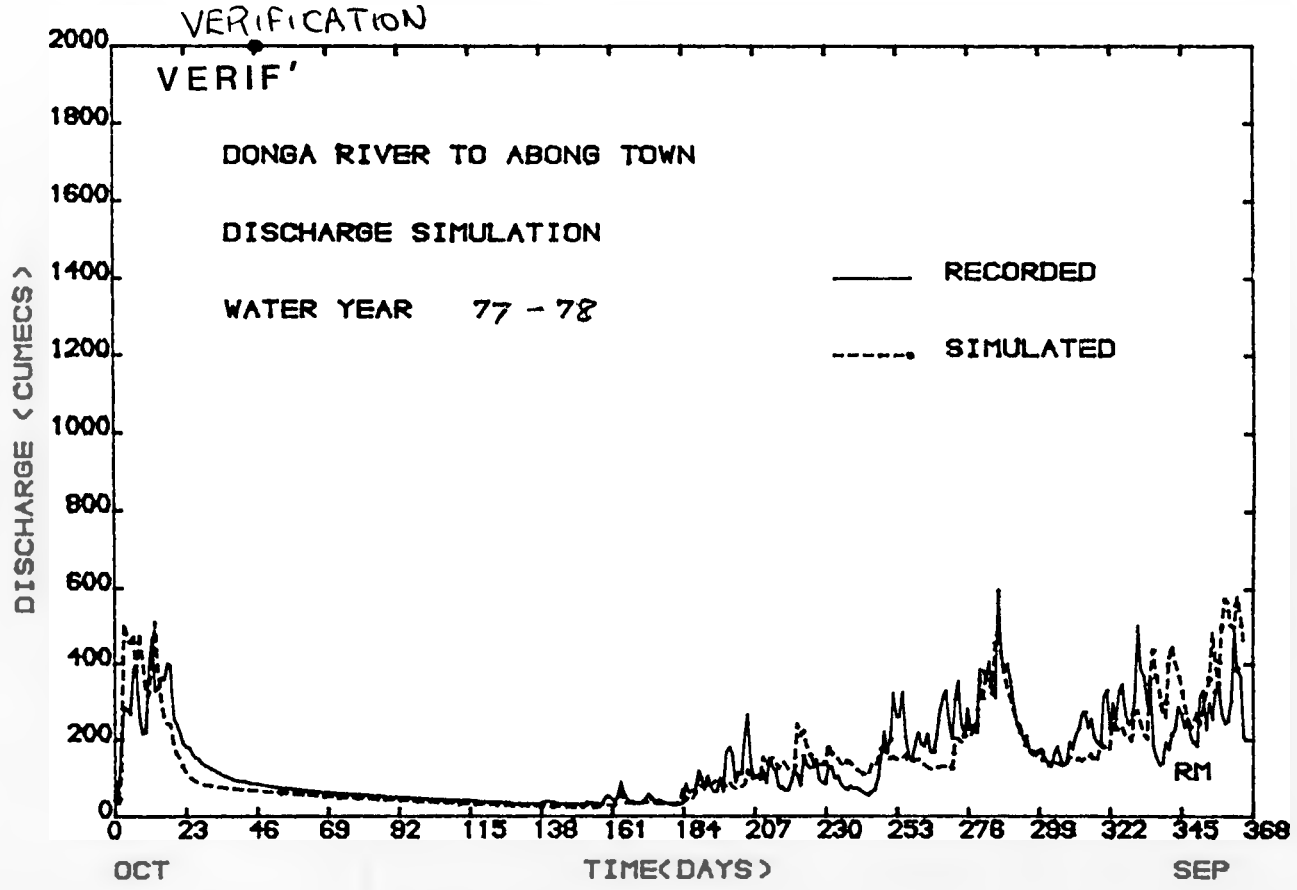


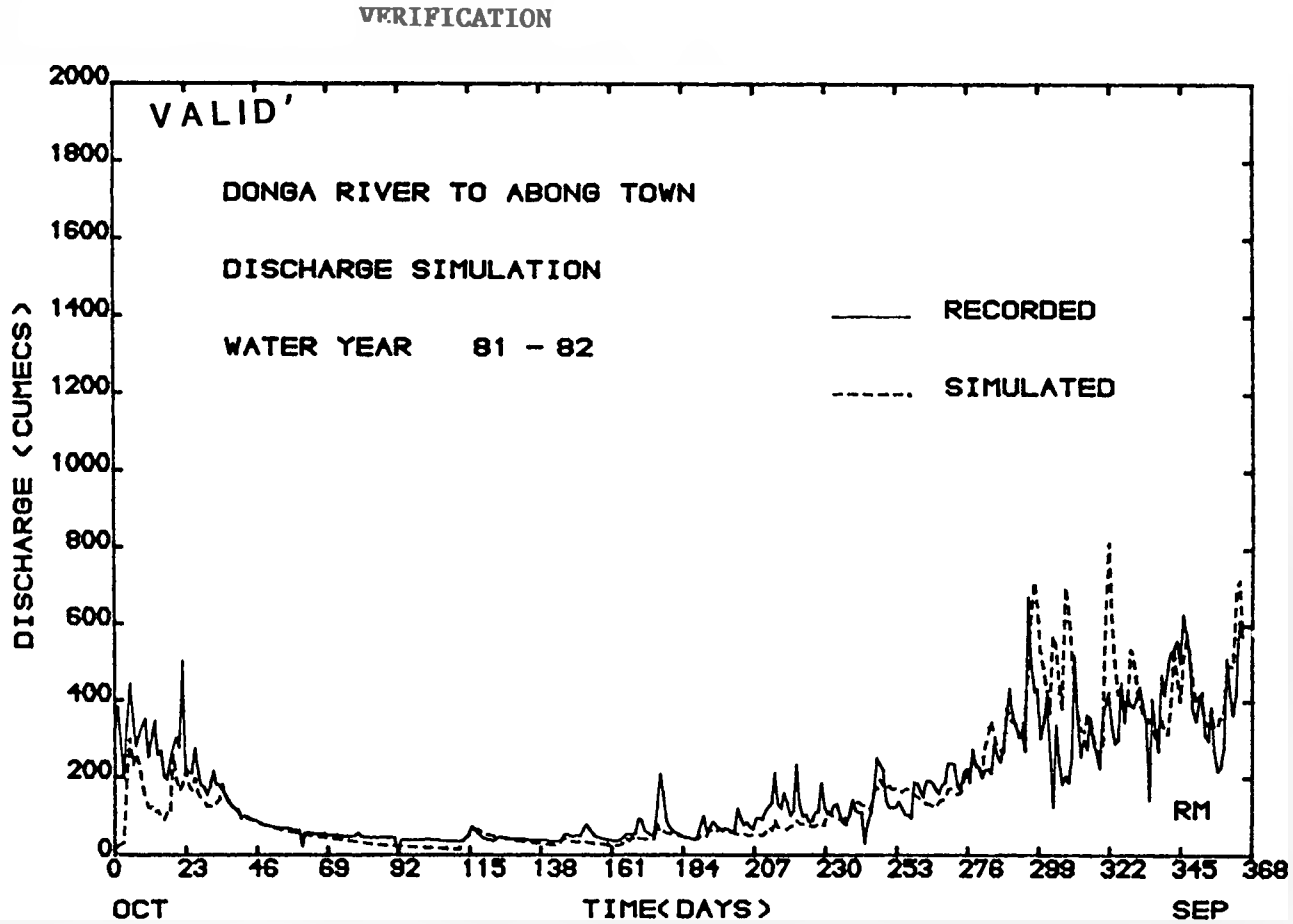
Fig. 8.5 (b)

VERIFICATION



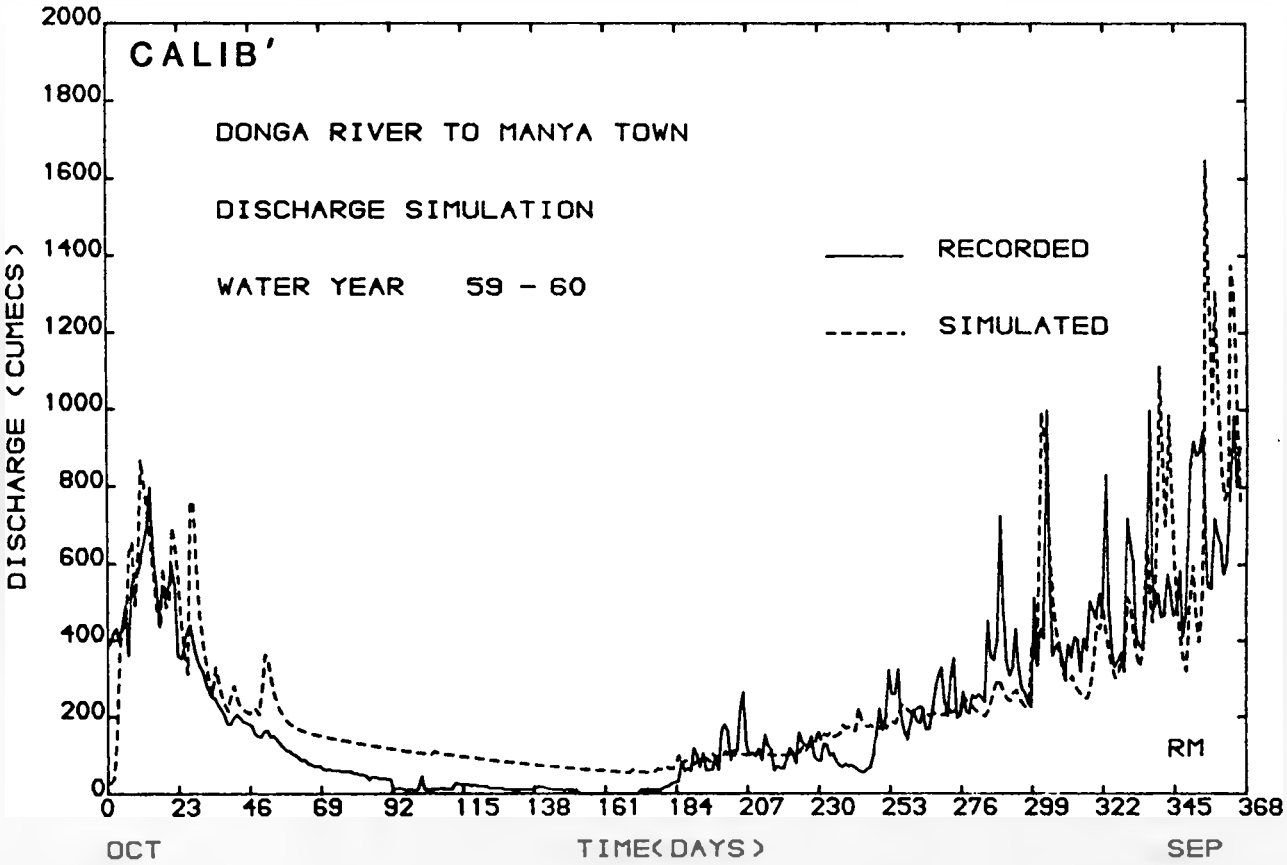
USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 69265 QUEUED ON 19/07/88 AT 17:26 PLOTTED ON 19/07/88 AT 17:28

Fig. 8.5 (c)



USER: CWAV61 PLOTFILE: PFILE ENTRY NO. 81766 QUEUED ON 02/12/88 AT 20:35 PLOTTED ON 02/12/88 AT 21:48

Fig. 8.6 (a)



USER: CHAR19 PLOTFILE: PFILE ENTRY NO. 84895 QUEUED ON 22/12/88 AT 10:51 PLOTTED ON 22/12/88 AT 11:22

Fig. 8.6 (b)

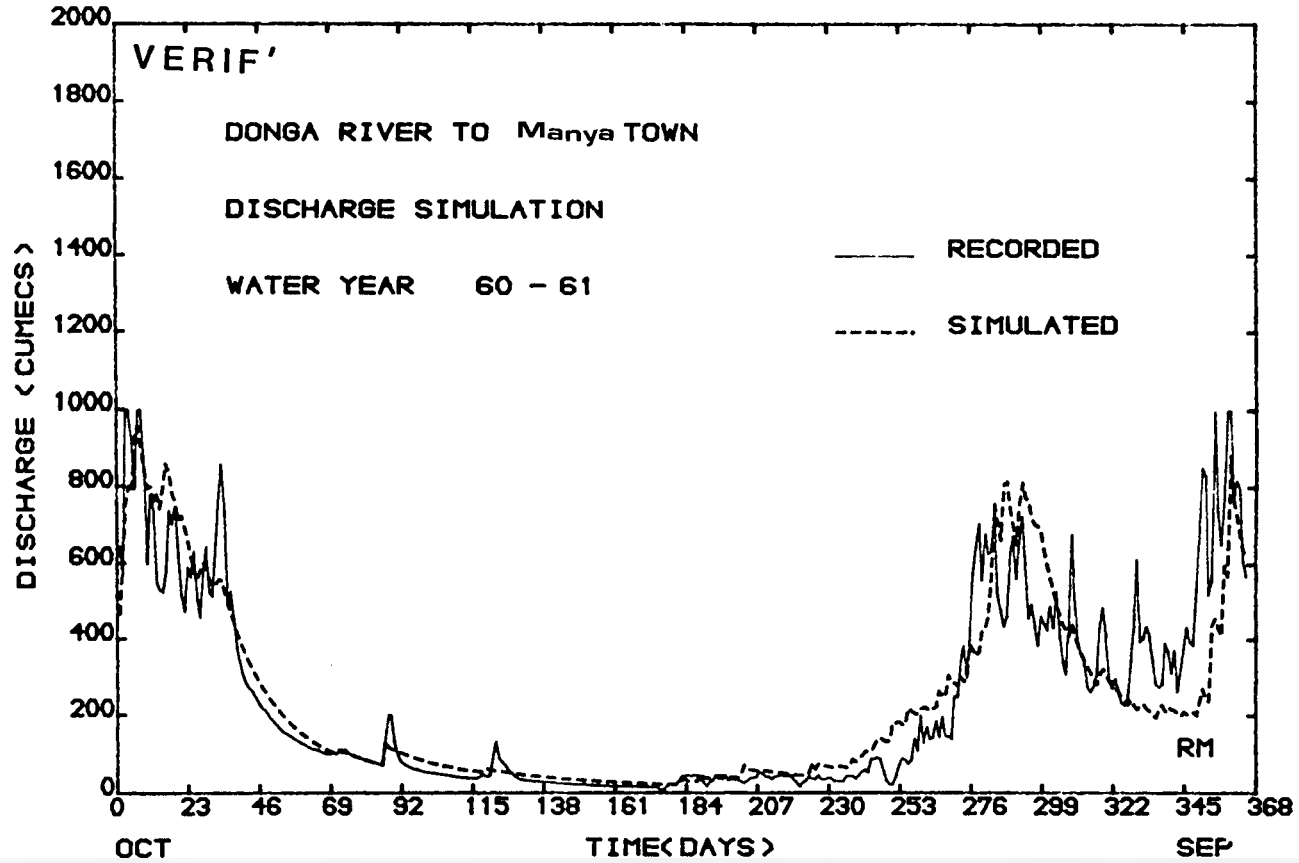
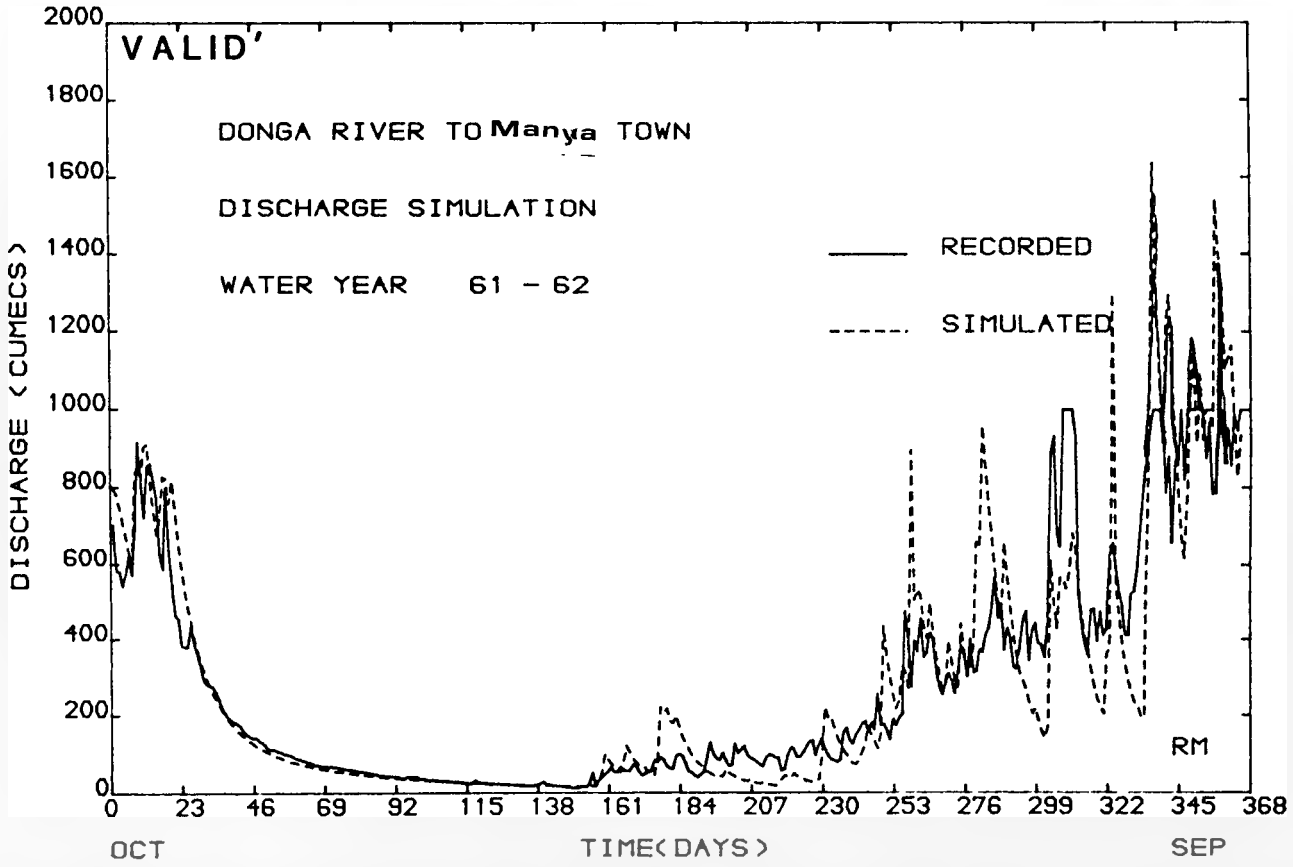


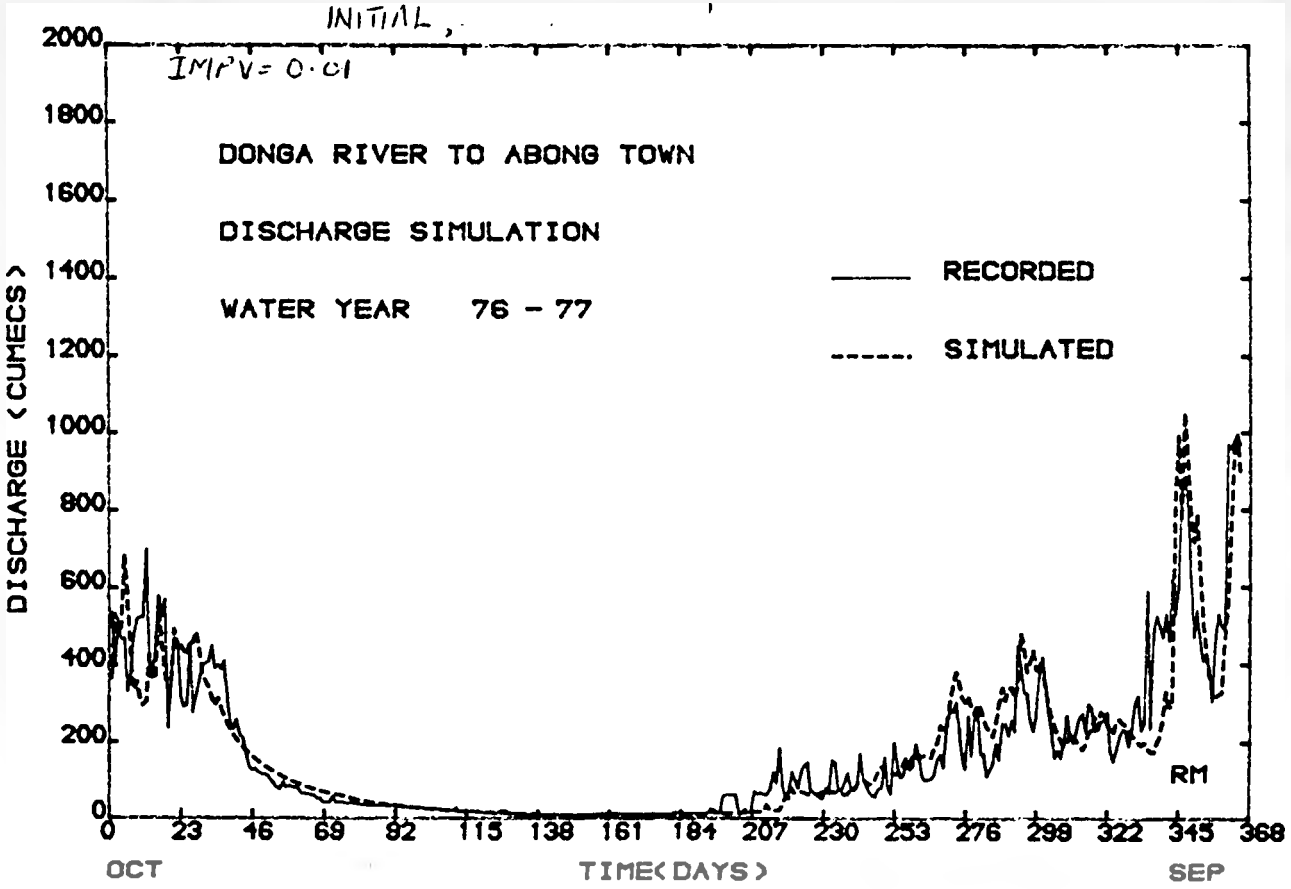
Fig. 8.6 (c)

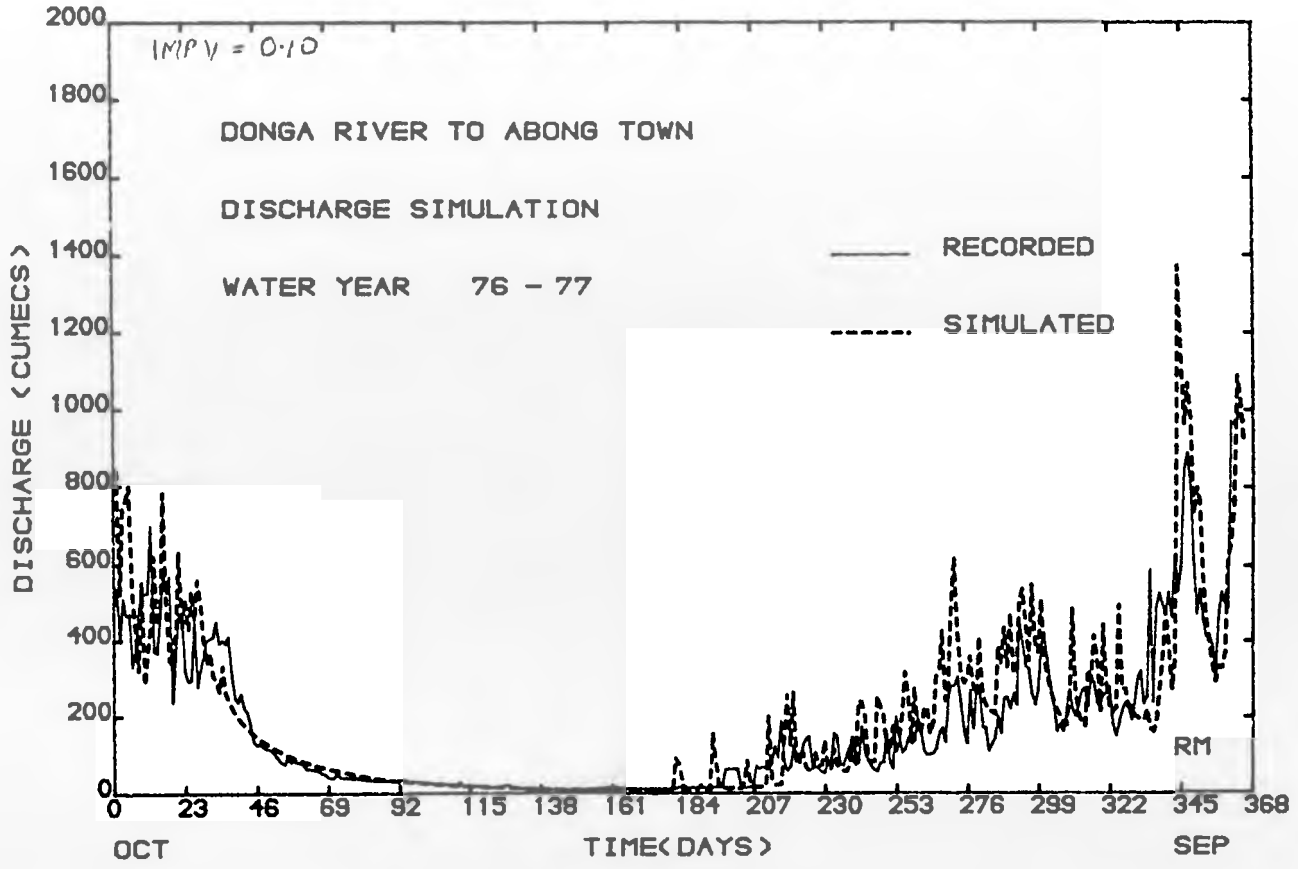

VALID'
DONGA RIVER TO Manya TOWN
DISCHARGE SIMULATION
WATER YEAR 61 - 62
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
23
46
69
92
115
138
161
184
207
230
253
276
299
322
345
368

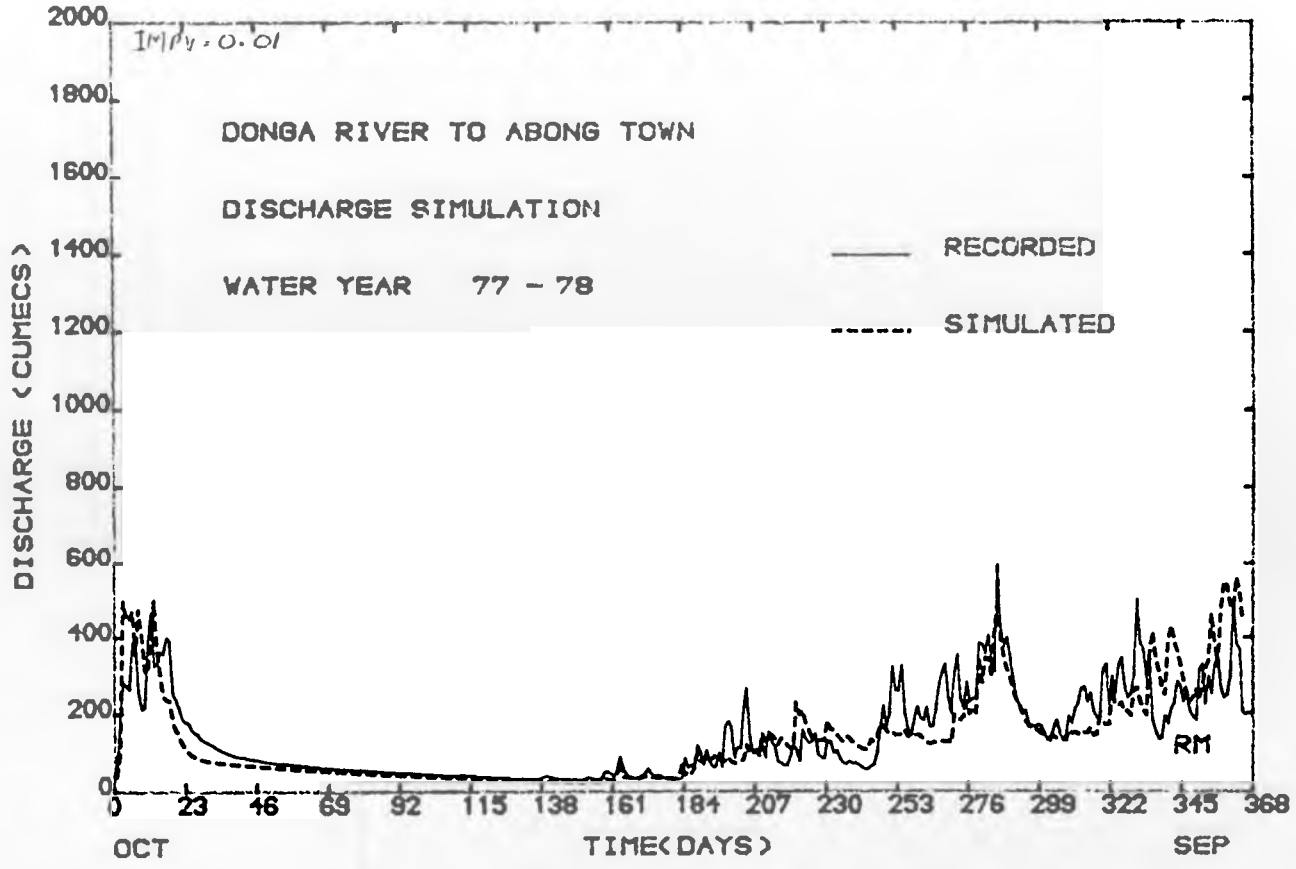
TESTING LANDUSE CHANGES, IMPV AND EPXM

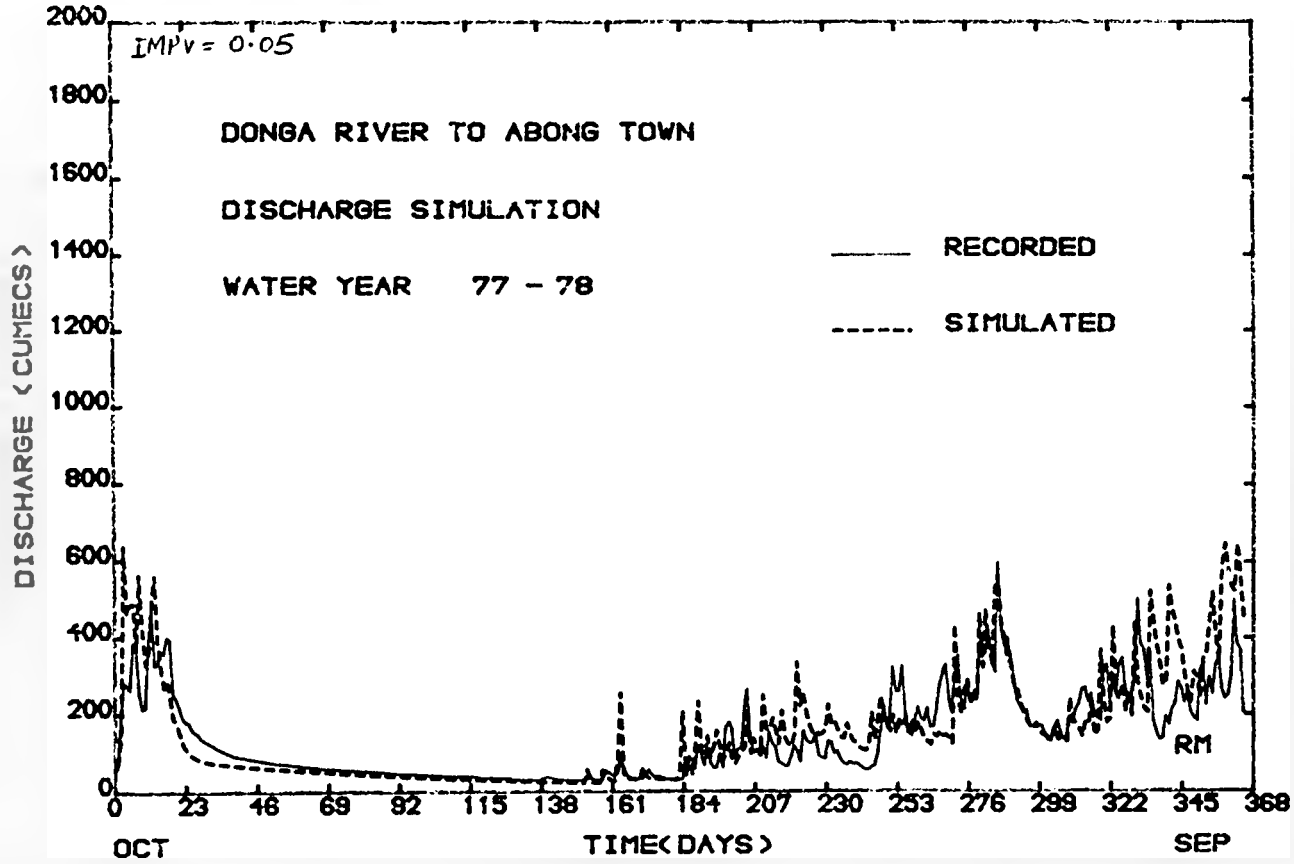
IMPV - FIGURES 8.7 (a) - HYDROGRAPHS

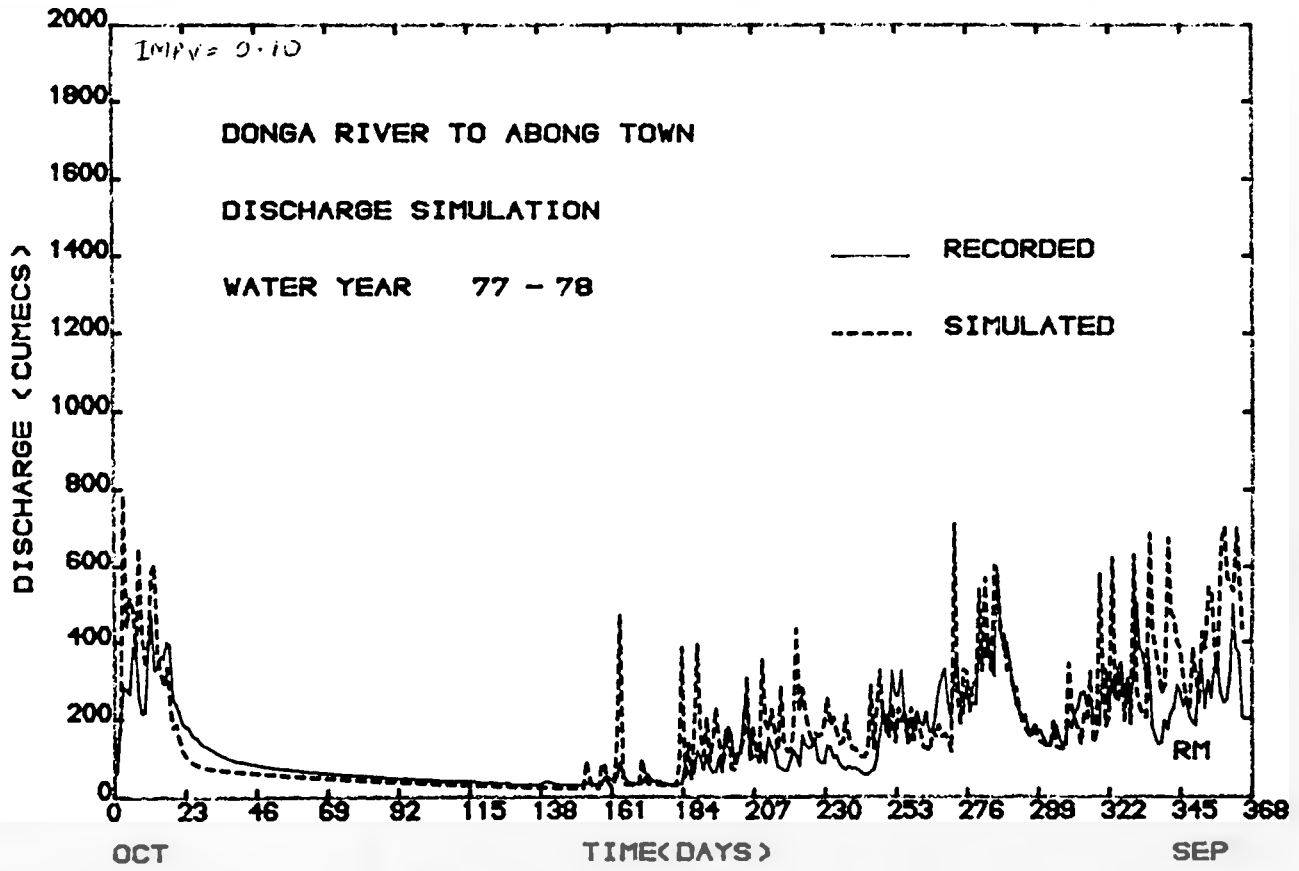
EPXM - FIGURES 8.7 (b) - HYDROGRAPHS

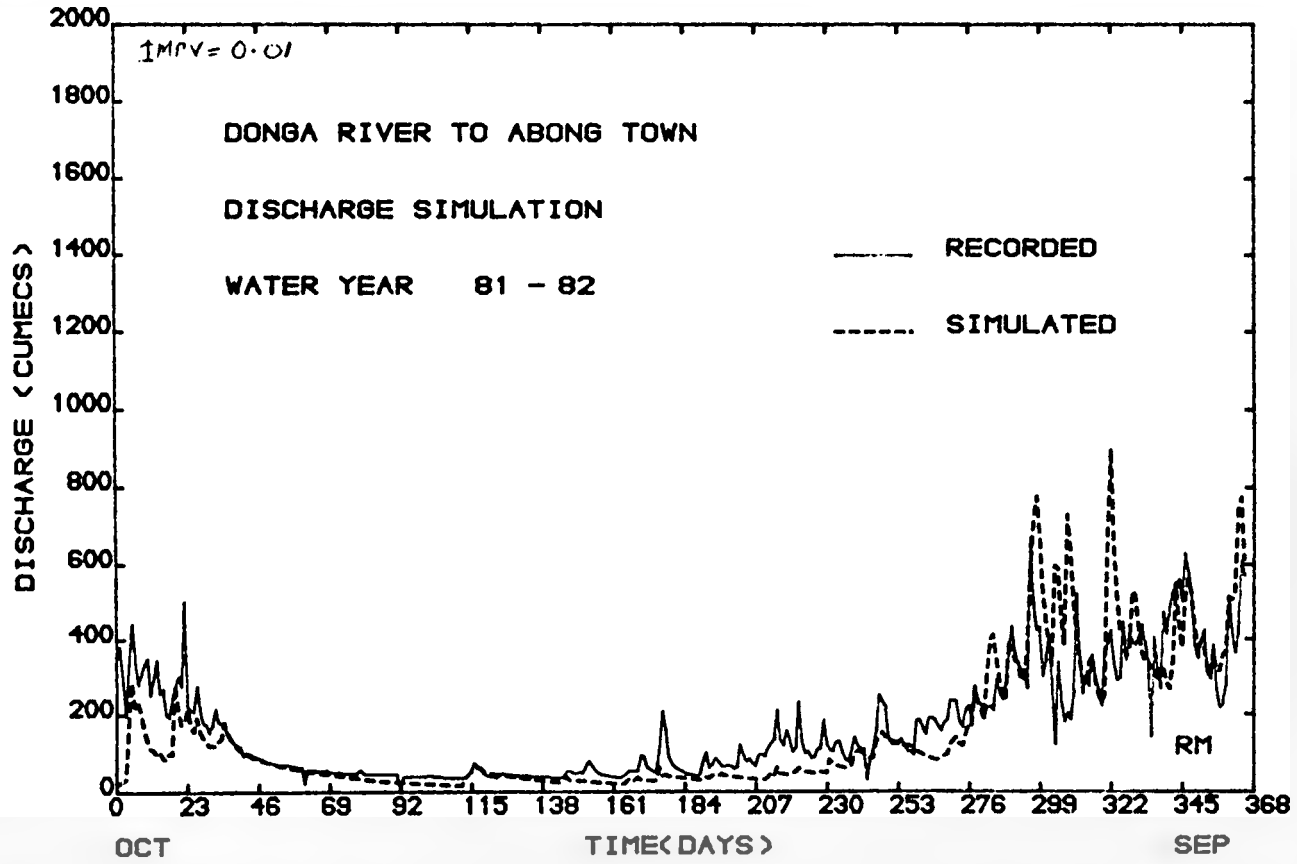

INITIAL,
IMPV = 0·01
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 76 - 77
RECORDED
SIMULATED
DISCHARGE (CUMECS)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368
OCT
TIME(DAYS)
SEP
RM

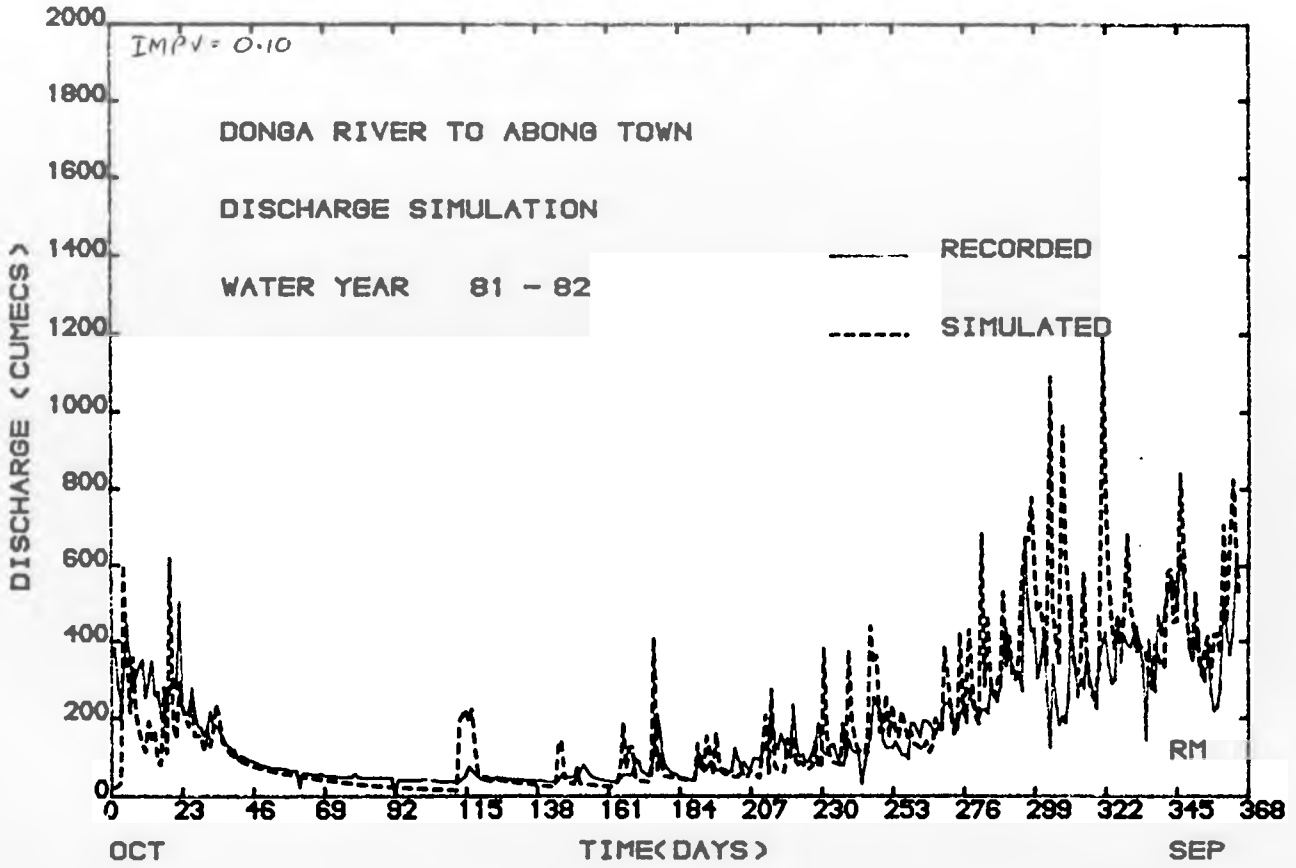

IMPV = 0.10
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 76 - 77
RECORDED
SIMULATED
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
RM
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368

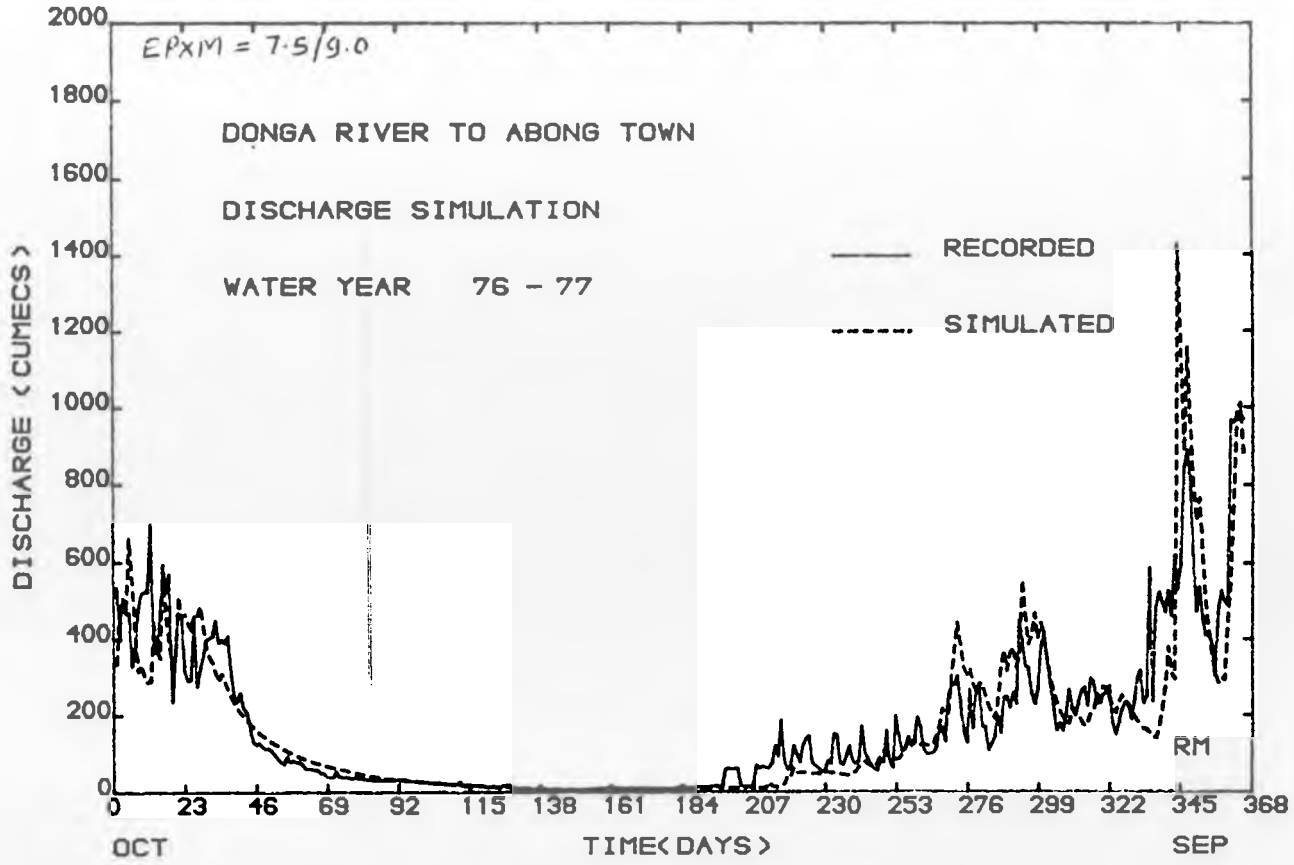

IMPy = 0.01
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 77 - 78
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
23
46
69
92
115
138
161
184
207
230
253
276
299
322
345
368

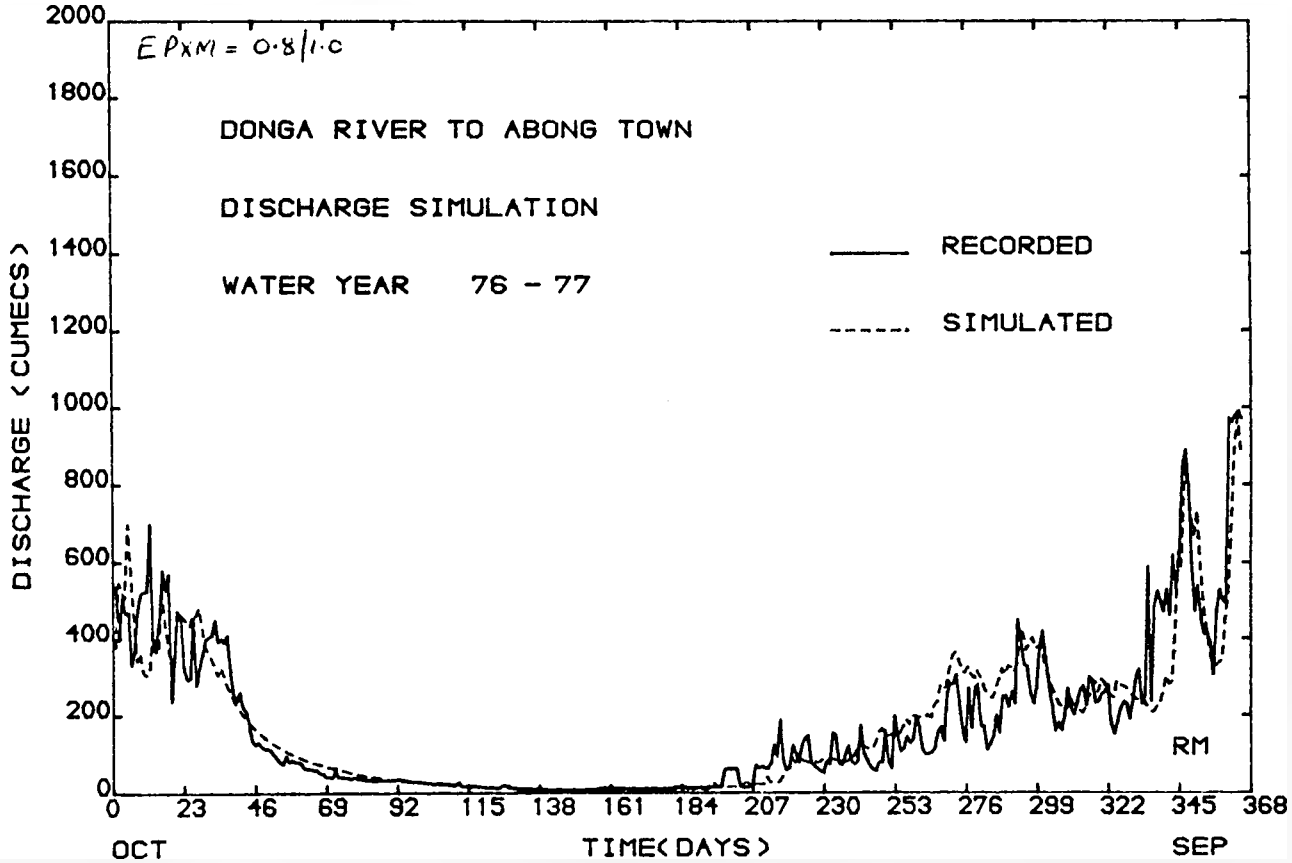

IMPv = 0·05
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 77 - 78
RECORDED
SIMULATED
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
RM
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
23
46
69
92
115
138
161
184
207
230
253
276
299
322
345
368

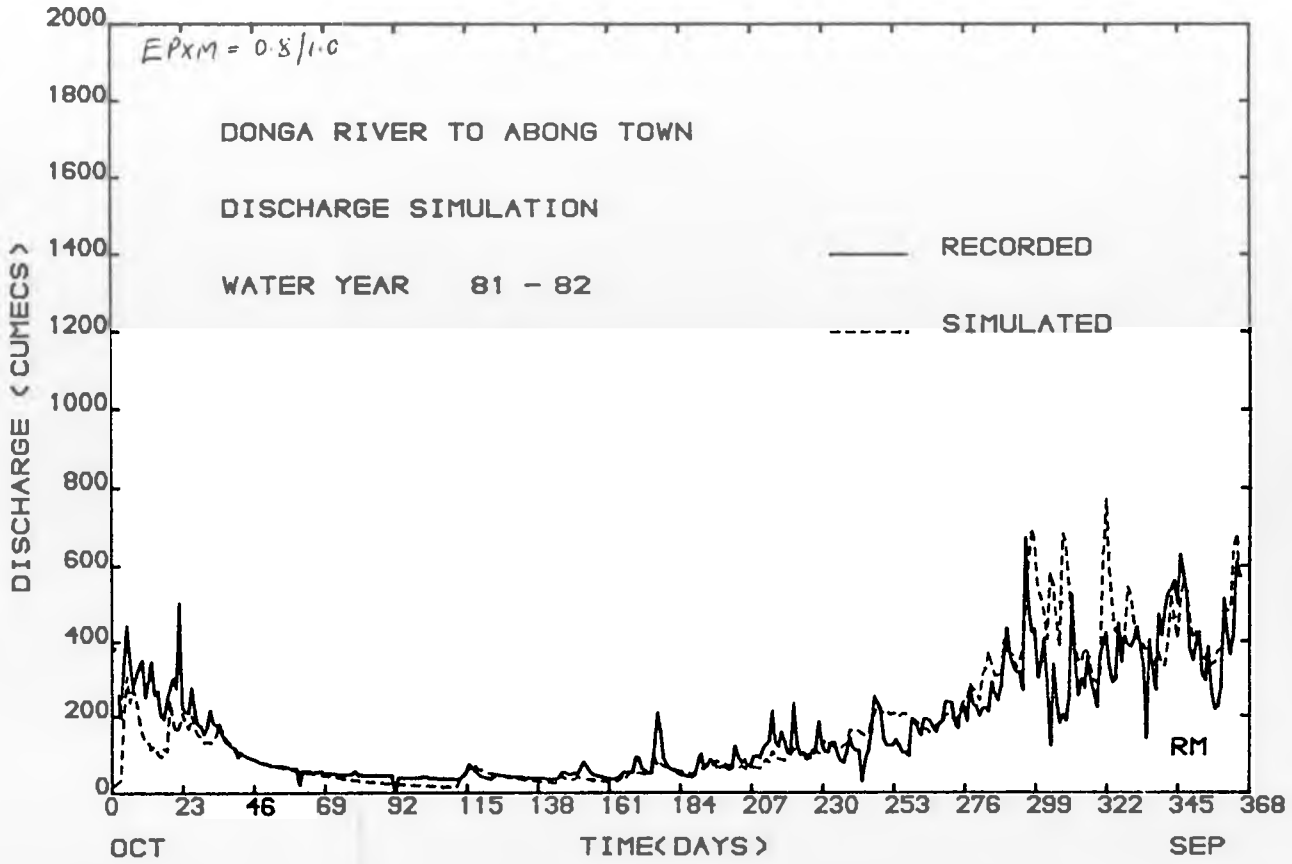

IMPV = 0·10
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 77 - 78
RECORDED
SIMULATED
DISCHARGE (CUMECS)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368
OCT
TIME(DAYS)
SEP
RM

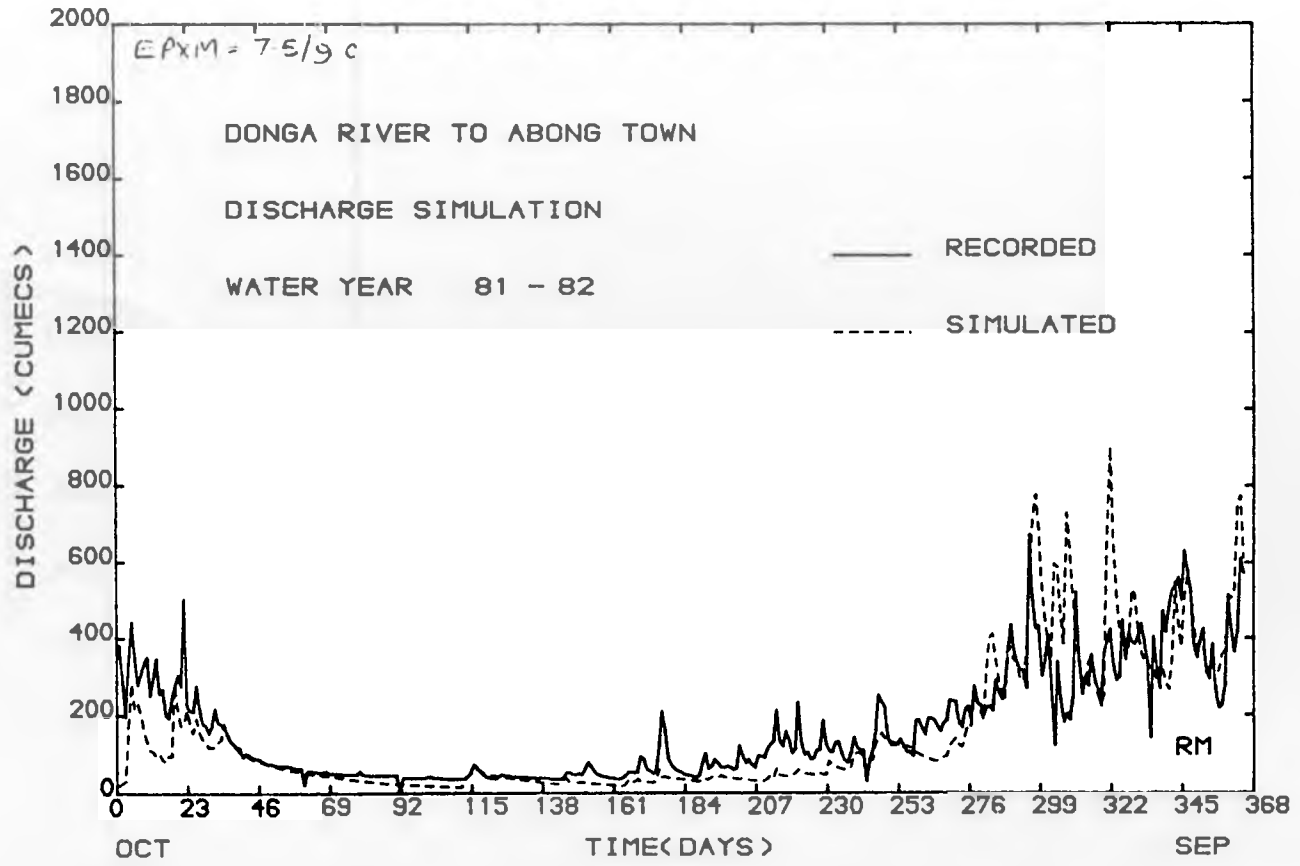

1MRV= 0·01
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 81 - 82
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368
OCT
TIME(DAYS)
SEP


IMPV = 0.10
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 81 - 82
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
23
46
69
92
115
138
161
184
207
230
253
276
299
322
345
368

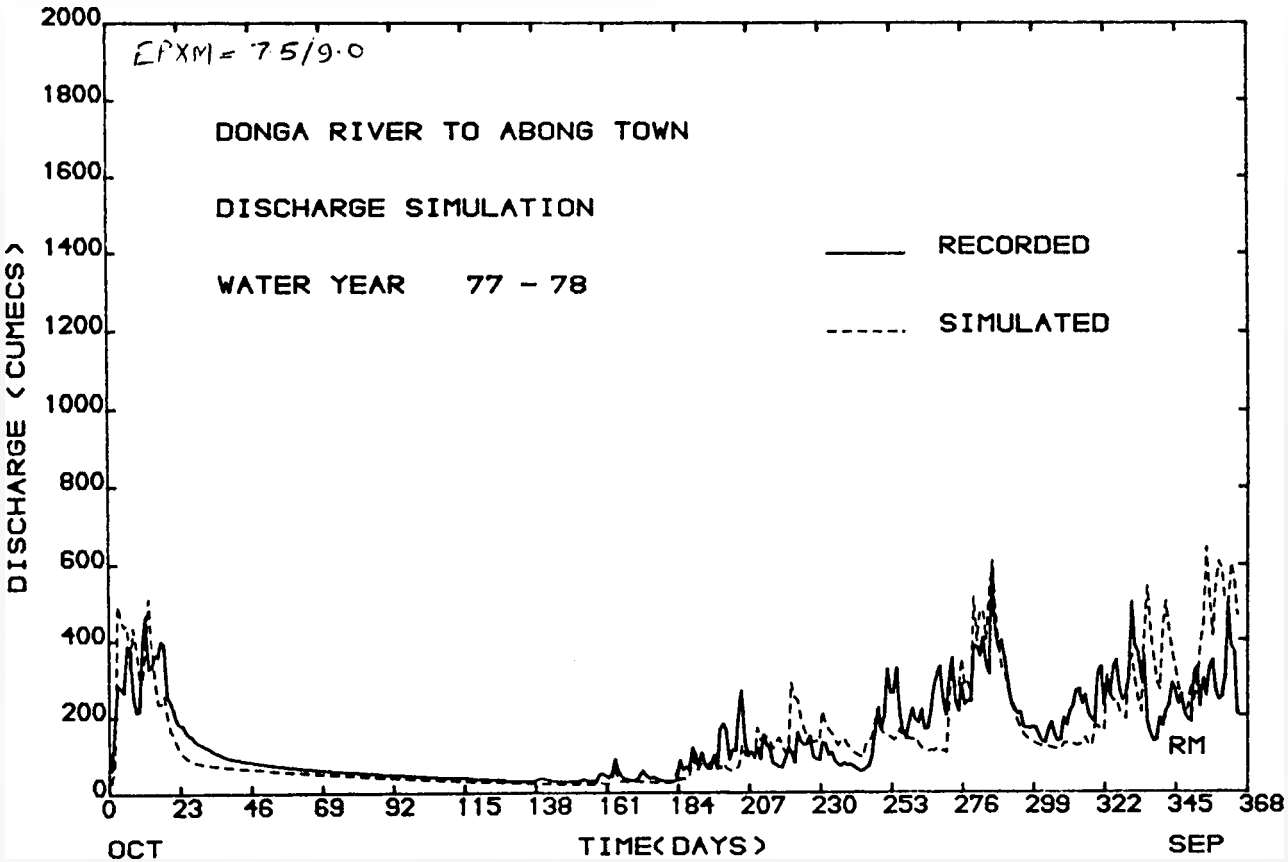

EPXM = 7.5/9.0
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 76 - 77
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368

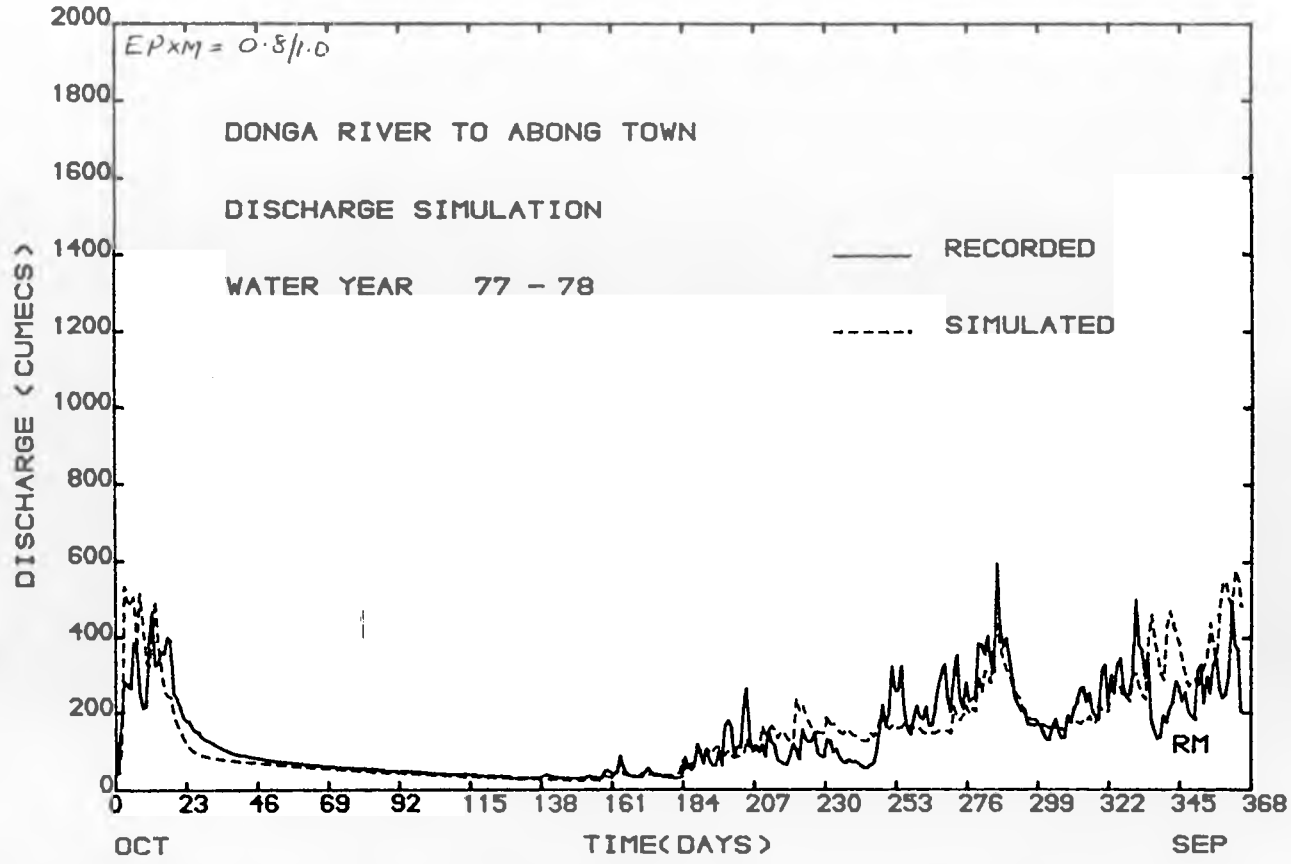

EPXM = 0.8/1.0
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 76 - 77
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
23
46
69
92
115
138
161
184
207
230
253
276
299
322
345
368


EPxM = 0.8/1.0
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 81 - 82
RECORDED
SIMULATED
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
RM
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368


EPxM = 75/9 c
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 81 - 82
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
23
46
69
92
115
138
161
184
207
230
253
276
299
322
345
368

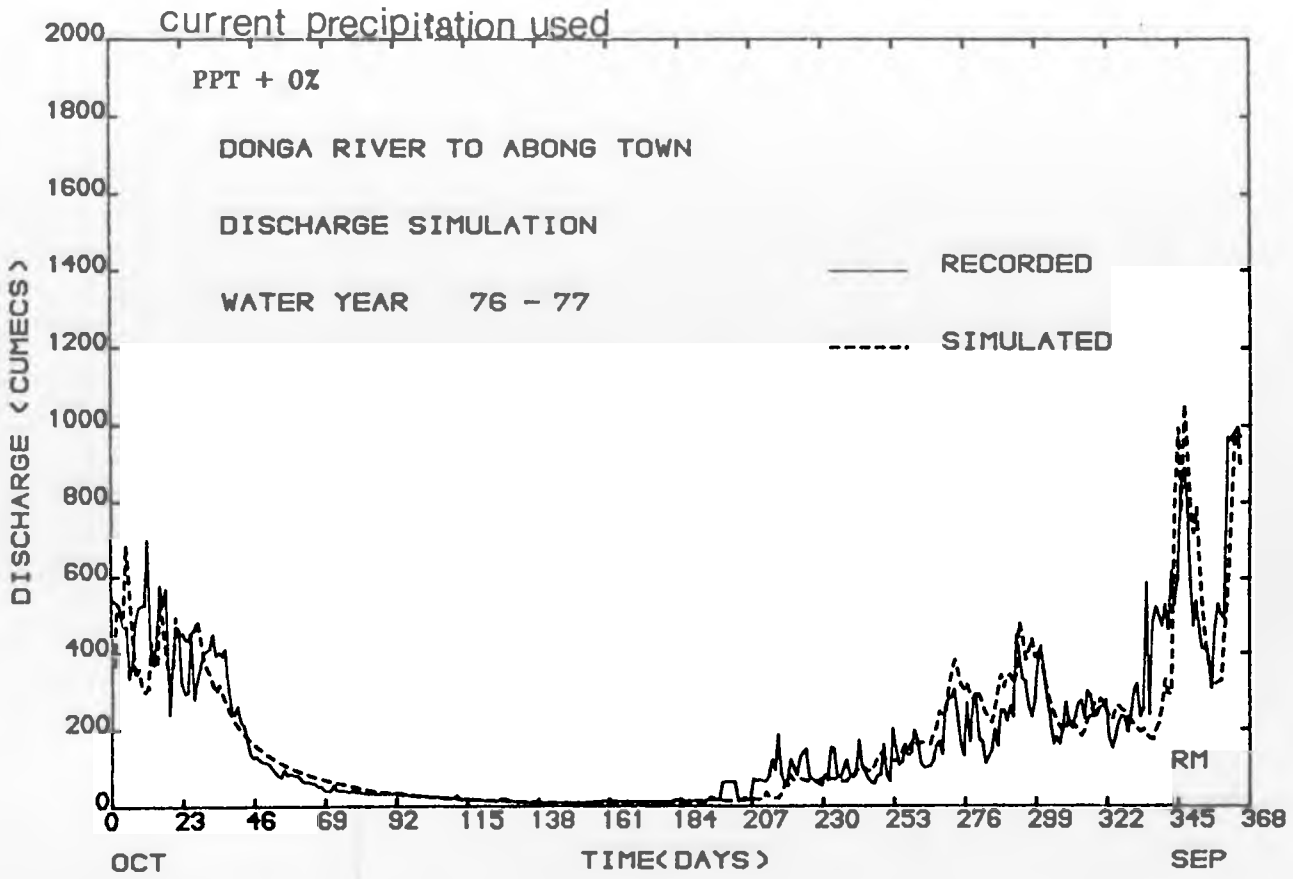
USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 82255 QUEUED ON 07/12/88 AT 11:51 PLOTTED ON 07/12/88 AT 11:53




