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COMMISSION FOR THE STUDY OF ALTERNATIVES TO THE PANAMA CANAL

THE STUDY OF ALTERNATIVES TO THE PANAMA CANAL

(COMPONENT STUDY : ENGINEERING AND COST ESTIMATES)

▼

PHASE 2

PRELIMINARY ENGINEERING AND COST ESTIMATES

FINAL REPORT

1/2

COMISION DE ESTUDIO DE LAS
ALTERNATIVAS AL CANAL DE PANAMA
CENTRO DE INFORMACION

EXECUTIVE SUMMARY

PART I : INTRODUCTION AND OBJECTIVES

PART II : GENERAL (Common Condition and Assumption)

APPENDIX

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YACHIYO ENGINEERING CO., LTD.
under the contract with
JAPAN INTERNATIONAL COOPERATION AGENCY

TABLE OF CONTENTS

Page

PREFACE

EXECUTIVE SUMMARY

PART I INTRODUCTION AND OBJECTIVES

I. Chapter 1 - Study Alternatives

1.1 General Description of the Alternatives..... I.1-1
1.2 High Rise Lock Canal..... I.1-1
1.3 Outline of the Sea Level Canal and Variations..... I.1-9

I. Chapter 2 - Phase 2 Study

2.1 Objectives..... I.2-1
2.2 Terms of Reference..... I.2-2
2.3 Study Schedule..... I.2-3

PART II GENERAL (Common Condition and Assumption)

PART II SECTION 1 ENGINEERING AND PRELIMINARY DESIGN

II.1 Chapter 1 - Basic Engineering Criteria

1.1 Design Ships..... II.1.1-1
1.2 Lock Chamber Dimensions..... II.1.1-2
1.3 Canal Cross Section Dimensions..... II.1.1-4
1.4 Canal Bank Slopes..... II.1.1-7
1.5 Number of Ship Transits and Lock Flushes.... II.1.1-17
1.6 Tide Levels and Level Datum..... II.1.1-21
1.7 Anchorages, Emergency Tie-Up Stations and Moorings..... II.1.1-23

II.1 Chapter 2 - Canal Alignment

2.1 Canal Alignments..... II.1.2-1

II.1 Chapter 3 - Seismology

3.1 Design Seismic Coefficient..... II.1.3-1
3.2 Earthquake in Panama..... II.1.3-3

COMISION DE ESTUDIO DE LAS
ALTERNATIVAS AL CANAL DE PANAMA
CENTRO DE INFORMACION

Page

II.1 Chapter 4 - Hydrology and Existing Water Supply

4.1 Rainfall..... II.1.4-1
4.2 Evaporation..... II.1.4-1
4.3 Watershed Yield..... II.1.4-1
4.4 Sediment Yield..... II.1.4-2
4.5 Design Flood..... II.1.4-2
4.6 Storage Capacity of Existing Dams..... II.1.4-3
4.7 Existing Water Use..... II.1.4-3
4.8 Monthly Discharge and Design Years..... II.1.4-4

II.1 Chapter 5 - Existing Population and Infrastructure

5.1 Population and Land Use..... II.1.5-1
5.2 Existing Road System..... II.1.5-6
5.3 Port Facilities..... II.1.5-11
5.4 Other Infrastructure..... II.1.5-12

PART II SECTION 2 CONSTRUCTION PLANS

II.2 Chapter 1 - General Construction Plan

1.1 Canal Excavation..... II.2.1-1
1.2 Aggregate for Concrete..... II.2.1-2
1.3 Arrangement of Construction Equipment and Materials..... II.2.1-2

II.2 Chapter 2 - Equipment for Excavation..... II.2.2-1

2.1 Dry Excavation..... II.2.2-1
2.2 Dredging..... II.2.2-13

II.2 Chapter 3 - Basic Conditions of Construction Plans

3.1 Working Days and Hours..... II.2.3-1
3.2 Load Factor, Unit Weight..... II.2.3-6
3.3 Dewatering System..... II.2.3-8
3.4 Placing Areas..... II.2.3-11
3.5 Drilling and Blasting of Guidelines..... II.2.3-16
3.6 Aggregate Production..... II.2.3-31
3.7 Concrete Production..... II.2.3-34

II.2 Chapter 4 - Fabrication of Gate Leaves

4.1 Introduction..... II.2.4-1
4.2 Material Source and Place of Fabrication.... II.2.4-1
4.3 Constructability..... II.2.4-4
4.4 Fabrication Procedure..... II.2.4-4
4.5 Transportation Procedures..... II.2.4-6
4.6 Possibility of Fabricating Gate Leaves in Panama and Immediate Region..... II.2.4-8
4.7 Work Schedule..... II.2.4-9

Page

PART II SECTION 3 COST ESTIMATES

II.3 Chapter 1 - Estimate Basis and Assumptions

1.1 Type of Contract..... II.3.1-1
1.2 Estimate Date and Currency..... II.3.1-1
1.3 Payment..... II.3.1-1
1.4 Taxes and Custom Duties..... II.3.1-1
1.5 Social Benefit and Overtime Premium..... II.3.1-1
1.6 Gate Leaves and Related Items..... II.3.1-2
1.7 General Conditions..... II.3.1-2
1.8 Supporting Construction..... II.3.1-3
1.9 Indirect Costs..... II.3.1-4

II.3 Chapter 2 - Unit Price

2.1 Materials..... II.3.2-1
2.2 Manpower..... II.3.2-4
2.3 Equipment..... II.3.2-5

II.3 Chapter 3 - Operation and Maintenance Costs

3.1 Introduction..... II.3.3-1
3.2 Previous Studies..... II.3.3-1
3.3 O & M Costs for Existing Canal
- PCC Annual Reports..... II.3.3-2
3.4 Phase One Reports
- Operation and Maintenance Costs..... II.3.3-9
3.5 High Rise Lock Canal
- O & M Cost Factors..... II.3.3-12
3.6 Sea Level Canal
- O & M Cost Factors..... II.3.3-16

COMISION DE ESTUDIO ... ALTERNATIVAS AL CANAL DE PANAMA
CENTRO DE INFORMACION

Page

APPENDIX

Appendix 1 - Terms of Reference

1. Introduction..... App.1-1
2. Objectives..... App.1-2
3. Scope of Services..... App.1-2
3.1 Engineering..... App.1-2
3.2 Preliminary Design..... App.1-8
3.3 Construction Planning..... App.1-11
3.4 Faasibility Cost Estimate..... App.1-14
4. Reports..... App.1-18
5. Staffing..... App.1-18
6. Schedule..... App.1-18

Appendix 2 - Basic Engineering Criteria

2.1 Canal Cross-Section and Bank Alignment..... App.2-1
2.2 Anchorages and Emergency Tie-Up Stations.... App.2-5
2.3 Calculation of Ship Transits and Lock Flushes..... App.2-11

Appendix 3 - Alternative to Tidal Lock and Tidal Gate

3.1 Discussion on Basic Designs and Alternative Designs..... App.3-1
3.2 Torsion Type Tidal Gate..... App.3-5

Appendix 4 - Cost Estimates

4.1 Additional Subsurface Investigations for Sea Level Canal..... App.4-1

4.2 Comparison of Studied Result (Sea Level Canal Route 10 Case b with Nagano Plan)..... App.4-2

4.3 High Rise Lock Canal..... App.4-4

Appendix 5 - Computer Outputs of Water Balance Simulation..... App.5-1

BASE MAPS

1. CONSTRUCTION PLAN AND TRANSPORTATION PLAN FOR HIGH RISE LOCK CANAL (1:100,000)

2. CONSTRUCTION PLAN AND TRANSPORTATION PLAN FOR ROUTE 10 SEA LEVEL CANAL (1:100,000)

3. CATCHMENT AREAS (1:250,000)

LIST OF TABLES

Page

SUMMARY

Table S.1-1 Main Features of Alternatives..... S-1
Table S.1-2 Summary for the Year 2020..... S-2
Table S.1-3 Summary for the year 2060..... S-3
Table S.1-4 Summary of Options Study..... S-3
Table S.2-1 Dams and Pump Stations..... S-8
Table S.2-2 150,000 DWT Ship Case(85Feet)..... S-10
Table S.2-3 200,000 DWT Ship Case(85Feet)..... S-10
Table S.2-4 Dam Dimensions in Gatun Watershed... S-10
Table S.2-5 Indio Dam Dimensions..... S-10
Table S.2-6 Excavation Volume..... S-12
Table S.2-7 Main Gate Dimensions (200,000 DWT Case)..... S-19
Table S.2-8 Main Gate Dimensions (150,000 DWT Case)..... S-19
Table S.3-1 Excavation Volume..... S-24
Table S.3-2 Dimension and Concrete Volume of Tidal Structure (2020)..... S-27
Table S.3-3 Main Gate Dimensions (250,000 DWT Vessel)..... S-27
Table S.3-4 Barrier Dam Dimensions..... S-30
Table S.3-5 River Improvement Measures..... S-35

PART II.Sect.1-Ch.1

Table II.1.1-1 Design Ship Dimensions..... II.1.1-1
Table II.1.1-2 Lock Chamber Dimensions..... II.1.1-2
Table II.1.1-3 Canal Cross Section Dimensions(m)... II.1.1-6
Table II.1.1-4 Comparison of Excavation Slope Gradient Criteria Proposed by Various Studies..... II.1.1-8
Table II.1.1-5 Adopted Number of Annual Ship Transits and Lock Flushes for Phase 2 Study..... II.1.1-18
Table II.1.1-6 Spacing of Anchoring Points..... II.1.1-24

PART II.Sect.1-Ch.2

Table II.1.2-1 High Rise Lock Canal Alignment..... II.1.2-2
Table II.1.2-2 Route 10 Sea Level Canal Alignment.. II.1.2-2

PART II.Sect.1-Ch.3

Table II.1.3-1 Factors for Subsoil Condition..... II.1.3-1
Table II.1.3-2 Subsoil Classes..... II.1.3-2
Table II.1.3-3 Coefficients of Importance..... II.1.3-2
Table II.1.3-4 Seismic Coefficient..... II.1.3-4

Page

PART II.Sect.1-Ch.4

Table II.1.4-1 Average Rainfall in Gatun Watershed II.1.4-6
Table II.1.4-2 Average Watershed Yield..... II.1.4-8

PART II.Sect.1-Ch.5

Table II.1.5-1(a) Population Date-Province of Colon... II.1.5-2
Table II.1.5-1(b) Population Date-Province of Panama II.1.5-3
Table II.1.5-2 Traffic Volume Date..... II.1.5-10

PART II.Sect.2-Ch.2

Table II.2.2-1 Comparisons for Earthwork System.... II.2.2-2
Table II.2.2-2(a) Case-1 Main Equipment :Shovel(30M3)+Dump Truck(240T)..... II.2.2-8
Table II.2.2-2(b) Case-2 Main Equipment :Shovel(18M3)+Dump Truck(195T)..... II.2.2-9
Table II.2.2-2(c) Case-3 Main Equipment :Shovel(15M3)+Dump Truck(150T)..... II.2.2-10
Table II.2.2-2(d) Case-4 Main Equipment :Motor Scraper(34M3)..... II.2.2-11
Table II.2.2-2(e) Case-Belt Conveyor System Main Belt W1=2.8M Branch Belt W=2=2.0M..... II.2.2-11
Table II.2.2-3(a) Combination of Dredging and Transport Equipment..... II.2.2-13
Table II.2.2-3(b) Combination of Dredging and Transport Equipment..... II.2.2-14
Table II.2.2-3(c) Combination of Dredging and Transport Equipment..... II.2.2-14
Table II.2.2-3(d) Combination of Dredging and Transport Equipment..... II.2.2-15
Table II.2.2-3(e) Combination of Dredging and Transport Equipment..... II.2.2-16
Table II.2.2-3(f) Combination of Dredging and Transport Equipment..... II.2.2-17
Table II.2.2-3(g) Combination of Dredging and Transport Equipment..... II.2.2-17

PART II.Sect.2-Ch.3

Table II.2.3-1(a) Available Working Days per Annum (Pacific Side)..... II.2.3-3
Table II.2.3-1(b) Available Working Days per Annum (Atlantic Side)..... II.2.3-4
Table II.2.3-1(c) Available Working Days per Annum for Dredging Works..... II.2.3-5
Table II.2.3-2(a) Load Factor and Unit Weight..... II.2.3-6
Table II.2.3-2(b) Load Factor and Unit Weight..... II.2.3-7
Table II.2.3-3 Drilling Equipment..... II.2.3-17
Table II.2.3-4 Characteristics of Explosives..... II.2.3-19
Table II.2.3-5 Application of Explosives..... II.2.3-19
Table II.2.3-6 Method of Secondary Blasting..... II.2.3-22

Page

Table II.2.3-7 Burden and Spacing Ratio..... II.2.3-26
Table II.2.3-8 Sources of Aggregates..... II.2.3-27

PART II.Sect.2-Ch.4

Table II.2.4-1 Major Facilities Related with Gate Leaves..... II.2.4-1
Table II.2.4-2 Classification Zone for Cost Competitiveness and Technical Capability..... II.2.4-3
Table II.2.4-3 Classification of Cost Component.... II.2.4-3
Table II.2.4-4 Approx. Maximum Block Weight in Ton of Gate Leaves When Launched at Shop..... II.2.4-4
Table II.2.4-5 Gate Leaves to be Shipped after Dis-assembling at Shop..... II.2.4-5
Table II.2.4-6 Transportation Precedure..... II.2.4-7
Table II.2.4-7 Dimensions of Products when Shipped (200,000 DWT H.R.L.C. Option)..... II.2.4-7
Table II.2.4-8 Possibility of Local Fabrication.... II.2.4-8

PART II.Sect.3-Ch.2

Table II.3.2-1 Unit Price..... II.3.2-1
Table II.3.2-2 Unit Price..... II.3.2-2
Table II.3.2-3 Unit Price..... II.3.2-3
Table II.3.2.2-1 Manpower (Staff&Labor) Rates..... II.3.2-4
Table II.3.2.3-1 Equipment Costs for Earthworks (1) II.3.2-5
Table II.3.2.3-2 Equipment Costs for Earthworks (2) II.3.2-6
Table II.3.2.3-3 Equipment/Vessels Cost for Canal Dredging..... II.3.2-7
Table II.3.2.3-4 Equipment/Vessels Cost for Barrier Dam..... II.3.2-8
Table II.3.2.3-5 Equipment/Vessels Cost for Breakwater(1)..... II.3.2-9
Table II.3.2.3-6 Equipment/Vessels Cost for Breakwater(2)..... II.3.2-10

PART II.Sect.3-Ch.3

Table II.3.3-1 PCC Annual Report 1991, Operating Expenses..... II.3.3-4
Table II.3.3-2 Detailed Breakdown of Operating Expenses 1991..... II.3.3-5
Table II.3.3-3 Phase Report O&M Costs..... II.3.3-11
Table II.3.3-4 Year 2020 O&M Costs and Ratios..... II.3.3-20
Table II.3.3-5 Year 2060 O&M Costs and Ratios..... II.3.3-21

Page

APPENDIX 2

Table A.2-1	Minimum Lane Widths on Curves.....	App.2-2
Table A.2-2	Depths in Atlantic Approaches.....	App.2-4
Table A.2-3	Dimensions of Approaches.....	App.2-5
Table A.2-4	Vessel Grouping for Anchorage.....	App.2-6
Table A.2-5	Scope of Chain at Difference Water Depths.....	App.2-6
Table A.2-6	Spacing of Anchoring Points.....	App.2-7
Table A.2-7(a)	Total Transit Projections for 2020	App.2-12
Table A.2-7(b)	Total Transit Projections for 2060	App.2-13
Table A.2-8	CAS Data for Assignment of Transits between New and Existing Facilities	App.2-14
Table A.2-9	Adopted Number of Annual Ship Transits.....	App.2-18
Table A.2-10	Comparison of Assigned Annual Transits and Capacity.....	App.2-19
Table A.2-11	Volume of Lockage Water per Transit and Number of Flushes for Existing Locks.....	App.2-21

APPENDIX 3

Table A.3-1	Reduction Amounts for Todal Cost of Canal Alternatives under Studied.....	App.3-4
Table A.3-2	Dimensions and Weights of Operating Gate and Emergency Gate.....	App.3-6
Table A.3-3	Design Heads in Meter of Operating Gate and Emergency Gate.....	App.3-8
Table A.3-4	Cost Estimate for Gate and Related Facilities 250,000 DWT Sea Level Canal Case (b) - Alternative (2020).....	App.3-16
Table A.3-5	Cost Estimate for Gate and Related Facilities 250,000 DWT Sea Level Canal Case (c) - Alternative (2020).....	App.3-17

LIST OF FIGURES

Page

REFACE

FIG.1 Overall Work Plan Flow..... (iii)

SUMMARY

Fig.S.1-1 Layout of High Rise Lock Canal (200,000 DWT)..... S-4
Fig.S.1-2 Layout of Sea Level Canal [Case (a) and (c)]..... S-5
Fig.S.1-3 Case of the Sea Level Canal..... S-6
Fig.S.2-1 Dam Locations..... S-11
Fig.S.2-2 Typical Cross Section of Fill Type Dam.S-11
Fig.S.2-3 Excavation Volume for Each Case..... S-13
Fig.S.2-4 Excavation Volume for Each Zone (85'and 200,000DWT Case)..... S-13
Fig.S.2-5 Zone Map of High Rise Lock Canal..... S-14
Fig.S.2-6 New Lock Layout Plan(Pacific Side).... S-17
Fig.S.2-7 New Lock Layout Plan(Atlantic Side)... S-17
Fig.S.2-8 Typical Cross Section of Middle Operating Gate (200,000DWT)..... S-18
Fig.S.2-9 Road Crossing Location..... S-21
Fig.S.3-1 Excavation Volume for Each Case..... S-24
Fig.S.3-2 Excavation Volume for Each Zone (Case a)..... S-24
Fig.S.3-3 Zone Map of Sea Level Canal..... S-25
Fig.S.3-4 Tidal Lock Layout..... S-29
Fig.S.3-5 Tidal Gate Layout..... S-29
Fig.S.3-6 Barrier Dam Locations..... S-31
Fig.S.3-7 Typical Section of Barrier Dam..... S-32
Fig.S.3-8 Diversion Scheme Layout..... S-34
Fig.S.3-9 Typical Section of Breakwater..... S-36

PART I Ch.1

Fig.I.1-1 Route of the High Rise Lock Canal..... I.1-4
Fig.I.1-2 Schematic Layout of the High Rise Lock Canal..... I.1-5
Fig.I.1-3 Route of the Sea Level Canal [Case (a)]..... I.1-10
Fig.I.1-4 Schematic Layout of the Sea Level Canal..... I.1-11

Page

PART II Sect.1-Ch.1

Fig.II.1.1-1(a) Definition of L1 & L2 for SLC..... II.1.1-3
Fig.II.1.1-1(b) Definition of L1 & L2 for HRLC..... II.1.1-3
Fig.II.1.1-2 Typical Canal Sections..... II.1.1-5
Fig.II.1.1-3 Slope Criteria for Type 1..... II.1.1-11
Fig.II.1.1-4 Slope Criteria for Type 2..... II.1.1-11
Fig.II.1.1-5 Slope Criteria for Type 3..... II.1.1-12
Fig.II.1.1-6 Slope Criteria for Type 4..... II.1.1-12
Fig.II.1.1-7 Slope Criteria for Type 5..... II.1.1-13
Fig.II.1.1-8 IOCS Slope Criteria..... II.1.1-14
Fig.II.1.1-9 Nagano Plan Slope Criteria..... II.1.1-15
Fig.II.1.1-10 Proposed Criteria by ECE Contractor... II.1.1-16
Fig.II.1.1-11 Tide Levels and PLD Level Datum..... II.1.1-22
Fig.II.1.1-12 Layout of Emergency Tie-Up Station.... II.1.1-26

PART II Sect.1-Ch.3

Fig.II.1.3-1 Panama Earthquake (7 Sep.1882)..... II.1.3-4

PART II Sect.1-Ch.4

Fig.II.1.4-1 Rainfall and Discharge Gauge Locations in Gatun Watershed..... II.1.4-5
Fig.II.1.4-2 Specific Runoff..... II.1.4-7
Fig.II.1.4-3 Gatun and Madden Area Capacity Curve . II.1.4-9

PART II Sect.1-Ch.5

Fig.II.1.5-1 Administrative Areas..... II.1.5-4
Fig.II.1.5-2 Population Densities..... II.1.5-5
Fig.II.1.5-3 Existing Road Network..... II.1.5-7
Fig.II.1.5-4 Power Stations and Electricity Distribution Lines..... II.1.5-13

PART II Sect.2-Ch.2

Fig.II.2.2-1 Standard Excavation Plan..... II.2.2-4
Fig.II.2.2-2 Earthwork System Using Dump Truck..... II.2.2-5
Fig.II.2.2-3 Earthwork System Using Conveyor Belt . II.2.2-6
Fig.II.2.2-4 Earthwork System Using Motor Scraper . II.2.2-7
Fig.II.2.2-5 Unit Cost (S/m3)..... II.2.2-12
Fig.II.2.2-6 Typical Dredging Procedure..... II.2.2-18
Fig.II.2.2-7 Type of Dredgers..... II.2.2-19

Page

PART II Sect.2-Ch.3

Fig.II.2.3-1	Layout of Bench	II.2.3-8
Fig.II.2.3-2	Section of Canal and Location of Ditches	II.2.3-9
Fig.II.2.3-3	Installation of Dewatering Pump	II.2.3-10
Fig.II.2.3-4(a)	General Layout for Drainage	II.2.3-12
Fig.II.2.3-4(b)	General Layout for Sedimentation Pond	II.2.3-13
Fig.II.2.3-4(c)	General Plan and Section of Ditch	II.2.3-14
Fig.II.2.3-5	Illustration of Drilling Equipment	II.2.3-17
Fig.II.2.3-6	Typical Section of Standard Bench Cut	II.2.3-18
Fig.II.2.3-7	Typical Section of Bench Cut Charge	II.2.3-21
Fig.II.2.3-8	Wide Space Hole Blasting Patterns	II.2.3-25
Fig.II.2.3-9	Large Size Fragmentation Patterns	II.2.3-25
Fig.II.2.3-10	Typical Section Bench Cut Burden and Spacing	II.2.3-26
Fig.II.2.3-11	Location of Quarries	II.2.3-28
Fig.II.2.3-12	Flow Chart of 600t/h Crushing Plant	II.2.3-29
Fig.II.2.3-13	Layout of Concrete Production Plant	II.2.3-31

APPENDIX 2

Fig.A.2-1	Layout of Emergency Tie-Up Stations	App.2-9
Fig.A.2-2(a)	Number of Lock Flushes (by Dr. K. KOBUNE, CAS)	App.2-22
Fig.A.2-2(b)	Number of Lock Flushes (by Dr. K. KOBUNE, CAS)	App.2-23
Fig.A.2-2(c)	Number of Lock Flushes (by Dr. K. KOBUNE, CAS)	App.2-24

APPENDIX 3

Fig.A.3-1	General Arrangement of Tidal Gate (Torsion Type) Sea Level Canal 250,000 DWT Canal Option	App.3-7
Fig.A.3-2	Result of Stractural Analysis of Tidal Gate (Torsion Type) Principal Stress Table Sea Level Canal 250,000 DWT Canal Option	App.3-12
Fig.A.3-3	Result of Structural Analysis of Tidal Gate (Torsion Type) Displacement Diagram Sea Level Canal 250,000 DWT Canal Option	App.3-13
Fig.A.3-4	Result of Structural Analysis of Tidal Gate (Torsion Type) Principal Stress Diagram Sea Level Canal 250,000 DWT Canal Option	App.3-14

ABBREVIATION

Abbreviation		Term
OCCE Study	:	Operating Characteristics and Capacity Evaluation Study
CTP Study	:	Commodity and Traffic Projections Study
ECE Study	:	Engineering and Cost Estimation Study
CAS	:	Commission for the Study of Alternatives to the Panama Canal
PCC	:	Panama Canal Commission
HRLC	:	High Rise Lock Canal
SLC	:	Sea Level Canal Route 10
AASHTO	:	American Association of State Highway and Transportation Officials
CALM	:	Catenary Anchor Leg Mooring
CAPAC	:	Camana Panamena De La Construction
IOCS	:	Inter Ocean Canal Studies
JAX	:	Department of the Army Jacksonville District, Corps of Engineers
JSSC	:	Japanese Society of Steel Construction
PIANC	:	Permanent International Association of Navigation Congress
SCADA	:	Supervising Control and Data Acquisition
SQ	:	Status Quo, Existing Panama Canal
Km	:	Kilometers
m	:	meters
mm	:	millimeters
ft	:	feet
in	:	inches

Abbreviation		Term
ha	:	hector
φ	:	diameter
KW	:	Kilowatts
Kg	:	Kilograms
DWT	:	Dead Weight Tonnage
HWL	:	High Water Level
MWL	:	Mean Water Level
LWL	:	Low Water Level
US$ or $	:	US dollars

PREFACE

The Study of Alternatives to the Panama Canal
(Component Study : Engineering and Cost Estimates)

Phase 2 : Preliminary Engineering and Cost Estimates

PREFACE

The Phase 2 Study is part of the Study of Alternatives to the Panama Canal being carried out under a Commission representing the Governments of the Republic of Panama, the United States of America, and Japan. A Secretariat composed of technical staff from the member nations was also established.

After a selection process, the Commission chose a consortium of consultant firms representing the three countries (SINBOL) to carry out a Feasibility Analysis/Final Report (FA/FR) Study of Alternatives to the Panama Canal. The Detailed Plan of the FA/FR Study was produced in September 1990 and included an overall work flow chart (Fig. 1). The component studies included the Engineering and Cost Estimates Study (ECE Study), identified in Fig. 1 as work items B.2.1, B.2.2, B.3.2 and B.3.3.

In Phase 1, corresponding to work items B.2.1 and B.2.2, a broad range of study cases was collated and developed so that the engineering feasibility and cost of each study case could be better assessed for prescreening. Yachiyo Engineering Company Limited (YEC) completed the Phase 1 study on Engineering and Cost Estimates and submitted their Final Report in February 1992.

In accordance with the results of the prescreening carried out by the Commission at the end of Phase 1, the Phase 2 Study focuses on the following two alternatives:

1. High Rise Lock Canal
2. Route 10 Sea Level Canal

The purpose of the Phase 2 Study, corresponding to work items B.3.2 and B.3.3 of Fig. 1, is to provide sufficient engineering and cost estimate information, data, analysis, and conclusions to enable the Commission, after considering the results of all component studies, to select the best and most viable alternative to the present Panama Canal.

The first report to be submitted in Phase 2 was the Inception Report. Following meetings in Panama in August/September 1992 with the CAS Secretariat, the Inception Report was revised to reflect agreements reached at the meetings.

The Interim Report dated October 1992 was submitted on October 21, 1992. Following submission, discussions were held with the CAS Secretariat in Panama between November 11 and November 19, and the agreements reached are described in the Memorandum of Meeting dated November 19, 1992. Subsequent monitoring meetings were held with a representative of the CAS Secretariat between December 15 and December 18. The agreements reached during these meetings are described in the Memorandum of Meeting dated December 18, 1992.

The Draft Final Report which was submitted on February 6, 1993 included substantial completion of most items to be addressed on the Phase 2 Study. The Draft Final Report was jointly reviewed with the CAS Secretariat in Panama between 17 and 25 February 1993 and agreements reached were Summarized in the Memorandum of Meeting dated 25 February 1993. The Final Report was submitted on 20 April 1993.

The alternatives under consideration, study outline and objectives are briefly explained in Part I. The basic conditions and assumptions, etc. commonly used in developing the preliminary designs and costing in Part III and Part IV are described in Part II. The technical examination results, construction plans and cost estimates for the new High Rise Lock Canal and Sea Level Canal are given in Part III and Part IV respectively.

Final Report 1/2

Summary
Part I : Introduction & Objectives
Part II : General
Appendix
Base Maps

Final Report 2/2

Part III : High Rise Lock Canal
Part IV : Sea Level Canal

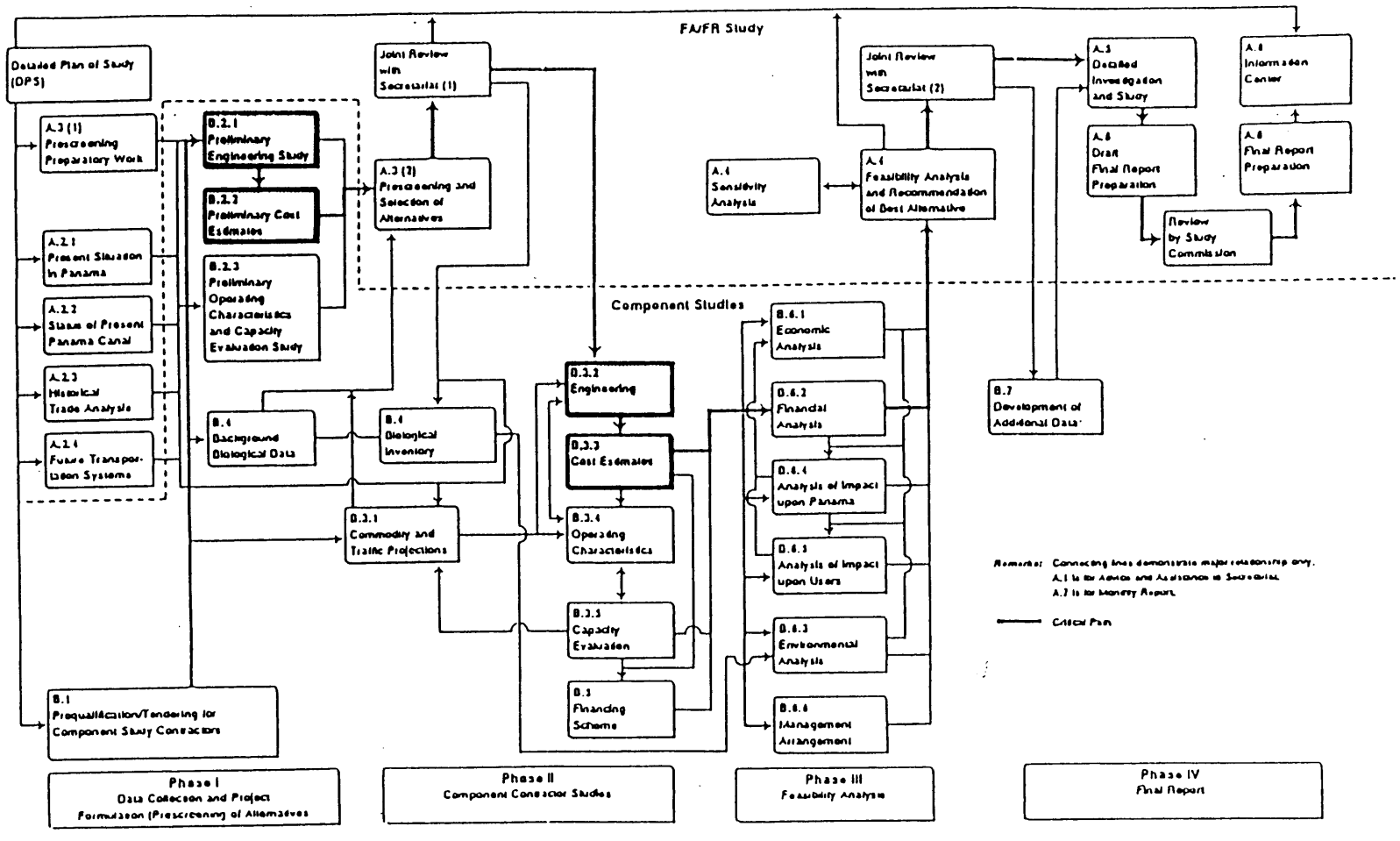


Fig. 1 Overall Work Plan Flow Chart

EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

TABLE OF CONTENTS

Page

1. GENERAL S-1

   1.1 Features of Alternatives S-1
   1.2 Result of the Study S-2

2. HIGH RISE LOCK CANAL S-7
   (Part III. Section 1 – Engineering and Preliminary Design)

   2.1 General S-7
   2.2 Water Management Plan S-7
   2.3 Canal Excavation Plan S-12
   2.4 Lock Structures and Mechanical Equipment S-15
   2.5 Incidental Structure S-20

3. SEA LEVEL CANAL S-22
   (Part IV. Section 1 – Engineering and Preliminary Design)

   3.1 General S-22
   3.2 Water Management Plan S-22
   3.3 Canal Excavation Plan S-23
   3.4 Tidal Locks and Tidal Gates S-26
   3.5 Barrier Dams S-30
   3.6 Diversion Scheme S-33
   3.7 Incidental Structures S-36

4. MEMBERS OF THE STUDY TEAM FOR PHASE 2 STUDY S-38

5. SUMMATION TABLE S-39

EXECUTIVE SUMMARY

1. GENERAL

The objective of the study in Phase 2 is to provide engineering, preliminary design, construction planning and feasibility engineering, cost estimates for the following two alternatives:

1) High Rise Lock Canal
2) Sea Level Canal

1.1 Features of Alternatives

The main features of the High Rise Lock Canal and Sea Level Canal are listed in the table below. Details of these two alternatives are summarized in the summation tables (1) and (2) attached at the end of this Executive Summary.

Table S.1-1 Main Features of Alternatives

Alternatives		High Rise Lock Canal		Sea Level Canal		
Alignment		Along Existing Canal		Route 10 from IOCS Study		
		Shown as Fig. S.1-1		Shown as Fig. S.1-2		
Maximum Ship Size		150,000	200,000	250,000 DWT		
Number of Ship Transits	(2020) Existing C.	16,200	16,200	16,200		
	New Canal	2,000	2,000	2,100		
	Total	18,200	18,200	18,300		
	(2060) Existing C.	20,000	20,000	19,400		
	New Canal	4,600	4,600	5,100		
	Total	24,600	24,600	24,500		
Number of Lanes		2		1 (approach 2)		
Number of Lifts		2		0		
Water Level		85 feet & 90 feet		Sea level		
Tidal Lock & Gate		n.a.		Case (a)	Case (b)	Case (c)
				parallel 4 locks	1 lane 2 gates	parallel 4 locks & 1 gate
				Shown in Fig. S.1-3		

n.a.: not applicable

1.2 Result of the Study

The Study proposals are phased to meet transit requirements for the years 2020 and 2060 or beyond, and it has been assumed that the existing canal and locks will continue to operate in addition to the new canal. Construction unit costs have been adjusted to take into account the enormous scale of construction.

The results of the study are as follows.

1) Details of the Study are summarized in the summation tables attached at the end of the Executive Summary.

2) Total construction cost for the year 2020 and the additional cost for the year 2060 with main facilities included in the cost are listed in the Tables S.1-2 and S.1-3.

3) Results of the sensitivity analysis for the canal bank slope and the bottom width of channel are shown in Table S.1-4 for Option Study.

Table S.1-2 Summary for the Year 2020

Item	High Rise Lock (85 feet)		Sea Level Canal		
	150,000 DWT	200,000 DWT	Case (a)	Case (b)	Case (c)
Total Cost (Mill.US$)	7,374	8,546	13,519	12,747	14,159
New Facilities Included:					
Canal	2 lane canal		1 lane canal & 2 lane approach		
Lock and Gate	third lane lock (one lane of new locks)		parallel 3 locks	1 lane 2 gates	parallel 3 locks & 1 gate
Water Supply	n.a.	Indio	n.a.		
Pump Station (MCM)	n.a.				
Barrier Dam	n.a.		3		
Freshwater Barrier	n.a.		considered		
Diversion Scheme	n.a.		considered		
Crossing Bridges	2 places		3 places		
Breakwater	existing available		additional new		

n.a.: not applicable

Table S.1-3 Summary for the Year 2060

(Additions to the Year 2020)

Item	High Rise Lock (85 feet)		Sea Level Canal		
	150,000 DWT	200,000 DWT	Case (a)	Case (b)	Case (c)
Total Cost (Mill.US$)	1,407	1,350	1,717	775	1,717
Lock and Gate	fourth lane lock (later than 2060)		fourth lane lock	n.a	fourth lane lock
Water Supply	2 dams Ciri, Indio	1 dam Trinidad	1 dam Upper Gatun		
Pump Station	2 places	2 places	n.a.		
Crossing Bridges	2 places		4 places		

n.a.: not applicable

Table S.1-4 Summary of Options Study

Item	Additional Cost (Mill. US$)				
	High Rise Lock (85 feet)		Sea Level Canal		
	150,000 DWT	200,000 DWT	Case (a)	Case (b)	Case (c)
Canal Bank Slope Cotangent 0.4 Increase	n.s.	277	837	n.s.	837
Increase Bottom Width of Channel from 7B to 8B	n.s.	555	129	n.s.	129

n.s.: not studied

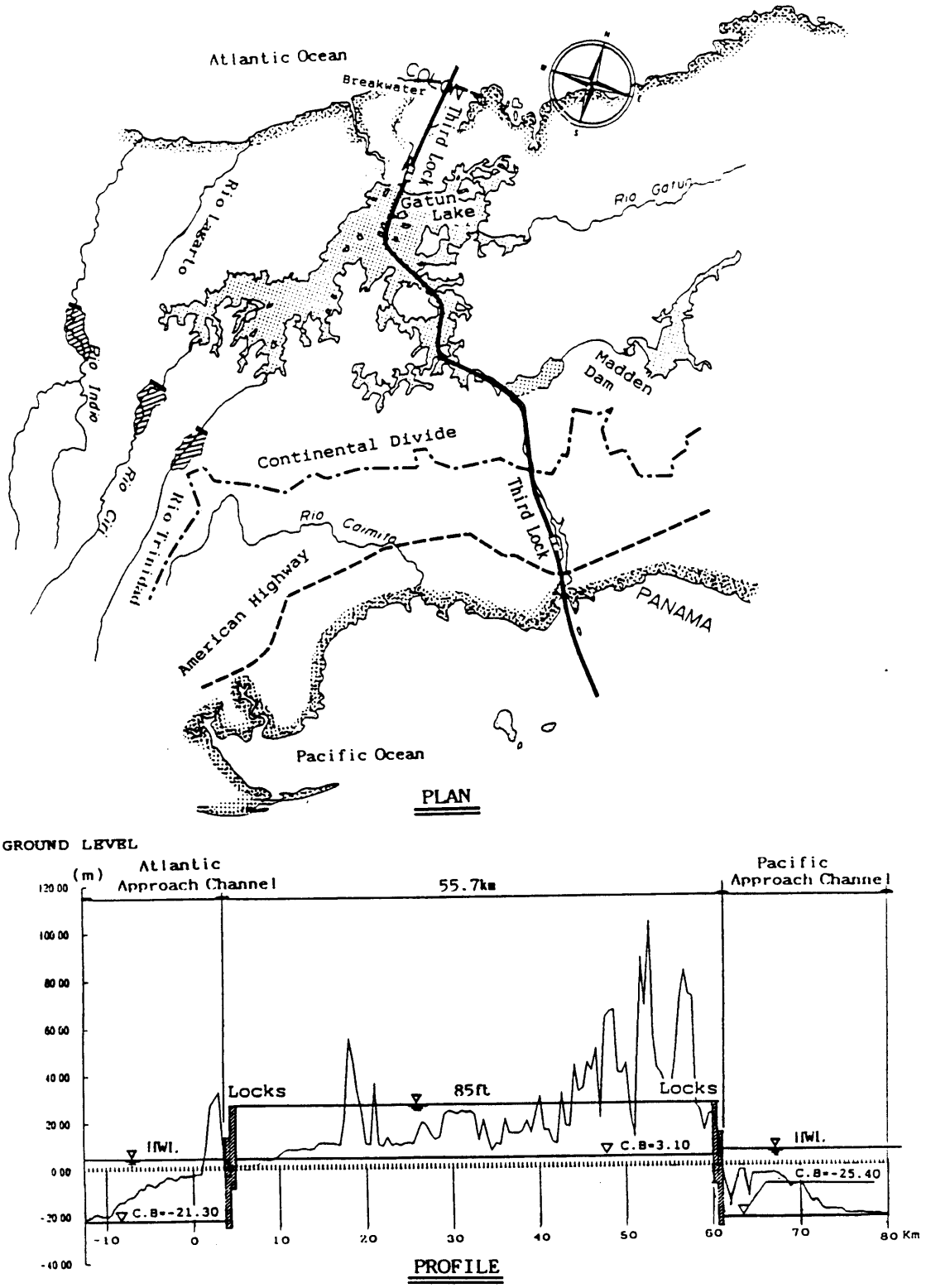


Fig. S.1-1 Layout of High Rise Lock Canal (200,000 DWT)

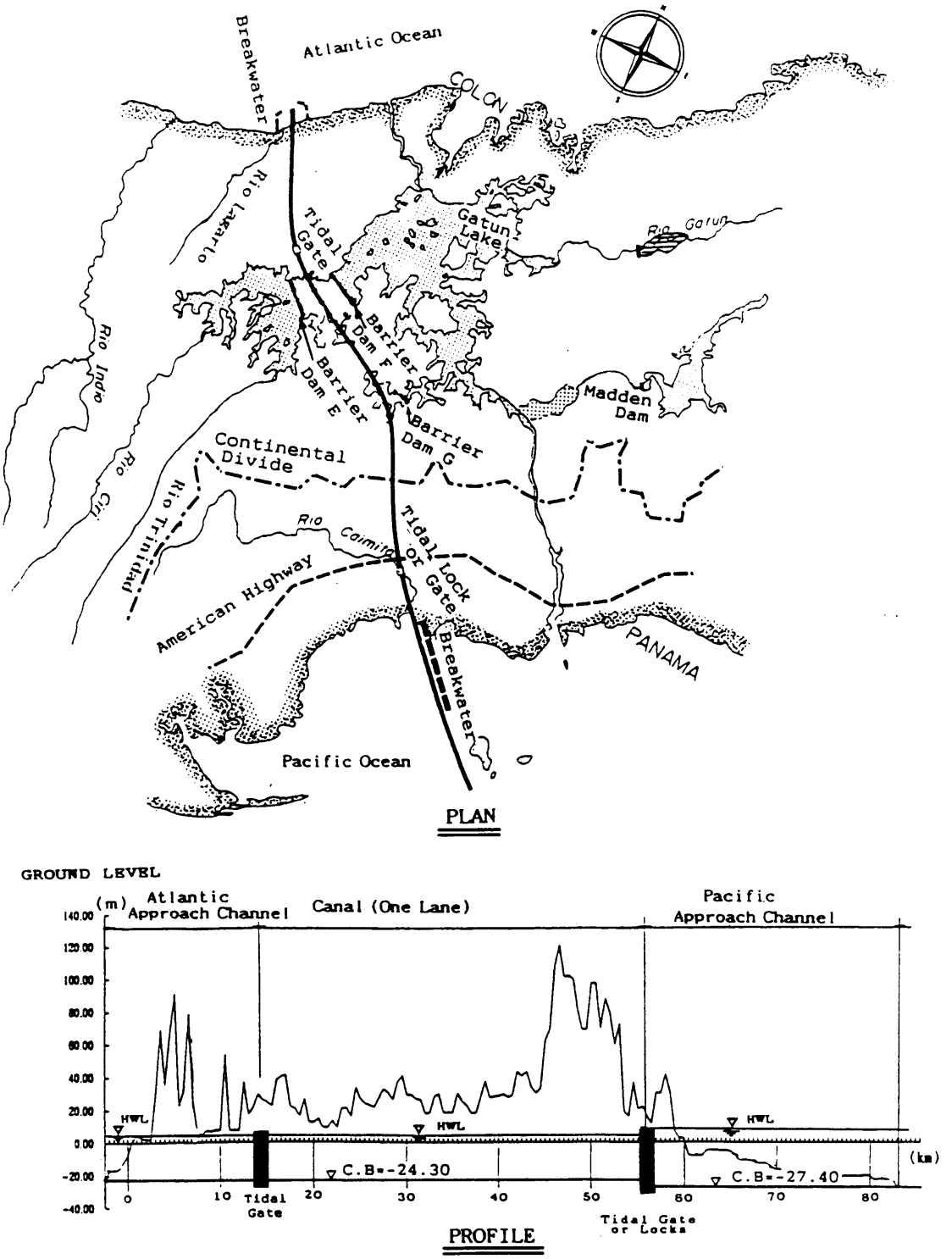


Fig. S.1-2 Layout of Sea Level Canal [Case (a) and (c)]

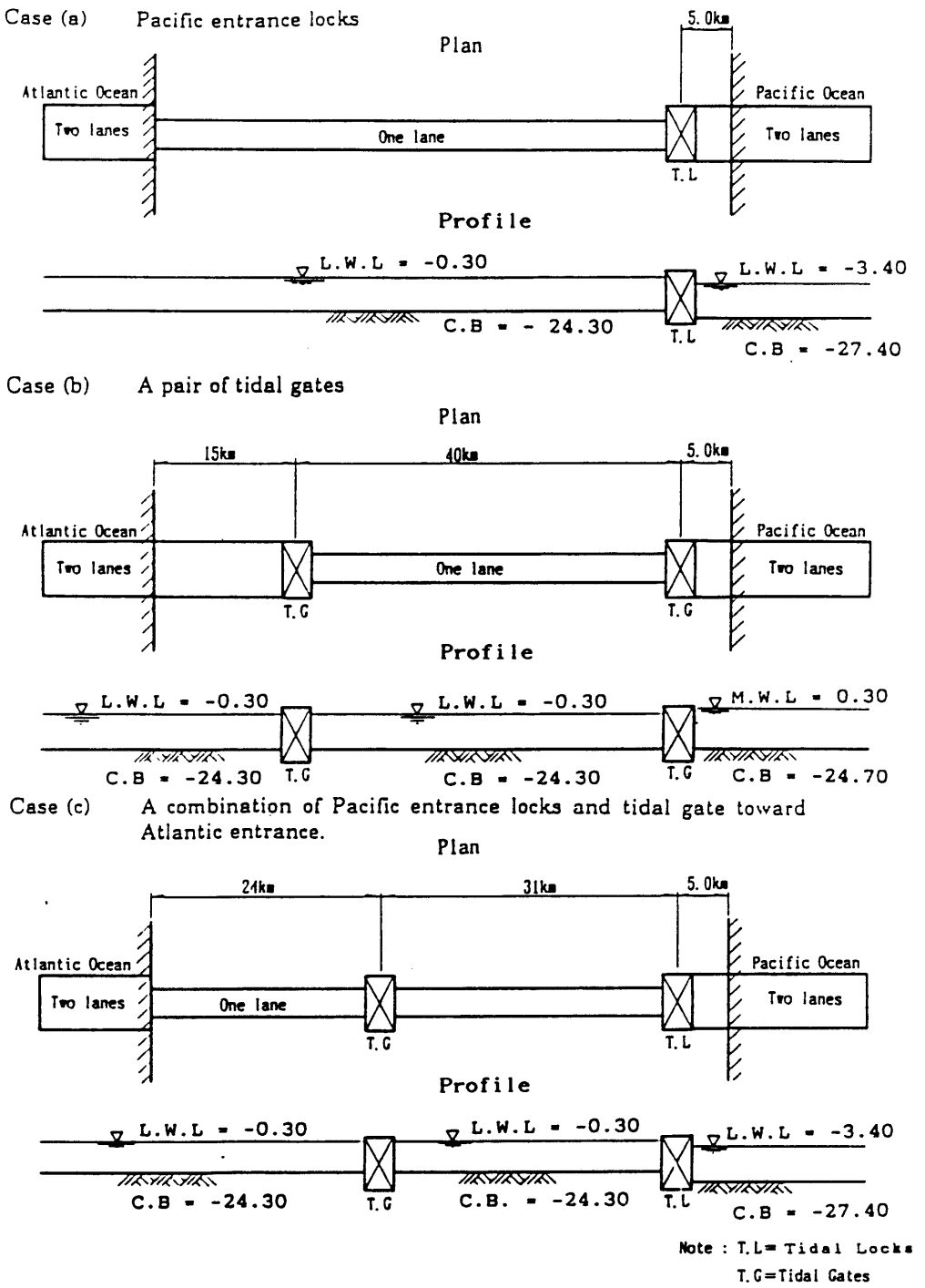


Fig. S.1-3 Case of the Sea Level Canal

2. HIGH RISE LOCK CANAL
(Part III, Section 1 Engineering and Preliminary Design)

With regard to the High Rise Lock Canal alternative, two case studies were conducted, for prospective ship sizes of 150,000 DWT and 200, 000 DWT. Two water levels, i.e. 85' and 90', were also considered for Gatun lake.

2.1 General

General engineering conditions used in Engineering and Preliminary Design are summarized in the Summation Table (1) attached in the last page. The Schematic Layout of the High Rise Lock Canal is shown in Fig. S.1-1.

2.2 Water Management Plan

2.2.1 Annual Yield

Calculation of the annual yield of the Gatun watershed of 3,339km^2 is necessary to determine the available lockage water volume for the High Rise Lock Canal. The annual yield minus municipal use (4%) and evaporation loss (10%) gives the available lockage water volume. Based on analysis of the runoff of the last 51 years, the average annual runoff (yield) of the Gatun watershed is 6.6 billion m^3. After reduction for municipal use and evaporation loss, the available lockage water volume is 5.6 billion m^3.

2.2.2 Water Balance

From the calculations, the following have become clear.

1) 150,000 DWT in the year 2020 case

   The required water volume (3,881 million m^3) for the existing and new locks can be supplied from the existing catchment and dams, and no special counter measures are needed.

2) 150,000 DWT in the year 2060 case

   The required water volume (5,872 million m^3) cannot be supplied from the existing catchment and dams, and a new water source (Indio) and regulating dam (Ciri) are necessary.

3) 200,000 DWT in the year 2020 year case

   The required water volume (4,068 million m^3) cannot be supplied from the existing catchment and dams in a dry year, and Indio Dam is needed as a new water source.

4) 200,000 DWT in the year 2060 case

The required water volume (6,303 million m^3) cannot be supplied from the existing catchment and dams even in an average year. Trinidad dam as a regulating reservoir and Indio dam as a new water source are necessary. A recycling pump system is also needed.

5) Facilities for water supply dams and pump stations are summarized in Table S.2-1.

Table S.2-1 Dams and Pump Stations

Year	Rain fall	Max. Ship Size DWT		
		High Rise Lock Canal (85 feet)		Sea Level Canal
		150,000 DWT	200,000 DWT	250,000 DWT
2020	Wet year	n.r.	n.r.	n.r.
	Average year	n.r.	n.r.	n.r.
	Dry year	n.r.	Indio	n.r.
2060	Wet year	n.r.	n.r.	n.r.
	Average year	Indio Ciri	Indio Trinidad Pump S	n.r.
	Dry year	Indio Ciri Pump S	Indio Trinidad Pump S	Upper Gatun

Note: n.r. : not required.
wet year : 1981 (maximum yield in last 10 years)
average year : average yield in 51 years
dry year : 1982 (minimum yield in last 10 years)

The relationship between the required water volume for the existing and new locks and the available water volume in an average year is shown in Table S.2-2 and S.2-3.

In all cases there will be no water shortage except for special situations for the high rise cases. The water shortage may occur near 2060 and especially during dry years. However with addition dams and/or pump stations there will be sufficient water for all situations.

2.2.3 Dams in Gatun Watershed (Acting as Regulating Reservoirs)

Any shortage in the lockage water cannot be compensated for regardless of how many dams are constructed in the Gatun watershed. The best way of compensating is the use of water from the Indio dam located outside the Gatun watershed or pumping to recycle water. Consequently, such planned dams as the Ciri, Trinidad or Upper Gatun, all located in the Gatun watershed, should act as regulating reservoirs for the effective use of the available water for lockages. The feasibility of such dams acting as regulating reservoirs was examined at the 3 sites as shown in Table S.2-4. The elevation of the crown, dam height and effective storage of each dam site are given in the table while the locations of these sites are shown in Fig. S.2-1. The typical dam cross section is shown in Fig. S.2-2. It was decided that the Ciri and Trinidad dams for ship size 150,000 DWT and 200,000 DWT respectively should be used as a regulating dam.

2.2.4 Indio Dam (Acting as New Water Source)

Planning of the Indio dam in the upper reach of Rio Indio will provide a new water source. The water stored by this dam will be guided to the western end of Gatun lake via an open channel. The main specifications of this dam are given in Table S.2-5 while its typical cross-section is shown in Fig. S.2-2.

2.2.5 Water Circulation Plan

This system guides the lockage water from the lock's lower chamber through an open channel to a recycling basin for temporary storage, instead of discharging it directly to the sea. The stored water is pumped to the Gatun lake side. The pump capacity used in the calculation was $30m^3$/sec with a pipe diameter of 3,600mm and a motor output of 9,200kw.

Pumping is not required for the year 2020. For the year 2060, two pump stations are proposed, one on the Atlantic side and one on the Pacific side. Allowing for outages, two pumps are required at each pump station.

It is necessary to provide a recycling basin and pump station on both the Atlantic and Pacific sides since the volume of spent lockage water discharged from the new locks on one side only is less than the volume of water required to be recycled. Separate recycling ponds and pump stations will also be an advantage in the event of an emergency.

Table S.2-2 150,000 DWT Ship Case (85 Feet)

Description	Year	2020	2060
1. Available Water (Average Year)	MCM	5,647	5,653
2. Lockage Water (for Existing Locks)	"	2,867	3,540
3. Lockage Water (for New Locks)	"	1,014	2,332
4. Total Lockage Water (2 & 3)	"	3,881	5,872
5. Water Balance	"	+1,766	-219

Table S.2-3 200,000 DWT Ship Case (85 Feet)

Description	Year	2020	2060
1. Available Water (Average Year)	MCM	5,647	5,653
2. Lockage Water (for Existing Locks)	"	2,867	3,540
3. Lockage Water (for New Locks)	"	1,201	2,763
4. Total Lockage Water (2 & 3)	"	4,068	6,303
5. Water Balance	"	+1,579	-650

Table S.2-4 Dam Dimensions in Gatun Watershed

Name	Dam Crest Elevation	Dam Height	Effective Storage Volume
Ciri Dam	125 m	60 m	120 million m^3
Trinidad Dam	145 m	80 m	475 million m^3
Upper Gatun Dam	120 m	95 m	340 million m^3

Table S.2-5 Indio Dam Dimensions

Name	Dam Crest Elevation	Dam Height	Effective Storage Volume
Indio Dam (H)	95 m	90 m	710 million m^3
Indio Dam (L)	85 m	80 m	550 million m^3

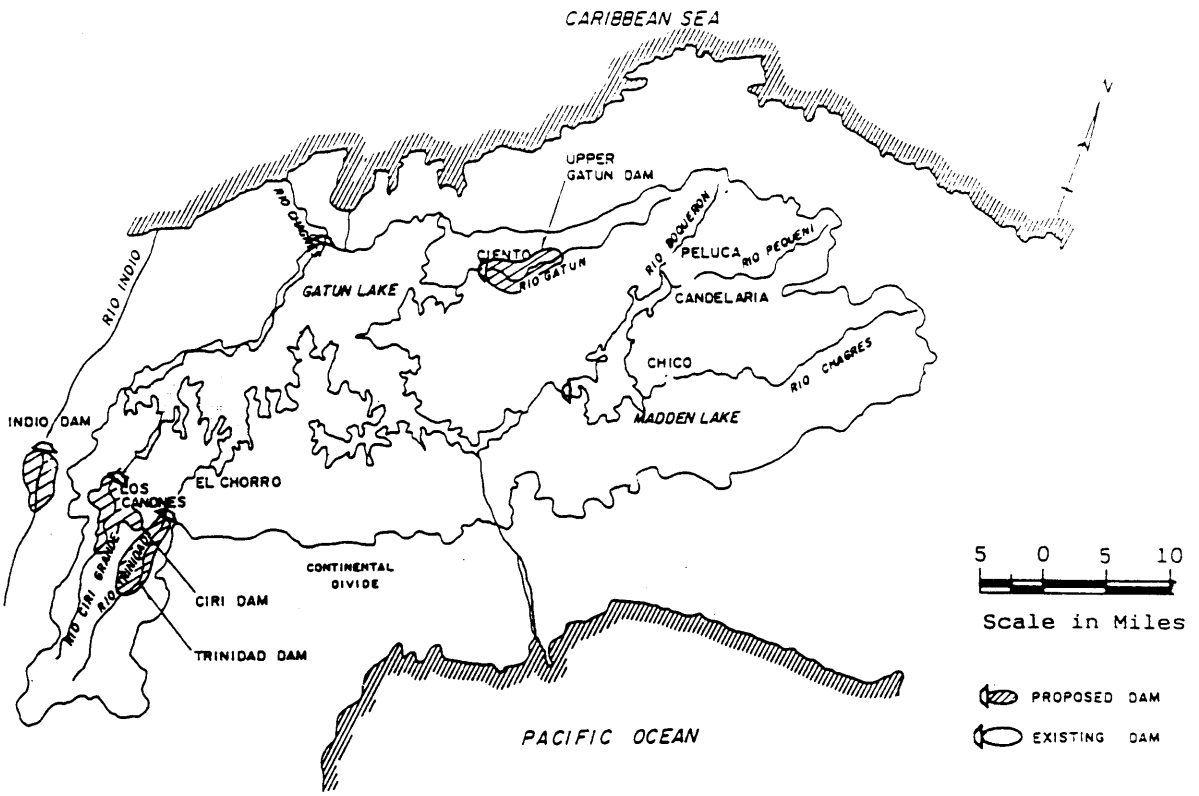


Fig. S.2-1 Dam Locations

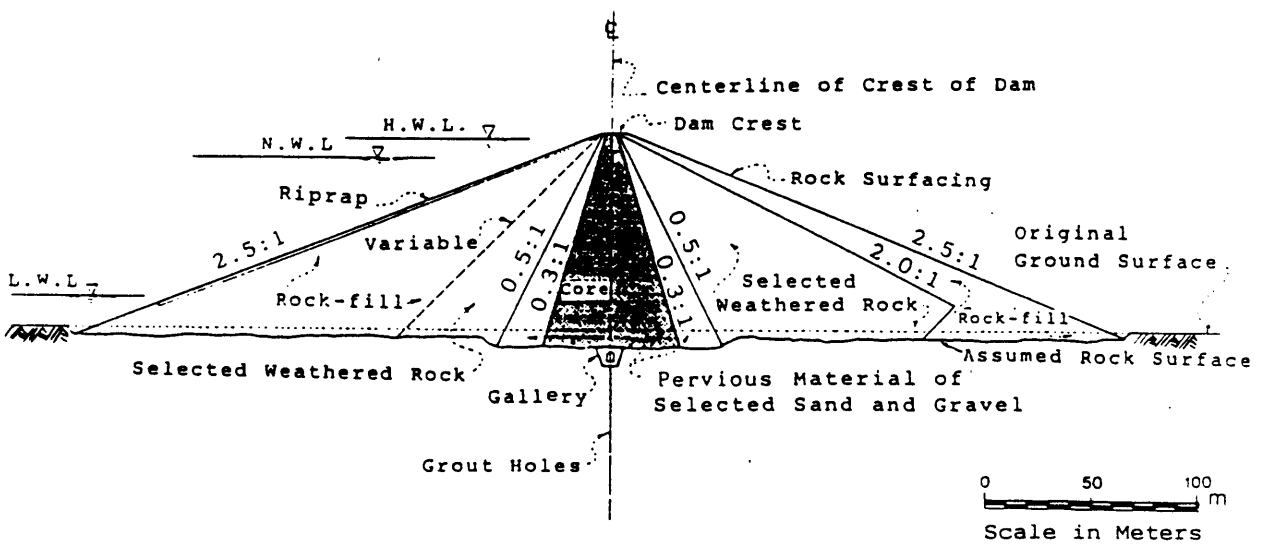


Fig. S.2-2 Typical Cross Section of Fill Type Dam

2.3 Canal Excavation Plan

2.3.1 Description

Calculation of the excavation volume was conducted for 4 different cases involving two ship sizes, i.e. 150,000 DWT and 200, 000 DWT, and two lake water levels, i.e. 85' and 90'. Topographical maps and sea charts (with a scale of 1/25,000) were used for the calculation. Moreover, five geographical types were introduced to represent the soil and rock characteristics of each zone. (See Fig. S.2-5)

2.3.2 Calculation Results

Calculation of the excavation volume was conducted separately for ground (dry) excavation and dredging. The results are given in Table S.2-6 and Fig. S.2-3. The relevant figures for Sea Level Canal Case (a) are also given for reference purposes. The excavation volume for each zone with a water level of 85' and a ship size of 200,000 DWT are given in Fig. S.2-4. Both the dry and wet excavation conditions are further classified in terms of the geological characteristics, i.e. soil, soft rock and hard rock, etc. Please refer to the main report for a more detailed description.

Table S.2-6 Excavation Volume

Water Level	Ship Size (DWT)	Dredging Volume (million m^3)	Dry Excavation Volume (million m^3)	Total Excavation Volume (million m^3)
85'	200,000	358	242	600
85'	150,000	295	217	512
90'	200,000	346	229	575
90'	150,000	286	205	491

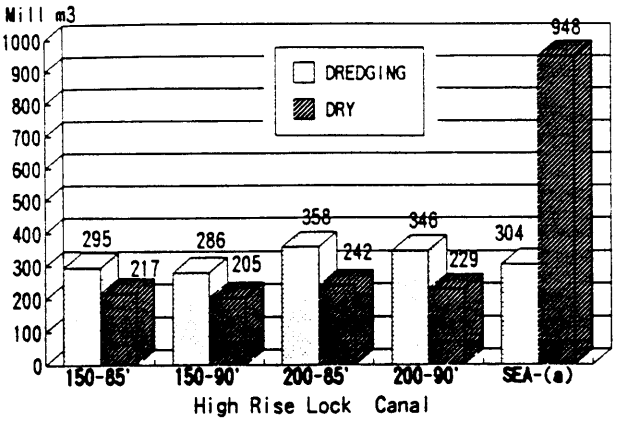


Fig. S.2-3 Excavation Volume for Each Case

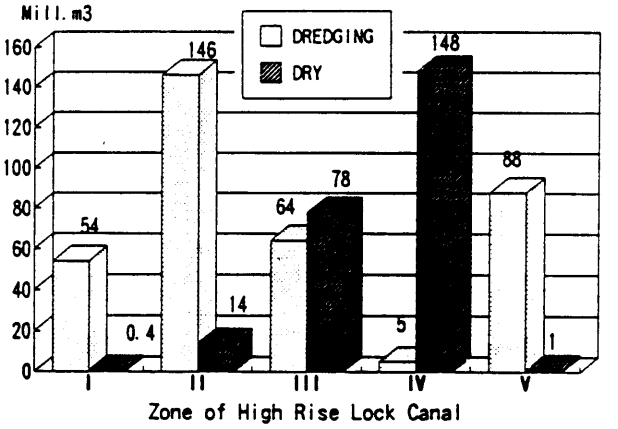


Fig. S.2-4 Excavation Volume for Each Zone (85' and 200,000 DWT Case)

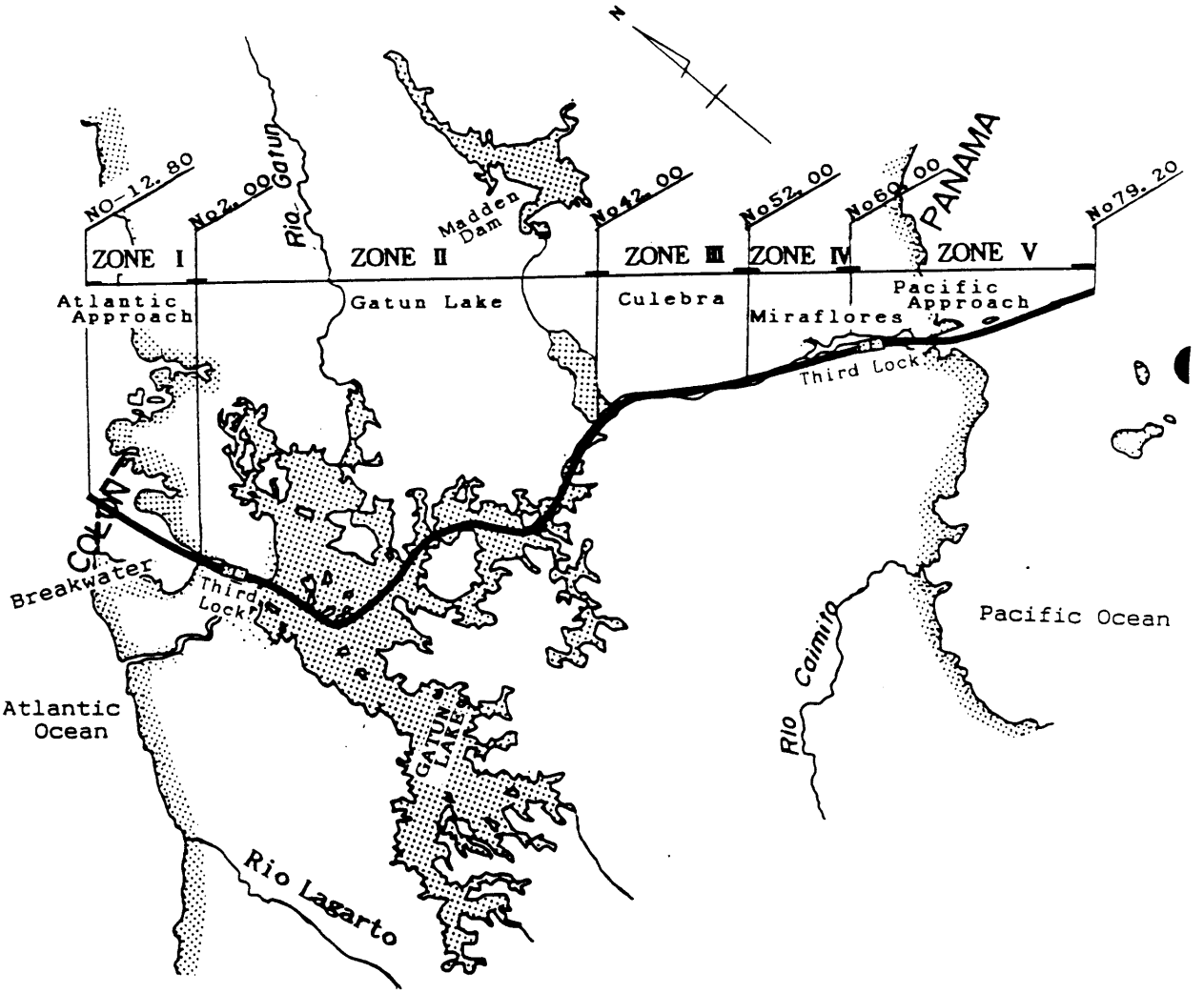


Fig. S.2-5 Zone Map of High Rise Lock Canal

2.4 Lock Structures and Mechanical Equipment

2.4.1 Lock Structures (Concrete) and Incidental Facilities

1) Lock Locations

Fig. S.2-6 and Fig. S.2-7 show the planned lock locations on the Pacific side and Atlantic side respectively. The location of the new high rise lock on the Pacific side has been slightly moved westward from the previous location suggested in Phase I because of the partial alteration of the canal alignment to enable the smoother navigation of vessels. As a result, the location of the new lock is a distance of some 1km from the existing lock. This separation is preferred on safety grounds as the canal side water level of the high rise lock is 85' which is much higher than the water level of Miraflores lake of 55'. A closure bank is introduced (as shown in Fig. S.2-6) next to the new lock to prevent the overflow of water from the new canal side to the existing lock side.