EPXM = 0·8/1·0
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 77 - 78
RECORDED
SIMULATED
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
RM
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368

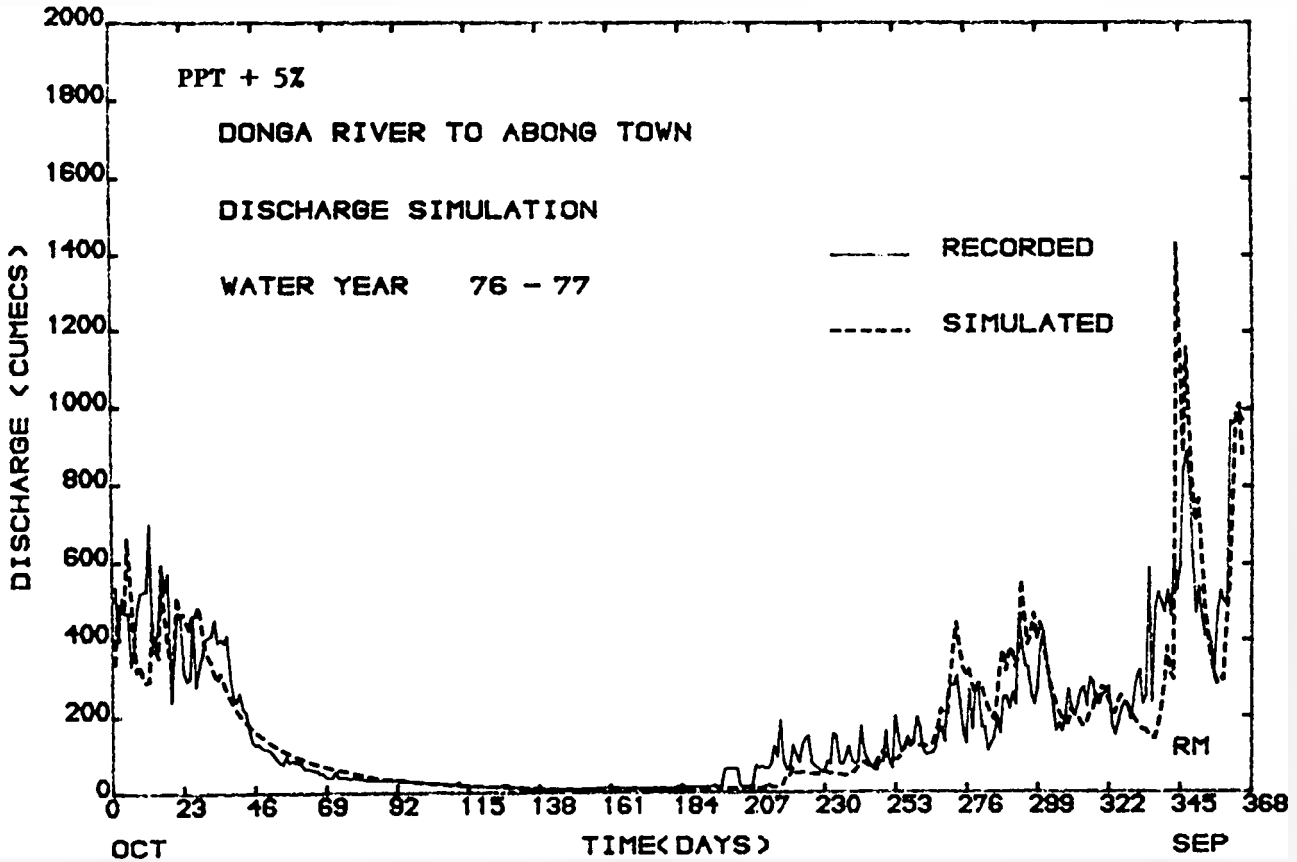
USER: CHAV61 PLOTFILE: PFILE ENTRY NO. 79665 QUEUED ON 13/10/88 AT 16:19 PLOTTED ON 13/10/88 AT 16:24

Fig. 8.8 (1)



USER: CMAV81 PLOTFILE: PFILE ENTRY NO. 80529 QUEUED ON 25/11/88 AT 11:35 PLOTTED ON 25/11/88 AT 16:42

Fig. 8.8 (2)



USER: CWAV61 PLOTFILE: PFILE ENTRY NO. 80528 QUEUED ON 25/11/88 AT 11:24 PLOTTED ON 25/11/88 AT 16:39

Fig. 8.8 (3)

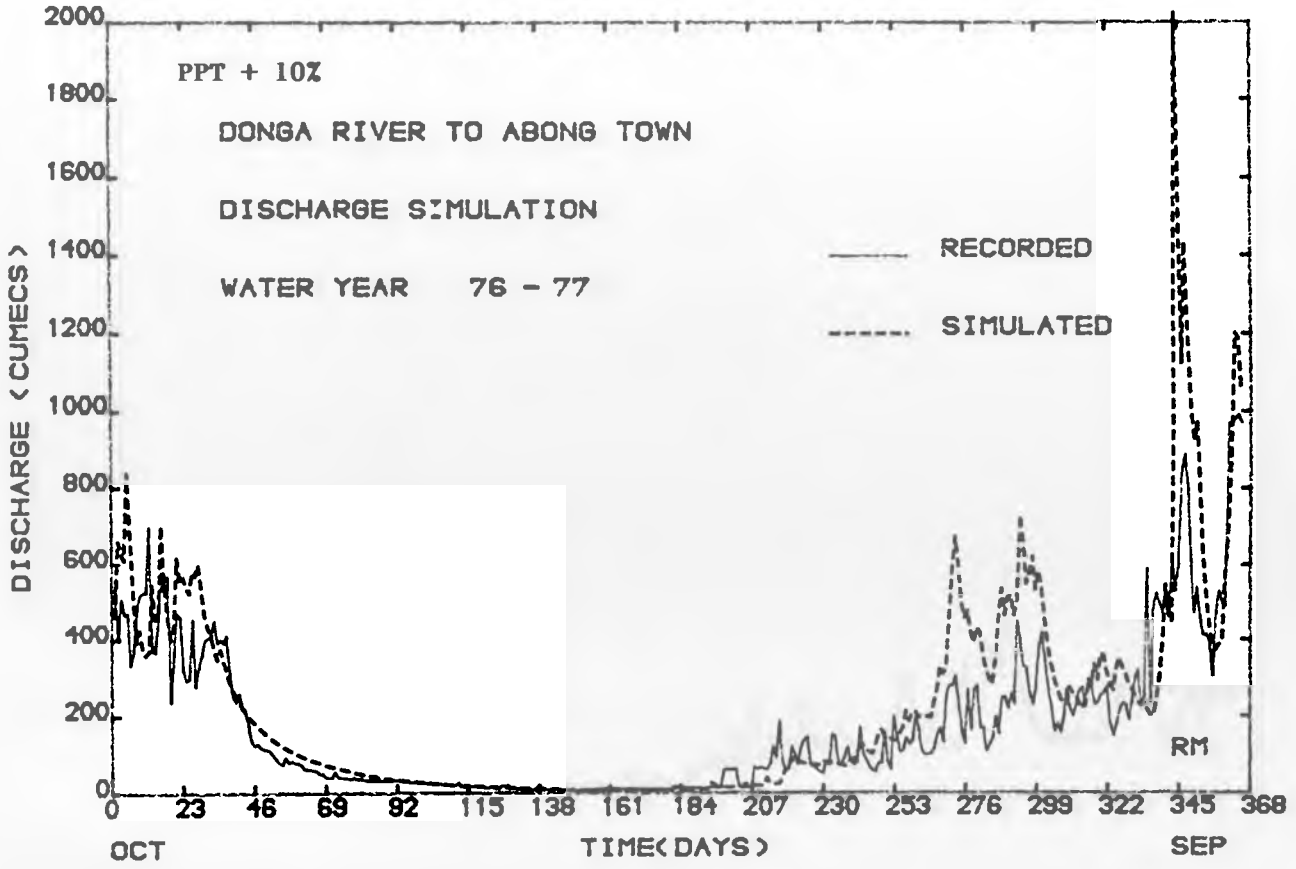


Fig. 8.8 (4)

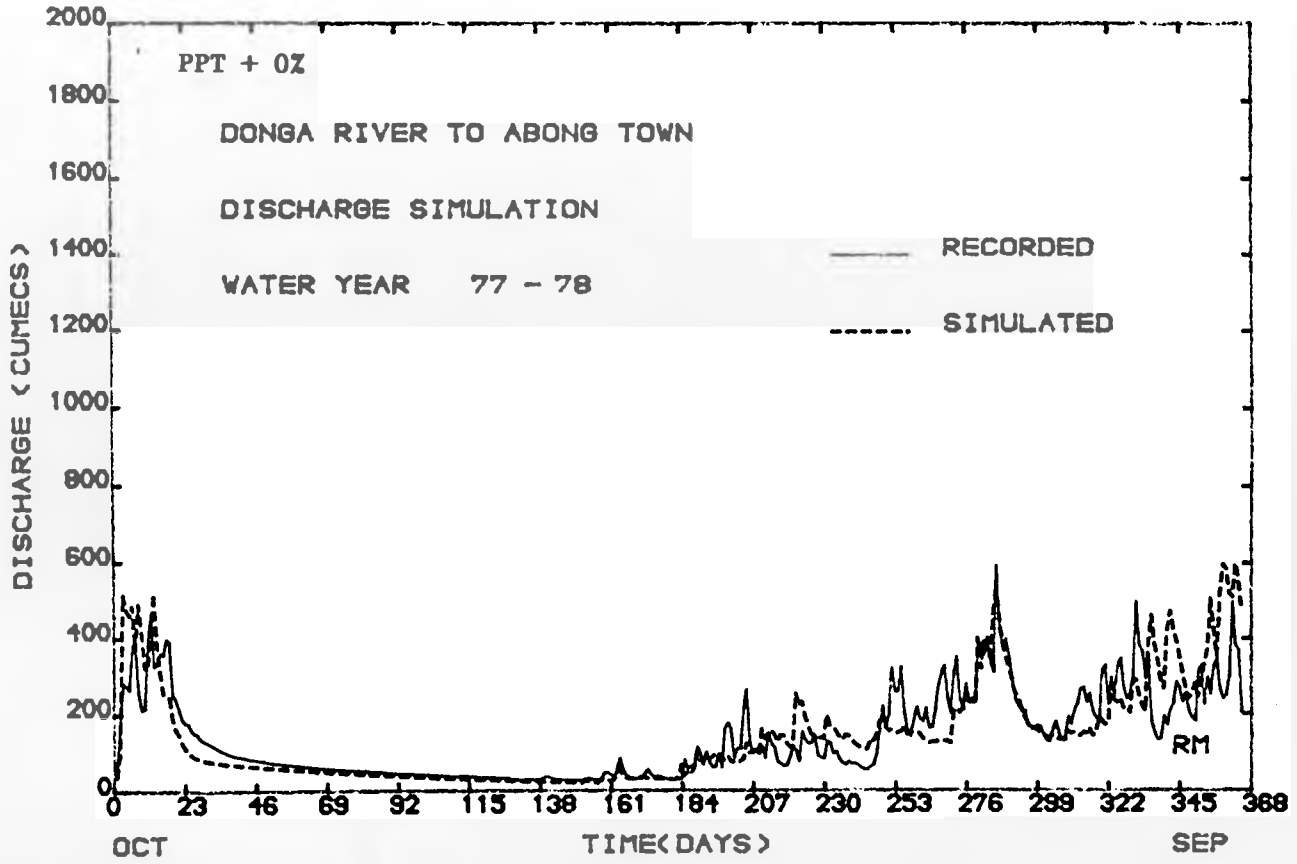

PPT + 0%
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 77 - 78
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
23
46
69
92
115
138
161
184
207
230
253
276
299
322
345
368

Fig. 8.8 (5)

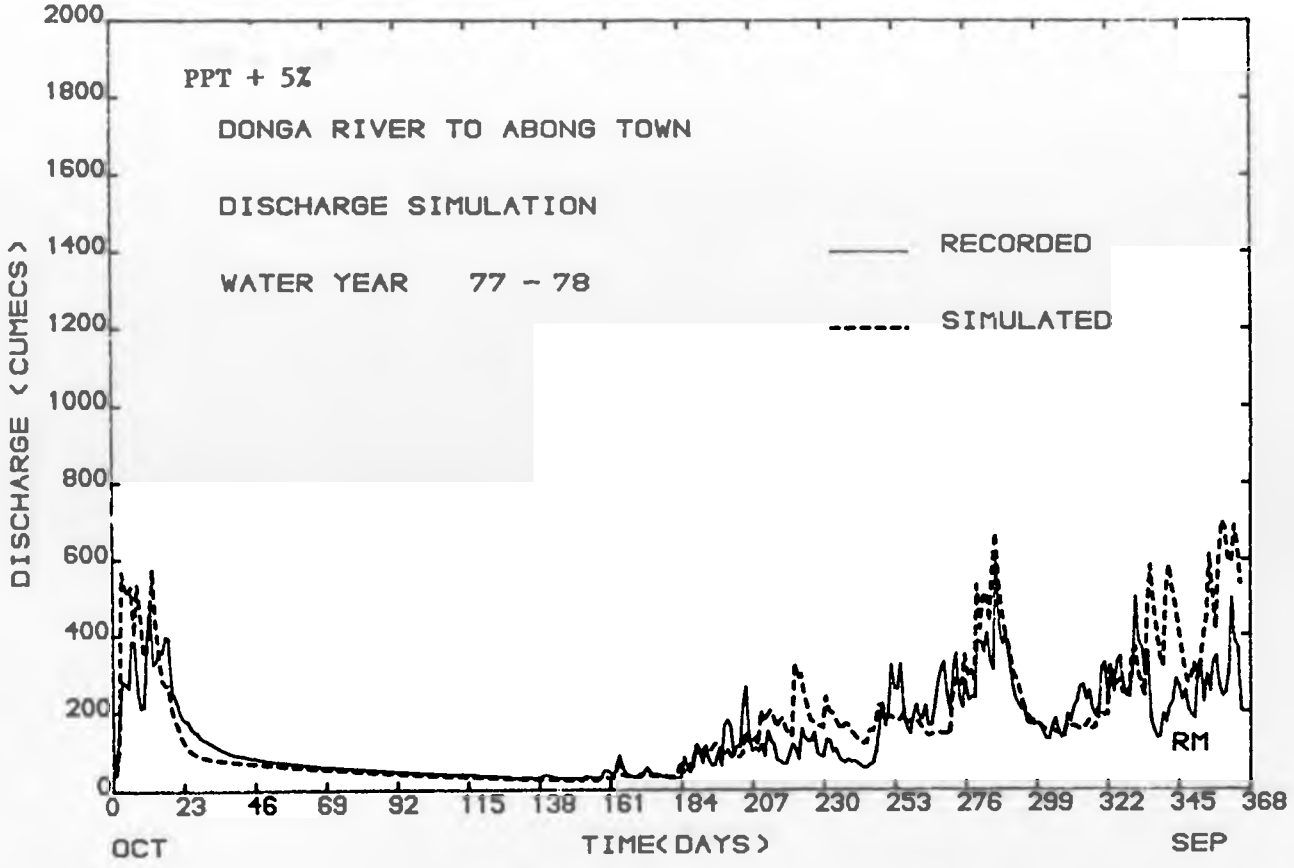

PPT + 5%
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 77 - 78
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368

Fig. 8.8 (6)

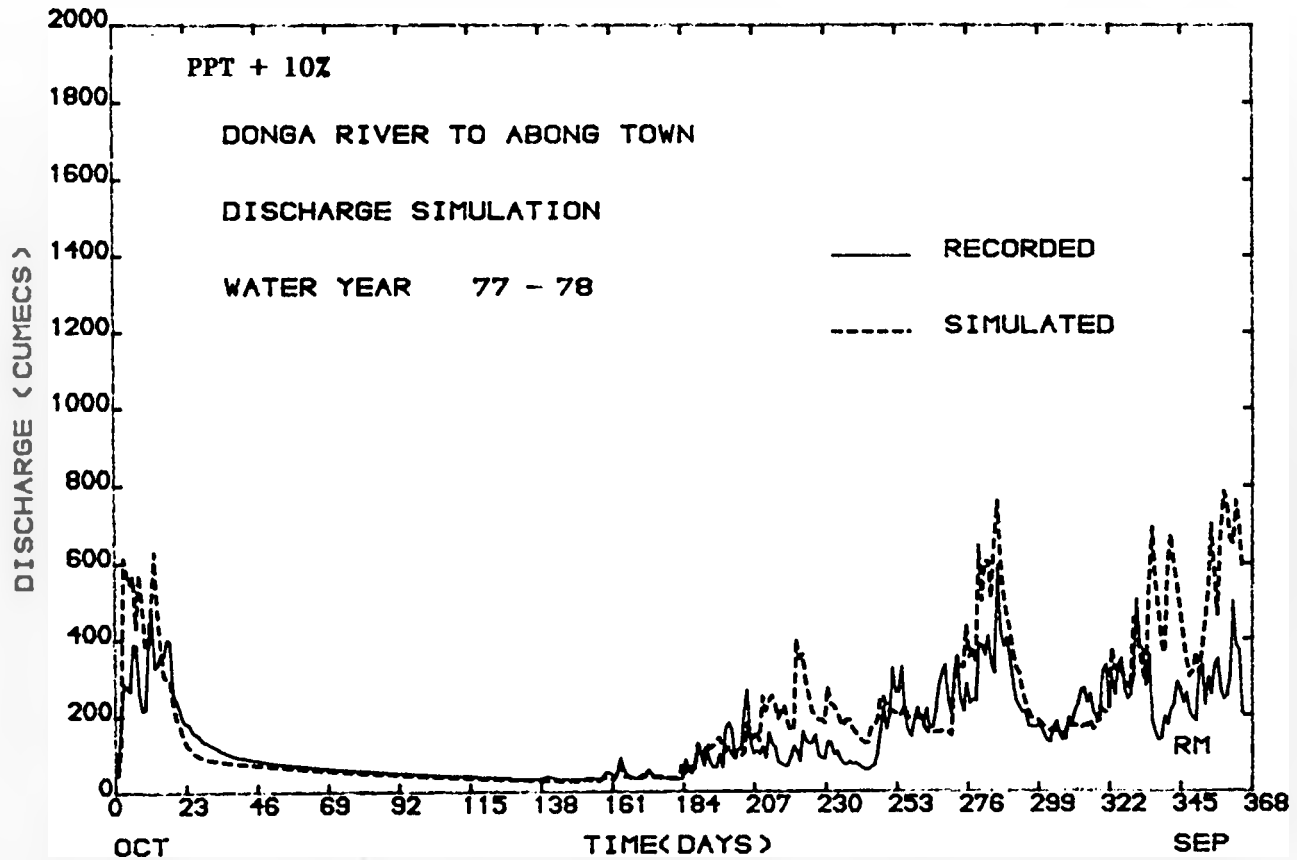
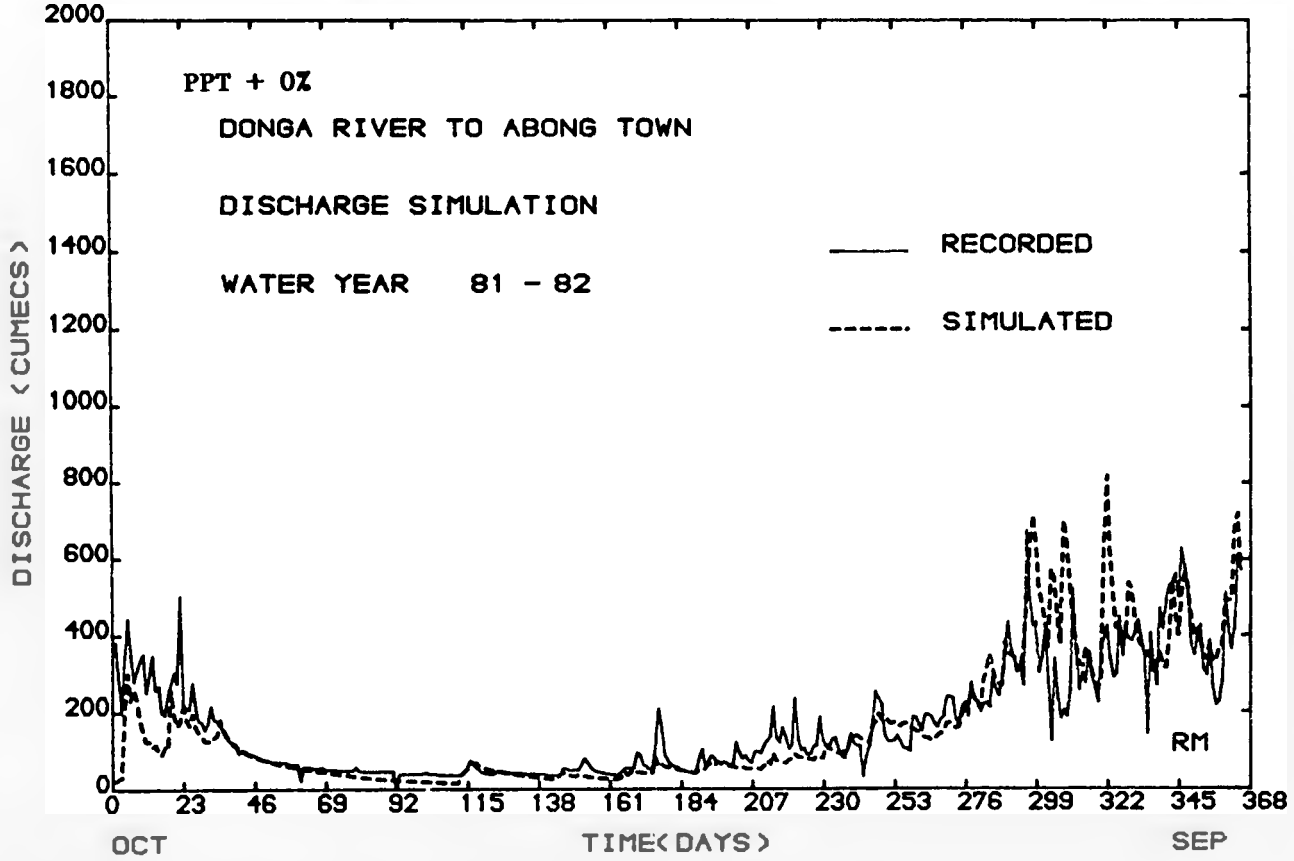

PPT + 10%
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 77 - 78
RECORDED
SIMULATED
DISCHARGE (CUMECS)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368
OCT
TIME(DAYS)
SEP
RM

Fig. 8.8 (7)


PPT + 0%
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 81 - 82
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0
23
46
69
92
115
138
161
184
207
230
253
276
299
322
345
368

USER: CMAV81 PLOTFILE: PFILE ENTRY NO. 80748 QUEUED ON 27/11/86 AT 16:49 PLOTTED ON 28/11/86 AT 16:33

Fig. 8.8 (8)

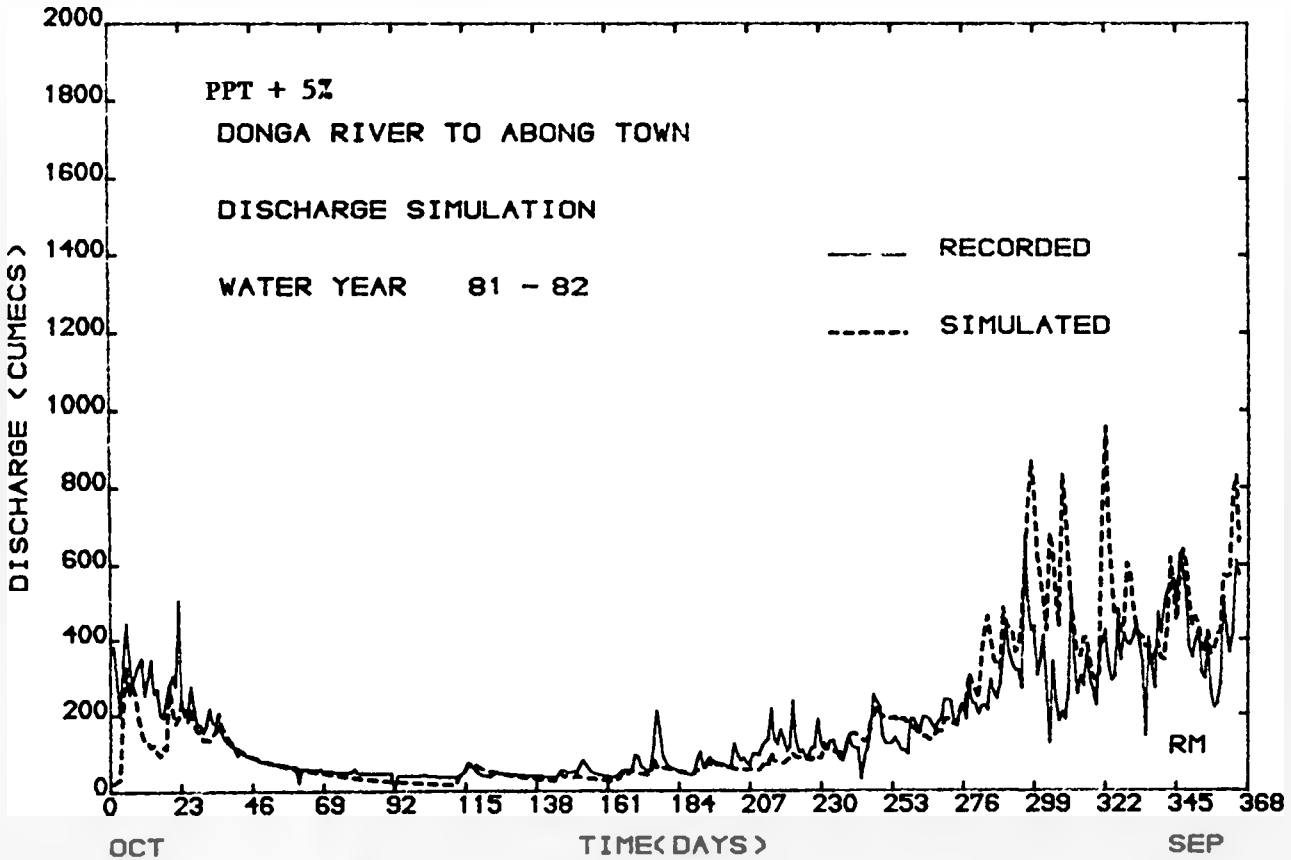
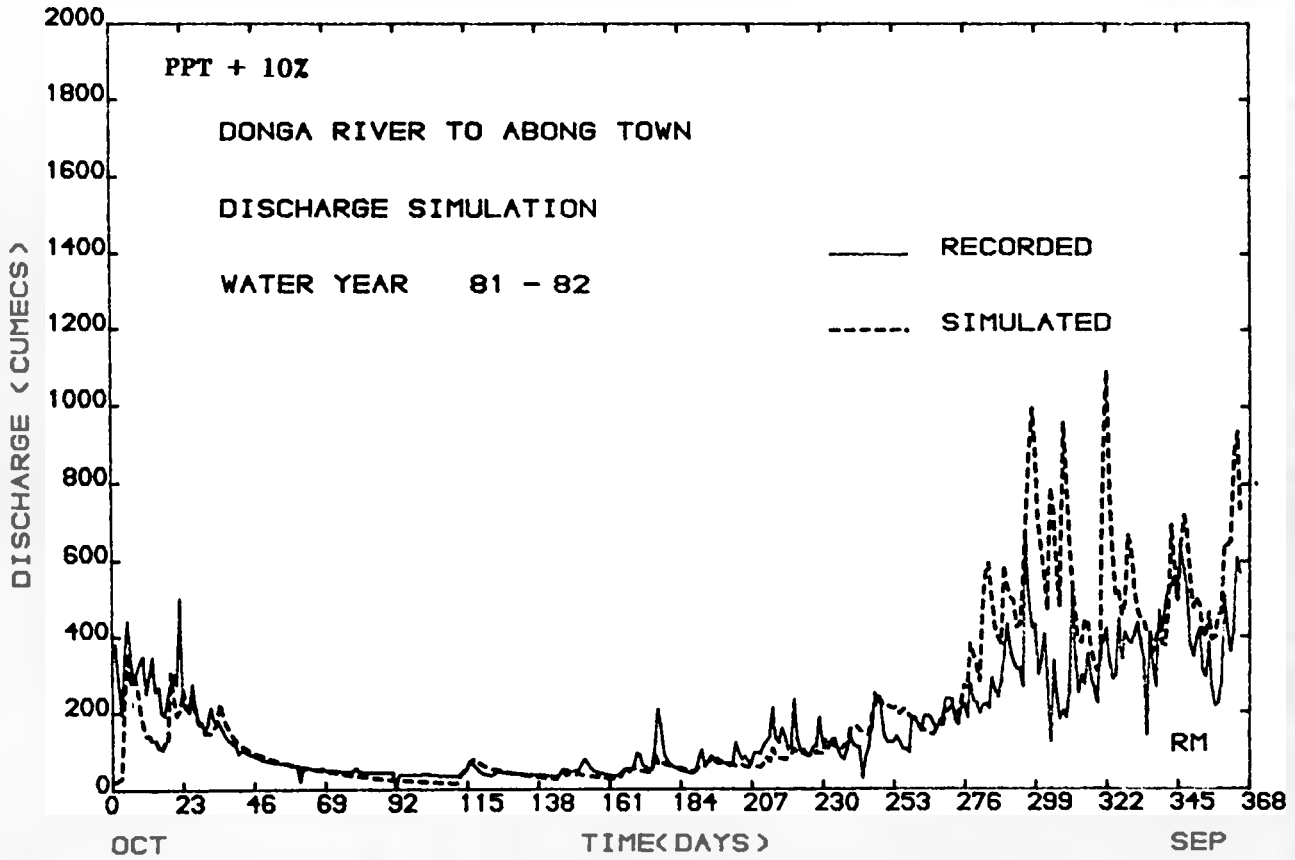


Fig. 8.8. (9)


PPT + 10%
DONGA RIVER TO ABONG TOWN
DISCHARGE SIMULATION
WATER YEAR 81 - 82
RECORDED
SIMULATED
RM
DISCHARGE (CUMECS)
TIME(DAYS)
OCT
SEP
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0 23 46 69 92 115 138 161 184 207 230 253 276 299 322 345 368

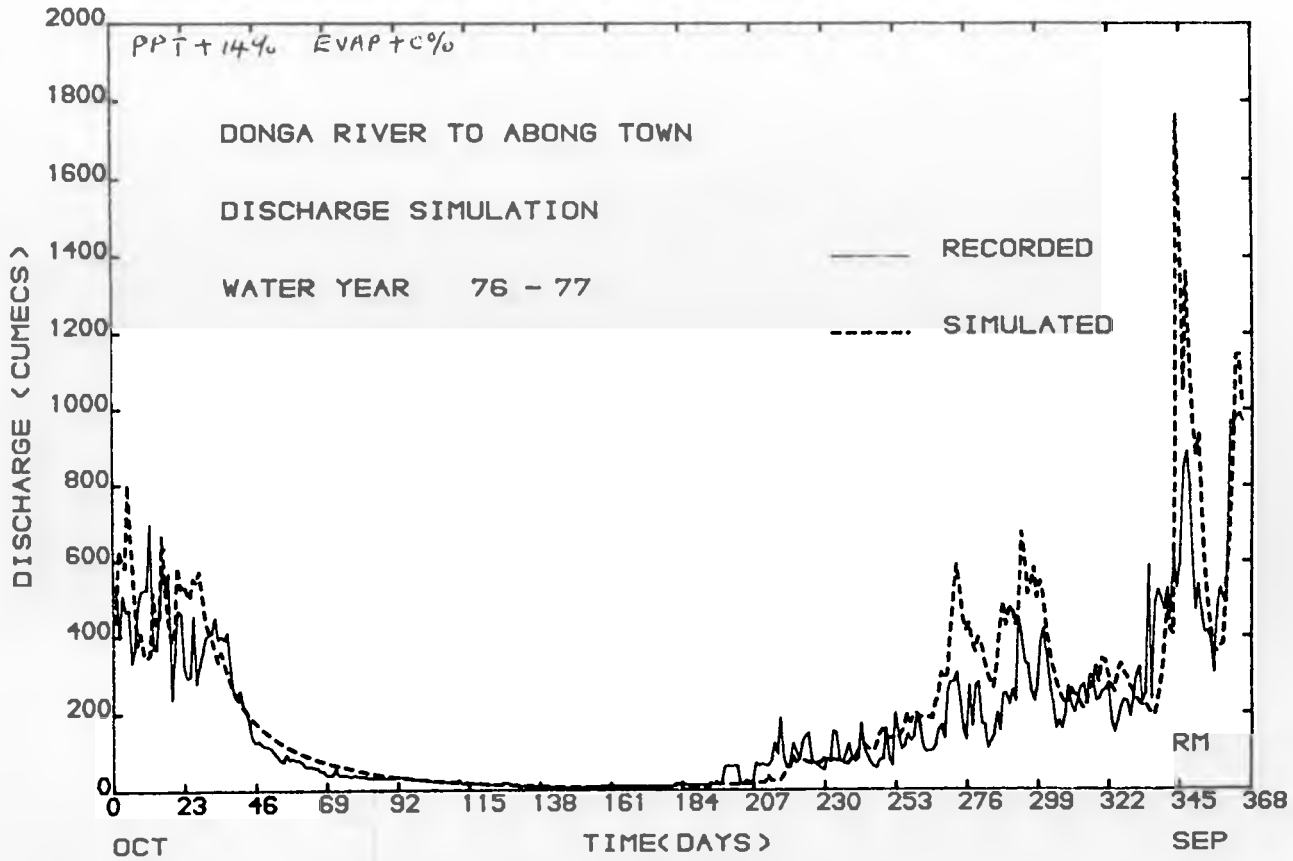
TESTING THE COMBINATION OF RESERVOIR AND GLOBALLY INDUCED CLIMATE CHGANGES

FIGURES 8.9 - HYDROGRAPHS

USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 80235 QUEUED ON 24/11/88 AT 11:14 PLOTTED ON 24/11/88 AT 13:15

Fig. (a)

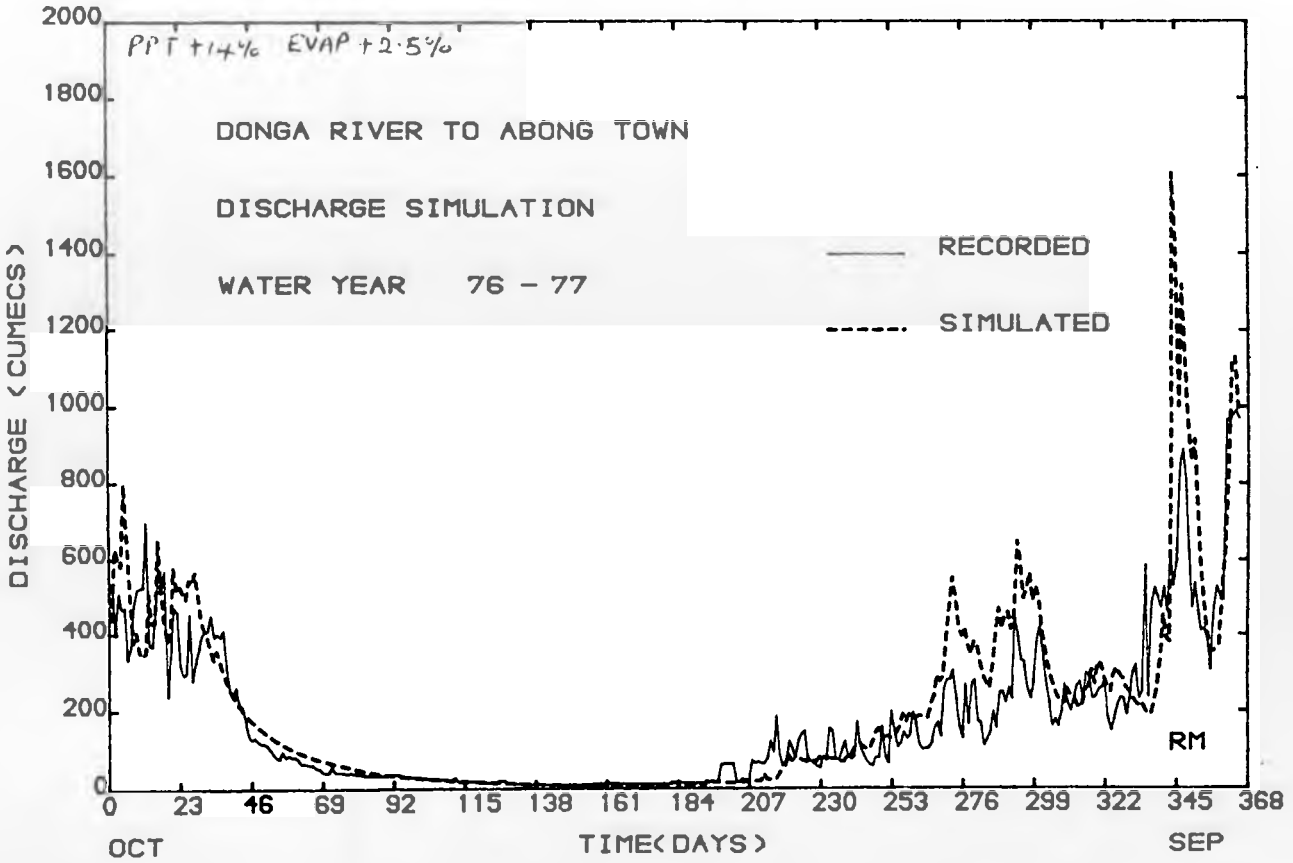
PRECIPITATION + 14%
EVAPORATION + 0%



USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 80246 QUEUED ON 24/11/88 AT 11:34 PLOTTED ON 24/11/88 AT 13:18

Fig. (b)

PRECIPITATION + 14%
EVAPORATION + 2.5%



USER: CNAV81 PLOTFILE: PFILE ENTRY NO. 80248 QUEUED ON 24/11/88 AT 11:48 PLOTTED ON 24/11/88 AT 14:24

Fig. (c)

PRECIPITATION + 14%

EVAPORATION + 5%

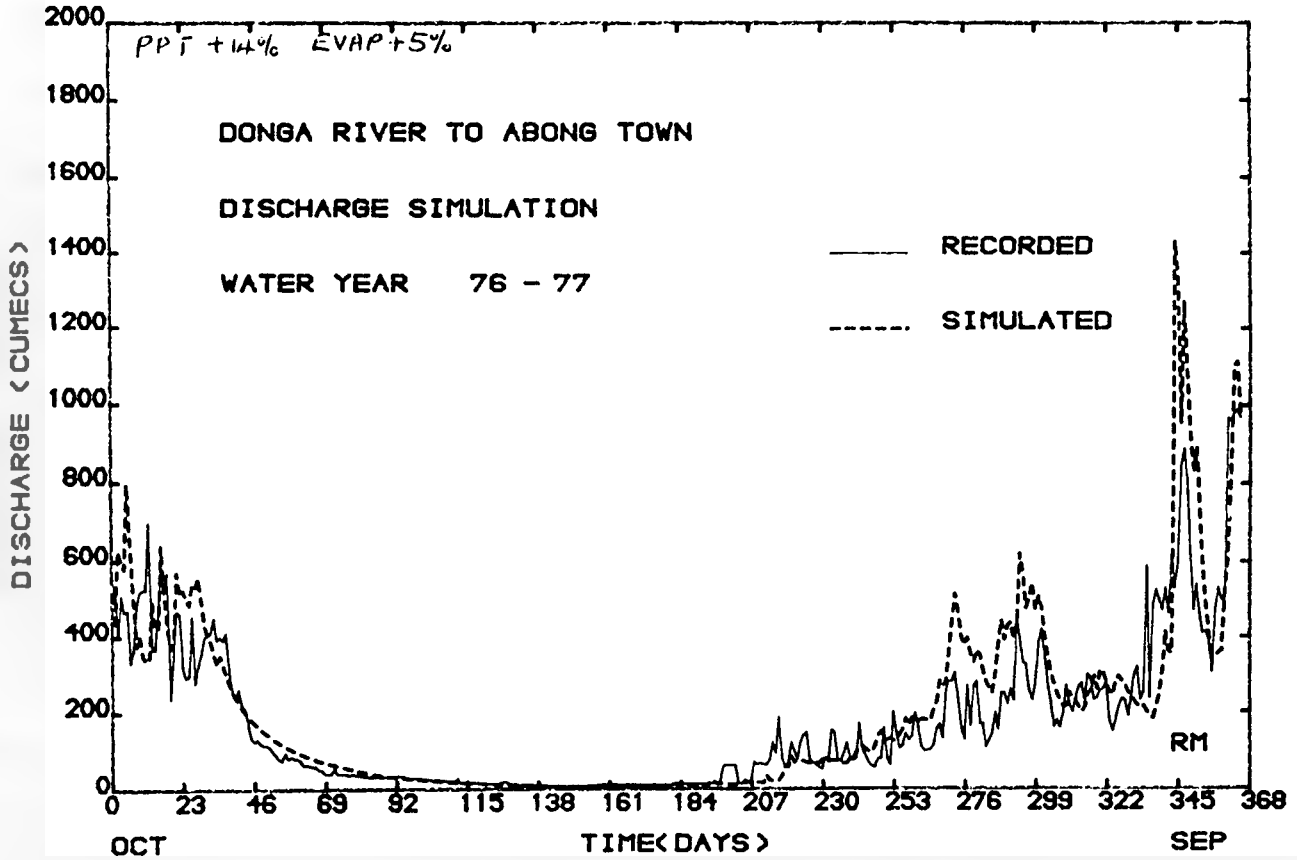


Fig. (d)

PRECIPITATION + 14%
EVAPORATION + 7.5%

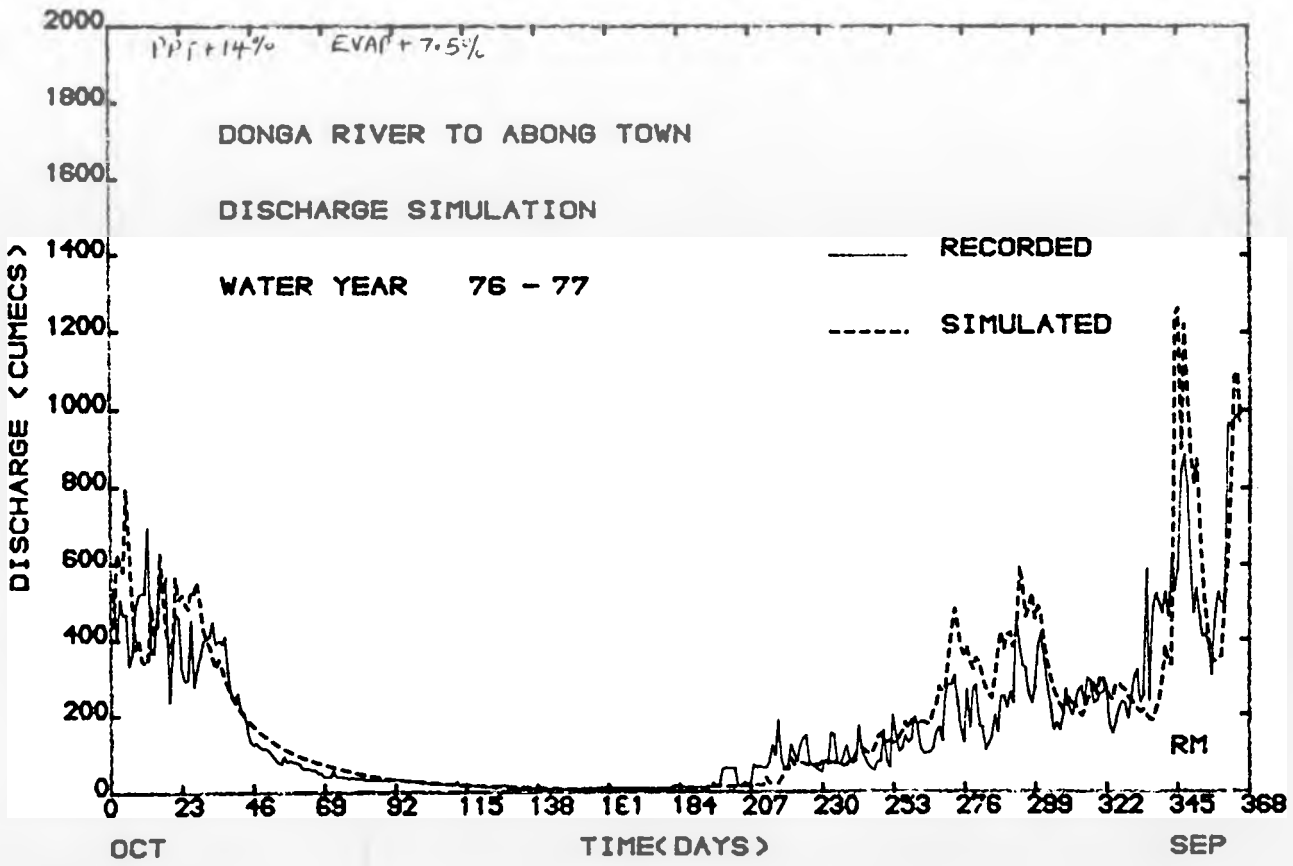
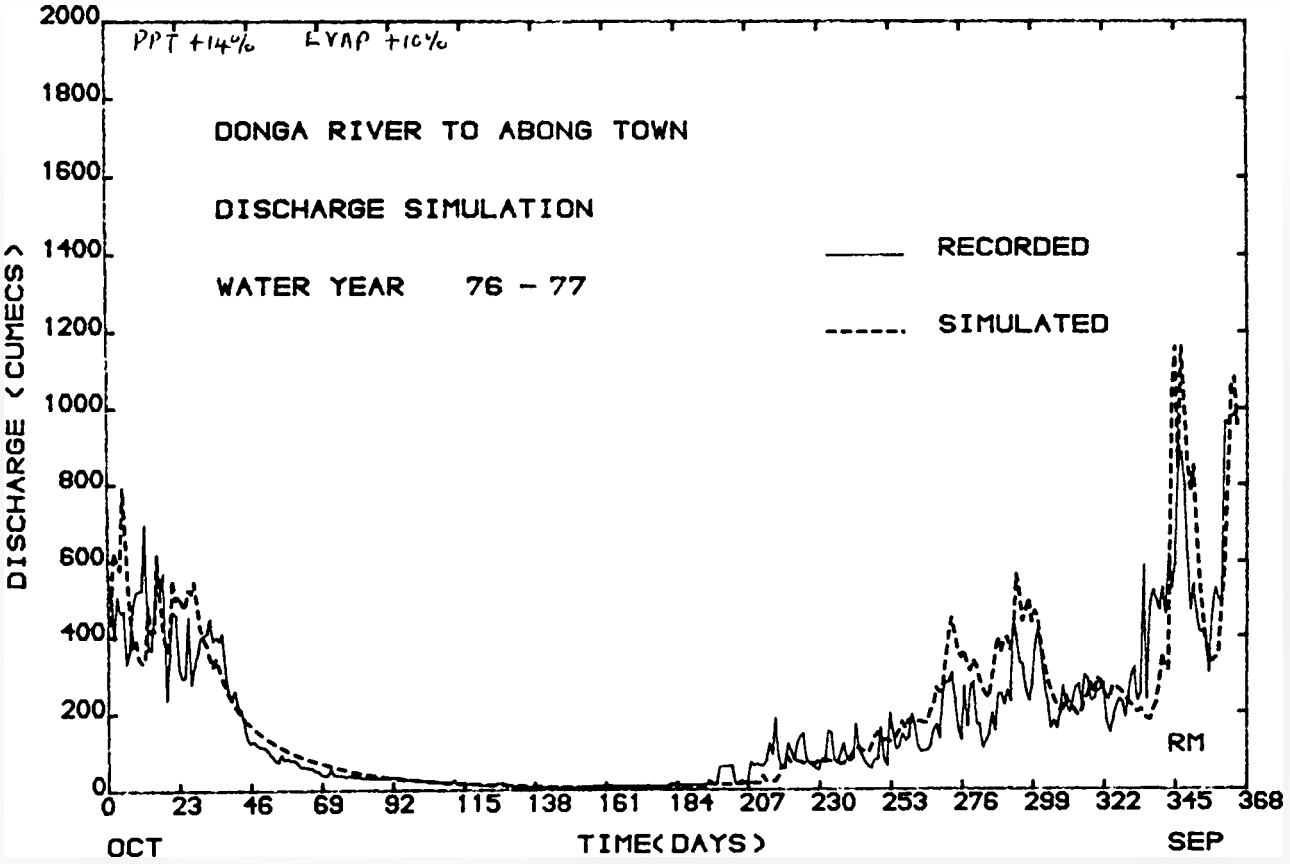


Fig. (e)

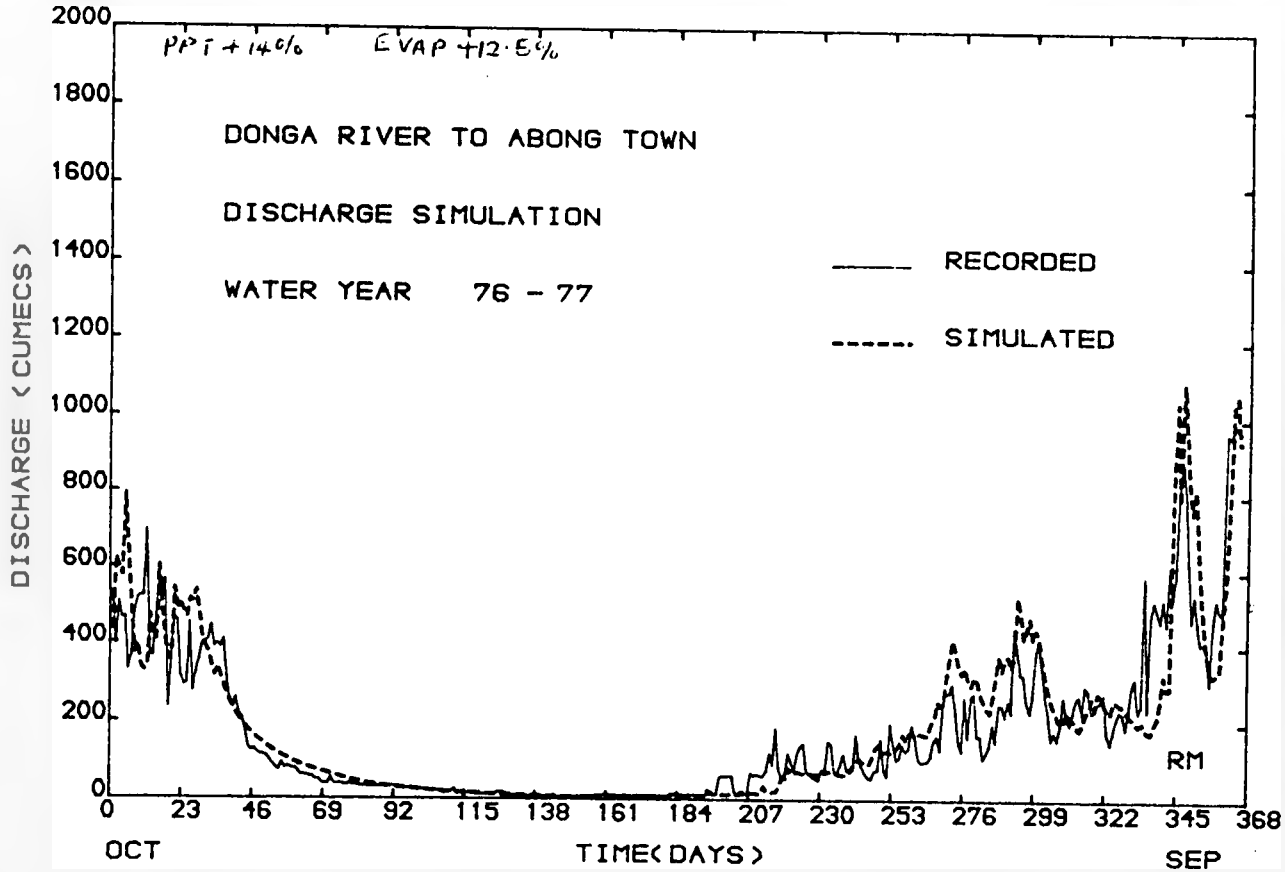
PRECIPITATION + 14%
EVAPORATION + 10.0%



USER: CHAV61 PLOTFILE: PFILE ENTRY NO. 80252 QUEUED ON 24/11/88 AT 12:14 PLOTTED ON 24/11/88 AT 16:49

Fig. (f)

PRECIPITATION + 14%
EVAPORATION +12.5%



USER: CNAV81 PLOTFILE: PFILE ENTRY NO. 80443 QUEUED ON 24/11/88 AT 18:13 PLOTTED ON 25/11/88 AT 12:46

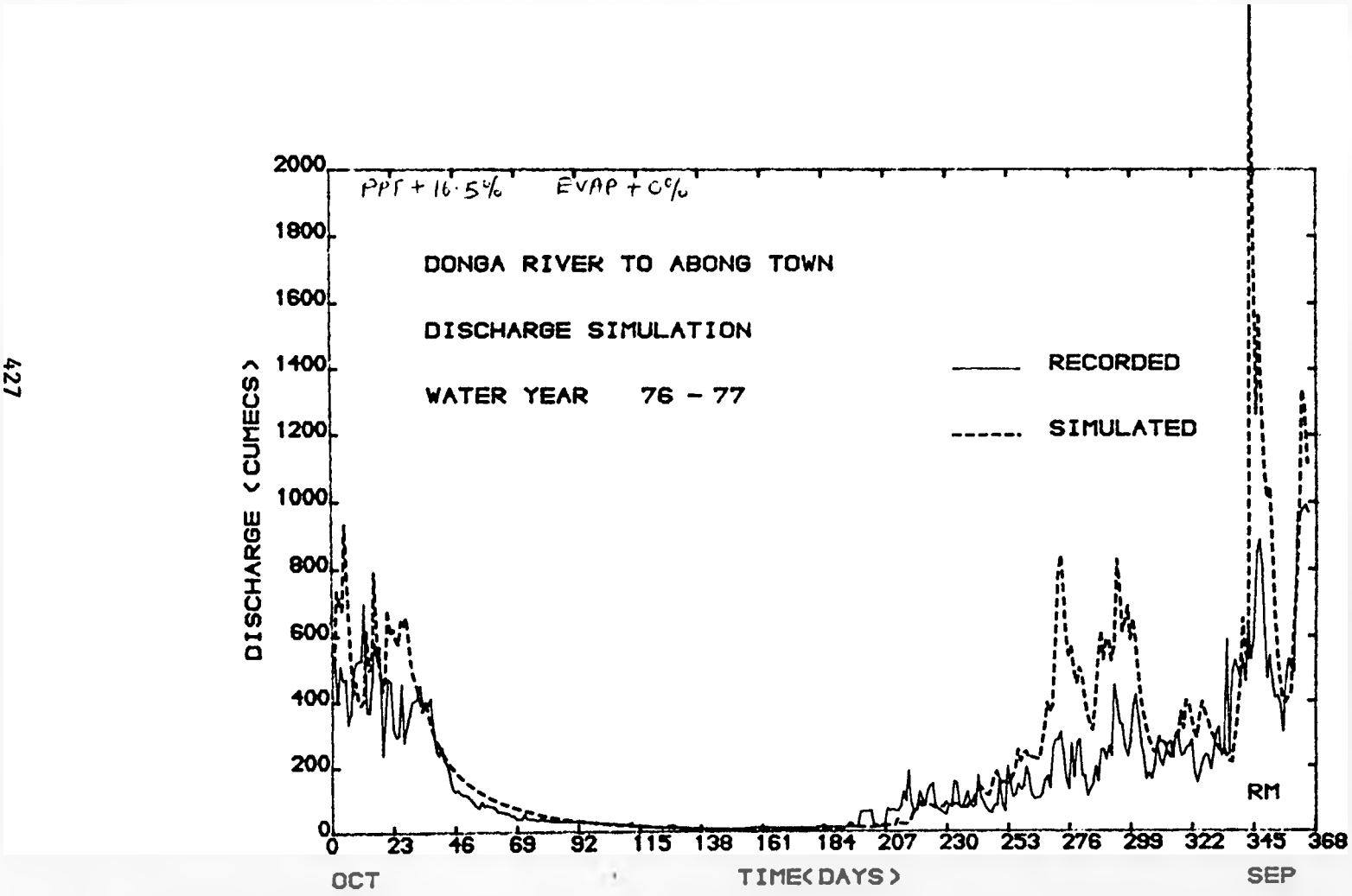


Fig. (a)
PRECIPITATION + 16.5%
EVAPORATION + 0%

Fig. (b)

PRECIPITATION + 16.5%
EVAPORATION + 2.5%

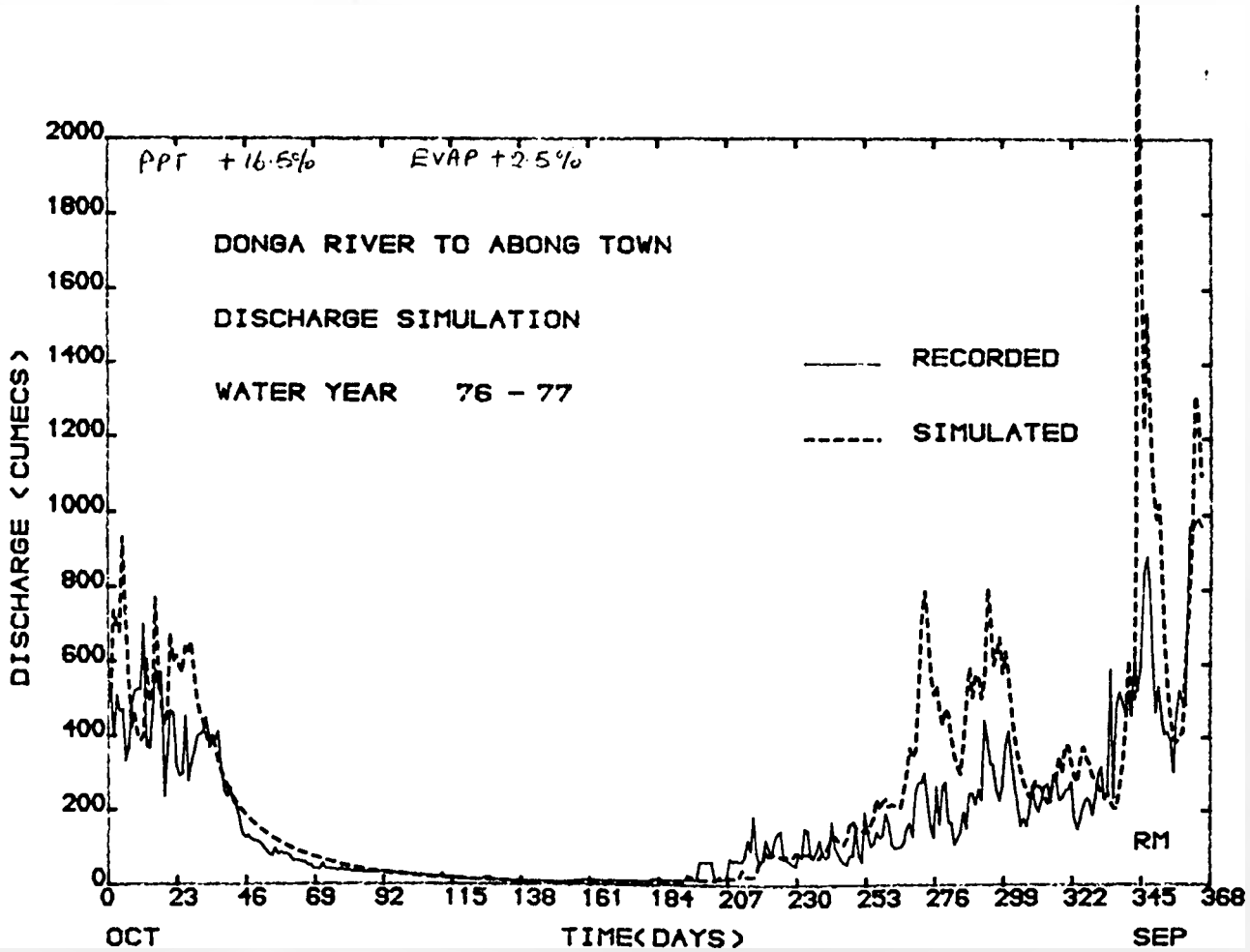
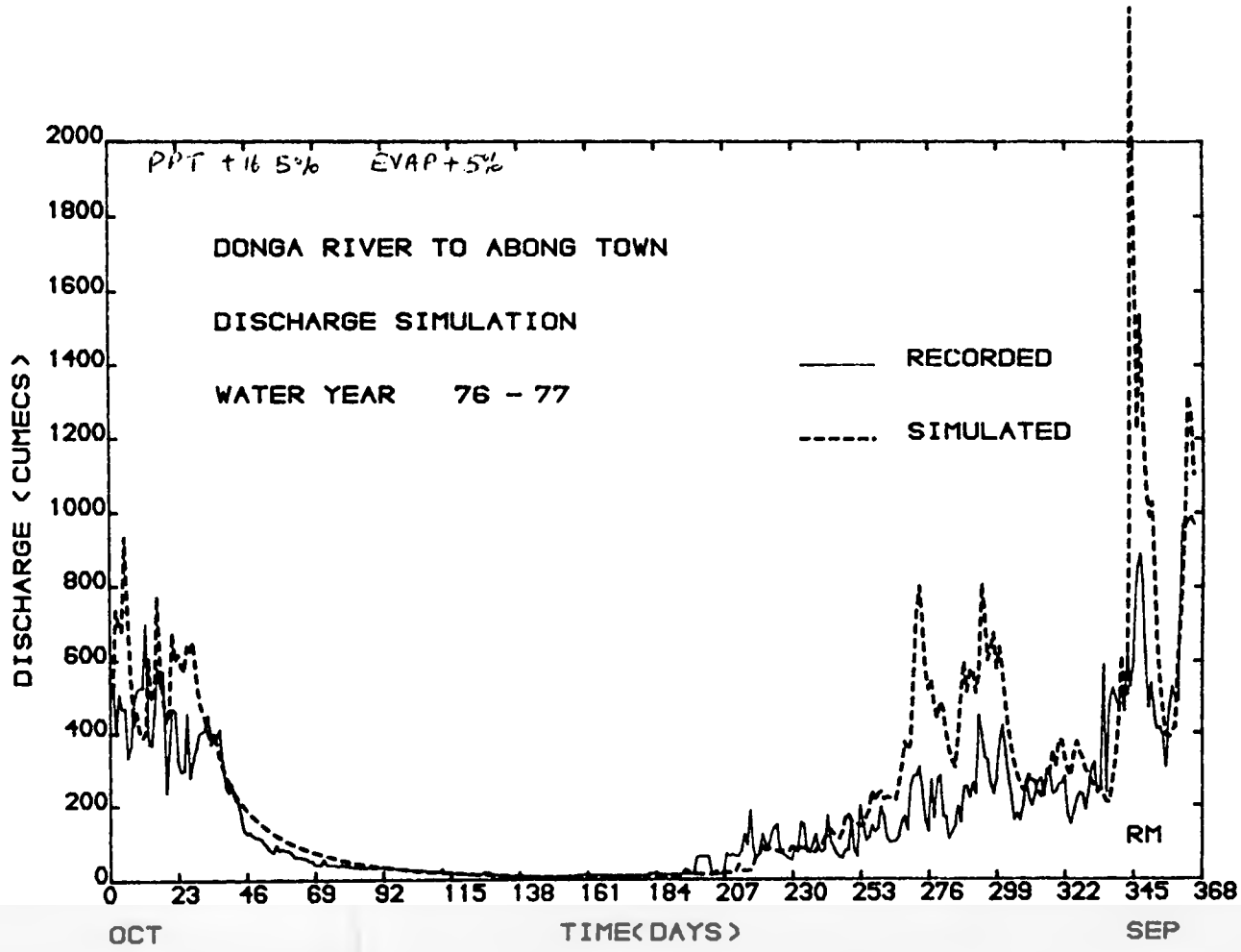


Fig. (c)

PRECIPITATION + 16.5%
EVAPORATION + 5.0%



USER: CHAV61 PLOTFILE: PFILE ENTRY NO. 80465 QUEUED ON 24/11/88 AT 16:36 PLOTTED ON 25/11/88 AT 14:58

Fig. (d)

PRECIPITATION + 16.5%
EVAPORATION + 7.5%

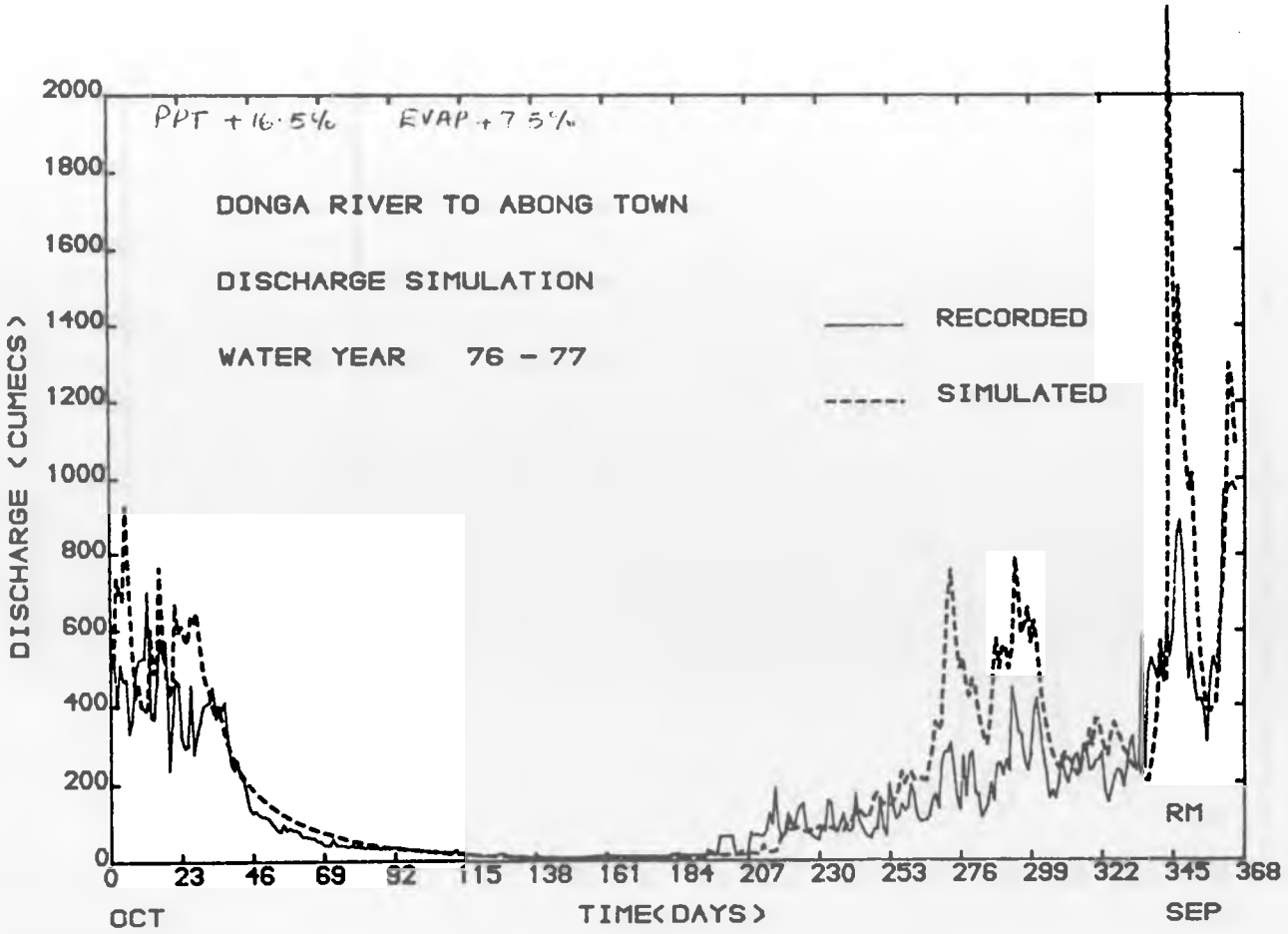


Fig. .(e)

PRECIPITATION + 16.5%
EVAPORATION + 10%

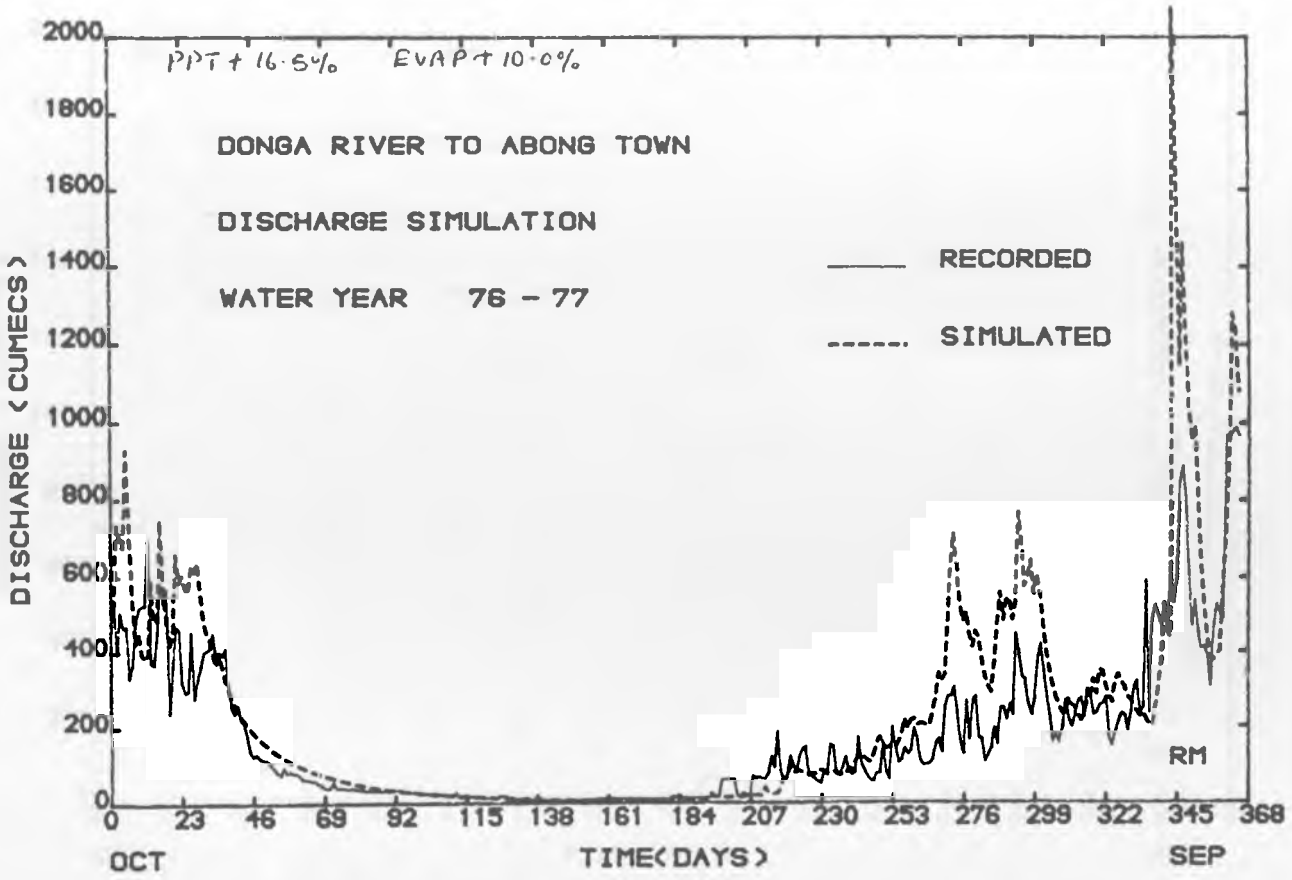
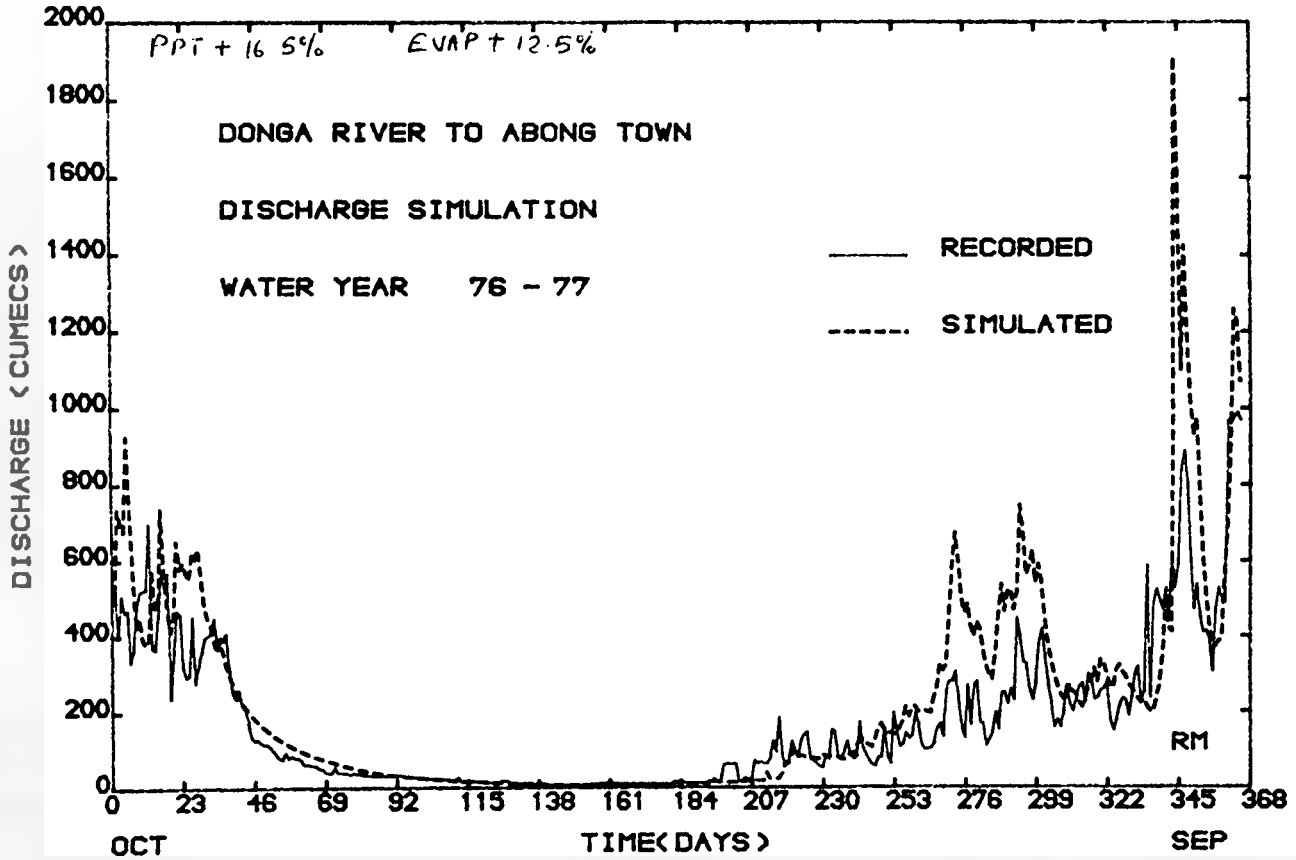


Fig. (f)

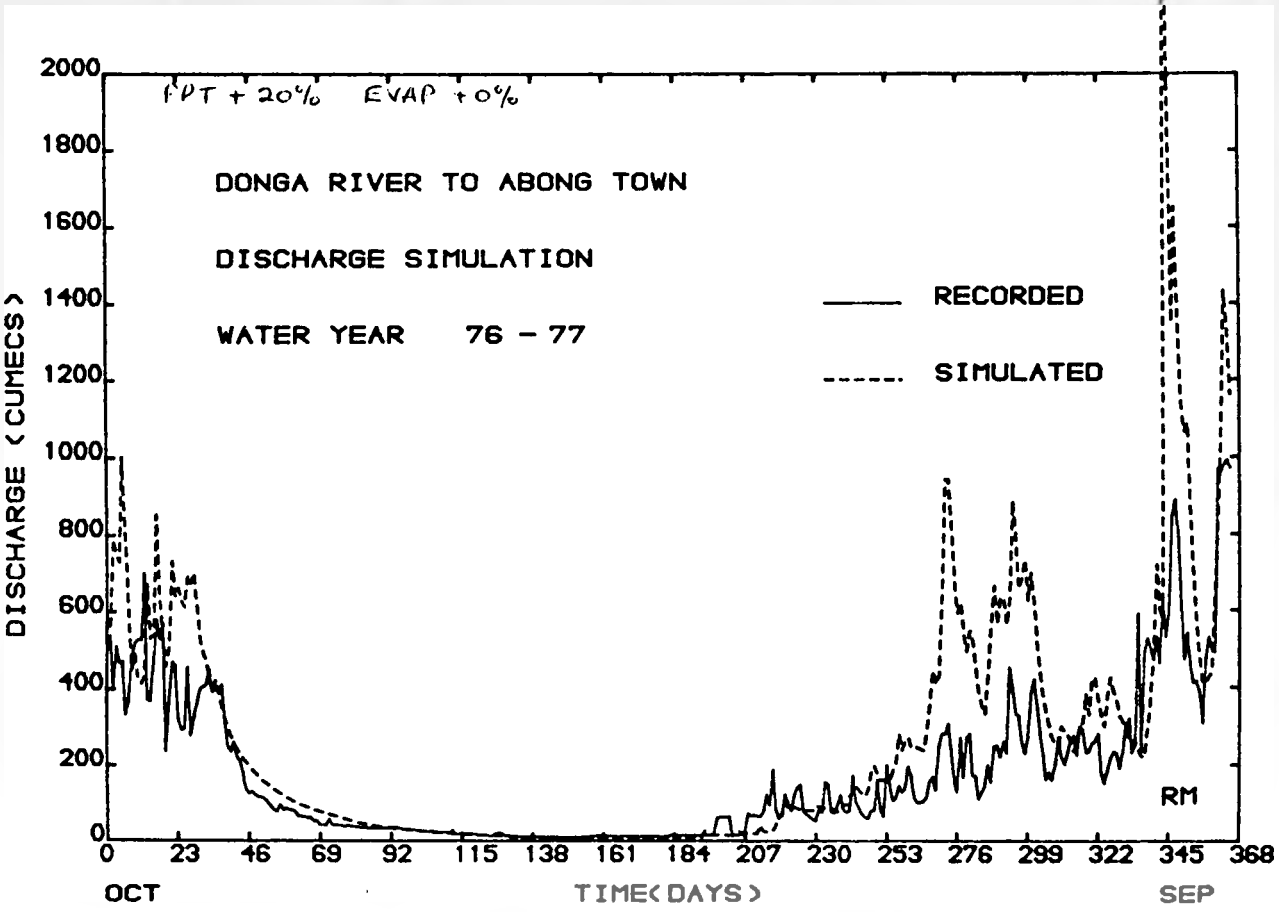
PRECIPITATION + 16.5%
EVAPORATION + 12.5%



USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 74257 QUEUED ON 17/10/88 AT 17:07 PLOTTED ON 20/10/88 AT 08:27

Fig. 5.2 (a)

PRECIPITATION + 20%
EVAPORATION + 0%



USER: CHAV61 PLOTFILE: PFILE ENTRY NO. 60344 QUEUED ON 24/11/88 AT 15:53 PLOTTED ON 25/11/88 AT 10:41

Fig. (b)

PRECIPITATION + 20%
EVAPORATION + 25%

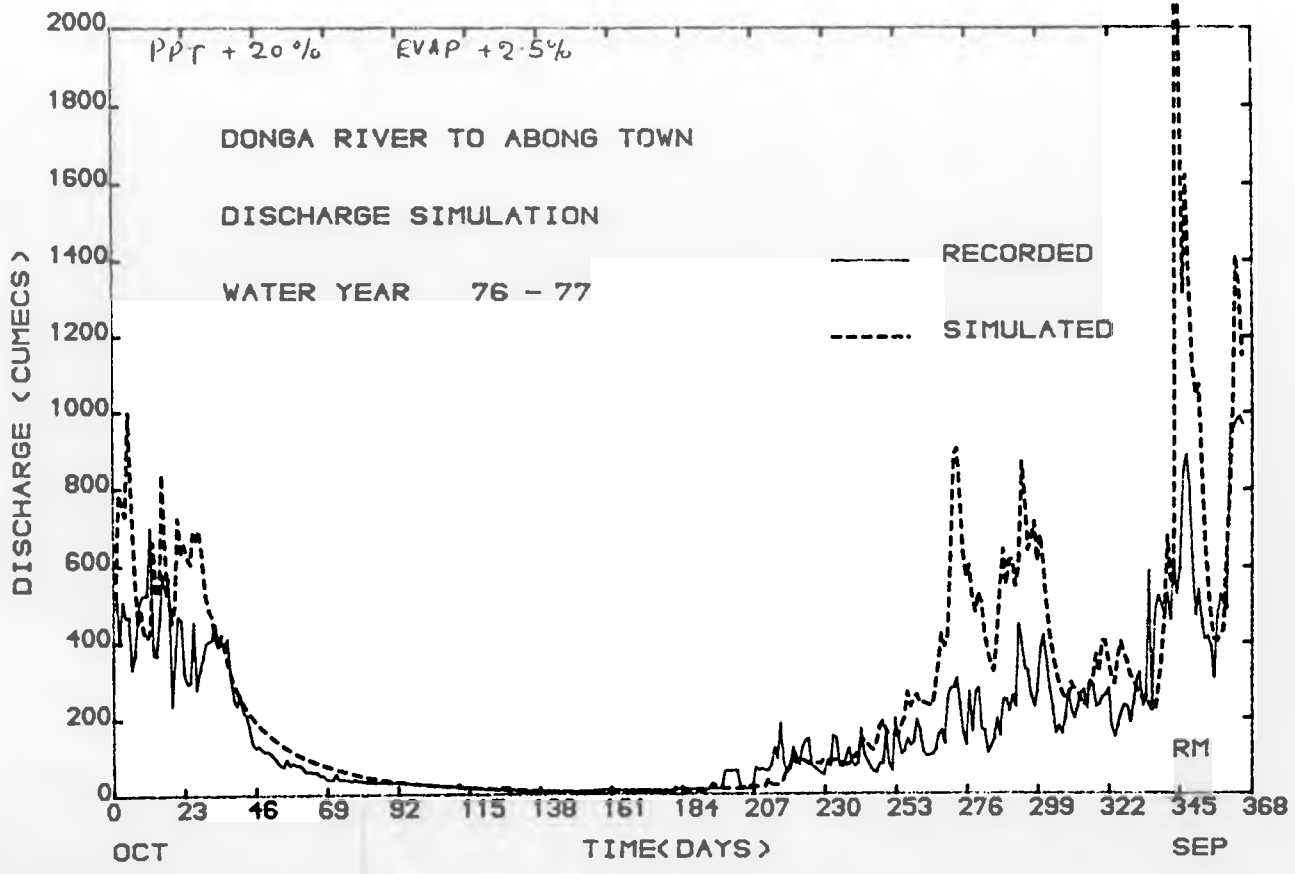


Fig. (c)

PRECIPITATION + 20%
EVAPORATION + 5.0%

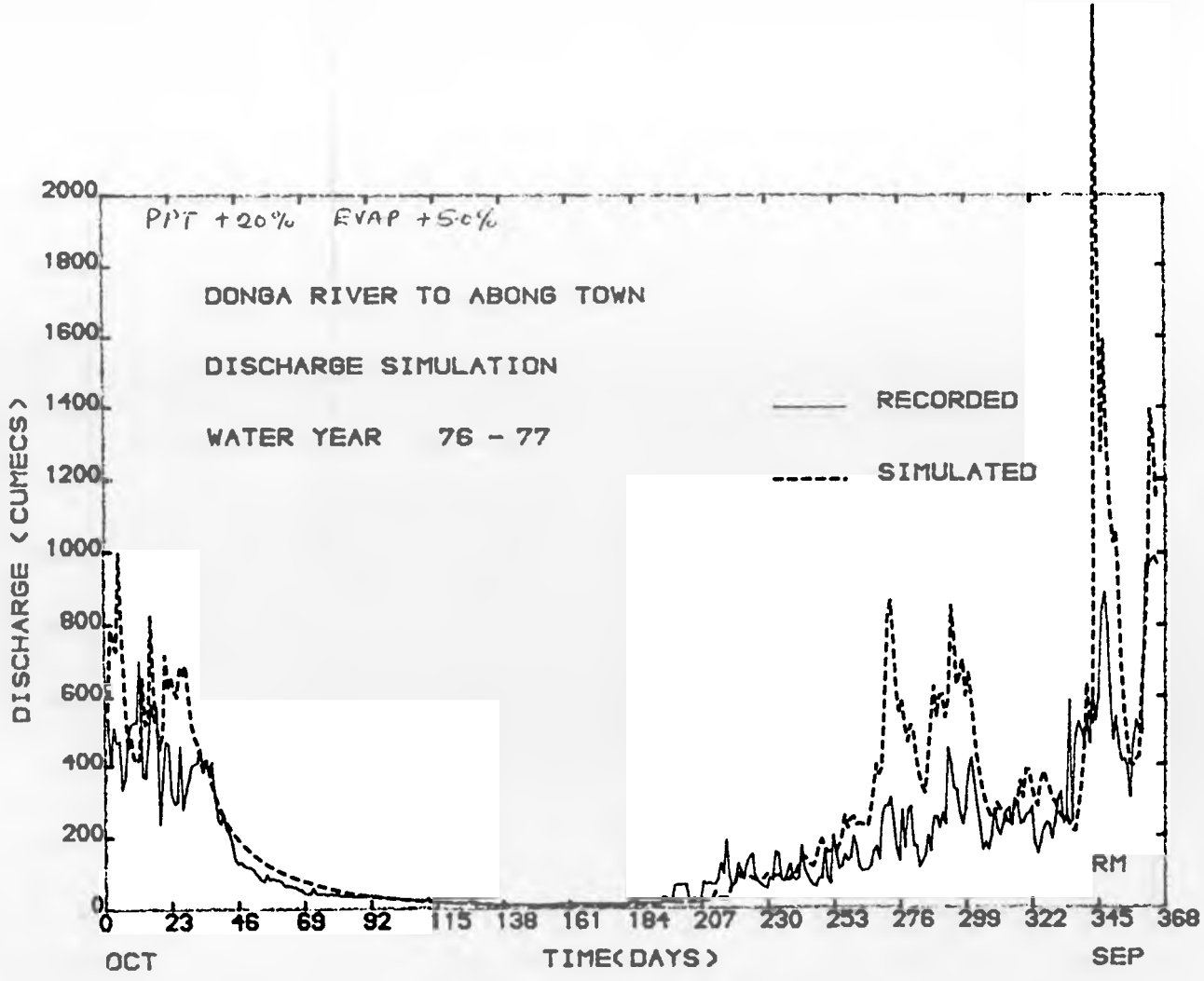
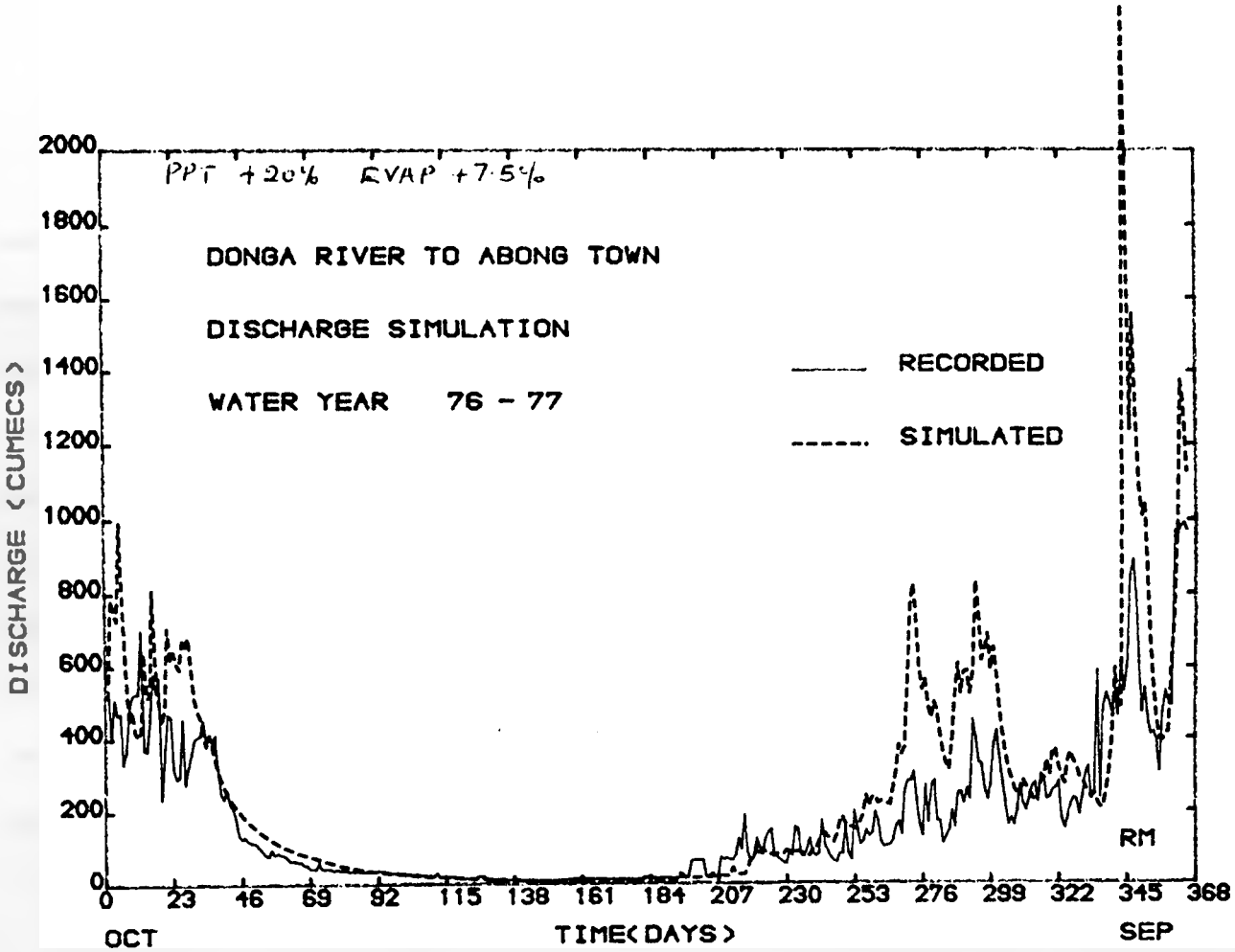


Fig. (d)

PRECIPITATION + 20%
EVAPORATION + 7.5%



USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 80269 QUEUED ON 24/11/88 AT 12:51 PLOTTED ON 24/11/88 AT 16:52

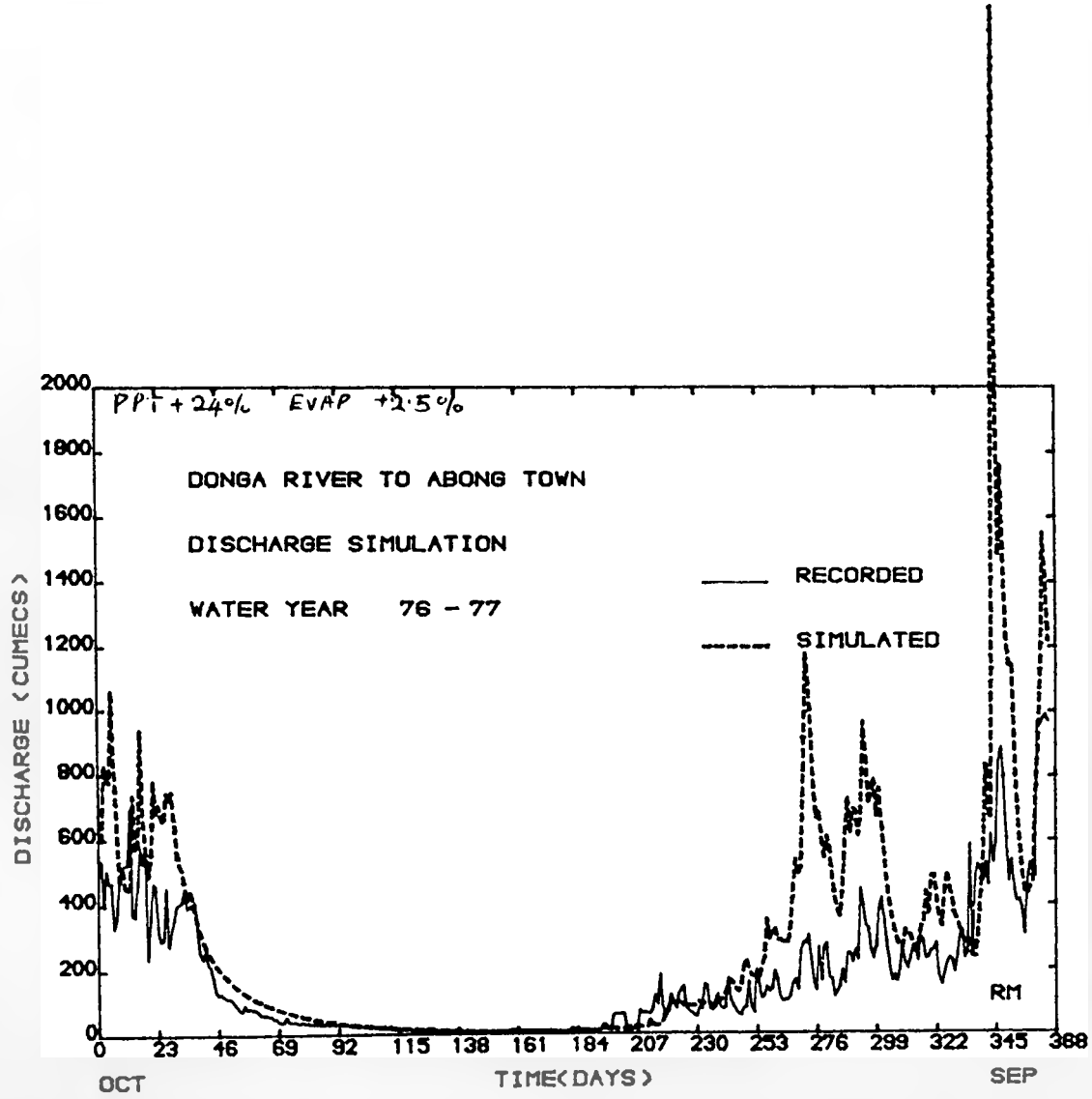


Fig. (a)
PRECIPITATION + 24%
EVAPORATION + 2.5%

USER: CHAV81 PLOTFILE: PFILE ENTRY NO. 74007 QUEUED ON 15/10/88 AT 11:46 PLOTTED ON 18/10/88 AT 14:35

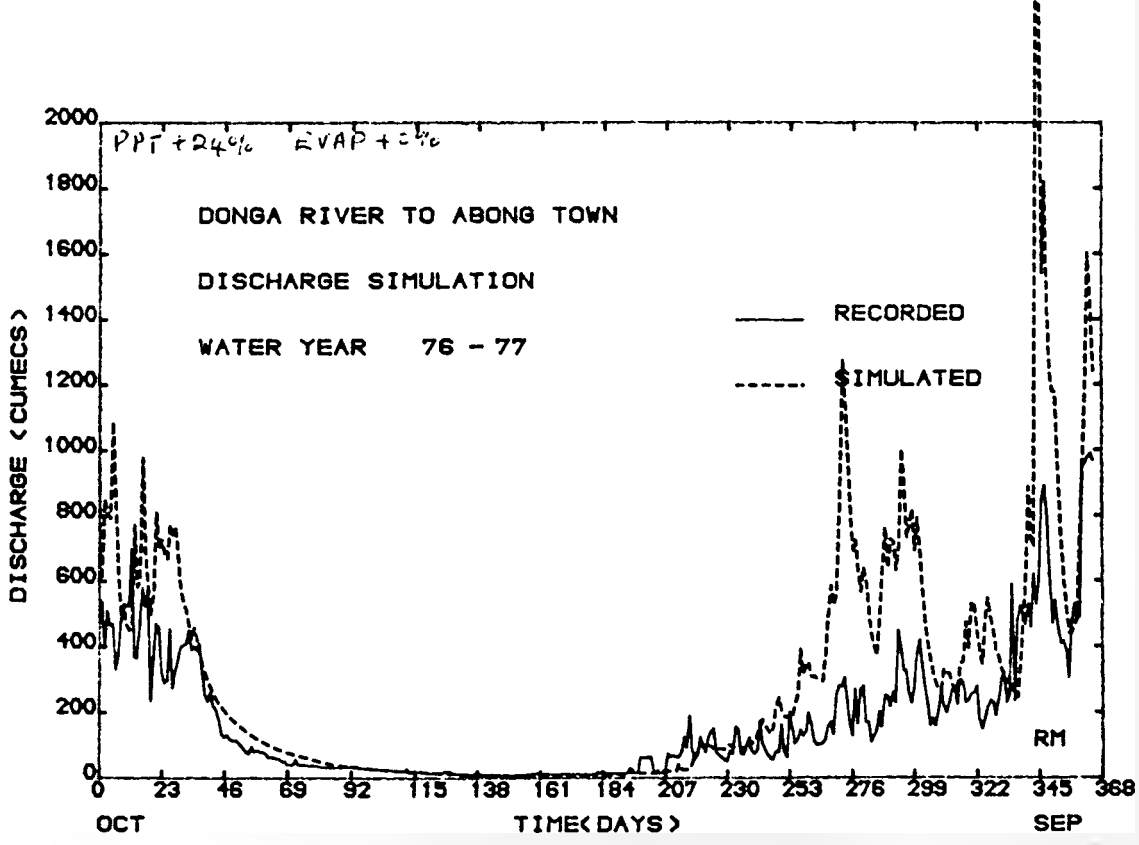
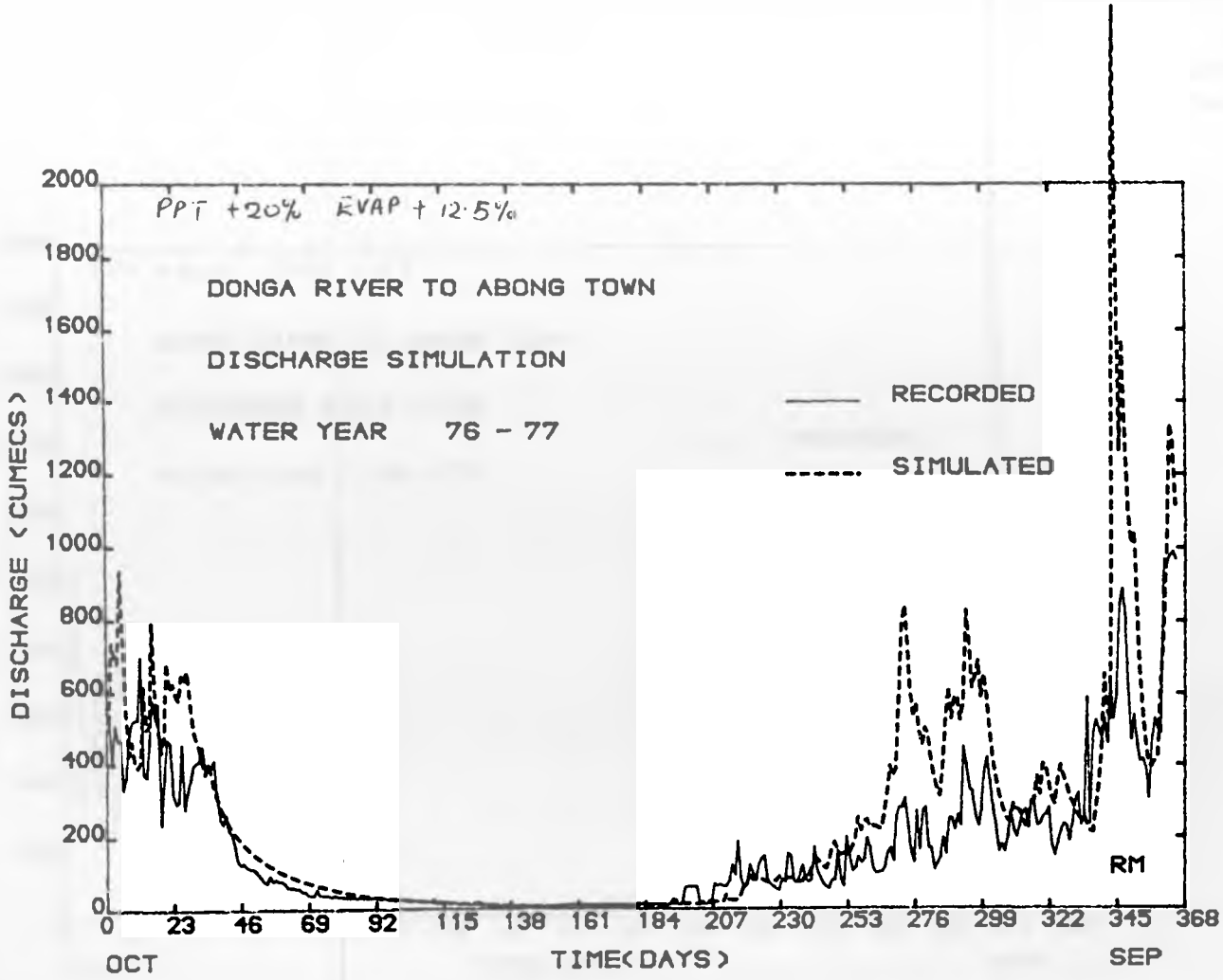


Fig. (f)

PRECIPITATION + 20%
EVAPORATION + 12.5%



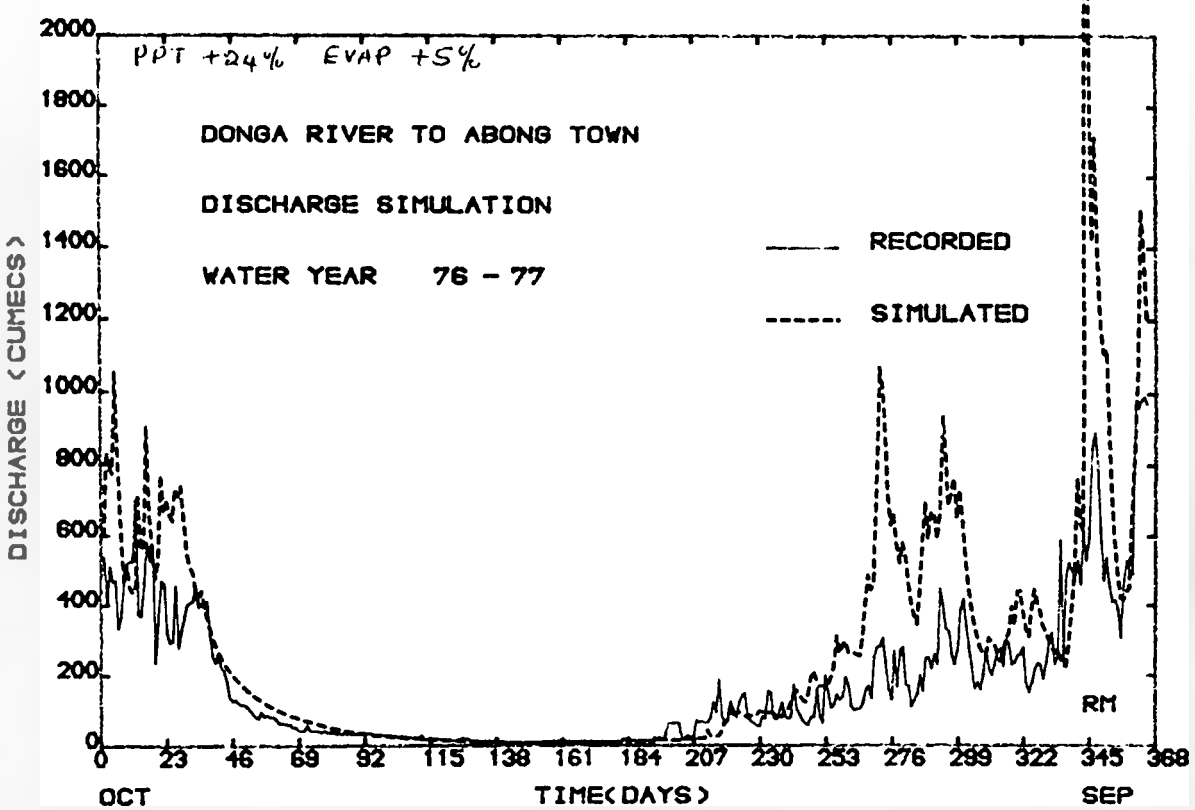


Fig. (b)

PRECIPITATION + 24%
EVAPORATION + 5.0%

Fig. ... (c)

PRECIPITATION + 24%
EVAPORATION + 7.5%

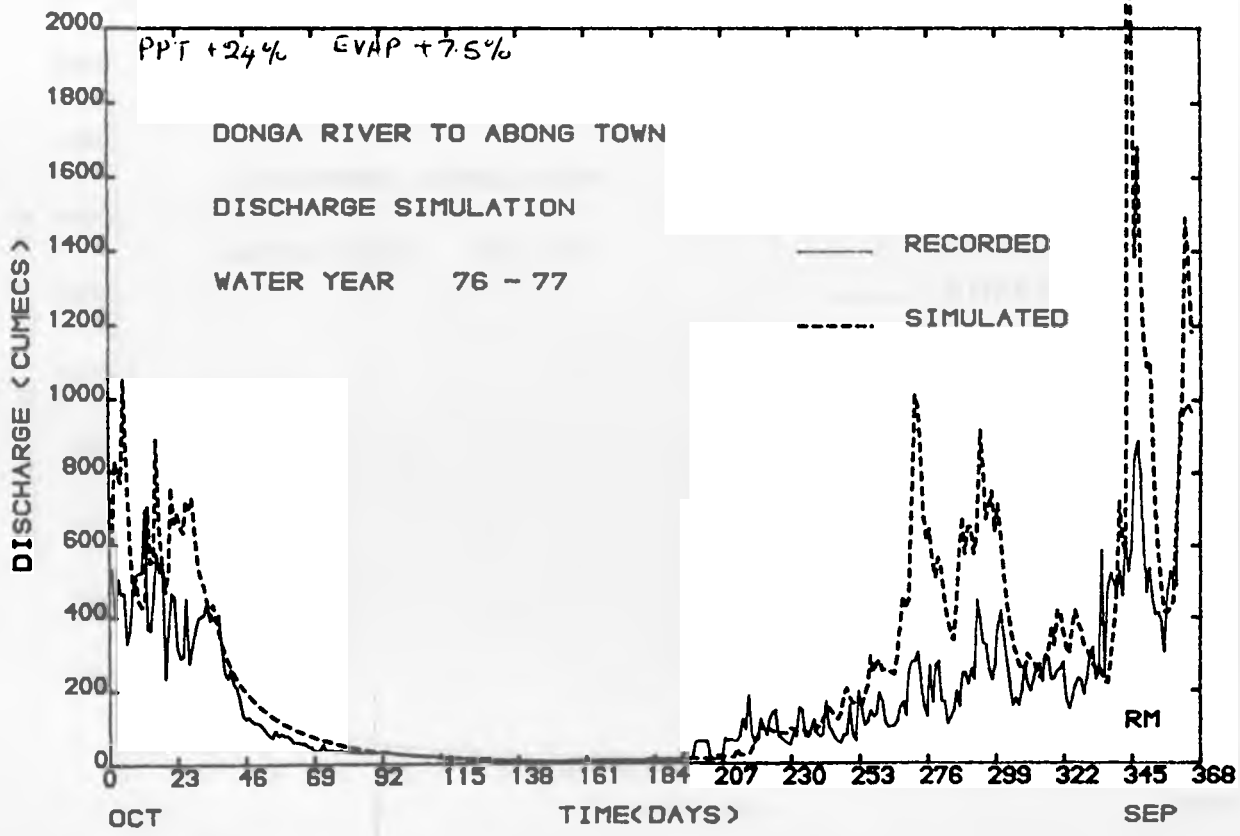
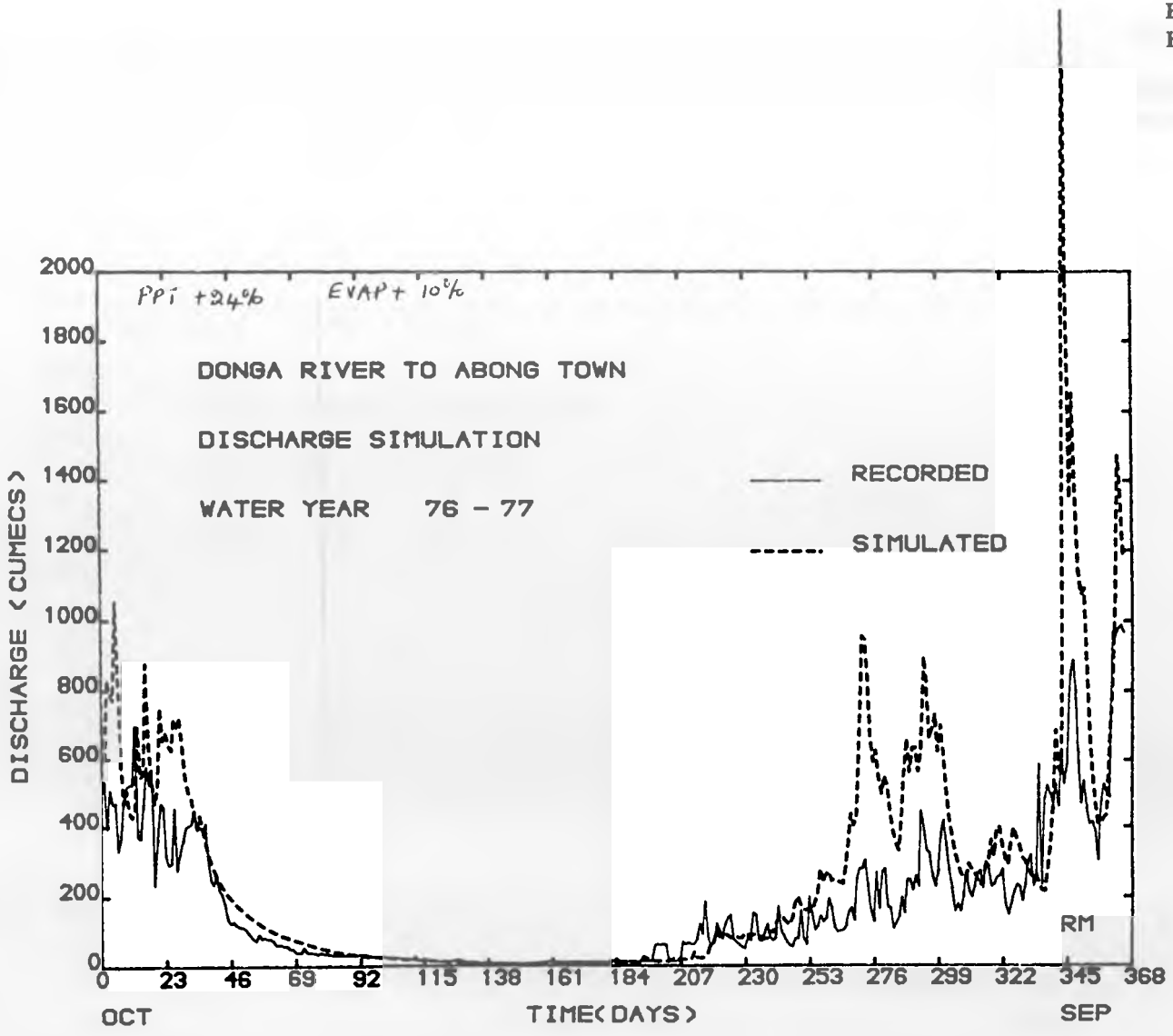


Fig. (d)
PRECIPITATION + 24%
EVAPORATION + 10%



USER: CMAV61 PLOTFILE: PFILE ENTRY NO. 80338 QUEUED ON 24/11/88 AT 15:05 PLOTTED ON 25/11/88 AT 10:07

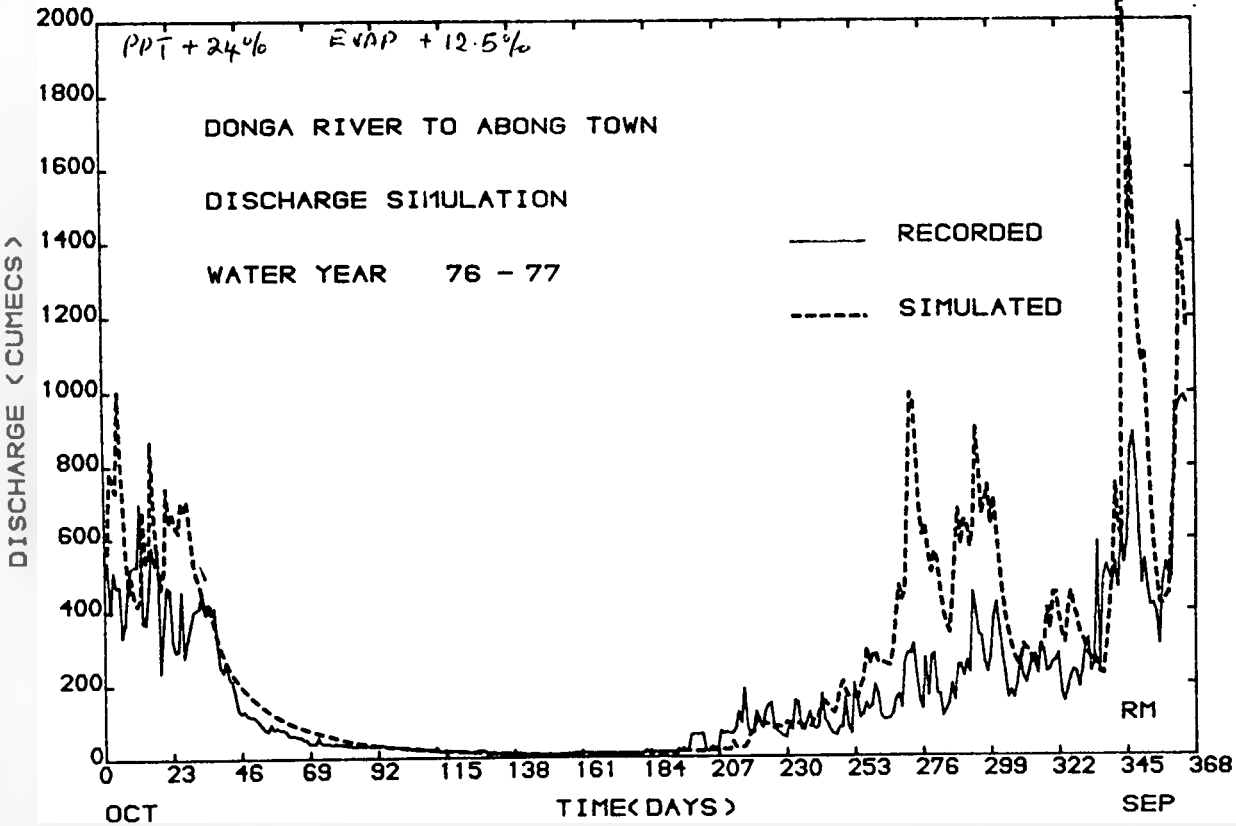
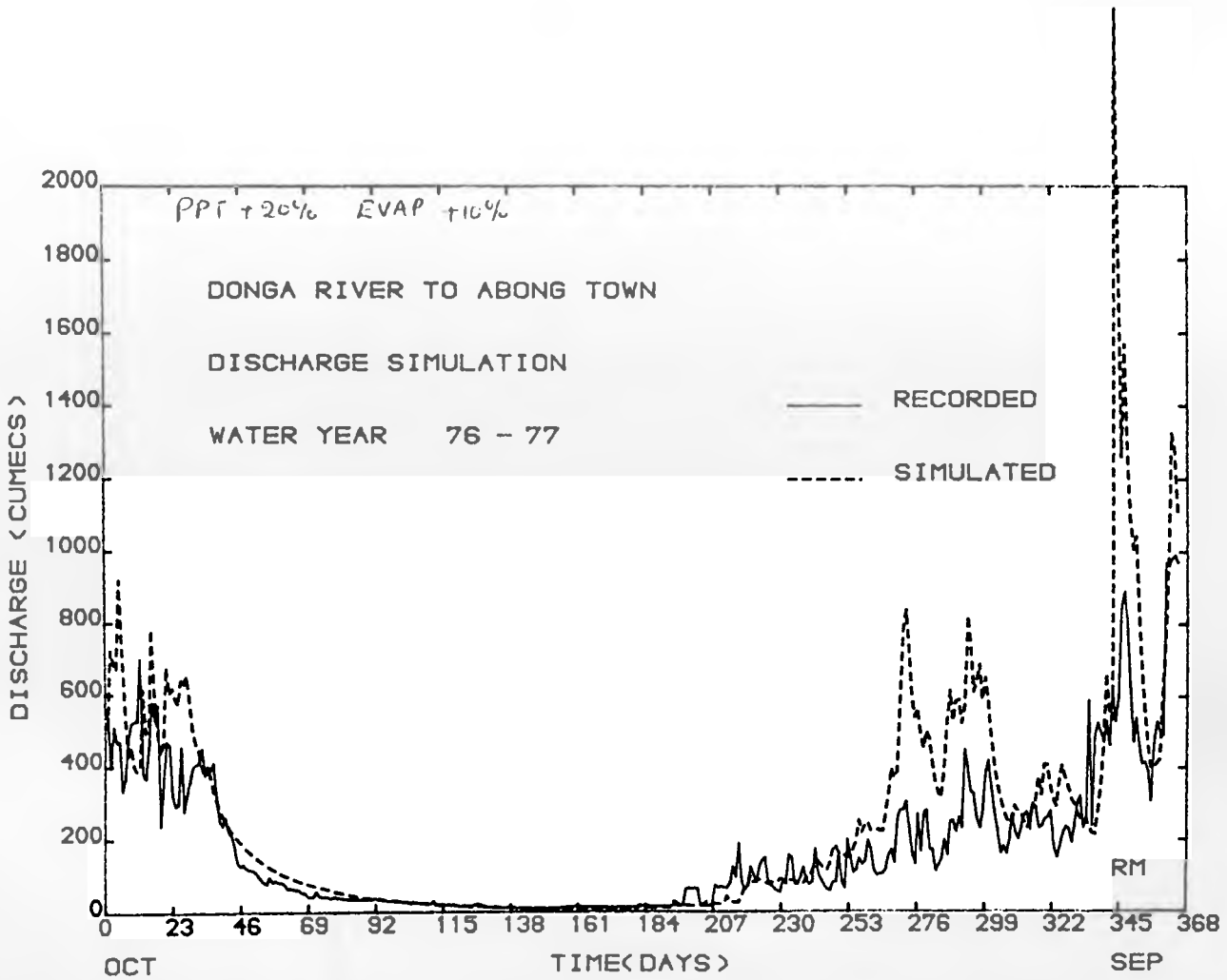


Fig. [illegible] (e)

PRECIPITATION + 24%
EVAPORATION + 12.5%

PRECIPITATION + 20%
EVAPORATION + 10%



CHAPTER 9

CHAPTER 9: DONGA BASIN SEDIMENT ANALYSIS

In this chapter, the magnitude of the sediment problem in the Donga Basin, with particular reference to the sedimentation in proposed reservoirs is assessed. In carrying out this objective, the chapter focusses on the discussion of the response of sediment erosion and deposition obtained from trial simulations involving appropriate changes in the magnitude of climate and landuse parameters in the Donga Basin. Section 9.1 outlines the need for accurate predictions of sediment erosion, yields, and deposition in a river basin with a focus on reservoir sedimentation . In section 9.2, the sediment problem in the Donga Basin is discussed with an assessment of previous work. The results of the calibration of the erosion model are presented in section 9.3. The simulated results as obtained from tests involving changes in landuse, reservoir-induced and globally induced climatic change are presented, analysed, and discussed in sections 9.4, 9.5, and 9.6 respectively. Section 9.7 concludes the results of the research findings in this chapter. Prior to the discussion of sediment process theory, an outline highlighting the need for a computer system in the accurate prediction of sediment erosion in river basins and deposition patterns in reservoir systems is first discussed.

9.1 Introduction

The sediments entering reservoirs from river basins emanate from croplands, pastures, rangelands, forestlands, streambanks, road dishes, urban sites and bare surfaces. The inflow of sediments into reservoirs may be inevitable but sedimentation may be accelerated by poor management of soils in the river basin. Live storage capacities are severely reduced, despite the large and sometimes wasteful dead storage spaces specifically allocated to reservoir sedimentation by engineers. Other sediment effects in reservoirs include temperature changes induced by turbidity and increased growth of water consuming plants due to sediment related nutrients. The net effect of these last two sediment damages on water storages, are relatively small, difficult to quantify, and not treated herein.

To minimise the economic impact of the sediment problem, accurate predictions of the catchment sediment yields and reservoir sedimentation patterns are important in surface water storage designs. The sediment erosion and delta formation models of the River Basin Management System are used herein in accomplishing this. Results of sediment yields obtained from empirical methods and field methods are also used to calibrate, verify, and validate the models.

It is important to note that, in predicting sediment yields and deposition patterns, the river basin management system uses the flow responses from its two water balance

components, the watershed and channel routing models. Prior to describing the erosion and delta simulation models, a brief description of the interacting sediment processes in a river basin, inclusive of reservoirs is first presented.

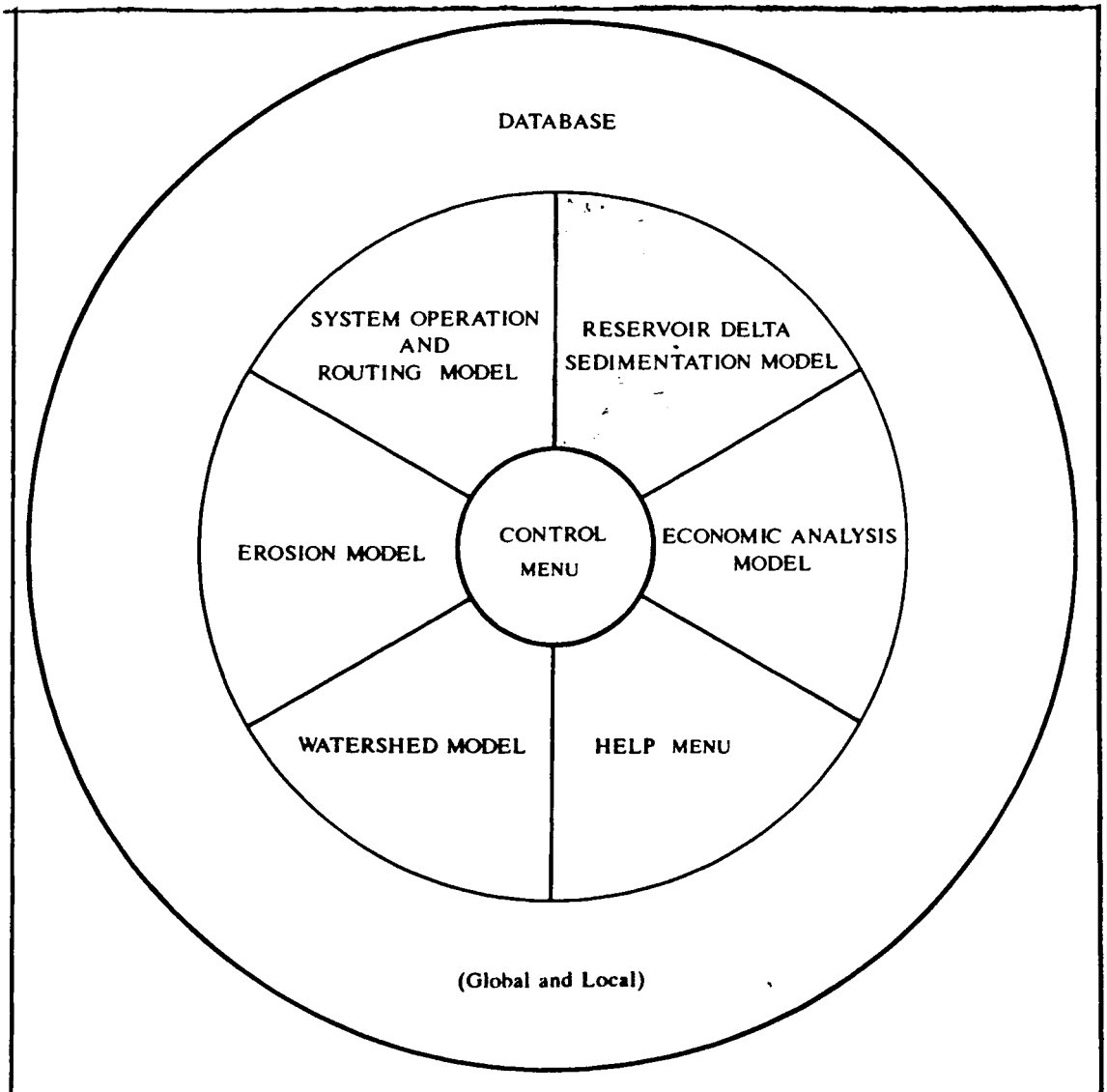


Fig. 9 RIVER BASIN MANAGEMENT SYSTEM

This system is available for use on both micro and mainframe(VAX) computers.

9.2 Sediment Yield of the Donga Basin.

Sediment studies on the Donga river and basin dates back to the late 1950's when NEDECO (1959) took sediment samples in the Donga at or near the confluence with the Benue, at a place called Nyankawola. They estimated the silt loads at the time to be 1.1 million tonnes annually and observed that water from the R.Donga was much clearer than that of the Benue and that a strip of clearer water, obviously entering from the Donga, was observed at some distance along the Benue, beyond the Donga-Benue confluence. In recent times however, the reverse has been the case as visual and measured observations confirm. The River Donga now carries a much higher silt content than the Donga. Plates D, E, and F shows the severe erosion problem in the Donga basin.

MRT (1978) took samples at several locations in the lower Donga including five at Manya reservoir site. In the upper Donga, sediment samples were taken at a number of sites in 1977 and early 1988, by LIDECO (1978). Hudson (1982) reported on soil erosion on the mambilla plateau and sedimentation in the proposed Gembu reservoir. Diyam (1985) took samples and studied the sediment behaviour of most of the Donga basin from 1979-82. UBRBDA (1983) has a continous data collection programme on the whole of the basin.

Nwogute (1987) utilised 1977 suspended sediment measurements in his recent sediment yield determination for the Donga Basin. His methodology involved the use of plots of sediment concentration against discharge, from

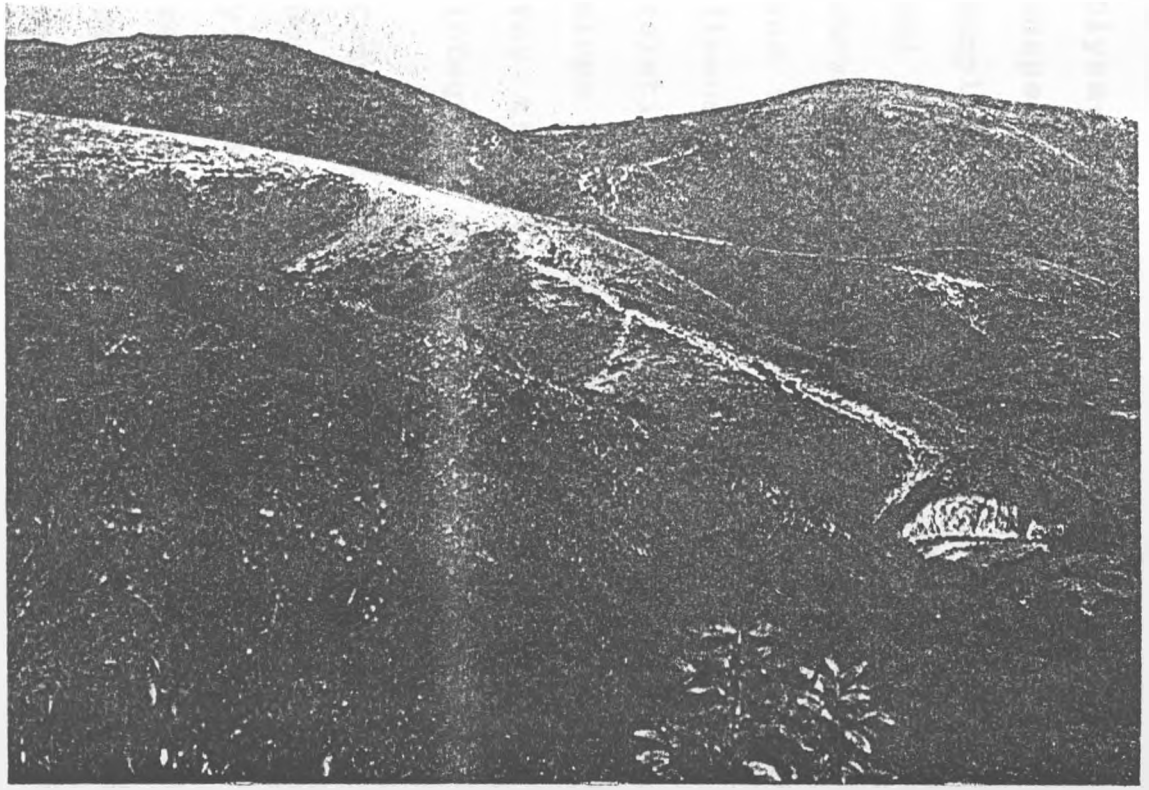


PLATE D SHEET AND RILL EROSION DONGA BASIN

which he deduced the sediment loads at Manya and Donga as 5.41 and 6.15 million tonnes respectively. The respectve annual sediment concentration at these two settlements are 0.59 and 0.62 gm/l. In an earlier study, MRT (1977), using Fleming's relationships estimated the annual sediment at Manya at 7.5 and 8.4 million tonnes at Donga.

In their 1985 study, using 1982 field measurements, Diyam-Binies (1985) plotted the relationships between suspended sediment concentration from depth integrated sampling and discharge for the gauging stations at Manya and Tepkwar as shown in figures 9.1 and 9.2. Figs 9.3 shows sediment/discharge relationships for the River Donga and two tributaries after discharges have been non-dimensionalised by division with mean daily flows. The relationship at Tepkwar is quite well defined and the slope of the line has been used to delineate relationships for the other locations in the basin. Again depth integrated samples have been utilised.

Computation of the annual suspended sediment load on the Donga at Tepkwar, near Gembu, and at Manya by Diyam (1985) yielded values of 3.6 and 8 million tonnes per year respectively which represents an average respective rates of 1860 and 890 tonnes/year/sq km. This value has been computed by combining the sediment discharge relationship on figures 9.1-9.3 with the average flow duration curve for the Tepkwar flow record of fig 9.4 (Diyam, 1985). The duration curves are based on daily flow data over the period 1977/80 for Tepkwar and the 8 years, 1963-65, 1967, and 1978-81 for Manya. In a similar way, the average

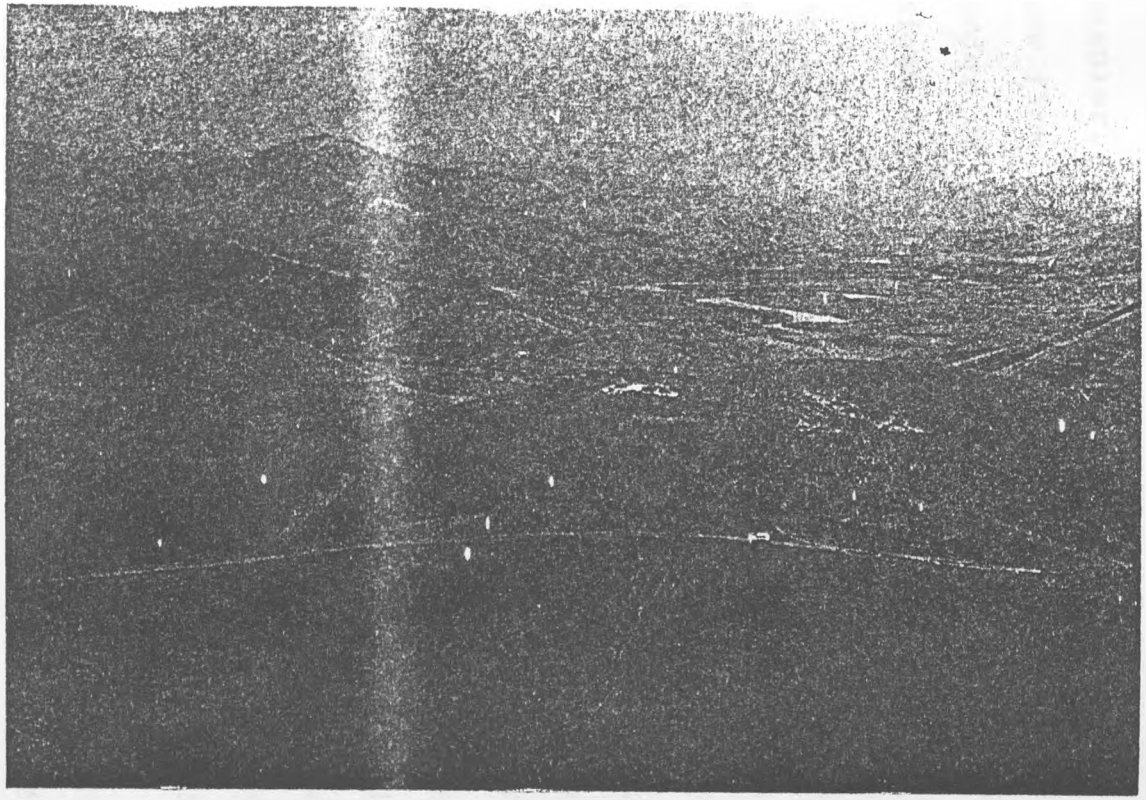


PLATE E GULLY AND RILL EROSION DONGA BASIN

annual suspended sediment yield in each of the rivers Sumsum and Nghu (reservoir sites), was estimated at 1200 tonnes/year/sq km.

Despite the fact that bed load measurements are difficult to make and hence susceptible to errors, attempts were made by LIDECO (1978) in the upper Donga using a Shamov sampler. Results of their study indicate that during the low flow season, the bed load in the upper Donga is of the order of 2-5% of the suspended load. Studies on the R. Niger by Dekker (1961) put this ratio at 5-7%.

An alternative approach to estimating bedloads is to use empirical formulae which relates particle size and density to the hydraulic characteristics of the channel. MRT (1978) used the Meyer-Peter-Mueller relation together with particle size data from the Manya reservoir site area and estimated bed load to be 5.25% of suspended load. Nwogute (1987) investigated bedloads and developed bedload curves at Manya and Donga on the Donga. Using these curves together with the Meyer-Peter-Mueller equation, and the flow duration curve for 1977, he obtained bedload to suspended load ratios of 5.25 and 2.71% respectively.

Diyam-Binies (1985) applied the Ackers-White (1980) equation, to particle size data from bed samples taken at Tepkwar cableway and the Manya-Nyivu bridge. Results indicated that bedload might be a much higher percentage of suspended load than suggested by other studies. Diyam-Binies stated that more samples are needed to confirm this observation, particularly as the particle size

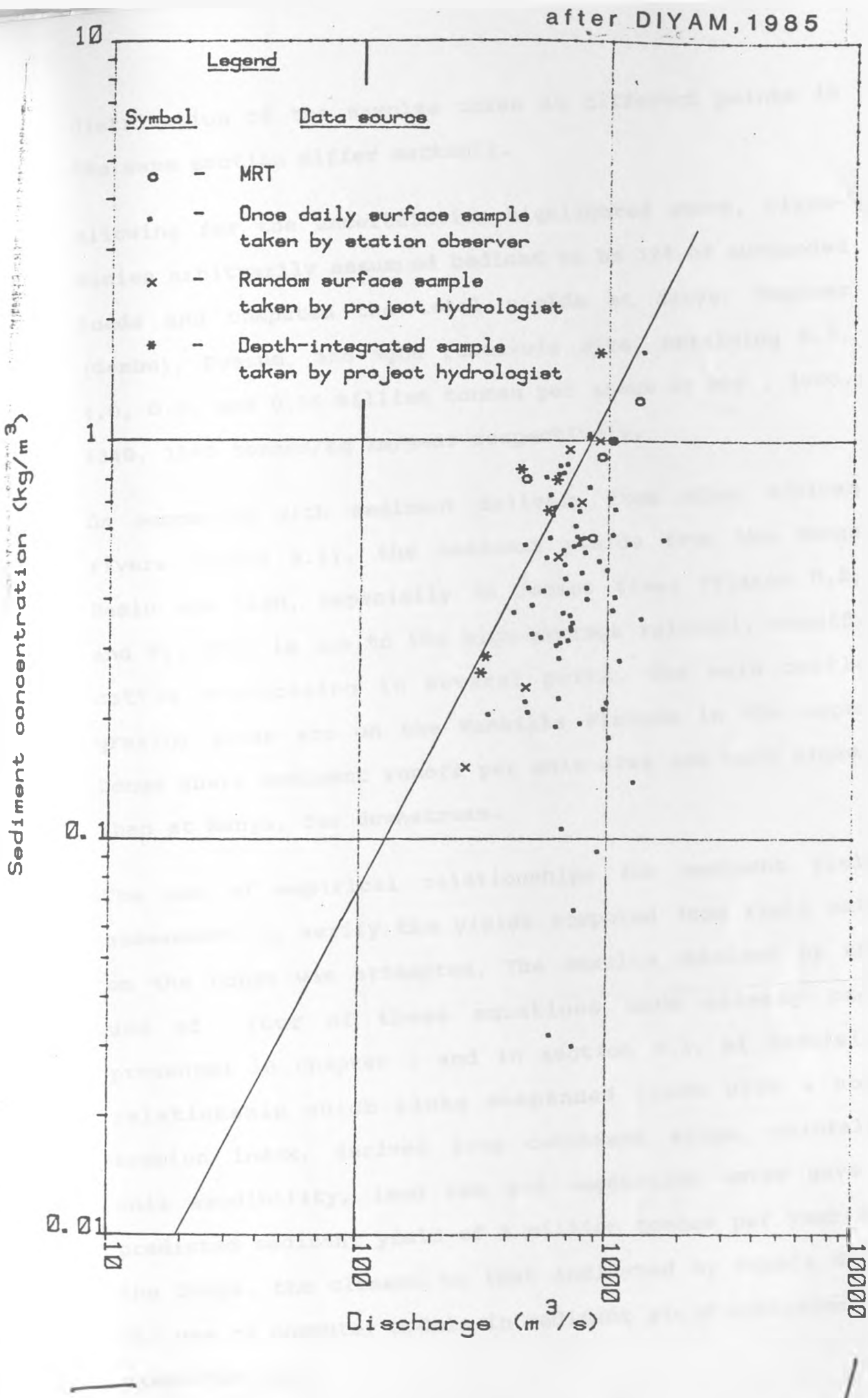


FIG 9.1 RIVER DONGA AT NYIVU
SEDIMENT/DISCHARGE RELATIONSHIP

distribution of the samples taken at different points in the same section differ markedly.