2) Surge Basin

Hydro-dynamic impact caused by the surging phenomenon may occur on the Pacific side when the canal water pours into the lock chamber in a relatively short period of time. To reduce the effects of such surge on vessels, a surge basin in proposed in the area of the Rio Cocoli valley of the new Pacific side lock using the natural topography as much as possible. A surge basin is not required on the Atlantic side since Gatun lake is immediately adjacent to the upstream end of the new locks.

3) Recycling Basin and Pump Station

The introduction of a recycling pond is proposed for the year 2060 on both the Pacific and Atlantic sides to store the spent lockage water (mainly fresh water) for subsequent recycling by pumping. A pump station with a capacity of $60m^3$/sec will also be provided on both ocean sides between the recycling basin and surge basin.

4) Lock Structure

All locks and approach walls will be the gravity type with varying cross-sections to meet the stability requirements and to incorporate a hydraulic tunnel system. The seismic coefficients used are Kh=0.1 and Kv=2/3Kh. Fig. S.2-8 shows the typical cross-section of the area around the middle operating gate.

5) Fourth Lane of Locks

Initially, just one lane of new, large locks is required at Gatun and Miraflores, however the excavation for the fourth lane of locks (ie. a second lane of larger locks) is included and costed as part of the initial works for 2020.
The fourth lane of locks may be required by or after 2060 and construction costs for the fourth lane have been included in Appendix 4.

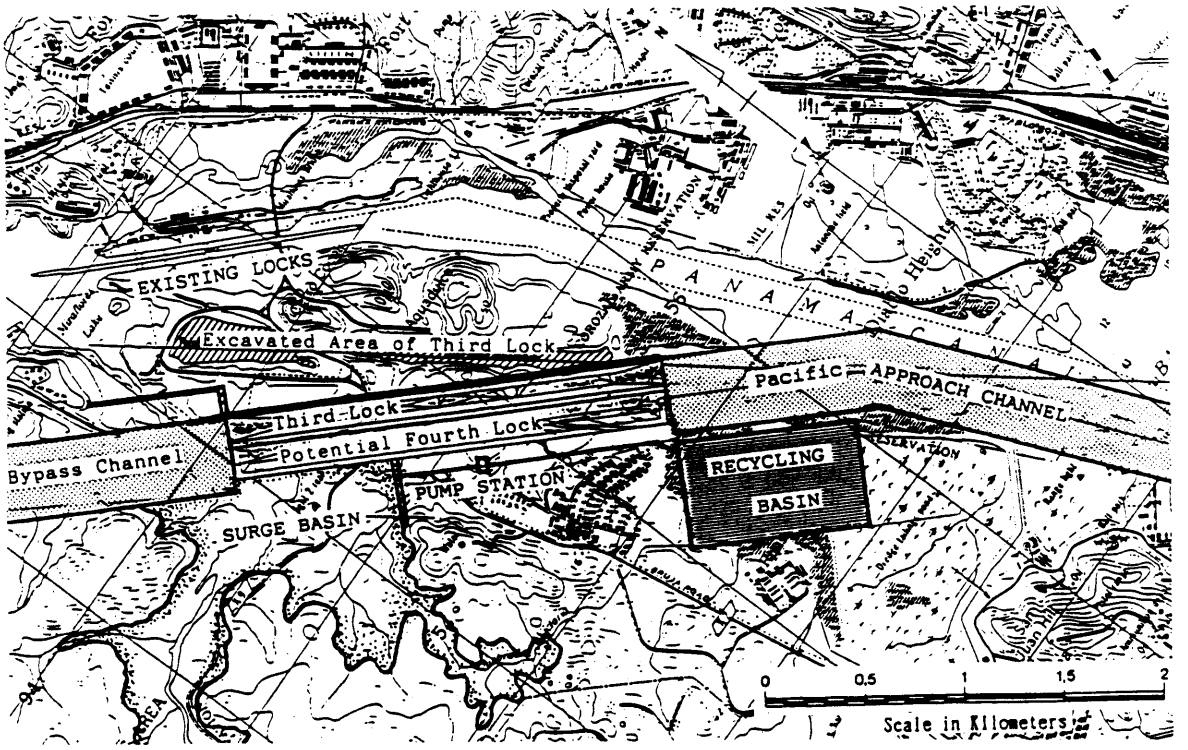


Fig. S.2-6 New Lock Layout Plan (Pacific Side)

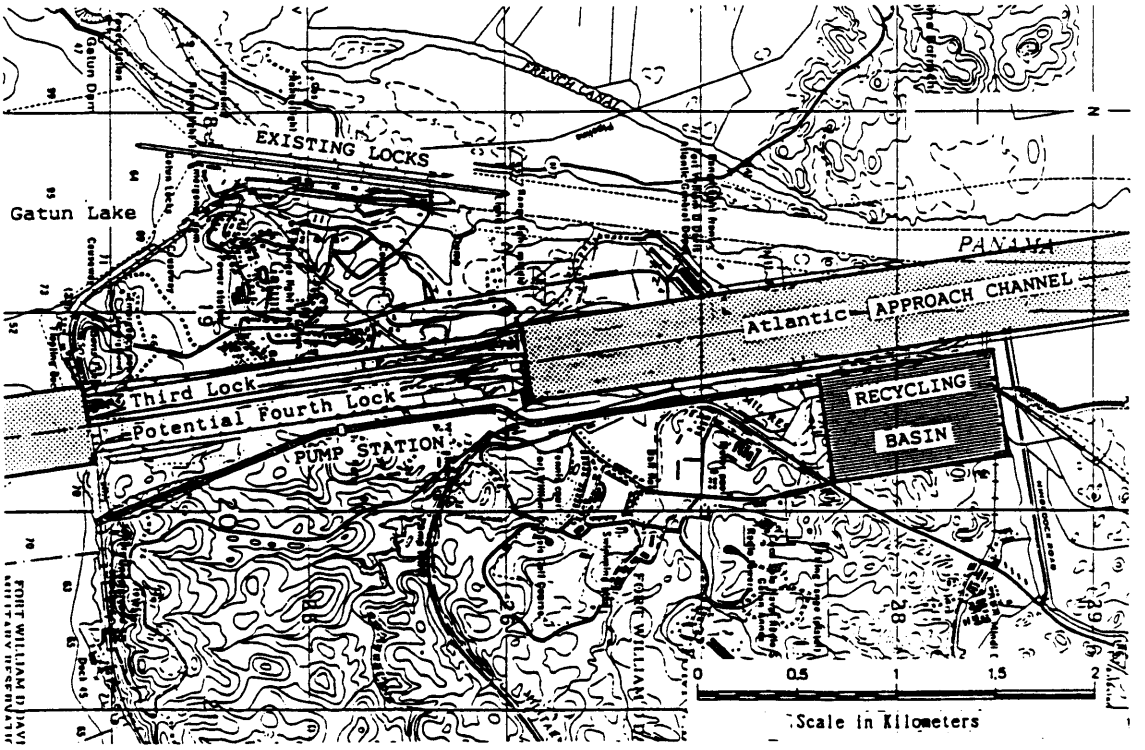


Fig. S.2-7 New Lock Layout Plan (Atlantic Side)

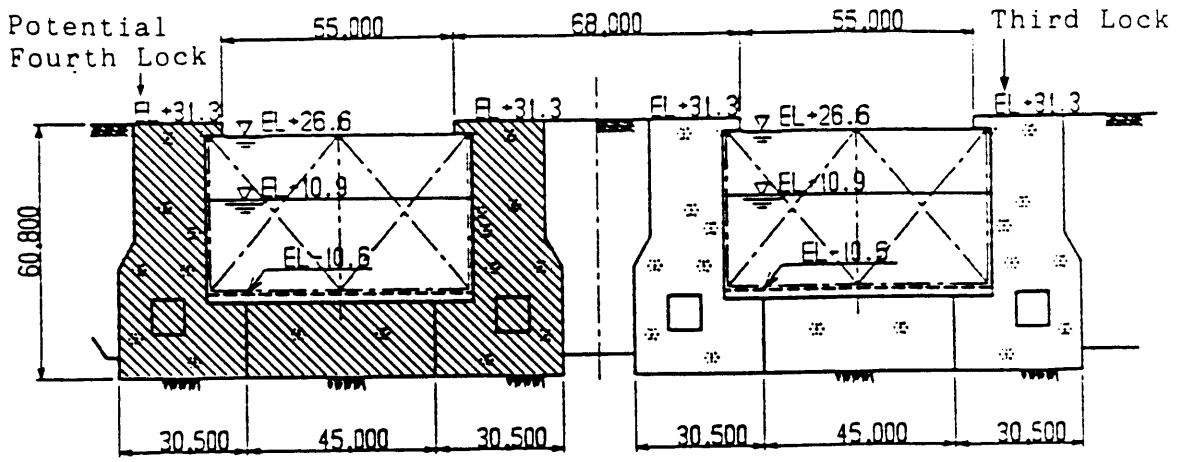


Fig. S.2-8 Typical Cross Section at middle operating Gate (200,000 DWT)

2.4.2 Gate Leaves

1) Description

The proposed gate type is the Miter type gate which was examined in Phase 1. Particular emphasis during the design stage of the main gate was put on the allowable stress for fatigue. Compared to the Phase 1 design, the number of gate operations has been reduced with the effect of increasing the allowable stress for fatigue, in turn lightening the gate weight. The assumed time required to open or close the gate is five minutes and the gate operation mechanism has been designed on this basis.

2) Main Gate Dimensions

The dimensions of the main gate are given in Table S.2-7 and S.2-8.

Table S.2-7 Main Gate Dimensions (200,000 DWT Case)

Position of Gate	Pacific Side W(m) x H(m) x W(tons)	Atlantic Side W(m) x H(m) x W(tons)
Upper Operating Gate	55.0 x 23.6 x 2,000	55.0 x 23.6 x 1,800
Middle Operating Gate	55.0 x 37.8 x 4,200	55.0 x 36.3 x 3,900
Lower Operating Gate	55.0 x 40.6 x 4,100	55.0 x 36.0 x 3,300

Table S.2-8 Main Gate Dimensions (150,000 DWT Case)

Position of Gate	Pacific Side W(m) x H(m) x W(tons)	Atlantic Side W(m) x H(m) x W(tons)
Upper Operating Gate	50.0 x 22.1 x 1,650	50.0 x 22.1 x 1,650
Middle Operating Gate	50.0 x 36.3 x 3,400	50.0 x 34.8 x 3,150
Lower Operating Gate	50.0 x 39.1 x 3,400	50.0 x 34.5 x 2,800

Note: .1) The gate weight indicates the weight of a pair of gate leaves and includes neither the weight of the gate frame which is built in to the supporting concrete structure nor the weight of the gate operation mechanism.

2) W (m) : gate width in meters
H (m) : gate height in meters
W (tons) : gate weight in tons

2.4.3 Mules

A total of 20 mules with a traction power of 64 tons will be used for both the Pacific side and Atlantic side of the new locks (totalling 40 mules). The specifications and number to be installed are the same for both the 200,000 DWT case and 150,000 DWT case.

2.5 Incidental Structures

2.5.1 Breakwater

A breakwater is currently provided on the Atlantic side to protect the approach channel to the canal and this breakwater can be used for the new High Rise Lock Canal. On the Pacific side, three islands, i.e. Naos, Perico and Falmenco, and the road connecting these islands act as a breakwater protecting the approach channel. As a result, no new breakwaters are required.

2.5.2 Berthing Facility for Small Boats

The construction of piers for the mooring of small boats, including tug boats to assist the navigation of passing vessels, is planned. These 50m long and 10m wide jetties will have a pile foundation with concrete slabs. A total of seven such piers will be constructed along the canal, including the approach channels.

2.5.3 Anchorage Area

Vessels wishing to use the canal must sometimes anchor offshore while awaiting permission to navigate the canal. The introduction of an anchorage area on the ocean side of the approach channel is necessary for the new canal as in the case of the existing canal. The planned area is sufficiently deep and no dredging is required. As the vessels use their own anchors, there is no need for the construction of new anchorage facilities.

2.5.4 Transportation Plan

With the construction of the new canal, the existing Bridge of the Americas on the Pan American Highway must be replaced by a new bridge to allow a wider approach channel and an increased bridge height for the passage of large vessels. Four canal crossing points, i.e. replacement of the Bridge of the Americas and three further crossings at Van Dam Bypass, Gamboa and Atlantic coast area, are proposed in the present plan for the widening of the canal and the construction of new high rise lock facilities. The approximate locations of these crossing points are shown in Fig. S.2-9. For the year 2020, it has been agreed that only the replacement of the Bridge of the Americas (Pacific side) and the provision of a new bridge on the Atlantic side will be costed. Addition of the other two bridges is included for the year 2060. Realignment

of some existing roads and the construction of new work roads are also required as part of the new high rise lock construction work. The construction cost for these bridges and roads, etc. is included in the estimated cost of the High Rise Lock Canal.

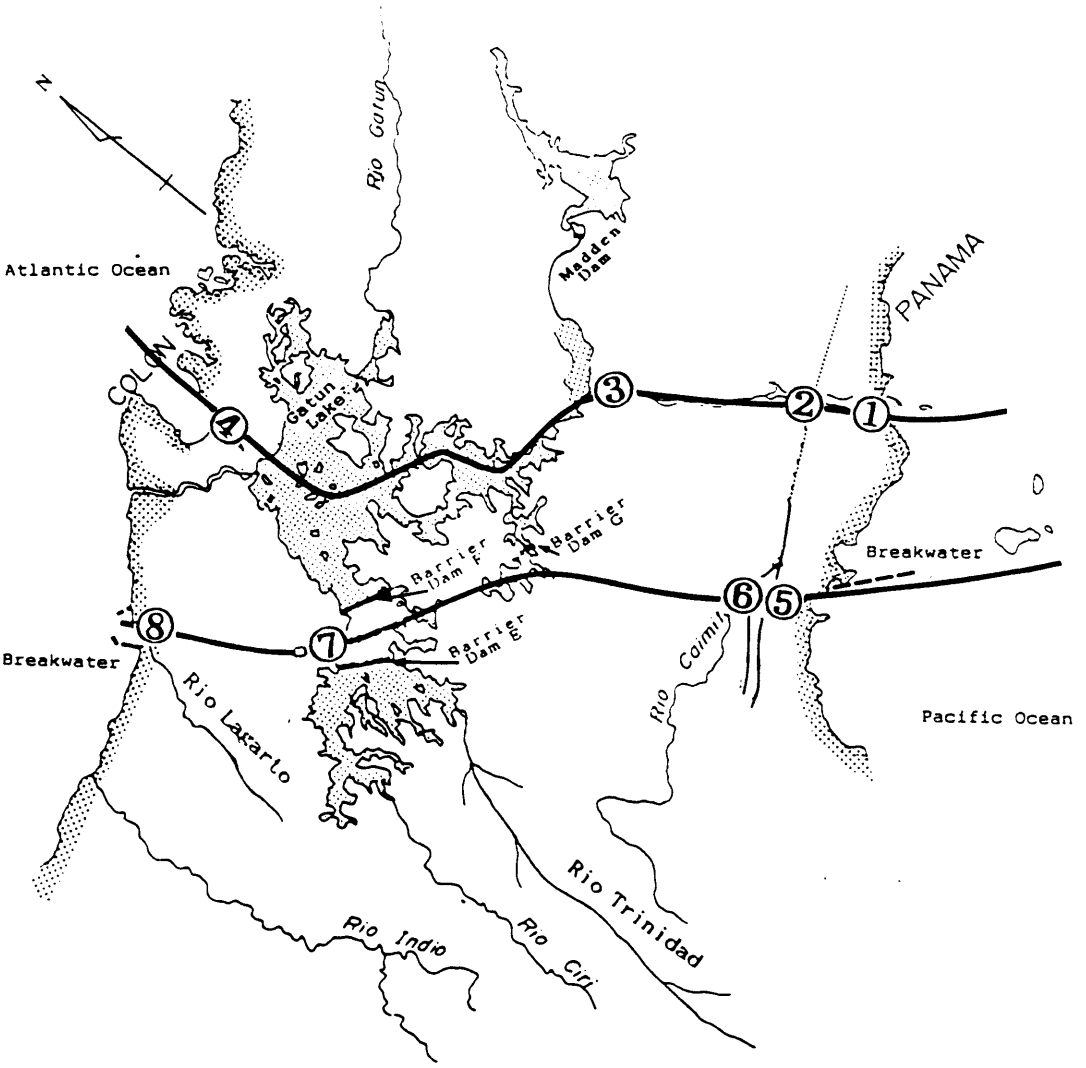


Fig. S.2-9 Road Crossing Location

For High Rise Lock Canal, road crossings at locations ① to ④ shown above are planned. For Sea Level Canal, road crossing at locations ② to ⑧ shown above are planned.

3. SEA LEVEL CANAL
(Part IV. Section 1. Engineering and Preliminary Design)

The Sea Level Canal is designed to accommodate 250,000 DWT class vessels. The case variations include tidal locks on the Pacific side, a tidal gate on the Atlantic side and combinations of these. The configurations of these different cases are shown in Fig. S.1-3. The Layout of the Sea Level Canal is shown in Fig. S.1-2.

3.1 General

General engineering conditions used in Engineering and Preliminary Design are summarized in the Summation Table (1) attached in the last page. The schematic layout of the Sea Level Canal is shown in Fig. S.1-2.

There is a possibility that the Sea Level Canal could cause an environmental problem on a large scale because of the movement of seawater between the Pacific and Atlantic. The introduction of a freshwater barrier has been examined as a method, of solving this potential problem. The study also examines the possible volume of freshwater to be poured into the canal and by what method, but the likely effects of this have not yet been studied. A more detailed examination is, therefore, required in the future prior to the official acceptance of the freshwater barrier.

3.2 Water Management Plan

3.2.1 Available Lockage Water for Existing Canal

The main water management item in the case of the Sea Level Canal is examination of the available water volume for the existing locks in view of the reduced size of Gatun lake (due to the construction of a barrier dam) upon completion of a Sea Level Canal. Upper Gatun Dam is planned to supply water to the reduced Gatun Lake as described in IV.1 Chapter 2 Water Balance.

3.2.2 Freshwater Barrier System

The freshwater barrier system proposed here to solve the problem of seawater movement between the two oceans involves the collection of rain, mainly from the watershed at the west side of the canal, and discharging as much freshwater as possible into the canal or tidal lock chambers to form a freshwater barrier to restrict the movement of seawater. However, it is questionable whether or not the movement of seawater between the two oceans will be successfully prevented by the freshwater barrier system alone. (Setting of the canal water level above HWL +3.6m on the Pacific side with the installation of tidal locks on both the Pacific and Atlantic sides could achieve the level of performance shown by the existing locks and may considerably reduce the possible adverse impact on the environment. Consideration of this option was not required as part of this study.)

3.2.3 Available Freshwater Volume

Only the available volume of water to be used by the freshwater barrier system and the water collection method are examined in the Report. The main analysis item is to determine the amount of freshwater which can be made available after meeting water requirements for the existing canal. In short, the amount of water available from reservoir (E) for the freshwater barrier is approximately 1,200 million m^3/year in an average year, plus about 700 million m^3/year from other rivers.

3.2.4 Freshwater Supply System

Freshwater for the creation of a freshwater barrier could be discharged either into the canal or into the tidal lock chambers.

1) Discharge into the Canal

Freshwater would be discharged into the canal via outlet structures from dams W-3-D, W-4-D, W-6-D and W-7-D (refer section 3.6) and from barrier dam (E). Water from other rivers such as Rio Pescado, Rio Martin Sanches, and Rio Lagarto (East) would also be discharged into the canal.
It would also be possible to discharge water directly into the tidal lock chambers as described below.

2) Discharge into Tidal Lock Chambers

The discharge into the canal could be supplemented by the discharge of freshwater directly into the tidal lock chambers. This method would probably not be very effective on its own but may be an effective supplementary measure to the discharge into the canal described in 1) above.

3.3 Canal Excavation Plan

3.3.1 Description

The excavation volume was calculated for the three different cases, i.e. (a), (b) and (c), using topographical maps and sea charts (scale: 1/25,000). Five geological types were introduced to represent the soil and rock characteristics of each zone. (See Fig. S.3-3)

3.3.2 Calculation Results

The excavation volume was separately calculated for dry excavation and dredging, and the results are given in Table S.3-1. The excavation volume for each case and the excavation volume for each zone are shown in Fig. S.3-1 and Fig. S.3-2 respectively. Both the dry excavation and dredging conditions are further classified in terms of the geological characteristics, i.e. soil, soft rock and hard rock, etc. Please refer to the main report for further details.

Table S.3-1 Excavation Volume

Case	Ship Size (DWT)	Dredging Volume (million m^3)	Dry Excavation Volume (million m^3)	Total Excavation Volume (million m^3)
Case (a)	250,000	304	948	1,252
Case (b)	250,000	351	1,034	1,385
Case (c)	250,000	304	948	1,252

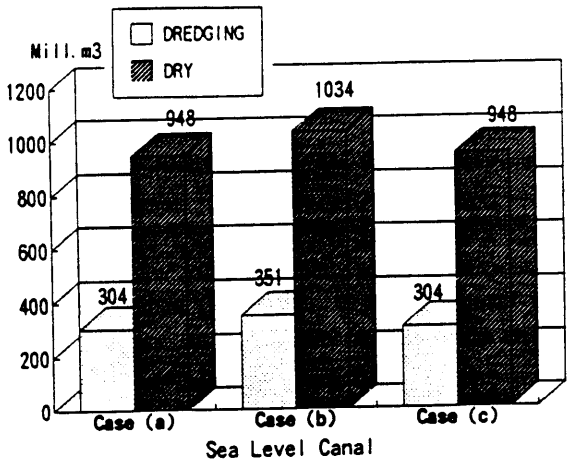


Fig. S.3-1 Excavation Volume for Each Case

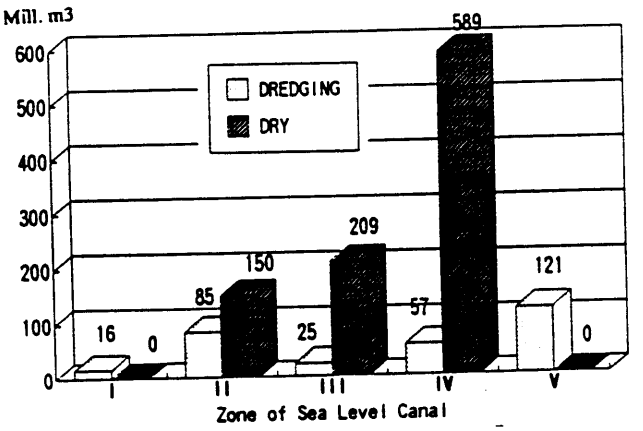


Fig. S.3-2 Excavation Volume for Each Zone (Case a)

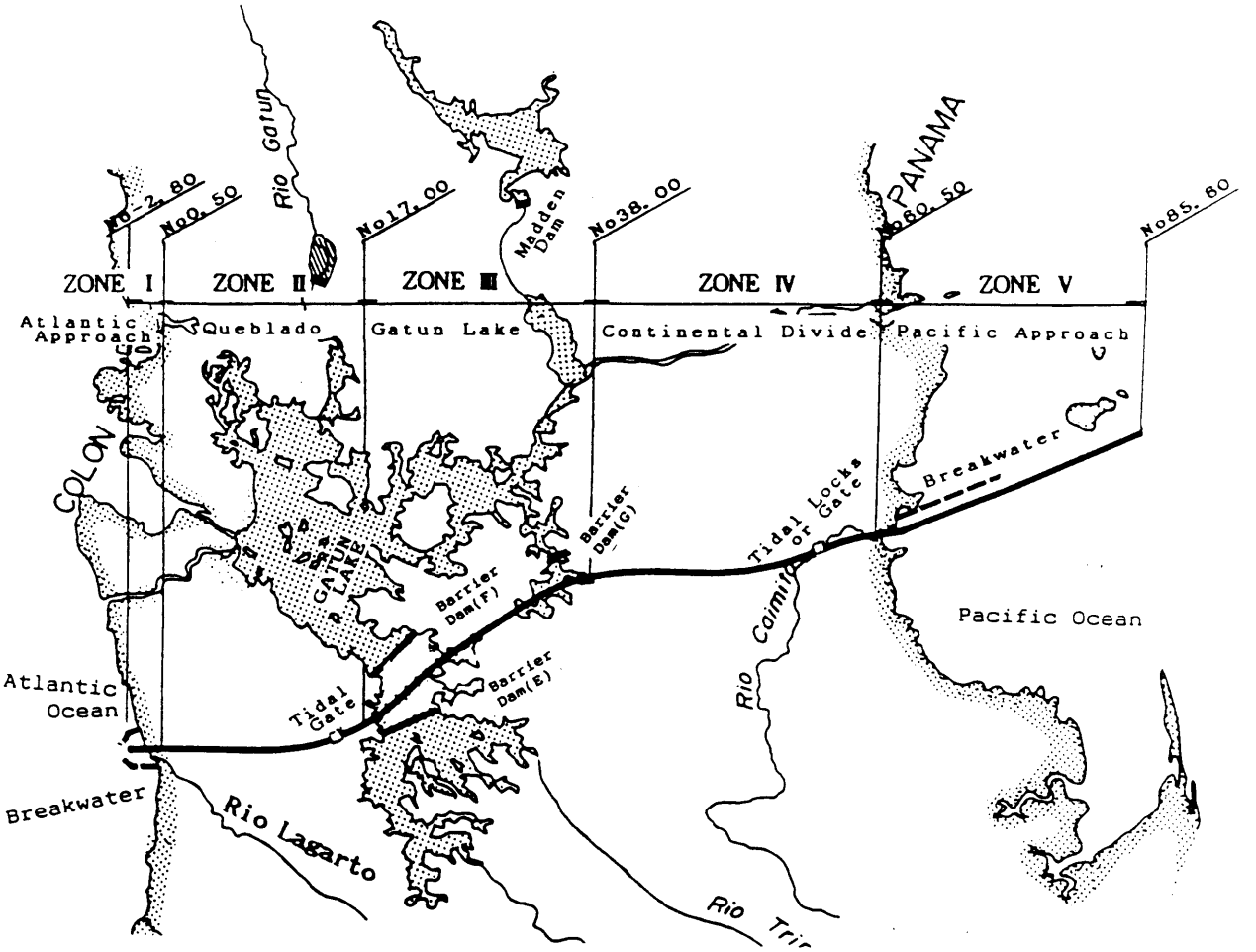


Fig. S.3-3 Zone Map of Sea Level Canal

3.4 Tidal Locks and Tidal Gates

3.4.1 Study Cases

For the Sea Level Canal, three cases were studied, i.e. (a) the introduction of tidal locks at only the canal entrance on the Pacific side, (b) the introduction of tidal gates at both the Pacific and Atlantic sides and (c) the combination of tidal locks and tidal gate.

In Case (a), the tidal locks are located 5km inland from the river mouth of Caimito river on the Pacific side and no tidal lock is constructed on the Atlantic side. As shown in Fig. S.3–4, three–lane tidal locks will be constructed by 2020 and an additional lane will be constructed by 2060, resulting in a total of four lanes.

In Case (b), tidal gates are located on both the Pacific and Atlantic sides. The location of the Pacific side tidal gates is the same as that of the tidal locks in Case (a) while the location of that on the Atlantic side is some 15km inland from the coast, near Escobal village and the existing Gatun Lake shoreline.

In Case (c), the tidal locks are constructed in the same position as in Case (a) while the tidal gate is constructed near Lagarterita village on the Atlantic side, about 24 km from the Atlantic shoreline.

3.4.2 Structures

1) Tidal Locks of Case (a) and Case (c)

The tidal lock structure will consist of a recess for torsion–type sliding gates, a lock chamber wall and guide wall. As mentioned earlier, three lanes will be constructed for 2020 and an additional lane will be constructed for 2060 to complete the 4–lane lock. The layout plan for the tidal locks is shown in Fig. S.3–4.

All structures and approach walls will be the gravity type with varying cross–sections to meet the stability requirements. The seismic coefficients to be applied are Kh=0.1 and Kv=2/3 x Kh. Table S.3–2 gives the main dimensions of the tidal lock structures and the concrete volume.

2) Tidal Gate of Case (b) and Case (c)

The tidal gate structures will consist of a recess for a triangular type rolling gate and abutment. All structures will be made of concrete. The plan for the tidal gate is given in Fig. S.3–5 while Table S.3–2 gives the main dimensions of the tidal gate structures and the required concrete volume.

Table S.3-2 Dimensions and Concrete Volume of Tidal Structures (2020)

	Dimensions (m) L	W	D	No.	Concrete Volume (m^3)
Case (a) Tidal Lock	430	60	23	3	6,000,000
Case (b) Tidal Gate	-	190	23	2	1,960,000
Case (c) Tidal Lock	430	60	23	3	6,980,000
Tidal Gate	-	190	23	1	

3.4.3 Gates

The proposed gate type for the tidal lock is the torsion type sliding gate. In view of possible emergencies, the provision of two operating gates and two emergency gates, i.e. two each at the Pacific approach channel side and canal side of the lock chamber, is proposed. Particular emphasis was put on the allowable stress for fatigue during the design stage. The assumed time required to open or close the gates is five minutes and the gate operation mechanism has been designed on this basis.

In the case of tidal gates, the triangular-type rolling gate is proposed. Two gates are planned at each tidal gate site to allow the emergency use of an extra gate. The assumed time required to open or close the gates is twenty minutes. The main dimensions of the tidal gate are given in Table S.3-3.

Table S.3-3 Main Gate Dimensions (250,000 DWT Vessel)

Structure	Dimensions W (m) x H (m)		No. per Site	Weight (tons)
Operating Gate for Tidal Lock	Sea side	60 x 30.6	4	1,650
	Canal side	60 x 27.5		1,450
Tidal Gate	190 x 27.5		2	8,500

Notes: 1) The gate weight indicates the weight of the a gate leaf and includes neither the weight of the gate frame which is built into the supporting concrete structure nor the weight of the gate operation mechanism.

2) W (m) : gate width in meters
H (m) : gate height in meters

3.4.4 Mules

A total of ten mules with a traction power of 64 tons each will be used for each tidal lock (40 units for four tidal locks). The specifications are the same as those to be used for the new high rise lock.

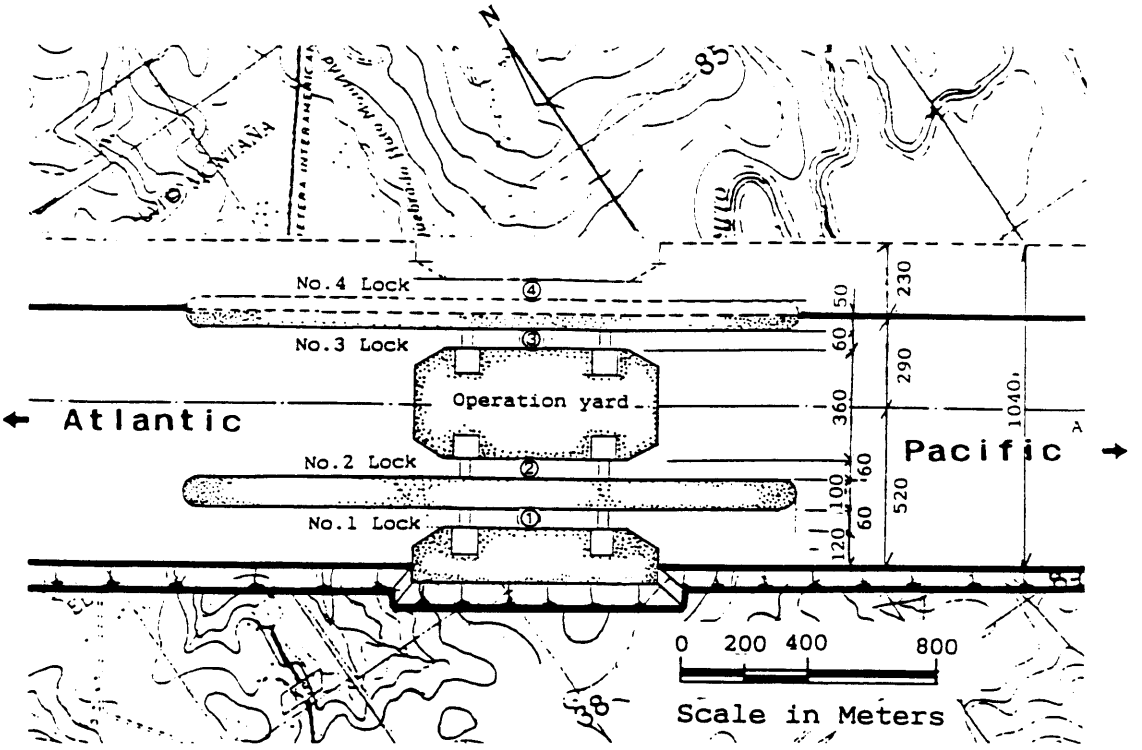


Fig. S.3-4 Tidal Lock Layout

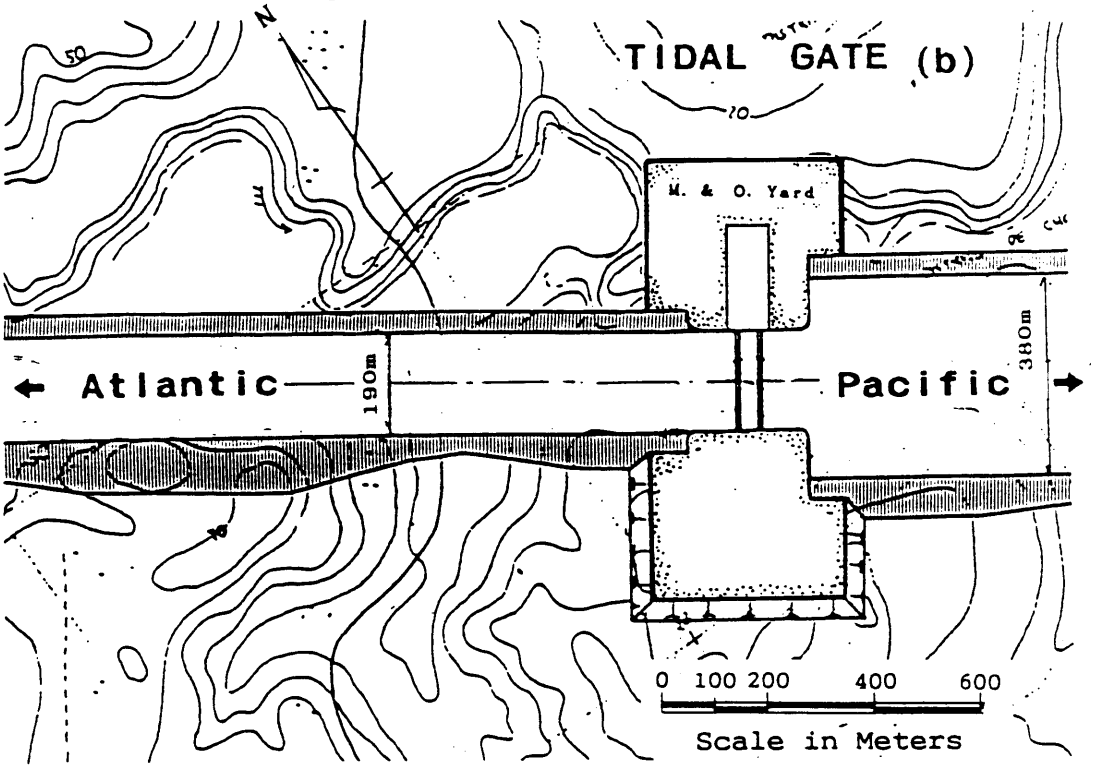


Fig. S.3-5 Tidal Gate Layout

3.5 Barrier Dams

3.5.1 Description

The main barrier dams are two almost parallel dams to be constructed in Gatun lake. These dams will ensure the present water level of Gatun lake while allowing the water level between the barrier dams to be lowered to the seawater level. The Sea Level Canal passing between these two dams can be constructed with minimal adverse effects on Gatun lake. The barrier dams will start near Escobal village at the southern tip of Gatun lake and end near Cano Gigante on the opposite side of the lake. Apart from these main barrier dams, another barrier dam is planned on the opposite coast of La Laguna village to bring the total number of barrier dams to three. Fig. S.3-6 shows the location of the barrier dams.

3.5.2 Barrier Dam Structure

All the barrier dams to be constructed will be the rock fill type. Structurally, each will consist of a core, filter and armour layer to ensure proper and stable functioning. The materials to be used are clayey soil, sand and 500-600 kg rocks for the core, filter and armour layer respectively. The foundation depth is designed to be some 10m lower than the lake bottom elevation to ensure dam stability. Instrumentation to measure subsidence will be attached to each dam to check the occurrence of dam subsidence.

3.5.3 Barrier Dam Dimensions

The main dimensions of the barrier dams are given in Table S.3-4, and Fig. S.3-7 shows a typical section of the barrier dam.

Table S.3-4 Barrier Dam Dimensions

Name	Dam Height	Crest Level	Crest Length	Volume (million m^3)
E	30 m	EL 32 m	4,200 m	13.0
F	32 m	EL 32 m	5,100 m	14.2
G	25 m	EL 32 m	900 m	2.0

3.5.4 Reservoir

An area to the west of the main barrier dam will be separated from Gatun lake to form a reservoir (E) which will be used as a freshwater supply source for the Freshwater barrier. The water level of the reservoir will be 26.6m above sea level which is the same as that of Gatun lake. The reservoir surface area will be about 80km^2 and the storage volume will be approximately 500 million m^3. The watershed serving this reservoir has an area of 685km^2 and rainwater will mainly flow into the reservoir via Rio Ciri and Rio Trinidad. The freshwater of the reservoir will flow down into the canal via a freshwater conveyance structure ordinarily. The freshwater volume from the reservoir will be 1,159 million m^3 in an average year. Surcharge water during a flood will be discharged to the Atlantic via the spillway and Rio Lagarto (see Fig. S.3-6).

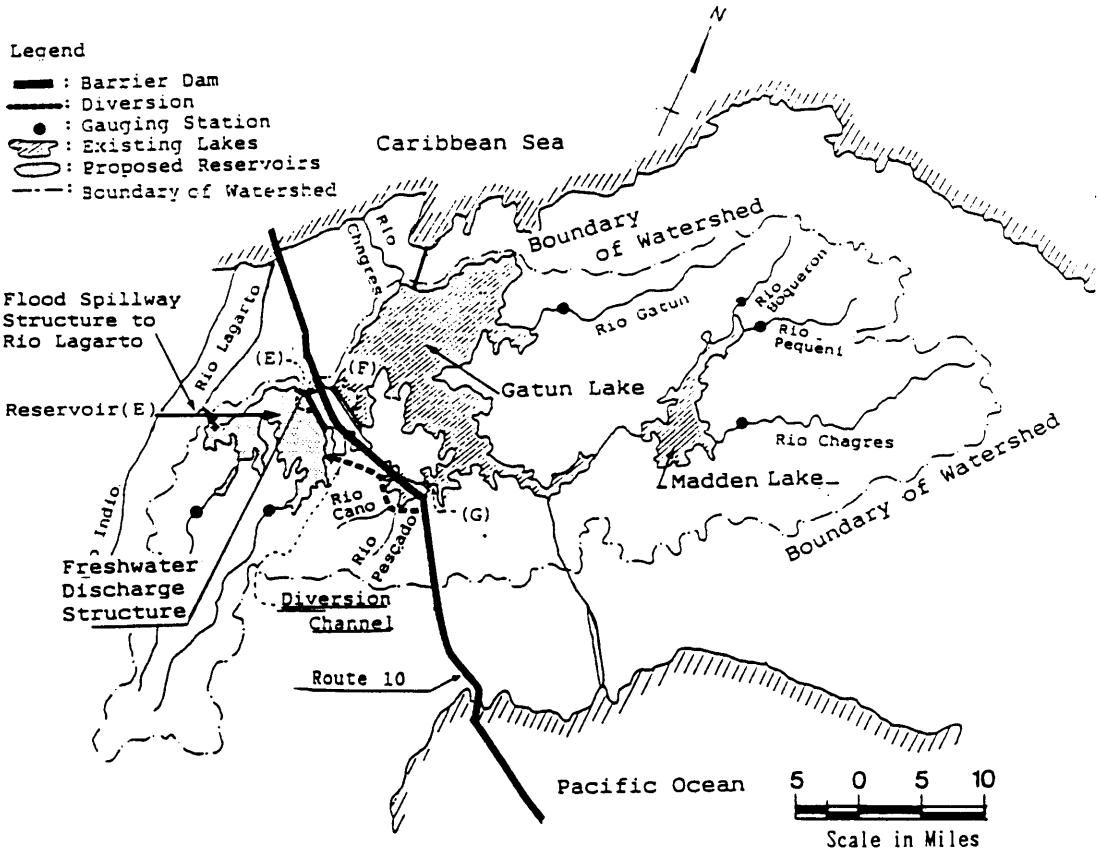


Fig. S.3-6 Barrier Dam Locations

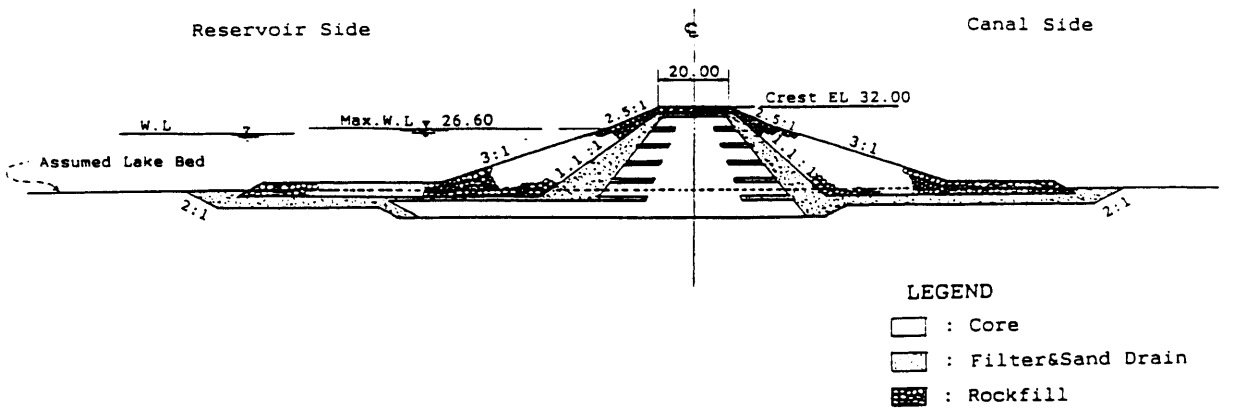


Fig. S.3-7 Typical Section of Barrier Dam

3.6 Diversion Scheme

3.6.1 Description

In view of the requirement to prevent flood water flowing into the canal during construction and to inject a huge amount of water into the canal or tidal lock chamber as freshwater, it is necessary to temporarily store freshwater from rivers in a reservoir and other facilities so that a regular supply of freshwater is available for discharge into the canal for the barrier system. Reservoir (E) is proposed for this purpose. Freshwater is obtainable from Rio Trinidad and Rio Ciri, both of which directly flow into Reservoir (E). In addition, the use of Rio Los Hules, Rio Quebrada Chica and Rio Cano is proposed to provide water for Reservoir (E) via diversion channels.

Rio Caimito, the watershed of which differs from the Gatun watershed, also has a large runoff which can be used for the freshwater barrier system. As it is difficult to guide water from the proposed Rio Caimito dam to Reservoir (E) due to topographical reasons, water from Rio Caimito will be discharged into the canal or tidal lock chambers independently of reservoir (E). Rio Lagarto (East) will be dammed near the Atlantic Ocean and a diversion channel constructed to divert water to the canal. During major floods however the water would be diverted to the Atlantic Ocean outside the Atlantic breakwaters. Water from other rivers (Rio Lagarto East, Rio Pescado and Rio Martin Sanches) will be discharged directly into the canal. Major rivers flowing into the Sea Level Canal are listed below for reference purposes.

1) Rio Lagarto (East)
2) Rio Lagarto (West)
3) Rio Trinidad
4) Rio Ciri
5) Rio Los Hules
6) Rio Quebrada Chica
7) Rio Cano
8) Rio Pescado
9) Rio Caimito (East)
10) Rio Caimito (West)
11) Rio Martin Sanchez

3.6.2 Diversion Channels and Cofferdams

The locations of rivers listed in 3.6.1 are shown in Fig. S.3–8 while the improvement measures for each river are given in Table S.3–5.

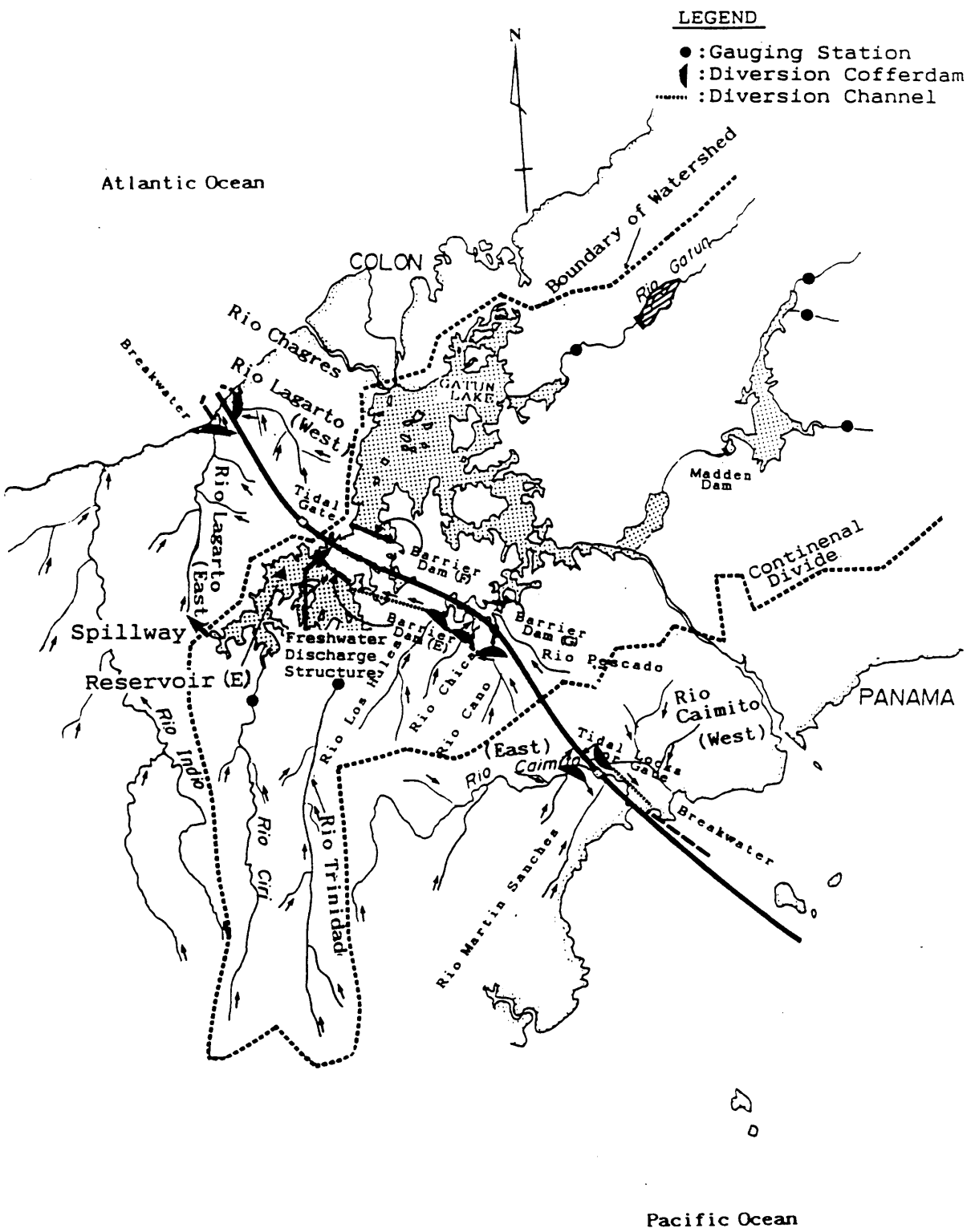


Fig. S.3-8 Diversion Scheme Layout

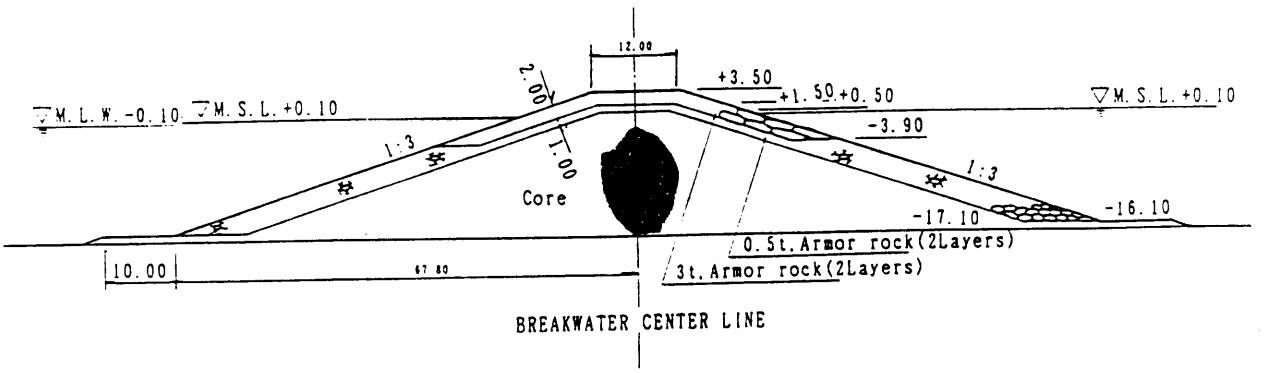
Table S.3-5 River Improvement Measures

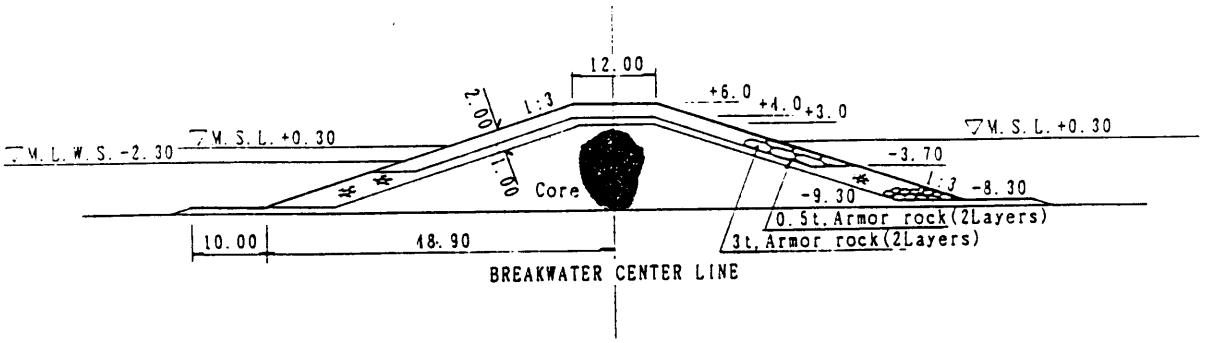
Name of River	Description	Dam	Channel
Rio Lagarto (East)	discharge directly to canal	-	-
Rio Lagarto (West)	dam and diversion channel	W-1-D	W-1-C
Rio Trinidad	directly to reservoir (E)	-	-
Rio Ciri	directly to reservoir (E)	-	-
Rio Los Hulos	dam and diversion channel	W-3-D	W-3-C
Rio Quebrada Chica	dam and diversion channel	W-4-D	W-4-C
Rio Cano	dam and diversion channel	W-6-D	W-5-C
Rio Pescado	discharge directly to canal	-	-
Rio Caimito (East)	dam and diversion channel	E-2-D	E-2-C
Rio Caimito (West)	dam and diversion channel	W-7-D	E-2-C
Rio Martin Sanchez	discharge directly to canal	-	-

3.7 Incidental Structures

3.7.1 Breakwaters

New breakwaters are proposed for the Sea Level Canal on both the Pacific and Atlantic sides. On the Pacific side, a breakwater on the eastside only is planned, as is provided for the existing canal near Panama City. The proposed slope gradient is 1:3, Fig. S.3-9 shows the typical section of the breakwaters.





Note 1: All dimensions are in meters.
2: Datum for all levels is P.L.D.(meters)

Fig. S.3-9 Typical Section of Breakwaters

3.7.2 Jetty for Small Boats

The construction of piers for the mooring of small boats, including tug boats to assist the navigation of passing vessels, is planned. These 50m long and 10m wide jetties will have a pile foundation with concrete slabs. A total of six such piers will be constructed along the canal, including the approach channels.

3.7.3 Anchorage Area

Vessels wishing to use the canal must sometimes anchor offshore while awaiting permission to navigate the canal. The introduction of an anchorage area on the ocean side of the approach channel is necessary for the new canal as in the case of the existing canal. The planned area is sufficiently deep and no dredging is required. As the vessels use their own anchors, there is no need for the construction of new anchorage facilities.

3.7.4 Transportation Plan

With the construction of the Sea Level Canal, the construction of new bridges/tunnels is essential to link the east and west sides of the canal. Four canal crossings are proposed, at the Pan American Highway, autopista, near Escobal and at the Atlantic coast area. The total number of crossings will be seven for the Sea Level Canal as three additional crossings will be provided over the existing canal. The cost of the construction of these bridges, realignment of existing roads and the construction of new roads, is included in the estimated cost for the Sea Level Canal.

The locations of the 7 proposed road crossings are ② to ⑧ shown in Fig. S.2-9.

For the year 2020, it has been agreed that three bridges over the Sea Level Canal, i.e. the Atlantic bridge, Escobal bridge, and one bridge on the Pacific side, will be costed. The remaining bridge over the Sea Level Canal and the three bridges over the existing canal are included in the costs for the year 2060.

4. MEMBERS OF THE STUDY TEAM FOR PHASE 2 STUDY

The Study Team consisted of the following 19 members;

Assignment	Name	Organisation
Project Manager	Dr. Akira Ishido	YEC
Structural Engineer (1)	Mr. Shuji Sekigushi	YEC
Structural Engineer (2)	Mr. Masashi Bessho	YEC
Geologist	Mr. Nobuhiko Uchiseto	YEC
Geotechnical Engineer	Mr. Murao Toyama	YEC
Civil/Hydraulics Engineer	Mr. Toshinobu Nakano	SANYU C.
Mechanical Engineer (1)	Mr. Hiroshi Terata	MHI
Mechanical Engineer (2)	Mr. Toshio Aketa	MHI
Waterways Specialist	Mr. John Read	MAUNSELL
Safety Operation Specialist	Mr. Takayuki Fukui	JAPMA
Port/Harbor Engineer	Mr. Satoru Nishino	YEC
Highway Specialist	Mr. Toshihiro Hotta	YEC
Transport Engineer	Mr. John Hamilton	YEC
Blasting Specialist	Mr. Lewis L Oriard	PBI
Construction Engineer	Mr. Masahiko Hatayama	SEIWA
Design/Quantities Engineer	Mr. Masaaki Naito	SEIWA
Cost Engineer (1)	Mr. Jun Iwasawa	SEIWA
Cost Engineer (2)	Mr. Koichi Nagayoshi	YEC
Electrical/Control Engineer	Mr. Noritune Chiba	YEC

YEC : Yachiyo Engineering Co., Ltd.
SANYU C. : Sanyu Consultants Inc.
MHI : Mitsubishi Heavy Industrial Co., Ltd.
MAUNSELL : Maunsell & Partners (UK)
JAPMA : Japan Association for Preventing Marine Accidents
PBI : Parsons Brinkerhoff Pty. Ltd.
SEIWA : Seiwa Consultants Co., Ltd.

Comparative Summation Table (1)

File :sum

ITEMS	Unit	HIGH RISE LOCK ALTERNATIVES								SEALEVEL CANAL ALTERNATIVES					
		Case (1)		Case (2)		Case (3)		Case (4)		Case (a)		Case (b)		Case (c)	
		2020	2060	2020	2060	2020	2060	2020	2060	2020	2060	2020	2060	2020	2060
A. FEATURES OF ALTERNATIVES															
1. Design Ship	1000DWT	150	150	200	200	150	150	200	200	250	250	250	250	250	250
2. Route		I	I	I	I	I	I	I	I	10	10	10	10	10	10
3. Lake Level	ft	85	85	85	85	90	90	90	90	-	-	-	-	-	-
4. Number of Lift	no.	2	2	2	2	2	2	2	2	-	-	-	-	-	-
5. Canal Width	m	320	320	350	350	320	320	350	350	190	190	190	190	190	190
6. Approach Channel Width	m	320	320	350	350	320	320	350	350	380	380	380	380	380	380
7. Total Route Length	Km	84.1	84.1	87.6	87.6	84.1	84.1	87.6	87.6	88.4	88.4	86	86	88.4	88.4
- Pacific Approach	Km	14.3	14.3	17.5	17.5	14.3	14.3	17.5	17.5	25.1	25.1	22.7	22.7	25.1	25.1
- Channel	Km	55.7	55.7	55.7	55.7	55.7	55.7	55.7	55.7	60	60	60	60	60	60
- Atlantic Approach	Km	14.1	14.1	14.4	14.4	14.1	14.1	14.4	14.4	3.3	3.3	3.3	3.3	3.3	3.3
8. Channel Depth	m	21	21	22	22	21	21	22	22	24	24	24	24	24	24
B. WATER MANAGEMENT															
1. Abailable Water															
a. Average year	Mill M3	5,647	5,653	5,647	5,653	5,611	5,653	5,613	5,617	4,331	4,333	4,331	4,337	4,331	4,333
b. Dry year	Mill M3	3,951	3,952	3,952	3,952	3,316	3,952	3,916	3,916	3,148	3,149	3,148	3,149	3,148	3,149
2. Water Consumption for Existing Canal															
a. Ship Transits	no.	16,200	20,000	16,200	20,000	16,200	20,000	16,200	20,000	16,200	19,400	16,200	19,400	16,200	19,400
b. Water Volume Required	Mill M3	2,867	3,540	2,867	3,540	3,033	3,744	3,033	3,744	2,867	3,434	2,867	3,434	2,867	3,434
3. Water Consumption for New Lock															
a. Ship Transits	no.	2,000	4,600	2,000	4,600	2,000	4,600	2,000	4,600	-	-	-	-	-	-
b. Water Volume Required	Mill M3	1,014	2,332	1,201	2,763	1,073	2,467	1,271	2,922	-	-	-	-	-	-
4. Total Water Consumption: 2.b + 3.b	Mill M3	3,881	5,872	4,068	6,303	4,105	6,211	4,303	6,666	2,867	3,434	2,867	3,434	2,867	3,434
5. Available Freshwater (as barrier)															
a. Freshwater from Gatun Watershed	Mill M3	-	-	-	-	-	-	-	-	1,159	1,159	1,159	1,159	1,159	1,159
b. Freshwater from Outside Gatun	Mill M3	-	-	-	-	-	-	-	-	737	737	737	737	737	737
6. Countermesures to be taken															
a. Upper Gatun Dam (as regulating)	Mill M3	-	-	-	-	-	-	-	-	-	285	-	285	-	285
b. Trinidad Dam (as regulating)	Mill M3	-	-	-	396	-	392	-	466	-	-	-	-	-	-
c. Ciri Dam (as regulating)	Mill M3	-	102	-	-	-	-	-	-	-	-	-	-	-	-
d. Indio Dam (as water source)	Mill M3	-	461	591	-	558	-	591	102	-	-	-	-	-	-
e. Pumping Station	Mill M3	-	1459	-	1890	-	1798	-	2289	-	-	-	-	-	-
C. MAIN ELEMENTS OF ALTERNATIVES															
1. Lock & Gate															
(1) High Rise Lock															
a. Lock Chamber (LxWxD)	note*1	(A)	-	(B)	-	(A)	-	(B)	-	-	-	-	-	-	-
b. Gates															
- Max. Operating Gate Dimensions (BxH)	m	50x39.1	-	55x40.6	-	50x39.5	-	55x41.0	-	-	-	-	-	-	-
- Operating Gate Total Number(in pair)	nos.	6	-	6	-	6	-	6	-	-	-	-	-	-	-
- Operating Gate Total Weight	1000 ton	15.8	-	19.2	-	16.1	-	19.7	-	-	-	-	-	-	-
- Culvert Gate Number	nos.	18	-	18	-	18	-	18	-	-	-	-	-	-	-
- Culvert Gate Dimensions (BxH)	m	7.5x8.1	-	8.2x8.4	-	7.7x8.3	-	8.4x8.6	-	-	-	-	-	-	-
- Maintenance Gate (weight x number)	ton	5600 x 1	-	6700x1	-	5600 x 1	-	6700x1	-	-	-	-	-	-	-
- Emergency Gate (weightx number)	ton	2500 x 2	-	2950x2	-	2500 x 2	-	2950x2	-	-	-	-	-	-	-
c. Number of Mules	nos.	40	-	40	-	40	-	40	-	-	-	-	-	-	-
(2) Tidal Lock															
a. Lock Chamber (LxWxD)	m	-	-	-	-	-	-	-	-	(c)	(c)	-	-	(c)	(c)
b. Gates		-	-	-	-	-	-	-	-			-	-		
- Operating Gate Dimensions (BxH)	m	-	-	-	-	-	-	-	-	60x30.6	60x30.6	-	-	60x30.6	60x30.6
- Operating Gate Total Number	nos.	-	-	-	-	-	-	-	-	6	2	-	-	6	2
- Operating Gate Total Weight	1000 ton	-	-	-	-	-	-	-	-	9.3	3.1	-	-	9.3	3.1
- Culvert Gate Number	nos.	-	-	-	-	-	-	-	-	12	4	-	-	12	4
- Culvert Gate Dimensions (BxH)	m	-	-	-	-	-	-	-	-	6.0x6.0	6.0x6.0	-	-	6.0x6.0	6.0x6.0
- Maintenance Gate Weight	1000 ton	-	-	-	-	-	-	-	-	22.2	7.4	-	-	22.2	7.4
c. Number of Mules	nos.	-	-	-	-	-	-	-	-	30	10	-	-	30	10
(3) Tidal Gate															
- Gate Dimensions (BxH)	m	-	-	-	-	-	-	-	-	-	-	190x27.5	-	190x25.0	-
- Gate Total Number	nos.	-	-	-	-	-	-	-	-	-	-	4	-	2	-
- Gate Total Weight	1000 ton	-	-	-	-	-	-	-	-	-	-	33.8	-	11.6	-
2. Barrier Dams (E),(F),(G)in Volume	mill M3	-	-	-	-	-	-	-	-	29	-	29	-	29	-
3. Canal Excavation															
- Dry Excavation	mill M3	217	-	242	-	205	-	229	-	948	-	1,034	-	948	-
- Dredging	mill M3	295	-	358	-	286	-	346	-	304	-	351	-	304	-
- Total Volume	mill M3	512	-	600	-	491	-	575	-	1,252	-	1,385	-	1,252	-
4. Diversion Scheme															
- Cofferdams	nos.	-	-	-	-	-	-	-	-	7	-	7	-	7	-
- Channels	km	-	-	-	-	-	-	-	-	39	-	39	-	39	-
5. Canal Crossing (Bridges)	place	2	2	2	2	2	2	2	2	3	4	3	4	3	4