Allowing for the uncertainties highlighted above, Diyam-Binies arbitrarily assumed bedload to be 12% of suspended loads and computed the total yields at Manya, Tepkwar (Gembu), Sumsum, and Nghu reservoir site, obtaining 9.0, 4.0, 0.2, and 0.06 million tonnes per annum or 890 , 2080, 1340, 1340 tonnes/sq km/year respectively.

On comparing with sediment delivery from other African rivers (table 9.1), the sediment yields from the Donga Basin are high, especially in recent times (Plates D,E, and F). This is due to the high average rainfall, runoff, cattle overgrazing in several parts. The main cattle grazing areas are on the Mambilla Plateau in the upper Donga where sediment runoff per unit area are much higher than at Manya, far downstream.

The use of empirical relationships for sediment yield assessment to verify the yields computed from field data on the Donga was attempted. The results obtained by the use of four of these equations have already been presented in chapter 3 and in section 9.3. Al Kadhimi's relationship which links suspended loads with a soil erosion index, derived from catchment slope, rainfall, soil erodibility, land use and vegetation cover gave a predicted sediment yield of 8 million tonnes per year for the Donga, the closest to that indicated by sample data. The use of computer models in sediment yield assessment is presented next.

R. DONGA AT TEPKWAR

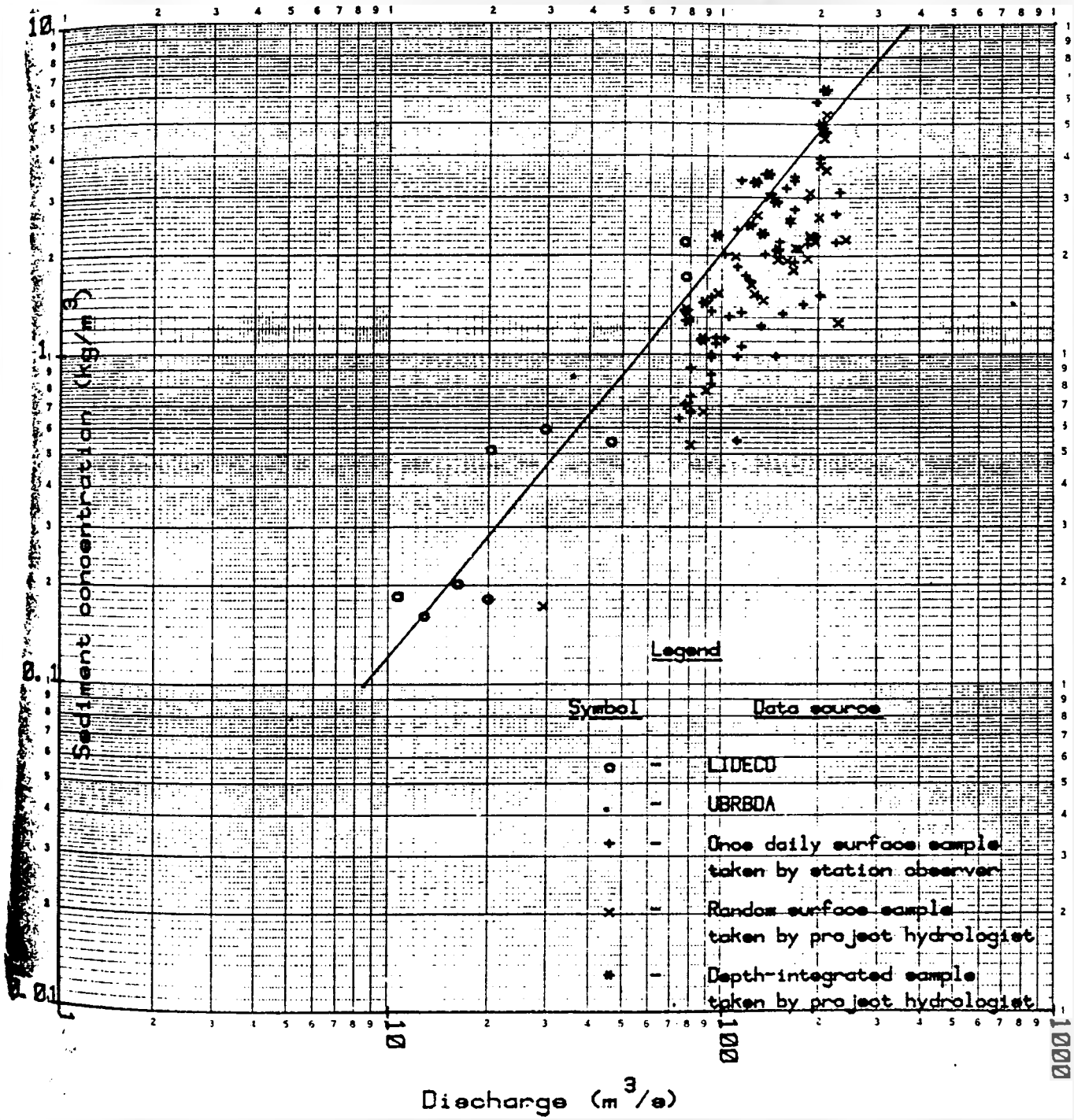


FIG 9.2 R. DONGA AT TEPKWAR
DISCHARGE / SUSPENDED SEDIMENT
CONCENTRATION RELATIONSHIPS

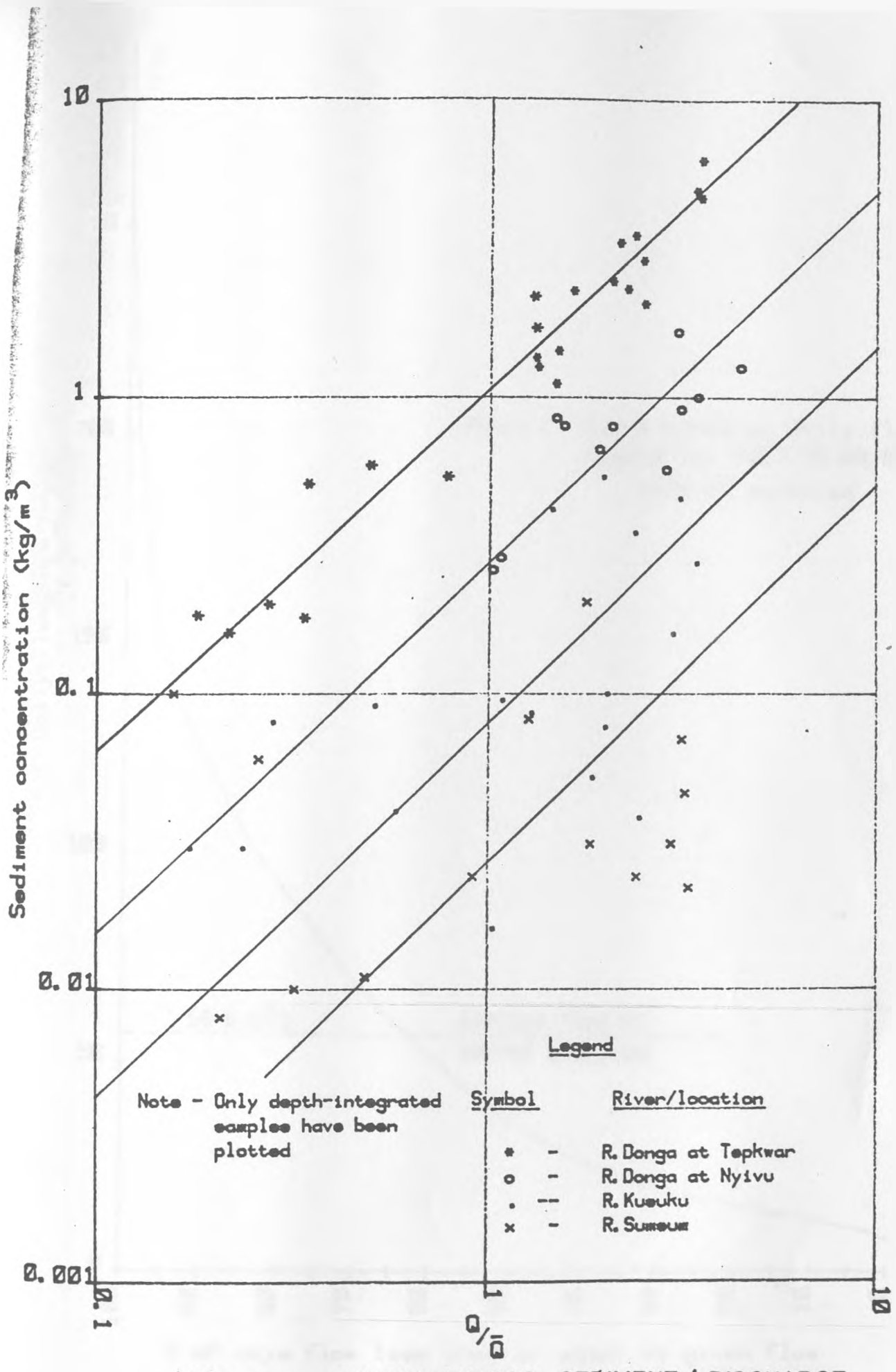


FIG 9.3 DIMENSIONLESS SEDIMENT / DISCHARGE RELATIONSHIPS FOR THE DONGA BASIN

Figure 9.

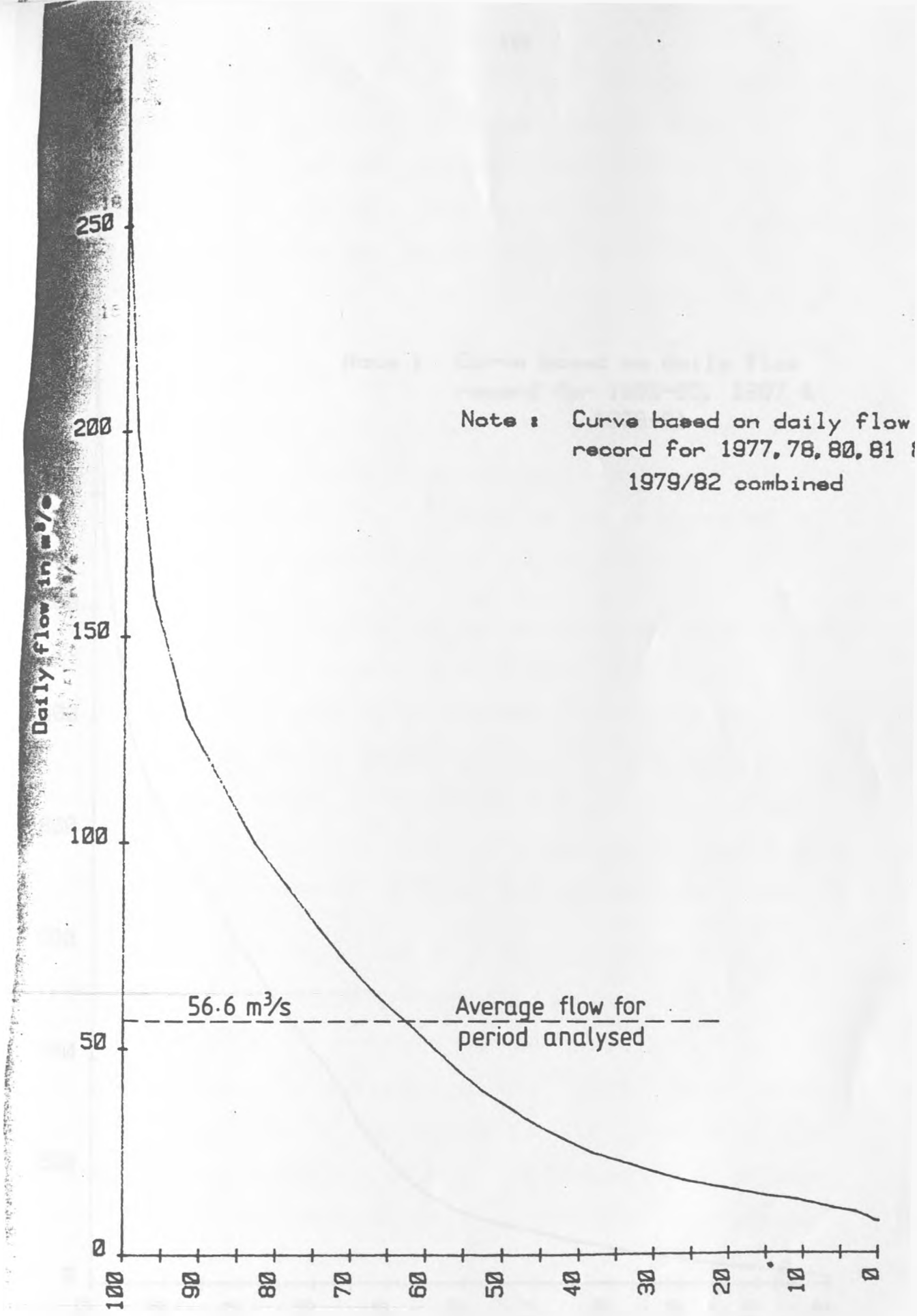


FIG 9.4a R.DONGA AT TEPKWAR: FLOW DURATION CURVE

Figure 5.

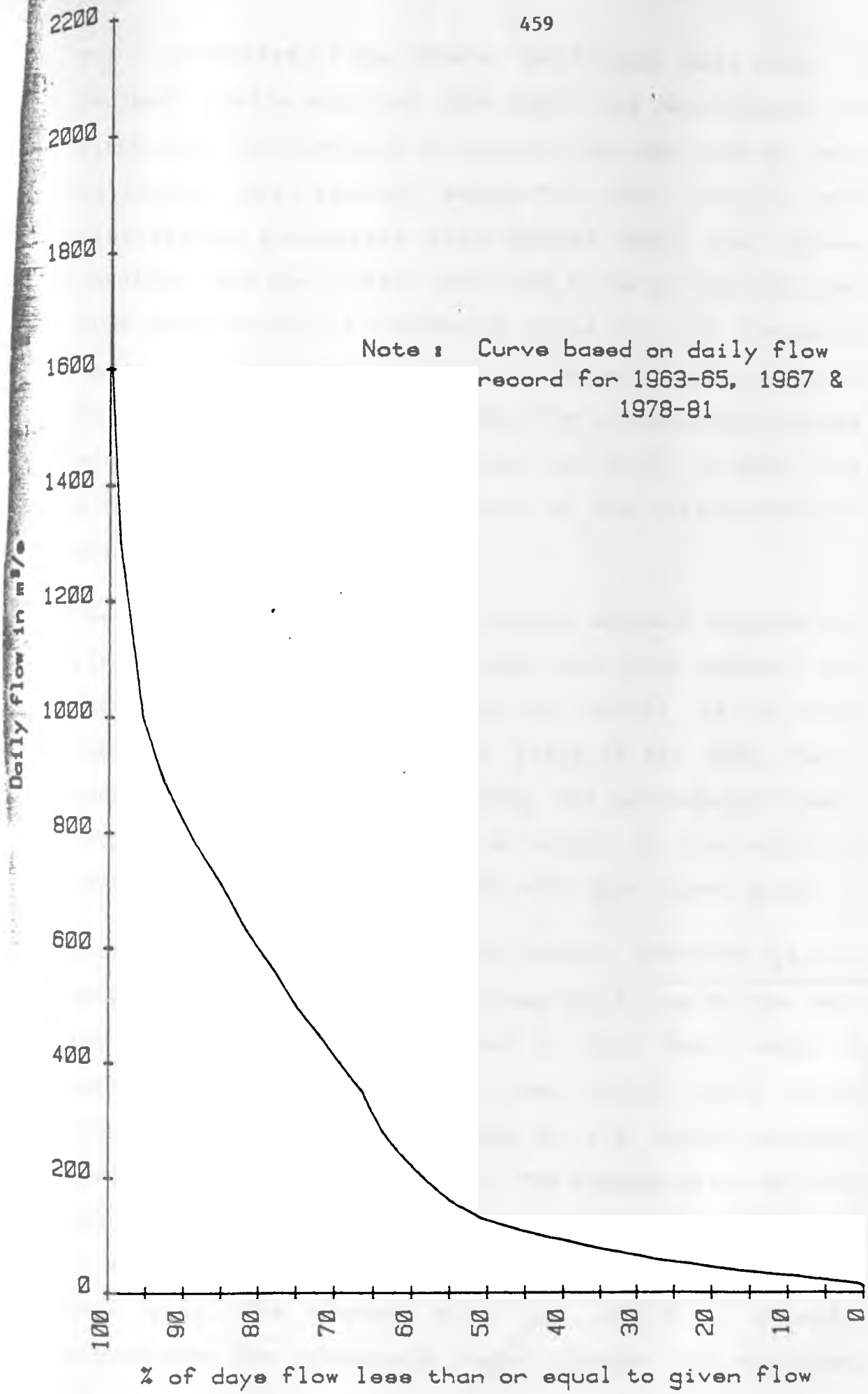


FIG 9.4b RIVER DONGA AT NYIVU
FLOW DURATION CURVE

9.3 Calibration of the Erosion Model-Donga Basin Case

Sediment yields obtained from the field measurements of discharges, and sediment discharges has been used to test calibrate the erosion model.The Rill, Clift, and disturbance parameters were varied until the values obtained from the erosion model are close to that obtained from measurements. A calibrated yield of 2,200 tonnes/sq km/year was obtained compared to a recorded yield of 2,080 tonnes/sq km/ year (Diyam, 1985). The corresponding values of rill, clift, and disturbance are 0.013 ,0.9940, and 0.0001 respectively. The results of the calibration run are presented in table 9.2.

The results indicate that the maximun sediment erosion and yields in a year occurs in months with peak rainfall and flows. From the calibration run of 1976-77, it is clear that over 95% of the sediment yield of the Donga Basin occur in the months of june/july and september/october. Interestingly, over 80% of this occurs in the month of September alone, being the month with the highest peaks.

From the table 9.3, the low annual sediment yields obtained for 1977-78 (5.3 tonnes/sq km) is due to the very low peaks produced or observed in that year, being a drought year. The slightly higher runoff peaks which occured in 1981-82 is responsible for its higher sediment yield value (447 tonnes/ sq km). The highest value of 2200 tonnes/sq km was obtained for the simulation of the 1976-77 water yields due to the sharp flow peaks experienced that year. The figures 8.4, 8.5, and 8.6 clearly illustrates the hydrograph peaks recorded and simulated

Table 9.1 Typical sediment loads in African rivers

River	Location	Catchment Area km^2	Suspended and washload tonnes/km^2/yr	Bed load tonnes/km^2/yr	Total sediment load tonnes/km^2/yr	Region	Data source
Mayo Kereng	–	0.066	165	475	640	N. Cameroun	Touchebeuf de Lussigny (1961)[33]
La Maggia	–	0.055		400		Niger	"
Le Banian	–	2.5		12		Madagascar	"
Mayo Boulore	–	3.75	4.5			N. Cameroun	"
Tero	–	32	650			Dahomey	"
Teloua	Razelmamaoulmi	1170	25			Niger	"
Benue	Kinada	64,000	21		22	N. Cameroun	"
Benue	Ouro Boki	104,000	39		41	Nigeria	"
Faro	Kossel	27,000	100		105	N. Cameroun	"
Logone	Lai	23,300	43			Chad	"
Mangoky	Banian	54,000	100–350			Madagascar	"
Niger	Baro	730,370	12.5	0.8	13.3	Nigeria	NEDECO (1959)[3]
Benue	Yolo	107,220	59.1	3.7	63	Nigeria	"
Benue	Makurdi	288,782	75.8	4.2	80	Nigeria	"
Niger	Shintaku	106,190	30?	16.6	324	Nigeria	"
Niger	Onitsha	1,113,690	31.8	1.8	33	Nigeria	"
Sanaga	Nachtigal	77,000	28			Cameroun	Olivry JC(1980)[34]
Mbam	Goura	42,300	85			Cameroun	"
Tsanaga	Bogo	1535	210			Cameroun	"
Djerem	Mbakaou	–	70			Cameroun	"
Noun	Bamileke	–	600			Cameroun	"
Katsina	Bolang	–	30–1500			Cameroun	"
Katsina	Monkie	–	16–110			Cameroun	"
Benue	Garpau	–	200			Cameroun	"

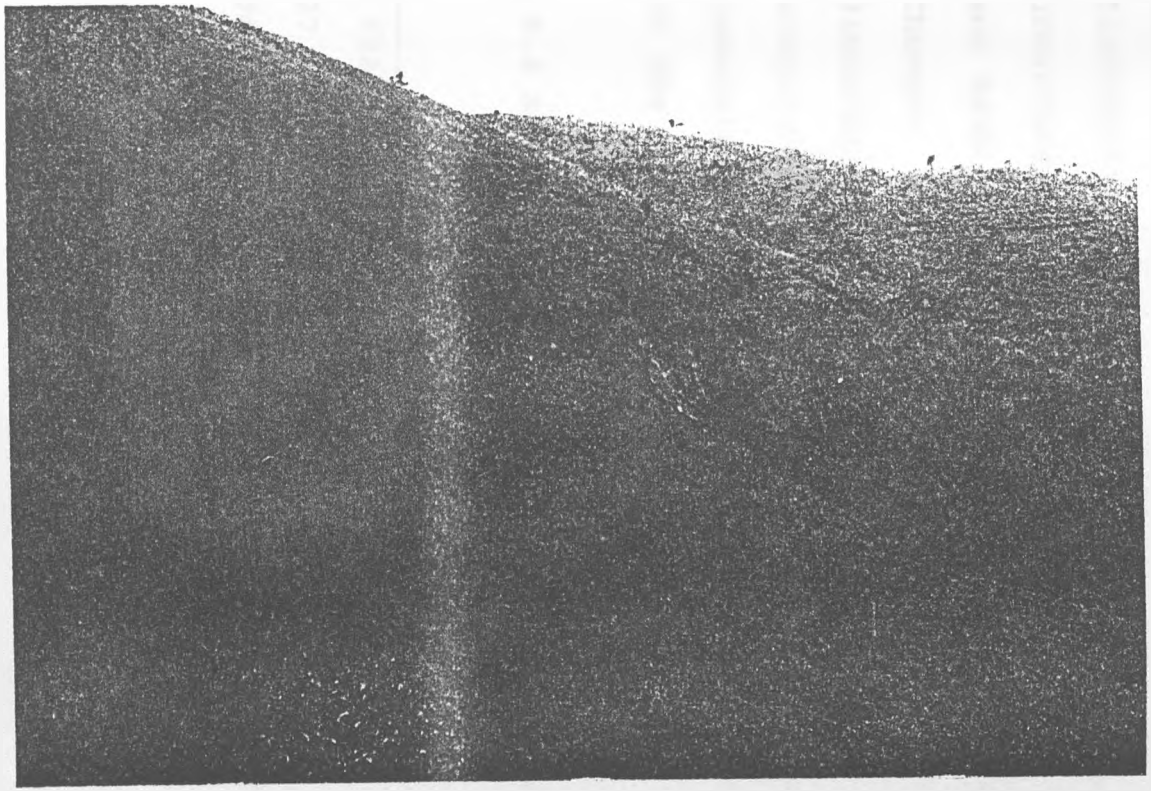


PLATE F GULLY EROSION DONGA BASIN

for these years.

The response of sediment yields to changes in precipitation, evaporation, land use, urbanisation, afforestation is assessed herein as part of an overall economic optimisation study involving the use of the entire river basin management system. This is similar to the assessment involving water yields and flow distribution presented in chapter six. The changes in urbanisation and afforestation involves changing the IMPV and EPXM parameters in the watershed model respectively. Changes in the land use are effected by changing the disturbance parameters DIST in the erosion model. The evaporation and precipitation changes used indicate possible influence due to water inundations and the impact of the future global climate.

TABLE 9.3 SEDIMENT YIELDS SIMULATED FOR THE UPPER DONGA BASIN BY YEAR (A COMPARISON)

WATER YEAR	SIMULATED YIELD	PEAKS	COMMENT
1976-77	2200.0 tons/sq Km	High	Mean Year
1977-78	5.3 ,,	V.Low	Severe Drought Yr
1981-82	447.0 ,,	Low	Drought Year

In this chapter, the objective is and has been to determine the behaviour of sediment processes in a changing river basin. Tables 9.4a-9.4d illustrates the effect of process and dynamic changes on sediment yields. These yields are expected to silt up proposed reservoirs

1977 ANNUAL SUMMARY TABLE,SEGMENT NO 1

	OCT	NOV	DEC	JAN	FEB	MARCH	APRIL	MAY	JUNE	JULY	AUGUSTSEPT	
MONTHLY ESTIMATED EROSION - TONNES	.799E+02	0.27E-03	0.00E+00	0.00E+00	0.00E+00	0.11E-04	0.63E-04	0.16E-03	0.20E-02	0.16E+00	0.35E-02	0.22E+04
MONTHLY PRECIPITATION MM	79[illegible]	13.0	0.0	0.0	0.0	12.4	65.8	177.7	280.0	198.4	211.6	390.6
SURFACE RUNOFF PRECIPITATION RATIO	[illegible]	0.0003	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0008	0.0001	0.0410
% AREA OF VEGETATION	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
CANOPY DENSITY OF VEGETATION	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
VEGETATION STAND METRES	5.0400	5.0400	5.0400	5.0400	5.0400	5.0400	5.0400	5.0400	5.0400	5.0400	5.0400	5.0400
VEGETATION STALK DENSITY	0.30E+03	0.30E+03	0.30E+03	0.30E+03	0.30E+03	0.30E+03	0.30E+03	0.30E+03	0.30E+03	0.30E+03	0.30E+03	0.30E+03
DISTURBANCE FACTOR	0.27E-03	0.26E-03	0.27E-03	0.27E-03	0.24E-03	0.27E-03	0.26E-03	0.27E-03	0.26E-03	0.27E-03	0.27E-03	0.26E-03

ANNUAL EROSION RATE= 0.2226E+04TONNES/80.KM/YEAR

PARTICLE SIZE DISTRIBUTION OF ERODED SOIL	0.0793	0.1394	0.1702	0.1020	0.2490	0.1080	0.0845	0.0224	0.0452
PARTICLE SIZE DISTRIBUTION OF SOIL TOP LAYER	0.2207	0.2519	0.2541	0.2544	0.2574	0.2889	0.2962	0.3118	1.0000

1977 ANNUAL SUMMARY TABLE, SEGMENT NO 2

	OCT	NOV	DEC	JAN	FEB	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPT
MONTHLY SIMULATED EROSION, TONNES	0.18E+01	0.19E-03	0.00E+00	0.00E+00	0.00E+00	0.32E-04	0.50E-03	0.78E-03	0.25E-01	0.15E+02	0.11E-02	0.17E+04
MONTHLY PRECIPITATION MM	395.3	13.0	0.0	0.0	0.0	9.9	85.8	174.5	282.3	198.4	211.4	330.7
SURFACE RUNOFF/ PRECIPITATION RATIO	0.0126	0.0012	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0018	0.0444	0.0001	0.1490
% AREA OF VEGETATION	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
CANOPY DENSITY F VEGETATION	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
VEGETATION STAND METRES	5.5200	5.5200	5.5200	5.5200	5.5200	5.5200	5.5200	5.5200	5.5200	5.5200	5.5200	5.5200
VEGETATION STALK DENSITY	0.16E+03	0.16E+03	0.16E+03	0.16E+03	0.16E+03	0.16E+03	0.16E+03	0.16E+03	0.16E+03	0.16E+03	0.16E+03	0.16E+03
DISTURBANCE FACTOR	0.27E-03	0.26E-03	0.27E-03	0.27E-03	0.24E-03	0.27E-03	0.26E-03	0.27E-03	0.26E-03	0.27E-03	0.27E-03	0.26E-03

ANNUAL EROSION RATE= 0.1575E+04TONNES/80.KM/YEAR

PARTICLE SIZE DISTRIBUTION OF ERODED SOIL	0.0067	0.0268	0.1051	0.6213	0.1886	0.0165	0.0279	0.0042	0.0030
PARTICLE SIZE DISTRIBUTION OF SOIL TOP LAYER	0.2633	0.4792	0.4794	0.4805	0.5151	0.6417	0.7014	0.7447	1.0000

1977 MONTHLY SEDIMENT YIELD

	OCT	NOV	DEC	JAN	FEB	MARCH	APRIL	MAY	JUNE	JULY	AUGHST	SEPT
MONTHLY SIMULATED SEDIMENT YIELD, TONNES	0.77E+02	0.27E-03	0.00E+00	0.00E+00	0.00E+00	0.12E-04	0.84E-04	0.19E-03	0.31E-02	0.87E+00	0.34E-02	0.22E+04

ANNUAL SIMULATED SEDIMENT YIELD = 0.2E+04 TONNES

TAB 9.2 EROSION MODEL CALIB' RESULTS

ABCNG

in the test Donga Basin while their consequent economic impact are presented in chapters 10 and 11.

9.4 Testing Landuse Changes in the Short Term

(1) IMPV

The responses of the Donga Basin sediment yields to short term changes in IMPV, the impervious and urbanisation parameter were investigated by changing its value from 0.01 (original) to 0.10 and 0.00. Tables 9.4 show the results obtained by year. The results indicate that sediment yields are moderately sensitive to small IMPV changes in the basin. This is the case because the numerous new hydrograph peaks generated by the substantial increase in the parameter are 'low to moderate' in magnitudes. The magnitudes of the existing peaks were moderately increased by an appreciable extent by IMPV changes, and the number of small peaks increased with the rise in IMPV values. It is the presence of very large peak runoffs and to a less extent high rainfall that causes large sediment erosion to occur. This is due to the fact that the two critical factors in soil erosion are the instantaneous runoff volumes and the rainsplash impact (refer chapters 2 and 5). Tables 8.7 and figures 8.7 (a) of chapter 8 clearly illustrates the peaks generated by increases in IMPV. The slight decrease in IMPV lowers hydrograph peaks and volume only slightly. The practical significance of this is that small and short term increases in urbanisation and impervious areas (roads) are unlikely to have much impact on the catchment sediment yields.

TABLE 9.4 (a) Total Simulated Annual Sediment Yield by Landuse Parameter in Tonnes/sq km

Parameter	INITIAL	EPXM		IMPV		DIST	
Values		.8/1	7.5/9	.00	.10	.000	.001
Year							
1976-77	2200	181	11890	2200	2200	2229	2273
1977-78	5.3	5.3	5.3	5.3	5.3	5.3	5.3
1981-82	447	362.2	459.2	447	447	447	447

TABLE 9.4 (b) Percentage Increase of Total Simulated Annual Sediment Yield by Landuse Parameter

Parameter	INITIAL	EPXM		IMPV		DIST	
Value		.8/1	7.5/9	.00	.10	.000	.001
Year							
1976-77	-	-1200	450	0	0	0	0
1977-78	-	0	0	0	0	0	0
1981-82	-	-22	+25	0	0	0	0

Initial Values EPXM=2.5/3 ; IMPV=0.01 ; DIST=0.0001

ORIGINAL NO CHANGE IMPV = 0.01, EPXM=3 DIST = 0.000	IMPV (x 5 = 0.05)	IMPV = 0.00	EPXM x3-0	EPXM x 0.33	DIST CHANGED FROM 0.000 TO 0.1000
0.6759E+100.1311E+04	0.6759E+100.1311E+04	0.6759E+100.1311E+04	0.6498E+100.1311E+04	0.6945E+100.1311E+04	0.6[illegible]E+100.1311E+04
0.6069E+100.1310E+04	0.6069E+100.1310E+04	0.6069E+100.1310E+04	0.5807E+100.1310E+04	0.6255E+100.1310E+04	0.5807E+100.1310E+04
0.5379E+100.1309E+04	0.5379E+100.1309E+04	0.5379E+100.1309E+04	0.5118E+100.1309E+04	0.5565E+100.1309E+04	0.5118E+100.1309E+04
0.4691E+100.1308E+04	0.4691E+100.1308E+04	0.4691E+100.1308E+04	0.4429E+100.1308E+04	0.4877E+100.1308E+04	0.4429E+100.1308E+04
0.4035E+100.1307E+04	0.4035E+100.1307E+04	0.4035E+100.1307E+04	0.3784E+100.1307E+04	0.4221E+100.1307E+04	0.3784E+100.1307E+04
0.3425E+100.1306E+04	0.3425E+100.1306E+04	0.3425E+100.1306E+04	0.3195E+100.1306E+04	0.3609E+100.1306E+04	0.3195E+100.1306E+04
0.2874E+100.1305E+04	0.2874E+100.1305E+04	0.2874E+100.1305E+04	0.2685E+100.1305E+04	0.3046E+100.1305E+04	0.2685E+100.1305E+04
0.2380E+100.1304E+04	0.2380E+100.1304E+04	0.2380E+100.1304E+04	0.2224E+100.1304E+04	0.2534E+100.1304E+04	0.2224E+100.1304E+04
0.1946E+100.1303E+04	0.1946E+100.1303E+04	0.1946E+100.1303E+04	0.1810E+100.1303E+04	0.2068E+100.1303E+04	0.1810E+100.1303E+04
0.1556E+100.1302E+04	0.1556E+100.1302E+04	0.1556E+100.1302E+04	0.1434E+100.1302E+04	0.1645E+100.1302E+04	0.1434E+100.1302E+04
0.1197E+100.1301E+04	0.1197E+100.1301E+04	0.1197E+100.1301E+04	0.1074E+100.1301E+04	0.1276E+100.1301E+04	0.1074E+100.1301E+04
0.8739E+090.1300E+04	0.8739E+090.1300E+04	0.8739E+090.1300E+04	0.7608E+090.1300E+04	0.9505E+090.1300E+04	0.7608E+090.1300E+04
0.6068E+090.1299E+04	0.6068E+090.1299E+04	0.6068E+090.1299E+04	0.5191E+090.1299E+04	0.6671E+090.1299E+04	0.5191E+090.1299E+04
0.3997E+090.1298E+04	0.3997E+090.1298E+04	0.3997E+090.1298E+04	0.3309E+090.1298E+04	0.4469E+090.1298E+04	0.3309E+090.1298E+04
0.2458E+090.1297E+04	0.2458E+090.1297E+04	0.2458E+090.1297E+04	0.1925E+090.1297E+04	0.2784E+090.1297E+04	0.1925E+090.1297E+04
0.1309E+090.1296E+04	0.1309E+090.1296E+04	0.1309E+090.1296E+04	0.9666E+080.1296E+04	0.1529E+090.1296E+04	0.9666E+080.1296E+04
0.4599E+080.1295E+04	0.4599E+080.1295E+04	0.4599E+080.1295E+04	0.2667E+080.1295E+04	0.5605E+080.1295E+04	0.2667E+080.1295E+04
0.1056E+070.1294E+04	0.1056E+070.1294E+04	0.1056E+070.1294E+04	0.1056E+070.1294E+04	0.6580E+070.1294E+04	0.1056E+070.1294E+04
0.0000E+000.1293E+04	0.0000E+000.1293E+04	0.0000E+000.1293E+04	0.0000E+000.1293E+04	0.0000E+000.1293E+04	0.0000E+000.1293E+04
0.0000E+000.1292E+04	0.0000E+000.1292E+04	0.0000E+000.1292E+04	0.0000E+000.1292E+04	0.0000E+000.1292E+04	0.0000E+000.1292E+04
0	0	0	0	0	0

TAB 9.4c TYPICAL
GEMBU RESERVOIR CAPACITY ELEVTION CURVE RELATIONS AS AFFECTED BY LAND USE CHANGES

(2) EPXM

An increase in the vegetation parameter, EPXM from 2.5-3 to 7.5-9 , has a slight effect on peaks (figure 8.7 (b) and tables 8.7) and an equally small impact on the sediment yields as seen from the simulated results in table 9.4. The effect is greatest in the wettest year (1976-77) and minimal in the drought year (1977-78). Despite that increases in EPXM slightly reduces runoff volumes, the respective slight and moderate increases in the magnitude of runoff peaks for 1981-82 and 1976-77 respectively has a similar effect on their annual sediment yields. The values for 1981-82 increases from 447 to 459.2t/sq km. The 1976-77 value rises from 2,200t/sq km to 11,890t/sq km.

Decreases in EPXM from 2.5-3 to 0.8-9 has an opposite reducing effect as seen from table 9.4. Here, as seen from figures 8.7, though runoff volumes are increased slightly, peaks on the other hand are reduced and this has a reducing effect on sediment yields. The effect of a decrease is far less profound than the case of a rise.

9.5 Testing the impact of Impoundment Induced Climate Change

The analysis of precipitation increases from this effect given in chapter 8 (tables 8.8 and figures 8.8), indicated that both runoff volumes and peaks will be significantly increased. This has led to the multiplying impact on sediment yields. A 5% increase in precipitation increases

ORIGINAL NO CHANGE	RAIN +10%	RAIN -10%	EVAP +10%	EVAP -10%
.6759E+100.1311E+04	0.6498E+100.1311E+04	0.6972E+100.1311E+04	0.6964E+100.1311E+04	0.6498E+100.1311E+04
.6069E+100.1310E+04	0.5807E+100.1310E+04	0.6289E+100.1310E+04	0.6278E+100.1310E+04	0.5807E+100.1310E+04
.5379E+100.1309E+04	0.5118E+100.1309E+04	0.5604E+100.1309E+04	0.5592E+100.1309E+04	0.5118E+100.1309E+04
.4691E+100.1308E+04	0.4429E+100.1308E+04	0.4919E+100.1308E+04	0.4907E+100.1308E+04	0.4429E+100.1308E+04
.4025E+100.1307E+04	0.3784E+100.1307E+04	0.4269E+100.1307E+04	0.4255E+100.1307E+04	0.3784E+100.1307E+04
.3425E+100.1306E+04	0.3195E+100.1306E+04	0.3562E+100.1306E+04	0.3646E+100.1306E+04	0.3195E+100.1306E+04
.2874E+100.1305E+04	0.2685E+100.1305E+04	0.3104E+100.1305E+04	0.3086E+100.1305E+04	0.2685E+100.1305E+04
.2380E+100.1304E+04	0.2224E+100.1304E+04	0.2590E+100.1304E+04	0.2573E+100.1304E+04	0.2224E+100.1304E+04
.1946E+100.1303E+04	0.1810E+100.1303E+04	0.2124E+100.1303E+04	0.2108E+100.1303E+04	0.1810E+100.1303E+04
.1556E+100.1302E+04	0.1434E+100.1302E+04	0.1703E+100.1302E+04	0.1696E+100.1302E+04	0.1434E+100.1302E+04
.1197E+100.1301E+04	0.1074E+100.1301E+04	0.1330E+100.1301E+04	0.1314E+100.1301E+04	0.1074E+100.1301E+04
.8739E+090.1300E+04	0.7608E+090.1300E+04	0.1001E+100.1300E+04	0.9862E+090.1300E+04	0.7608E+090.1300E+04
.6068E+090.1299E+04	0.5191E+090.1299E+04	0.7201E+090.1299E+04	0.7044E+090.1299E+04	0.5191E+090.1299E+04
.3997E+090.1298E+04	0.3309E+090.1298E+04	0.4844E+090.1298E+04	0.4720E+090.1298E+04	0.3309E+090.1298E+04
.2458E+090.1297E+04	0.1925E+090.1297E+04	0.2969E+090.1297E+04	0.2892E+090.1297E+04	0.1925E+090.1297E+04
.1309E+090.1296E+04	0.9666E+080.1296E+04	0.1551E+090.1296E+04	0.1544E+090.1296E+04	0.9666E+080.1296E+04
.4599E+080.1295E+04	0.2667E+080.1295E+04	0.5905E+080.1295E+04	0.5876E+080.1295E+04	0.2667E+080.1295E+04
.1056E+070.1294E+04	0.1056E+070.1294E+04	0.7331E+070.1294E+04	0.7110E+070.1294E+04	0.1056E+070.1294E+04
.0000E+000.1293E+04	0.0000E+000.1293E+04	0.0000E+000.1293E+04	0.0000E+000.1293E+04	0.0000E+000.1293E+04
.0000E+000.1292E+04	0.0000E+000.1292E+04	0.0000E+000.1292E+04	0.0000E+000.1292E+04	0.0000E+000.1292E+04
0	0	0	0	0

TAB 9.5c

GEMBU RES CAPACTY ELEVATION RELATIONSHIPS AS AFFECTED BY CLIMATE CHANGES

TYPICAL

sediment yields for the 1976-77 run by 500%; for 1981-82 by 100% and 400% for 1977-78 (table 9.5).

From table 9.5, it is evident that a further 5% increase in precipitation doubles the sediment yields obtained from the 5% increase case. The low values obtained from the simulation of the 1977-78 water year is not unconnected with the low daily precipitation values of that year which has led to the absence of erosion causing high runoff peaks. The sediment yields obtained from 1981-82 water year simulation is intermediate between those of the other two years.

The practical implication of this is that climate change from impoundments in the Donga Basin will have a significant and far reaching impact on erosion and sediment yields of the catchment, leading to high increases. The severity of the economic effect of this cannot be overemphasized and is assessed in chapter 10.

9.6 Testing the Impact of Global Climatic Change

The scenarios tested here are the same as performed for water balance in chapter 8. Table 9.6 indicates that no matter the scenario of global warming, sediment yields in the Donga Basin will be substantially increased. There will be a 7 to 22 fold increase in erosion and sediment yields. This will result due to the huge increases in annual and monthly runoff volumes and runoff peaks as indicated in chapter 8 in tables 8.9 and figures 8.9. .

The Donga Basin must be prepared for these huge increses

TABLE 9.5 (a) Total Annual Simulated Sediment Yields by Various Scenarios of Impoundment-Induced Climate Change. Values are in Tonnes/sq km

Water Year	1976-77	1977-78	1981-82
Precipitation+			
+0%	2,200	5.27	447
+5%	12,150	25.14	835.1
+10%	26,850	82.12	1,595.0
+11.6%	27,280	123.84	11,085.6

TABLE 9.5 (b) Number of Folds of Increase in Annual Simulated Sediment Yield by Various Scenarios of Impoundment Induced Climate Change.

Water Year	1976-77	1977-78	1981-82
Precipitation+			
+0%	0.0	0.0	0.0
+5%	4.5	3.5	2.0
+10%	11.0	15.2	2.7
+11.6%	**12.4**	23.5	24.8

in sediment loss as a result of the future climatic change. The design of water schemes must accomodate these huge sediment and water yields and floods if they must perform efficiently.

9.7 Conclusions and Final Remarks

The results above indicate that only significant and possibly extreme changes in landuse parameters in the short term, will have any appreciable effect on catchment sediment yields. The changes in EPXM and increases in IMPV tested herein led to some moderate changes in sediment yields. In real life these changes tested are not likely to occur in the short term as they were a bit extreme. It can be concluded therefore that the small short term changes in these parameters are not likely to have any devasting effect on the catchment erosion and sediment yields.

The results and analysis indicate that the significant increases in runoff volumes and peaks obtained from simulations of reservoir and the greenhouse effect induced climate change will lead to enormous catchment erosion. The magnitude of the erosion indicates the certainty of a future economic disaster with regard to both agricultural output and water resources developments in the Donga Basin due to

(1) the short term impact of the construction of the numerous reservoirs proposed for the catchment

TABLE 9.6 (a) Total Simulated Annual Sediment Yields from Various Scenarios of Global Climate Change. Long term conditions (50 yrs) Values are in tonnes/sq km.

EVAP +	0%	2.5%	5.0%	7.5%	10.0%	12.5%
Precip +						
0.0%	2,200					
14.0%		32,480	29,580	25,880	21,270	16,150
16.5%		37,610	35,340	32,510	28,860	24,250
20.0%		47,870	45,600	43,210	40,440	37,230
24.0%		59,030	56,390	53,700	51,010	48,030

TABLE 9.6 (b) Number of Folds of Increase of Total Annual Sediment Yields for various Scenarios of Global Climate Change.

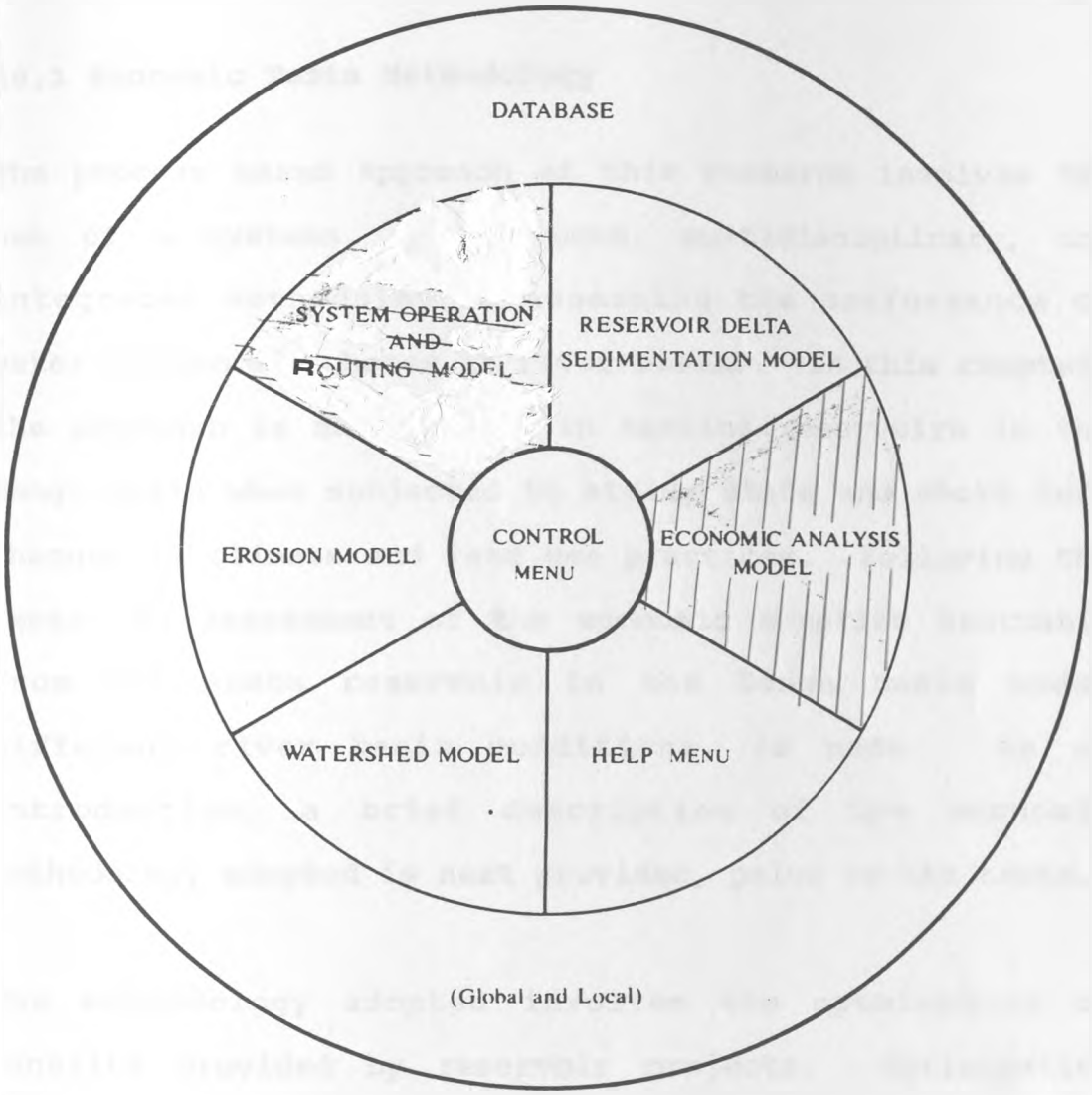
EVAP +	0%	2.5%	5.0%	7.5%	10.0%	12.5%
Precip +						
0.0%	0%					
14.0%		14.8	13.5	11.8	9.7	7.3
16.5%		17.1	16.1	14.8	13.1	11.0
20.0%		21.8	20.7	19.6	18.4	16.9
24.0%		26.8	25.6	24.4	23.2	21.2

(2) the long term effect of the greenhouse effect coupled with the impoundment induced climate change.

The disaster will particularly affect the storage schemes planned. The economic implications of the disaster from case (1) and case (2) with respect to the reservoirs planned in the basin are respectively assessed in chapters 10 and 11 by the use of the economic simulation model.

CHAPTER 10: GEMBU RESERVOIR OPTIMISATION STUDY STEADY STATE AND SHORT TERM SCENARIO ANALYSIS AND DISCUSSION OF RESULTS

In completing the Donga River Basin Management which include the presentation of water and sediment yield responses in chapter 8 and 9 respectively, the results of a similar simulation study involving the use of economics is presented herein for the proposed Gembu reservoir project for steady state and short term changes. The main tool of the management system active herein is the economic simulation model developed during the course of this research and described in chapter 6 (figure 10.0). The routing and system operation model is also active herein to a lesser extent since it provides the reservoir releases (figure 10.0). Section 10.1 briefly describes and discusses the economic tests methodology adopted herein. In section 10.2, various scenarios involving the multipurpose allocation of water to users is tested and their economics are assessed. In section 10.3, the economic sensitivity of various scenarios of dead storage allocation are presented and discussed. The effect of short term landuse changes by year is assessed and presented in section 10.4. The influence of reservoir construction on local climate change is tested by year. The results are presented and discussed in section 10.5. A general discussion of the results inclusive of implications are provided in section 10.6. Section 10.7 summarises the research findings and concludes the



MODELS ACTIVE IN STEADY & SHORT TERM SCENARIOS

Fig.10 RIVER BASIN MANAGEMENT SYSTEM

This system is available for use on both micro and mainframe(VAX) computers.

chapter. A computer output of the results of a typical simulation run is given in Table 10.0.

10.1 Economic Tests Methodology

The process based approach of this research involves the use of a systems, multipurpose, multidisciplinary, and integrated methodoloy is assessing the performance of water resources schemes in river basins. In this chapter, the approach is mainly used in testing reservoirs in the Donga Basin when subjected to steady state and short term changes in climate and land use practices. Following the tests, an assessment of the economic benefits accruable from the Gembu reservoir in the Donga Basin under different river basin conditions, is made. As an introduction, a brief description of the economic methodology adopted is next provided, prior to the tests.

The methodology adopted involves the optmisation of benefits provided by reservoir projects. Optimisation involves the attainment of economic efficiency by the use of such criteria as the maximisation of economic benefits and the minisation of costs using standard economic methodology which are incorporated into the economic simulation package discussed in chapter 6. These techniques include the present worth, annual worth, cost-benefit, and the rate of return techniques. These are briefly described in chapter four. As a point of caution, many social benefits and costs are difficult to estimate,

	HYDROPOWER	EVAP LOSS	SPILLAGES
DISCHARGE CMS	0.2424E+02	0.7903E+00	0.0000E+00
MEAN POWER MW	0.1700E+03	0.6160E+01	0.0000E+00
MEAN ENERGY KWH	0.1490E+10	0.5396E+02	0.0000E+00

CAPITA 0.2273E+05 HECT 0.7540E+02

FIRST COSTS	0.2550E+09	0.2550E+08
ALTNT COSTS	0.4292E+09	0.4292E+08

NONPOWR GR.TOTAL	POWER	IRRIGATION	W.SUPPLY	OTHERS	TOT.NPW	TOTAL
PRESENT VALUES-COSTS	0.3061E+09	0.6710E+05	0.8370E+06	0.6253E+08	0.6343E+08	0.4420E+09
PRESENT VALUES-BENFTS	0.8380E+09	0.2121E+05	0.1918E+07	0.7306E+07	0.9245E+07	0.8472E+09
PRESENT WORTHS VALUES	0.5319E+09	-.4589E+05	0.1081E+07	-.5522E+08	-.5419E+08	0.4053E+09
BENEFIT COST RATIO	0.2738E+01	0.3161E+00	0.2291E+01	0.1169E+00	0.1458E+00	0.1917E+01
ANNUAL COSTS VALUES	0.2473E+08	0.5422E+04	0.6763E+05	0.5052E+07	0.5125E+07	0.3571E+08
ANNUAL BENEFIT VALUES	0.6771E+08	0.1714E+04	0.1549E+06	0.5903E+06	0.7470E+06	0.6846E+08
ANNUAL WORTH VALUES	0.4298E+08	-.4466E+04	0.8731E+05	-.4462E+07	-.4378E+07	0.3275E+08
ANNUAL BEN.COST RATIO	0.2738E+01	0.3161E+00	0.2291E+01	0.1169E+00	0.1458E+00	0.2432E+01
UNDISCOUNTED COSTS	0.8030E+09	0.1417E+06	0.1815E+08	0.2223E+09	0.1485E+10	0.2288E+10
UNDISCOUNTED BENFTS	0.1730E+11	0.4629E+06	0.4186E+08	0.4061E+09	0.4484E+09	0.1731E+11
BEN.CST RATIO-UNDISCOUNTED	0.2154E+02	0.8748E+01	0.2307E+01	0.1827E+01	0.3020E+00	0.7565E+01

TABLE 10.0 TYPICAL ECON&OPERATION OUTPUT

and such have not been included in this analysis. For example, the cultural inconvenience of displacing populations from reservoir sites and the benefits of employment in a project location (or due to a particular project) are difficult to assess. The actual relocation and resettlement costs, however, inclusive of the costs of land can easily be estimated and are included.

The reservoir management problem considered in this work involves an investigation of the impact of sedimentation and water inflows emanating from river basins. In using Management System in tackling the problem, the main focus herein will be on the areas that influence reservoir economic performance. These are listed below.

(a) The economic effect of possible long and short term climate shifts due to the direct environmental impact of the imposition of a lake or a combination of lakes within a catchment or its vicinity, on reservoir economic efficiency.

(b) The effect of the position of the minimum operating levels

(c) The effect of short term landuse changes

(d) The effect of water allocation policy to various uses.

(e) The long term economic effects caused by global climate changes such as the greenhouse, ozone shield, and acid rain effects, among others, is investigated and presented in Chapter 11 while the first four effect (a) - (d) are treated in this chapter.

The results are presented, analysed and discussed in section 10.2 to 10.5.

10.2 Testing Water Allocation to Various Users for both Highhead and Lowhead Cases.

The simulations here involve subjecting the high head (960m) Gembu reservoir project to five cases of water allocation, namely,

(a) 99% to power generation and the rest to non-power uses;

(b) 75% to power generation and the rest to other uses;

(c) 50% to power and the rest to non-power uses;

(d) 25% to power generation and the balance to other uses

(e) 99% to non-power uses with only 1% being allocated to power.

Table 10.1 summarises the economic returns from these runs. In carrying out the simulation, the following points should be noted.

(a) The reservoir rule curve utilised relates water withdrawals directly with the elevation capacity. In addition, no special part of the reservoir capacity is allocated to flood control or any other use. In short, the reservoir is allowed to build up and spill freely during operation. Conservation of water is the stated policy during use.

(b) The simulations covers the 50 year capitalisation

period of the project using the 1976-77 water year flows as the mean for the period. This year is chosen because it has mean annual flows (57 cumecs) that are closest to the long term mean (57.8 cumecs) (see chapter 11, fig 11.4).

Table 10.1 (a) and (b) summarises the simulation results while the details are listed in computer prints provided in the appendix 10. The summarised results of tables 10.1 (a) are (b) are plotted in figure 10.1 (a) - (c).

TABLE 10.1 (a) Highhead Case

% Allocation		99/1	75/25	50/50	25/75	1/99
Priority		Power	Power	Multi	Nonpow	Nonpow
P.V.BNFTS	m$	468.300	382.400	292.800	203.300	117.300
ANN BNFTS	m$	37.840	30.890	23.660	16.420	9.478
B.C.R		1.666	1.301	0.954	0.635	0.352
N.P.W	M$	187.200	88.540	-14.240	-117.000	-215.70
N.A.W.	m$	15.130	7.154	-1.150	-9.455	-17.430

The values of the economic returns are the total for all purposes

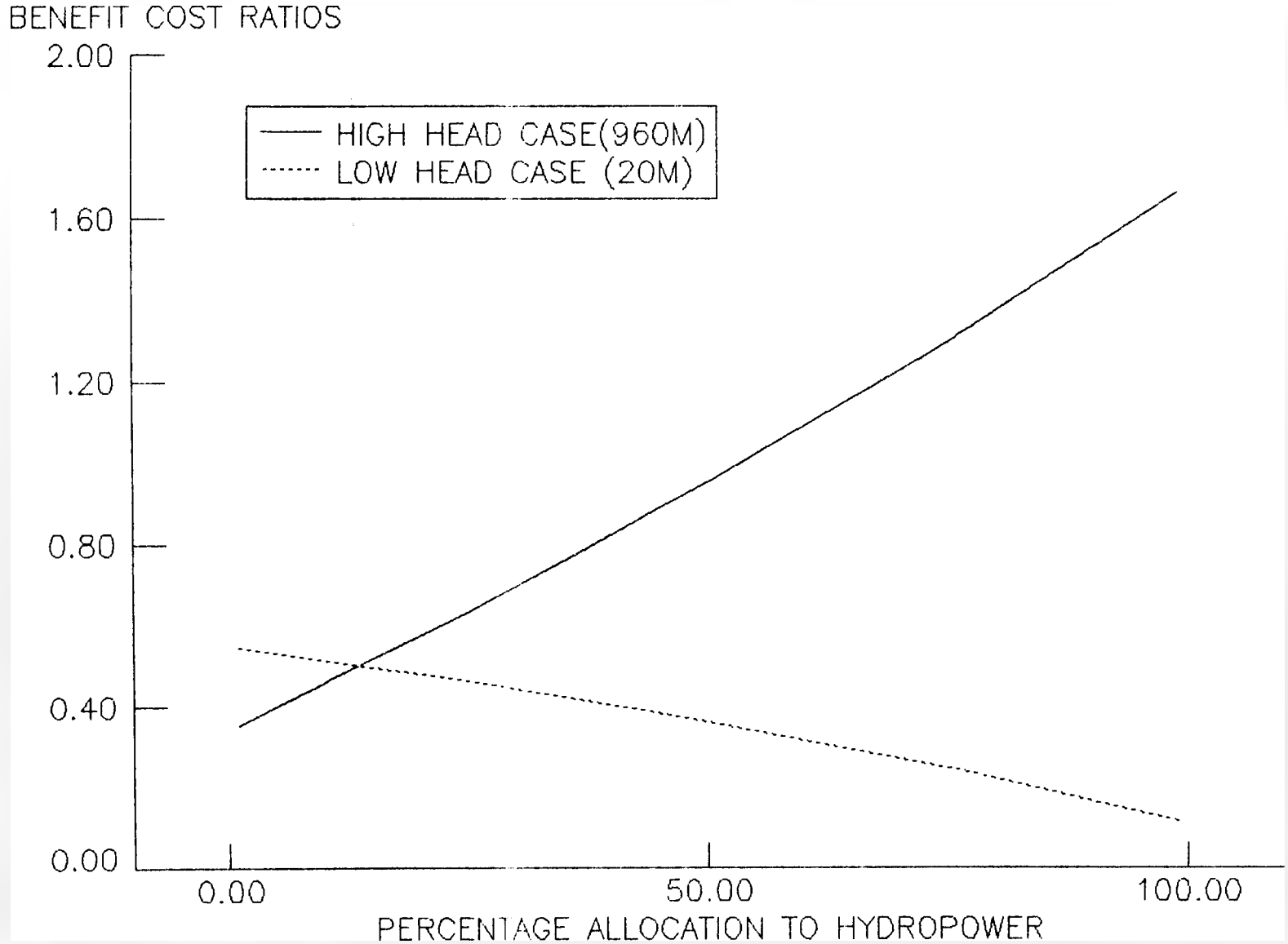
B.C.R. is the benefit cost ratio

N.P.W is the net present worth in millions of U.S dollars

N.A.W is the net annual work in millions of U.S dollars.

GEMBU RESERVOIR ECONOMIC ANALYSIS

EFFECT OF WATER ALLOCATION TO USERS



THE BALANCE IS ASSIGNED TO IRRIGATION AN D WATER SUPPLY

Fig. 10.1(a)

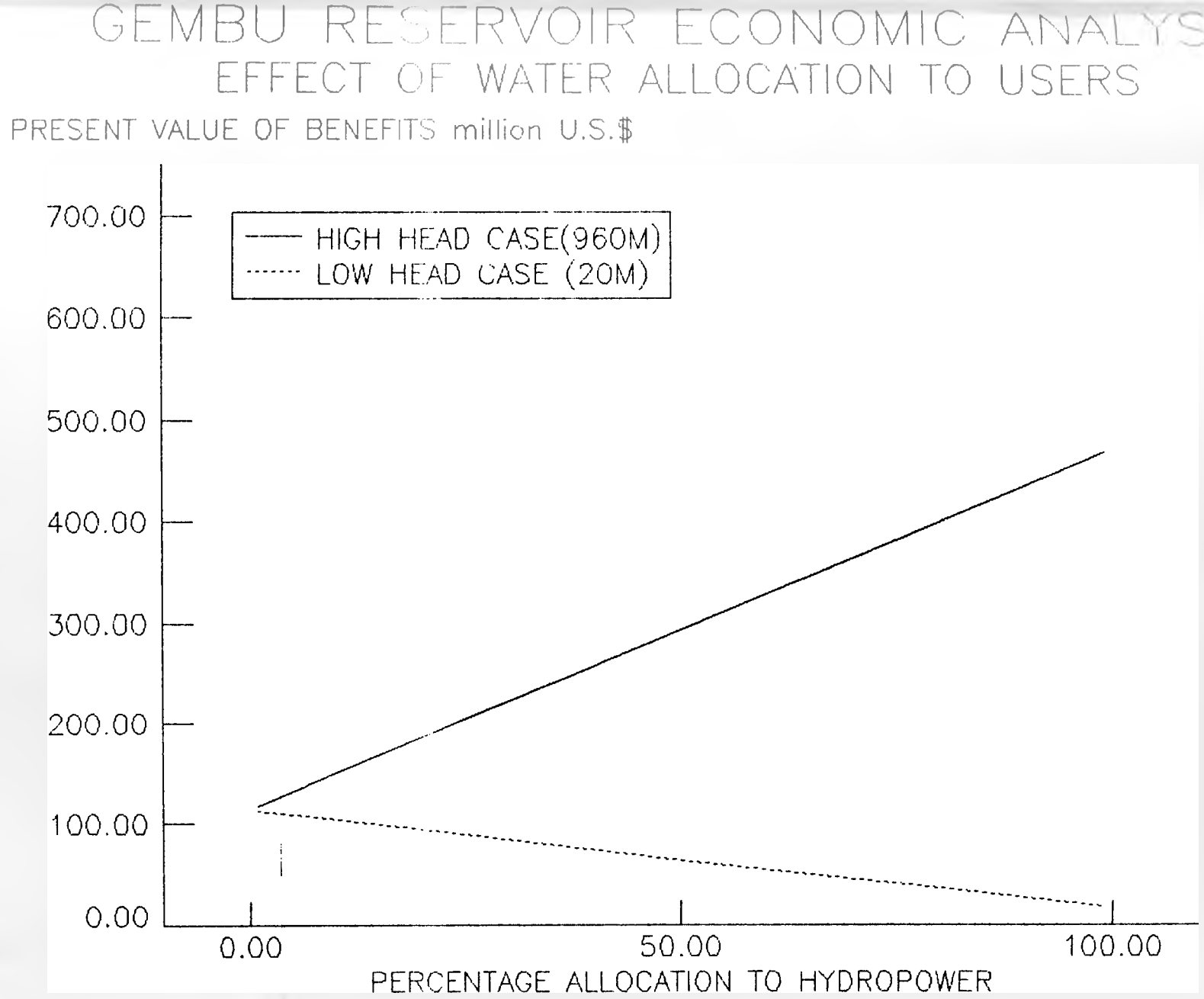

GEMBU RESERVOIR ECONOMIC ANALYSIS
EFFECT OF WATER ALLOCATION TO USERS
PRESENT VALUE OF BENEFITS million U.S.$
HIGH HEAD CASE(960M)
LOW HEAD CASE (20M)
700.00
600.00
500.00
400.00
300.00
200.00
100.00
0.00
0.00
50.00
100.00
PERCENTAGE ALLOCATION TO HYDROPOWER
THE BALANCE IS ASSIGNED TO IRRIGATION AN D WATER SUPPLY

Fig. 10.1 (b)

GEMBU RESERVOIR ECONOMIC ANALYSI

EFFECT OF WATER ALLOCATION TO USERS

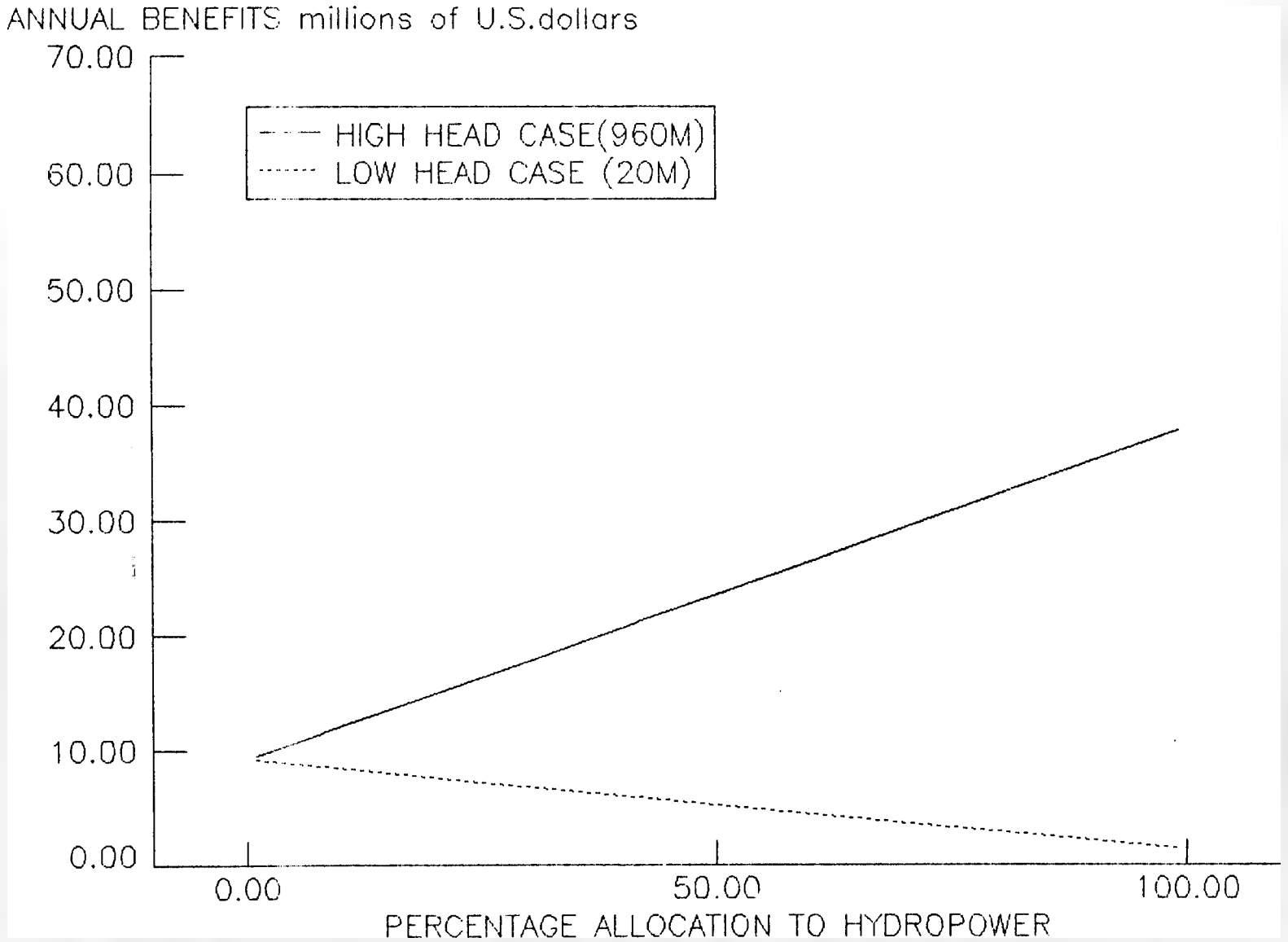


Fig. 10.1 (c)

THE BALANCE IS ASSIGNED TO IRRIGATION AN D WATER SUPPLY

TABLE 10.1 (b) Lowhead Case

%Allocation	99/1	75/25	50/50	25/75	1/99
Priority	Power	Power	Multi	Nonpow	Nonpow
P.V. BNFTS m$	17.900	41.170	63.60	89.540	112.800
ANN BNFTS m$	1.450	3.326	5.281	7.235	9.111
B.C.R.	0.117	0.247	0.363	0.464	0.548
N.P.W m$	-136.000	-125.500	-114.500	-105.500	-93.030
N.A.W m$	-10.990	-10.140	-9.252	-8.367	-7.517

From the simulation results and plots above, the project benefits are higher in case (a) when hydropower generation is assigned priority allocation because of the high head (960m) involved. The benefits reduce progressively until they become losses, that is when practically no water is allocated to power.