Note*1: (A)=390x50x20, (B)=420x55x21.5, (C)=430x60x23

Comparative Summation Table (2)

I T E M S	Unit	HIGH RISE LOCK ALTERNATIVES								SEALEVEL CANAL ALTERNATIVES					
		Case (1)		Case (2)		Case (3)		Case (4)		Case (a)		Case (b)		Case (c)	
		2020	2060	2020	2060	2020	2060	2020	2060	2020	2060	2020	2060	2020	2060
Design Ship	1000DWT	150	150	200	200	150	150	200	200	250	250	250	250	250	250
Route		1	1	1	1	1	1	1	1	10	10	10	10	10	10
Lake Level	ft	85	85	85	85	90	90	90	90	-	-	-	-	-	-
6. New Roads	km	32		32		32		32		71		71		71	
7. Breakwater	mill. m3									18		18		18	
8. Navigation Support System															
a. Jetty for Small Boats	nos.	8	-	8	-	8	-	8	-	7	-	5	-	7	-
b. Berthing Dolphin	nos.	8	-	8	-	8	-	8	-	8	-	8	-	8	-
c. Navigation Bouy	set	74	-	74	-	74	-	74	-	78	-	78	-	78	-
d. Canal Lighting	set	60	-	60	-	60	-	60	-	786	-	786	-	786	-
e. Tug Boats															
- 6000 HP	nos.	23	-	23	-	23	-	23	-	16	-	10	-	16	-
- 3000 HP	nos.	2	-	2	-	2	-	2	-	9	-	7	-	9	-
D. CONSTRUCTION COST															
1. Canal Excavation															
-Dry Excavation	mill US$	719	-	802	-	687	-	766		3,198	-	3,499	-	3,198	-
-Dredging	mill US$	1,851	-	2,171	-	1,768	-	2,077		2,051	-	2,721	-	2,051	-
-Slope Treatment etc.	mill US$	442	-	504	-	422	-	482		995	-	1,153	-	995	-
Total Canal Excavation	mill US$	3,012	-	3,477	-	2,877	-	3,325	0	6,244	-	7,373	-	6,244	-
2. Placing Area	mill US$	88	-	88	-	83	-	83	-	189	-	189	-	189	-
3. Lock & T. Gate structure	mill US$	991	-	1,127	-	991	-	1,127	-	2,037	477	429	-	2,293	477
4. Lock & T. Gate (Mechanical)															
-Gates Related	mill US$	889	-	1,011	-	898	-	1,023	-	950	316	738	-	1,252	316
5. New Dams															
a. Upper Gatun Dam (as regulating)	mill US$	-	-	-	-	-	-	-	-	-	90	-	90	-	90
b. Trinidad Dam (as regulating)	mill US$	-	-	-	93	-	93	-	61	-	-	-	-	-	-
c. Ciri Dam (as regulating)	mill US$	-	28	-	-	-	-	-	75	-	-	-	-	-	-
d. Indio Dam (as water source)	mill US$	-	173	237	-	195	-	207	-	-	-	-	-	-	-
6. Pumping Stations	mill US$	-	149	-	149		149		149	-	-	-	-	-	-
7. New Parmanent Road	mill US$	9	-	9	-	9	-	9	-	19	-	19	-	19	-
8. Canal Crossing	mill US$	450	777	501	839	450	777	501	839	222	548	309	548	222	548
9. Barrier Dams	mill US$	-	-	-	-	-	-	-	-	494	-	494	-	494	-
10. Diversion Scheme	mill US$	-	-	-	-	-	-	-	-	75	-	75	-	75	-
11. Navigation Support System	mill US$	414	-	414	-	414	-	414	-	546	-	477	-	546	-
12. Breakwater	mill US$	-	-	-	-	-	-	-	-	402	-	402	-	402	-
13. Supporting Construction	mill US$	425	88	425	85	425	88	425	85	574	66	574	38	574	66
14. Other Works *1	mill US$	-	-	-	-	474	-	474	-	-	-	-	-	-	-
15. Indirct Cost	mill US$	1,096	192	1,257	184	1,096	192	1,256	184	1,767	220	1,668	99	1,849	220
Total Project Cost	mill US$	7,374	1,407	8,546	1,350	7,912	1,299	8,844	1,393	13,519	1,717	12,747	775	14,159	1,717
E. O & M COST															
1. Electrical Power	mill US$	10.4	0.9	10.8	1.4	10.4	0.9	10.8	1.4	7.8	2.6	0.6	-	8.2	2.6
2. Man Power and Other O&M Costs	mill US$	103.9	59.9	112.2	59.9	103.9	59.9	112.2	59.9	154.6	51.2	144.3	51.2	159.7	51.2
3. Total O&M Cost	mill US$	114	61	123	61	114	61	123	61	162	54	145	51	168	54
F. CONSTRUCTION ASPECTS															
1. Adaptablity of Partial Construction	grade	Possible		Possible		Possible		Possible		Impossible		Impossible		Impossible	
2. Sufficiency of Engineering/Cost Data	grade	Fair		Fair		Fair		Fair		Poor		Poor		Poor	
3. Interference by Existing Canal Operations	grade	Heavy		Heavy		Heavy		Heavy		None		None		None	
4. Construction Period	year	10.0		10.5		10.0		10.5		15.0		15.0		15.0	
G. IMPACT ON EXISTING SITUATION															
1. Impact on Existing Panama Canal															
a. Interferences to Existing Canal Operations	grade	Considerable		Considerable		Considerable		Considerable		None		None		None	
b. Modification Works to Existing Panama Canal	grade	Minor		Minor		Minor		Minor		None		None		None	
2. Potential Envilonmental Impact															
a. Possibile Gatun Lake Salinity Increase	grade	Same with SQ		Same with SQ		Same with SQ		Same with SQ		None		None		None	
b. Both Oceans Intermixing	grade	None		None		None		None		Not Studied		Not Studied		Not Studied	
c. Ecologicaly Sensitive Areas	grade	Very Low		Very Low		Very Low		Very Low		Low		Low		Low	
3. Socioeconomic Impacts															
a. Relocation of Residents	grade	Few		Few		Few		Few		Small		Small		Small	
b. Increased Work Opportunity	grade	Large		Large		Large		Large		Very Large		Very Large		Very Large	

Note *1: Other Works include existing lock modification.
Gatun Dam modification. etc.

PART I – INTRODUCTION & OBJECTIVES

I. CHAPTER 1 - STUDY ALTERNATIVES

1.1 General Description of the Alternatives

The ECE Phase 2 Study is being carried out on the two conceptual alternatives which survived the prescreening process. The two alternatives are the High Rise Lock Canal (HRLC) and the Route 10 Sea Level Canal (SLC). The routes are shown in Figs. I.1-1 and I.1-3 respectively and the schematic layouts are shown in Figs. I.1-2 and I.1-4 respectively. The main features of the alternatives, and the variations to be considered are described below.

1.2 High Rise Lock Canal

The High Rise Lock Canal represents a new fundamental alternative derived from the results of the Phase 1 work. It is characterized by improvements to the existing canal to allow ships of larger size to transit. Two variations for the maximum ship size are considered; 150,000 DWT and 200,000 DWT. The ability for the larger ships to transit will be achieved by the construction of a lane of larger, two-lift locks at Gatun and Miraflores, and by widening and deepening the canal waterway elsewhere. Bypass canals will be constructed at Gatun and Miraflores connecting the new lane of locks to the existing canal. A variation in the level of Gatun Lake from 85 feet to 90 feet will also be considered to determine the effects on costs. In all variations, the existing canal and locks will continue to operate both during and after construction of the canal improvements.

The alternative and variations described above do not correspond exactly to any of the Study Cases from Phase 1. The high rise lock canal for the 150,000 DWT ships closely corresponds to the Phase 1 Study Case L4 (Gatun Lake 85') and L5 (Gatun Lake 90'). While Phase 1 Study Case L7 and L8 considered 250,000 DWT ships with Gatun Lake levels of 85' and 90' respectively, the Phase 1

Study did not consider 200,000 DWT ships as a separate case. A further major difference is that all Study Cases in Phase 1 assumed that two parallel lanes of new locks would be constructed. In Phase 2 however, only the excavation for the second lane of new locks will be included.

The route of the High Rise Lock Canal is shown in Fig. I.1-1 and closely follows the existing canal except at the bypass canal sections. The layout of the canal improvements and the relationship to the existing canal are shown schematically on Fig. I.1-2 and further details are described below.

1.2.1 Operational Requirements

Two variations of the High Rise Lock Canal are considered. One caters for ships to a maximum nominal size of 150,000 DWT and the other for ships to a maximum nominal size of 200,000 DWT. The canal navigation width is sufficient for two-way operation throughout the canal of the adopted maximum ship size.

1.2.2 Existing Canal

The existing canal will continue in operation during and after the canal improvements, and new construction and dredging works will be carried out in and beside the existing canal. The preliminary design and construction planning must ensure that disruption to existing canal navigation, operation and maintenance are minimized. The cost estimates should reflect any higher costs associated with restrictions on construction works.

1.2.3 Canal Transits

The predicted numbers of transits and lockages through both the existing locks and the new locks are required for the preliminary designs. Design areas affected by transits and lockages include the analysis of lockage water requirements, fatigue design of the lock gates, and estimation of anchorage area requirements.

Transit and lockage predictions are based on information provided to the ECE Contractor by the CAS Secretariat. The information provided by the Secretariat was based on data from the other on-going component studies; the Operating Characteristics and Capacity Evaluation Study (OCCE Study) and the Commodity and Traffic Projections Study (CTP Study).

Transit information is available for the years 2020 and 2060, and will be used to consider possible phasing of the construction of new dams and pump stations for lockage water supply.

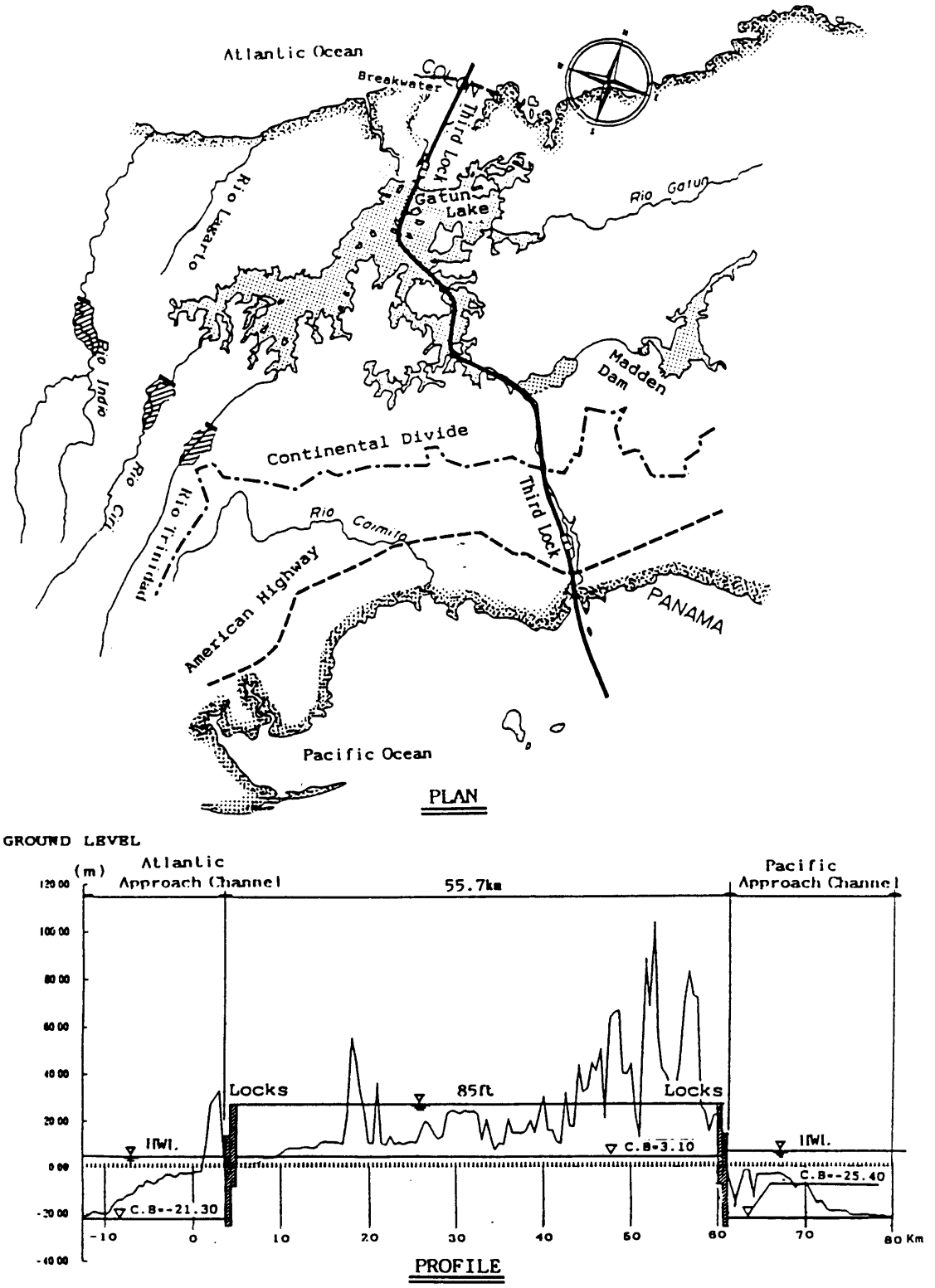
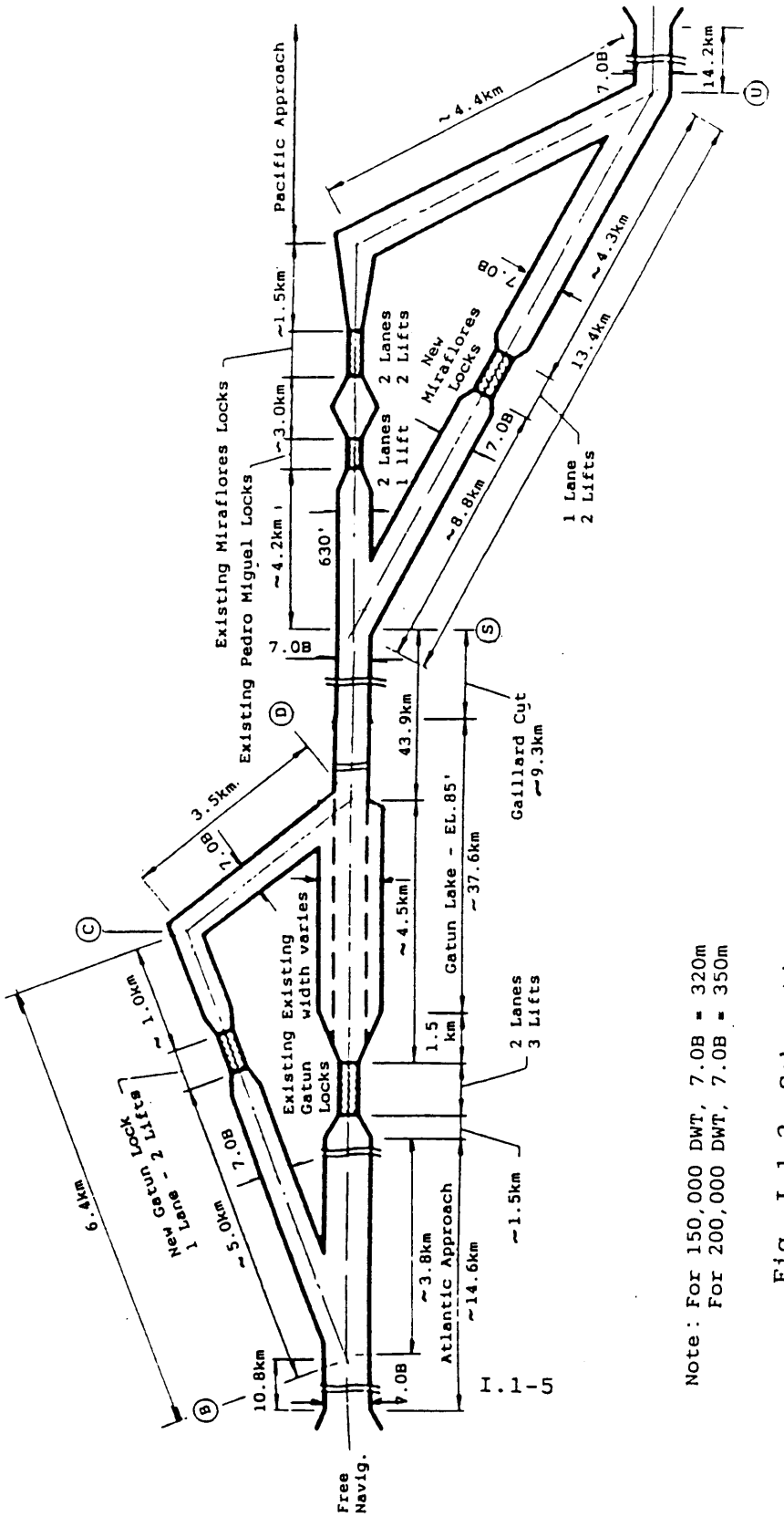


Fig. I.1-1 Route of the High Rise Lock Canal (200,000 DWT)



Note : For 150,000 DWT, 7.0B = 320m
For 200,000 DWT, 7.0B = 350m

Fig. I.1-2 Schematic Layout of the High Rise Lock Canal

I.1-5

1.2.4 Water Management

The existing dams and lakes are able to provide sufficient lockage water for the number of ships now using the canal. However, this situation is likely to change if the High Rise Lock Canal improvements are implemented and the water management analysis is an important component of the Phase 2 Study.

In Phase 1, potential new dam sites were identified on the Ciri Grande, Trinidad, Indio and Gatun rivers. The Phase 2 Study reviews the need for the new dams and considers where necessary the possibility of supplementing the available water supply with pumping systems and reservoirs for the recycling of fresh water.

After allowing for evaporation and municipal usage, the water management analysis is based on the following order of priority for water usage:

1. Lockage water (highest priority), then
2. Water for hydro-electric power generation.

Dam spillage will only be allowed when unavoidable.

As mentioned in Section 1.2.3 above, the water management analysis will be carried out for the years 2020 and 2060. Recommendations for the staged construction of new dams and pumping stations will be made if appropriate.

1.2.5 Gatun Lake Water Level

By increasing the level of Gatun Lake from the existing 85 feet to 90 feet, the amount of dredging required to allow the passage of 150,000 DWT and 200,000 DWT ships across Gatun Lake is reduced. This variation to the High Rise Lock Canal is included to investigate if the savings in dredging costs compensate for other problems. Disadvantages to be investigated include higher lockage water requirements and additional lock construction costs. Modifications would be required to existing locks, dams, spillways and other infrastructure affected to suit the higher water level. Other infrastructure affected would include sections of the Panama-Colon railway, the PCC

lakeside facilities at Gamboa, and low level roads and bridges beside the lake. In addition, the effect of the higher water level on the environment should be considered.

1.2.6 Fourth Lane of Locks

It is anticipated that the addition of a fourth lane of locks will be required in the future. The construction planning and cost estimates for the High Rise Lock Canal will be based on construction of just one new lane of larger locks at Gatun and Miraflores. However, the excavation for the fourth lane of locks, ie. for a second lane of larger locks, is included and costed.

Preliminary designs and a separate cost estimate are also required for the fourth lane of locks as a guide for long term planning. The construction of the fourth lane is included in the costs for the year 2060.

The preliminary designs and cost estimates for the third and fourth lanes of locks include culverts to allow the direct transfer of lockage water between adjacent locks. This feature will allow water conservation, likely to be critical if transit numbers required the fourth lane.

1.2.7 PCC Culebra Cut Widening Contract

In July 1991 the PCC approved proposals for the widening and straightening of the 13 km length of Gaillard Cut. The widening is from 152m (500') to 192m (630') on straight sections and to a maximum width of 223m (730') on curves. Completion of this major improvement will allow virtually unrestricted two-way transit of PANAMAX size ships throughout the Panama Canal. Excavation works commenced in 1992 and will continue over a number of years. (Source: Panama Canal Commission Annual Report, Fiscal Year ended September 30, 1991.)

For the purposes of this Study, the improvements are considered to have been completed. The excavation volume for the Widening Contract is not included in the final volumes and cost estimates for the High Rise Lock Canal.

1.2.8 Associated Facilities

The preliminary design and costing for the High Rise Lock Canal includes the design of associated facilities necessary for operation of the improved canal. These facilities include emergency gates, maintenance gates, anchorage and mooring areas, breakwaters, port facilities, and emergency lay-by facilities (tie-up stations). Improvements to the existing road network and new bridges over the canals are also defined and costed.

In addition a lockage guidance system and a navigation aid system based on requirements of the OCCE Contractor is included in the cost estimates.

1.3 Outline of the Sea Level Canal and Variations

The fundamental alternative for the Sea Level Canal under consideration in Phase 2 was derived from the results of the Phase 1 work. It represents a refinement of the sea level canals considered in Phase 1. The route follows Route 10 of the IOCS Study and is characterized by a maximum ship size of 250,000 DWT. Across the isthmus the canal will be one-way, with two-lane approaches on the Pacific and Atlantic sides. For tidal control three variations of tidal locks and tidal gates will be considered. For the control of movement of biota between the two oceans, the tidal barriers required for tidal control will be utilized to assist in the formation of a fresh water barrier.

Since the existing lock canal continues in operation during and after construction of the Sea Level Canal, barrier dams must be constructed across Gatun Lake to retain the lake water at existing levels.

The route of the Sea Level Canal is shown in Fig. I.1-3 and closely follows Route 10 from the IOCS Study. The layouts of the three variations of the proposed canal are shown schematically on Fig. I.1-4 and further details are described below.

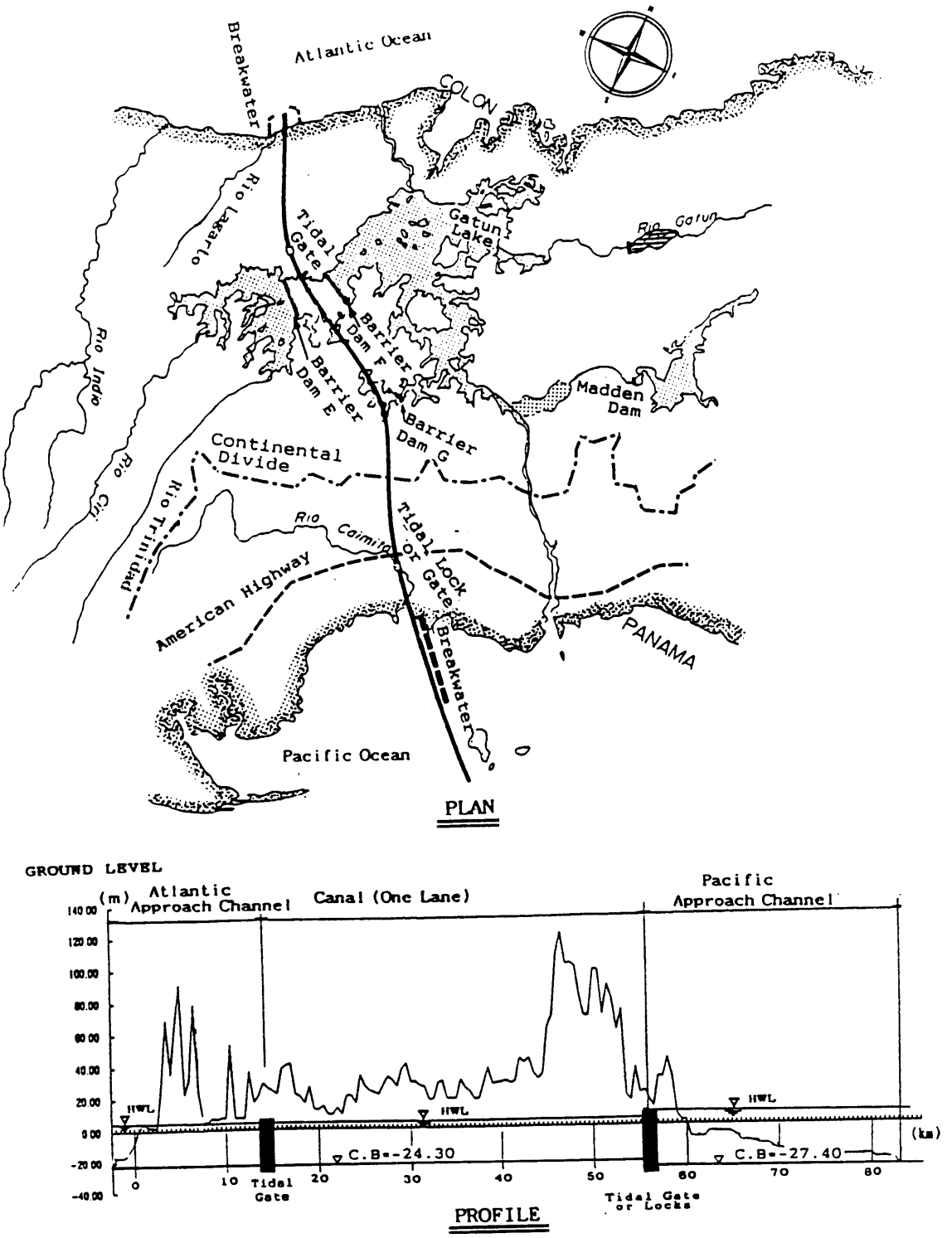


Fig. I.1-3 Route of the Sea Level Canal [Case (a)]

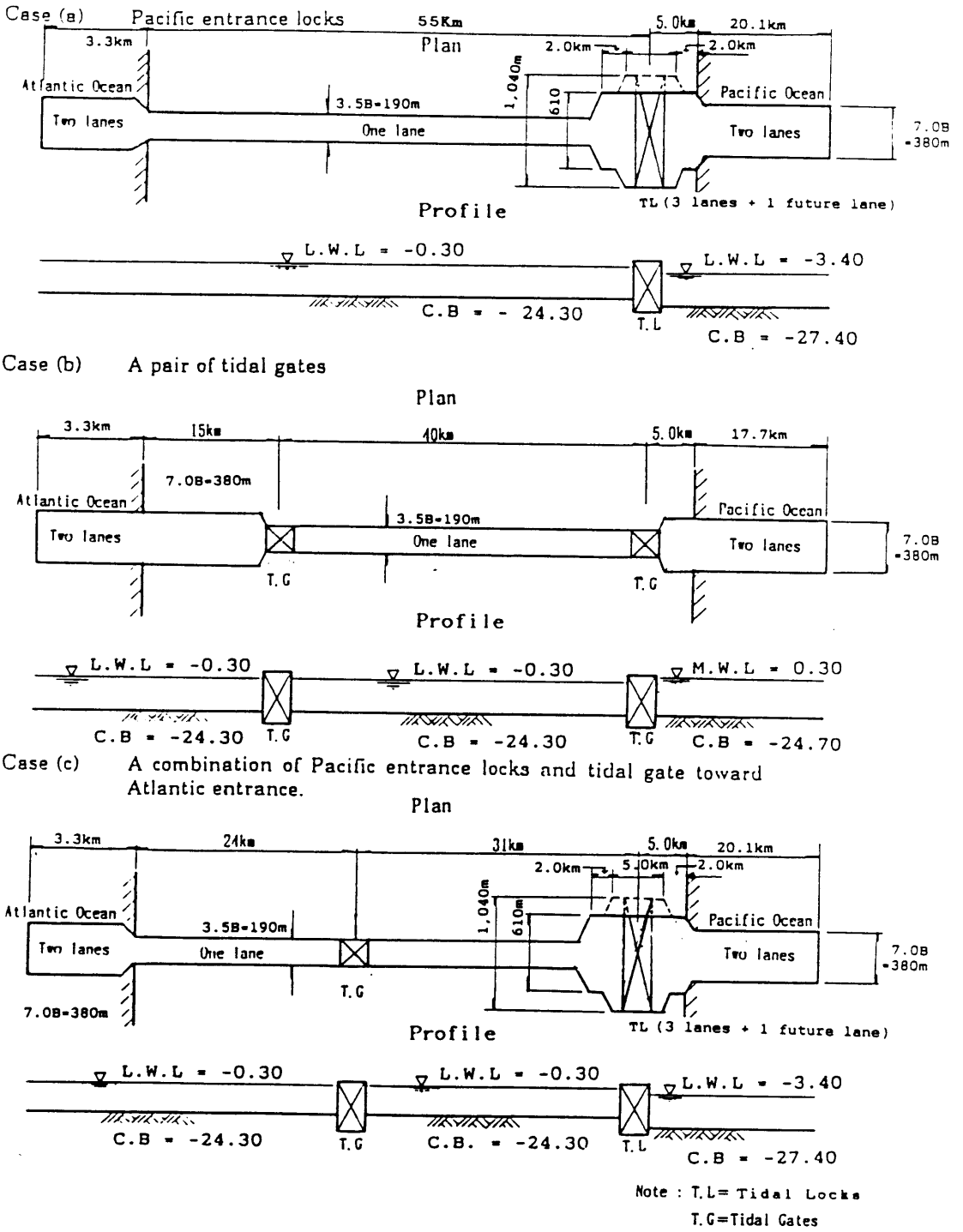


Fig. I.1-4 Schematic Layout of the Sea Level Canal

The Sea Level Canal alternative and variations described briefly above do not correspond exactly to any of the Study Cases from Phase 1. The Phase 1 study cases most similar to the Sea Level Canal now under consideration were Study Case S-3 (250,000 DWT, one-way for full length) and Study Case S-14 (250,000 DWT, 50% one-way and 50% two-way).

1.3.1 Operational Requirements

The Sea Level Canal caters for 250,000 DWT vessels and the navigation width allows for one-way operation except for the approaches, which are two-way. The position of the transition from the one-way canal to the two-way ap proaches depends on the variation under consideration and is described in Section 1.3.2 below.

1.3.2 Tidal Locks and Tidal Gates

Three variations are considered for the control of the tidal currents that would be generated in an unrestricted sea level canal. The variations were defined during discussions with the CAS Secretariat in Panama following submission of the Inception Report. The three variations are shown schematically in Fig. I.1-4 and are further described below.

Case (a) Pacific Tidal Locks

Tidal control is provided by one set of tidal locks (three lanes wide with an option for a fourth lane). The locks are located about 5 km from the Pacific shoreline at a position with suitable topography and soil conditions. The two-way approach on the Pacific side terminates at the locks and the two-way Atlantic approach channel terminates at the Atlantic shoreline.

Case (b) A Pair of Tidal Gates

Tidal control is provided by tidal gates located near each shoreline. The location of the Pacific gates is as in Case (a) while the Atlantic gates are approximately 40 km from the Pacific Gates. The one-way canal length of 40 km is based on the operating cycle of the gates and the sailing time of convoys through the one-way section. Both the Pacific and Atlantic approaches to the gates are two-way and the transition to the one-way canal occurs on the ocean side of the gates.

Case (c) Pacific Locks and Atlantic Tidal Gates

This case represents a slight variation of Case (a). Tidal locks are provided on the Pacific side according to Case (a) and the limits of the one-way and two-way canal sections are also according to Case (a). However, a tidal gate is added on the Atlantic side to facilitate the formation of a fresh water barrier, described in Section 1.3.3 below. The operations and capacity of Case (c) are the same as for Case (a), with the tidal gate opening and closing to suit the transit of the ship convoys. The site of the tidal gate does not affect the canal capacity. It has been located 31 km from the Pacific locks at a site suitable from a construction viewpoint and where it can contribute effectively to creation of the freshwater barrier. This aspect is further described in Part IV - Section 1 - Chapter 2.3.

In accordance with advice from the CAS Secretariat, the tidal locks in Cases (a) and (c) have three parallel lock chambers. However the designs allow for the future addition of a fourth parallel lock chamber. The cost of adding the fourth chamber for 2060 is estimated to assist in future planning.

The preliminary designs and costing of the tidal locks assume that a mule system, rather than tugs alone, will be adopted for assisting vessels through the locks.

The tidal gates in Cases (b) and (c) provide the same clear width as the bottom width of the one-way canal.

1.3.3 Canal Transits

The predicted number of transits and lockages through both the existing lock canal and through the Sea Level Canal is required for the preliminary designs. The numbers will affect important aspects of the preliminary designs, including the lockage water analysis for the existing locks, lockage water usage for the tidal locks, fatigue design of the tidal locks and tidal gates, and estimation of anchorage area requirements.

As in the case of the High Rise Lock Canal, transit and lockage predictions are based on information provided to the ECE Contractor by the CAS Secretariat. The CAS information was derived from data produced by component contractors for the OCCE and CTP Studies.

Transit information is available for the years 2020 and 2060. It will be used to consider possible phasing of the construction of any new dams or pump stations required for continued operation of the existing locks.