10.3 Testing the Sensitivity of the Dead Storage Capacities Gembu Reservoir Scheme

The tables 10.2 (a) below shows the economic returns obtained from trial simulation runs for the Gembu reservoir when the minimum operating levels are progressively lowered by a few metres at a time. The conditions of the simulation runs are listed below.

(a) The run covers the 50 year operation of the reservoir using the 1976-77 water year as the mean. As indicated earlier, this water year has a mean annual flow (57 cumecs) closest to the long term mean (57.8 cumec) (Diyam, 1985-see chapter 11 for graphical comparison).

(b) The rule curve utilised herein also relates water withdrawals directly with the capacity elevation curve of the reservoir without any restrictions on reservoir build up, i.e a conservation operating policy.

(c) Hydropower generation has been given priority in these runs because of the extremely high head available and used. Table 10.2 (a) provides the summary results for this highhead case.

TABLE 10.2 (a) Minimum level change and economic returns for highhead scheme.

Minlev (masl)	1300	1297	1294	1292
ENERGY (mKwh)	817.600	817.600	817.600	817.600
POWER (Mw)	93.330	93.330	93.330	93.330
P.V BNFTS m$	460.000	460.000	460.000	460.000
ANN BNFTS m$	37.160	37.160	37.160	37.160
B.C.R	1.503	1.503	1.503	1.503
N.P.W m$	154.000	154.000	154.000	154.000
N.A.W m$	12.430	12.430	12.430	12.430

B.C.R is the benefit cost ratio
N.P.W is the net present worth in millios of United State $
N.A.W is the net annual worth in United States dollars

The summary of the results of the simulation runs are shown in table 10.2 (a).

From the simulation results shown in the table 10.2 (a)

above, it is clear that the reservoir performance is not sensitive to changes in minimum operating levels, for very high head storage projects. This may be explained by the reasons given below.

(1) The generation head does not change due to changes in operation levels because of the high head involved. The rise in the water depth caused by a drop in minimum operating levels is neutralised by a reduction in head between the intake at the dam body and the turbines 900m below (for this Gembu case).

(2) The slight increase in capacity due to dead storage lowering is not reflected in the economic returns. This may be due to the nature of the operation rule curve. This is the case because as the reservoir surface reaches lower elevations, water withdrawals are curtailed accordingly. The loss in benefits from this curtailment plays a dominant role when compared to the gains expected from a larger live storage capacity.

10.3.2 Testing the Sensitivity of the Dead Storage Capacity by lower Minimum Operating Levels Gembu Reservoir - Low Head Case.

The test carried out in 10.3.1 is repeated with a low head version of the Gembu Reservoir which involves the initial use of a head of 12m and this is increased gradually by lowering the minimum operating levels. In this simulation, 50% of the water is allocated to power generation and the

other 50% is allocated to non power uses. The table 10.2 (b) summarises the economic returns from computer simulation runs involving the lowering of the minimum dead storage levels.

TABLE 10.2 (b)

MinLev (masl)	1300	1297	1294	1292
ENERGY (mKwh)	5.162	6.452	7.742	8.603
POWER (Kw)	590.000	736.500	883.800	982.000
P.V. BNFTS m$	63.420	64.150	64.870	65.360
ANN BNFTS m$	5.124	5.183	5.241	5.281
B.C.R	0.345	0.353	0.360	0.364
N.P.W m$	-120.000	-117.700	-115.700	-114.500
N.A.W m$	-9.700	-9.507	-9.347	-9.252

From the results in tables 10.2 and figures 10.2, it is evident that economic returns from the Gembu reservoir improves with a lowering of the dead storage levels. The trend is linear. The economic benefits are generally low in this case because the start head is low, only 12m but this can be increased substantially by the gradual lowering of the minimum operating levels until a sizeable fraction of the 91m height of the dam is harnessed. Benefits improve substantially with the minimum level lowering as observed from figure 10.2.

10.4 Testing the Effect of Changing Landuse Parameters

Table 10.3 provides a summary of the results obtained from

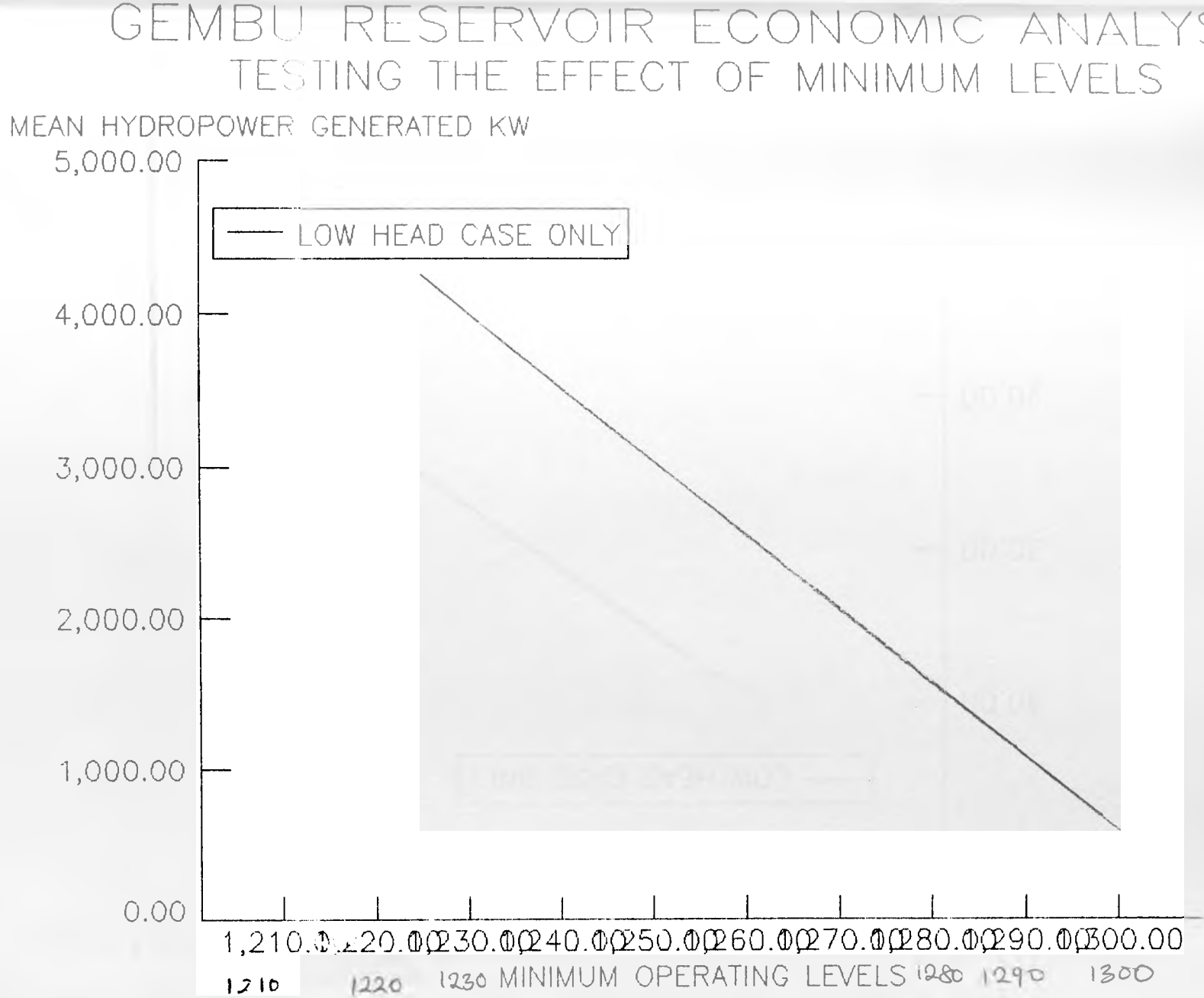

GEMBU RESERVOIR ECONOMIC ANALYSI
TESTING THE EFFECT OF MINIMUM LEVELS
MEAN HYDROPOWER GENERATED KW
5,000.00
4,000.00
3,000.00
2,000.00
1,000.00
0.00
LOW HEAD CASE ONLY
1,210.00
1,220.00
1,230.00
1,240.00
1,250.00
1,260.00
1,270.00
1,280.00
1,290.00
1,300.00
1210
1220
1230
MINIMUM OPERATING LEVELS
1280
1290
1300

Fig. 10.2 (1)

GEMBU RESERVOIR ECONOMIC ANALYSIS

TESTING THE EFFECT OF MINIMUM LEVELS

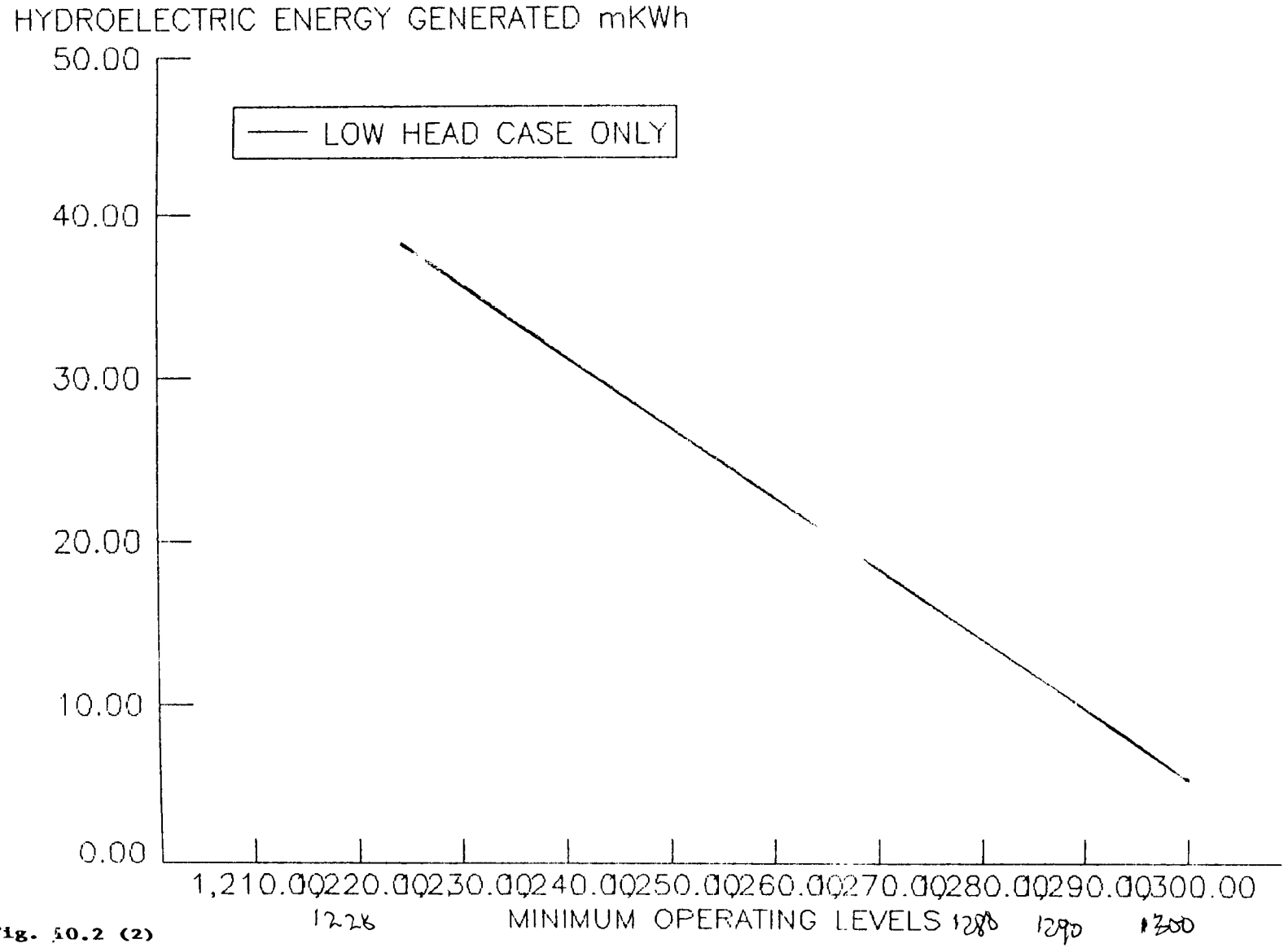


Fig. 10.2 (2)

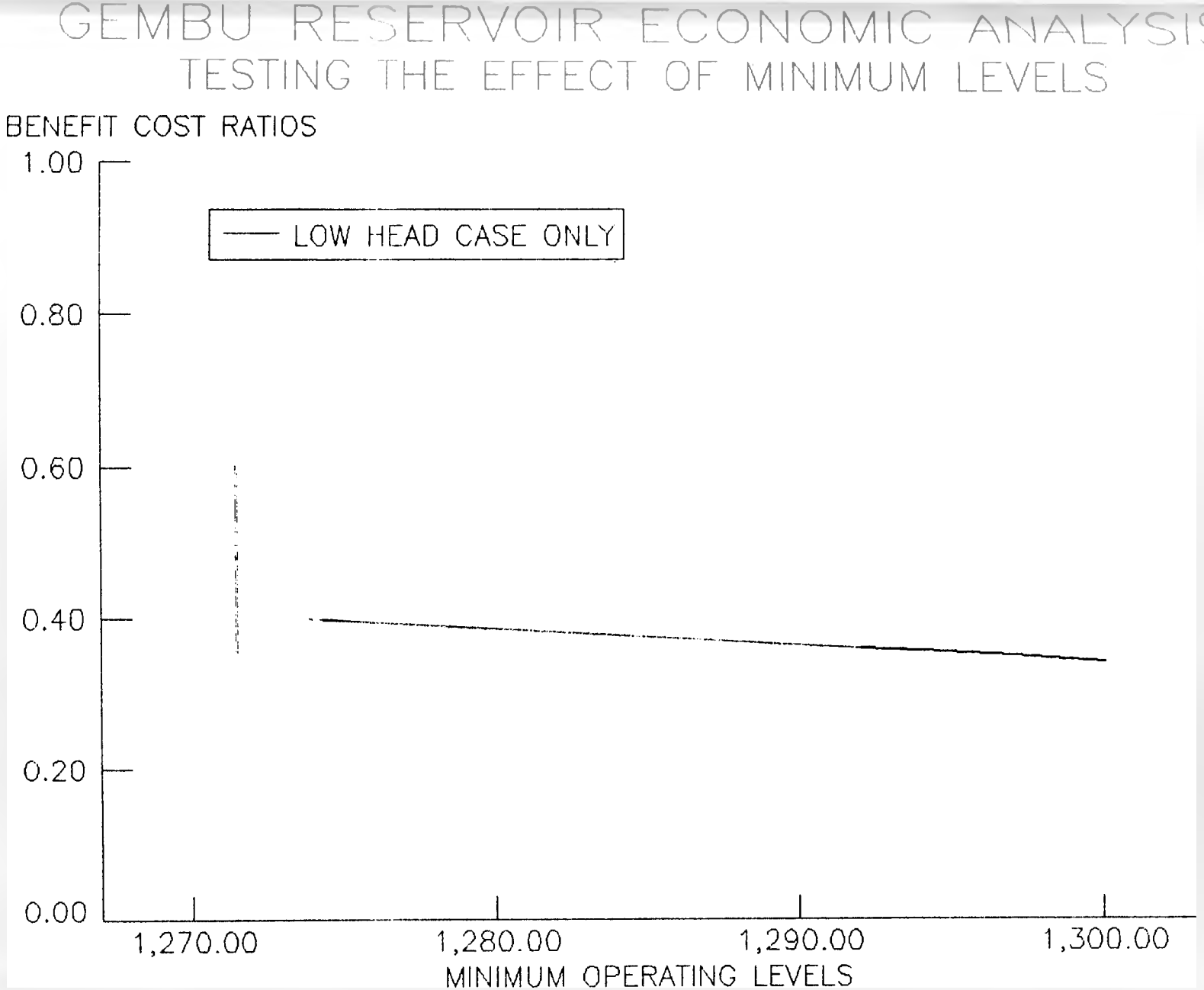

GEMBU RESERVOIR ECONOMIC ANALYSIS
TESTING THE EFFECT OF MINIMUM LEVELS
BENEFIT COST RATIOS
1.00
0.80
0.60
0.40
0.20
0.00
LOW HEAD CASE ONLY
1,270.00
1,280.00
1,290.00
1,300.00
MINIMUM OPERATING LEVELS

Fig. 10.2 (3)

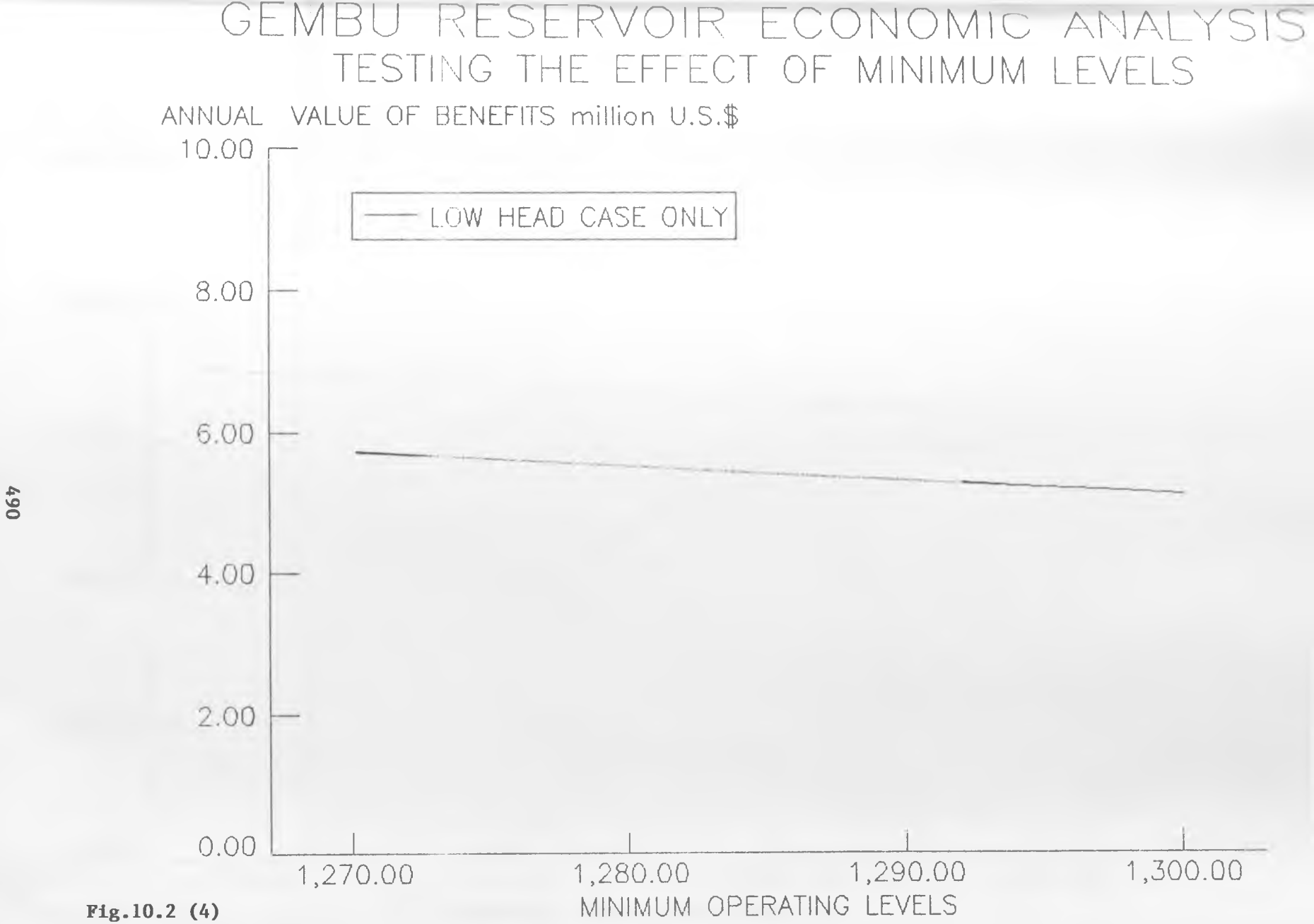

GEMBU RESERVOIR ECONOMIC ANALYSIS
TESTING THE EFFECT OF MINIMUM LEVELS
ANNUAL VALUE OF BENEFITS million U.S.$
10.00
8.00
6.00
4.00
2.00
0.00
LOW HEAD CASE ONLY
1,270.00
1,280.00
1,290.00
1,300.00
MINIMUM OPERATING LEVELS

Fig.10.2 (4)

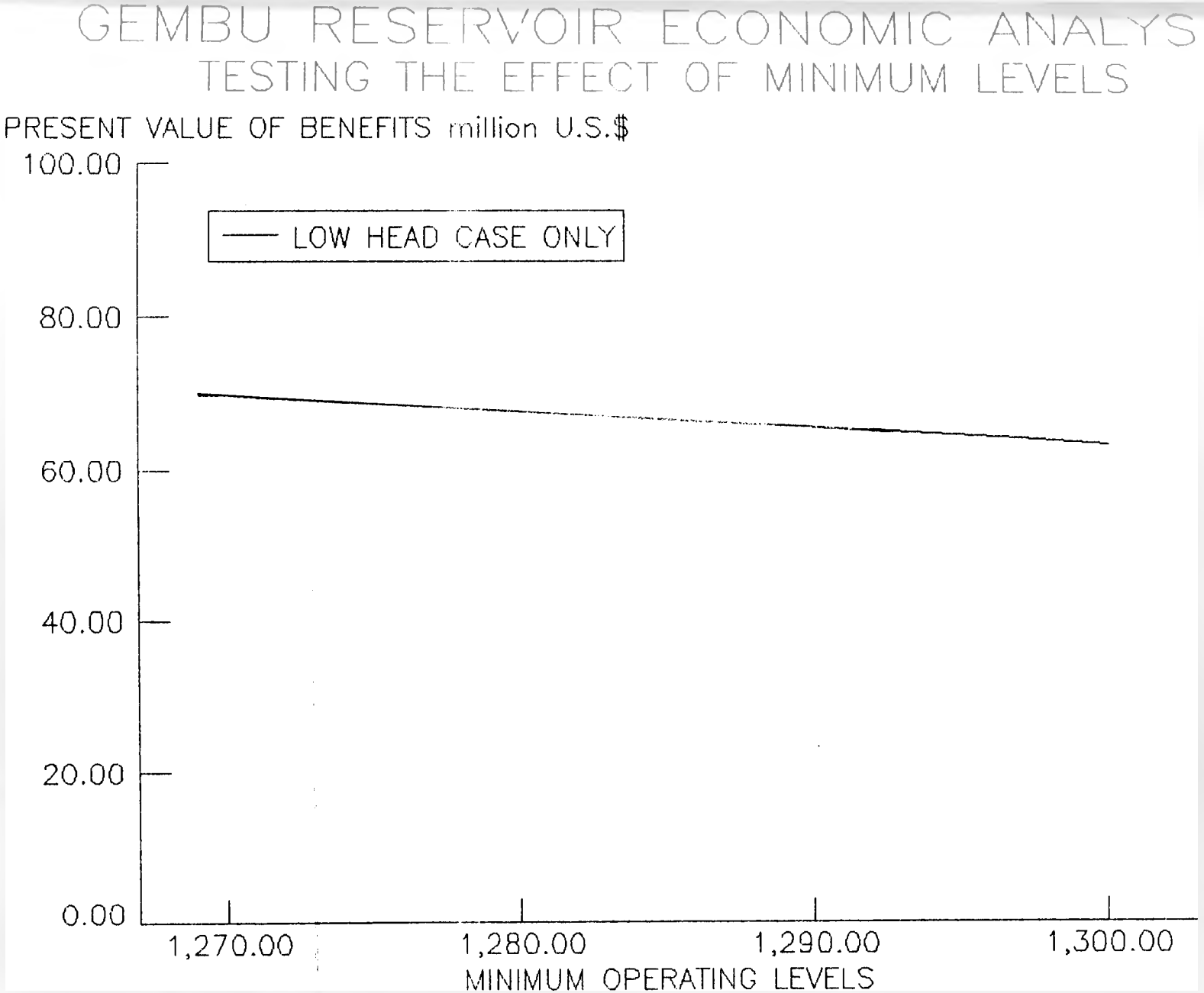

GEMBU RESERVOIR ECONOMIC ANALYSIS
TESTING THE EFFECT OF MINIMUM LEVELS
PRESENT VALUE OF BENEFITS million U.S.$
100.00
80.00
60.00
40.00
20.00
0.00
LOW HEAD CASE ONLY
1,270.00
1,280.00
1,290.00
1,300.00
MINIMUM OPERATING LEVELS

Fig. 10.2 (5)

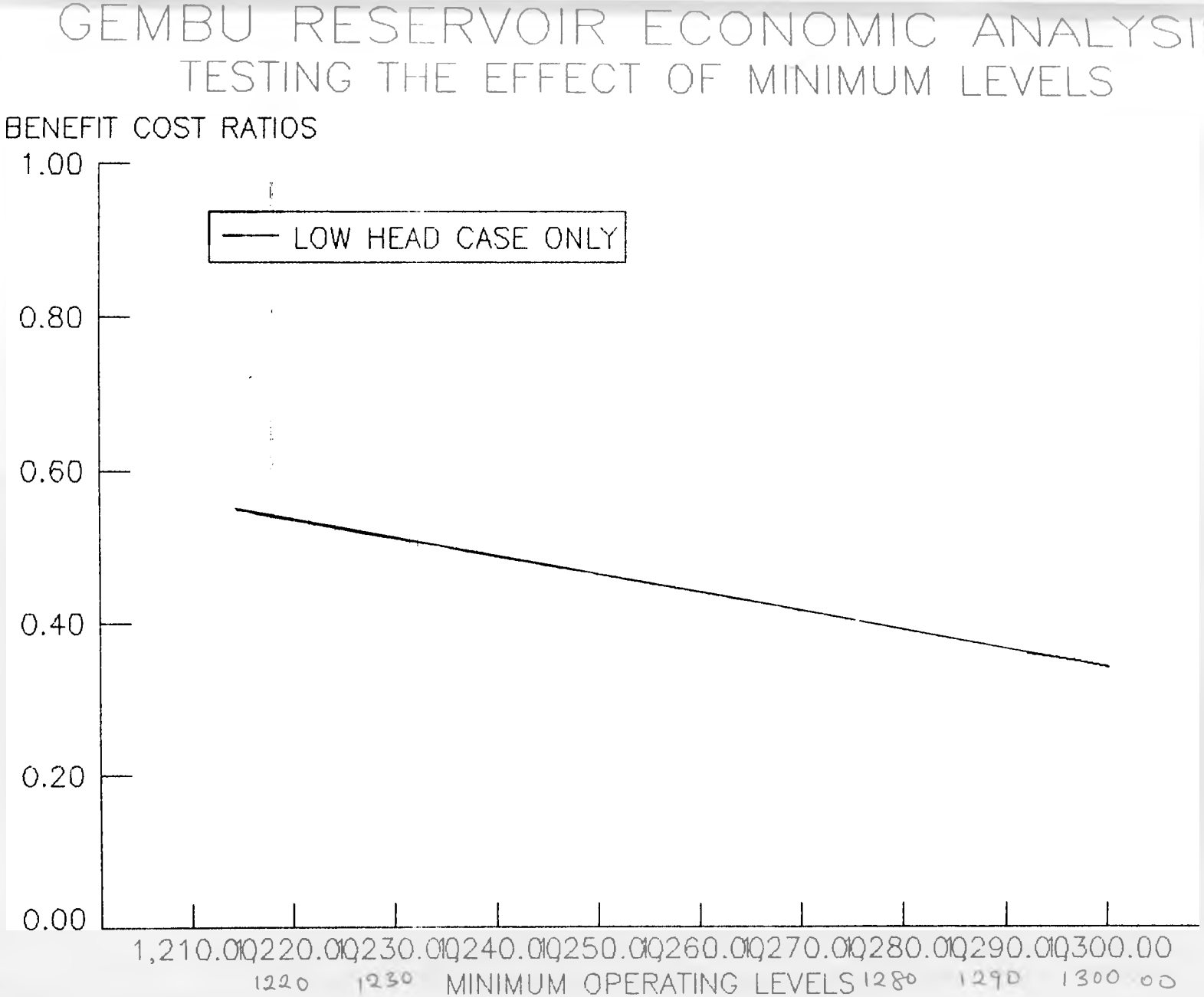

GEMBU RESERVOIR ECONOMIC ANALYSIS
TESTING THE EFFECT OF MINIMUM LEVELS
BENEFIT COST RATIOS
1.00
0.80
0.60
0.40
0.20
0.00
LOW HEAD CASE ONLY
1,210.00 220.00 230.00 240.00 250.00 260.00 270.00 280.00 290.00 300.00
1220 1230 MINIMUM OPERATING LEVELS 1280 1290 1300 00

Fig. 10.2 (6)

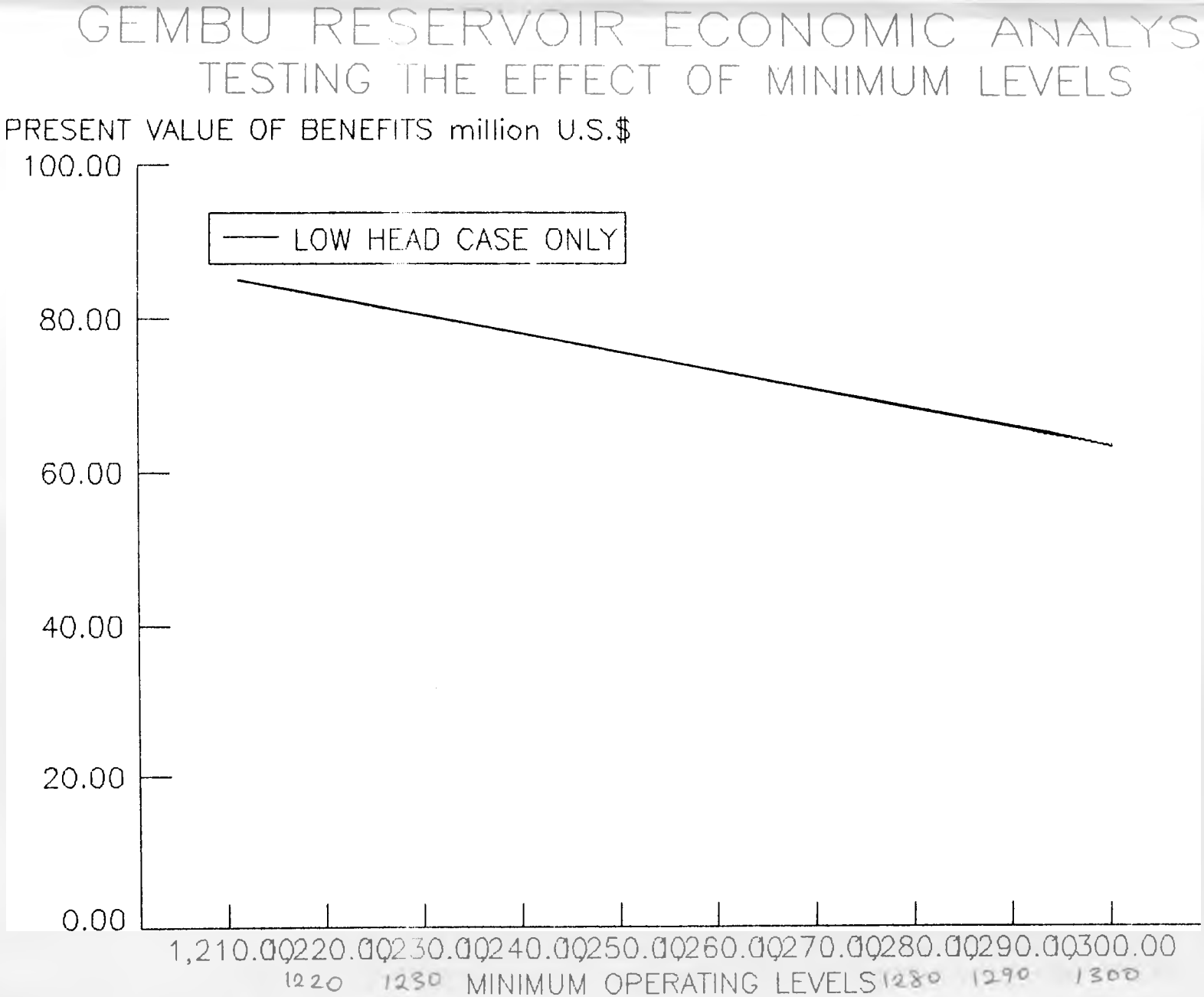

GEMBU RESERVOIR ECONOMIC ANALYSIS
TESTING THE EFFECT OF MINIMUM LEVELS
PRESENT VALUE OF BENEFITS million U.S.$
100.00
80.00
60.00
40.00
20.00
0.00
LOW HEAD CASE ONLY
1,210.00 220.00 230.00 240.00 250.00 260.00 270.00 280.00 290.00 300.00
1220 1230 MINIMUM OPERATING LEVELS 1280 1290 1300

Fig. 10.2 (7)

GEMBU RESERVOIR ECONOMIC ANALYS

TESTING THE EFFECT OF MINIMUM LEVELS

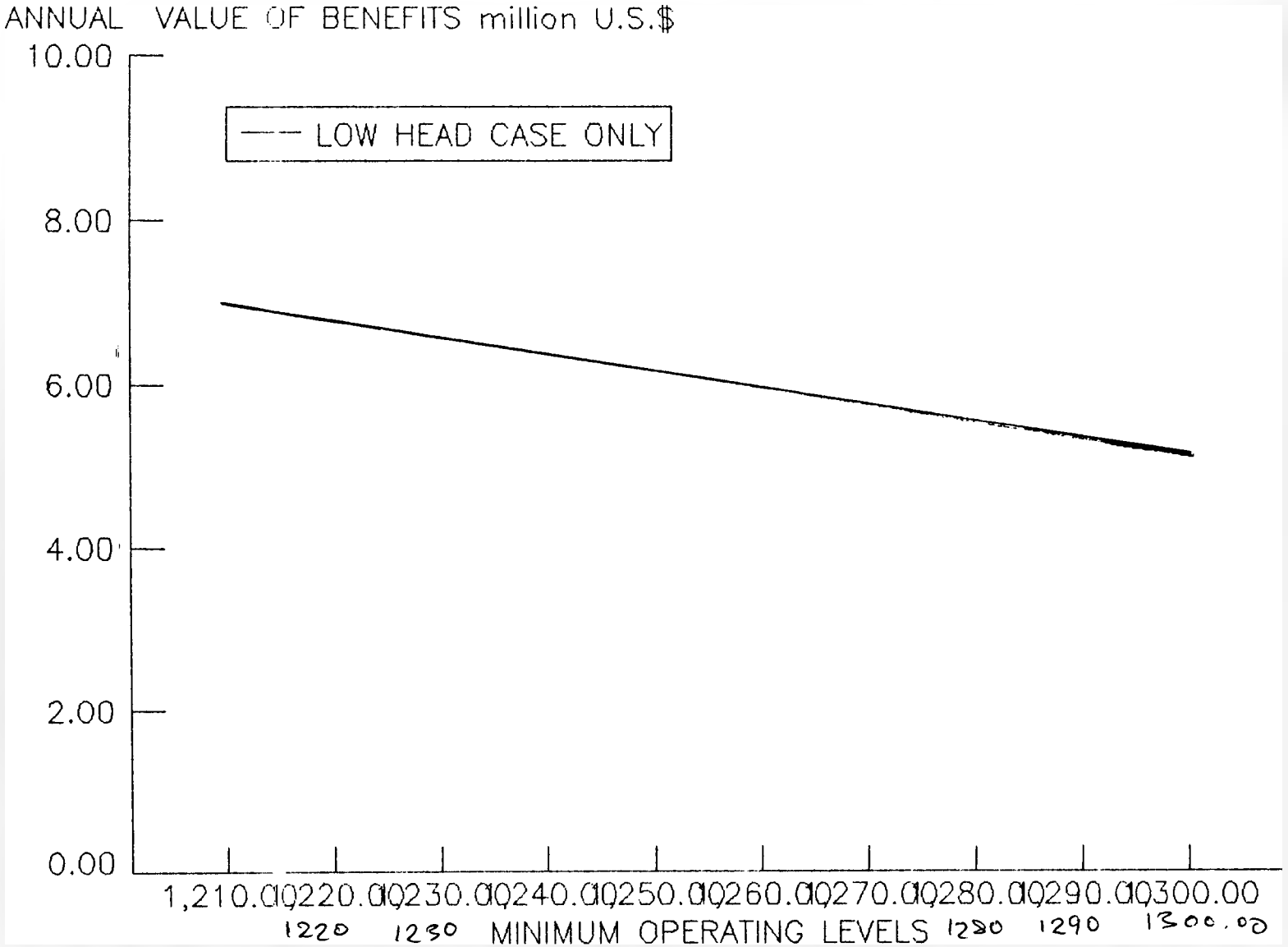


Fig. 10.2 (8)

simulation runs involving short term landuse changes using the 1976-77, 1977-78, and 1981-82 water years hydrological data for the Donga Basin. The landuse parameters tested are described below.

(a) EPXM, the vegetation parameter had an initial value of 2.5-3.0 (2.5 for catchment upper segment and 3.0 for the lower segment) for the first simulation run, representing current conditions. In practical terms, this parameter represents afforestation, deforestation, crop and cropping types, shifting cultivation and bush burning etc. This was increased initially by multiplying by 3 giving a value 7.5-9.0 for the second simulation run. The third simulation run utilised a value of 0.8-1.0. This was obtained by multiplying the initial values by 0.3. This EPXM parameter is used in the watershed model.

(b) Three simulation runs involving IMPV, the fraction of total catchment area which is considered impervious, were carried out. This parameter does represent urbanisation in a catchment to a certain extent in practical terms. The initial value of 0.01 is first used and thereafter values of 0.10 are used for the second and third trial simulation runs respectively. The IMPV parameter is used in the watershed model.

(c) Three simulation runs involving DIST, the land disturbance parameter were also carried out. Cultivation practices such as tilling, terracing, etc are represented by this parameter. The original calibration value of

0.00001 was first used. A value of 0.0000 and 0.0010 were then used for the second and third simulation runs respectively. These values are extremely low because the Donga Basin catchment being tested has very little physical soil disturbances at this point in time. It is quite virgin with very few people inhabiting the area. This parameter is utilised in the erosion model.

TABLE 10.3 SHORT TERM ECONOMIC SIMULATION RUNS RESULTS

PRESENT VALUE OF BENEFITS

PARAMETER	INITIAL	EPXM	EPXM	IMPV	IMPV	DIST	DIST
VALUE		7.5/9	0.8/1	.00	.10	.0000	.0010
YEAR							
76-77	47.55	53.18	46.27	47.54	45.55	47.55	47.55
77-78	46.15	46.38	46.15	46.15	46.15	46.15	46.15
81-82	47.31	49.05	47.10	47.31	47.31	47.31	47.31

All benefits are in millions of United States dollars.
The above results are plotted in figure 10.3.

From the table above, the economic benefits are sensitive to changes in EPXM, the vegetation and forest parameter, especially when annual runoffs are not too low as in 1976-77. IMPV and DIST, the urbanisation and the cultivation disturbance parameters will not influence reservoir performance to any appreciable extent in the short term in the Donga basin.

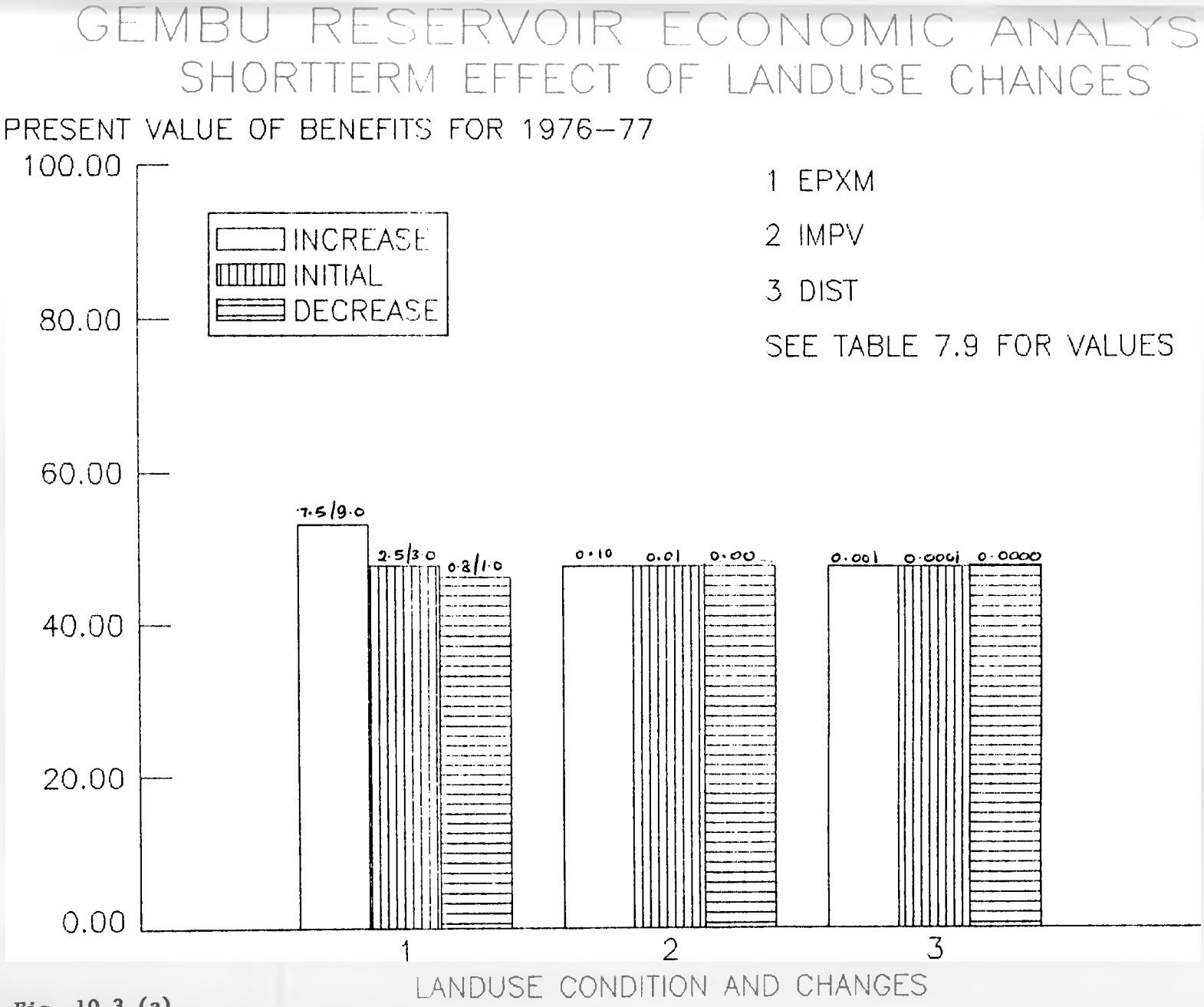

GEMBU RESERVOIR ECONOMIC ANALYSIS
SHORTTERM EFFECT OF LANDUSE CHANGES
PRESENT VALUE OF BENEFITS FOR 1976–77
100.00
80.00
60.00
40.00
20.00
0.00
INCREASE
INITIAL
DECREASE
1 EPXM
2 IMPV
3 DIST
SEE TABLE 7.9 FOR VALUES
7.5/9.0
2.5/3.0
0.2/1.0
0.10
0.01
0.00
0.001
0.0001
0.0000
1
2
3
LANDUSE CONDITION AND CHANGES

Fig. 10.3 (a)

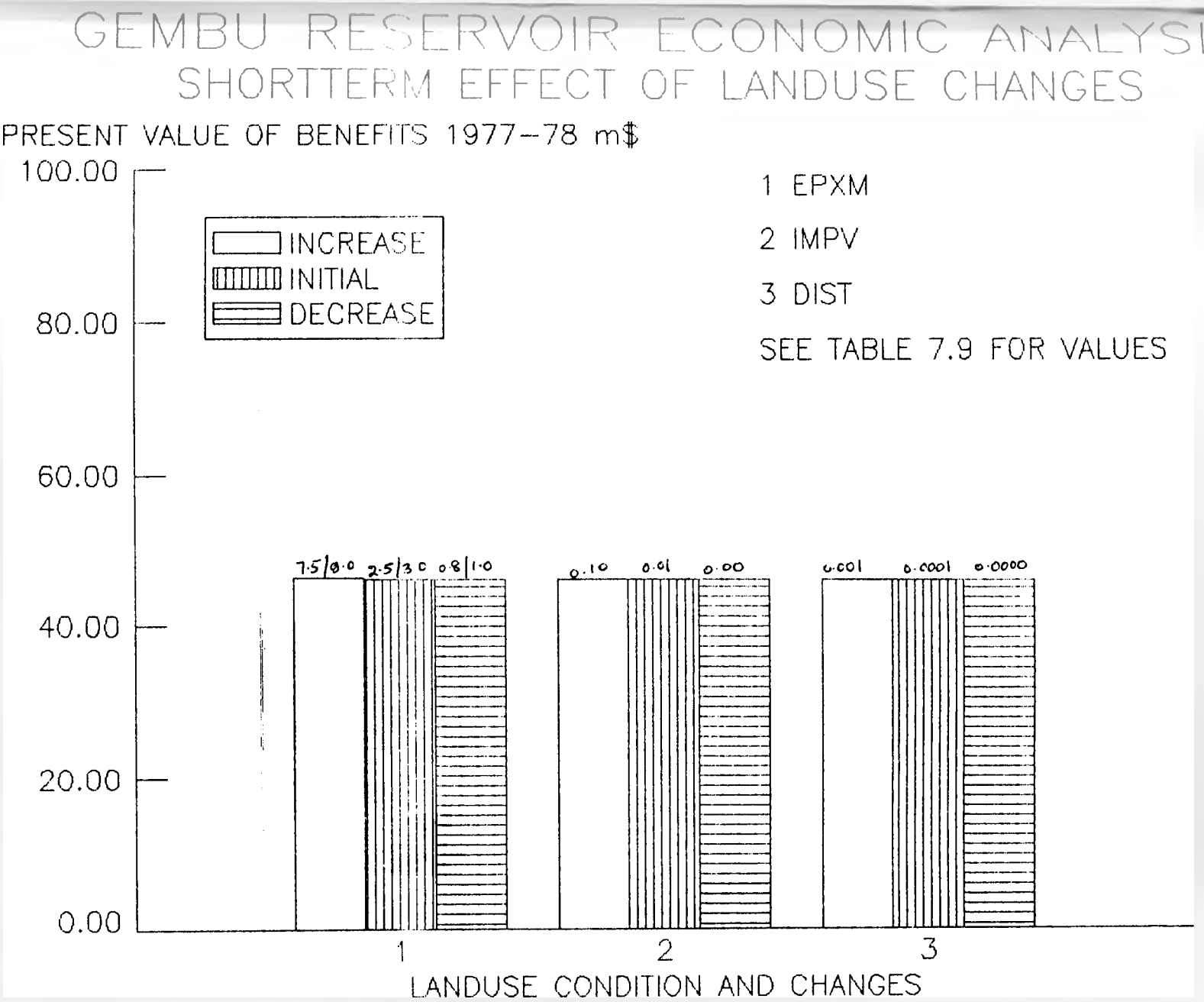

GEMBU RESERVOIR ECONOMIC ANALYSIS
SHORTTERM EFFECT OF LANDUSE CHANGES
PRESENT VALUE OF BENEFITS 1977–78 m$
100.00
80.00
60.00
40.00
20.00
0.00
INCREASE
INITIAL
DECREASE
1 EPXM
2 IMPV
3 DIST
SEE TABLE 7.9 FOR VALUES
7.5/0.0
2.5/3.0
0.8/1.0
0.10
0.01
0.00
0.001
0.0001
0.0000
1
2
3
LANDUSE CONDITION AND CHANGES

Fig. 10.3 (6)

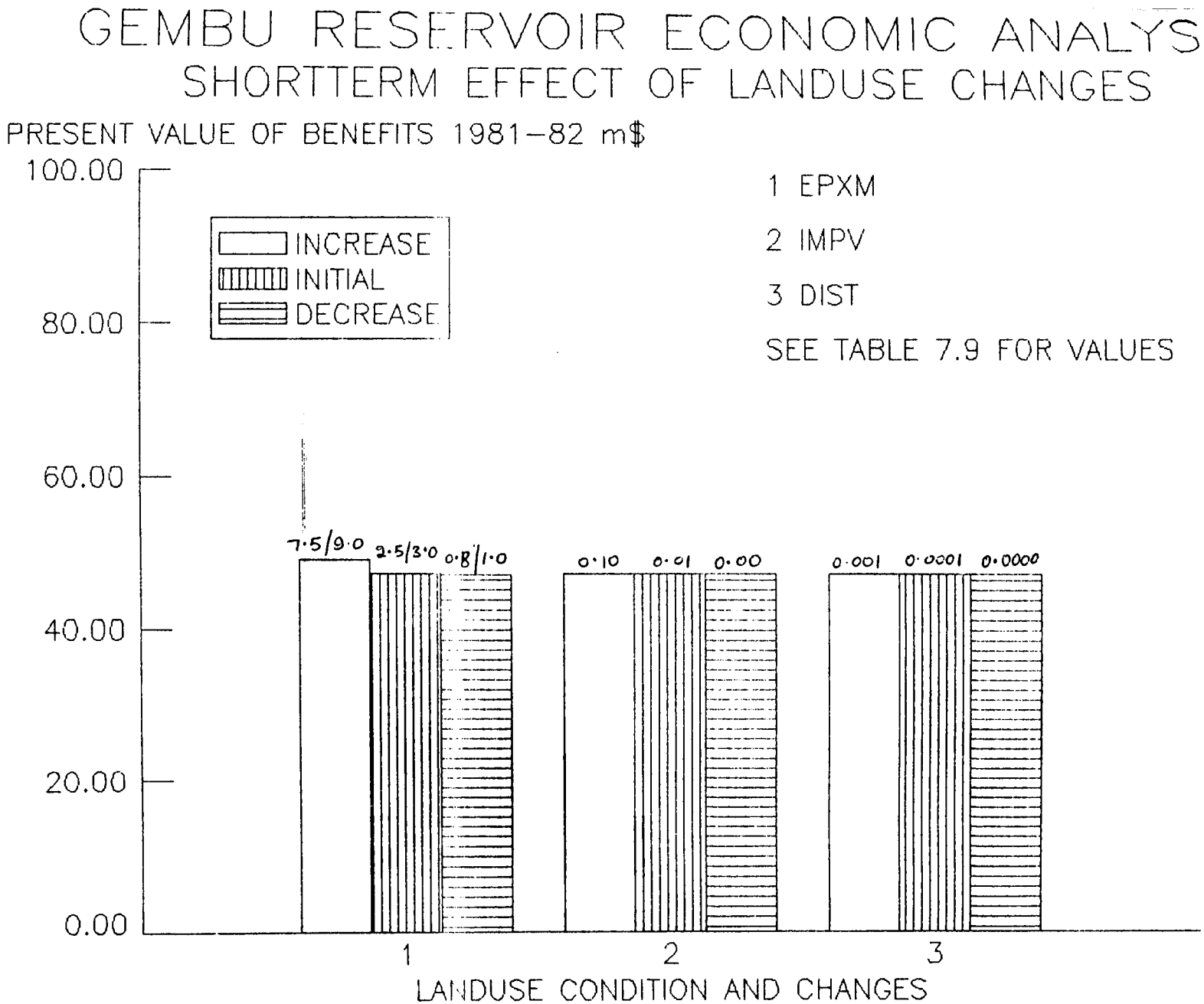

GEMBU RESERVOIR ECONOMIC ANALYSIS
SHORTTERM EFFECT OF LANDUSE CHANGES
PRESENT VALUE OF BENEFITS 1981–82 m$
100.00
80.00
60.00
40.00
20.00
0.00
INCREASE
INITIAL
DECREASE
1 EPXM
2 IMPV
3 DIST
SEE TABLE 7.9 FOR VALUES
7·5/9·0
2·5/3·0
0·8/1·0
0·10
0·01
0·00
0·001
0·0001
0·0000
1
2
3
LANDUSE CONDITION AND CHANGES

Fig. 10.3 (c)

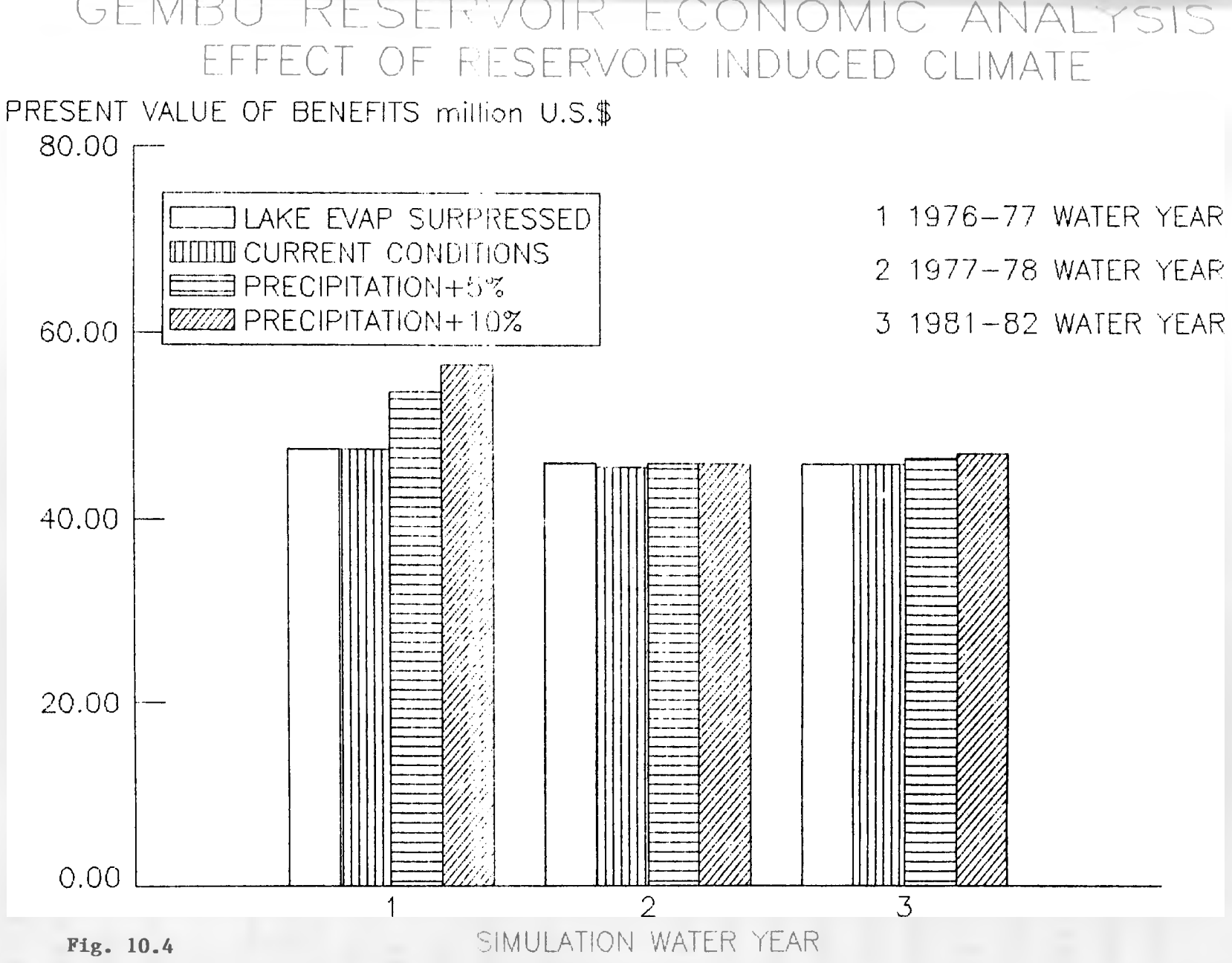

GEMBU RESERVOIR ECONOMIC ANALYSIS
EFFECT OF RESERVOIR INDUCED CLIMATE
PRESENT VALUE OF BENEFITS million U.S.$
80.00
60.00
40.00
20.00
0.00
LAKE EVAP SURPRESSED
CURRENT CONDITIONS
PRECIPITATION+5%
PRECIPITATION+10%
1 1976-77 WATER YEAR
2 1977-78 WATER YEAR
3 1981-82 WATER YEAR
1
2
3
SIMULATION WATER YEAR

Fig. 10.4

10.5 Testing Influence of Local Climate Change Resulting from Reservoir Construction

In assessing the economic response of the proposed Gembu Reservoir in the Donga Basin to local climate changes from the presence of large water or reservoir surfaces, the three percentage (increase) scenarios, namely, 0% 5% 10%, and 11.6% were tested. Reservoir evaporation is automatically accounted for by the model (see Computer print out in table 10.0). In the simulations, hydropower generation has been given priority over other uses because of the high head involved (960). The simulation results of economic benefits as obtained for the water years 1970-77, 1977-78, and 1981-82 are summarised in table 10.4, below:

Tab 10.4

WATER YEAR[a]	76-77		77-78		81-82	
PPT+	No Evap	+ Evap	No Evap	+ Evap	No Evap	+Evap
0%	47.56	47.48	46.09	45.74	46.13	46.11
5%		53.72		46.06		46.70
10%		56.63		46.12		47.29
(11.6)						

All benefits are in millions of United States dollars.

From the table 10.4, the annual economic benefits from Gembu reservoir simulations generally increase with higher values of catchment precipitation as assigned. In the 1977-78 water year, however, the increases in benefits are marginal. From the runoff hydrographs of that year given in chapter 8, it is

clear that the runoff is very small. This can be explained from the fact that simulated reservoir levels are lower than normal (in 1977-78) and thus the water abstractions by the chosen model rule curve are correspondingly lower and hence the resultant reductions in benefits.

In general,the increases in benefit with increasing precipitation is due to corresponding increases in water elevations and the volumes of water available in lieve storage. The result is that more water is available for allocation to power generation and other uses.

The general observation for the Gembu reservoir from simulation results is that annual benefits will improve substantially with the construction of the reservoirs, provided runoff for a specified year is not much below that of an average year, which is this case is the 1976-77 water year. Any year with a mean annual runoff below average are unlikely to have improved benefits from climate modification by impoundments.

From the table 10.4, it is clear that lake evaporation losses have some effect on economic returns from the Gembu reservoir. The table indicates that annual losses could be as much as 350,000 U.S. dollars.

10.6 General Discussion of Results

The factors influencing reservoir economic performance are many and complex. These are outlined below and

illustrated in figure 11.4.

(a) Sedimentation in reservoirs tends to reduce benefits due to reduced reservoir capacity. This reduction in benefits is conditional on the temporal and spatial location of the deposited sediments. Benefits tend to reduce when sediments are deposited in the live storage and especially so if the deposits do remain there for long durations. The deposited sediments tend to move into the dead storage region with time.

(b) Increasing inflows bring in higher sediment loads and this tends to reduce benefits.

(c) Sediment deposits in reservoirs do raise water elevations and heads for hydropower generation. Raised elevations tend to increase water abstraction in accordance with the specified rule curve. Consequently, economic benefits are improved.

(d) The increased inflows from higher runoffs due to precipitation increases raises the quantity of water in live storage and its elevation. The resultant effect is that more water available for allocation in accordance with the chosen rule curves. (This condition of increased reservoir water storage and elevations is only tenable in cases where the reservoirs are large enough to accommodate the increased inflows. In cases where the available reservoir capacity is small, the extra water inflows are spilled off. In this Gembu reservoir case study, the

reservoir is large enough to accommodate the inflows assigned, and no spillages are experienced (see table 10.0 for typical simulated spill output). Thus, water allocation from the reservoir and associated benefits rise when precipitation increases).

(e) Further, increased discharges from rise in precipitation are associated with higher water velocities. These direct the deposition of sediments far into the lake and also have the tendency to sweep deposited sediments from the live storage to the dead storage zone, thus improving benefits.

(f) The strength of each of these factors, and the final economic return for a particular reservoir, depends on the size, shape, surface area, and slope of that reservoir, and the physical, hydrological, and climatological characteristics of the catchment.

With the above discussion of the factors competing for prominence and pre-eminence in the reservoir regime, the results obtained from simulation tests for the Gembu reservoir can be explained thus;

(1) **Water Allocation**

The economic returns from the various water allocation scenarios does not depend on the above mentioned factors but on the head of the scheme (for power generation) and on the amount of water specifically allocated to each use, be it power, irrigation, water supply or others. In the

high head Gembu scheme, a large allocation of water to power improves total project economic benefits. This conclusion is reached from the comparisons of the results in table 10.1 (a). These results are expected as high heads improve power benefits and vice versa.

Similarly, a high allocation of water to the non-power uses in the low head scheme improves economic benefits. Thus the ability of the model to pinpoint an optimal allocation policy for a particular set of conditions or design parameters cannot be overemphasized. In this light, the use of the model is recommended for the provision of an optimal allocation policy. The construction of allocation/design charts for different reservoir conditions and sizes similar to figures 10.1 is feasible and may be explored by future researchers.

(2) **Minimum Operating Levels**

The maximisation of the economic returns from the policy of dead storage minimisation occurs more readily in conventional low to medium head dams. This is no particularly substantial or effective in the high head case for reasons already given. The potential for the development of charts relating minimum level lowering and percentage (increase) in benefits by the use of the River Basin Management System exists.

(3) **Landuse**

EPXM

Afforestation and increased vegetation tend to improve benefits from reservoirs slightly in the short and long terms, especially when annual runoffs are moderate to large (e.g 1976-77 water year). The possible explanation is that the pattern of sedimentation from afforestation improves benefits despite the observation that runoff and sediment yields are not much affected (refer chapter 8 and 9). It must be noted that the EPXM change applied here are extreme and are not likely to occur in the short term. This parameter is therefore relatively insensitive in realistic terms (figure 10.4).

IMPV and DIST

Changes in the urbanisation and the cultivation disturbance parameters, IMPV and DIST respectively, do not have much influence on reservoir benefits as observed from figures 10.4. This may be due to the fact that the start values of these parameters in the catchment are low. The changes applied to them are also low but realistic because in a period as short as one year, there are not likely to be large changes.

(4) Reservoir Induced Climate

From rise in short term benefits from the effect of changing climate due to impoundments in the Donga Basin is due to the fact that the influence of increased runoff neutralises the effect of sediment deposition in the live storage of the reservoir. This rise is small when considered on an annual basis because of the large influence of reservoir sedimentation.

10.7 Summary of Research Findings and Conclusions

From the results, analysis, and discussions above, it is clear that the River Basin Management System, in its present form, has a great potential for analysing alternative reservoir design options and in the decision making process concerning the optimal design choice of projects in River Basin Development. By analysing the effect of water allocation, the location of the minimum operating levels, reservoir evaporation and sedimentation, landuse effect, and the effect of local climatic change from reservoir impoundments, the system offers itself as a useful tool for the optimisation of reservoir design and operations.

The allocation policy adopted in Reservoir management should be closely related to such reservoir parameters as head, size, main objectives of the reservoir (i.e. flood control, irrigation, power etc.), so as to obtain an optimal strategy for the use of the available water.

The policy of optimisation of minimum operating levels does improve benefits substantially. This policy of reduced dead storage capacity, is a more realistic and beneficial option than that of current practice, which specifies an excessively high operating levels is recommended especially in times of adverse climatic conditions.

The effect of afforestation, deforestation, and vegetation on the economic returns reservoirs are insignificant in the short term. Benefits tend to improve slightly with afforestation and increased catchment vegetation. They tend to reduce with deforestation. Deforestation is a common practice in Nigeria, the Donga Basin is not excluded. This should be discouraged if proposed storage project schemes are to give improved performance.

As catchment disturbance from agricultural practices are almost non existent in the Donga Basin, small test changes in these parameters do not influence the reservoir economic response significantly

Similarly, when a large catchment area, such as the Donga, is considered, the influence of urbanisation, which is currently small, does not seem to be significant from model results. In the short term, urbanisation is not likely to increase to any appreciable extent.

The short term influence of reservoir induced increases in precipitation is significant. Economic benefits tend to increase especially in years fairly high mean annual runoff and runoff peaks. The influence of reservoir evaporation alone is less significant in the shot term than when it is combined with precipitation increases. In the long term, however, the cumulative effect may be different and substantial. The steady state and short term scenarios tested herein will be complicated,

(a) in the long term by unsteady state changes introduced by the greenhouse effect.

(b) long term reservoir sedimentation

These aspects are analysed in chapter 11.

CHAPTER 11

CHAPTER 11 : GEMBU RESERVOIR MANAGEMENT, DONGA RIVER BASIN TESTING LONGTERM ECONOMIC VIABILITY FROM CLIMATE CHANGES ANALYSIS AND DISCUSSION

The main objective of this chapter is to confirm by testing, the adequacy of the proposed Gembu reservoir, as designed by Diyam and Binies (1985), in withstanding the future climatic changes expected throughout its life in the next half century using the River Basin Management System. This is to complement the tests involving steady state and short term scenarios undertaken in chapter 10. Section

11.1 Introduction

In chapter 1 the problem of the ongoing changes in the World's climate was introduced. These changes have been generally excluded in River Basin Management Practice uptill now because of the lack of information on their course, causes and effects. Their effects may be significant and can make the difference between the viability and the non-viability of a water resources project in the long term. In Chapter 7, a detailed description of these climatic changes including their probable causes, course and effects, accompanied by facts and figures are given. The expected long term climate changes in the Donga Basin are deduced from Global projections as obtained from the literature. These are used herein for the reservoir design and management economic analysis problem for the case study, just as was done for landuse, water and sediment balance in the preceeding chapters.

The Gembu reservoir is singled out and used for the analysis because it appears to be the most viable in the basin for both hydropower and multipurpose benefits because of its large storage volume and the presence of a highhead potential (960m). Further, hydrological and landuse data in the reservoir area are well documented (Diyam, 1985; MRT,1978; ALTS/LIDECO, 1978). The shaded portion of the River Basin Management System shown in figure 10.1 depicts the economic model which carries out complete economic analysis of the case study presented in this chapter when subjected to both current and future conditions.