1.3.4 Water Management

The existing canal will continue in operation during and after construction of the Sea Level Canal. The existing dams and lakes provide sufficient lockage water for the number of ships now using the existing canal but a review is necessary. The water area, storage capacity, and catchment area of Gatun Lake are all reduced by construction of the barrier dams for the Sea Level Canal. Therefore a water management analysis for the reduced lake storage and catchment area is required for continued operation of the existing canal and locks. The upper Gatun Dam was identified in Phase 1 as a possible dam

site and the Phase 2 Study reviews the need for this dam or for a pumping system to supplement the available water supply for the existing locks.

The water management analysis for the existing locks is based on the same assumptions adopted for the High Rise Lock Canal. After allowing for evaporation and municipal usage, the order of priority is:

1. Lockage water (highest priority)
2. Water for hydro-electric power generation.

Dam spillage will only be allowed when unavoidable.

As mentioned in Section 1.3.3 above, the water management analysis will be carried out for the years 2020 and 2060. Recommendations for the staged construction of new dams and pumping stations will be made as appropriate.

For the Sea Level Canal itself, methods for run-off storage, diversion or discharge into the canal are required both during and after construction. The post-construction proposals must be designed to support the formation of a freshwater barrier as described below.

1.3.5 Freshwater Barrier

Studies have indicated that a freshwater barrier may be required to prevent the transfer of biota between the two oceans.

The three variations of tidal gates and tidal locks described in 1.3.2 above will assist in the control of biota transfer. However, they are unlikely to be fully effective without additional measures such as the establishment of a permanent freshwater zone within the Sea Level Canal. It has been agreed with the CAS Secretariat that establishing design concepts for the freshwater barrier, or assessing the effectiveness of the freshwater barrier, are outside the Scope of Works of the ECE Contractor.

For the purposes of the Study, preliminary engineering designs and cost estimates are prepared for the diversion and discharge of as much freshwater as possible from the adjacent catchments into the Sea Level Canal. Assessment of the effectiveness of the freshwater barriers will be carried out by others at a later date, based on the freshwater diversion and discharge proposals developed in this Study.

It should be noted that the provision of tidal locks on both the Atlantic side and the Pacific side, coupled with the discharge of sufficient freshwater between the two locks to maintain a permanent freshwater canal between the locks, would be likely to provide the most effective control. This option is not directly considered in the Study.

1.3.6 Associated Facilities

The preliminary designs and costing for the Sea Level Canal include the design of associated facilities necessary for operation of the new canal. These facilities include emergency gates, maintenance gates, anchorage and mooring areas, breakwaters, port facilities, emergency lay-by facilities (tie-up stations), improvements to the existing road network and provision of new bridges over the sea level and existing canals. Facilities required for the continued operation of the existing canal are also included.

In addition a lockage guidance system, and a navigation support system based on requirements of the OCCE Contractor, will be included in the cost estimates.

I. CHAPTER 2 - PHASE 2 STUDY

2.1 Objectives

The purpose of the Phase 2 component study of Engineering and Cost Estimates (ECE) is to study the two Conceptual Alternatives and variations described in Chapter 1 and to provide the CAS Secretariat with engineering and cost estimate information, data, analysis and conclusions. These findings will enable the Secretariat, after considering the results of all Phase 2 component studies, to select and recommend to the Commission the best and most viable alternative to the present Panama Canal.

To meet the broad objectives described above and to assist the Secretariat and the Commission in their assessment, the Study methodology and reporting should be consistent with the following more detailed objectives for the Phase 2 Study:

- The engineering and preliminary designs, construction planning and cost estimates should include the appurtenant facilities required for operation of the canal. This will ensure that cost estimates for each conceptual alternative and variation are complete and can be fairly compared, providing the basis for appropriate recommendations.

- The construction planning and cost estimates should give due consideration to the scale of the project, including evaluation of current construction equipment and techniques, and consideration of possible future improvements and innovation. Examples could include:

  i) Consideration of equipment larger than that currently available to take advantage of economies of scale.

ii) Use of alternative methods for excavation and transport of material, eg. rail, conveyor systems, etc.

iii) Bulk purchase and delivery of materials and equipment.

- The Study results and cost estimates should be as comparable as possible for each conceptual alternative and variation. In cases where results are not fully comparable, due for example to a lack of survey information for a conceptual alternative or variation, the dissimilarity in available data should be clearly identified and the possible effect on the results should be considered.

- All assumptions necessary in deriving the results and cost estimates should be clearly defined in the relevant section of the report.

- Sources of data and information used during the Study should be clearly cited in the report.

- The report should be compatible with the assumptions, findings and requirements of the other component contractors

2.2 Terms of Reference

The scope of works to be carried out by the ECE Contractor is fully described in the Terms of Reference appended to this report (Appendix 1). Minor modifications were agreed during the meetings in Panama following submission of the Inception Report and the Interim Report, and are described in the respective Memorandum of Meeting.

In the Terms of Reference the Scope of Services has been summarized in terms of four main areas of activity: Engineering, Preliminary Design, Construction Planning, and Cost Estimating. Detailed requirements are described in the Scope of Services under each heading. As agreed during the Interim Report meetings in Panama, the report

is structured as far as possible to follow this basic order.

2.3 Study Schedule

The proposed Working Schedule was produced for the Phase 2 Inception Report. A description of each study item and the study methodology on which the schedule was based were included in the Inception Report.

The Study work was mainly carried out in Japan. Periodic visits to Panama provided opportunities for discussions with the Secretariat and other concerned parties to confirm the study direction, submit reports, obtain data and information, and to coordinate with other component contractors.

PART II – GENERAL

PART II SECTION 1 ENGINEERING AND PRELIMINARY DESIGN

II.1 CHAPTER 1 - BASIC ENGINEERING CRITERIA

In order to be able to carry out the Phase 2 Study it has been necessary to decide the basic features and parameters that control the development of the preliminary designs. Some features were designated in the Terms of Reference, some were agreed during the subsequent meetings with the CAS Secretariat and others have been investigated and developed by the ECE Contractor. These basic features are critical to the Phase 2 Study and are described below.

1.1 Design Ships

Ship sizes to be adopted for the alternative studies are 150,000 and 200,000 DWT for the High Rise Lock alternative and 250,000 DWT for the Route 10 Sea Level Canal alternative. The design ship sizes were provided by the Secretariat during the Inception Report meeting and are shown in Table II.1.1-1 below.

Table II.1.1-1 Design Ship Dimensions

Ship Size (DWT)	Ship Dimensions (m) Length	Beam	Draft
65,000	295	32	12
150,000	306	46	18
200,000	333	50	19
250,000	356	54	21

1.2 Lock Chamber Dimensions

The dimensions of the lock chambers for 150,000 DWT, 200,000 DWT and 250,000 DWT ships as provided by the Secretariat early in Phase 2 are shown in Table II.1.1-2 and Fig. II.1.1-1 overleaf.

As agreed during the discussions in Panama following the Interim Report submission, the length of the tidal locks for 250,000 DWT ships assumes that movement of ships through the locks will be aided by mules rather than tugs.

Table II.1.1-2 Lock Chamber Dimensions

Canal Alternative	Ship Size (DWT)	Lock Chamber Dimensions (m) Length L1*	Length L2**	Width	Depth
HRLC	150,000	365	390	50	20.0
HRLC	200,000	395	420	55	21.5
SLC	250,000	420	430	60	23.0

* Provided by the Secretariat.
** Determined by the ECE Contractor considering emergency space.

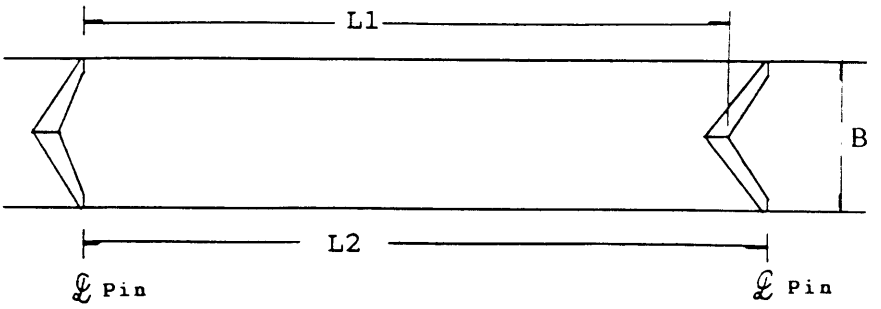


Fig. II.1.1-1(b) Definition of L1 & L2 for HRLC (Miter Gates)

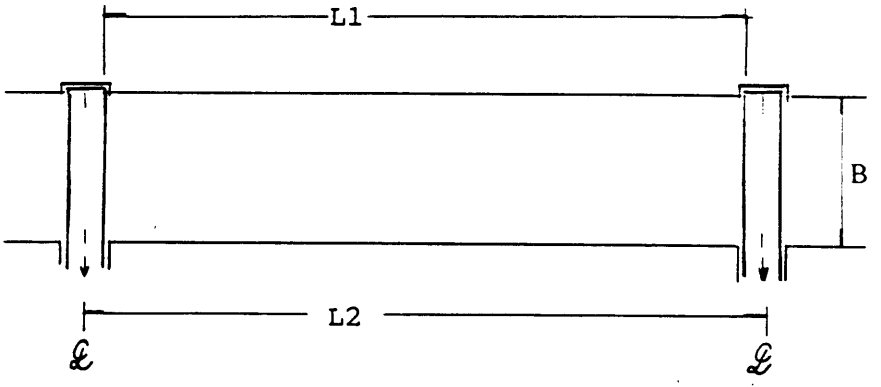


Fig. II.1.1-1(a) Definition of L1 & L2 for SLC (Torsion Type Sliding Gates)

1.3 Canal Cross Section Dimensions

Two canal schemes are considered in Phase 2. A single lane Sea Level Canal with two-lane approaches for ships of 250,000 DWT and, on the high level route, the canal is to be enlarged to two lanes for ships of 150,000 and 200,000 DWT. Thus, the design of dual and single lane channels is required for 250,000 DWT ships and dual lane for 150,000 and 200,000 DWT vessels.

Cross section dimensions for the High Rise Lock Canal and the Sea Level Canal to be adopted for the Phase 2 Study were confirmed by the CAS Secretariat during the meetings in Panama which followed the Interim Report submission. The dimensions adopted are a function of the design ship sizes as follows;

Width at Base = W_s = 3.5 B (single lane canal)

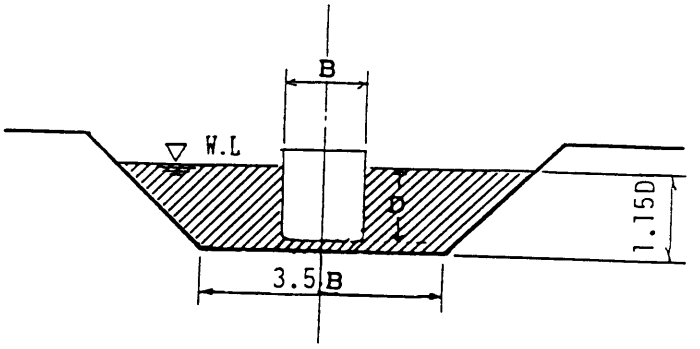
W_d = 7.0 B (double lane canal)

Depth = 1.15 D ;

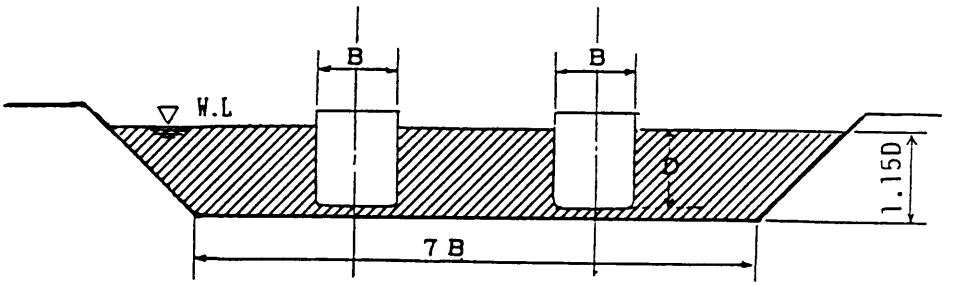
where B and D are the Beam and Draft respectively of the design ship.

The channel width for two-lane traffic was selected by CAS on the basis of (1) U.S. Army Corps of Engineers and U.S. Navy channel design literature and (2) on the basis of the design ship width of the Panama Canal Culebra Cut width as determined through computer ship simulations at the U.S. Merchant Marine Academy.

Fig. II.1.1-2 shows graphically the typical canal sections for one-way and two-way canals.



(a) One-Way



(b) Two-Way

Fig. II.1.1-2 Typical Canal Sections

Using the formulae above, the required cross section dimensions have been calculated and rounded slightly. The resulting dimensions are shown in the following Table II.1.1-3.

Table II.1.1-3 Canal Cross Section Dimensions (m)

	High Rise Lock Canal 150,000 DWT	High Rise Lock Canal 200,000 DWT	Sea Level Canal 250,000 DWT
Width	320	350	190 (1-way) 380 (2-way)
Depth	21	22	24

1.4 Canal Bank Slopes

1.4.1 General

An excavation slope is a slope which has been artificially created by cutting the natural slope and its stability depends on the original ground characteristics. Only the slope gradient can be artificially controlled.

The final decision on the criteria to determine an excavation slope gradient can only be taken after slope stability analysis based on soil engineering constants which are identified by a soil engineering survey in the subject area. In addition, such a decision must also take experience with similar soil characteristics, slope size and project purposes into proper consideration. The study findings on the gradient criteria for cut slope to be created by the canal excavation have been made public through several reports. The most useful reports in this regard are the Atlantic Pacific Interoceanic Study (IOCS, 1970) and the Nagano Plan (1981). Both propose five sectional forms based on rock quality using the same criteria. The gradient criteria proposed by the Study are sightly gentler than those of the IOCS and Nagano Plan, considering the reports associated with Culebra cut widening project (Ref; "Study of clay shale slopes along the Panama Canal" - Report IV, History, Geology, and mechanics of development of Slides in Gaillard. -) as compared in Table II.1.1-4.

Table II.1.1-4 Comparison of Excavation Slope Gradient Criteria Proposed by Various Studies

Type	Rock Type	IOCS and Nagano Plan (cotangent)	Proposed Crit. (cotangent)
1	High quality rock	0.4	0.6 to 0.7
2	Intermediate quality rock	0.6	1.0 to 1.3
3	Low quality rock	1.0 to 2.8[1)]	2.0 to 4.4
4	Soft rock	1.0 to 6.7 (Case A curve)[1)]	2.0 to 4.4
5	Unconsolidated sediments	2.5[2)]	2.5
		10[3)]	

Note: 1) The slope in question is defined in Fig.II.1.1-9
2) Slope in navigation prism
3) Slope above navigation prism

1.4.2 Differences in Criteria

The differences between the IOCS slope criteria and the proposed slope criteria are summarized as follows.

(1) High quality rock (Type 1) is either intrusive ballast or hard agglomerate and is represented by the excavation slope gradient at Gold Hill where the slope gradient of 2H/3V corresponds to the IOCS criterion for intermediate quality rock.

(2) An example of intermediate quality rock (Type 2) is the Gatun formation at the No.3 lock former excavation site. This type is considered to be a stable slope with sedimentary rock. In the case of unconsolidated rock of some height, adoption of the Type 2 slope with a gradient of 45° and 15m wide berms at 50m intervals is recommended.

(3) The IOCS criterion for low quality rock (Type 3) corresponds to the current slope gradient. This criterion should be applied to shale is relatively more stable than soft agglomerate containing soft clay shale.

(4) The IOCS slope gradient criterion for soft rock (Type 4) roughly corresponds to the present slope gradient. The average gradient is approximately 3H/1V. The minimum excavation volume is achieved by introducing a 100m wide bench at each excavation height of 50m and the adoption of a slope gradient of 2H/1V between wide benches is proposed in view of preventing landslides due to the excavation of high elevation areas along the canal.

The Culebra formation is classified as low quality rock by the IOCS. However, it is reclassified into the soft rock category (Type 4) because of compatibility with the present slope gradient and the criterion for low quality rock and also because of a history of landslides at the present slopes.

(5) The proposed criterion for unconsolidated sediments (Type 5) is the same as the IOCS criterion (Type 5).

1.4.3 Standard Slope Configurations

The Study did not conduct any special soil engineering survey and the standard slope configurations are simply based on existing data. Their applicability should remain within the scope of the Feasibility Study. The introduction of 5 different types is proposed depending on the rock and sediment properties as shown in Figs. II.1.1-3 to 7.

Empirical knowledge on natural slope gradients and excavated slopes should be referred to in conducting the detailed design and actual construction work in order to determine the slope type together with constant examination of PCC records. Actual construction work must be based on careful surveying and monitoring of the work.

Type 1: Strong unaltered or slightly altered volcanic rocks, e.g. basalt and hard agglomerate limestone; slopes of moderately consolidated intact sedimentary rocks and tuff not exceeding 50m in height (Fig. II.1.1-3)

Type 2: High slopes of moderately consolidated intact sedimentary rocks and tuff; altered but still solid volcanic rocks (Fig. II.1.1-4)

Type 3: Soft volcano-sedimentary rocks, including soft agglomerate, with partial clay shale but relatively stable to form high massive hills (Fig. II.1.1-5)

Type 4: Soft volcano-sedimentary rocks, including soft agglomerate with frequent development of clay shale; technical highly distorted and sheared, located in landslide-afflicted areas (Fig. II.1.1-6)

Type 5: Unconsolidated deposits (Fig. II.1.1-7)

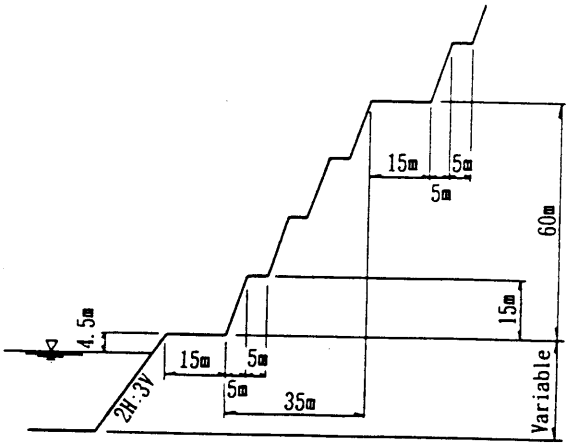


Fig. II.1.1-3 Slope Criteria for Type 1

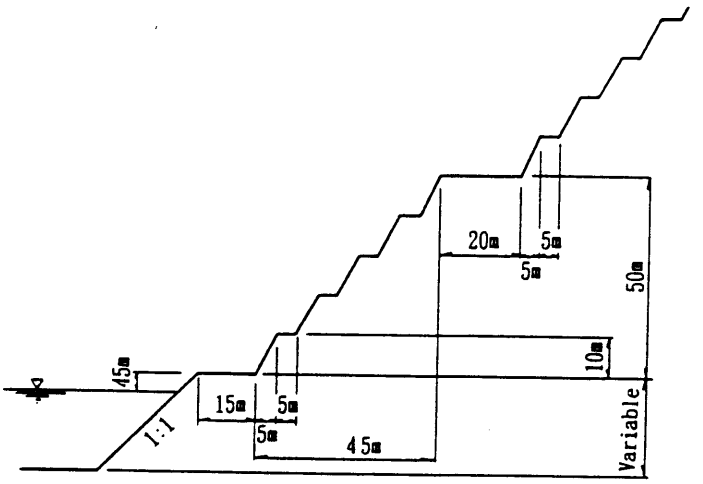


Fig. II.1.1-4 Slope Criteria for Type 2

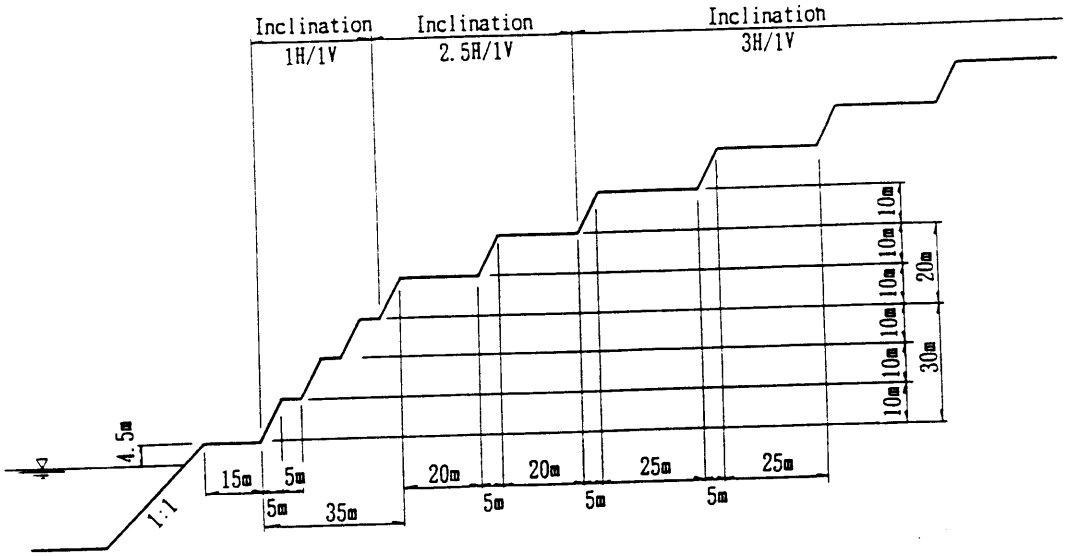


Fig. II.1.1-5 Slope Criteria for Type 3

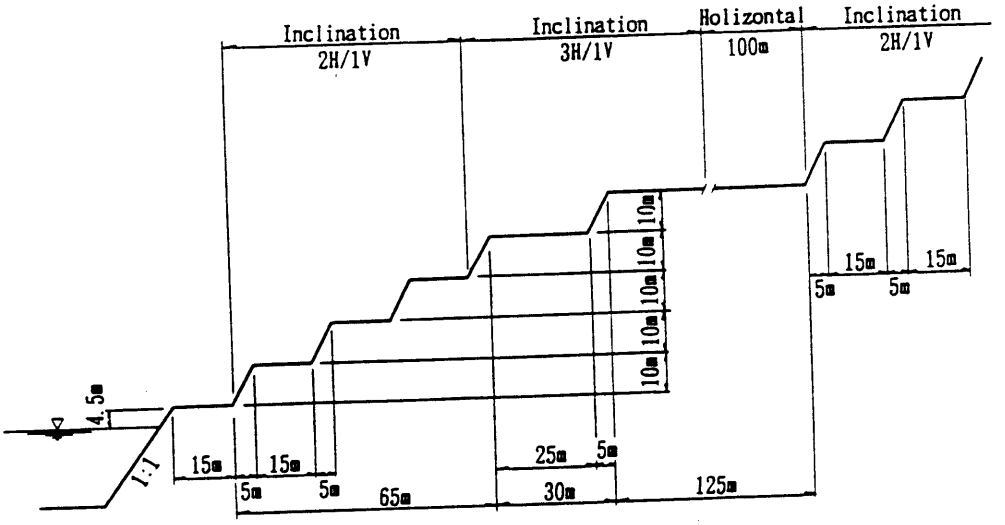


Fig. II.1.1-6 Slope Criteria for Type 4

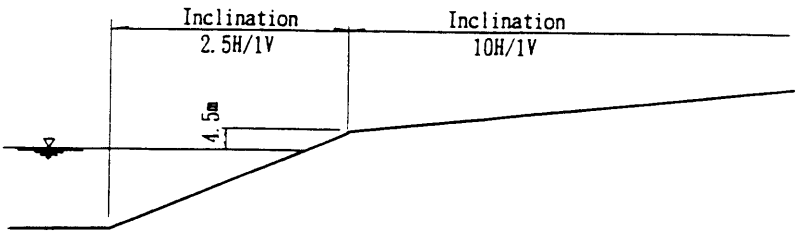
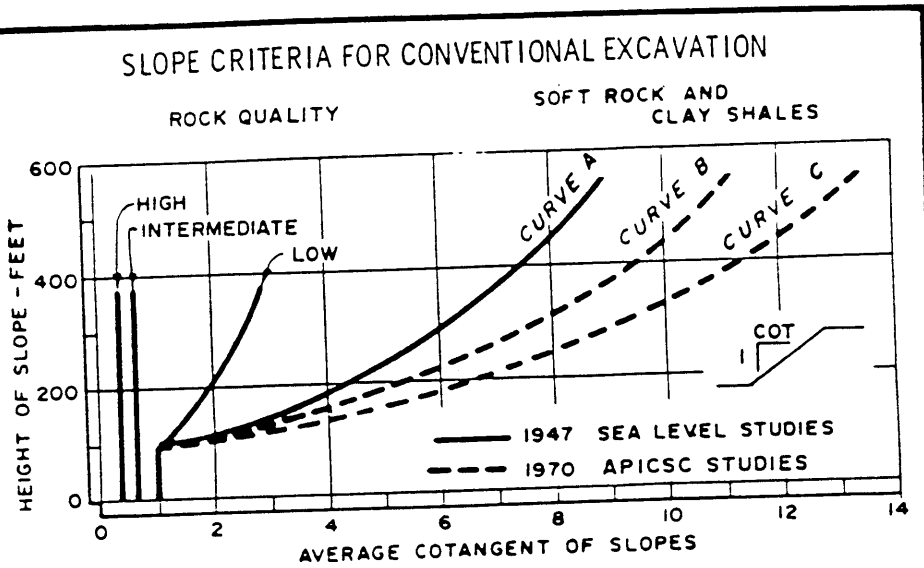


Fig. II.1.1-7 Slope Criteria for Type 5



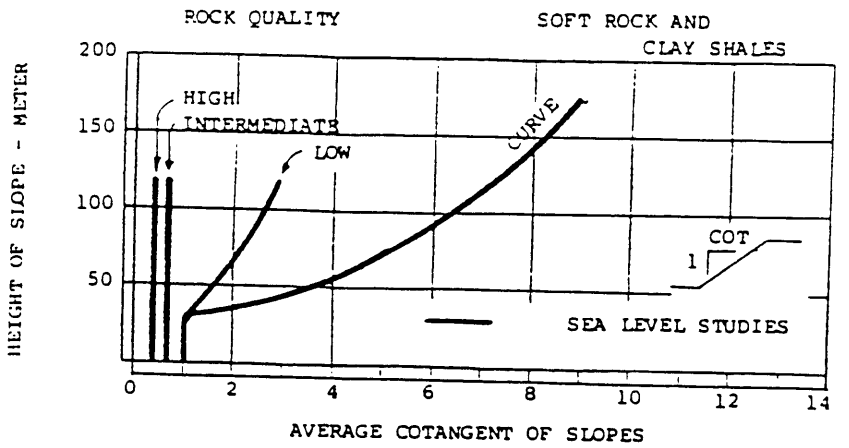
APPLICATION OF CRITERIA

	ROUTE 10	ROUTE 14
HIGH QUALITY ROCK	BASALT (EXTRUSIVE) (INTRUSIVE)	BASALT PEDRO MIGUEL AGGLOMERATE BAS OBISPO
INTERMEDIATE QUALITY ROCK	CHAGRES GATUN CAIMITO	GATUN BOHIO CAIMITO
LOW QUALITY ROCK	NONE IDENTIFIED	LAS CASCADES CULEBRA LA BOCA PANAMA TUFF
SOFT ALTERED VOLCANIC ROCKS AND CLAY SHALE	A HIGHLY ALTERED AGGLOMERATES & TUFFS	B (14-S) C (14-C) CUCARACHA
SOILS	RESIDUAL SOIL ATLANTIC MUCK PACIFIC MUCK	RESIDUAL SOIL ATLANTIC MUCK PACIFIC MUCK

FIGURE E-2

Source: IOCS Appendix 9 (1970)

Fig. II.1.1-8 IOCS Slope Criteria

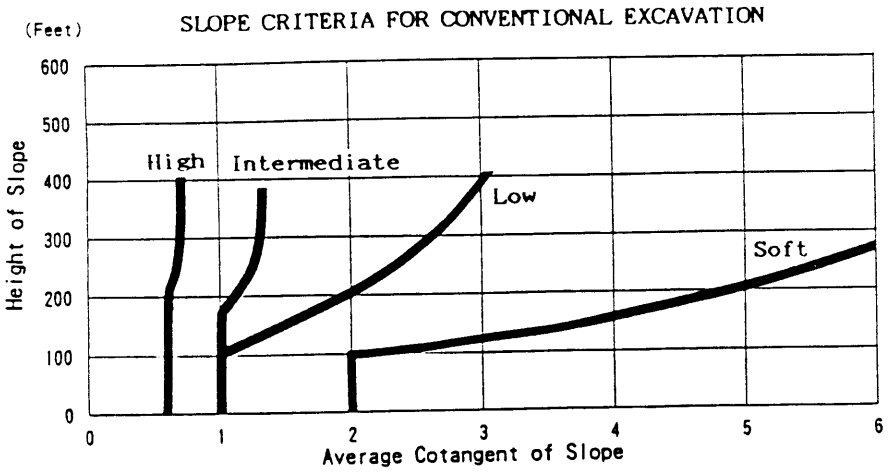


APPLICATION OF CRITERIA

HIGH QUALITY ROCK	BASALT (EXTRUSIVE) (INTRUSIVE)
INTERMEDIATE QUALITY ROCK	CHAGRES GATUN CAIMITO
LOW QUALITY ROCK	NONE IDENTIFIED
SOFT ALTERED VOLCANIC ROCKS AND CLAY SHALE	HIGHLY ALTERED AGGLOMERATES & TUFFS
SOILS	RESIDUAL SOIL ATLANTIC MUCK PACIFIC MUCK

Source: The Sea Level Canal Connecting The Atlantic and Pacific Oceans (KPTM, 1981)

Fig. II.1.1-9 Nagano Plan Slope Criteria



APPLICATION OF CRITERIA

Rock or Formation		Category of Quality
Route - 10	High Rise Lock Canal	
Basalt (extrusive) (Tb) (intrusive) Volcanic rocks (Tv) undifferentiated	Basalt (extrusive) (Tb) (intrusive) Dacite (Td) Pedro Miguel F. (Tpa) Bas Obispo F. (Tba)	High Quality Rock
Chagres F. (Tc) Toro limestone (Tct) Gatun F. (Tg) Marine rocks (Tue) Caimito F. (Tcm)	Gatun F. (Tg) Caimito F. (Tcm) Marine rocks (Tue) Bohio F. (Tbm)&(Tbo)	Intermediate Quality Rock
	Las cascadas F. (Tlc) La boca F. (Tl) Panama F. (TP)	Low Quality Rock
Severely altered agglomerate and tuff in (Tb) and (Tv)	Cucaracha F. (Tca) Severely altered agglomelate and tuff in (Tb) and (Tv) Culebra F. (Tcb)	Soft Rock
Debris (Overburden) Atlantic Muck Pacific Muck	Debris (Overburden) Atlantic Muck Pacific Muck	Earth

Fig. II.1.1-10 Proposed Criteria by ECE Contractor

1.5 Number of Ship Transits and Lock Flushes

1.5.1 Introduction

The annual number of ship transits and lock flushes are of particular importance in the Phase 2 Study since they define the lock operation frequencies used in both the water management studies and the lock gates fatigue design. Lock flush is defined as an action to flow out lockage water into the open sea.

1.5.2 Number of Ship Transits and Flushes

The annual number of ship transits and lock flushes adopted for the Phase 2 Study are shown in Table II.1.1.5 below.

The number of transits and the transit split between the existing and new locks, or between the existing canal and the sea level canal, were calculated based on information provided by CAS.

Table II.1.1-5 shows number of ship transits and lock flushes. More detailed explanations are included in Appendix 2 of this Report.

Table II.1.1.5 - Adopted Number of Annual Ship Transits and Lock Flushes for Phase 2 Study

Canal (DWT)	Locks	A.D. 2020		A.D. 2060	
		Flushes	Transits	Flushes	Transits
H.R.L. (150,000)	Existing Locks	32400	16200	40000	20000
	3rd Set of Locks	4000	2000	9200	4600
	Total Transits		18200		24600
H.R.L. (200,000)	Existing Locks	32400	16200	40000	20000
	3rd Set of Locks	4000	2000	9200	4600
	Total Transits		18200		24600
S.L.C. (250,000)	Existing Locks	32400	16200	38800	19400
	Tidal Lock	2100	2100	5100	5100
	Total Transits		18300		24500

Note : Number of lock flushes given in the table is the total number of flushes necessary for the stated number of transits to pass through the canal.