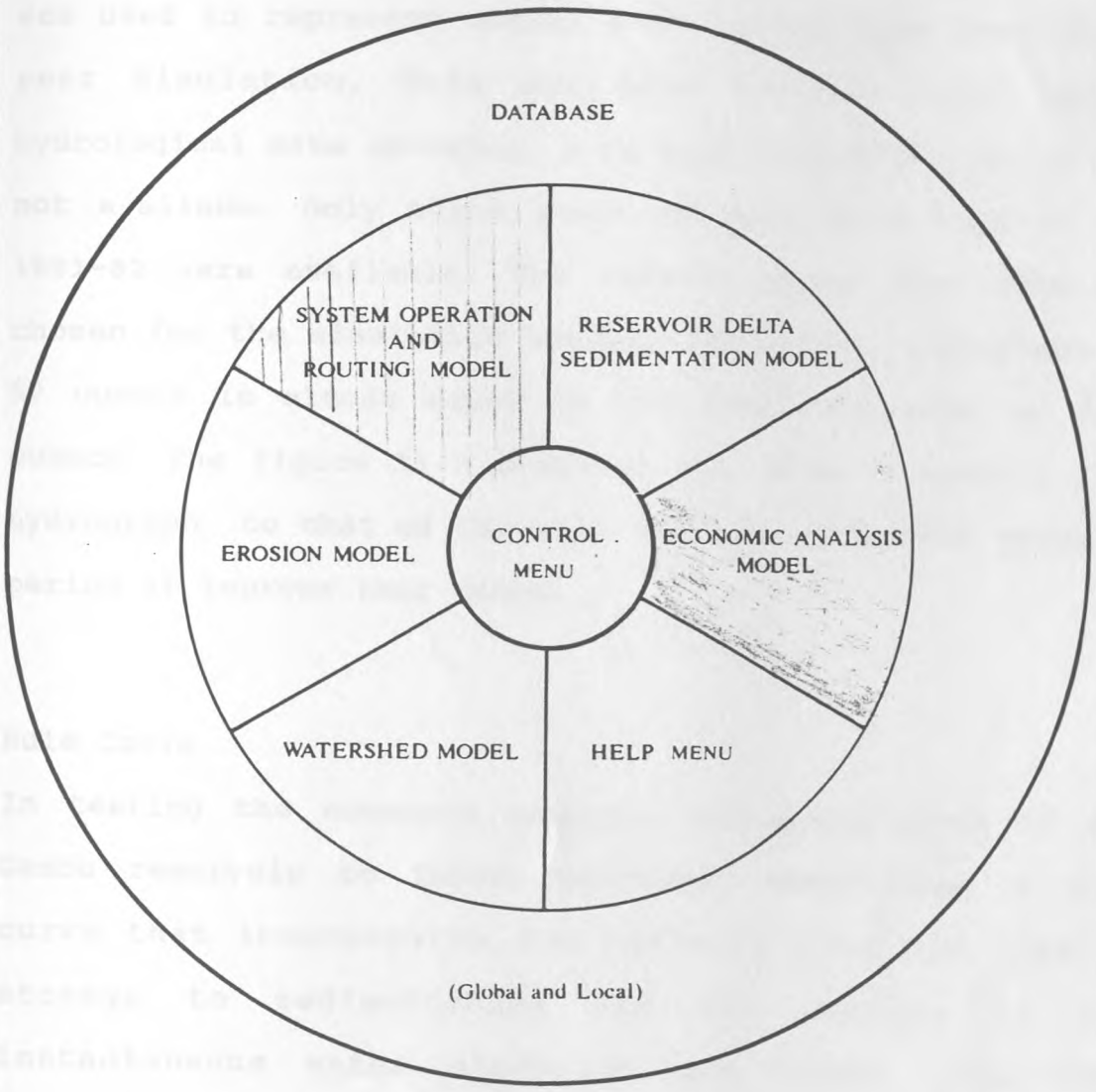


Fig. 11.0 RIVER BASIN MANAGEMENT SYSTEM

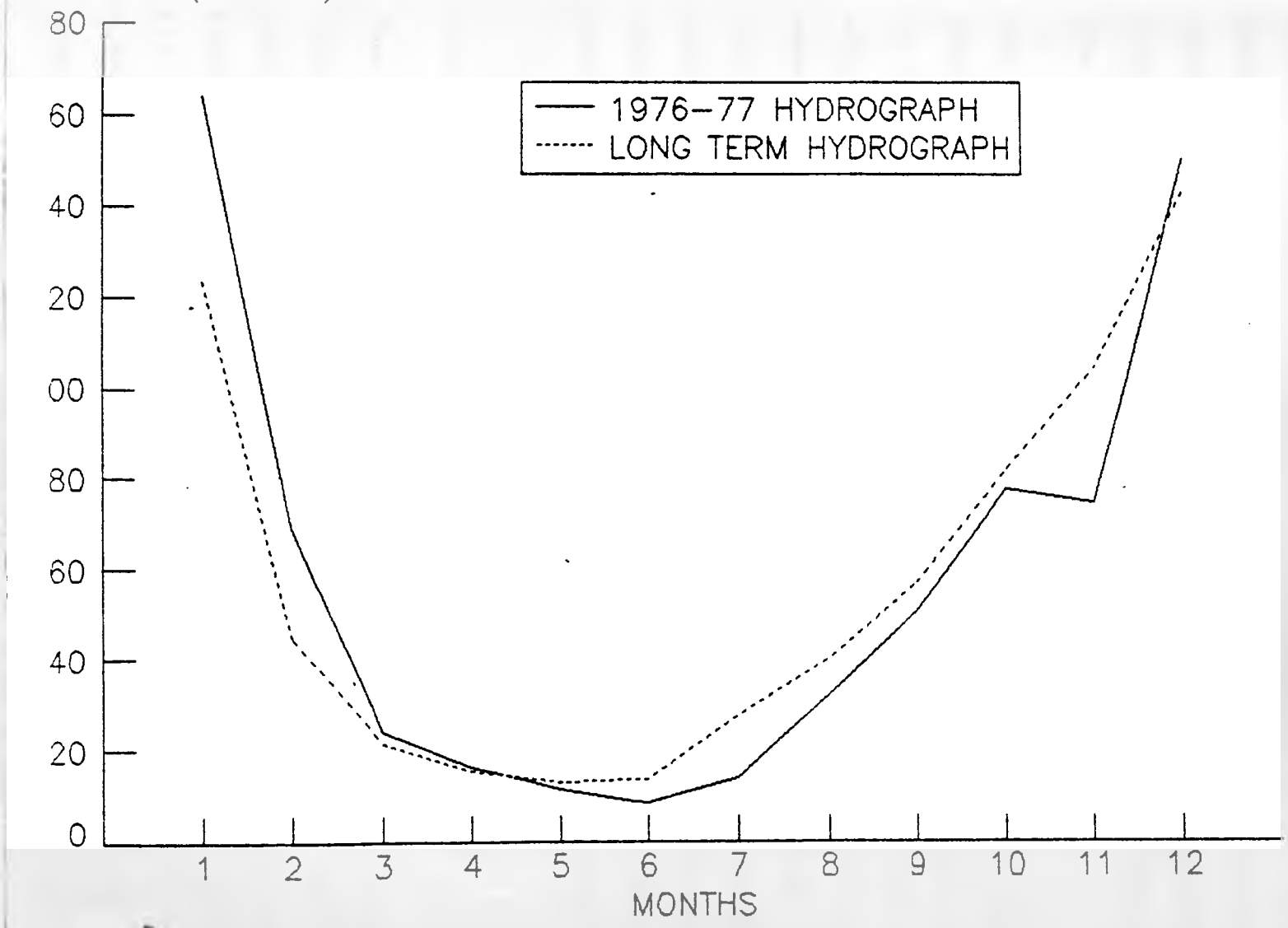
This system is available for use on both micro and mainframe(VAX) computers.

In carrying out the longterm simulations, the 1976-77 water year which has conditions similar to the longterm average was used to represent annual time series data for the 50 year simulation. This was done becuase time series hydrological data covering a 50 year simulation period was not availabe. Only three years of data from 1976-78 and 1981-82 were available. The 1976-77 water year data was chosen for the simulation because its annual mean flows of 57 cumecs is almost equal to the long term mean of 57.8 cumecs. The figure 11.2 compares the 1976-77 monthly flow hydrograph to that of the mean for the 1959-1981 standard period at Tepkwar near Gembu.

Rule Curve

In testing the economic response and performance of the Gembu rescrvoir to future catchment conditions, a rule curve that incorporates and reflects both the loss of storage to sedimentation and the changes in the instantaneous water elevation was chosen. The rule basically involves withdrawing water instantaneously at any elevation at a rate proportional to the amount of water in storage at that instance and at that elevation. It does not restrict any fraction of the reservoir capacity to any use in particular and the lake is allowed to build up and spill freely.This will help in monitoring, virtually, all the macroprocesses interacting in the reservoir regime in order to predict their behaviour and effect on water yields. A similar rule curve was utilised by Acres (1985)

g 11.1 COMPARISON OF HYDROGRAPHS
LONG TERM AND 1976–77 WATER YEAR H.GRAPH



and Merz and McLellan (1986) in a study review of the performance of the Akosombo reservoir in Ghana and the study of the rehabilitation of the Tema thermal power station respectively.

Evapotranspiration and precipitation increases in the Donga Basin are expected to occur, due to both the greenhouse effect and the presence of numerous impoundments within the catchment and in the two neighbouring Basins, the Teraba to the east and the Katsina-Ala to the west (chapter 7).

The precipitation increases expected over the 50 year period due to these two factors range from a minimum of 14% to a maximum of 24%. The Gembu reservoir scheme will be subjected to a series of simulation experiments involving several test evaporation and precipitation increases. In this subsection, increases of 0%, 7.5%, 10%, and 12.5% for evapotranspiration are considered and used for the economic simulation study. Precipitation increases of 14.0%, 16.5%, 20.0%, and 24.0% are progressively used in the simulations.

11.2 Presentation of Gembu Reservoir simulation results-Case 1

The summary of the results obtained for all the simulation runs for the first Gembu reservoir design are given in tables 11.1 (1) - (5) for benefit cost ratios, net present worths, and net benefits/net annual worths respectively. All money values are in United States dollars. The computer outputs which show the details and breakdown of each run are provided in tables A (a)-(x) in the appendix

	HYDROPOWER	EVAP LOSS	SPILLAGES
DISCHARGE CMS	0.4941E+02	0.7929E+00	0.0000E+00
MEAN POWER MW	0.3466E+03	0.6180E+01	0.0000E+00
MEAN ENERGY KWH	0.3036E+10	0.5414E+02	0.0000E+00

CAPITA 0.4632E+05 HECT 0.1537E+03

FIRST COSTS	0.2759E+09	0.2759E+08
ALTNT COSTS	0.4292E+09	0.4292E+08

NONPOWR GR.TOTAL	POWER	IRRIGATION	W.SUPPLY	OTHERS	TOT.NQN PW	TØTAL
PRESENT VALUES-COSTS	0.3061E+09	0.6710E+05	0.1700E+07	0.6253E+08	0.6429E+08	0.4428E+09
PRESENT VALUES-BENFTS	0.1708E+10	0.4323E+05	0.3909E+07	0.7306E+07	0.1126E+08	0.1719E+10
PRESENT WORTHS VALUES	0.1402E+10	-.2387E+05	0.2209E+07	-.5522E+08	-.5303E+08	0.1277E+10
9ENEFIT COST RATIO	0.5581E+01	0.6442E+00	0.2299E+01	0.1169E+00	0.1751E+00	0.3883E+01
ANNUAL COSTS VALUES	0.2473E+08	0.5422E+04	0.1373E+06	0.5052E+07	0.5195E+07	0.3578E+08
ANNUAL BENEFIT VALUES	0.1380E+09	0.3493E+04	0.3158E+06	0.5903E+06	0.9096E+06	0.1389E+09
ANNUAL WORTH VALUES	0.1133E+09	-.5189E+04	0.1785E+06	-.4462E+07	-.4285E+07	0.1031E+09
ANNUAL BEN.COST RATIO	0.5581E+01	0.6442E+00	0.2299E+01	0.1169E+00	0.1751E+00	0.4927E+01
UNDISCOUNTED COSTS	0.8030E+09	0.1851E+06	0.3698E+08	0.2223E+09	0.1504E+10	0.2307E+10
UNDISCOUNTED BENFTS	0.3526E+11	0.9436E+06	0.8532E+08	0.4061E+09	0.4924E+09	0.3527E+11
BEN.CST RATIO-UNDISCOUNTED	0.4391E+02	0.9680E+01	0.2307E+01	0.1827E+01	0.3274E+00	0.1529E+02

TABLE 11.0 ECONOMIC ANALYSIS&OPERATIONS OUTPUT

11. The values given in tables 11.1 (1)-(5) are plotted in figures 11.3 (1)-(5) for visual comparisons.

The reservoir capacity of the Gembu reservoir option tested herein is about 6,000 million cumecs and the maximum operating elevation is 1310 masl. The reservoir depositional area is represented by a regular square grid system of side 200m each by the delta simulation model which simulates sediment deposition in the reservoir.

TABLE 11.1 (1) BENEFIT COST RATIOS FROM ECONOMIC SIMULATOR

EVAP-Global + 0% PPT +	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	2.884	2.507	2.111	1.507	0.922	0.105
16.5%	3.319	3.054	2.738	2.309	1.762	1.080
20.0%	4.188	3.956	3.714	3.381	2.992	2.473
24.0%	5.581	5.366	5.152	4.904	4.616	4.294

VALUE FROM CURRENT GLOBAL CONDITIONS+0%PPTN= 6.372
N.B RESERVOIR EVAPORATION IS INCLUDED BY MODELS IN ALL SIMULATIONS

TABLE 11.1 (2) NET PRESENT WORTH VALUES millions of U.S.dollars

EVAP-Global+ PPT +	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	554.4	416.1	340.3	179.8	-23.8	-274.1
16.5%	709.7	628.6	531.9	400.7	233.3	24.5
20.0%	975.6	904.8	830.6	728.7	609.6	450.0
24.0%	1402.0	1336.0	1271.0	1195.0	1107.0	1008.0

VALUE FROM CURRENT CONDITIONS+0%PPTN = 1644.0
N.B. RESERVOIR EVAPORATION IS INCLUDED BY MODEL IN ALL SIMULATIONS

TABLE 11.1 (3) NET ANNUAL WORTH VALUES millions of U.S. dollars

EVAP-Global+ PPT +	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	47.79	37.26	27.50	14.53	-1.93	-22.14
16.5%	82.08	50.79	42.98	32.38	18.85	1.98
20.0%	78.83	73.11	67.11	58.88	49.26	36.42
24.0%	113.30	108.00	102.70	96.54	89.42	81.47

VALUE FROM CURRENT CONDITIONS+0%PPTN = 132.80

N.B. RESERVOIR EVAPORATION IS INCLUDED BY MODEL IN ALL SIMULATIONS

TABLE 11.1 (4) PRESENT VALUE OF BENEFITS millions of U.S.dollars.

EVAP/Global+ PPT+	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	860.40	767.20	646.40	485.90	282.20	32.00
16.5%	1016.00	934.70	838.00	706.80	539.40	330.60
20.0%	1282.00	1211.00	1137.00	1035.00	915.70	765.90
24.0%	1708.00	1642.00	1577.00	1501.00	1413.00	1314.00

VALUE FROM CURRENT CONDITIONS+0%PPTN = 2321.00

N.B. RESERVOIR EVAPORATION IS INCLUDED BY MODEL IN ALL SIMULATIONS.

TABLE 11.1 (5) ANNUAL BENEFITS millions of U.S.dollars

EVAP/Global+ PPT+	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	69.52	61.99	52.23	39.26	22.80	2.59
16.5%	82.08	75.52	67.71	57.11	43.58	26.71
20.0%	103.60	97.84	91.84	83.61	73.99	61.89
24.0%	138.00	132.70	127.40	121.30	114.20	106.20

VALUE FROM CURRENT CONDITIONS+0%PPTN = 187.50

N.B. RESERVOIR EVAPORATION IS INCLUDED IN ALL SIMULATIONS.

GEMBU RESERVOIR ECONOMIC ANALYSIS
EFFECT OF CLIMATE CHANGE ON BENEFITS

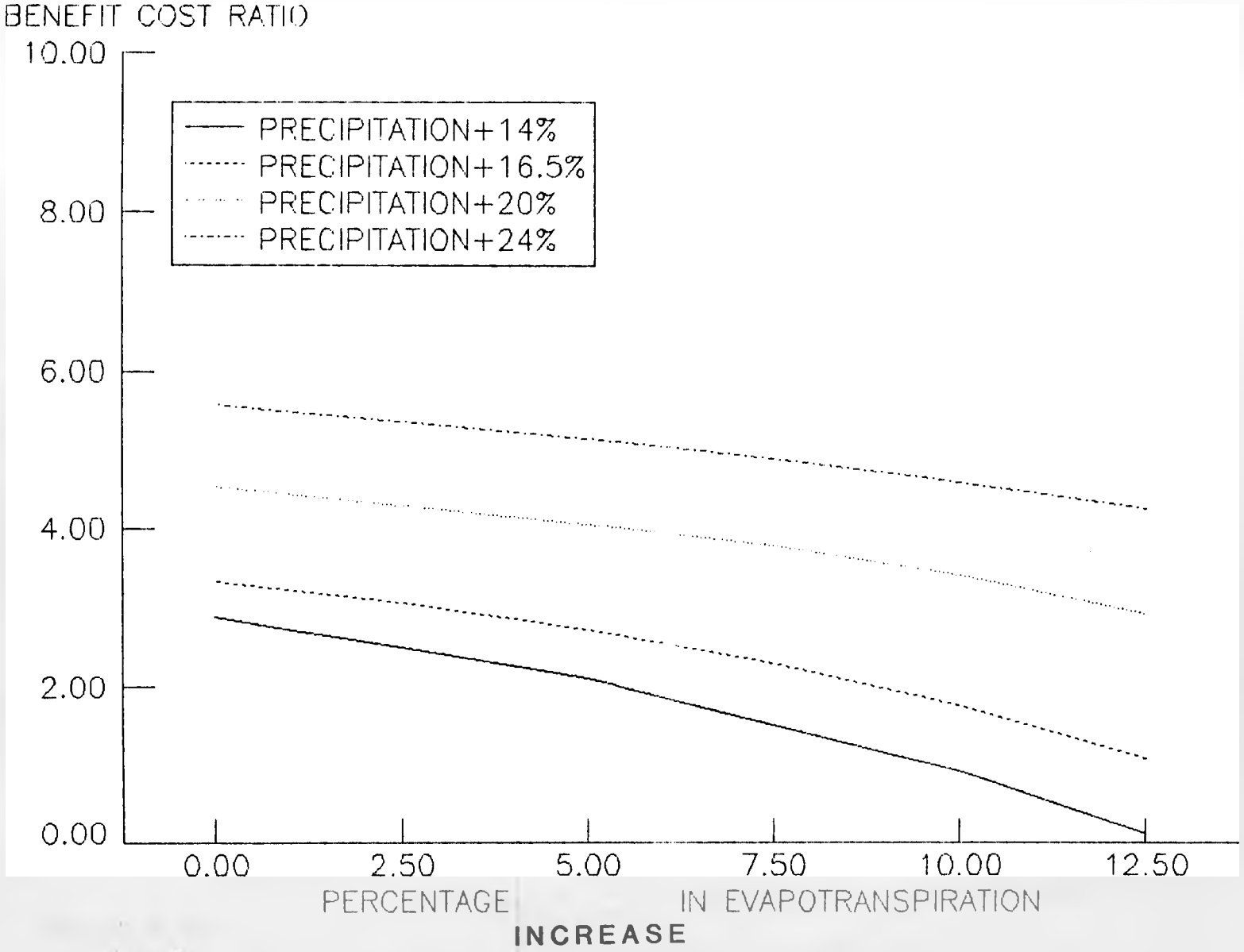


Fig. 11.2(a)

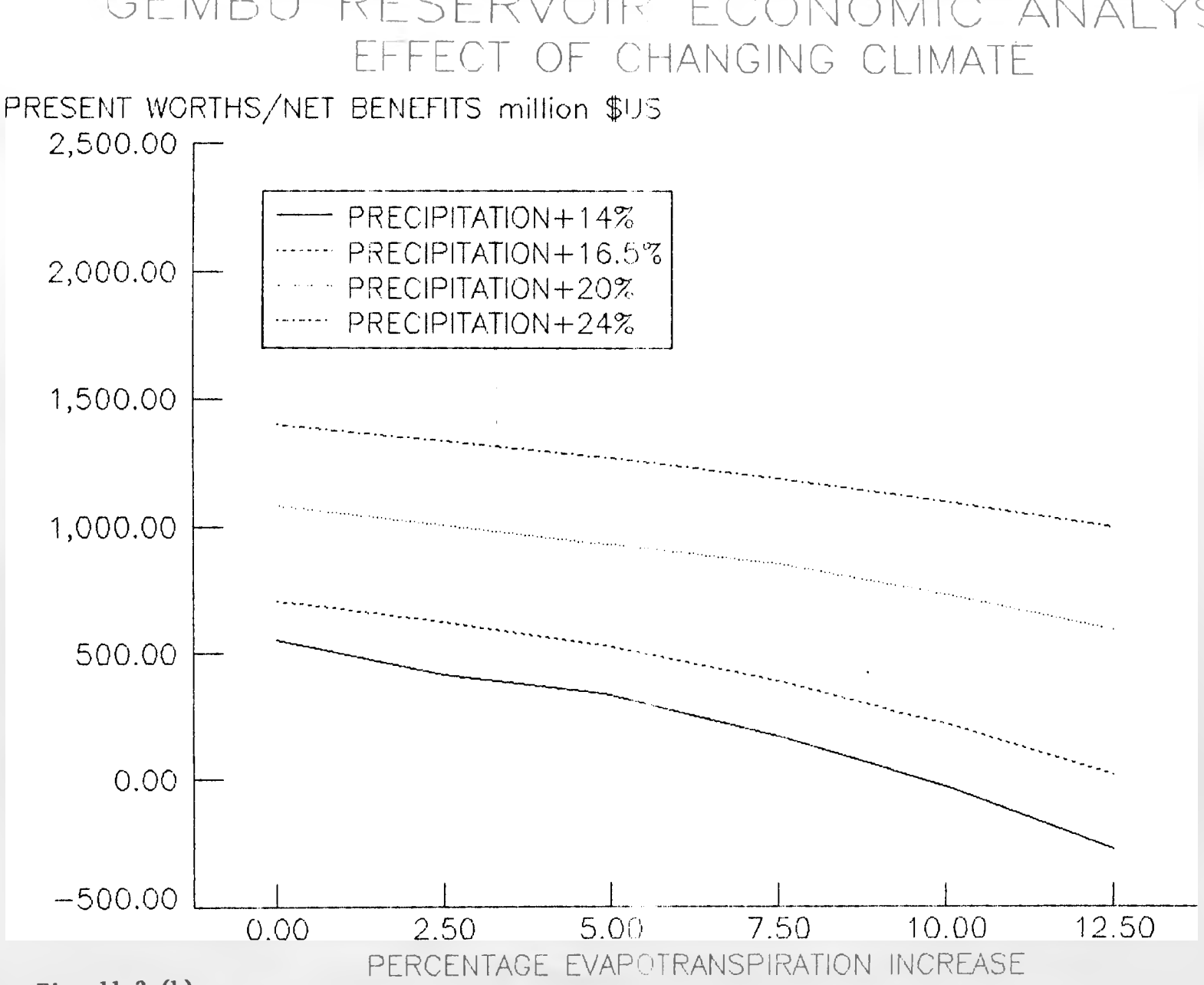

GEMBU RESERVOIR ECONOMIC ANALYSIS
EFFECT OF CHANGING CLIMATE
PRESENT WORTHS/NET BENEFITS million $US
2,500.00
2,000.00
1,500.00
1,000.00
500.00
0.00
-500.00
PRECIPITATION+14%
PRECIPITATION+16.5%
PRECIPITATION+20%
PRECIPITATION+24%
0.00
2.50
5.00
7.50
10.00
12.50
PERCENTAGE EVAPOTRANSPIRATION INCREASE

Fig. 11.2 (b)

GEMBU RESERVOIR ECONOMIC ANALYSIS
EFFECT OF CLIMATIC CHANGE

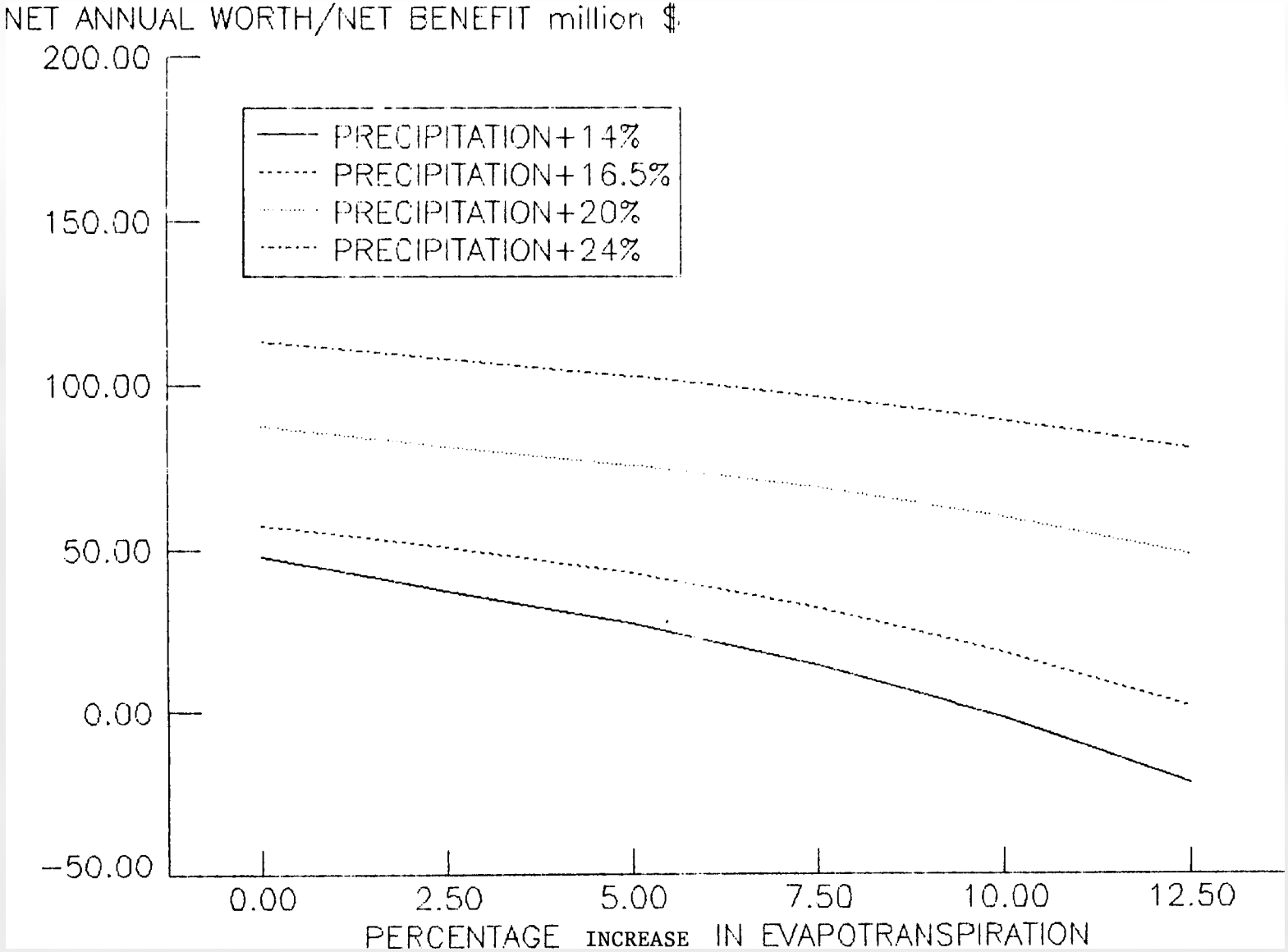


Fig. 11.2 (c)

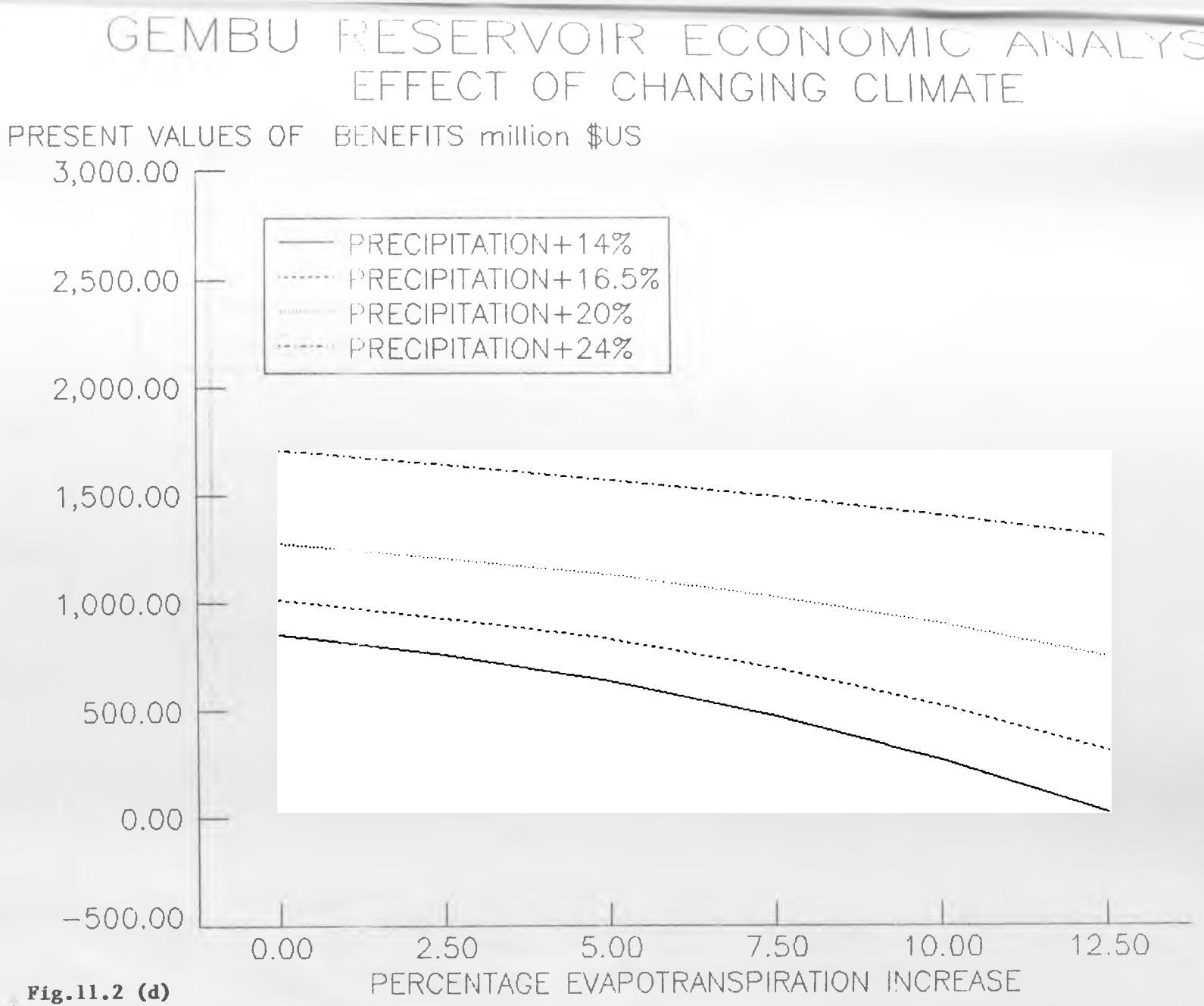

GEMBU RESERVOIR ECONOMIC ANALYSIS
EFFECT OF CHANGING CLIMATE
PRESENT VALUES OF BENEFITS million $US
3,000.00
2,500.00
2,000.00
1,500.00
1,000.00
500.00
0.00
−500.00
PRECIPITATION+14%
PRECIPITATION+16.5%
PRECIPITATION+20%
PRECIPITATION+24%
0.00
2.50
5.00
7.50
10.00
12.50
PERCENTAGE EVAPOTRANSPIRATION INCREASE

Fig.11.2 (d)

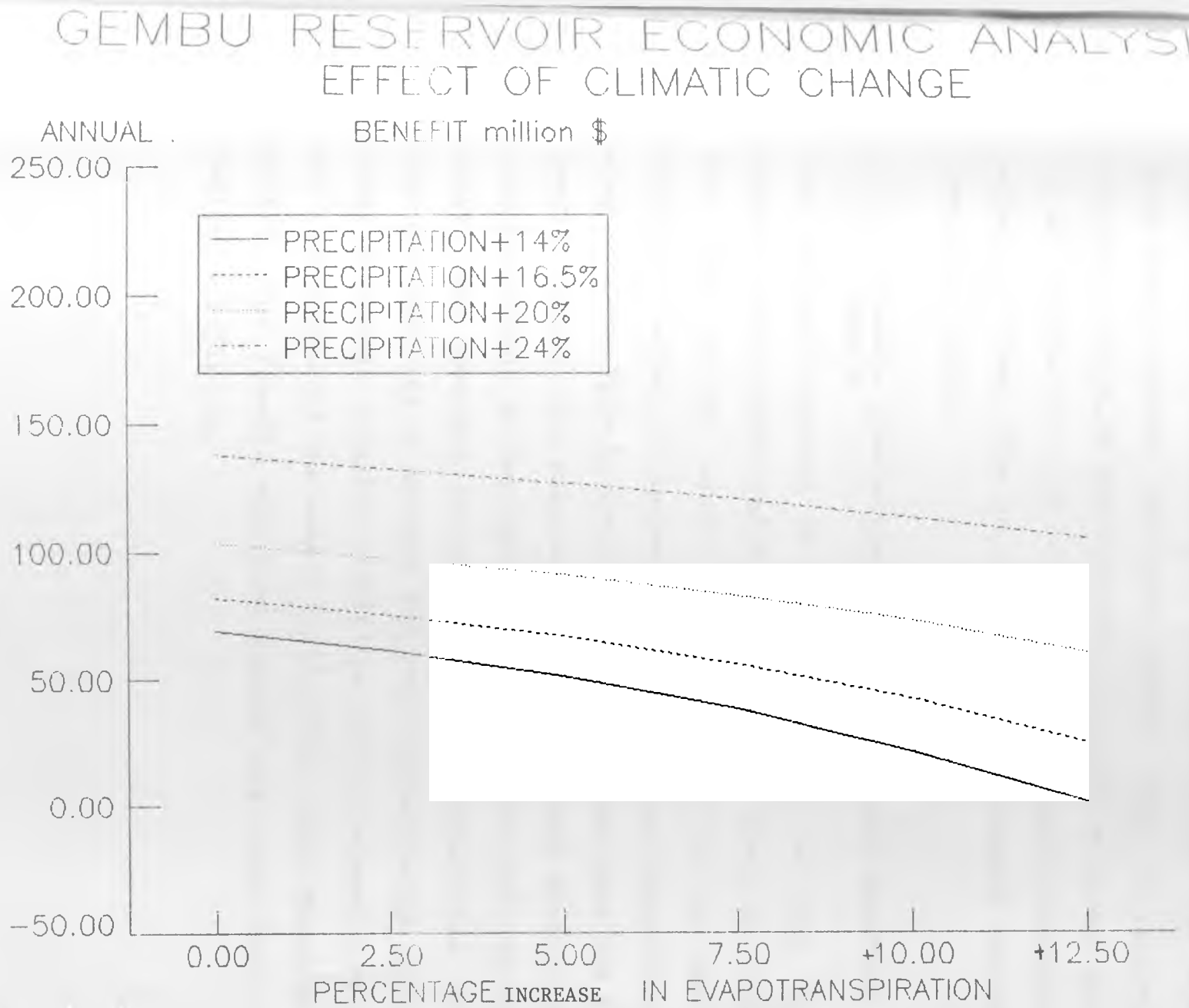

GEMBU RESERVOIR ECONOMIC ANALYSIS
EFFECT OF CLIMATIC CHANGE
ANNUAL BENEFIT million $
250.00
200.00
150.00
100.00
50.00
0.00
-50.00
PRECIPITATION+14%
PRECIPITATION+16.5%
PRECIPITATION+20%
PRECIPITATION+24%
0.00
2.50
5.00
7.50
+10.00
+12.50
PERCENTAGE INCREASE IN EVAPOTRANSPIRATION

Fig. 11.2 (e)

It is important to note that in all runs, reservoir evaporation is automatically accounted for by the model. From model outputs, the mean annual value of reservoir evaporation is 0.98 cubic metres per second or 114mm, which is an increase of 11.6% over current catchment evaporation values of 983mm. This current evaporation value is obtained from Diyam (1985).

11.3 Economic Simulation Results for Gembu Reservoir - Case 2

The proposed capacity of this new design is about 7600 million cubic metres and the maximum lake level is 1313m. The size of the square grid utilised by the Delta Simulation model in representing the reservoir depositional area is 220m. This is 20m (10%) more than that used for the smaller design. The runs were carried out with exactly the same conditions as that performed for the original design, above. This exercise is carried out for comparisons with the original case to examine the more optimal design option under different climatic conditions and to assess some of the limitations of the models in the River Basin Management System. The results obtained herein are summarised in tables 11.2 (1)-(5).

TABLE 11.2 (1) BENEFITS COST RATIOS

EVAP/Global+ PPT+	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	3.014	2.981	2.800	2.642	2.444	2.213
16.5%	3.246	3.155	3.054	2.929	2.771	2.580
20.0%	3.663	3.562	3.454	3.347	3.217	3.071
24.0%	4.177	4.064	3.945	3.821	3.697	3.562

VALUE FROM CURRENT OR TODAYS CONDITIONS+0%PPTN = 7.584
N.B RESERVOIR EVAPORATION IS INCLUDED IN ALL SIMULATIONS.

TABLE 11.2 (2) PRESENT WORTH OF BENEFITS in millions of U.S$

EVAP/Global+ PPT+	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	616.5	587.10	550.80	502.5	442.00	371.20
16.5%	687.0	659.70	628.60	590.6	542.20	483.50
20.0%	815.2	784.10	751.30	718.4	678.70	633.80
24.0%	972.4	937.80	901.50	863.5	825.50	784.00

VALUE FROM CURRENT CONDITIONS+0%PPTN = 2015.0

BRESERVOIR EVAPORATION IS INCLUDED IN ALL SIMULATIONS

TABLE 11.2 (3) ANNUAL WORTH OF BENEFITS in millions of U.S$

EVAP/Global+ PPT+	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	49.81	47.44	44.50	40.60	35.71	29.99
16.5%	55.53	53.30	50.79	47.72	43.81	39.06
20.0%	65.87	63.35	60.70	58.05	54.84	51.21
24.0%	78.57	75.77	72.84	69.77	66.70	63.35

VALUE FROM CURRENT CONDITIONS+0%PPTN = 162.80

TABLE 11.2 (4) PRESENT VALUE OF BENEFITS in millions of U.S$

EVAP/Global+ PPT+	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	922.60	892.20	856.90	808.50	748.10	677.30
16.5%	993.40	965.70	934.60	896.60	848.30	789.60
20.0%	1121.00	1090.00	1057.00	1024.00	984.70	939.80
24.0%	1278.00	1244.00	1208.00	1170.00	1132.00	1090.00

VALUE OF FROM CURRENT CONDITIONS+0%PPTN = 2321.00

TABLE 11.2 (5) ANNUAL BENEFITS in millions of U.S dollars

EVAP/Global+ PPT+	0%	2.5%	5.0%	7.5%	10.0%	12.5%
14.0%	74.54	72.17	69.23	65.33	60.44	54.72
16.5%	80.27	78.03	75.52	72.45	68.54	63.79
20.0%	90.60	88.08	85.43	82.78	79.57	75.94
24.0%	103.30	100.50	97.57	94.50	91.43	88.08

VALUE FROM CURRENT CONDITIONS+0%PPTN = 187.50

The results of the larger Gembu reservoir simulation summarised in tables 11.2 (1) - (5) are plotted in figures 11.4 (a) - (e) for visual appreciation.

11.4 General Analysis of Results with Comments

The figure 11.5 show some of the processes acting within the reservoir regime that influences its economic performance. These interacting processes listed and discussed below.

(1) The loss of live storage capacity to sedimentation tends to reduce economic benefits. This is discussed extensively in chapters one and two.

(2) The deposition of sediments in the reservoir raises water elevations. Heads for hydropower generation are increased accordingly. When a capacity/elevation rule is used for water withdrawals to various users, the raised elevations tend to increase the water yields and benefits. The increase in head also increases power benefits.

(3) The instantaneous water elevations also depends on the quantity of water inflows into the lake. The higher the inflows, the higher the instantaneous levels and the corresponding water abstractions.

(4) The ratio of the live storage to the dead storage capacity affects the benefits accruable from reservoirs as was indicated earlier in Chapter 1.

(5) In general, an increase in evapotranspiration reduces while a rise in precipitation increases water and sediment inflows into reservoirs.

(6) The above listed factors are always in action in reservoirs, counteracting and sometimes aiding one another. The resultant economic effect depends on the factors with the greatest influence under a particular set of condirtions.

11.5 Analysing Case 1 Results

General Pattern of Results for Case 1

From tables 11.1 and figures 11.3, it is observed that for each of the precipitation scenarios during the 50 year life of the project, the economic benefits reduce when the scheme is subjected to increases in evapotranspiration values. The reason may be that the influence of reduced runoff from evapotranspiration increases is greater than the effect of reduced sedimentation (see figure 9.6).

Tables 11.1 indicates that

(1) the economic returns obtainable from the Gembu reservoir will generally reduce in the long term with changing climatic conditions. The benefit cost ratio obtained, when current conditions are used in the simulation,is 6.372 compared to the range of 0.105 to 5.580 obtained by the use of expected future scenarios.

(2) benefits tend to improve with large increases in long term precipitation. This is because the large size of the Gembu reservoir allows the incomming large runoff to build up water elevations without spillage. This large amount of water in storage and the higher elevation allows for larger allocation of water to users.

Reservoir Failure Conditions for Case 1.

Three critical 'economic failure' conditions of future climate changes are observed for the smaller case. These three critical conditions as observed from thesimulation

results above (tables 9.6) are listed below.

(a)The reservoir completely fails, economically, during a 14.0% increase in precipitation and the 12.5% evapotranspiration increase simulation run, yielding a benefit cost ratio of only 0.105 compared to 6.37 for the no increase (current conditions) run.

(b) The reservoir just fails during the 14% precipitation and the 10% evapotransiration increase simulation run. The benefit cost ratio obtained is 0.922 which is far less than the 6.372 value obtained by the run involving current conditions.

(c) Complete financial failure was just averted in the run involving 16.5% and 12.5% increases in precipitation and evapotranspoiration respectively, with an economic benefit ratio of only 1.080.

The condition for failure utilised is the standard one used for the cost benefit analysis of engineering projects, that is, a failure occurs if the benefit cost ratios obtained from economic analysis is less than unity.

Changing Benefit Cost Ratios with Climate-Design Case 1

It is worthwhile to note from the results presented in tables 11.1 (1)- (5) that benefit cost ratios for various evaporation scenarios for case 1,

(1) reduce considerably from 6.372 to between 2.884 and 0.105 for a 14% increase in precipitation. This reduction ranges between 55% and 98%.

(1) reduce from 6.372 for current conditions to the range of 3.319 to 1.080 for the 16.5% precipitation increase case, a reduction in ratio of between 48% and 83%.

(2) for the 20% precipitation case range reduce from 4.537 to 2.986 for various evaporation scenarios and represent a 29%-53% reduction when compared to the 6.372 value for the current conditions run.

(3) for the 24% precipitation increase lie between 5.581 and 4.294 for various evaporation scenarios. These last range of figures are nearer the 6.372 value obtained from simulations involving current conditions, but this still constiutes a benefit cost ratio loss of 12%-33%.

(4) gradually increase with increasing runoff as rainfall increases. This increase is however not high enough to restore economic benefits to what is obtained when current conditions are simulated.

11.6 Analysing Case 2 vis-a-vis Case 1 results.

The results of simulations involving the use of a bigger capacity reservoir design for Gembu given in tables 11.2 confirm the general trend observed for the smaller capacity case, discussed above. Small differences are observed, though. The observations are listed below.

(a) The trend shows that economic benefits from reservoirs tend to reduce with increasing global evaporation values and tends to increase with large increases in precipitation.

(b) In general, however, in this case as in the last, benefits fall drastically from a value of benefit cost ratios of 7.79 to the range of 2.21 to 4.17 when this improved design of the reservoir is subjected to simulations involving projected future climatic changes. The actual monetary values involved are indicated in the tables 11.2 (2)-(5).

(c) For relatively small increases in precipitation (14-20%), benefits from the slightly larger reservoir design are a marked improvement on those obtained from the smaller case (original).

(d) In the larger reservoir case, though benefits are generally higher than less than the the smaller lake case, they are slightly less when precipitation increases are very large (from 20 to 24%) and as evaporation decreases from 10% to 0% inclusive (tables 11.1 and 11.2). This can be explained progressively by the points listed below.

(1) These are conditions of increasing and very large runoffs and therefore high sediment loads and deposition in the reservoirs.

(2) The larger reservoir, has about 10% more of depositional area at the inflow points. This allows for

more and a wider spread of sedimentation in its live storage region.

(3) The additional sediment deposits are less likely to be swept into the dead storage zone, since they are deposited at the reservoir bed under more or less stable conditions.

(4) This broader depositional pattern and the stability of the deposits in the live storage tend to reduce benefits.

(5) In this particular case, the effect of capacity reduction from sedimentation seems to be more influencial than the capacity gain from increased reservoir size.

From the above analysis results and the comparisons of two different reservoir designs, it is clear that the smaller design is more optimal for cases of extreme globally induced increases in catchment precipitation. In cases of moderate precipitation increases, clearly, a slight increase in the reservoir capacity may be required for achieving a more optimal performance in the long term.

In general, the drop in the values of economic benefits in both cases is much and this indicates that both designs are going to be greatly strained by future climate conditions, even with today's landuse practice patterns which is utilised in the analysis. The situation will be further aggravated if future landuse patterns become extensive and land management practices such as cattle ranching and grazing -land allocation, devegetation, farming (shifting cultivation), etc remain poor.

GEMBU RESERVOIR ECONOMIC ANALYSIS(large

EFFECT OF CLIMATE CHANGE ON BENEFITS

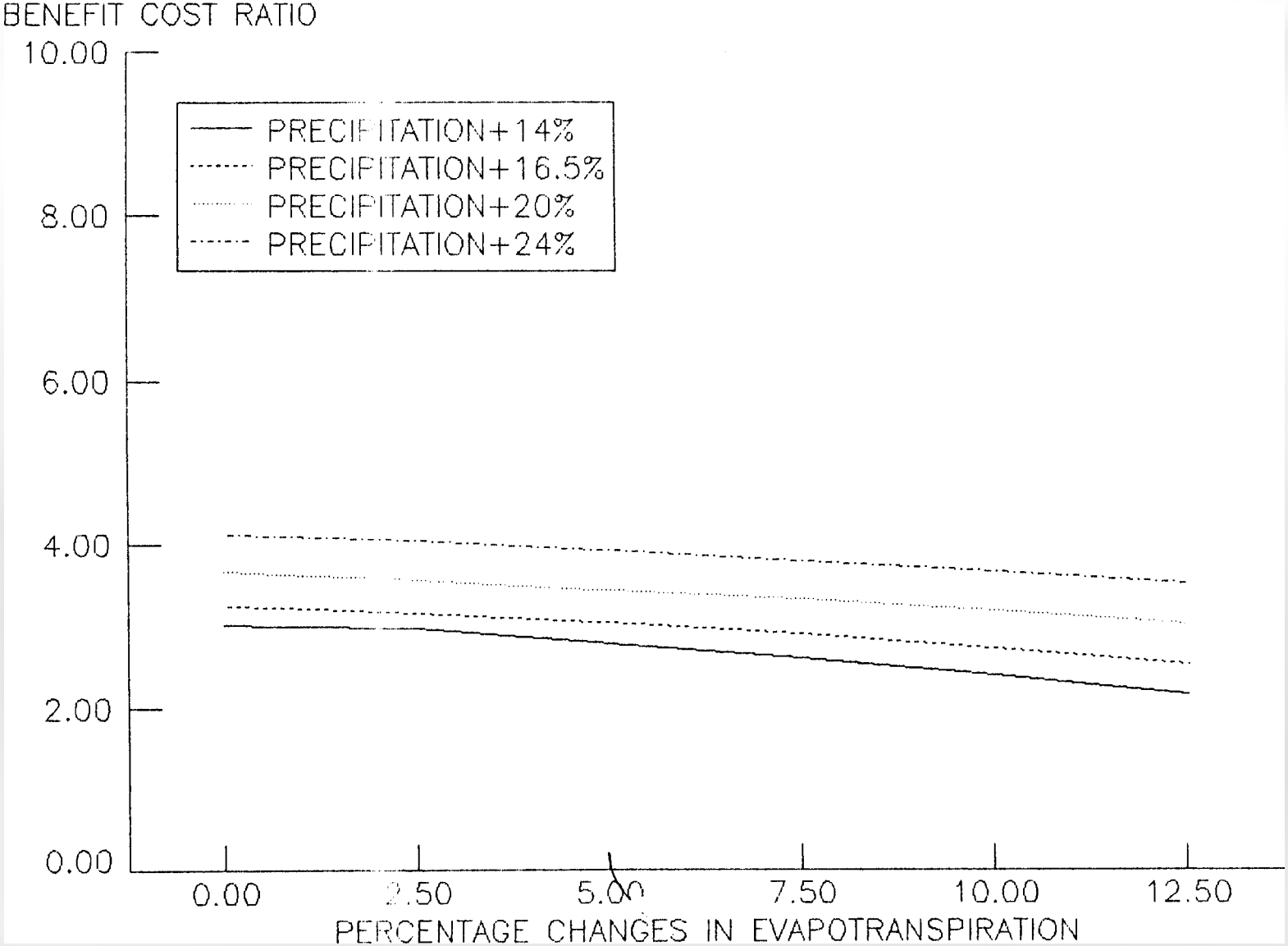


Fig. 11.3(a)

GEMBU RESERVOIR ECONOMIC ANALYSIS(larg

EFFECT OF CHANGING CLIMATE

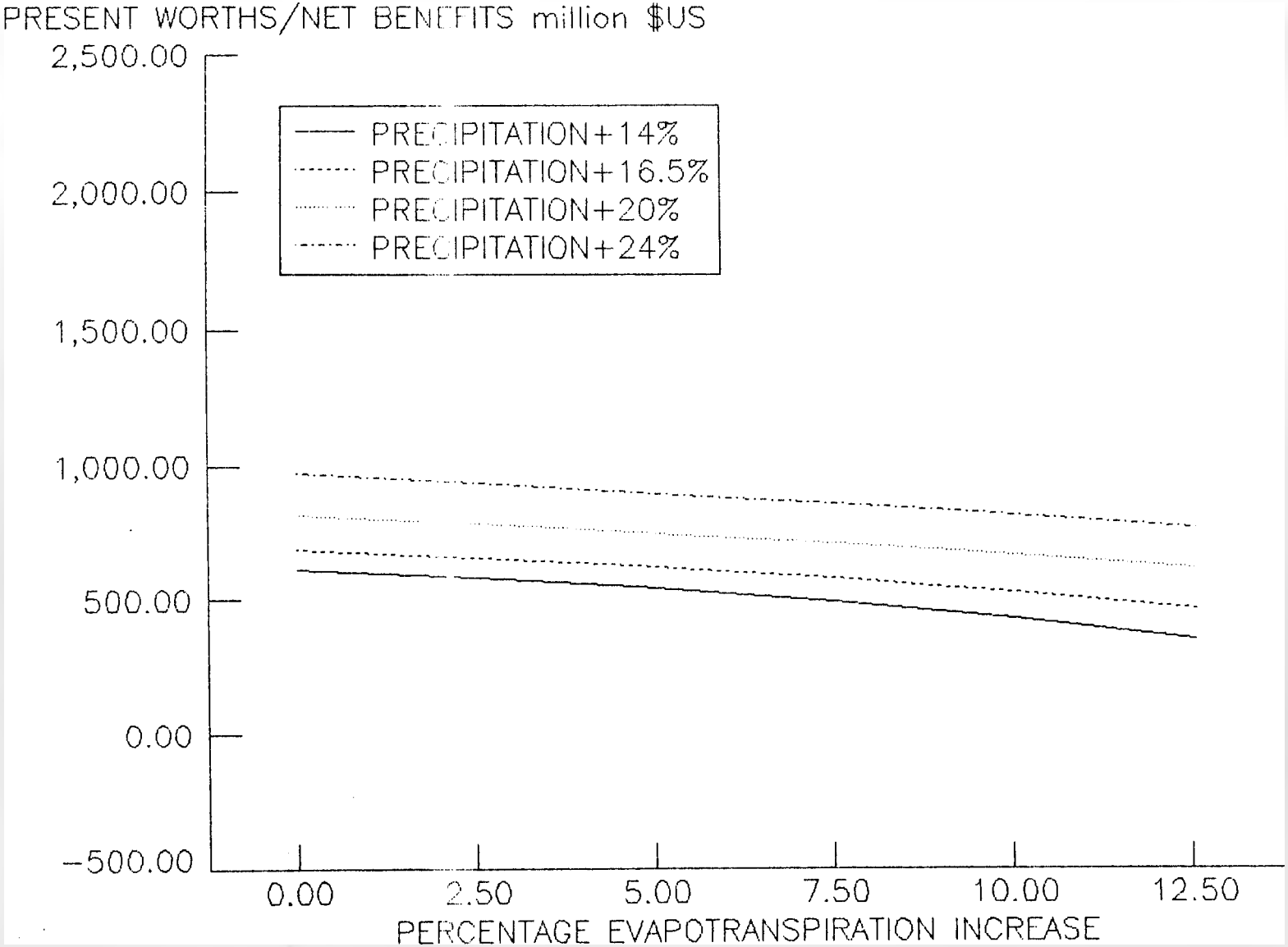


Fig. 11.3(b)

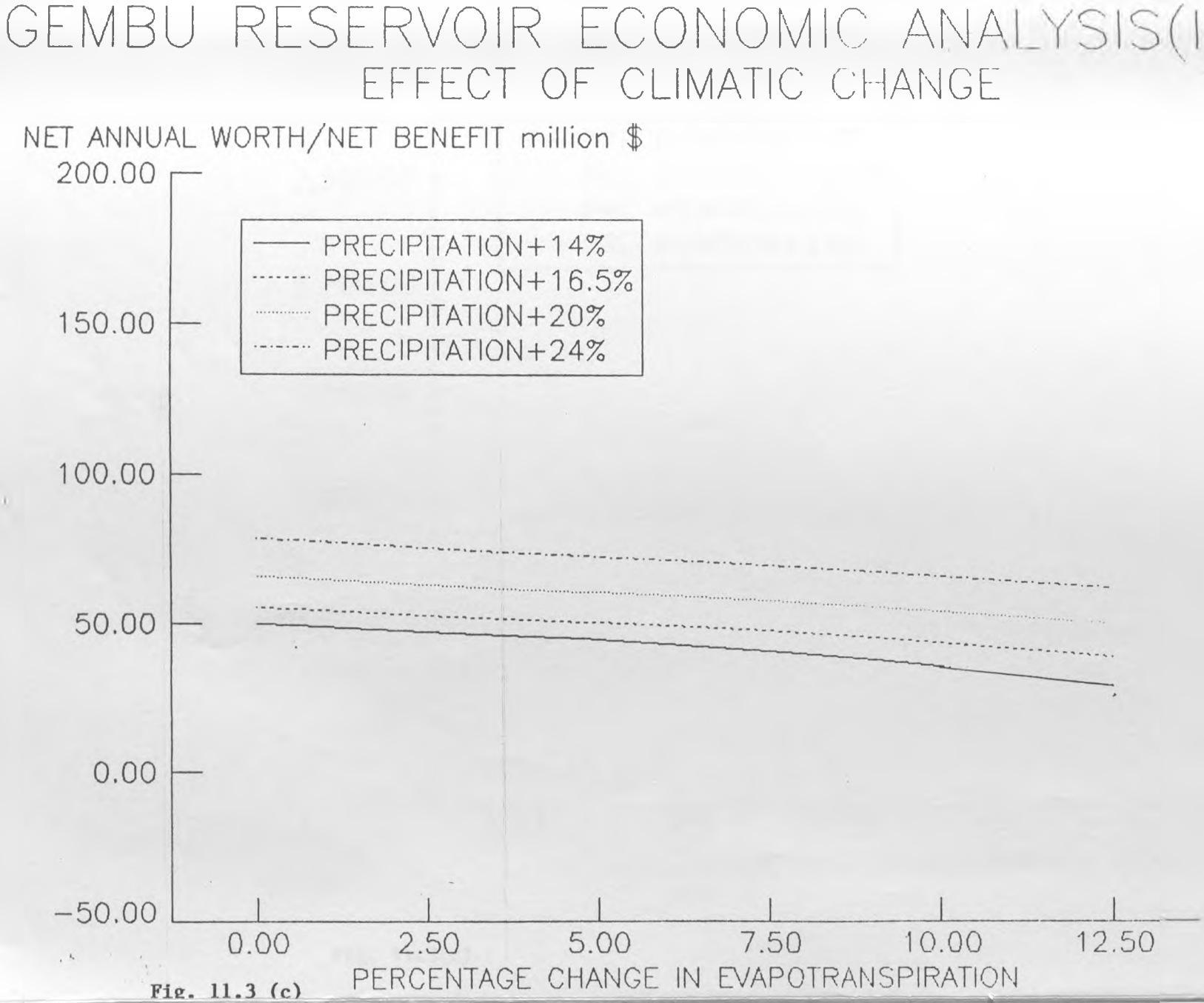

GEMBU RESERVOIR ECONOMIC ANALYSIS(larg
EFFECT OF CLIMATIC CHANGE
NET ANNUAL WORTH/NET BENEFIT million $
200.00
150.00
100.00
50.00
0.00
-50.00
PRECIPITATION+14%
PRECIPITATION+16.5%
PRECIPITATION+20%
PRECIPITATION+24%
0.00
2.50
5.00
7.50
10.00
12.50
PERCENTAGE CHANGE IN EVAPOTRANSPIRATION

Fig. 11.3 (c)

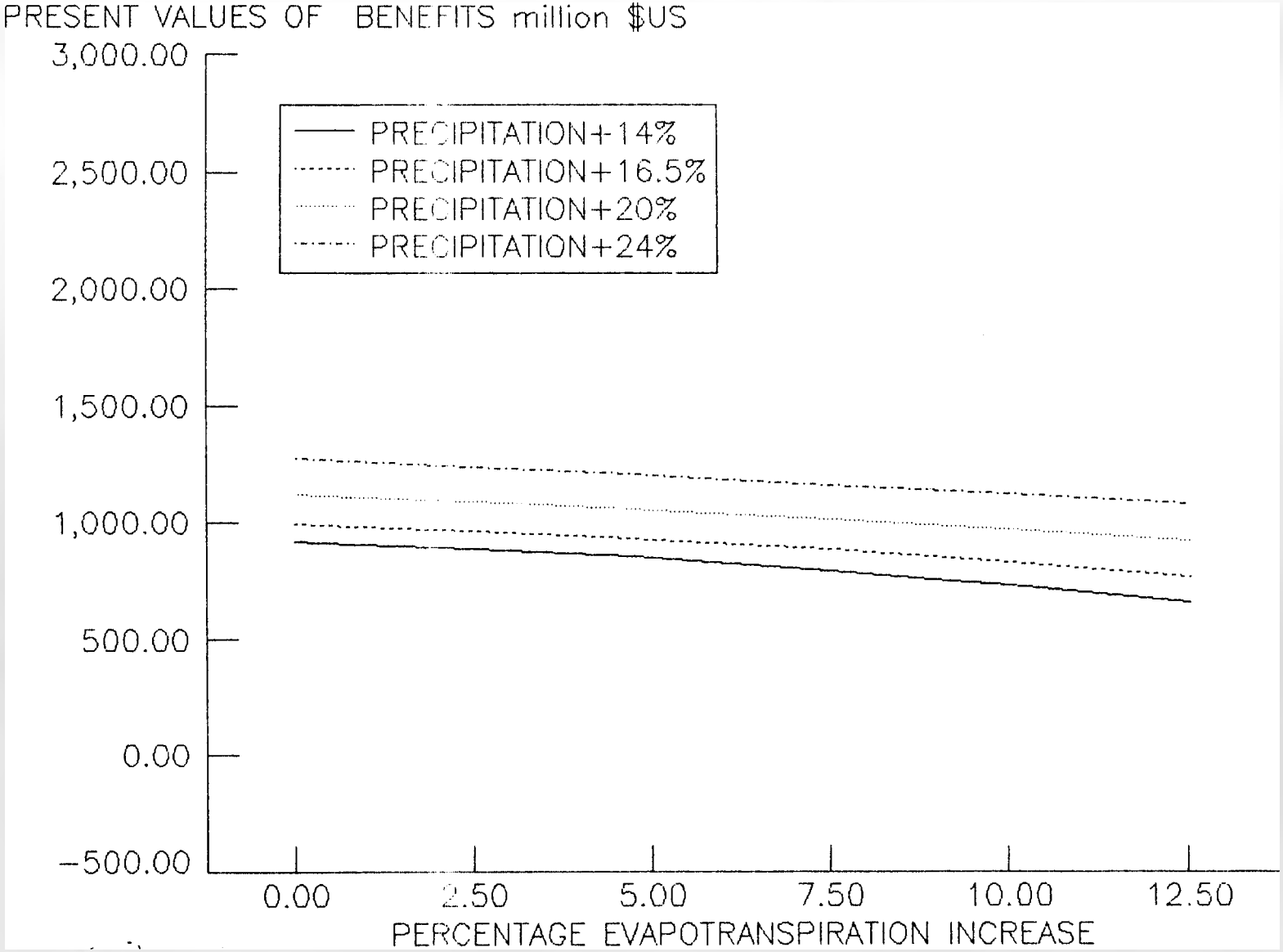

PRESENT VALUES OF BENEFITS million $US
3,000.00
2,500.00
2,000.00
1,500.00
1,000.00
500.00
0.00
-500.00
PRECIPITATION+14%
PRECIPITATION+16.5%
PRECIPITATION+20%
PRECIPITATION+24%
0.00
2.50
5.00
7.50
10.00
12.50
PERCENTAGE EVAPOTRANSPIRATION INCREASE

Fig. 11.3(d)

GEMBU RESERVOIR ECONOMIC ANALYSIS
EFFECT OF CLIMATIC CHANGE

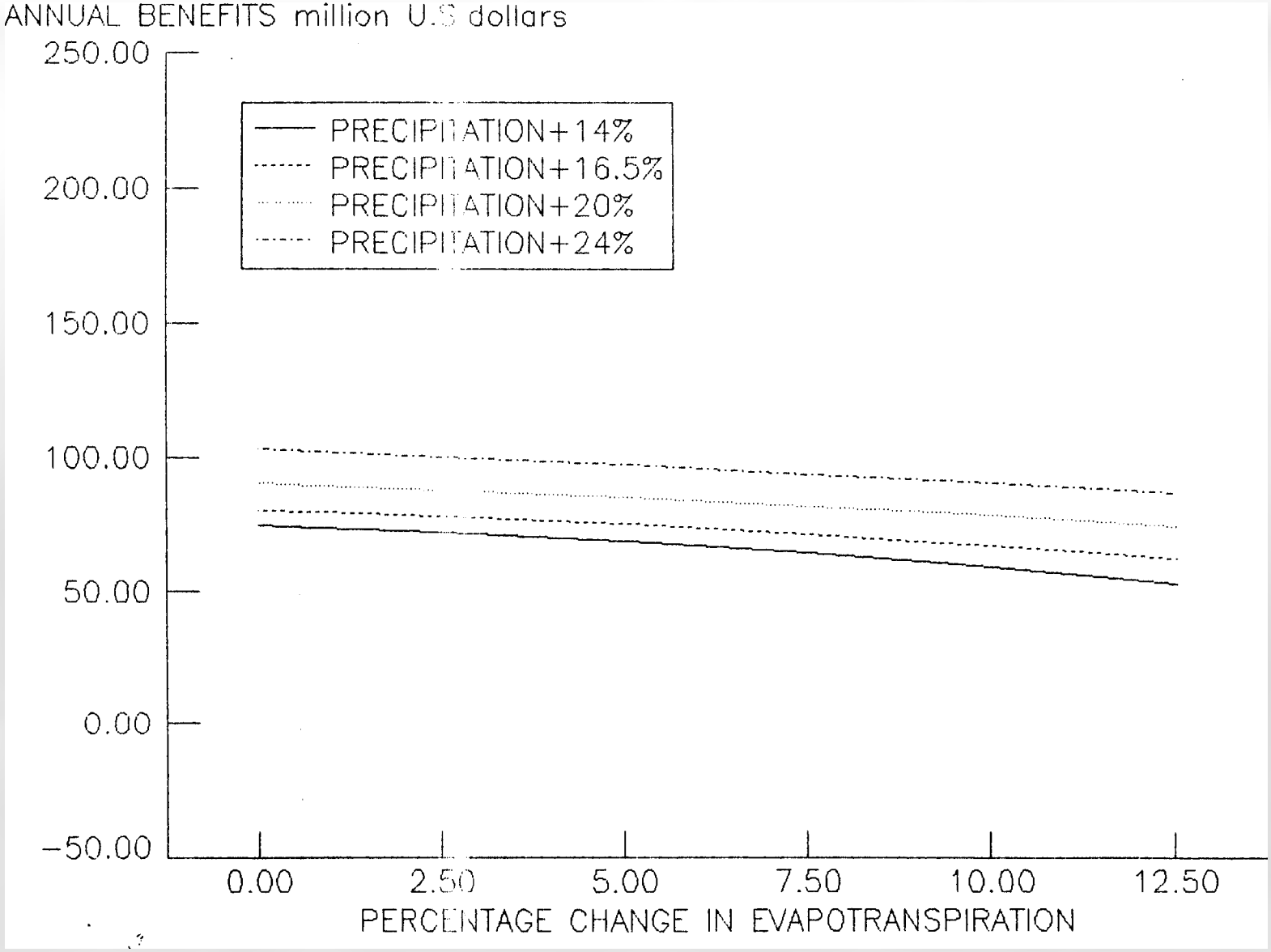


Fig. 11.3(e)

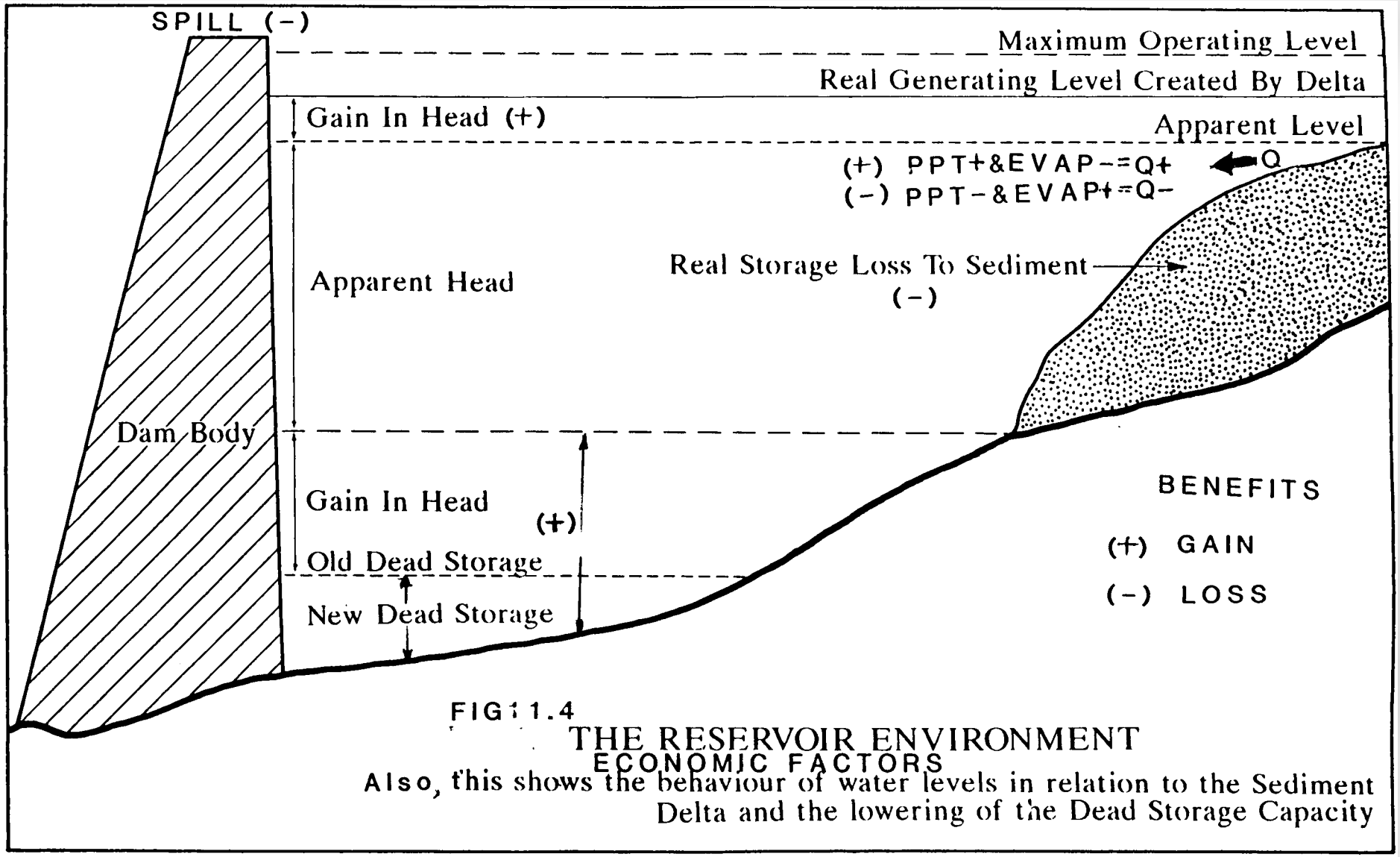


FIG 11.4

THE RESERVOIR ENVIRONMENT

ECONOMIC FACTORS

Also, this shows the behaviour of water levels in relation to the Sediment Delta and the lowering of the Dead Storage Capacity

11.7 Conclusions

From the above simulation results and their graphical representations, a trend is evident. Both alternative designs of the Gembu reservoir will be severely strained by expected stresses (changes) in global and local climates in the future. This trend is not unexpected, and thus implies that the River Basin Management System used in the analysis as well as the process and dynamically based computer simulation approach for assessing long term performance of reservoirs works satisfactorily in engineering design.