1.5.3 Water Volume in One Lock Flush

The volume of water required for one lock flush can be calculated from the formula given below:

$$V = B \times L \times DH$$
$$B \times L \times (H/N)$$

where,
V = volume of water required for one flush
B = width of lock in metres
L = length of lock in metres
DH = high of one lift (DH = H/N)
H = total rise
N = number of lifts

(assuming that all chambers are approx the same size)

1.5.4 Derivation of Number of Flushes Per Ship Transit

The number of flushes per ship transit was discussed and it was concluded that two flushes per ship transit for the High Rise Lock Canal would be suitable for the Phase 2 Study. It means the number of flushes for each of the High Rise Lock alternatives is equal to twice the number of allocated transits.

The number of lock flushes varies depending on the number of lifts per lock canal and whether traffic direction through the locks is one way or alternate. One way traffic means all vessels passing through a lane of locks are going in the same direction, ie. all northbound or all southbound. One way lockages are sometimes termed relay lockages. Alternate traffic means one northbound ship is followed by one southbound ship, then another northbound ship etc. Table II.1.1-6 below shows the results of analysis of traffic direction and number of lifts on water consumption per lockage.

Table II.1.1-6 Number of Flushes and Relative Water Consumption per Ship Transit

(a) Number of Flushes per Ship Transit

Traffic Direction	Number of Lifts 1	2	3
Alternate	1	2	3
One Way	2	2	2

Note: For the High Rise Lock Canal, see text below for flushes per transit.

(b) Relative Water Consumption per Ship Transit

Traffic Direction	Number of Lifts 1	2	3
Alternate	1	1	1
One Way	2	1	0.667

Note: Assumes total life is the same in all cases.

Since the proposed locks for the new High Rise Lock Canal will be two lifts on each ocean side, it can be seen from the above table that two flushes per transit are required regardless of whether the new locks are operated with one way or alternate traffic.

There are other factors which could affect the number of flushes required for each transit, including:

(a) Maintenance works on the locks,
(b) Trial gate operations,
(c) A switch in direction of traffic at Gatun locks,
(d) Tandem lockages where two vessels share a lock chamber, and
(e) Introduction of water saving methods such a the transfer of water between adjacent parallel lock chambers.

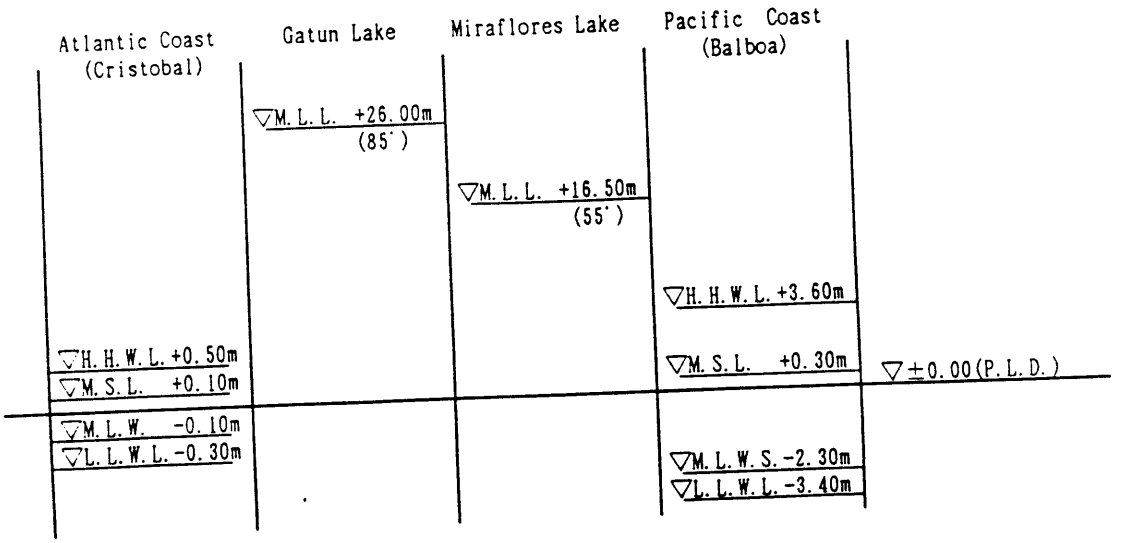
The first three factors would tend to increase water usage while the last two would reduce water usage, so that assumption of two flushes per transit still appears reasonable.

The assumption can be verified by checking existing water usage and transit records kept by PCC. Actual water usage does in fact approximate two flushes per transit.

The tidal locks is Sea Level Canal Case (a) and (c) have one lift only generally operating under one way flow, and the locks are on the Pacific side only. Therefore one flush per transit has been adopted, as shown in Table II.1.1-5.

1.6 Tide Levels and Level Datum

Tide levels and datum levels adopted for the Phase 2 Study are based on PCC information and are summarized in Fig. II.1.1-11. Throughout this report the Precise Level Datum (PLD) has been adopted as the datum for all levels, including the ocean approach channels. It should be noted that the datum levels for tidal elevations and hydrographic charts at the Pacific and Atlantic entrances are slightly different to the PLD datum. Adjustments have been made as necessary in calculating the earthwork volumes.



H.H.W.L. : Highest High Water Level
(Extreme High Water Level)

M.S.L. : Mean Sea Level

M.L.W. : Mean Low Water Level
(The Datum for Tidal Elevations and Hydrographic Charts at Atlantic Entrance)

M.L.W.S. : Mean Low Water Springs
(The Datum for Tidal Elevations and Hydrographic Charts at Pacific Entrance)

L.L.W.L. : Lowest Low Water Level
(Extreme Low Water Level)

M.L.L. : Mean Lake Level

P.L.D. : Precise Level Datum as adopted by the Panama Canal Commission

Source: The Panama Canal Pilot, by Captain George A. Markham, Chief of Maritime Training, Panama Canal Commission, 1990 (page 462).

Fig. II.1.1-11 Tide Levels and PLD Level Datum

1.7 Anchorages, Emergency Tie-Up Stations and Moorings

General proposals for anchorages, emergency tie-up stations, and moorings are summarized below. For a more detailed description refer to Appendix 2.

1.7.1 Anchorages

Ships awaiting passage require safe anchorage close to the canal entrance and in sufficient, but not excessive, depths of water. The anchorage area will be determined by the number of vessels to be accommodated and the swinging radius of each ship about its anchor. The swinging radius will increase with the depth of water.

A spacing between anchoring position of 2.5 times the swinging radius has been assumed. This will allow for errors in positioning the anchor and differences in the amount of chain which is veered.

Predicted traffic includes vessels from less than 100,000 DWT to 250,000 DWT with drafts varying from 8m to 21m. These have been grouped into three categories based on overall length for the purposes of estimating the area required for anchorage. The resulting spacings between anchorage positions for the three ship sizes chosen are defined in Table II.1.1-6 below. These spacings are used in Parts III and IV to determine the anchorage area requirements for the High Rise Lock Canal and Sea Level Canal respectively.

Table II.1.1-6 Spacing of Anchoring Points

Max Ship Size(DWT) Group	40,000 A (m)	100,000 B (m)	200,000 C (m)
Ship Draft	11.5	16.5	21
Ship Length	200	300	330
Min water depth	15	21	27
Max depth(Pacific H.Tide)	20	26	32
Anchorage Spacing(Atlantic)	877	1196	1328
Anchorage Spacing(Pacific)	932	1243	1372

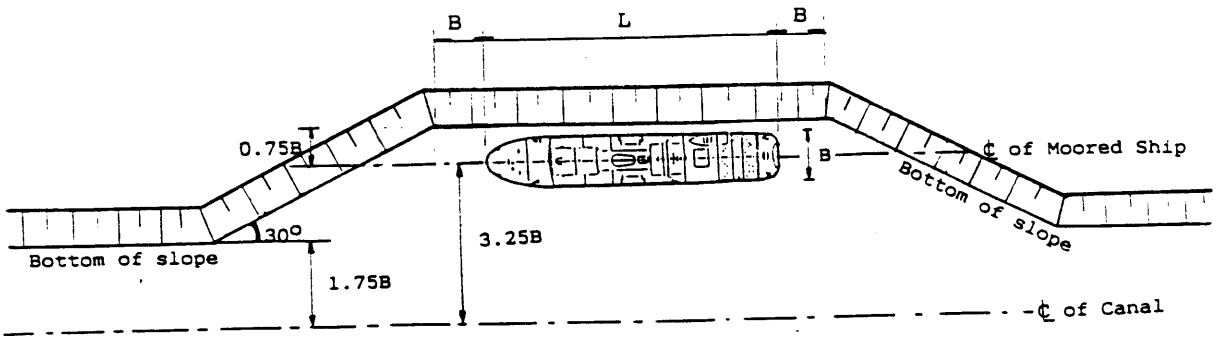
1.7.2 Emergency Tie-Up Stations

Provision must be made to enable a disabled ship to be removed from the navigation channel to minimise the disruption which it would otherwise cause to canal traffic.

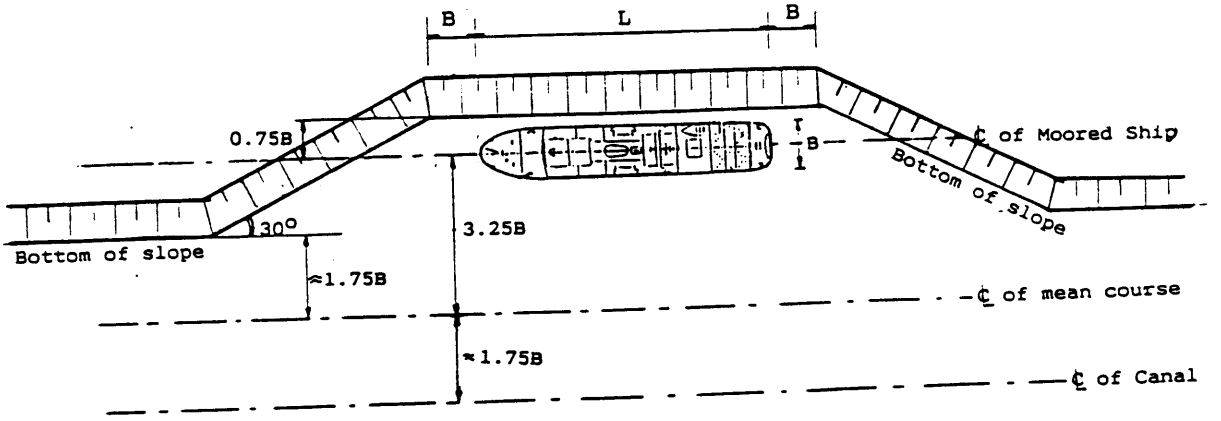
Passing ships induce forces on a nearby moored vessel which may be large enough to break moorings if the separation of the vessels is small. An emergency tie-up station (sometimes termed emergency lay-by) needs to be large enough to accommodate the design vessel without excessive capital cost yet far enough from the nearest navigation lane for mooring forces to remain within the capacity of the design ship's normal equipment.

For each tie-up station, two breasting dolphins against which the ship should be provided. The breasting dolphins would be any structure designed to resist the forces exerted by the ship during and subsequent to the act of mooring. Usually a heavy concrete deck on raking piles is provided. In addition, an on-shore mooring bollard should be provided near each end of a moored ship.

The proposed layouts for emergency tie-up stations beside single and two lane canals are shown in Figure II.1.1-13. Further explanation of the arrangements shown is given in Appendix 2.



(a) Single Lane Canal



(b) Two Lane Canal

Fig. II.1.1-12 Layout of Emergency Tie-Up Station

II.1 CHAPTER 2 - CANAL ALIGNMENT

2.1 Canal Alignments

Coordinated alignments for the Sea Level Canal and the High Rise Lock Canal were provided by the Commission for the Phase 1 Study. These alignments have been reviewed in detail using the available 1:25,000 and 1:50,000 scale mapping.

For the calculation of earthworks, preliminary criteria have been established for canal curvature at bends in the canal. To allow for ship navigation at the bends a radius of five and a half (5.5) times the length of the design ship has been inserted where the deflection angle is 30° or more. This situation only occurs on the High Rise Lock Canal alignment and the 200,000 DWT ship length of 333m results in a centerline radius of 5.5 x 333m = 1832m.

Minor refinements of the high rise lock canal alignment in Gatun Lake, and the sea level canal alignment considering curvature of the waterway, were proposed by the ECE contractor at the Inception Report meeting and were accepted by the Secretariat. The revised intersection point coordinates are shown in Tables II.1.2-1 and 2.

The alignment of the High Rise Lock Canal is shown in the 1:50,000 fold out maps which follow. Drawings for the Sea Level Canal are also provided; canal widths shown are for Case (b) but the centerline for Case (a) and Case (c) is the same.

The alignment and modifications are described below.

Table II.1.2-1 High Rise Lock Canal Alignment

Point	Station No. (km)	Intersection Angle(°)	Coordinates North (km)	Coordinates East (km)	Distance (km)
A	-12.50		1,040.00	618.70	10.40
B	-2.10	10	1,029.60	618.70	6.39
C	4.29	35	1,023.30	619.75	3.49
D	7.78	28	1,020.20	618.15	2.00
E	9.78	60	1,018.20	618.20	6.74
F	16.52	52	1,014.85	624.05	2.96
G	19.48	62	1,015.95	626.80	4.30
H	23.77	39	1,013.20	630.10	5.32
I	29.09	67	1,008.00	631.20	4.28
J	33.37	50	1,007.20	635.40	1.42
K	34.79	30	1,008.10	636.50	1.02
L	35.81	19	1,008.30	637.50	4.96
M	40.77	20	1,007.50	642.40	1.57
N	42.34	32	1,006.80	643.80	2.87
O	45.20	10	1,004.30	645.20	1.71
P	46.91	18	1,002.70	645.80	0.81
Q	47.72	21	1,002.00	646.20	1.44
R	49.16	21	1,001.20	647.40	2.50
S	51.66	10	999.20	648.90	10.68
T	62.34	24	991.50	656.30	2.69
U	65.03	18	989.00	657.30	15.20
X	80.23		977.30	667.00	

Table II.1.2-2 Route 10 Sea Level Canal Alignment

Point	Station No. (km)	Intersection Angle(°)	Coordinates North (km)	Coordinates East (km)	Distance (km)
A	-2.70		1,024.40	599.00	12.61
B	9.91	15	1,014.30	606.55	3.00
C	12.91	15	1,011.55	607.75	5.34
D	18.25	16	1,007.85	611.60	6.53
E	24.78	12	1,004.95	617.45	5.19
F	29.97	16	1,003.55	622.45	4.17
G	34.14	8	1,001.45	626.05	3.31
H	37.45	13	999.40	628.65	3.51
I	40.96	14	996.75	630.95	9.52
J	50.48	15	988.00	634.70	3.51
K	53.99	16	985.30	636.95	5.63
L	59.62	22	982.20	641.65	5.25
M	64.88	29	977.50	644.00	21.91
N	86.79		965.00	662.00	

(1) High Rise Lock Canal Alignment

The basic principle in the design of the High Rise Route has been the maximum use of the existing canal route. In those places where a sharp deflection angle is required, making navigation difficult, the inside bank will be widened by the adoption of a curved design, as described above.

(a) Atlantic Approach Zone
(No.-12.80 km - No.2.00 km)

The existing approach is used between Point A and Point B while the section between Point B and Point C follows the alignment of the former excavation site for the Gatun 3rd Lock. Although this plan involves a partial loss of residential areas near Gatun Lock, it has been selected because of such advantages as the minimization of residential property loss and the minimum excavation work required due to the use of the former excavation site.

(b) Gatun Lake Zone
(No.2.00 km - No.42.00 km)

The section between Point C and Point D is a new passageway linking the existing route and the new route. Although the passageway joins the B-C line at an angle of 35°, widening at the intersection of the canals will ensure smooth navigation. The section between Point D and Point F is along the existing canal while the section between Point F and Point H avoids Barro Island and De Lesseps Island, both of which are subject to environmental conservation as recommended by the Smithsonian Tropical Research Institute. This environmental consideration makes excavation of the ground on the outside of the curve near Point G necessary. The section

between Point H and Point N again follows the existing canal, but widening is basically on the south side only.

(c) Culebra Zone
(No.42.00 km - No.52.00 km)

The section between Point N and Point S follows the existing canal. The present northeastern bank of the canal remains unchanged while the southwestern bank will be pushed back to widen this section.

(d) Miraflores Zone
(No.52.00 km - No.60.00 km)

The section between Point S and Point T is to the south of the present route, avoiding Miraflores Lake so that traffic through the present canal will not be disrupted. Only a small part of the 3rd Lock, the former excavation site near Miraflores can be used due to the requirement to separate the new canal and Miraflores Lake. This is necessary because of the significant water level differences between Miraflores Lake (55' WL) and the new canal (85'-90' WL).

(e) Pacific Approach Zone
(No.60.00 km - No.79.20 km)

The section between Point T and Point X basically follows the existing approach although the alignment of the section has been slightly modified to make it straighter. Along this section, the southwestern bank will be pushed back to increase the width while the northeastern bank remains unchanged. Some of the existing facilities on the southwestern bank of the canal will require modification to accommodate this widening of the canal.

(2) Sea Level Canal Alignment

Following the Inception Report meetings, an aerial survey using a helicopter was conducted to fly over the alignment proposed in Phase 1. The flight confirmed the suitability of the alignment adopted in Phase 1. Observation of the surrounding areas confirmed that any significant shift to adjacent terrain would be very unlikely to lower construction costs. The navigability has been properly considered and every effort has been made to ensure the alignment passes through lowland as much as possible. Consequently, it was decided that the alignment proposed in Phase 1 would be adopted in Phase 2.

(a) Atlantic Approach Zone
(No.-2.80 km - No.0.50 km)

This section is some 3 km in length and for easy navigation if follows the same bearing as the section from the Atlantic shoreline to Point B.

(b) Quebrado Zone
(No.0.50 km - No.17.00 km)

The section between the Atlantic shoreline and Point C follows Quebrado River from its mouth. In order to ensure good navigability, a straight alignment has been adopted. About 4.5 Km of this section passes through a hills but, while routes requiring less excavation might be possible, they would be undesirable from a navigation point of view. The section between Point C and Point D is a short route to Gatun Lake and follows an area of gentle topography.

(c) Gatun Lake Zone
(No.17.00 km - No.38.00 km)

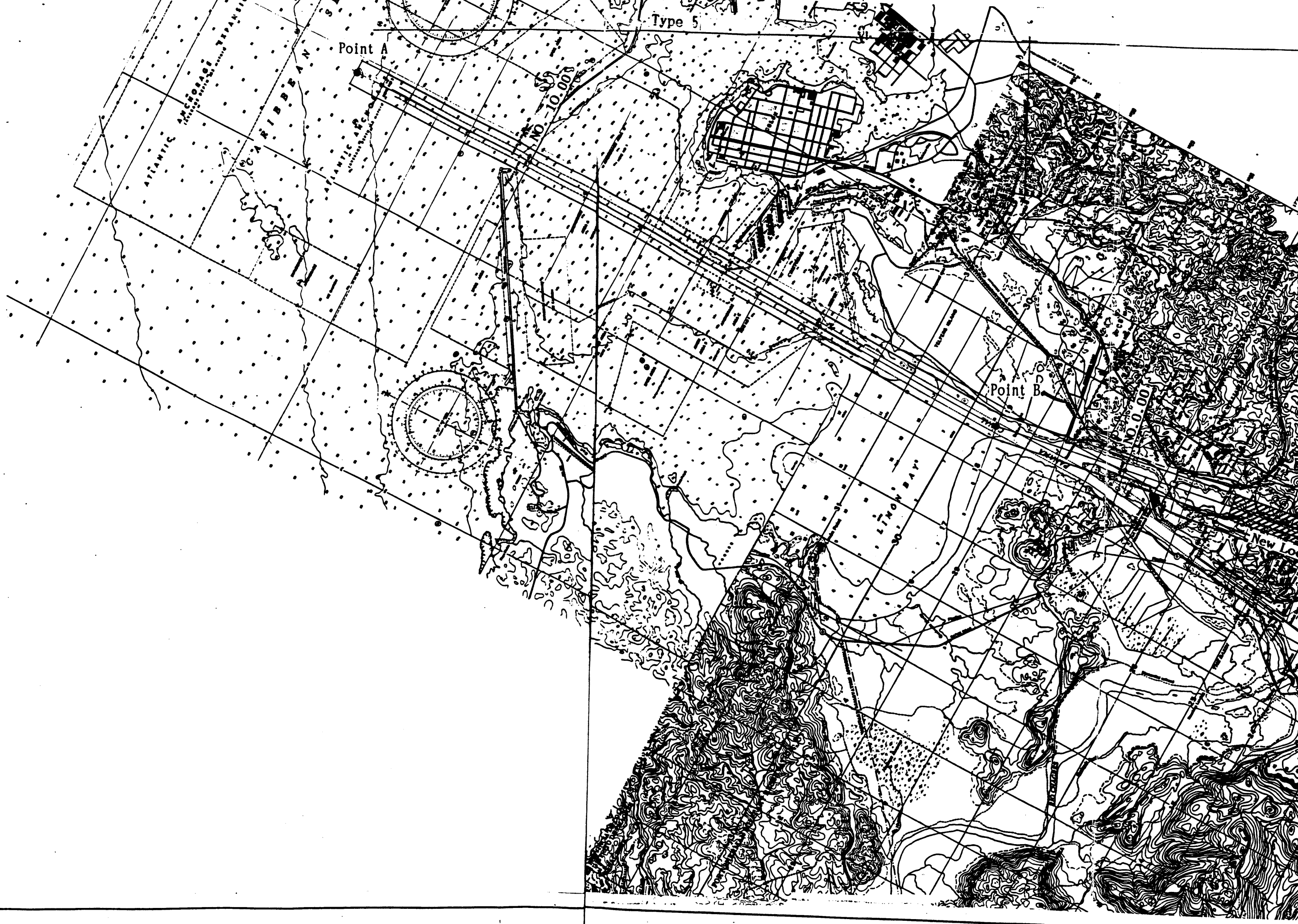
The section between Point D and Point H passes through Gatun Lake and Gigante Valley, reaching the mouth of Pescado River.

(d) Galboa Zone
(No.38.00 km - No.60.50 km)

The section between Point H and Point J follows alongside Pescado River and the section between Point J and Point L follows alongside Caimito River, ending at the mouth of Caimito River.

(e) Pacific Approach Zone
(No.60.50 km - No.85.50 km)

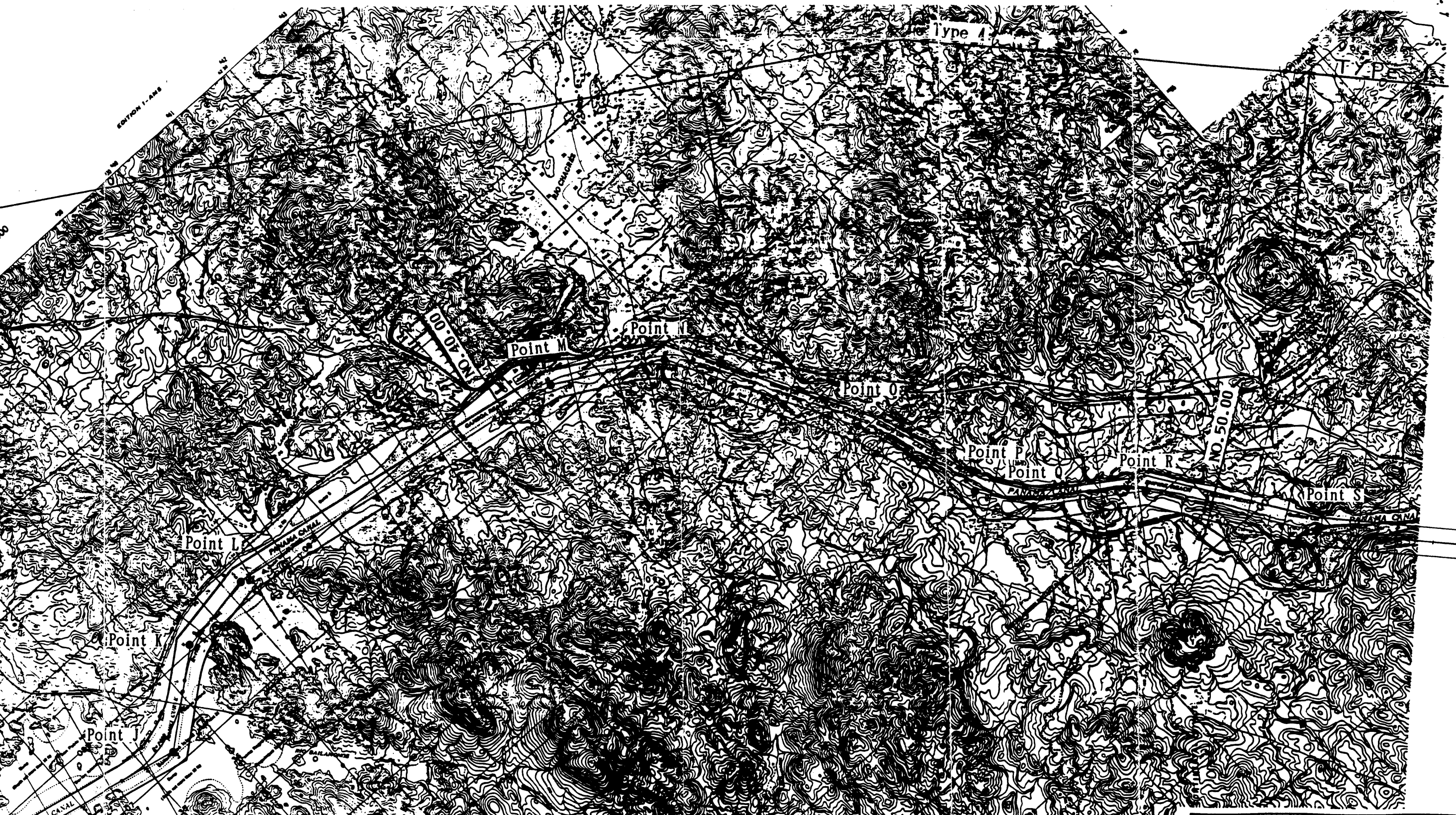
The section between Point L and Point N is planned to ensure the minimum length of the Pacific Approach.


Type 5
Point A
Point B
NO. -10.00
NO. 0.00
CARIBBEAN
ATLANTIC ANCHORAGE
LIMON BAY

Type 1

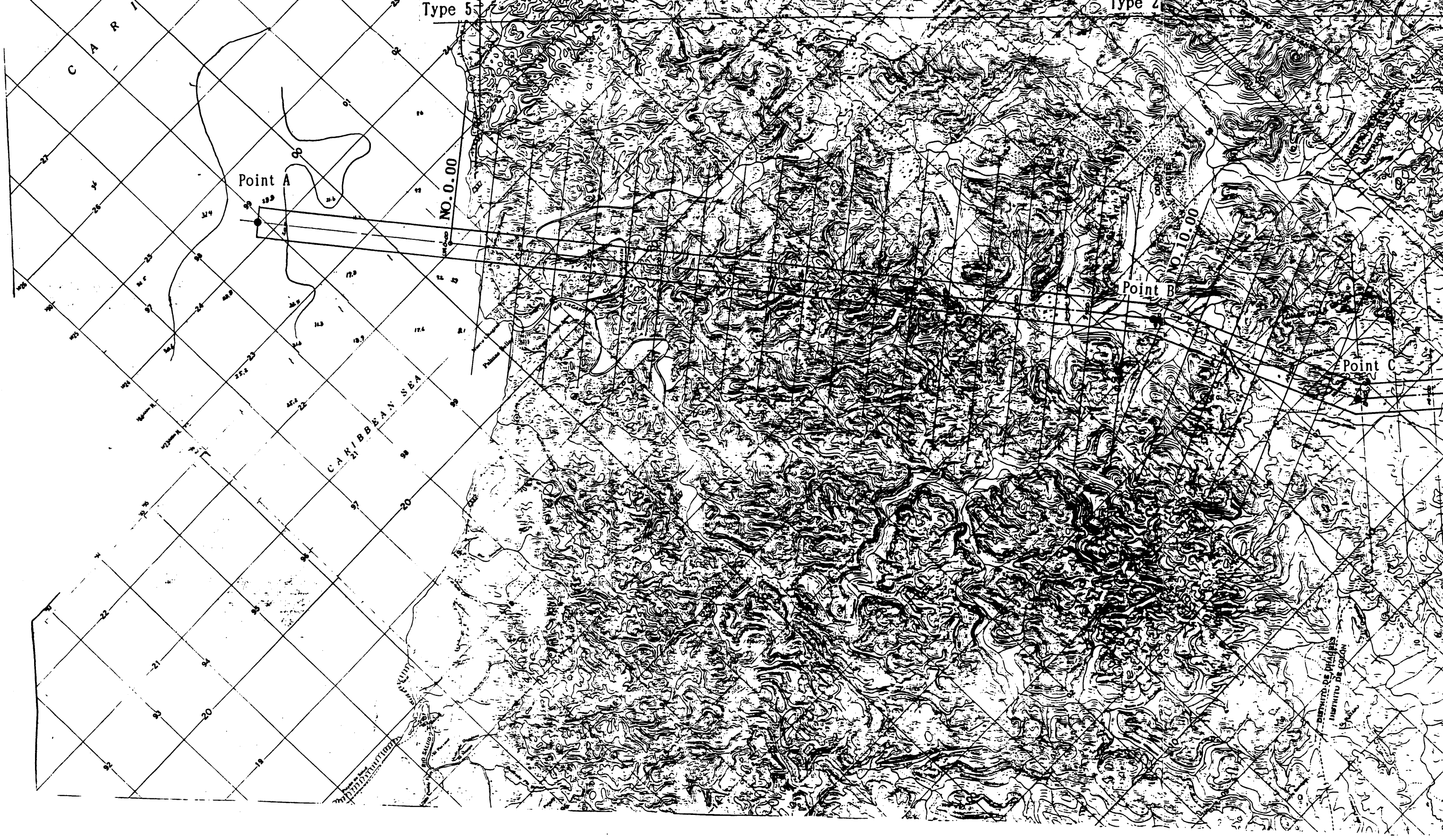
THE STUDY OF ALTERNATIVES TO THE PANAMA CANAL			
TITLE	HIGH RISE LOCK CANAL ALIGNEMENT		
DATE	April .1993	SCALE	1/50,000
YACHIYO ENGINEERING CO., LTD.			D.W.G NO. 1/3

PROVINCE OF COLON
DISTRICT OF COLON
EDITION 1-AMS
Point H
Point I
Point J
Point K
Point L
Point M
Point N
Point O
N0.30.00
N0.40.00
PANAMA CANAL
SALIDO BAY
RIO CHAGRES



THE STUDY OF ALTERNATIVES TO THE PANAMA CANAL			
TITLE	HIGH RISE LOCK CANAL ALIGNEMENT		
DATE	April .1993	SCALE	1/50,000
YACHIYO ENGINEERING CO., LTD.			D.W.G NO.