The situation will be more critical for the case 1 alternative if future precipitation increases are modest and mild (say 14%-16.5%) and if increases in potential evapotranspiration are severe (say 12.5% global). This critical situation is due to the unacceptable losses in benefits observed from simulation, a drop from 6.732 to the range 0.105 - 0.922 (table 11.1 (1)). Case 2 design appears to be a marginal improvement on case 1, as benefits are higher during the critical (mild) scenarios of climate change. In severe scenarios of warming, the case 2 design simulations gave slightly worse benefits than case 1. The design and management of the Gembu reservoir must take these critical conditions into consideration to avoid complete loss of benefits, should such conditions occur. The choice of the more optimal of these two defined cases depends on the exact scenario under which long term operation of the project will occur. The decision on the

exact scenario is left to designers and future researchers. It depends very much on policies government and inter-governmental organisations with regard to controlling the rate of accumulation of greenhouse gases in the atmosphere.

As an exercise for interested future researchers, it may be worthwhile to test more Gembu reservoir design options and then subject them to the same simulations scenerios as was done for the two design options above. This would assist in the determination of the most optimal design in a changing climatic environment.

For obtaining an optimal scheme, serious consideration must be given to watershed management and reservoir designs which will specifically reduce capacity losses to sedimentation and evaporation, the main culprits hampering long term reservoir performance. In general, there may be a need to reconsider a redesign of the facillities and structures of the proposed scheme to include

(1) sediment traps or sediment diversion structures

(2) sediment flushing facilities in the lower part of the dam body.

(2) the possible raising of the height of the dam to increase reservoir capacity.

(3) lowering the specified minimum dead storage levels to improve real time reservoir yields.

(4) measures for reducing reservoir evaporation by the use

of special oils or alcohols, evaporation reducing powder, or plastic boards/cover if this is practicable and affordable.

(5) foresting the windward sides of the reservoir to reduce evaporation losses.

(6) introduce far reaching recommendations on the need for improved land management practices in the Donga Basin

(7) For smaller reservoirs in the Donga Basin, frequent dredging could provide fertile materials for landfill and cultivation, if assessed and found to be economical.

CHAPTER 12

CHAPTER 12 : GENERAL CONCLUSIONS AND RECOMMENDATIONS

This study is multidisciplinary, cutting across professions, ranging from the individual areas of hydrology to economics. It forms an extension of a continuing research programme at Strathclyde University for assessing the hydrology and sediment processes in river basins and involves the addition of the concepts of real time reservoir sedimentation and economics to existing modelling technology. The specific areas covered by the study include an assessment of global and river basin meteorology and hydrology, landuse practices, sediment erosion and reservoir sedimentation and focusses on the economics of reservoir operations.

This thesis has been concerned mainly with the changing river basin processes and climatic conditions with a focus on the impact of these on reservoir management by the use of comprehensive and computer based techniques. One of the principal objectives of this research is to construct a practical water management system for the analysis of river basins and to assist in the realistic and optimal design and management of water storage schemes. The fundamentals involved in achieving this aim were that the mathematical models of the system; existing, modified, or newly developed, had to use parameters or data which are measurable or available. Thus, the models must be functional on data.

Conclusions have been expressed throughout the thesis in independent chapters. In this chapter, the whole study will draw together these conclusions and assess their

impact both practically and philosophically in the form of achievements and contributions, general conclusions, specific conclusions concerning the Donga Basin case study, and recommendations with suggestions for further research.

12.1 Study Achievements and Contributions

For the work outlined in this thesis, an advancement of design and management techniques in the water resources area has been made. This is mainly reflected by the provision of a practical physically based tool for assessing total catchment hydrology and sediment response, reservoir sedimentation and their combined economic impact. This tool is the River Basin Management System described in chapters 5 and 6.

Existing physically based theories have been utilised by previous researchers at Strathclyde and Stanford Universities to model and simulate the component hydrological and sediment processes which characterise the land, channel, and reservoir phases of river basins. Here, these processes have been linked with appropriate economic concepts by means of a new and comprehensive economic analysis model into a viable computer based management system of models. The system is capable of simulating the interaction between these processes, and their resultant economic response and impact.

The system itself consists of a watershed, erosion, modified system operation and routing, modified reservoir delta, and a newly developed economic simulation model. Modifications were made in all models to effect the interlinks. Thus, this total model system is a comprehensive tool which represents the complex and comprehensive interaction of many interplaying processes, both natural and man-induced, in a practical fashion. The input data requirement, though extensive, are simple and practical. Consequently, the system should be applicable under various physical environment and capable of predicting the effect of time related physical catchment changes on the hydrological and sediment regimes as well as the economics involved.

The system, having been extensively tested herein, works satisfactorily and its complete documentation is available. The documentation is felt to be of sufficient detail to enable a new user to operate the system.

12.2 Specific Conclusions for the Donga Basin Study

This research has used the Donga Basin as a case study because the catchment has great but untapped potentials and is on the verge of a huge and expensive development programme. This requires a long term assessment of the economic viability of proposals for hydroelectric power and irrigation. The system was tested in three main parts, and analysed for appropriate parameter responses and sensitivities. The tests involve assessing the short term

effect of changes in landuse practices and reservoir induced climate as well as the long term impact of global and impoundment related climate shifts. The scope of the study was conditioned to the asssessment of both quantitative results and relative effects, thus helping to confirm the reliability of the system utilised.

The three aspects analysed are

(1) catchment water balance and hydrograph response

(2) sediment yield response

(3) the operations and the economic performance of the Gembu reservoir project.

From the results of these tests involving various scenarios of change, this study has reached a major conclusion.

This main conclusion is that

the greenhouse effect coupled with impoundment related changes will have serious hydrological and hydraulic impact with resultant economic implications for the proposed developments in the Donga Basin, irrespective of the scenario of global warming. Huge floods ranging from severe to extreme will be the rule (commonplace) rather than the exception, from year to year.

Consequently, soil erosion, already a major problem in the catchment will be aggravated to disaster levels. The proposed storage schemes will face certain economic

disasters if current designs are carried out with benefit-cost ratios dropping from relatively high levels (6.372-7.300) for current conditions to unacceptable figures (0.922-0.105) for the future cases. An improved design tested by the writer raised minimum benefit cost ratios from 0.105 to 2.2.

A review of proposed designs is not only necessary but mandatory in the light of current knowledge provided by the results of this study. New designs which must be combined with the provision of adequate watershed management, sediment flushes and traps,etc will be required to combat the predicted floods and soil losses.

Watershed management may require improved vegetation cover and afforestation in appropriate places and the adequate control of the movements and numbers of rapidly increasing cattle population, especially around proposed reservoirs after construction. Inadequate management of cattle populations may aggravate the soil erosion losses even far more than predicted.

In addition, the impoundment related climate change will have a profound short term effect on the catchment water and sediment balance and the economic performance of the Gembu reservoir. The short term effect of realistic changes in landuse practices has a relatively less significant effect on the catchment floods, sediment yields or reservoir economics.

On application, each individual model within the system and the system as a whole has given good and realistic results. The results of the tests and the conclusions of the research has contributed knowledge to the organisations and 'parastatals' planning the gigantic reservoir projects in the basin (NEPA and URBDA). These conclusions are also likely to attract the interest of the Nigerian Federal Government and the government of Gongola State of Nigeria whose attention is now heavily directed to the implementation of their massive development proposals on the catchment. The implication is that proposed projects may now be reviewed in the light of future changes in catchment climate and landuse conditions as appropriate.

12.3 General Conclusions

The successful application of the proposed assessment approach and the computer system to a practical situation appraises and confirms the capability and usefulness of both the approach and the system. By using the computer system 'outwith' a pure research situation; experience and a greater understanding of the problems encountered on application of such techniques to a changing basin has been gained. Computer simulation of the responses of the river basin to landuse and climate changes has highlighted the dangers of using the traditional approach of dealing with a 'static' river basin in the design and management of projects. Economic

benefits are usually 'bloated' due to the use of unrealistic basin conditions by the traditional approach.

From the overall study, the complexity of the reservoir and river basin management problem vis-a-vis hydrological changes and sediment processes has led to a conclusion that the best approach to their solution is by comprehensive process modelling. Thus, this research is considered as one premier attempt in this direction. Thus, the modelling system established herein provides an excellent tool for studying hydrological and sediment processes vis-a-vis their economic impact in river basins.

This modelling approach, a promising attempt at basin management for short and long term scenarios, should be widely used by the professional community considering the immense potentials of the modelling system.

The model system is capable of comprehensively assessing the real time and long term

(a) water and sediment response of river basins

(b) reservoir operations and economics

and their response to the time variant changes in climate and landuses. More potentials include multi-purpose and multi-reservoir operations.

With a wider use of the modelling system, more knowledge of its limitations will be obtained and subsequent modifications and development can thereafter follow.

12.4 Recommendations

The movement of sediment on land surfaces and in reservoirs and channels is a complex process vis-a-vis the movement of water coupled to their interaction with economics. It is the writer's opinion that the assessment of these interactions lies in the construction of detailed knowledge of the processes, the interlinking of this knowledge by the use of computational techniques. The use of the process based system established in this thesis for basin management complies with this opinion and the results obtained provide a precedence for water resources managers.

It is evident from the analysis in this thesis that this white box approach which has been adopted, gives more realistic results and thus should be the basis of river basin management. For example, the consideration of the delta formation in the live storages of reservoirs has led to the need for the optimisation of the dead storage specifications which will considerably improve the economic returns from conventional dams. A major gain from this policy will be a substantial increase in the amount of energy obtained from conventional hydroelectric power plants. Thus, this rise in hydropower energy efficiency will help reduce the reliance on fossil fuels and nuclear plants, while awaiting further development in the use of alternative energy sources. A reduction in the rate of accumulation of greenhouse gases will result from a cut in the use of coal and oil for energy generation. The dangers

of the problem of the control of nuclear waste, unstable nuclear power stations, and radiation will be diminished if hydropower output rises.

As there are many optimisation models in the field of reservoir management which tend to lack the detail of catchment and reservoir processes, it is recommended herein that this approach and indeed the system be incorporated into existing optimisation models for long and real time operations and design. Alternatively, the optimisation of operations can be included in process based deterministic models, such as the model system utilised herein.

Further, it has been demonstrated in this study and elsewhere (UNEP/WMO/ICSU, 1987) that the use the traditional fixed river basin approach in management is open to serious error of judgement. The approach proposed, herein, which has been tested thoroughly can be adopted by the professional community.

It is also recommended that the complete model system requires further testing in different environments to enhance confidence. The watershed, erosion, and routing components have already been tested extensively with data from various catchments.

The River Basin Management System has one major limitation which should be adressed by future researchers. The reservoir delta simulation model requires a modification which will allow it to accept time series data of water

discharges and sediment loads as an alternative to the lump data option currently in use, prior to the tracing of the depositional path of individual sediment particles. The writer feels that this will give more realistic results of sediment deposits in the live storage zones of reservoirs, a major factor in reservoir performance.

Satellite remote sensing techniques can be linked with the erosion model of the System to provide data on areal vegetation cover, canopy stand, etc at regular time intervals by means of Landsat series (18 days) or SPOT series (10 days). This will improve the efficiency and accuracy of the model, and indeed the whole System.

Finally, further modification and testing of the System for broader use to include small reservoirs may be necessary as the economics involved may be different as compared to that of large reservoirs tested herein. The small capacities involved may make sedimentation a major hazard. Thus, regular dredging may be required to keep the reservoirs in operation. As dredging activities are expensive, benefit cost ratios and net benefits may be drastically lowered. Furthermore, to enhance benefits, the dredged materials can be used as agricultural landfills, brick making material, etc. This will necessitate additional modifications in the economic model to include these aspects, as required.

LIST OF REFERENCES

ACKOFF, R.L 1961
Progress in Operations Research, Vol 1
John Wiley, New, 1961

ACRES INTERNATIONAL LTD 1985
Ghana Generation Planning Study
Acres International Ltd, January 1985

ALBERTSON, M.L et al 1950
Diffusion of Submerged Jets :
Trans American Society of Civil Engineers., v 115, p. 639-664

AL KHADIMI, A 1982
Landuse, Water Yield and Soil Erosion Simulation of Cause and Effect
PHD Thesis, Department of Civil Engineering, University of Strathclyde Glasgow Scotland.

ALTS, 1978
Donga and Taraba Upper Basins, Hydroelectric Power Potential Pre-feasibility Study, Report for Upper Benue River Basin Development Authority, January, 1978.

ALTS, 1981
Feasibility Studies for The Lower Teraba Basin, Nigeria
Report for the Upper Benue River Basin Development Authority of Nigeria, April 1981

AMEIN, M and FANG, C.S 1969
Streamflow Routing with Application to North Carolina Rivers
University of North Carolina, Water Resources Institute
Report No 17, The University, Raleigh, N.C

AMEIN, M and FANG, C.S 1970
Implicit Flood Routing in Natural Channels
Proc American Society Civil Engineers, Journal of Hydraulic Division 96 (Hy 12), 2481-2500

ANANDALE 1987
Reservoir Sedimentation

ASCE 1970
Sedimentation Engineering Chapter IV, Sediment Sources and Sediment Yields
Task Commitee on the Preparation of Manual on Sedimentation
Proceedings ASCE, Journal of Hydraulics Division, 96, HYD PP 1283 - 1329. **(also ASCE ,1974: OPTIONAL WATER RESOURCES PLANNING).**

AUSTRALIAN WATER RESOURCES COUNCIL 1969
The Representative Basin Concept in Australia
Australia Dept of Natl Resources Hydrology Serial No 2
Australia Govt Pub, Canberra.

BATES, C.C 1953
Rational Theory of Delta Formation ;

American Association of Petroleum Geologists Bull., v. 37 No 9, P 2110-2162

BAGNOLD, R.A 1966
An Approach to the Sediment Transport Problem from General Physics
U.S.G.S Professional Paper 422-I

BBCTV October 3, 1988
The Flood Problem in Bangladesh

BEER, C.E; FARNHAM, C.W; HEINEMANN, H.G 1966
Evaluating Sedimentation- Prediction Techniques in Western Iowa
Trans America Society of Agricultural Engineers, 9,6, pp 828–833

BISWAS, A.K 1976
Systems Approach to Water Management
McGraw-Hill, Inc

BLACK, R.E 1973
Simulation of Flow Records for an Emphemeral Stream
MSc Thesis, Dept of Civil Eng, Univ of Strathclyde, Glasgow

BONHAM-CARTER, G and SUTHERLAND, A.J, 1968
Mathematical Model and Fortran IV Program for Computer Simulation of Deltaic Sedimentation.
Department of Geology, Stanford University

BURNASH, R.J.C., FERRAL, R.L and MCGUIRE, R.A 1973
A Generalised Streamflow Simulation System : Conceptual Modelling for Digital Computers. U.S Dep Commerce, National Weather Service in coop with Carlifornia Dept of Water Resources.

CHANG, H.H and HILL, J.C 1977
Minimum Stream Power for Rivers and Deltas
Journal of Hydraulic Division, American Society of Civil Engineers, Dec,1977

CLARKE, R.T 1973
A Review of Some Mathematical Models Used in Hydrology with Observations on their Calibration and Use.
Journal of Hydrology 19 (1), 1-20, 1973

COLE, J.A 1975
Assessment of Surface Water Sources
ICE Engineering Hydrology Today 73-85.

COOMES, R.T 1979
Regulation of Arkansas Basin Reservoir
Paper Presented at Proceedings on the National Workshop on Reservoir System's Operation pp 254-265, University of Colorado, Boulder, Colorado, Aug 13-17, 1979

CRAWFORD, N.H and LINSLEY, R.K 1966
Digital Simulation in Hydrology - Stanford Watershed model IV
Tech Rept No 39, Dept of Civil Eng, Stanford Univ, Carlifornia

CROWDER, B.M 1987
Economic Costs of Reservoir Sedimentation : A Regional Approach to Estimating Cropland Erosion Damages
Journal of Soil and Water Conservation, May-June, 1987

DEKKER, G 1961
Sediment Transport Measurement and Computations on the Niger
Section 11 of Inter-African Conference on Hydrology, Nairobi. 1961 256-265

DIYAM and BINIES, 1985 (or 1983)
Mambilla Hydroelectric Development Feasibility Report
Gembu Scheme

DIYAM and BINIES, 1985
Mambilla Hydroelectric Development Feasibility Report
Manya Scheme

DRAGOUN, F.J 1967
Rainfall Energy as Related to Sediment Yield
Journal of Geophysical Research, 1967, Vol 67, pt 4, April 1962

EICHERT, B.S and BONNER, V.R 1979
HEC Contribution to Reservoir System Operation
Technical Paper 63, Hydrological Engineering Centre
U.S Army Corps of Engineers, Davia, Carlifornia, August 1979.

EICHERT, B.S and DAVIS, D.W 1976
Sizing Flood Control Reservoir Systems by System Analysis
Technical Paper 44, Hydrologic Engineering Centre
U.S Army Corps of Engineers, Davis, Carlifornia, March 1976.

EMERY, D.A and MEEK, B.I 1960
The Simulation of a Complex River System in
Les Choix Economiques, Edited by P. Rosenstiel and A. Ghouila-Houri pp 237-255, Dunod, Paris, 1960.

EVANSON, D.E and MOSELEY, J.C 1970
Simulation/Optimisation Techniques for Multi-Basin Water Resource Planning
Water Resources Bulletin, 6(5), 725-736, 1970

FAHMY, M.A.N.E 1976
The Method of Dynamic Elements Applied to Watershed Sedimentation
PHD Thesis. Dept of Civil Engineering, Univ of Strathclyde, Glasgow

FAO-FOOD AND AGRICULTURAL ORGANISATION, 1974
The Sediment Problem by Fleming, G

FEDRICK, A.J and BEARD, R.L 1972
System Simulation of Integrated Use of Hydroelectric and Thermal Power Generation

Technical Paper 33, Hydrologic Engineering Centre, U.S Army Corps of Engineers, Davis, Carlifornia, Oct 1972.

FLEMING, G 1975
Computer Simulation Techniques in Hydrology

Elsivier Publishing Company

FLEMING, G 1969
Design Curves for Suspended Load Estimation
Proceedings Institution of Civil Engineers 43, 1-9

FLEMING, G 1969
The Clyde Basin : Hydrology and Sediment Transport
PHD Thesis, Department of Civil Engineering, The University of Strathclyde, Glasgow, 1969

FLEMING, G 1970
The Sediment Balance of the Clyde Estuary
Journal of the Hydraulics Division, American Society of Civil Engineers, Vol 96, No. HY 11, Proc. Paper 7676, Nov. 1970, pp 2219 - 2230

FLEMING, G 1970
Simulation of Streamflow in Scotland
Bulletin of the International Association of Scientific Hydrology XV 1, pp 53-59

FLEMING, G and FAHMY, M 1973
Some Mathematical Concepts for Simulating the Water Sediment Systems of Natural Watershed Areas
Report HO-73-26, Dept of Civil Engineering, the University of

Strathclyde, Glasgow, U.K. Sept, 1973

FLEMING, G 1974

Approaches to controlling Sediment Erosion in Rural Areas
Proceedings of the Symposium "Celebration of the Tercentary of Scientific Hydrology"
UNESCO/WHI/IAHS, Paris. September 9-12

FLEMING, G 1977
The Sediment Problem
Internal Publication, University of Strathclyde, Glasgow, Scotland

FLEMING, G 1979
Deterministic Models in Hydrology
Food and Agricultural Organisation of the United Nations

FLEMING, G and MCKENZIE, R.S 1982, 1983
River Basin Model for Water and Sediment Assessment, User Guide, Vol 1-The Watershed Model. Vol2- The Erosion and Routing Models.
Internal Publication No HY-82-7, Department of Civil Engineering, University of Strathclyde, Glasgow

FOURNIER, F 1960
Climat et Erosion - La relation contre l'erosion du Sol par L'eau etles Precipitations Atmospheriques. Presses Universitaires de France, Paris 201 p

FOURNIER, F 1954
La Parcelle Experimentale; Methode D'etude Experimentale de la

Conservation du Sol, de L'Erosion de Russelement

FREENY, G.B 1979
Managing Conflicts on the Lower Colorado River System
Paper Presented at the Proceedings of the National Workshop on Reservoir Systems Operations, University of Colorado, Boulder, Colorado. Aug 13-17, 1979.

GILL, M.A 1974
Hydrological Characteristics of the Sokoto-Rima Basin, Savanna, Vol 3, No 1, June 1974

GOODMAN, A.S 1984
Water Resources Planning
Prentice-Hall Inc, Englewood Cliffs, New Jersey 07632

GORDON, J.L 1978
Small Hydro Sets can Yield Competitive Energy
Energy International, Vol 14, Aug 1978

GORDON, J.L and PENMAN, A.C 1979
Quick Estimating Techniques for Small Hydro Potential
International Water Power and Dam Construction, Sept, 1979

GORDON, J.L 1983
Hydropower Cost Estimates
Water Power and Dam construction, Nov 1983

GULLIVER, J.S and GARVER, R.J 1981
A Comparison of Preliminary and Comprehensive Feasibility Estimates
Waterpower'81 An International Conference on Hydropower, US Army Corps of Engineers, Washington D.C June 22-25, 1981

HALCROW, W (Sir) and PARTNERS 1956
The Volta River Project Vol 111, Engineering Report to the Preparatory Commision.

HALL and DRACOUP, 1970
Water Resources Systems Engineering
McGraw-Hill, New York, 1970

HENDERSON, F.M 1966,63
Open Channel; Flow
The McMillan company, New York.

HYDROLOGIC ENGINEERING CENTRE 1971
HEC-3 Reservoir System Analysis

Technical Report, U.S Army Corps of Engineers, Davis, Carlifornia

HYDROLOGIC ENGINEERING CENTRE 1979
HEC-5 Reservoir System Operation for Flood Control and Conservation User Manual
Technical Report, U.S Army Corps of Engineers, Davis Carlifornia, June 1979

HOLY, M 1987
Expected Climatic Changes in the Atmosphere due to Greenhouse Effect
ICID Bulletin vol 36 No 2 July, 1987
Professor, Technical University of Prague and President Honoraire, ICID

HUDSON, N.W 1982
Soil Erosion on the Mambilla Plateau and Sedimentation into the Gembu Reservoir
National College of Agricultural Engineering

HUFSCHMIDT, M.M and FIERING, M.B 1966
Simulation Techniques for the Design of Water Resource Systems
Havard University Press, Cambridge, Massachussetts, U.S.A, 1966

HUFSCHMIDT, M.M 1966
Analysis by Simulation : Examination by Response Surface, in Design of Water Resource Systems
Havard University Press, Cambridge, Massachussetts 1966

HURST, H.E; BLACK, R.P; and SIMAIKE, Y.M 1965
Long-Term Storage: An Experimental Study
Constable, 145pp

HYDROCOMP INC 1969
Hydrocomp Simulation Program - Operations Manual
Palo Alto, Carlifornia.

HYDROCOMP INC 1976
The Derwent Catchment Study- A Report to the Rivers Division of the North West Water Authority in England
Internal Report, Glasgow.

INTERNATIONAL INSTITUTE FOR ENVIRONMENT AND DEVELOPMENT 1987
An Assessment of the Resource Base that Supports the Global Economy.
World Resources Institute

IRELAND, H.A; SHARP, C.F; and EAGLE, D.H 1939
Principles of Gully Erosion in the Piedmont of South Carolina.
U.S.D.A Technical Bulletin,633

IRI ITALSTAT 1983
Transaqua (Zaire-Chari-Chad Water Transfer) A Idea for the Sahel

IWE-INSTITUTION OF WATER ENGINEERS 1969
Manual of British Water Engineering Practice 4th Edition Vol 11 655pp

JAMES, L.D and LEE R.R 1971
Economics of Water Resources Planning
Tata McGraw-Hill Publishing Company Ltd, New Delhi.

JONES, I.J 1979
Hydro System Seasonal Regulation Program
Paper Presented at the Proceedings of the Workshop on Reservoir Systems Operations, University of Colorado, Boulder. Aug 13-17 1979.

KAISER, H.J COMPANY 1959
Reassessment Report on the Volta River Project for the Government of Ghana

KIRKBY, M.J 1978
Hillslope Hydrology
John Wiley and Sons

LAVENDER, S.T and ANDERSON, J.E 1984
The Recent African Drought Within the Context of Earlier African Hydrological Experience
Acres International Ltd, Niagara Falls, Canada Oct 1984.

LIDECO 1978
Mambilla Plateau Development Plan
Report for Federal ministry of Agriculture and Rural development, Nigeria. June 1978

LINSLEY R.K and FRAZINI 1979
Water Resources Engineering
McGraw-Hill Publishing Company

LIU, C.S et al 1972
Multipurpose Operation Studies of a Canal-Lake-River System-Oswego River System, New York.
Water Resources Research 8(2), 349-358, 1972.

LOUCKS, D.P and SIGVALDASON 1982
Multiple-Reservoir Operation in North America in " The Operation of Multiple Reservoir Systems edited by Kaczmarek, Z and Kindler, J
International Institute for Applied Systems Analysis, Luxemburg, Austria, 1982

MAHMOOD, K 1986
Reservoir Sedimentation, Impact, Extent, and Mitigation
World Bank and the Department of International Water Resources George Washington University

MAASS et al (Eds) 1962
Design of Water Resource Systems
Havard University Press, Cambridge, Massachussetts 1962

MAJOR, D.C and LENTON, R.L 1979
Water Resource Systems Planning
Prentice-Hall Inc, Englewood Cliffs, New Jersey 07632

MATHER, J.R 1984

Water Resources Distribution, Use and Management
John Wiley and Sons Inc and V.H Winston and Sons

MAZUNDER, 1983 " Irrigation Engineering", Tata-McGraw-Hill.

MCDONALD, J.E 1962

The Evaporation-Precipitation Fallacy
Weather, 17(5):1-9

MCKENZIE, R.S 1983
The Effects of Snow Processes and Landuse Changes On Hydrological Response
PHD Thesis, Civil Eng Dept, Univ of Strathclyde, Glasgow.

MCMAHON, T.A and MEIN, R.G 1978
Reservoir Capacity and Yield
Elsevier, 213pp

MERZ and MCLELLAN CONSULTING ENGINEERS 1986
Republic of Ghana, Tema Power Station, Appraisal of Rehabilitation Project, May 1986

MEYER, L.d and WEISMEIER, W.H 1969
Mathematical Simulation of the Process of Soil Erosion by Water
Trans. A.S.A.E., Vol 12, 6, pp 754 - 758

MORRICE, H.A.W and ALLAN, W.N 1959
Planning for the Ultimate Hydraulic Development of the Nile Valley.
Proceedings Institution of Civil Engineers 14, 101-156 (Discussion in 16, 288-317).

MRT CONSULTING ENGINEERS (NIGERIA) LTD, 1978
Donga River, Pre-Feasibility Study
Report for Upper Benue River Basin Development Authority, July, 1978

MUSTAFA, S 1981
Stochastic-Deterministic Model for Drought Assessment in Water Storage Design
PHD thesis, University of Strathclyde, Glasgow

MUTCHLER, C.K and HANSEN, L.H 1970
Splash of a Water Drop at Terminal velocity
Science, Vol 169, pp 1311-1312

MUTCHLER, C.K and YOUNG, R.A 1975
Soil Detachment by Raindrops
Present and Prospective Technology for Predicting Sediment Yields

and Sources.
U.S Dept of Agriculture Publication No ARS - S - 40, pp 113-117

MUTREJA, K.N 1986
Applied Hydrology

Tata-McGraw-Hill, New Delhi

MYERS, N 1985
The GAIA Atlas of Planet Management for Today's Caretakers of Tomorrow's World.
Pan Books

NACE, R.L 1969
Arrogance toward the Landscape- A problem in Water PLanning.
Bulletin Atomic Scientists 25, No 10 (1969) P 13

NACE, R.L 1974
Environmental Hazards of Large Scale Water Develpoments. A paper in the Book,"Priorities in Water Management" edited by Leversedge, F. M. Western Geography Series. Volume 8. Series Editor Foster, W.D

NEGEV, M 1967
A Sediment Model on a Digital Computer
Technical Report No 76, Dept of Civil Engineering, Stanford University, Carlifornia, U.S.A March 1967 pp109

NFMST-Nigerian Federal Ministry of Science and Technology, 1985
Proceedings of the National Workshop on Ecological Disasters, Kano, Nigeria, Dec 1985.

NWOGUTE, 1987
The Sediment Balance of the Donga Basin Unpublished.

PATTERSON, M.S 1984
Water Resource Planning and Development
Prentice-Hall, Inc, Englewood Cliffs, New Jersey 07632

PENMAN, H.L 1956
Evaporation from Lake Volta, Volta River Project- Volume 111, Engineering Report to the Preparatory Commision, Appendix 11

PIEST, R.F, BRADFORD, J.M and SPOMER, R.G 1972
Mechanism of Erosion and Sediment Movement from Gullies
Proceeding USDA Sediment Yield Workshop, Oxford, Mississippi, Nov 1972, Government Printing Press, Washington, D.C.

POWER, G 1988
Dam Siltation - The Silent Enemy
Water Power and Dam Construction, June 1988

SHAW, E.M 1988
Hydrology in Practice
VNR International

SHELTON, R.A 1979
Management of TVA Reservoir System
Paper Presented at the Proceedings on the National Workshop on Reservoir Systems Operations, Univ of Colo, Boulder Aug 13-17 1979

SILGAVALDSON, O.T 1976
A Simulation Model for Operating a Multipurpose MultiReservoir System.
Water Resources Research, 12(2), 263-278, 1976.

SILGAVALDASON, O.T et al 1975
The Trent River System : An Improved Operational Strategy
Paper Presented at the Proceedings of the Second World Congress on Water Resources, New Delhi, Dec 1975

SLEDGE, H.K 1979
Duke Power hydro System Reservoir Operation
Paper Presented at the Proceedings of the National Workshop on Reservoir Systems Operations, Univ of Colorado, Boulder, Aug 13-17 1979.

SMITH, S.E 1986
Drought and Water Management: The Egyptian Response
Journal of Soil and Water Conservation, Sept-Oct, 1986

SMITH, S.E and BEDFORD, K.E 1986
Effect of African Drought on the Water Resource Management in Egypt
in Water Forum '86, World Water Issues in Evolution, 1988

THAMBYAPILLAY, F.R 1985
Droughts and Desertification in Sahelo-Soudan Nigeria; The Climatic Parameter, Paper presented in the National Workshop on Ecological Disasters, 9-11 Dec, kano, Nigeria.

THOMPSON, J.R 1964
Quantitative Effect of Watershed Variables on the Rate of Gully Head Advancement
Trans. A.S.A.E, Vol 7, No 1, pp 54-55

TOEBES, G.H and CHANG, T.P
Simulation Model for the Upper Wabash Surface Water System
Tech Report 27 Dept of Civil Eng, Purdue university, Lafayette, Indiana July, 1972

UBRBDA, NIGERIA 1983 & 85
Five and Twenty Year Development Plan for Upper Benue Basin.

UMOLU, J.C and OKE, V.O 1986
Zaire-Chad-Niger Interbasin Water Transfer Scheme: A proposal for Subregional Water Resources Planning
Proceedings of the International Conference on Water Resources Needs and Planning in Drought Prone Areas organised by Sudan Engineering Society; International Association for Hydraulic Research; and UNESCO/UNDP

UNCNR-United Nations Commitee for National Resources, 1980
Proceedings of the Seventh Session of the Commitee on Natural Resources, Water Resources Series No 54

UN/ESCAP/WRS, 1985
UNCNR-United Nations Commitee for National Resources, 1985
Proceedings of the Tenth Session of the Commitee on Natural Resources, Water Resources Series No 59

UNEP/WMO/ICSU, 1985, 1986, 1987, 1988,1989, 1990 Conferences on the Global Climate change (Annual).

USDA 1966
Proceedures for Determining Rates of Land Drainage, Land Depreciation, and Volume of Sediment Produced by Gully Erosion
Soil Conservation Service, Technical Release No 32, Geology, July 1966

WRA-WATER RESEARCH ASSOCIATION, 1966
Proceedings of the Reservoir Yield Symposium, September 1965, 2 volumes

WALKER, R.A 1978

Regional Soil Erosion by Mathematical Modelling

PHD Thesis, Department of Civil Engineering, University of Strathclyde, Glasgow, Scotland.

WALKER, R.A and FLEMING, G 1979
The Strathclyde Sediment Model I - User Guide
Internal Publication No HHCD-79-10, Department of Civil Engineering, University of Strathclyde, Glasgow.

WILSON, E.M 1983
Engineering Hydrology
The McMillan Press Ltd

WISCHMEIR, W.H and SMITH, D.D 1965
Predicting Rainfall-Erosion Losses from the Cropland East of the Rocky Mountains
Agricultural Handbook 282, USDA, ARS 47 pp.

WISCHMEIR, W.H and SMITH, D.D 1960
A Universal Soil-Loss Equation to Guide Conservation Farm Planning
7th International Congress of Soil Science, Madison, Winsconsin 1960

WORLD METEOROLOGICAL ORGANISATION / NIGER BASIN AUTHORITY 1987
The Niger River Basin Model
NBA Meeting, Durbar Hotel Lagos Nigeria.

WORLD METEOROLOGICAL ORGANISATION 1977
Hydrological Model for the Upper Nile Basin, Vol. 2a, User Manuals for the Catchment, channel and Lakes Models.
Geneva

YEH, W.W.G 1985
Reservoir Management and Operation Models: A State of the Art Review
Water Resources Research Vol 21, No 12, p 1797-1818, 1985

MORE REFERENCES

DIGITAL RESEARCH, 1983
DR Graph graphic application user's

GRAF, W.H. 1971,1978
Hydraulics of Sediment Transport,
Mcgraw-Hill

FIERING, M.B. and JACKSON, B.B 1971
Synthetic Streamflows, American Geophsical Union,
Water Resources Monograph 1, Washington D.C

MUSGRAVE, G.W. 1947
The Quantitative Evaluation of Factors in Water Erosion,
A First Approximation. Journal of Soil and Water Conservation,
Vol 2,PP 133-138.

YEVJEVICH, V 1972
Probability and Statistics in Hydrology,
Water Resources Publications, Fort Collins, Colorado.

Stochastic Processes in Hydrology,
Water Resources Publications, Fort Collins, Colorado.

FOOD AND AGRICULTURAL ORGANISATION 1986
Irrigation Reports, FAO Publications, Rome.

SNOWY MOUNTAIN ENGINEERING COMPANY 1977
Upper Nile Catchment Model Reports, SMEC Publications
Australia.

WARDLAW, R.B 1978
The Development of a deterministic Integrated Surface/
Subsurface Hydrological Response Model, Ph.D Thesis,
Dept of Civil Engineering, University of Strathclyde
Glasgow, Scotland.

BURGES S.J. 1970,1979
Use of Stochastic Hydrology to determine Storage

Requirements for Reservoirs:A critical Analysis
Stanford University Report and Review

THOMAS, H.A and FIERING, M.B. 1963
The Nature of Storage Yield Function in " Operations
Research in Water Quality Management". Havard Water Program.

THOMAS, H.A and FIERING, M.B. 1962
Mathematical Synthesis of Streamflow sequences

for the analysis of River Basins by Simulation:
Chapter 12, Design of Water resource Systems edited
by Maass, A; Hufschmidt,R; Dorman, R; Thomas,H; Marglin,S;
Fair,G. Harvard University Press, Cambridge.

Other References cited but not stated herein are available in more recent references cited in the text and in the literature

MORE REFERENCES

CLARK, E.H. ; HAVERKAMP, J.A. ; CHAPMAN, W 1985
Eroding Soils:The off-farm Impacts, The Conservation Foundation, Washington. D.C

FIERING, M.B. and HUFSCHMIDT, M.M.F 1967
Simulation Techniques for the Design of Water Resource Systems. Mcmillan.

FIERING, M.B. 1967
Streamflow Synthesis
Mcmillan.

ASKEW, A.J. ; YEH, W.W.G. ; HALL, W.J 1971
Use of Monte Carlo Techniques in the Design and Operation of a Multipurpose Reservoir System, Water Resource Research ,6(4), 819-826, 1971.

APPENDIX

```
C*****************************************************************
C  SYSTEM OPERATION AND ROUTING MODEL
C  SECOND EDIT ON CHANIT TO ROUT1 DATED 8TH JUNE 1984
C  MODIFIED FURTHER IN 1988 BY V.O. OKE
C*****************************************************************
      REAL RESINFO(7,20,5)
      DIMENSION BKFLLQ(40),BKFLLA(40),
     &CR1(40),
     &CY1(40),CY2(40),CY3(40)
      DIMENSION SDAFLO(31,12),ACTQ(11),HRFLO(25,3),
     &          ADAFLO(31,12),BASIN(20)
      DIMENSION LAST(2,12),IPRINT(40),IQTIME(40)
      INTEGER ILASEND
      INTEGER BGNYR,BGNMON,
     &        ENDYR,ENDMON,
     &        YEAR,DAY,HOUR,DT,DTM,PRYR,PRMN,PRDY,DTSED,IRUL
      INTEGER NTRI
      REAL LOP,CP,Q,RY,GHEAD
      REAL VSED
      REAL MNCH(40),MNBK(40),LENGTH(40)
      INTEGER KRES
      COMMON /V/ RRATIO(7)
      COMMON /W/ SEDHD
      COMMON /T/ KRES
      COMMON /Q/ RAREA(7)
      COMMON /A1/ RS(40)
      COMMON /A/ CS(40),QIN(40),QSIDE(40),QOUT(40),QDIV(40,12,31),
     &        TOTDIV(40,12),ATRIB(3,40),DEPTH(3),CONFAC,ARE
     &        HDAM(6),SO(40),QINP(40),
     &VSTART(6),VSPILL(6),IRESN,
     &        MONTH,DAY,HOUR,MIN,DT,
     &        IADDTO(40),IRES(40),IR,NREACH,NSEGS,ISEDML,IDIV,ICRASH,IRUL
      COMMON /B/ CVELUP,VELMIN,
     &        C1LEN(6),C2LEN(6),
     &        TATRIB(40),QDEPTH(40),
     &        ITMSED,ICOUN1,DTSED,DEPLEN(40,6)
      COMMON /P1/ YDIVQ(7),TQSPILL(7),SPILPWR(7),SPILENG(7)
      COMMON /P/ TOTQD(7),TQEVAP(7),EVAPWR(7),EVAPENG(7)
      COMMON /P4/ LOP,CP,Q,YL,RY,ORF,PRFRA,PK,GHEAD,ECOST,PCOST,
     &TOTST,DMCM,GFRACT,KA,WC,T,FIIC,C,KC,FC,NC,FFC,
     &Y,MP,ZQ,FY,FMP,FCB,
     &DENS,FRACT,PHEADI,EFFI,PHEADG,EFFG,ALPHA,
     &TRACT,TCOST,CSPW,PSCF,CUD,DE,DA,PLEN,
     &DENST,CST,TC,
     &TLEN,LENC,CCHR,CCNR,TAXINS,ILASEND
      COMMON /C/ SMOFLO(12),AMOFLO(12),TYPE(40),
     &        IPRRCH,NRES,YEAR
      COMMON /D/ IPRINT,PRYR,PRMN,PRDY,NPR,NPRINT
      COMMON /E/ WID1(6),WID2(6)
      COMMON /F/ RESINFO(7,20,5),VOL,ELEV,QSPILL,AREAR,SDIVR
      COMMON /G/ LEAPYR,ISW,INCDT(6),INC
      COMMON /H/ SAREAR(7),DIVR(7)
      COMMON /K/ AQOUT(40)
      COMMON /J/ VSED
      COMMON /L/ DPM
C INITILISATION
      DATA CHAN,RES/4HCHAN,4H RES/
      DATA LAST/31,31,28,29,31,31,30,30,31,31,30,30,31,31,31,31,30,30,
     &          31,31,30,30,31,31/
      CALL INIZIA
      TOTQD(IR)=0.0
      YDIVQ(IR)=0.0
      TQSPILL(IR)=0.0
      TQEVAP(IR)=0.0
      DIVR(IR)=0.0
```

```
      SEDHD=(VSED/RAREA(NRES))/2660000
      IF(IDIV.EQ.0) GOTO 75
      DO 76 I=1,36
      READ(5,907) MO,ICN,ACTQ
      IFINAL=ICN*10
      IBASE=IFINAL-9
      IF(ICN.EQ.3) IFINAL=LAST(LEAPYR,MO)
      N=IBASE-1
      DO 77 IDA=IBASE,IFINAL
      TOTDIV(IR,MO)=TOTDIV(IR,MO)+ACTQ(IDA-N)
      QDIV(IR,MO,IDA)=ACTQ(IDA-N)
      TOTQD(IR)=TOTQD(IR)+ACTQ(IDA-N)
      TOTQD(IR)=TOTQD(IR)+QDIV(IR,MO,IDA)
      PRINT*,IR,TOTQD(IR)
 77   CONTINUE
 76   CONTINUE
      IF(TYPE(IR).EQ.RES) CALL SPOWER
 75   CONTINUE
      READ(5,901) HDAM(NRES),WID1(NRES),WID2(NRES),SO(IR)
      WRITE(6,24)
      WRITE(6,538)HDAM(NRES),WID1(NRES),WID2(NRES),SO(IR)
      READ(5,901) (ATRIB(IS,IR),IS=1,NSEGS)
      WRITE(6,25)
      WRITE(6,531)(ATRIB(IS,IR),IS=1,NSEGS)
      READ(5,902) IADDTO(IR)
      WRITE(6,26)
      WRITE(6,532)IADDTO(IR)
      READ(5,901) C1LEN(NRES),C2LEN(NRES),CVELUP,VELMIN
      WRITE(6,27)
      WRITE(6,541)C1LEN(NRES),C2LEN(NRES),CVELUP,VELMIN
      READ(5,901) VSTART(NRES),VSPILL(NRES)
       WRITE(6,901) VSTART(NRES),VSPILL(NRES)
      DO 81 I=1,20
      READ(5,901) (RESINFO(NRES,I,J),J=1,5)
      WRITE(6,901) (RESINFO(NRES,I,J),J=1,5)
 81   CONTINUE
      IF(NRES.EQ.KRES)THEN
      DO 881 I=20,1,-1
      IF(NRES.EQ.KRES)READ(21,901)(RESINFO(NRES,I,J),J=1,2)
      IF(NRES.EQ.KRES)WRITE(6,901) (RESINFO(NRES,I,J),J=1,5)
 881  CONTINUE
      ELSE
      CONTINUE
      ENDIF
      TATRIB(IR)=0.0
      DO 104 IS=1,NSEGS
      TATRIB(IR)=TATRIB(IR)+ATRIB(IS,IR)
      AREA(IS)=AREA(IS)+ATRIB(IS,IR)
 104  CONTINUE
      GO TO 100
 102  CONTINUE
C CHANNEL INPUT DATA
      READ(5,533) IDIV
      WRITE(6,70) IDIV
      IF(IDIV.EQ.0) GOTO 72
      DO 73 I=1,36
      READ(5,907) MO,ICN,ACTQ
      IFINAL=ICN*10
      IBASE=IFINAL-9
      IF(ICN.EQ.3) IFINAL=LAST(LEAPYR,MO)
      N=IBASE-1
      DO 74 IDA=IBASE,IFINAL
      TOTDIV(IR,MO)=TOTDIV(IR,MO)+ACTQ(IDA-N)
      QDIV(IR,MO,IDA)=ACTQ(IDA-N)
 74   CONTINUE
 73   CONTINUE
```

```
      CONTINUE
      READ(5,901) LENGTH(IR),SO(IR),MNCH(IR),MNBK(IR)
      WRITE(6,29)
      WRITE(6,530)LENGTH(IR),SO(IR),MNCH(IR),MNBK(IR)
      READ(5,901) (ATRIB(IS,IR),IS=1,NSEGS)
      WRITE(6,25)
      WRITE(6,531)(ATRIB(IS,IR),IS=1,NSEGS)
      READ(5,902) IADDTO(IR)
      WRITE(6,26)
      WRITE(6,532)IADDTO(IR)
      READ(5,901) CR1(IR),BKFLLA(IR)
      WRITE(6,32)
      WRITE(6,530)CR1(IR),BKFLLA(IR)
      READ(5,901) CY1(IR),CY2(IR),CY3(IR)
      WRITE(6,33)
      WRITE(6,530)CY1(IR),CY2(IR),CY3(IR)
C CALCULATION OF FLOW TIMES AT BANKFULL STAGE
      HRC=CR1(IR)*BKFLLA(IR)
      QTIME=LENGTH(IR)*MNCH(IR)/HRC**0.667/SQRT(SO(IR))
      IQTIME(IR)=QTIME/60.0
      BKFLLQ(IR)=BKFLLA(IR)*LENGTH(IR)/QTIME
      TATRIB(IR)=0.0
      DO 119 IS=1,NSEGS
      TATRIB(IR)=TATRIB(IR)+ATRIB(IS,IR)
      AREA(IS)=AREA(IS)+ATRIB(IS,IR)
119   CONTINUE
100   CONTINUE
      WRITE(6,34)
      WRITE(6,35)
      DO 123 IR=1,NREACH
      IF (TYPE(IR).EQ.CHAN) WRITE(6, 47) IR,IQTIME(IR),BKFLLQ(IR)
      IF (TYPE(IR).EQ.RES) WRITE(6,544) IR,IRES(IR)
123   CONTINUE
C TIME EQUALS START TIME
      YEAR=BGNYR
      MONTH=BGNMON
      DO 107 IR=1,NREACH
107   QIN(IR)=0.0
C INITIAL CONDITIONS
      READ(5,901) (HRFLO(25,IS),IS=1,NSEGS)
      WRITE(6,36)
      WRITE(6,511)(HRFLO(25,IS),IS=1,NSEGS)
      DO 103 IR=1,NREACH
      Q=0.0
      DO 121 IS=1,NSEGS
      DEPTH(IS)=HRFLO(25,IS)
      Q=Q+DEPTH(IS)*ATRIB(IS,IR)*CONFAC
121   CONTINUE
      Q=Q+QIN(IR)
      IF (TYPE(IR).EQ.CHAN) CS(IR)=(Q*Q*MNCH(IR)*MNCH(IR)/(CR1(IR)**1.33
     &                              *SO(IR)))**0.3*LENGTH(IR)
      IRV=IRES(IR)
      IRESN=IRV
      IF (TYPE(IR).EQ.RES) CS(IR)=VSTART(IRV)
C     THIS PROGRAM IS SET TO READ START VOLUMES
      ITRIB=IADDTO(IR)
      IF (ITRIB.GT.NREACH) GO TO 103
      QIN(ITRIB)=QIN(ITRIB)+Q
103   CONTINUE
      DO 109 IR=1,NREACH
      QOUT(IR)=0.0
      QSIDE(IR)=0.0
109   QIN(IR)=0.0
C READ SEDIMENT DATA
      ISEDML=0
C START YEARLY LOOP
```

```
 995 CONTINUE
 READ RECORDED DAILY FLOW
       DO 110 MO=1,12
       AMOFLO(MO)=0.0
       SMOFLO(MO)=0.0
       DO 110 IDA=1,31
       ADAFLO(IDA,MO)=0.0
       SDAFLO(IDA,MO)=0.0
 10    CONTINUE
       DO 111 I=1,36
       READ(5,907,END=99990)MO,ICN,ACTQ
       IFINAL=ICN*10
       IBASE=IFINAL-9
       IF (ICN.EQ.3) IFINAL=LAST(LEAPYR,MO)
       N=IBASE-1
       DO 111 IDA=IBASE,IFINAL
       AMOFLO(MO)=AMOFLO(MO)+ACTQ(IDA-N)
       ADAFLO(IDA,MO)=ACTQ(IDA-N)
 111   CONTINUE
 99990 CONTINUE
 C START MONTH LOOP
       ISTART = 1
       IF(MONTH.EQ.BGNMON)ISTART=IDA1
       LASTDY=LAST(LEAPYR,MONTH)
       IF(MONTH.EQ.ENDMON)LASTDY=IDA2
       DO 99991 DAY=ISTART,LASTDY
 C READ TIME SERIES DATA FOR CHANNEL SIMULATION
       DO 118 IS=1,NSEGS
       HRFLO(1,IS)=HRFLO(25,IS)
 108   CONTINUE
       IFL=IS+10
       READ(IFL,900) IYR,IMO,IDA,(HRFLO(IHR,IS),IHR=2,25)
       IF (IYR.NE.YEAR.OR.IMO.NE.MONTH.OR.IDA.NE.DAY) GO TO 108
 118   CONTINUE
 C START DAY LOOP
       DO 99992 HOUR=1,24
 C READ TIME SERIES DATA FOR SEDIMENT
 C CALCULATE TIME INCREMENT OF CHANNEL SIMULATION
       RMAX=0.0
       DO 115 IS=1,NSEGS
       R=(HRFLO(HOUR+1,IS)+HRFLO(HOUR,IS))/2.0
       IF (R.GT.RMAX) RMAX=R
 115   CONTINUE
       INC=NINCDT
       IF (RMAX.GT.0) INC=CINCDT/RMAX+1.0
       IF (INC.GT.NINCDT) INC=NINCDT
       IF (INC.LT.1) INC=1
       DTM=INCDT(INC)
       DT=DTM*60
 C START HOUR LOOP
       DO 99993 IMIN=1,60,DTM
       MIN=IMIN-1
       DO 122 IS=1,NSEGS
       DEPTH1=HRFLO(HOUR,IS)
       DEPTH2=HRFLO(HOUR+1,IS)
       DSEC=MIN*60
       DEPTH(IS)=(DSEC/3600.0)*(DEPTH2-DEPTH1)+DEPTH1
 122   CONTINUE
       DO 101 IR=1,NREACH
       IRESN=IRES(IR)
        IF (TYPE(IR).EQ.RES) CALL RESOIR
       IF (ICRASH.EQ.1) GOTO 80000
       IF (TYPE(IR).EQ.RES) GO TO 101
 C ***********************************************************************
 C          CHANNEL LEVEL EQUATION
       CS(IR)=CS(IR)+DT*(QIN(IR)+QSIDE(IR)-QOUT(IR)+QDIV(IR,MONTH,DAY))
```

```
      IF(ILASEND.LT.88) CS(IR)=CS(IR)+FRFRA*SDIVR
      IF (CS(IR).LE.0.0) WRITE(6,902)MONTH,DAY,HOUR,MIN,IR
      IF (CS(IR).LE.0.0) WRITE(6,901)CS(IR),QIN(IR),QSIDE(IR),QOUT(IR),
     &QDIV(IR,MONTH,DAY)
      IF (CS(IR).LT.0.1E-4) CS(IR)=0.1E-4
      IF (ICRASH.EQ.1) GOTO 80000
      QINP(IR)=QIN(IR)
      QIN(IR)=0.0
      QFP=0.0
C CHANNEL  AUXILIARIES AND RATES
      AC=CS(IR)/LENGTH(IR)
      YC=CY1(IR)*AC
      HRC=CR1(IR)*AC
      YFP=0.0
      IF (AC.LE.BKFLLA(IR)) GO TO 116
      YC=CY2(IR)+CY3(IR)*AC
      YFP=(YC-CY1(IR)*BKFLLA(IR))/2.0
      IF (YFP.LE.1.0E-4) YFP=1.0E-4
      AFP=AC-BKFLLA(IR)
      IF (AFP.LE.1.0E-4) AFP=1.0E-4
      IF(MNBK(IR).EQ.0.0)MNBK(IR)=1.0
      QFP=SQRT(SO(IR)*AFP*AFP*YFP**1.333/MNBK(IR)/MNBK(IR))
      AC=BKFLLA(IR)
      HRC=CR1(IR)*BKFLLA(IR)
116   CONTINUE
      YFP=YFP*2.0
      QC=SQRT(SO(IR)*AC*AC*HRC**1.333/MNCH(IR)/MNCH(IR))
      QOUT(IR)=QC+QFP
      QDEPTH(IR)=YC
      VELQ=QOUT(IR)*LENGTH(IR)/CS(IR)
      IF (AC.GE.0.99*BKFLLA(IR)) VELQ=BKFLLQ(IR)/BKFLLA(IR)
      QSIDE(IR)=0.0
      DO 120 IS=1,NSEGS
      QSIDE(IR)=DEPTH(IS)*ATRIB(IS,IR)*CONFAC+QSIDE(IR)
120   CONTINUE
      ITRIB=IADDTO(IR)
      IF (ITRIB.GT.NREACH) GO TO 106
      QIN(ITRIB)=QIN(ITRIB)+QOUT(IR)
106   CONTINUE
C PRINTING AND PLOTTING
      IF (IPRINT(IR).EQ.0) GO TO 101
      IF (YEAR.GT.PRYR) GO TO 105
      IF (YEAR.LT.PRYR) GO TO 101
      IF (MONTH.GT.PRMN) GO TO 105
      IF (MONTH.LT.PRMN) GO TO 101
      IF (DAY.GE.PRDY) GO TO 105
      GO TO 101
105   CONTINUE
      IF (NPR.GE.NPRINT) GO TO 101
      IF (NPR.GT.1) GOTO 700
C +++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
      WRITE(6,1000)
C     WRITE(6,701)
700   CONTINUE
      NPR=NPR+1
      WRITE(6,904) IR,YEAR,MONTH,DAY,HOUR,MIN,
     &CS(IR),QINP(IR),QSIDE(IR),QOUT(IR),VELQ,YC,YFP,QC,QFP,
     &QDIV(IR,MONTH,DAY)
101   CONTINUE
      SDAFLO(DAY,MONTH)=SDAFLO(DAY,MONTH)+QOUT(IPRRCH)*DT/86400.0
      SMOFLO(MONTH)=SMOFLO(MONTH)+QOUT(IPRRCH)*DT/86400.0
99993 CONTINUE
C END OF HOUR LOOP
99992 CONTINUE
C END OF DAY LOOP
```

```
99991 CONTINUE
C END OF MONTH LOOP
C CHECK CURRENT TIME AGAINST ENDTIME
      IF (MONTH.EQ.ENDMON.AND.YEAR.EQ.ENDYR) GO TO 99999
C ADVANCE THE TIME
      MONTH=MONTH+1
      IF (MONTH.EQ.10) GO TO 99999
      IF (MONTH.LE.12) GO TO 99990
      MONTH=1
      YEAR=YEAR+1
      GO TO 99990
99999 CONTINUE
      IF(IPLOT.EQ.0)GOTO 916
      WRITE(18,913)BASIN
      JYEAR=YEAR-1900
      WRITE(18,915)JYEAR,PLTMXQ
916   CONTINUE
      CALL DAILY(ADAFLO,SDAFLO,PLTMXQ,YEAR,1,IPLOT,
     &IDA1,IDA2,BGNMON,ENDMON)
      IF (MONTH.EQ.ENDMON.AND.YEAR.EQ.ENDYR) GO TO 99994
      IF (MONTH.EQ.10) GO TO 99995
99994 CONTINUE
      DO 777 IR=1,NREACH
      IRESN=IRES(IR)
C     IF(IRESN.GT.0)ICODE=IRESN
      IF(TYPE(IR).EQ.RES) CALL PIROUT
777   CONTINUE
1000  FORMAT(1H1)
C701  FORMAT(/2X,125HIR  YR   MO  DY  HR MIN   CHAN-STOR    INFL
C    &-FLOW  OUT-FLOW  VELOCITY   CHAN-Y      FP-Y      CHAN-Q
C    &   QDIV /)
900   FORMAT(I4,2I2,2X,2(12E10.4))
901   FORMAT(20X,6E10.4)
902   FORMAT(20X,12I5)
903   FORMAT(20X,40I1)
904   FORMAT(1X,I3,1X,5I4,2X,E10.4,6(1X,F9.3),3(1X,F9.2))
905   FORMAT(20X,10F6.4)
906   FORMAT(20X,6E10.4)
907   FORMAT(9X,I2,I1,11F6.2)
  500 FORMAT(34X,I5,9X,I5,9X,I5)
  502 FORMAT(33X,I5,10X,I5)
  503 FORMAT(35X,6E10.4)
  505 FORMAT(32X,I5)
  506 FORMAT(31X,12I5)
  507 FORMAT(35X,30I2)
  508 FORMAT(34X,I5,9X,I5,8X,I5)
  509 FORMAT(33X,I5,10X,I5)
  510 FORMAT(35X,E10.4,5X,5E10.4)
  530 FORMAT(35X,E10.4,5X,E10.4,5X,E10.4,4X,E10.4)
  531 FORMAT(35X,3(E10.4,5X))
  511 FORMAT(35X,3(E10.4,5X))
  526 FORMAT(32X,I5)
  513 FORMAT(32X,I5,11X,I5,11X,I5)
  515 FORMAT(35X,6E10.4)
  517 FORMAT(35X,10F6.4)
  061 FORMAT(1X,6HREACH ,I2,26X,6E10.4)
  060 FORMAT(1H0,6HREACH ,I2,26X,6E10.4)
  514 FORMAT(35X,6E10.4)
  541 FORMAT(35X,E10.4,5X,E10.4,5X,E10.4,4X,E10.4,3X,E10.4)
  544 FORMAT(1H0,34X,5HREACH,I4,16H IS RESERVOIR NO,I4)
  532 FORMAT(35X,12I5)
  525 FORMAT(35X,30I2)
533   FORMAT(20X,5I5)
   10 FORMAT(/1H0,19X,29HBASIC SIMULATION,INFORMATIONS)
   11 FORMAT(1H0,21HSIMULATION START TIME,13X,4HYEAR,11X,5HMONTH,11X,
     &3HDAY)
```

```
     1  FORMAT(1HO,22HSIMULATION BEGIN TIME,10X,4HYEAR,11X,5HMONTH,11X,
     &3HDAY)
    13 FORMAT(1HO,22HEXECUTE TIME INCREMENT,12X,5HNNCDT,10X,5HINCDT)
    14 FORMAT(1HO,19HINCREMENT PARAMETER,15X,6HCINCDT)
    15 FORMAT(1HO,17HCONVERSION FACTOR,17X,6HCONFAC)
    16 FORMAT(1HO,19HNO OF RIVER REACHES,15X,6HNREACH)
    17 FORMAT(1HO,19HNO OF LAND SEGMENTS,15X,4HNSEG)
    18 FORMAT(1HO,18HPRINT INSTRUCTIONS,16X,6HIPRINT)
    19 FORMAT(1HO,16HPRINT START TIME,18X,4HYEAR,11X,5HMONTH,9X,3HDAY)
    20 FORMAT(1HO,17HNO OF PRINT STEPS,17X,6HNPRINT,9X,7HIPREACH)
    21 FORMAT(1HO,16HPLOT INSTRUCTION,18X,4HFLOW,11X,8HSEDIMENT)
    22 FORMAT(1HO,49X,15HNETWORK SUMMARY)
    23 FORMAT(/1HO,5HREACH,I2,10HREACH TYPE,17X,A4/)
    29 FORMAT(1X,17HCHANNEL CONSTANTS,17X,9HLENGTH(M),6X,8HSLOPE(%),
     &7X,4HMNCH,10X,4HMNBK)
    25 FORMAT(1X,24HTRIBUTARY AREAS TO REACH,10X,11HSEG1(KM*KM),
     &4X,11HSEG2(KM*KM),4X,11HSEG3(KM*KM))
    26 FORMAT(1X,26HCURRENT REACH TRIBUTARY TO,8X,5HREACH)
   512 FORMAT(35X,E10.4,5X,E10.4,5X,E10.4,4X,E10.4,3X,E10.4)
    27 FORMAT(1X,22HRESERVOIR PARAMETER(1),12X,5HC1LEN,10X,5HC2LEN
     &,8X,6H VELUP,8X,6HVELMIN)
   538 FORMAT(35X,E10.4,5X,E10.4,5X,E10.4,3X,E10.4,
     &4X,E10.4)
    32 FORMAT(1X,26HHYDRAULIC RADIUS PARAMETER,8X,3HCR1
     &,11X,11HBKFLLA(M*M))
    33 FORMAT(1X,20HFLOW DEPTH PARAMETER,14X,3HCY1,12X,3HCY2,
     &12X,3HCY3)
    24 FORMAT(1X,15HREACH CONSTANTS,19X,7HHDAM(M),8X,7HWID1(M),
     &8X,7HWID2(M),7X,2HSO)
    28 FORMAT(1X,22HRESERVOIR PARAMETER(2),12X,6HC10CON,9X,6HC20LON
     &,9X,3HCYO,11X,3HEYO)
    36 FORMAT(1HO,21HINITIAL DEPTH OF FLOW,13X,4HSEG1,11X,4HSEG2,
     &11X,4HSEG3)
    37 FORMAT(1HO,28HINDEX OF SEDIMENT SIMULATION,6X,6HISEDML)
    47 FORMAT(1HO,34X,I4,11X,I4,5H MINS,4X,E10.3,7H CUMECS)
    34 FORMAT(1HO,48X,27HBANK FULL DISCHARGE SUMMARY)
    35 FORMAT(1HO,34X,8HREACH NO,7X,9HFLOW TIME,7X,14HBANK FULL FLOW)
 908   FORMAT(I4,3I2,9E10.4)
 909   FORMAT(1X,13E10.4)
 911   FORMAT(20X,A4)
 912   FORMAT(20X,2E12.4)
 913   FORMAT(20A4)
 914   FORMAT(5I5)
 915   FORMAT(3X,I2,F6.0)
 70    FORMAT(1HO,'DIVERSION INDEX (1=YES 0=NO)',I5)
 71    FORMAT(1HO,'DIVERSION INDEX (1=YES 0=NO)',I5)
 80000 CONTINUE
       IF (ICRASH.EQ.0) GOTO 80001
       WRITE(6,80)
 80    FORMAT(1HO,'DIVERSIONS EXCEED STORAGE IN REACH')
 80001 CONTINUE
       PRINT*,'AREAR',AREAR
       STOP
       END
       SUBROUTINE INIZIA
       COMMON /A1/ RS(40)
       COMMON /A/ CS(40),QIN(40),QSIDE(40),QOUT(40),QDIV(40,12,31),
      &        TOTDIV(40,12),ATRIB(3,40),DEPTH(3),CONFAC,AREA(3),
      &        HDAM(6),SO(40),QINP(40),
      &VSTART(6),VSPILL(6),IRESN,
      &        MONTH,DAY,HOUR,MIN,DT,
      &        IADDTO(40),IRES(40),IR,NREACH,NSEGS,ISEDML,IDIV,ICRASH,IRUL
       COMMON /B/ CVELUP,VELMIN,
      &        C1LEN(6),C2LEN(6),
      &        TATRIB(40),QDEPTH(40),
      &        ITMSED,ICOUN1,DTSED,DEPLEN(40,6)
```

```
      COMMON /E/ WID1(6),WID2(6)
      COMMON /F/ RESINFO(7,20,5),VOL,ELEV,QSPILL,AREAR,SDIVR
      REAL LANSED
      INTEGER DAY,HOUR,DT,DTSED
      INTEGER ILASEND
      DO 25 I=1,6
      DO 25 J=1,20
      DO 25 M=1,4
   25 RESINFO(I,J,M)=0.0
      DO 15 I=1,3
      AREA(I)=0.0

   15 CONTINUE
      DO 30 I=1,6
      VSTART(I)=0.0
      VSPILL(I)=0.0
      DO 30 J=1,40
      IRES(J)=0.0
      DEPLEN(J,I)=0.0
      DO 10 K=1,12
      TOTDIV(J,K)=0.0
      DO 10 L=1,31
      QDIV(J,K,L)=0.0
   10 CONTINUE
   30 CONTINUE
      END
      SUBROUTINE DAILY(ACTUAL,SIMVOL,PMAX,YEAR,
     &ITITLE,IPLOT,IDA1,IDA2,MO1,MO2)
      DIMENSION ACTUAL(31,12),SIMVOL(31,12),
     &LASTDA(2,12),MOCHWY(12),
     $YA(368),YS(368),P(11),LINE(103)
      INTEGER YEAR,START(12)
      INTEGER BL,DOT,DASH,CROS,R,S
      INTEGER ILASEND
      DATA MOCHWY/3HJAN,3HFEB,3HMAR,3HAPR,3HMAY,3HJUN,3HJUL,3HAUG,3HSEP,
     &3HOCT,3HNOV,3HDEC/
      DATA LASTDA/31,31,28,29,31,31,30,30,31,31,30,30,31,31,31,31,30,30,
     &31,31,30,30,31,31/
      DATA BL,DOT,DASH,CROS,R,S/1H ,1H.,1H-,1H+,1HR,1HS/
      LEAPYR=0
      IF((YEAR-4*(YEAR/4)).EQ.0)LEAPYR=1
      L=LEAPYR+1
      DELTA=PMAX*0.1
      IF(IPLOT.EQ.0)GOTO 705
      ICHAN=18
      IF(ITITLE.EQ.2)ICHAN=19
      DO 701 MO=10,12
      LAST = LASTDA(L,MO)
      DO 702 IDA=1,LAST
      WRITE(ICHAN,917)IDA,SIMVOL(IDA,MO),ACTUAL(IDA,MO)
  702 CONTINUE
  701 CONTINUE
      DO 704 MO=1,9
      LAST=LASTDA(L,MO)
      DO 704 IDA=1,LAST
      WRITE(ICHAN,917)IDA,SIMVOL(IDA,MO),ACTUAL(IDA,MO)
  704 CONTINUE
  705 IBASE = 0
      DO 801 MO= MO1,12
      START(MO) = IBASE
      ISTART = 1
      IF(MO.EQ.MO1)ISTART = IDA1
      LAST=LASTDA(L,MO)
      IF(MO.EQ.MO2)LAST = IDA2
      DO 841 IDA=ISTART,LAST
      IBASE = IBASE+1
```

```
      FA=(ACTUAL(IDA,MO)/PMAX)*10.0
      FS=(SIMVOL(IDA,MO)/PMAX)*10.0
      IF(FA.GT.10.0)FA=10.0
      IF(FS.GT.10.0)FS=10.0
      YA(IBASE)=FA
      YS(IBASE)=FS
  941 CONTINUE
      IF(MO.EQ.MO2)GOTO 804
  901 CONTINUE
      DO 803 MO = 1,MO2
      START(MO) = IBASE
      LAST = LASTDA(L,MO)
      IF(MO.EQ.MO2)LAST = IDA2
      DO 803 IDA = 1,LAST
      IBASE = IBASE + 1
      FA = (ACTUAL(IDA,MO)/PMAX)*10.0
      FS = (SIMVOL(IDA,MO)/PMAX)*10.0
      IF(FA.GT.10.0)FA=10.0
      IF(FS.GT.10.0)FS=10.0
      YA(IBASE) = FA
      YS(IBASE) = FS
  803 CONTINUE
  804 DO 802 I=1,11
  802 P(I)=(I-1)*DELTA
      WRITE(6,910)
      WRITE(6,916)YEAR
      WRITE(6,914)
      WRITE(6,911)P
      IBASE = 0
      DO 814 MO = MO1,12
      LAST = LASTDA(L,MO)
      IF(MO.EQ.MO2)LAST = IDA2
      DO 805 I=1,101
  805 LINE(I)=DASH
      DO 806 I=1,101,10
  806 LINE(I)=CROS
      JSTART = 1
      IF(MO.EQ.MO1)JSTART = IDA1
      DO 813 IDM=JSTART,LAST
      IDY=IBASE+IDM-JSTART+1
      IS=YS(IDY)*10
      IA=YA(IDY)*10
      LINE(IS+1)=S
      LINE(IA+1)=R
      LINE(102)=BL
      LINE(103)=BL
      IF(IS.EQ.IA)LINE(102)=R
      IF(IS.EQ.IA)LINE(103)=S
      MED = (LAST - JSTART)/2
      IF(IDM.EQ.MED)GOTO 807
      WRITE(6,912)LINE
      GOTO 808
  807 WRITE(6,913)MOCHWY(MO),LINE
  808 IF(IDM.EQ.JSTART)GOTO 809
      GOTO 812
  809 CONTINUE
      DO 810 I=1,101
  810 LINE(I)=BL
      DO 811 I=1,101,10
  811 LINE(I)=DOT
  812 LINE(IA+1)=BL
      LINE(IS+1)=BL
      IF((IS-10*(IS/10)).EQ.0)LINE(IS+1)=DOT
      IF((IA-10*(IA/10)).EQ.0)LINE(IA+1)=DOT
  813 CONTINUE
      IBASE=IBASE+LAST-JSTART+1
```

```
      IF(MO.EQ.MO2)GOTO 830
  814 CONTINUE
      DO 820 MO = 1,MO2
      LAST = LASTDA(L,MO)
      IF(MO.EQ.MO2)LAST = IDA2
      DO 821 I = 1,101
  821 LINE(I) = DASH
      DO 822 I = 1,101,10
  822 LINE(I) = CROS
      DO 829 IDM = 1,LAST
      IDY = IBASE + IDM
      IS = YS(IDY)*10
      IA = YA(IDY)*10
      LINE(IS+1)=S
      LINE(IA+1)=R
      LINE(102)=BL
      LINE(103)=BL
      IF(IS.EQ.IA)LINE(102)=R
      IF(IS.EQ.IA)LINE(103)=S
      MED = LAST/2
      IF(IDM.EQ.MED)GOTO 823
      WRITE(6,912)LINE
      GOTO 824
  823 WRITE(6,913)MOCHWY(MO),LINE
  824 IF(IDM.EQ.1)GOTO 825
      GOTO 826
  825 CONTINUE
      DO 827 I = 1,101
  827 LINE(I)=BL
      DO 828 I=1,101,10
  828 LINE(I)=DOT
  826 LINE(IA+1)=BL
      LINE(IS+1)=BL
      IF((IS-10*(IS/10)).EQ.0)LINE(IS+1)=DOT
      IF((IA-10*(IA/10)).EQ.0)LINE(IA+1)=DOT
  829 CONTINUE
      IBASE=IBASE+LAST
  820 CONTINUE
  830 CONTINUE
  815 WRITE(6,910)
C +++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
C SONO STATE ELIMINATE ALCUNE STAMPE INUTILI
C +++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
  910 FORMAT(1H1)
  911 FORMAT(1HO,4X,11F10.2)
  912 FORMAT(1H ,10X,103A1)
  913 FORMAT(1H ,5X,A3,2X,103A1)
  914 FORMAT(25X,43HRECORDED SIMULATED MEAN DAILY FLOW (CUMECS))
  916 FORMAT(50X,11HWATER YEAR ,I4)
  917 FORMAT(1H ,I5,2F20.1)
      RETURN
      END
      SUBROUTINE RESDIR
       DIMENSION EVAP(11),EVAPT(12,31),QEVAP(40),LAST(2,12)
      COMMON /A1/ RS(40)
      COMMON /V/ RRATIO(7)
      COMMON /A/ CS(40),QIN(40),QSIDE(40),QOUT(40),QDIV(40,12,31),
     &        TOTDIV(40,12),ATRIB(3,40),DEPTH(3),CONFAC,AREA(3),
     &        HDAM(6),SO(40),QINP(40),
     &VSTART(6),VSPILL(6),IRESN,
     &        MONTH,DAY,HOUR,MIN,DT,
     &        IADDTO(40),IRES(40),IR,NREACH,NSEGS,ISEDML,IDIV,ICRASH,IRUL
      COMMON /B/ CVELUP,VELMIN,
     &        C1LEN(6),C2LEN(6),
     &        TATRIB(40),QDEPTH(40),
     &        ITMSED,ICOUN1,DTSED,DEPLEN(40,6)
```

```
      COMMON /P1/ YDIVQ(7),TQSPILL(7),SPILPWR(7),SPILENG(7)
      COMMON /P/ TOTQD(7),TQEVAP(7),EVAPWR(7),EVAPENG(7)
      COMMON/P3/DEMAND(7),POWER(7),ANENG(7),ER(7),TPB(7),PC(7),OMC(7),
     &          TLTC(7),ENR(7),CBR(7),CRF(7),ANC(7),AAC(7),OMCPW(7),
     &          PWI(7),PWB(7),ZNAW(7),PW(7),ACBR(7),IDC(7),WI(7),
     &          HECT(7),WCA(7),CI(7),OMCI(7),TCI(7),WSC(7),TWSC(7),
     &          TNPC(7),MBA(7),WSBA(7),FBA(7),FCBA(7),TNPB(7),RPENR(7),
     &          ZNPCBR(7),ANCOST(7),ALCOST(7),QNPPWB(7),ZNPPWI(7),
     &          OMCIW(7),TLNPPW(7),AZNPCBR(7),PMPPI(7),PMPPG(7),RPW(7),
     &          CRPW(7),BPW(7),CBPW(7),PSCT(7),PPCI(7),PIPC(7),TNC(7),
     &          CLC(7),TPCPC(7),OMPCPC(7),TTPCPC(7),ANPCPC(7),WPCPC(7),
     &          OMPCPW(7),ALPCPC(7)
      COMMON /P4/ LOP,CP,Q,YL,RY,OMF,PRFRA,PK,GHEAD,ECOST,PCOST,
     &TOTST,DMCM,GFRACT,KA,WC,T,FIIC,C,KC,FC,NC,FFC,
     &Y,MP,ZQ,FY,FMP,FCB,
     &DENS,FRACT,PHEADI,EFFI,PHEADG,EFFG,ALPHA,
     &TRACT,TCOST,CSPW,PSCF,CUD,DE,DA,PLEN,
     &DENST,CST,TC,
     &TLEN,LENC,CCHR,CCNR,TAXINS,ILASEND
      COMMON /C/ SMOFLO(12),AMOFLO(12),TYPE(40),
     &          IPRRCH,NRES,YEAR
      COMMON /D/ IPRINT(40),PRYR,PRMN,PRDY,NPR,NPRINT
      COMMON /E/ WID1(6),WID2(6)
      COMMON /F/ RESINFO(7,20,5),VOL,ELEV,QSPILL,AREAR,SDIVR
      COMMON/G/ LEAPYR,ISW,INCDT(6),INC
      COMMON /H/ SAREAR(7),DIVR(7)
      COMMON /K/ AQOUT(40)
      COMMON /J/ VSED
      INTEGER DAY,HOUR,DT
      INTEGER DTSED
      INTEGER YEAR
      INTEGER PRYR,PRMN,PRDY
      INTEGER ILASEND
      DATA LAST/31,31,28,29,31,31,30,30,31,31,30,30,31,31,
     &          31,31,30,30,31,31,30,30,31,31/
      SDS=0.0
      RS(IR)=VSTART(IRESN)
      IF(ISW.GE.1)GO TO 500
      DO 60 I=1,36
      READ(5,55) MO,ICN,EVAP
55    FORMAT(9X,I2,I1,11F6.2)
      IFINAL=ICN*10
      IBASE=IFINAL-9
      IF(ICN.EQ.3)IFINAL=LAST(LEAPYR,MO)
      N=IBASE-1
      DO 57 IDA=IBASE,IFINAL
      EVAPT(MO,IDA)=EVAP(IDA-N)
      DEVNG=(EVAP(IDA-N)*0.001/(24*3600))*PRFRA*GHEAD
     & *9.81*0.836*1000
      WRITE(19,*) 'DAILY',IDA,DEVNG
57    CONTINUE
60    CONTINUE
500   CONTINUE
      TDVENG=TDVENG+DEVNG
      ISW=1
C     PRINT*,'AREAR',AREAR,'IR',IR,'SAREAR',SAREAR
      QEVAP(IR)=EVAPT(MONTH,DAY)*SAREAR(IR)*0.001/(24*DT*60/INCDT(INC))
      TQEVAP(IR)=TQEVAP(IR)+QEVAP(IR)
      TQEVAP(IR)=(TQEVAP(IR))*(60/INCDT(INC))*(1.0/8760.0)
C ***************************************************************
C RESERVOIR VOLUME EQUATION
      RS(IR)=RS(IR)+DT*(QIN(IR)+QSIDE(IR)-QOUT(IR)+QDIV(IR,MONTH,DAY))
     & -DT*QEVAP(IR)-DT*DIVR(IR)
C ***************************************************************
C     THE THREE LINES BELOW GIVE DEBUG OUTPUT IF RS(O
C     IF (RS(IR).LE.0.0) WRITE(6,900)MONTH,DAY,HOUR,MIN,IR
```

```
      IF (RS(IR).LE.0.0) WRITE(6,901)RS(IR),QIN(IR),QSIDE(IR),
     &QDIV(IR,MONTH,DAY)
      IF (RS(IR).LE.0.0) RS(IR)=0.0100E 01
      IF (ICRASH.EQ.1) GOTO 80000
C AUXILIARIES AND RATES
      VOL = RS(IR)
      CALL RESCALC
      IRV=IRES(IR)
      QOUT(IR)=QSPILL
      AQOUT(IR)=AQOUT(IR)+QOUT(IR)
      AQOUT(IR)=AQOUT(IR)/8760
C     PRINT*,'DISCHARGE FOR FLOODS AND NAVIGATION',AQOUT(IR)
      QSIDE(IR)=0.0
      DO 100 IS=1,NSEGS
      QSIDE(IR)=QSIDE(IR)+DEPTH(IS)*ATRIB(IS,IR)*CONFAC
100   CONTINUE
      ITRIB=IADDTO(IR)
      IF (ITRIB.GT.NREACH) GO TO 107
      QIN(ITRIB)=QIN(ITRIB)+QOUT(IR)
107   CONTINUE
101   CONTINUE
C PRINTING AND PLOTTING
      IF (IPRINT(IR).EQ.0) GO TO 108
      IF (YEAR.GT.PRYR) GO TO 109
      IF (YEAR.LT.PRYR) GO TO 108
      IF (MONTH.GT.PRMN) GO TO 109
      IF (MONTH.LT.PRMN) GO TO 108
      IF (DAY.GE.PRDY) GO TO 109
      GO TO 108
109   CONTINUE
      IF (NPR.GE.NPRINT) GOTO 108
      IF(NPR.GT.1) GOTO 702
      WRITE(6,703)
702   CONTINUE
      NPR=NPR+1
      WRITE(6,903)IR,YEAR,MONTH,DAY,HOUR,MIN,
     &RS(IR),QIN(IR),QOUT(IR),QSIDE(IR),QDIV(IR,MONTH,DAY),QEVAP(IR)
108   CONTINUE
      QIN(IR)=0.0
703   FORMAT(/4X,'IR   YR    MO   DY   HR MIN     RS (M3)       QIN      QSID
     &E      QOUT       QDIV       QEVAP   ')
900   FORMAT(20X,10I5)
901   FORMAT(20X,10E10.4)
902   FORMAT(20X,8E14.7)
903   FORMAT(1X,I3,1X,5I4,2X,E10.4,1X,F9.2,1X,F9.2,2(1X,F9.2),1X,F8.4)
80000 CONTINUE
      RETURN
      END
      SUBROUTINE RESCALC
      COMMON /V/ RRATIO(7)
      COMMON /T/ KRES
      COMMON /Q/ RAREA(7)
      COMMON /A1/ RS(40)
      COMMON /A/ CS(40),QIN(40),QSIDE(40),QOUT(40),QDIV(40,12,31),
     &TOTDIV(40,12),ATRIB(3,40),DEPTH(3),CONFAC,AREA(3),
     &HDAM(6),SO(40),QINP(40),
     &VSTART(6),VSPILL(6),IRESN,
     &MONTH,DAY,HOUR,MIN,DT,
     &IADDTO(40),IRES(40),IR,NREACH,NSEGS,ISEDML,IDIV,ICRASH,IRUL
      COMMON /P1/ YDIVQ(7),TQSPILL(7),SPILPWR(7),SPILENG(7)
      COMMON /P/ TOTQD(7),TQEVAP(7),EVAPWR(7),EVAPENG(7)
      COMMON/P3/DEMAND(7),POWER(7),ANENG(7),ER(7),TPB(7),PC(7),OMC(7),
     &          TLTC(7),ENR(7),CBR(7),CRF(7),ANC(7),AAC(7),OMCPW(7),
     &          PWI(7),PWB(7),ZNAW(7),PW(7),ACBR(7),IDC(7),WI(7),
     &          HECT(7),WCA(7),CI(7),OMCI(7),TCI(7),WSC(7),TWSC(7),
     &          TNPC(7),MBA(7),WSBA(7),FBA(7),FCBA(7),TNPB(7),RPENR(7),
```

```
     &              ZNPCBR(7),ANCOST(7),ALCOST(7),QNPPWB(7),ZNPPWI(7
     &              QMCIW(7),TLNPPW(7),AZNPCBR(7),PMPPI(7),PMPPG(7),
     &              CRPW(7),BPW(7),CBPW(7),PSCT(7),PPCI(7),PIPC(7),
     &              CLC(7),TPCPC(7),QMPCPC(7),TTPCPC(7),ANPCPC(7),WPCPC(7),
     &              QMPCPW(7),ALPCPC(7)
       COMMON /P4/ LOP,CP,Q,YL,RY,QMF,PRFRA,PK,GHEAD,ECOST,PCOST,
     &TOTST,DMCM,GFRACT,KA,WC,T,FIIC,C,KC,FC,NC,FFC,
     &Y,MP,ZQ,FY,FMP,FCB,
     &DENS,FRACT,PHEADI,EFFI,PHEADG,EFFG,ALPHA,
     &TRACT,TCOST,CSPW,PSCF,CUD,DE,DA,PLEN,
     &DENST,CST,TC,
     &TLEN,LENC,CCHR,CCNR,TAXINS,ILASEND
      COMMON /F/ RESINFO(7,20,5),VOL,ELEV,QSPILL,AREAR,SDIVR
      COMMON /G/ LEAPYR,ISW,INCDT(6),INC
      COMMON /H/ SAREAR(7),DIVR(7)
      COMMON /K/ AQOUT(40)
      COMMON /J/ VSED
      DIMENSION DIVINFO(7,20,5)
      INTEGER DAY,HOUR,DT,IRUL,IDIV
      INTEGER ILASEND
      DO 10 I=1,20
      IF(VOL.LT.RESINFO(IRESN,I,1)) GOTO 20
   10 CONTINUE
   20 CONTINUE
       RATIO=(VOL-RESINFO(IRESN,I-1,1))/(RESINFO(IRESN,I,1)
     &-RESINFO(IRESN,I-1,1))
C     THE FOLLOWING CALCULATES CURRENT VOL ELEV SPILL AREA
      ELEV=(RESINFO(IRESN,I,2)-RESINFO(IRESN,I-1,2))
     &*RATIO+RESINFO(IRESN,I-1,2)
      QSPILL=(RESINFO(IRESN,I,3)-RESINFO(IRESN,I-1,3))
     &*RATIO+RESINFO(IRESN,I-1,3)
      AREAR=(RESINFO(IRESN,I,4)-RESINFO(IRESN,I-1,4))
     &*RATIO+RESINFO(IRESN,I-1,4)
      SDIVR=(RESINFO(IRESN,I,1)-RESINFO(IRESN,I-1,1))
     &*(1.0/1.0E 07)*RATIO+RESINFO(IRESN,I-1,1)
     &*0.0*(1.0/1.0E 08)
      DIVR(IR)=SDIVR*5.0
      YDIVQ(IR)=YDIVQ(IR)+SDIVR*5.0
      TQSPILL(IR)=TQSPILL(IR)+QSPILL
      TQSPILL(IR)=(TQSPILL(IR)/8760)*(60/INCDT(INC))
      SAREAR(IR)=AREAR
      RETURN
      END
```

```
 SUBROUTINE SPOWER - ECONOMIC ANALYSIS MODEL-V OKE
 DIMENSION ADAFLO(31,12),BASIN(20)
 DIMENSION SDAFLO(31,12),ACTQ(11),HRFLO(25,3)
 DIMENSION LAST(2,12),IPRINT(40),INCDT(6),IQTIRE(40)
 INTEGER DAY,HOUR,DT,IDIV,ILASEND
 REAL LOP,CP,Q,RY,GHEAD
 REAL KA,FIIC,KC,NC,MP,LENC,ZQ
 REAL TOTST,MBA,MBAN
 COMMON /S/ NTRI
 COMMON /A1/ RS(40)
 COMMON /A/ CS(40),QIN(40),QSIDE(40),QOUT(40),QDIV(40,12,31),
&TOTDIV(40,12),ATRIB(3,40),DEPTH(3),CONFAC,
&AREA(3),HDAM(6),SO(40),QINP(40),
&VSTART(6),VSPILL(6),IRESN,
&MONTH,DAY,HOUR,MIN,DT,
&IADDTO(40),IRES(40),IR,NREACH,NSEGS,ISEDML,
&IDIV,ICRASH,IRUL
  COMMON /P1/ YDIVQ(7),TQSPILL(7),SPILPWR(7),SPILENG(7)
  COMMON /P/ TOTQD(7),TQEVAP(7),EVAPWR(7),EVAPENG(7)
  COMMON/P3/DEMAND(7),POWER(7),ANENG(7),ER(7),TPB(7),PC(7),OMC(7),
&           TLTC(7),ENR(7),CBR(7),CRF(7),ANC(7),AAC(7),OMCPW(7),
&           PWI(7),PWB(7),ZNAW(7),PW(7),ACBR(7),IDC(7),WI(7),
&           HECT(7),WCA(7),CI(7),OMCI(7),TCI(7),WSC(7),TWSC(7),
```

```
      &                 ZNPCBR(7),ANCOST(7),ALCOST(7),QNPPWB(7),ZNPPWI(7),
      &                 OMCIW(7),TLNPPW(7),AZNPCBR(7),PMPPI(7),PMPPG(7),RPW(7),
      &                 CRPW(7),BPW(7),CBPW(7),PSCT(7),PPCI(7),PIPC(7),TNC(7),
      &                 CLC(7),TPCPC(7),OMPCPC(7),TTPCPC(7),ANPCPC(7),WPCPC(7),
      &                 OMPCPW(7),ALPCPC(7)
        COMMON /P4/ LOP,CP,Q,YL,RY,OMF,PRFRA,PK,GHEAD,ECOST,PCOST,
      &TOTST,DMCM,GFRACT,KA,WC,T,FIIC,C,KC,FC,NC,FFC,
      &Y,MP,ZQ,FY,FMP,FCB,
      &DENS,FRACT,PHEADI,EFFI,PHEADG,EFFG,ALPHA,
      &TRACT,TCOST,CSPW,PSCF,CUD,DE,DA,PLEN,
      &DENST,CST,TC,
      &TLEN,LENC,CCHR,CCNR,TAXINS,ILASEND
          COMMON /K/ AQOUT(40)
          COMMON /L/ DPM
          REAL DPM
          DIMENSION OMPCPW(40),WPCPC(40)
C        PRINT*, 'DIVERSION'
C        PRINT*,IR, TOTQD(IR)
C
C
C
C POWER COST-BENEFIT
C
C
C
C PARARETERS CBR
C
C 00  continue
  WRITE(6,*) 'ENTERED SPOWER ROUTINE'
       READ(5,5000) LOP,CP,Q,OMF,YL,RY,PRFRA,PK,GHEAD,ECOST,
      &DPM,TOTST,DMCM,GFRACT,KA,WC,T,FIIC,C,KC,FC,NC,FFC,
      &Y,MP,ZQ,FY,FMP,FCB,ALCK,ZNB,RREF,WF,SF,TNL,TNNC,
      &DENS,FRACT,PHEADI,EFFI,PHEADG,EFFG,ALPHA,TRACT,TCOST,
      &CSPW,PSCF,CUD,DE,DA,PLEN,DENST,CST,TC,
      &TLEN,LENC,CCHR,CCNR,TAXINS,ILASEND,TRC,TRB,NTRI
       PRINT*,'LIFE OF PROJECT AND DISCOUNT RATE',LOP,R
 5000 FORMAT(3F6.3,F5.2,2F5.2,F4.2,F3.1,F6.2,3F8.3/F12.1,F3.1,F4.2,
      & F10.2,F8.2,F3.1,F3.0,F7.0,F4.1,F5.1,2F5.1,F3.1,F6.1/F6.3,
      &F7.0,F4.0,F5.0,F4.2,F6.0,3F4.2,F8.1,F6.1/F4.0,F4.2,F2.0,
      &F4.2,F4.1,2F4.2,F4.0,F6.0,F2.0,F5.2,
      &         F4.0,F2.0,f4.1,2F4.0,F3.0,F4.0/2f6.1,F2.0,f3.0,F4.2,I2
      &,F5.1,F6.0,I2)
       IF(IDIV.GE.2) GOTO 1007
 1007 WRITE(IRES(IR),*) LOP,CP,Q,OMF,YL,RY,PRFRA,PK,GHEAD,ECOST,PCOST,
      &DPM,TOTST,DMCM,GFRACT,KA,WC,T,FIIC,C,KC,FC,NC,FFC,
      &Y,MP,ZQ,FY,FMP,FCB,ALCK,ZNB,RREF,WF,SF,TNL,TNNC,
      &DENS,FRACT,PHEADI,EFFI,PHEADG,EFFG,ALPHA,TRACT,TCOST,
      &CSPW,PSCF,CUD,DE,DA,PLEN,DENST,CST,TC,
      &TLEN,LENC,CCHR,CCNR,TAXINS,ILASEND
       REWIND (IRES(IR))
        RETURN
        END
C
        SUBROUTINE PIROUT
        DIMENSION ADAFLO(31,12),BASIN(20)
        DIMENSION SDAFLO(31,12),ACTQ(11),HRFLO(25,3)
        DIMENSION LAST(2,12),IPRINT(40),IQTIRE(40)
        INTEGER DAY,HOUR,DT,IDIV,ILASEND
        REAL LOP,CP,Q,RY,GHEAD
        INTEGER NTRI
        REAL KA,FIIC,KC,NC,RP,LENC,ZQ
        REAL DPM
        COMMON /W/ SEDHD
        COMMON /V/ RRATIO(7)
        COMMON /S/ NTRI
        COMMON /A1/ RS(40)
        COMMON /A/ CS(40),QIN(40),QSIDE(40),QOUT(40),QDIV(40,12,31),
```

```
      &TOTDIV(40,12),AIRIB(3,40),DEPTH(3),CONFAC,
      &AREA(3),HDAM(6),SO(40),QINF(40),
      &VSTART(6),VSFILL(6),IRESN,MONTH,DAY,HOUR,MIN,DT,
      &IADDTO(40),IRES(40),IR,NREACH,NSEGS,ISEDML,
      &IDIV,ICRASH,IRUL
        COMMON /C/ SMOFLO(12),AMOFLO(12),TYPE(40),
      &     IFRMCH,NRES,YEAR
        COMMON /P1/ YDIVQ(7),TQSPILL(7),SPILPWR(7),SPILENG(7)
        COMMON /P/ TOTQD(7),TQEVAP(7),EVAPWR(7),EVAPENG(7)
        COMMON/P3/DEMAND(7),POWER(7),ANENG(7),ER(7),TPB(7),PC(7),OMC(7),
      &             TLTC(7),ENR(7),CBR(7),CRF(7),ANC(7),AAC(7),OMCPW(7),
      &             PWI(7),PWB(7),ZNAW(7),PW(7),ACBR(7),IDC(7),WI(7),
      &             HECT(7),WCA(7),CI(7),OMCI(7),TCI(7),WSC(7),TWSC(7),
      &             TNPC(7),MBA(7),WSBA(7),FBA(7),FCBA(7),TNPB(7),RPENR(7),
      &             ZNPCBR(7),ANCOST(7),ALCOST(7),QNPPWB(7),ZNPPWI(7),
      &             OMCIW(7),TLNPPW(7),AZNPCBR(7),PMPPI(7),PMPPG(7),RPW(7),
      &             CRPW(7),BPW(7),CBPW(7),PSCT(7),PPCI(7),PIPC(7),TNC(7),
      &             CLC(7),TPCPC(7),OMPCPC(7),TTPCPC(7),ANPCPC(7),WPCPC(7),
      &             OMPCPW(7),ALPCPC(7)
        COMMON /L/ DPM
        COMMON /T/ KRES
        COMMON /P4/ LOP,CP,Q,YL,RY,OMF,PRFRA,PK,GHEAD,ECOST,PCOST,
      &TOTST,DMCM,GFRACT,KA,WC,T,FIIC,C,KC,FC,NC,FFC,
      &Y,MP,ZQ,FY,FMP,FCB,
      &DENS,FRACT,PHEADI,EFFI,PHEADG,EFFG,ALPHA,
      &TRACT,TCOST,CSPW,PSCF,CUD,DE,DA,PLEN,
      &DENST,CST,TC,
      &TLEN,LENC,CCHR,CCNR,TAXINS,ILASEND
       COMMON /G/ LEAPYR,ISW,INCDT(6),INC
       COMMON /K/ AQOUT(40)
       COMMON /J/ VSED
       COMMON /Q/ RAREA(7)
       COMMON /F/ RESINFO(7,20,5),VOL,ELEV,QSPILL,AREAR,SDIVR
       DIMENSION PWIRG(7),PWWS(7),PWOTS(7),TPWPN(7)
       DIMENSION APB(7),AIRB(7),AWSB(7),AOTB(7),AFTNP(7)
       DIMENSION ARGC(7),AWSC(7),AOTC(7),ATNPC(7)
       DIMENSION AWIRG(7),AWWS(7),AWOTC(7),AWTNP(7)
       DIMENSION ARGCBR(7),AWSCBR(7),AOTCBR(7),ATNPCBR(7)
       DIMENSION TNER(7),TPWNP(7),RGCBR(7),WSCBR(7),OTCBR(7),TNPCBR(7)
       DIMENSION OTHB(7),OTHC(7)
       DIMENSION CKBR(7),CKIBR(7),CKSBR(7),CKOBR(7),CKNBR(7)
       DIMENSION TCIRG(7)
       REAL LATIO(1)
C
C      DIMENSION RGCBR(7),WSCBR(7),OTCBR(7),TNPCBR(7)
       DIMENSION MBAN(7),WSBAN(7),OTHBN(7),TNPBN(7)
       DIMENSION CIRG(7),WSCN(7),TNPCN(7)
       DIMENSION ERG(7),ERW(7),ORT(7),TNEP(7)
       DIMENSION OMCA(7),P(7),DEM(7),PCA(7),TLC(7),PWIA(7),OMCPA(7)
       DIMENSION CAP(7),OTHCN(7),PCT(7),PT(7)
        REAL TOTST,MBA,MBAN
        WRITE(*,*)'ENTER PIROUT'
       READ(IRES(IR),*)LOP,CP,Q,OMF,YL,RY,PRFRA,PK,GHEAD,ECOST,PCOST,
      &DPM,TOTST,DMCM,GFRACT,KA,WC,T,FIIC,C,KC,FC,NC,FFC,
      &Y,MP,ZQ,FY,FMP,FCB,ALCK,ZNB,RREF,WF,SF,TNL,TNNC,
      &DENS,FRACT,PHEADI,EFFI,PHEADG,EFFG,ALPHA,TRACT,TCOST,
      &CSPW,PSCF,CUD,DE,DA,PLEN,DENST,CST,TC,
      &TLEN,LENC,CCHR,CCNR,TAXINS,ILASEND
 5000 FORMAT(3F6.3,F5.2,2F5.2,2F3.1,I3,2F4.0/F12.1,2F3.1,F10.2,F8.2
      &         ,F3.1,F3.0,F7.0,F4.1,F5.0,2F5.0,F3.1,F6.1/F2.0,F7.0,F4.0,
      &F5.0,F4.2,F6.0/F4.0,F4.2,F2.0,f4.2,f4.1,2f4.2,F4.0,F6.0,F2.0,
      &         F4.0,F2.0,f4.1,2F4.0,F3.0,F4.0/2f6.1,F2.0,f3.0,F4.2,I2)
         IF(TYPE(IR).EQ.CHAN) GOTO 1001
        PRINT*,'SPECIAL RATIO PLEASE',RRATIO(1)
C       IF(1.EQ.KRES)THEN
C       YDIVQ(IR)=RRATIO(1)*YDIVQ(IR)
```

```
      ELSE
      CONTINUE
      ENDIF
      IF(YDIVQ(IR).EQ.0.0) YDIVQ(IR)=TOTQD(IR)
      PRINT*,'DIVERSION TEST PLEASE',YDIVQ(IR)
      WRITE(6,*) 'YDIVQ(IR)',YDIVQ(IR)
      YDIVQ(IR)=YDIVQ(IR)/(24*365)
      WRITE(6,*) IR,LOP,YL,RY,IDIV,TQEVAP(IR),TQSPILL(IR),
     &  TOTQD(IR),YDIVQ(IR)
      TOTQD(IR)=TOTQD(IR)/(365)
      WRITE(6,*) 'TOTAL ANNUAL DIVERSION'
      IF(TOTQD(IR).EQ.0.0) TOTQD(IR)=YDIVQ(IR)
      WRITE(6,*) TOTQD(IR)
      IF(IDIV.EQ.2) GOTO 1000
      IF(LOP.LT.1.OR.LOP.GT.100) LOP=50
      PRINT*,LOP,PRFRA,PK,GHEAD,CP,Q,YL,RY,PCOST,ECOST
      IF(PRFRA.GT.1.0) PRFRA=1.0
      WRITE(6,*) 'TQEVAP(IR)',TQEVAP(IR)
      TQEVAP(IR)=TQEVAP(IR)*8760
      GHEAD=GHEAD+SEDHD
      PRINT*,'NEW CALC HEAD',GHEAD
      EVAPWR(IR)=TQEVAP(IR)*PRFRA*GHEAD*9.81*0.836/1000
      WRITE(6,*) 'TQSPILL(IR)',TQSPILL(IR)
      TQSPILL(IR)=TQSPILL(IR)
      SPILPWR(IR)=TQSPILL(IR)*PRFRA*GHEAD*9.81*0.836/1000
      WRITE(6,*) 'YDIVQ',YDIVQ(IR)
      DEMAND(IR)=ABS(YDIVQ(IR))*PRFRA*PK*GHEAD*9.81*0.836/1000
      WRITE(6,4998) EVAPWR(IR)
 4998 FORMAT(20X,'EVAPOWER',10X,E12.4,'MWATTS')
      WRITE(6,4999) SPILPWR(IR)
 4999 FORMAT(20X,'SPILPWR',10X,E12.4,'MWATTS')
      WRITE(6,5001) DEMAND(IR)
 5001 FORMAT(20X,'DEMAND',10X,E12.4,'MWATTS')
      POWER(IR)=DEMAND(IR)
      WRITE(6,5002) POWER(IR)
 5002 FORMAT(20X,'POWER',10X,E12.4,'MWATTS')
      EVAPENG(IR)=EVAPWR(IR)*24*365.0/1000
      SPILENG(IR)=SPILPWR(IR)*24*365.0/1000
      ANENG(IR)=POWER(IR)*24.0*365.0/1000
      WRITE(6,4997) EVAPENG(IR)
 4997 FORMAT(20X,'EVAP ENERGY',10X,E12.4,'GWH')
      WRITE(6,5333) SPILENG(IR)
 5333 FORMAT(20X,'SPILENG',10X,E12.4,'GWH')
      WRITE(6,5003) ANENG(IR)
 5003 FORMAT(20X,'ANENG',10X,E12.4,'GWH')
      ANENG(IR)=ANENG(IR)*1000000
      ER(IR)=ANENG(IR)*ECOST/100
      WRITE(6,5004) ER(IR)
 5004 FORMAT(10X,'ER(IR)',10X,E12.4)
      TPB(IR)=(((1.0+YL)**(LOP+Q))-1.0)*ER(IR)/YL
      WRITE(6,5005) TPB(IR)
 5005 FORMAT(10X,'TPB(IR)',10X,E12.4)
      PCT(IR)=(((LOP+Q)*ANENG(IR)*PCOST)/100)+(TOTST*DMCM*PRFRA/100)
     & +TNL*TNNC
      PRINT*,'FIRST POWER COSTS',PCT(IR)
      WRITE(6,5006) PCT(IR)
      PC(IR)=PCT(IR)/CF
 5006 FORMAT(10X,'PCT(IR)',10X,E12.4)
      PRINT*,'ANNUAL POWER COST',PC(IR)
      P(IR)=(9600*(DPM*1000)**(0.82))*(GHEAD*3.4)**(-0.35)
      PT(IR)=SF*P(IR)+TNL*TNNC
      PCA(IR)=(SF*P(IR)+TNL*TNNC)/CP
      PRINT*,'ALTERNATIVE ANN POW COST',PCA(IR)
      OMCA(IR)=PCA(IR)*OMF
      OMC(IR)=PC(IR)*OMF
      WRITE(6,5007) OMC(IR)
```

```
5007  FORMAT(10X,'OMC(IR)',10X,E12.4)
      TLTC(IR)=CP*PC(IR)+OMC(IR)*(((1.0+YL)**(LOP+Q)-1.0))/YL
      WRITE(6,5008) TLTC(IR)
5008  FORMAT(10X,'TLTC(IR)',10X,E12.4)
      PRINT*,'LONG TERM COSTS',TLTC(IR)
      TLC(IR)=CP*PCA(IR)+OMCA(IR)*(((1.0+YL)**(LOP+Q)-1.0))
     &/YL
      PRINT*,'ALTERNAT LONG TERM COST',TLC(IR)
      TLTC(IR)=TLC(IR)
      ENR(IR)=TPB(IR)-TLC(IR)
      WRITE(6,5009) ENR(IR)
5009  FORMAT(10X,'ENR(IR)',10X,E12.4)
      CBR(IR)=(TPB(IR)/TLTC(IR))
      WRITE(6,5010) CBR(IR)
5010  FORMAT(10X,'CBR(IR)',10X,E12.4)
C     DO 333 RJ=RY,0.2,0.05
      CRF(IR)=(RY*(1.0+RY)**(LOP+CP))/((1.0+RY)**(LOP+CP)-1.0)
      WRITE(6,5011) CRF(IR)
5011  FORMAT(10X,'CRF(IR)',10X,E12.4)
5012  FORMAT(10X,'ANC(IR)',10X,E12.4)
      FACT=((1+RY)**(LOP+Q)-(1+YL)**(LOP+Q))/((RY-YL)*(1+RY)**(LOP+Q))
      OMCPW(IR)=OMC(IR)*FACT
      OMCPA(IR)=OMCA(IR)*FACT
      WRITE(6,5013) OMCPW(IR)
      OMCPW(IR)=OMCPA(IR)
C
      WRITE(6,5013)OMCPA(IR)
5013  FORMAT(10X,'OMCPW(IR)',10X,E12.4)
C
C
      PWB(IR)=ER(IR)*FACT*(1.0/(1.0+RY)**(CP-Q))
      FFACT=(1/(1+RY)**(CP-Q))
      FACT2=(((1.0+RY)**CP)-1.0)/(RY*(1.0+RY)**CP)
      PWI(IR)=OMCPW(IR)*FFACT+(PC(IR))*FACT2
      PRINT*,'PRESENT WORTH INV',PWI(IR)
      PWIA(IR)=OMCPW(IR)*FFACT+(PCA(IR)*FACT2)
      PRINT*,'ALTERNATIVE PRES WORTH OF INV',PWIA(IR)
      PWI(IR)=PWIA(IR)
      WRITE(10,5014) PWI(IR)
      ANC(IR)=CRF(IR)*PWI(IR)
      WRITE(6,5012) ANC(IR)
      PW(IR)=PWB(IR)-PWI(IR)
      WRITE(10,5064) PW(IR)
5064  FORMAT(10X,'PW',10X,E12.4)
      AAC(IR)=CRF(IR)*PWI(IR)
      WRITE(6,5062) AAC(IR)
5062  FORMAT(10X,'AAC',10X,E12.4)
5014  FORMAT(10X,'PWI(IR)',10X,E12.4)
      APB(IR)=CRF(IR)*PWB(IR)
      ZNAW(IR)=(APB(IR)-(ANC(IR)+AAC(IR))/2)
      WRITE(6,5063) ZNAW(IR)
5063  FORMAT(10X,'ZNAW',10X,E12.4)
      ACBR(IR)=PWB(IR)/PWI(IR)
      WRITE(6,5065) ACBR(IR)
5065  FORMAT(10X,'ACBR',10X,E12.4)
C
C
C
C
C IRRIGATION COSTS ANALYSIS
      IDC(IR)=(1.0-PRFRA)*TOTST*DMCM/100
      WRITE(6,5015) IDC(IR)
5015  FORMAT(10X,'IDC',10X,I12)
      WI(IR)=(1.0-PRFRA)*ABS(TOTQD(IR))*GFRACT*86400*RREF
      WRITE(6,5016) WI(IR)
5016  FORMAT(10X,'WI',10X,E12.4)
```

```
      HECT(IR)=WL*IR)/AA
      WRITE(6,5017) HECT(IR)
 5017 FORMAT(10X,'HECT',10X,E12.4)
      CI(IR)=FIIC*HECT(IR)*((1.0+(YL))**LOP)/(YL*(LOP+Q))
      WRITE(6,5018) CI(IR)
 5018 FORMAT(10X,'CI',10X,E12.4)
      OMCI(IR)=OMF*CI(IR)
      WRITE(6,5019) OMCI(IR)
 5019 FORMAT(10X,'OMCI',10X,E12.4)
      WCA(IR)=WC*HECT(IR)*T
      CIRG(IR)=WCA(IR)+CI(IR)+OMCI(IR)+IDC(IR)/CP
      TCIRG(IR)=WCA(IR)*(1+YL)**(LOP+Q)+CI(IR)+IDC(IR)
      FACT2=(((1.0+RY)**CP)-1.0)/(RY*(1.0+RY)**CP)
      TCI(IR)=(WCA(IR)+CI(IR)+OMCI(IR))*FACT1+(IDC(IR)*FACT2)/CP
C
C
      WRITE(6,5020) TCI(IR)
 5020 FORMAT(10X,'TCI',10X,E12.4)
      ARGC(IR)=CRF(IR)*TCI(IR)
C     KC=KC*(1.0+YL)**(LOP+Q)/YL
      CAP(IR)=((1.0-PRFRA)*(1.0-GFRACT)*WF*TOTQD(IR)/C)*10000000
      PRINT*,'CAPITAL-POPULATION',CAP(IR)
      WSCN(IR)=CAP(IR)*(KC*(1+YL)**(LOP+Q)/YL)+ALCK*(1-GFRACT)*IDC(IR)
      WSC(IR)=CAP(IR)*KC*FACT*FFACT+ALCK*(1-GFRACT)*IDC(IR)*FACT2/CP
      WRITE(6,5021) WSC(IR)
 5021 FORMAT(10X,'WSC',10X,E12.4)
      AWSC(IR)=CRF(IR)*WSC(IR)
      TWSC(IR)=TCI(IR)+WSC(IR)
      WRITE(6,5022) TWSC(IR)
 5022 FORMAT(10X,'TWSC',10X,E12.4)
      FACT1=((1+RY)**(LOP+Q)-(1+YL)**(LOP+Q))/((RY-YL)*(1+RY)**(LOP+Q))
      PRINT*,'DISCHGE PUR NAVGATY ET FLUD CNTRL',AQOUT(IR)
C     ENDIF
      IF(AQOUT(IR).LT.1.0) AQOUT(IR)=1.0
      PRINT*,ILASEND,'LLLLLLLLLLLLLLLLLLLLLLLLLLLLLL'
      IF(ILASEND.LT.88) AQOUT(IR)=AQOUT(IR)+YDIVQ(IR)+TOTQD(IR)
      PRINT*,'DISCHGE FOR NAVIGATION & FLOOD CNTRL',AQOUT(IR)
      TNPC(IR)=TCI(IR)+WSC(IR)+((0.000001*(FC+TRC)*TOTST)+
     & AQOUT(IR)*(FFC+NC))*FACT1
      WRITE(6,5023) TNPC(IR)
 5023 FORMAT(10X,'TNPC',10X,E12.4)
      OTHC(IR)=(0.000001*(FC+TRC)*TOTST+AQOUT(IR)*(FFC+NC))*FACT1*
     & FFACT+FACT2*(1-ALCK)*(1-GFRACT)*IDC(IR)/CP
      OTHCN(IR)=(0.000001*(FC+TRC)*TOTST+AQOUT(IR)*(FFC+NC))*
     & (LOP+Q)+(1-ALCK)*(1-GFRACT)*IDC(IR)
      AOTC(IR)=CRF(IR)*OTHC(IR)
      ATNPC(IR)=ARGC(IR)+AWSC(IR)+AOTC(IR)
      TNPCN(IR)=CIRG(IR)+WSCN(IR)+((OTHC(IR)*CRF(IR))*((1+YL)**(LOP+Q)
     &  -1)/YL)
C
C
C BENEFITS
      IF(ARGC(IR).EQ.0.0) ARGC(IR)=1.0
      MBA(IR)=HECT(IR)*Y*MP*FACT1*FFACT/100
      MBAN(IR)=HECT(IR)*Y*MP*(((1.0+YL)**LOP)/YL)/100
      WRITE(6,5024) MBA(IR)
 5024 FORMAT(10X,'MBA',10X,E10.4)
C     WSBAN(IR)=C*(1-PRFRA)*(1-GFRACT)*TOTQD(IR)*ZQ*
      WSBAN(IR)=CAP(IR)*ZQ*(((1.0+YL)**LOP)/YL)
C     WSBA(IR)=C*(1-PRFRA)*(1-GFRACT)*TOTQD(IR)*ZQ*FACT1
      WSBA(IR)=CAP(IR)*ZQ*FACT1*FFACT
C
      WRITE(6,5025) WSBA(IR)
 5025 FORMAT(10X,'WSBA',10X,E12.4)
      FYY=FY*.0000001*TOTST
      FBA(IR)=FYY*FMP*FACT1*FFACT+TRB*TOTST
```

```
      WRITE(6,5026) FBA(IR)
 5026 FORMAT(10X,'FBA',10X,E12.4)
      FCBA(IR)=(FCB+ZNB)*AQOUT(IR)*FACT1*FFACT
      WRITE(6,5027) FCBA(IR)
 5027 FORMAT(10X,'FCBA',10X,E12.4)
      TNPB(IR)=MBA(IR)+WSBA(IR)+FBA(IR)+FCBA(IR)
      WRITE(6,5028) TNPB(IR)
 5028 FORMAT(10X,'TNPB',10X,E12.4)
      OTHB(IR)=FCBA(IR)+FBA(IR)
      OTHBN(IR)=(OTHB(IR)/(FACT1*FFACT))*((1+RY)**(LOP+Q))/YL
      AOTB(IR)=OTHB(IR)*CRF(IR)
      AIRB(IR)=CRF(IR)*MBA(IR)
      AWSB(IR)=CRF(IR)*WSBA(IR)
      AFTNP(IR)=AIRB(IR)+AWSB(IR)+AOTB(IR)
      TNPBN(IR)=MBAN(IR)+WSBAN(IR)+OTHBN(IR)
      RPENR(IR)=TNPB(IR)-TNPC(IR)
      WRITE(6,5029) RPENR(IR)
 5029 FORMAT(10X,'RPENR',10X,E12.4)
      IF(TNPC(IR).EQ.0.0) TNPC(IR)=1.0
      WRITE(6,5030) ZNPCBR(IR)
 5030 FORMAT(10X,'ZNPCBR',10X,E12.4)
      ANCOST(IR)=((IDC(IR)/LOP)+CI(IR)*CRF(IR)+OMCI(IR)
     &           +TAXINS*((IDC(IR)/LOP)+CRF(IR)*CI(IR)))
      WRITE(6,5031) ANCOST(IR)
 5031 FORMAT(10X,'ANCOST',10X,E12.4)
      OMCIW(IR)=OMCI(IR)*FACT1
C
      WRITE(6,5032) OMCIW(IR)
 5032 FORMAT(10X,'OMCIW',10X,E12.4)
C
C
C
      ZNPPWI(IR)=TCI(IR)+WSC(IR)+OTHC(IR)
      IF(ZNPPWI(IR).EQ.0.0)ZNPPWI(IR)=1.0
      ZNPCBR(IR)=TNPB(IR)/ZNPPWI(IR)
      WRITE(6,5033) ZNPPWI(IR)
 5033 FORMAT(10X,'ZNPPWI',10X,E12.4)
C       ONPPWB(IR)=(TNPB(IR)/(LOP+Q))*FACT1
C       PRINT *,ONPPWB(IR)
        AWIRS(IR)=AIRS(IR)-(ARGC(IR)+ANCOST(IR))/2.0
        AWWS(IR)=AWSB(IR)-AWSC(IR)
        AWOTC(IR)=AOTB(IR)-AOTC(IR)
        AWTNP(IR)=AFTNP(IR)-ATNPC(IR)
C
         IF(TCI(IR).EQ.0.0) TCI(IR)=1.0
         ARGCBR(IR)=((MBA(IR)/TCI(IR))+(AIRB(IR)/ARGC(IR)))/2.0
         IF(WSC(IR).EQ.0.0) WSC(IR)=1.0
         IF(AWSC(IR).EQ.0.0) AWSC(IR)=1.0
         AWSCBR(IR)=((WSBA(IR)/WSC(IR))+(AWSB(IR)/AWSC(IR)))/2.0
         IF(OTHC(IR).EQ.0.0) OTHC(IR)=1.0
         IF(AOTC(IR).EQ.0.0) AOTC(IR)=1.0
         AOTCBR(IR)=((OTHB(IR)/OTHC(IR))+(AOTB(IR)/AOTC(IR)))/2.0
         IF(ATNPC(IR).EQ.0.0) ATNPC(IR)=1.0
         ATNPCBR(IR)=((AFTNP(IR)/ATNPC(IR)))
C
C PUMPING COSTS
      PMPPI(IR)=(DENS*(1-PRFRA)*TOTQD(IR)*PHEAD*EFFI)/1000.0
      WRITE(6,5034) PMPPI(IR)
 5034 FORMAT(10X,'PMPPI',10X,E12.4)
      PMPPG(IR)=(DENS*FRACT*TOTQD(IR)*PHEAD*EFFG)/1000.0
      WRITE(6,5035) PMPPG(IR)
 5035 FORMAT(10X,'PMPPG',10X,E12.4)
      RPW(IR)=ALPHA*(PMPPG(IR)+PMPPI(IR))
      WRITE(6,5036) RPW(IR)
 5036 FORMAT(10X,'RPW',10X,E12.4)
      CRPW(IR)=(RPW(IR)/TRACT)*TCOST
```

```
      WRITE(6,5037) CRPW(IR)
 5037 FORMAT(10X,'CRPW',10X,E12.4)
      BPW(IR)=PMPPG(IR)-RPW(IR)
      WRITE(6,5038) BPW(IR)
 5038 FORMAT(10X,'BPW',10X,E12.4)
      CBPW(IR)=BPW(IR)*CSPW
      WRITE(6,5039) CBPW(IR)
 5039 FORMAT(10X,'CBPW',10X,E12.4)
      PSCT(IR)=TOTQD(IR)*PSCF*CUD*3.0
      WRITE(6,5040) PSCT(IR)
 5040 FORMAT(10X,'PSCT',10X,E12.4)
      PPCI(IR)=PMPPI(IR)*CPW
      WRITE(6,5041) PPCI(IR)
 5041 FORMAT(10X,'PPCI',10X,E12.4)
      PIPC(IR)=((22.0/7.0)/4.0)*(DE**2.0-DA**2.0)*PLEN
     &          *DENST*CST
      WRITE(6,5042) PIPC(IR)
 5042 FORMAT(10X,'PIPC',10X,E12.4)
      TNC(IR)=TC*TLEN
      WRITE(6,5043) TNC(IR)
 5043 FORMAT(10X,'TNC',10X,E12.4)
      CLC(IR)=0.25*LENC*CCHR+LENC*CCNR
      WRITE(6,5044) CLC(IR)
 5044 FORMAT(10X,'CLC',10X,E12.4)
      TPCPC(IR)=CBPW(IR)+PSCT(IR)+PPCI(IR)+PIPC(IR)
     &            +CLC(IR)+TNC(IR)
      WRITE(6,5055) TPCPC(IR)
 5055 FORMAT(10X,'TPCPC',10X,E12.4)
      OMPCPC(IR)=OMF*TPCPC(IR)
      WRITE(6,5056) OMPCPC(IR)
 5056 FORMAT(10X,'OMPCPC',10X,E12.4)
      TTPCPC(IR)=OMPCPC(IR)+TPCPC(IR)
      WRITE(6,5057) TTPCPC(IR)
 5057 FORMAT(10X,'TTPCPC',10X,E12.4)
      ANPCPC(IR)=TPCPC(IR)*CRF(IR)+OMPCPC(IR)+TAXINS
     &            *TPCPC(IR)
      WRITE(6,5058) ANPCPC(IR)
 5058 FORMAT(10X,'ANPCPC',10X,E12.4)
      OMPCPW(IR)=OMPCPC(IR)*FACT1
C
      WRITE(6,5059) OMPCPW(IR)
 5059 FORMAT(10X,'OMPCPW',10X,E12.4)
      FACT3=1/((1+RY)**CP-1)/(RY*(1+RY)**CP)
      WPCPC(IR)=OMPCPW(IR)+TPCPC(IR)*FACT3
C
      WRITE(6,5060) WPCPC(IR)
 5060 FORMAT(10X,'WPCPC',10X,E12.4)
      ALPCPC(IR)=ZNPPWI(IR)*CRF(IR)
      WRITE(6,5061) ALPCPC(IR)
 5061 FORMAT(10X,'ALPCPC',10X,E12.4)
      ALCOST(IR)=CRF(IR)*ZNPPWI(IR)
      WRITE(6,5066) ALCOST(IR)
 5066 FORMAT(10X,'ALCOST',10X,E12.4)
      QNPPWB(IR)=(TNPB(IR)/LOP)*(1.0/((1.0+RY)**(CP-Q)))
      WRITE(6,5067) QNPPWB(IR)
 5067 FORMAT(10X,'QNPPWB',10X,E12.4)
      TLNPPW(IR)=QNPPWB(IR)-ZNPPWI(IR)
      WRITE(6,5068) TLNPPW(IR)
 5068 FORMAT(10X,'TLNPPW',10X,E12.4)
      AZNPCBR(IR)=QNPPWB(IR)/ZNPPWI(IR)
      WRITE(6,5069) AZNPCBR(IR)
 5069 FORMAT(10X,'AZNPCBR',10X,E12.4)
C     WRITE(6,*) 'SPOWER OUT'
C
C
C     SUMMARY
```

```
      FTC=TLTC(IR)+TNPCN(IR)+TPCPC(IR)
      FTCW=PWI(IR)+TNPC(IR)+TPCPC(IR)
      FTB=TPB(IR)+TNPB(IR)
      FTBW=PWB(IR)+TNPB(IR)
      FENR=FTB-FTC
      FFCBR=FTB/FTC
      FATB=CRF(IR)*FTBW
      FATC=CRF(IR)*FTCW
      FATW=FATB-FATC
      FAC=ANC(IR)+ANCOST(IR)+ANPCPC(IR)
      FPWI=PWI(IR)+ZNPPWI(IR)+WPCPC(IR)
      FPWB=(FTB/LOP)*KLCON*(1.0+RY)**(CP-Q))
      FTPW=FTBW-FTCW
      AFAC1=FPWI*CRF(IR)
      AFAC2=AAC(IR)+ALCOST(IR)+ANPCPC(IR)
      AFAC=(FAC+AFAC1+AFAC2)/3.0
      AFCBR=FPWB/FPWI
      FATB=FTB/(LOP+Q)
      AFACBR=FATB/AFAC
      PWIRG(IR)=MBA(IR)-TCI(IR)
      PWWS(IR)=WSBA(IR)-WSC(IR)
      PWOTS(IR)=OTHB(IR)-OTHC(IR)
      TPWNP(IR)=TNPB(IR)-ZNPPWI(IR)
      IF(CIRG(IR).EQ.0.0) CIRG(IR)=1.0
      RGCBR(IR)=MBAN(IR)/CIRG(IR)
      IF(WSCN(IR).EQ.0.0) WSCN(IR)=1.0
      WSCBR(IR)=WSBAN(IR)/WSCN(IR)
      IF(OTHC(IR).EQ.0.0)OTHC(IR)=1.0
      OTCBR(IR)=OTHBN(IR)/(OTHCN(IR))
      IF(TNPCN(IR).EQ.0.0)TNPCN(IR)=1.0
      TNPCBR(IR)=TNPBN(IR)/TNPCN(IR)
      ERG(IR)=MBAN(IR)-CIRG(IR)
      ERW(IR)=WSBAN(IR)-WSCN(IR)
      ORT(IR)=OTHBN(IR)-OTHC(IR)
      TNER(IR)=TNPBN(IR)-TNPCN(IR)
      IF(PWI(IR).EQ.0.0)PWI(IR)=1.0
      CKBR(IR)=PWB(IR)/PWI(IR)
      IF(TCI(IR).EQ.0.0)TCI(IR)=1.0
      CKIBR(IR)=MBA(IR)/TCI(IR)
      IF(WSC(IR).EQ.0.0)WSC(IR)=1.0
      CKSBR(IR)=WSBA(IR)/WSC(IR)
      CKOBR(IR)=OTHB(IR)/OTHC(IR)
      IF(ZNPPWI(IR).EQ.0.0) ZNPPWI(IR)=1.0
      CKNBR(IR)=TNPB(IR)/ZNPPWI(IR)
      CKFBR=FTBW/FTCW
      FTPW=FTBW-FTCW
      IF(IRESN.EQ.KRES)THEN
      DO 66 J=1,KRES
      WRITE(30,*)PWI(IR),TCI(IR),WSC(IR),OTHC(IR),ZNPPWI(IR),FTCW
      WRITE(31,*)PWB(IR),MBA(IR),WSBA(IR),OTHB(IR),TNPB(IR),FTBW
      WRITE(32,*)PW(IR),PWIRG(IR),PWWS(IR),PWOTS(IR),TPWNP(IR),FTPW
      WRITE(36,*)CKBR(IR),CKIBR(IR),CKSBR(IR),CKOBR(IR),CKNBR(IR),CKFBR
      WRITE(33,*)ANC(IR),ARGC(IR),AWSC(IR),AOTC(IR),ATNPC(IR),FATC
      WRITE(34,*)APB(IR),AIRB(IR),AWSB(IR),AOTB(IR),AFTNP(IR),FATB
      WRITE(35,*)ZNAW(IR),AWIRG(IR),AWWS(IR),AWOTC(IR),AWTNP(IR),FATW
      WRITE(37,*)ACBR(IR),ARGCBR(IR),AWSCBR(IR),AOTCBR(IR),ATNPCBR(IR)
     &,AFACBR
      WRITE(38,*)CBR(IR),RGCBR(IR),WSCBR(IR),OTCBR(IR),TNPCBR(IR),FFCBR(IR)
      WRITE(7,*)'                                        HYDROPOWER  EVAP LOSS
     &   SPILLAGES'
      WRITE(7,1)YDIVQ(IR),TQEVAP(IR),TQSPILL(IR)
1     FORMAT(1X,'DISCHARGE    CMS                  ',3(2X,E10.4))
      WRITE(7,2)POWER(IR),EVAPWR(IR),SPILPWR(IR)
2     FORMAT(1X,'MEAN POWER    MW                  ',3(2X,E10.4))
      WRITE(7,3)ANENG(IR),EVAPENG(IR),SPILENG(IR)
```

```
      WRITE(7,13)CAP(IR),HECT(IR)
   13 FORMAT(1X,'CAPITA',5X,E10.4,'      HECT',5X,E10.4,/////////////)
      WRITE(7,18)PCT(IR),PC(IR)
      WRITE(7,19)PT(IR),PCA(IR)
   18 FORMAT(1X,'FIRST COSTS',6X,2(4X,E10.4))
   19 FORMAT(1X,'ALTNT COSTS',6X,2(4X,E10.4),/////)
      WRITE(7,*)'                                    POWER    IRRIGATION
     & W.SUPPLY    OTHERS       TOT.NONPOWR    GR.TOTAL'
      WRITE(7,4)PWI(IR),TCI(IR),WSC(IR),OTHC(IR),ZNPPWI(IR)
     & ,FTCW
    4 FORMAT(1X,'PRESENT VALUES-COSTS       ',6(2X,E10.4))
      WRITE(7,5)PWB(IR),MBA(IR),WSBA(IR),OTHB(IR),TNPB(IR),FTBW
    5 FORMAT(1X,'PRESENT VALUES-BENFTS      ',6(2X,E10.4))
      WRITE(7,6)PW(IR),PWIRG(IR),PWWS(IR),PWOTS(IR),
     & TPWNP(IR),FTPW
    6 FORMAT(1X,'PRESENT WORTHS VALUES      ',6(2X,E10.4))
      WRITE(7,7)CKBR(IR),CKIBR(IR),CKSBR(IR),CKOBR(IR),CKNBR(IR)
     & ,CKFBR
    7 FORMAT(1X,'BENEFIT COST RATIO         ',6(2X,E10.4))
      WRITE(7,8)ANC(IR),ARGC(IR),AWSC(IR),AOTC(IR),ATNPC(IR),FATC
    8 FORMAT(1X,'ANNUAL COSTS VALUES        ',6(2X,E10.4))
      WRITE(7,9)APB(IR),AIRB(IR),AWSB(IR),AOTB(IR),AFTNP(IR),FATB
    9 FORMAT(1X,'ANNUAL BENEFIT VALUES      ',6(2X,E10.4))
      WRITE(7,10)ZNAW(IR),AWIRG(IR),AWWS(IR),AWOTC(IR),AWTNP(IR)
     & ,FATW
   10 FORMAT(1X,'ANNUAL WORTH VALUES        ',6(2X,E10.4))
      WRITE(7,11)ACBR(IR),ARGCBR(IR),AWSCBR(IR),AOTCBR(IR),ATNPCBR(IR)
     & ,AFACBR
      WRITE(7,14)TLTC(IR),TCIRG(IR),WSCN(IR),OTHCN(IR),TNPCN(IR),FTC
   14 FORMAT(1X,'UNDISCOUNTED COSTS         ',6(2X,E10.4))
      WRITE(7,15)TPB(IR),MBAN(IR),WSBAN(IR),OTHBN(IR),TNPBN(IR),FTB
   15 FORMAT(1X,'UNDISCOUNTED BENFTS        ',6(2X,E10.4))
   11 FORMAT(1X,'ANNUAL BEN.COST RATIO      ',6(2X,E10.4))
      WRITE(7,12)CBR(IR),RGCBR(IR),WSCBR(IR),OTCBR(IR),TNPCBR(IR)
     & ,FFCBR
   12 FORMAT(1X,'BEN.CST RATIO-UNDISCOUNTED',6(2X,E10.4),//////////)
   66  CONTINUE
      ELSE
      CONTINUE
      ENDIF
  200 FORMAT('1',10X,'POWER AND IRRIGATION COST-BENEFIT ANALYSIS',/,
     &10X,'LIFE OF PROJECT-LOP',43X,F7.3,/,
     &10X,'CONSTRUCTION PERIOD-CP',40X,F6.3,/,
     &10X,'PRODUCTION YEAR Q',48X,F6.3,/,
     &10X,'OPERATION &RTCE FACTOR - OMF',45X,F5.3,/,
     &10X,'FINANCIAL RATE -RY',49X,F3.1,/,
     &10X,'POWER EFFICIENCY FACTOR - PK',46X,F3.1,/,
     &10X,'RESERVOIR HEIGHT - GHEAD',44X,F8.3,/,
     &10X,'ENERGY COST(K/KWH) - ECOST',42X,F4.0,/,
     &10X,'PRODUCTION COST(K/KWH) - PCOST',38X,F4.0,/)
      WRITE(6,300)
     &       TOTST,DMCM,GFRACT,KA,WC,T,FIIC,CP,KC,FC,NC,FFC,Y,RP,Z0,FY,
     &       FMP,FCBR
C
C       POWER IRRIGATION OUTPUT
C
  300 FORMAT('1',30X,'POWER & IRRIGATION COST-BENEFIT ANALYSIS --INPUT
     &PARARETERS --',/,
     &10X,'TOTAL RESERVOIR STORAGE(CU.R) - TOTST',11X,F12.2,/,
     &10X,'DAM COST PER CU.R( /CU.R) - DMCM',16X,F4.1,/,
     &10X,'IRRI FRACTION OF DIVERSION - GFRACT',13X,F4.1,/,
     &10X,'ART OF WATER REQD. (CU.R) - KA',18X,F12.2,/,
     &10X,'WC-WATER CHARGE FOR IRRIGATION',19X,F10.2,/,
     &10X,'T-FRACTION OF YEAR FOR IRRIGATION',16X,F4.1,/,
     &10X,'FIIC-IRRIGATION INVESTMENT COSTS',18X,F7.2,/,
```

```
     &10X,'FC-FISH INVESTMENT COSTS',26X,F12.2,/,
     &10X,'NC-NAVIGATION INVESTMENT',26X,F7.2,/,
     &10X,'FFC-FLOOD CONTROL INVESTMENT',22X,F7.2,/,
     &10X,'Y-IRRIGATION YIELDS OR RETURNS',20X,F3.1,/,
     &10X,'MP-MARKET PRICE OF CROPS',27X,F6.2,/,
     &10X,'ZQ-WATER SUPPLY CHARGE PER CAPITA',18X,F4.2,/,
     &10X,'FY-FISH YIELD OR RETURN',29X,F7.0,/,
     &10X,'FMP-FISH MARKET PRICE IN DOLLARS',23X,F6.2,/,
     &10X,'FCBA-FLOOD CONTROL BENEFITS',16X,F7.2,/)
       PRINT *,OKE
301  FORMAT('1',10X,'(KG/CU.M) - DENS',15X,F6.2,/,
     &10X,'FRACT',15X,F1.2,/,
     &10X,'PHEADI',10X,F4.2,/,
     &10X,'EFFI',13X,F3.2,/,
     &10X,'PHEADG',22X,F4.2,/,
     &10X,'ALPHA',28X,F3.2,/,
     &10X,'TRACT',22X,F4.0,/,
     &10X,'TCOST',15X,F8.2,/,
     &10X,'CSPW',27X,F4.2,/,
     &10X,'PSCF',13X,F4.2,/,
     &10X,'CUD',19X,F4.0,/,
     &10X,'DE',17X,F2.0,/,
     &10X,'DA',17X,F3.1,/,
     &10X,'PLEN',26X,F6.2,/,
     &10X,'DENST',10X,F6.2,/,
     &10X,'CST',10X,F5.2,/,
     &10X,'TC',16X,F6.2,/)
302  FORMAT('1',10X,' -TLEN',27X,F8.2,/,
     &10X,'LENC',25X,F8.2,/,
     &10X,'CCHR',22X,F3.1,/,
     &10X,'CCNR',20X,F2.1,/)
       WRITE(6,301) DENS,FRACT,PHEADI,EFFI,PHEADG,ALPHA,TRACT,TCOST,
     &CSPW,PSCF,CUD,DE,DA,PLEN,DENST,CST,TC
       WRITE(6,302) TLEN,LENC,CCHR,CCNR
       WRITE(6,303) ER(IR),TPB(IR),PC(IR),OMC(IR),TLTC(IR),ENR(IR),
     &              CBR(IR),CRF(IR),ANC(IR),AAC(IR),OMCPW(IR),
     &                 PWI(IR),PWB(IR),ZNAW(IR),PW(IR),ACBR(IR)
       WRITE(6,304) IDC(IR),WI(IR),HECT(IR),WCA(IR),CI(IR),OMCI(IR),
     &              TCI(IR),WSC(IR),TWSC(IR),TNPC(IR),MBA(IR),WSBA(IR),
     &              FBA(IR),FCBA(IR),TNPB(IR),RPENR(IR),ZNPCBR(IR),
     &              ANCOST(IR),ALCOST(IR)
       WRITE(6,305) QNPPWB(IR),ZNPPWI(IR),OMCIW(IR),TLNPPW(IR),
     &           AZNPCBR(IR)
       WRITE(6,306) PMPPI(IR),PMPPG(IR),RPW(IR),CRPW(IR),BPW(IR),
     &             CBPW(IR),PSCT(IR),PPCI(IR),PIPC(IR),TNC(IR),
     &             CLC(IR),TPCPC(IR),OMPCPC(IR),TTPCPC(IR),ANPCPC(IR),
     &             ALPCPC(IR),WPCPC(IR),OMPCPW(IR)
       WRITE(6,307) FTC,FENR,FFCBR,CBRA,FAC,AFAC1,AFAC2,AFAC,FPWI,FPWB,
     &FPW,AFCBR
303  FORMAT('1',10X,'CALCULATED POWER-IRRIGATION COST-BENEFIT',/,
     &10X,'ER(IR)',26X,E12.6,/,
     &10X,'TPB(IR)',22X,E12.6,/,
     &10X,'PC(IR)',27X,E12.6,/,
     &10X,'OMC(IR)',15X,E12.6,/,
     &10X,'TLTC(IR)',20X,E12.6,/,
     &10X,'ENR(IR)',20X,E12.6,/,
     &10X,'CBR(IR)',22X,E12.6,/,
     &10X,'CRF(IR)',18X,F12.2,/,
     &10X,'ANC(IR)',11X,E12.6,/,
     &10X,'AAC(IR)',18X,E12.6,/,
     &10X,'OMCPW(IR)',17X,E12.6,/,
     &10X,'PWI(IR)',12X,E12.6,/,
     &10X,'PWB(IR)',17X,E12.6,/,
     &10X,'ZNAW(IR)',25X,E12.2,/,
```

```
     &10X,'PN(IR)',20X,E12.6,/,
     &10X,'ACBR(IR)',10X,F12.2,/)
 304  FORMAT('1',10X,'CALCULATED AND WRITTEN IRRIGATION OTHERS',/,
     &10X,'IDC(IR)',28X,I12,/,
     &10X,'WI(IR)',11X,E12.2,/,
     &10X,'HECT(IR)',52X,F12.2,/,
     &10X,'WCA(IR)',28X,E12.2,/,
     &10X,'CI(IR)',12X,E12.2,/,
     &10X,'OMCI(IR)',25X,E12.6,/,
     &10X,'TCI(IR)',19X,E12.6,/,
     &10X,'WSC(IR)',21X,E12.6,/,
     &10X,'TWSC(IR)',17X,E12.6,/,
     &10X,'TNPC(IR)',10X,E12.6,/,
     &10X,'MBA(IR)',11X,E10.4,/,
     &10X,'WSBA(IR)',19X,E12.6,/,
     &10X,'FBA(IR)',28X,E12.6,/,
     &10X,'FCBA(IR)',18X,E12.6,/,
     &10X,'TNPB(IR)',16X,E12.6,/,
     &10X,'RPENR(IR)',29X,E12.2,/,
     &10X,'ZNPCBR(IR)',17X,F12.2,/,
     &10X,'ANCOST(IR)',17X,E12.6,/,
     &10X,'ANCOSTA(IR)',12X,E12.6,/)
 305  FORMAT('1',10X,'NON POWER PRESENT WORTH OF BENEFITS -ONPPWB(IR)'27X,
     &F12.2,/,10X,'NON POWER PRESENT WORTH OF INVESTRENT - ZNPPWI(IR)',22X,
     &F12.2,/,10X,'NON POWER WORTH OF OPERATION AND RAINTENANCE COSTS -
     &OMCIW(IR)',11X,F12.2,/,
     &10X,'NONPOWER PRESENT WORTH - NET -TLNPPW(IR)',11X,E12.2,/,
     &10X,'ALTERNATIVE NON POWER COST BENEFIT RATIO - AZNPCBR(IR)',18X,
     &F12.2,/)
 306  FORMAT('1',10X,'PURPING COSTS - PARARETERS',/,
     &10X,'PMPPI',15X,E12.6,/,
     &10X,'PMPPG(IR)',26X,F12.2,/,
     &10X,'RPW(IR)',22X,F12.2,/,
     &10X,'CRPW(IR)',25X,F12.2,/,
     &10X,'BPW(IR)',16X,F12.2,/,
     &10X,'CBPW(IR)',29X,F12.2,/,
     &10X,'PSCF(IR)',21X,F12.2,/,
     &10X,'PPCI(IR)',21X,F12.2,/,
     &10X,'PIPC(IR)',10X,F12.2,/,
     &10X,'TNC(IR)',10X,F12.2,/,
     &10X,'CLC(IR)',11X,F12.2,/,
     &10X,'TPCPC(IR)',10X,F12.2,/,
     &10X,'OMPCPC(IR)',1X,F12.2,/,
     &10X,'TTPCPC(IR)',17X,F12.2,/,
     &10X,'ANPCPC(IR)',16X,F12.2,/,
     &10X,'ALPCPC(IR)',11X,F12.2,/,
     &10X,'WPCPC(IR)',15X,F12.2,/,
     &10X,'OMPCPW(IR)',18X,F12.2,/)
 307  FORMAT('1',10X,'SURMARY',/,10X,'FINAL TOTAL COST - FTC',22X,
     &E12.6,/,10X,'FINAL EXPECTED REVENUE - FENR',19X,E12.6,/,
     &10X,'FCBR',29X,F12.2,/,
     &10X,'CBRA',11X,F12.2,/,
     &10X,'FAC',22X,E12.6,/,
     &10X,'AFAC1',27X,E12.6,/,
     &10X,'AFAC2',27X,E12.6,/,
     &10X,'AFAC',10X,E12.6,/,
     &10X,'FPWI',27X,E12.6,/,
     &10X,'FPWB',29X,E12.6,/,
     &10X,'FPW',18X,E12.6,/,
     &10X,'AFCBR',21X,F12.2,/)

1001  CONTINUE
      WRITE(6,*)'PIROUT OUT'
      IF(FTPW.LE.0.0) WRITE(7,*)'RATE OF RETURN',RY
      IF(FTPW.LE.0.0) GO TO 334
333   CONTINUE
      RETURN
      END
```

```
Strathclyde VAX                 CMAV61          DISK07:[CMAV61.ES65]MODBAK.FOR;
Strathclyde VAX                 CMAV61          DISK07:[CMAV61.ES65]MODBAK.FOR;
Strathclyde VAX                 CMAV61          DISK07:[CMAV61.ES65]MODBAK.FOR;
Strathclyde VAX                 CMAV61          DISK07:[CMAV61.ES65]MODBAK.FOR;
Strathclyde VAX                 CMAV61          DISK07:[CMAV61.ES65]MODBAK.FOR;
Strathclyde VAX                 CMAV61          DISK07:[CMAV61.ES65]MODBAK.FOR;
Call duration: 00:05:37    Packets out: 0    Packets in: 1689
```