Type 3
Type 2
Type 5
PANAMA CANAL
NO. 60.00
Point T
New Locks
Point U
NO. 70.00
UNITED STATES NAVAL AMMUNITION DEPOT
HOWARD AIR FORCE BASE


Type 5
Type 2
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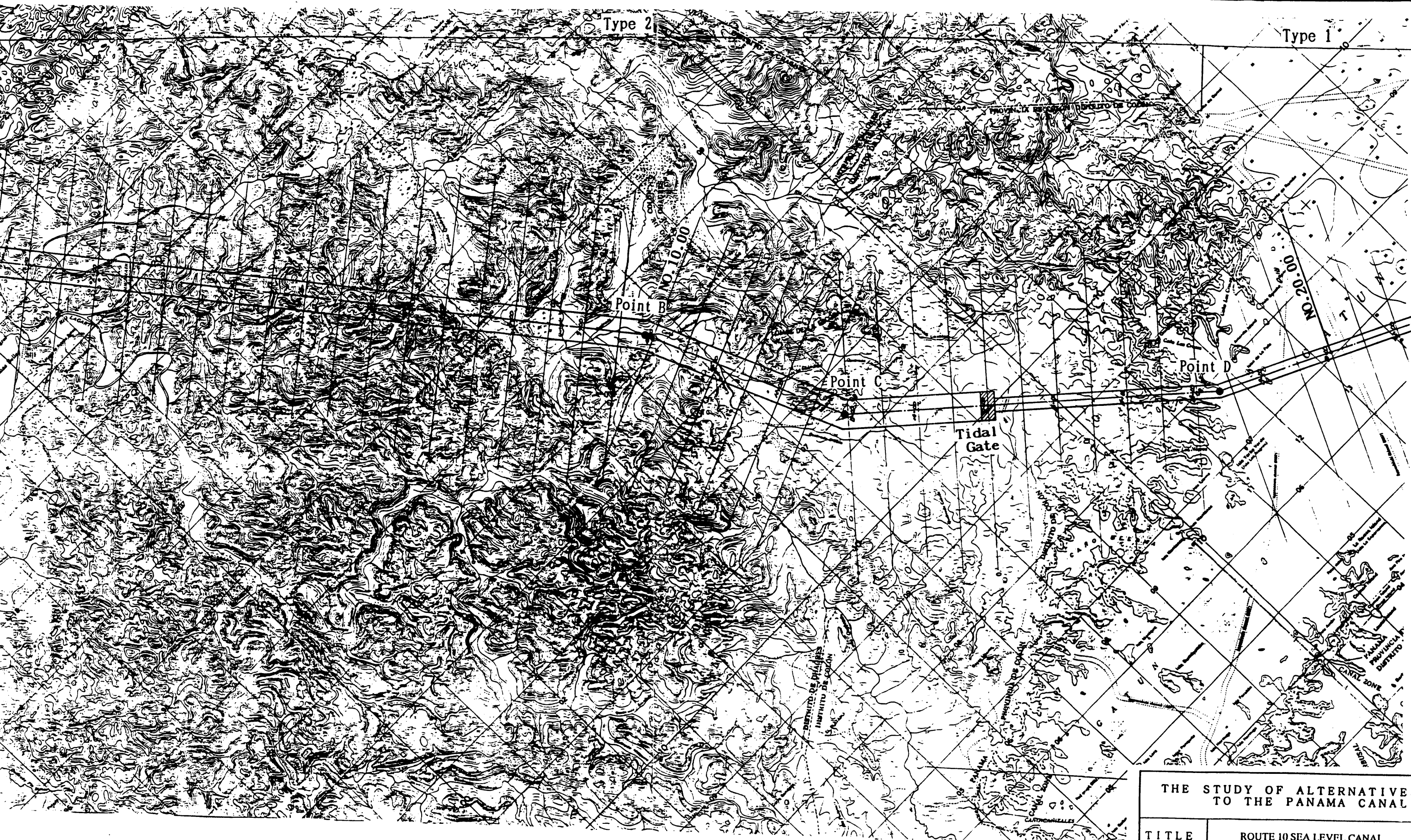
NO. 0.00
NO. 10.00
Point B
Point C
CARIBBEAN SEA
DISTRITO DE CHAGRES
DISTRITO DE COLON



THE STUDY OF ALTERNATIVES TO THE PANAMA CANAL			
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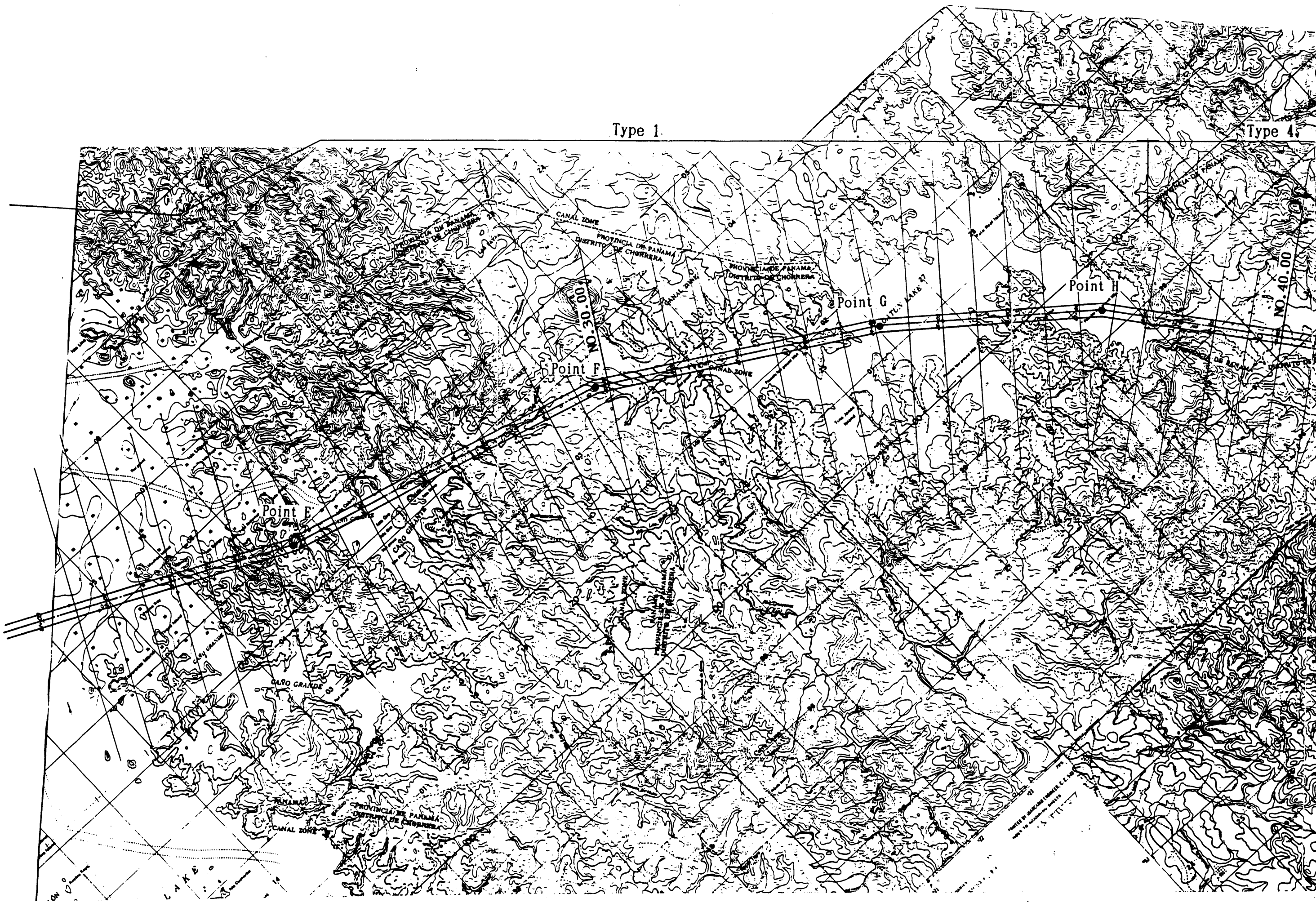
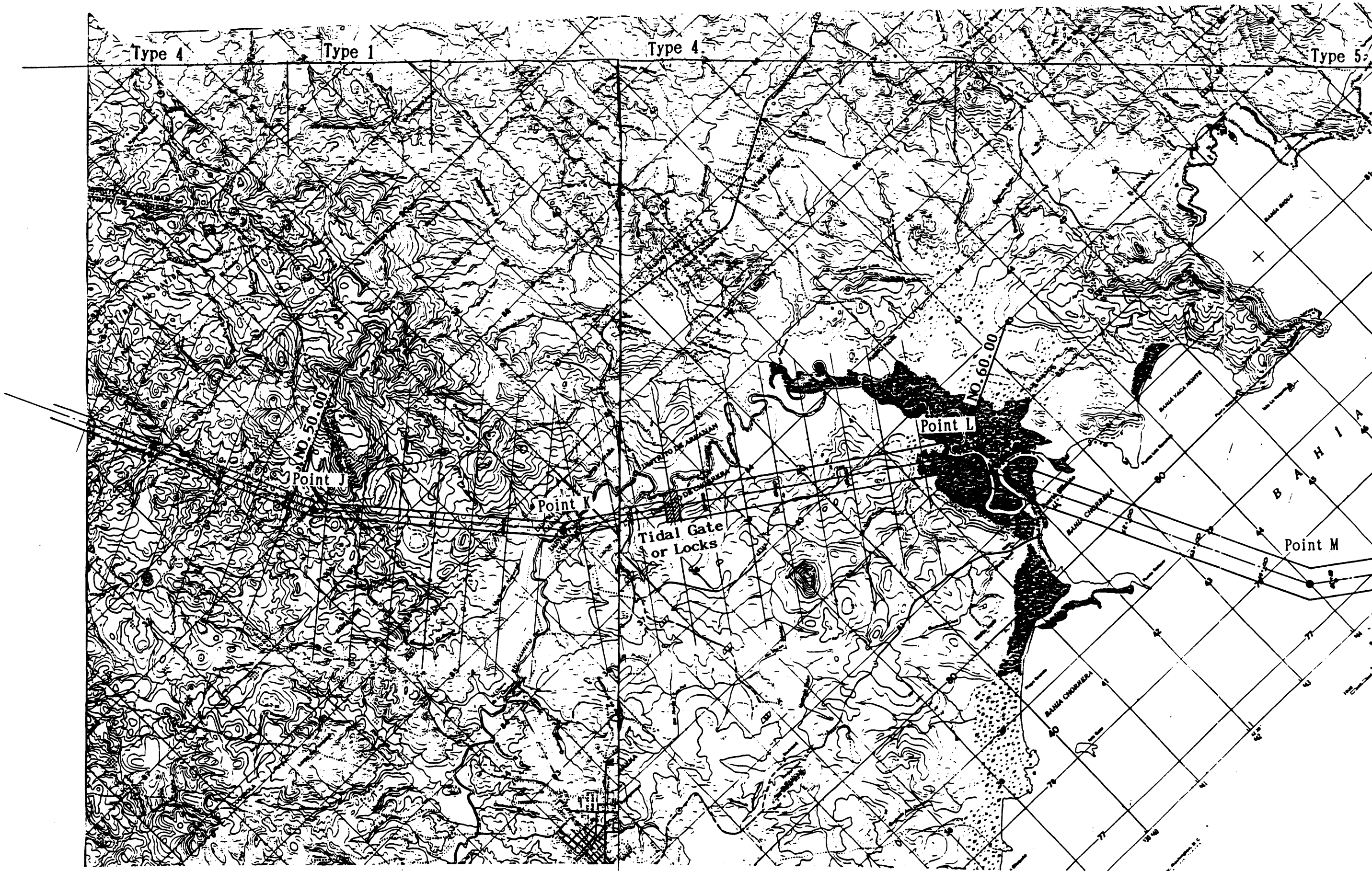
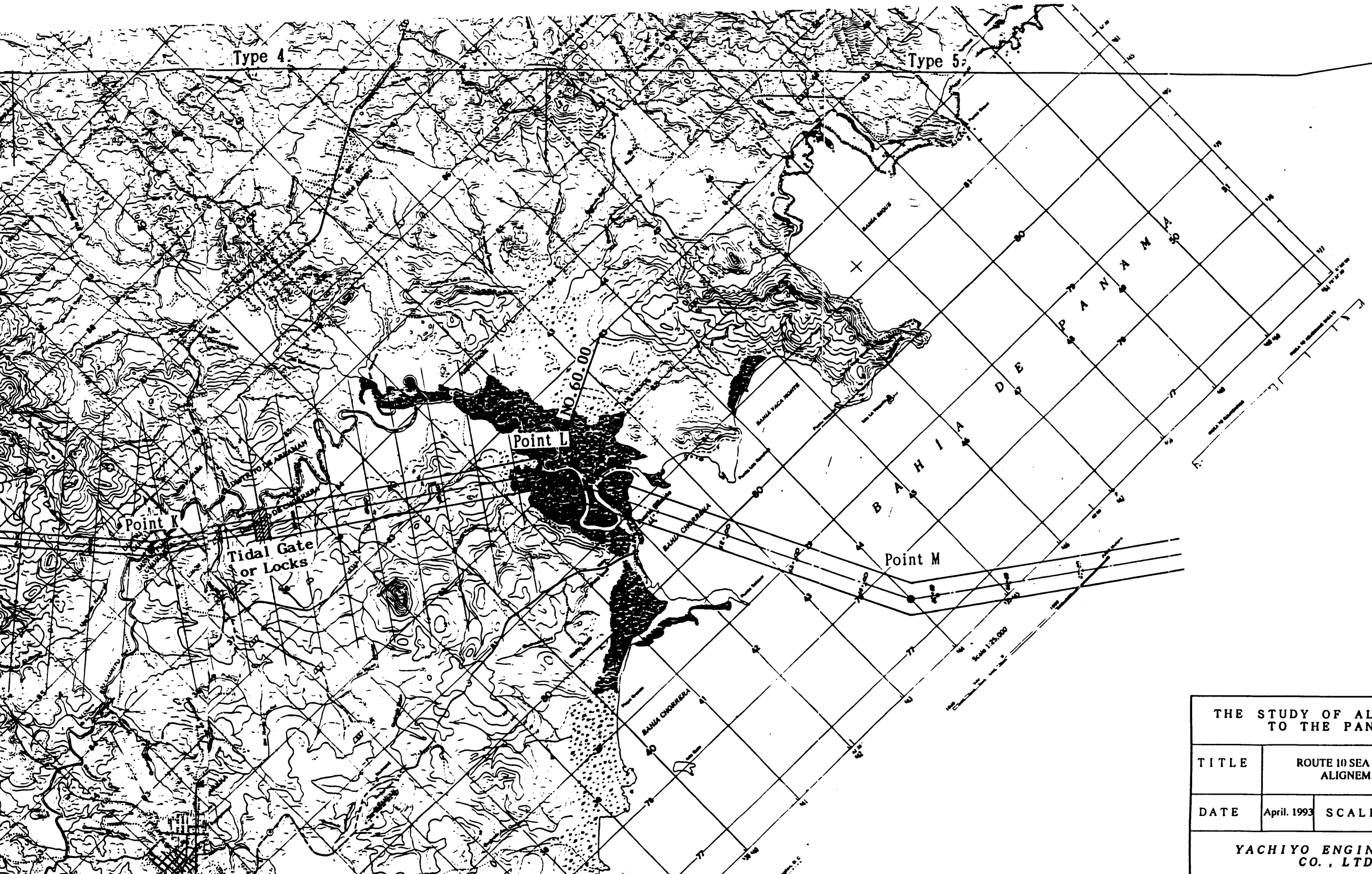
DATE	April. 1993	SCALE	1/50,000
YACHIYO ENGINEERING CO., LTD.			D.W.G NO.


Type 1
Type 4
Point E
Point F
Point G
Point H
NO. 30.00
NO. 40.00
CANAL ZONE
PROVINCIA DE PANAMA
DISTRITO DE CHORRERA
CAÑO GRANDE
CAÑO URACUE
LAKE


Type 4
Type 1
Type 4
Type 5
NO. 50.00
NO. 60.00
Point J
Point K
Point L
Point M
Tidal Gate
or Locks
BAHIA
BAHIA CHORRERA



THE STUDY OF ALTERNATIVES TO THE PANAMA CANAL			
TITLE	ROUTE 10 SEA LEVEL CANAL ALIGNEMENT		
DATE	April. 1993	SCALE	1/50,000
YACHIYO ENGINEERING CO., LTD.			D.W.G NO. 3/3

II.1 CHAPTER 3 - SEISMOLOGY

3.1 Design Seismic Coefficient

The design seismic coefficient was calculated in accordance with the "Technical Standards for Port and Harbor Facilities in Japan" by OCDI.

$$Kh = k * b * c$$

where: Kh = Design seismic coefficient
k = Regional seismic coefficient
b = Factor for subsoil condition
c = Coefficient of importance

However, for the design seismic coefficient, only the horizontal coefficient is considered unless otherwise required specifically from the structure. For the regional seismic coefficient, the values are from 0.05 to 0.15 in Japan.

For the factor for subsoil condition, the value is shown in Table II.1.3-1. Subsoil classes shall be determined according to Table II.1.3-2 in consideration of the thickness of the quarternary deposits such as alluvium and diluvium.

The coefficient of importance shall be determined in accordance with the values in Table II.1.3-3 depending on the degree of importance of the structure concerned.

Table II.1.3-1 Factors for Subsoil Condition

Subsoil Class	Class 1 Subsoil	Class 2 Subsoil	Class 3 Subsoil
Factor	0.8	1.0	1.2

Table II.1.3-2 Subsoil Classes

Thickness of Quarternary Deposits	Sand Gravel Layer	Ordinary Sand, Clay Subsoil	Poor Subsoil
Less than 5m	Class 1	Class 1	Class 2
5−25m	Class 1	Class 2	Class 3
More than 25m	Class 2	Class 3.	Class 3

Table II.1.3-3 Coefficients of Importance

Classification of Structure	Character of Structure	Coefficient of Importance
Special Class	Structures of which the chractors ①−③ are strongly evident among the characters in Class A.	1.5
Class A	① Structures tending to cause loss of life and property upon seismic damage. ② Structures playing an important role in recovery from earthquake disaster. ③ Structures handling hazardous materials and tending to cause serious damage to life or property upon seismic damage. ④ Structures causing serious influences on the economic and social activities of the areas concerned upon seismic damage. ⑤ Structures of which considerable difficulty is expected for recovery upon seismic damage.	1.2
Class B	Structures other than the Special Class, Class A and Class C.	1.0
Class C	Small structures other than the Special Class and Class A, and permitting recovery with ease.	0.5

The regional seismic coefficient (k) proposed for the studies in Phase 2 is k=0.10, taking into consideration the historical seismic data recorded on 7 September 1882. The proposed factor for subsoil condition is b=0.8 corresponding to subsoil class 1. The coefficient of importance is c=1.2, assuming class A importance. Thus, the design seismic coefficient to be applied becomes kh=0.1 from the formula mentioned above.

3.2 Earthquake in Panama

The strongest earthquake to hit Panama occurred on 7th September 1882.

This earthquake is reported in detail in "The North Panama Earthquake of 7 September 1882", by C Mendoza and S Nishenko - Bulletin of Seismological Society of America, Vol 79.

Other earthquake records on Panama are likewise reported in "Some Aspects of the Seismic Hazard in Panama City" by Vergara Munosz - Natural Hazards 3. 1990. And IOCS Memorandum PCC-3.

Fig. II.1.3-1 shows isoseismal contours inferred for the 1882 earthquake from the Modified Mericalli intensities. Contour lines separate zones of equal intensity during the 7 September 1882 earthquake. The principal tectonic features presently known for the Panama-Colombia region are shown in the inset map.

-Source- "The North Panama Earthquake of 7 September 1882" by C Mendoza and S Nishenko - Bulletin of Seismological Society of America, Vol 79.

Table II.1.3-4 shows a comparison between the Modified Mericalli Scale and seismic acceleration as a proportion gravity. -Source- IOCS Memorandum PCC-3.

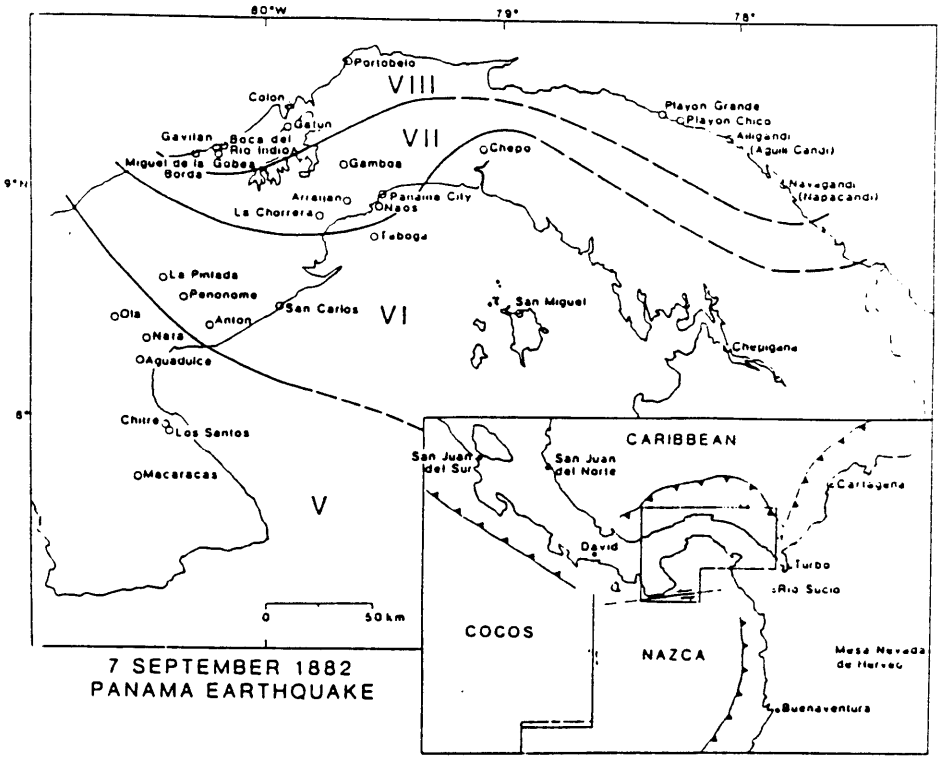


Fig. II.1.3-1 Panama Earthquake (7 Sep. 1882)

Table II.1.3-4 Seismic Coefficent

Modified Merical Scale	Acceleration as Proportion of Gravity
I	0.0001
II	0.0005
III	0.001
IV	0.002
V	0.01
VI	0.02
VII	0.05
VIII	0.10
IX	0.15
X	0.31

Source: IOCS Memorandum PCC-3

II.1 CHAPTER 4 - HYDROLOGY AND EXISTING WATER SUPPLY

4.1 Rainfall

Based on the hydro-meteorological data, the climate of Panama can be categorized into two seasons. January through April is considered the dry season, and May through December, the wet season. About 90% of the total annual rainfall occurs during the wet season, and the total annual rainfall on the Atlantic coast is about 150% of that for Pacific coast. The annual average rainfall for the Panama Canal Watershed is approximately 2,500mm. (see Fig. II.1.4-1 and Table II.1.4-1)

4.2 Evaporation

The amount of monthly evaporation in the watershed is given in the table below (Ref. IOCS JAX-50 Page 20).

(UNIT : *mm*)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
Gatun	140	149	166	156	109	93	89	95	90	94	89	114	1,384
Madden	137	145	162	153	107	91	87	93	88	92	87	112	1,354

4.3 Watershed Yield

The water supply for the operation of the Panama Canal is provided by a 3,339 sq.km drainage area of Gatun and Madden dams. The average annual yield for 51 years of record(1933-1983) according to IRHE (Instituto de Resursos Hideraulicos y Electrification) is about 6,593 million m^3. For the last 10 years, maximum and minimum yields of 9,948 and 4,892 million m^3 occurred in the years 1981 and 1982, respectively. Annual evaporation is about 670 million m^3. Therefore, the net annual average yield of the watershed would be about 5,923 (ie. 6,593-670) million m^3.

For some variations, the watershed yield of the Gatun and Madden catchments is insufficient to meet requirements (6,303 million m^3 for 200,000 DWT case for an average year by 2060) and the introduction of water from outside the catchment is necessary. A suitable dam site, on the Indio River, was identified in Phase 1. The watershed yield of the Indio river at the proposed dam site would be about 591, 461 and 988 million m^3 in average, dry and wet years respectively.

4.4 Sediment Yield

Sediment observations are not available for the canal area, but information about sediment yield is available in the IOCS Report(JAX-50). According to that report, the design sediment yield is estimated at about 500 $m^3/km^2/year$. Judging from land conditions, vegetation and rainfall compared to similar projects in other countries, this value seems reasonable for this study.

4.5 Design Flood

The maximum discharge data for the study area is available from 1979 to 1989 at BOCA DE URACILLO of Indio river. According to the Japanese Design Standard of Fill Dam, the value of 200 years probability multiplied 1.2 is adopted for the design flood. The probable flood discharge is calculated based on the Gumbel and Iwai methods, and the design flood of 285 $m^3/sec/100\ km^2$ was adopted by averaging the result of the two methods.

4.6 Storage Capacity of Existing Dams

The effective storage capacity in Gatun Lake is about 659 million m^3 between elevations ranging from 25.1m (82') to 26.6m (87') while the effective storage capacity in Madden Lake is about 539 million m^3 between elevations ranging from 61m (200') to 76.2m (252'). The total usable storage capacity is 1,198 million m^3. (see Fig. II.1.4-3)

4.7 Existing Water Use

Gatun and Madden lake water is utilized as lockage water, municipal water and generating hydroelectric power for Panama City, the Canal Area and Balboa. There are significant losses due to evaporation and spillage when runoff exceeds storage capacities. According to the PCC annual report (1991), the water use in fiscal year 1990 from these two lakes are as follows:

Item	Volume(MCM)	Percent(%)
Lockages	2431	46
Municipal	289	5
Evaporation	589	11
Power	1715	33*
Spillage	253	5
Total	5277	100

* does not include Madden Hydroelectric power

Under the assumptions to be adopted for the water management, water for power and spillage will be given the lowest priority.

Therefore the total volume used for power or spilled will be made available to provide additional lockage water.

4.8 Monthly Discharge and Design Years

The available data period of monthly average discharge data is from 1933 to 1983 in the Gatun watershed, and from 1958 to 1990 in the Indio watershed. In the water balance simulation, the monthly discharge pattern and three different annual discharges, ie. (1) average year, (2) dry year and (3) wet year, were defined as follows to evaluate the capacity of dams and pumps.

(1) Monthly Discharge Pattern

There are six gauging stations (Trinidad, Gatun, Ciri, Chagres, Boqueron and Pequeni) in the Gatun watershed. The pattern of monthly discharge was determined based on the yield of Gatun watershed by averaging the discharge each month for the six stations mentioned above. This monthly discharge pattern defines the runoff characteristics in the study area.(See Fig. II.1.4-2 and Table II.1.4-2)

(2) Design Year (Average, Dry and Wet year)

The average year is defined as the average yield over 51 years of record (1933-1983). From records over the last 10 years the year 1982, corresponding to the minimum yield, was selected as the dry year and 1981, corresponding to the maximum yield, was selected as the wet year. The distribution of monthly discharge in a dry year and wet year was calculated based on the above pattern. Generally the dry year controls the requirements for dams and pump stations.

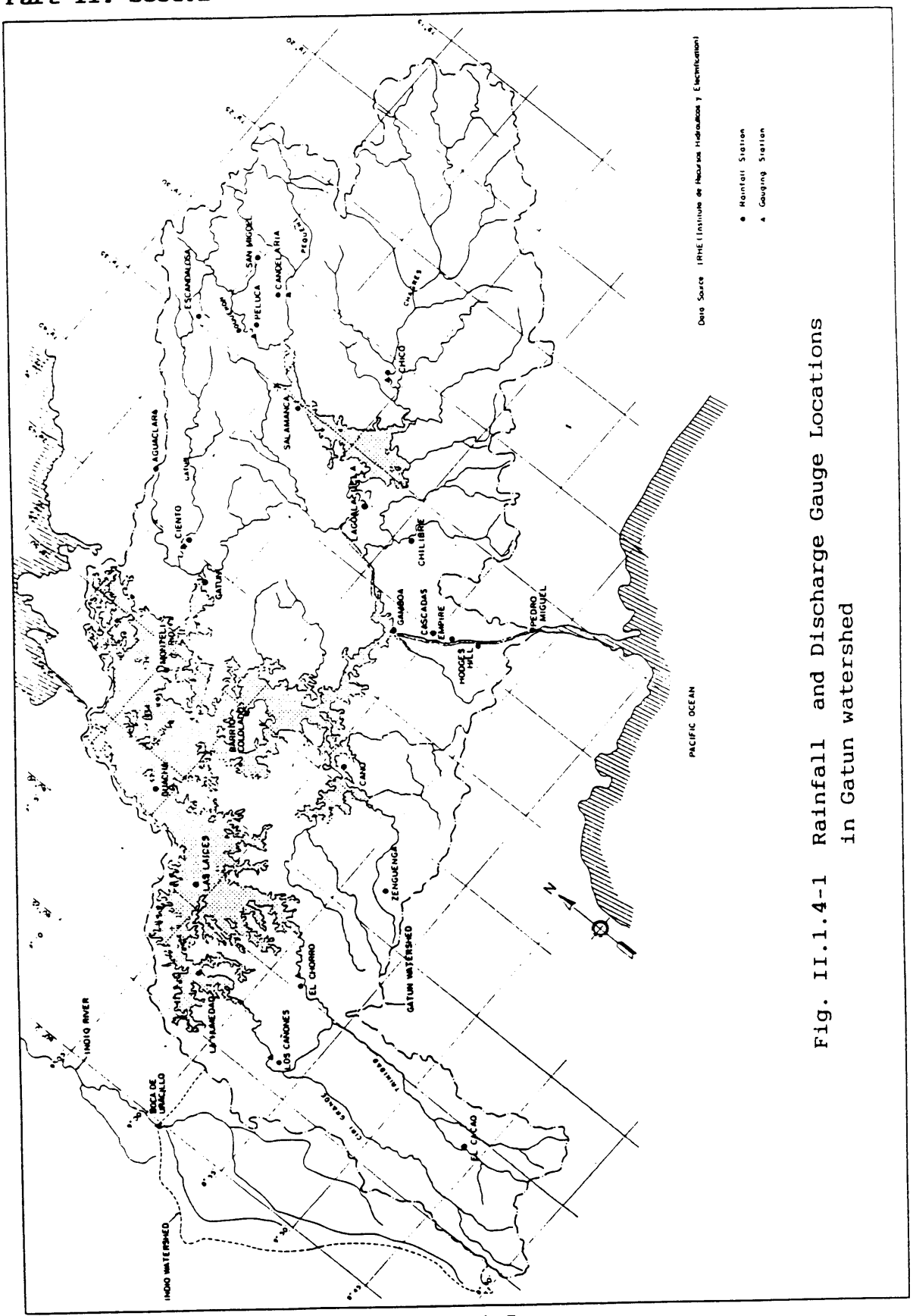


Fig. II.1.4-1 Rainfall and Discharge Gauge Locations in Gatun watershed

Table II.1.4-1 Average Rainfall in Gatun Watershed

(Unit : mm)

Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
Agua Clara	108.3	54.6	55.4	157.1	371.0	335.8	319.8	374.2	325.6	512.9	539.4	348.2	3,502.3
Barrio Colorado	65.3	31.8	27.1	93.7	274.5	263.5	271.3	306.8	259.1	347.4	421.1	242.8	2,604.4
Candelaria	114.7	57.7	45.3	165.8	336.5	341.5	330.7	350.2	320.4	350.2	388.9	302.3	3,104.2
Chico	39.6	14.4	13.3	83.6	278.8	310.0	317.5	337.5	322.3	387.6	345.3	174.1	2,624.0
Ciento	100.8	11.1	38.0	129.6	314.3	307.7	311.7	336.4	13.7	501.7	501.7	304.1	2,870.8
El Chorro	68.7	30.0	29.0	93.0	266.5	245.1	197.3	260.6	276.8	326.0	313.6	167.3	2,273.9
Escandalosa	133.9	78.6	65.2	204.7	351.6	338.2	343.5	320.5	283.2	333.6	436.9	313.6	3,203.5
Salamanca	10.9	13.8	15.2	93.5	251.7	286.3	256.7	287.7	275.9	326.6	331.8	167.3	2,317.4
San Miguel	152.1	74.4	74.8	222.4	379.6	354.8	352.1	358.3	324.4	348.4	424.9	356.4	3,422.6
La Humedad	71.8	39.8	31.1	115.9	252.7	247.0	194.3	235.1	254.5	303.1	324.9	183.1	2,253.3
Lago Alajuela	29.5	13.7	16.0	83.9	271.9	290.6	276.2	287.9	290.3	359.9	332.0	131.3	2,383.2
Empire	22.9	15.9	11.2	94.4	212.6	251.9	233.2	231.4	258.0	326.4	278.6	106.9	2,043.4
Guacha	78.9	32.5	31.0	133.2	231.1	221.2	213.1	252.8	221.3	309.0	327.6	206.1	2,257.8
Pedro Miguel	30.1	10.2	10.9	87.2	245.8	240.3	203.3	228.7	234.3	311.4	272.6	132.7	2,007.5
Hodges Hill	24.6	17.5	8.2	106.2	216.7	164.1	166.9	183.6	217.2	279.9	188.5	105.9	1,679.3
Gamboa	33.7	17.2	13.8	75.1	253.4	249.8	246.9	247.3	246.3	306.1	297.3	137.7	2,124.6
Gatún	90.1	42.0	40.9	124.0	314.3	282.9	325.1	335.3	293.5	392.6	518.0	317.8	3,076.5
Las Raices	61.6	23.0	27.6	101.2	239.7	217.1	206.0	231.1	237.8	302.2	310.8	189.5	2,147.6
Montelirio	69.3	31.7	32.5	92.9	276.8	255.2	275.2	319.5	271.7	377.9	452.0	280.1	2,734.8
Peluca	100.2	50.9	40.3	164.5	328.2	322.9	309.3	318.3	290.8	331.1	386.8	284.8	2,928.1
Chilibre	19.5	7.4	18.6	74.6	217.9	258.6	247.6	269.1	274.8	347.9	275.6	81.5	2,093.1
Caño	51.5	20.0	20.4	73.2	243.5	225.1	224.3	252.9	236.0	325.1	349.2	187.2	2,208.4
Los Cañones	111.4	46.1	37.2	110.8	309.7	280.8	285.4	295.4	374.8	362.3	340.1	204.7	2,758.7
El cacao	47.5	23.1	45.8	105.2	274.8	217.6	197.0	262.9	308.1	417.0	264.4	120.2	2,283.6
Ciri Grande	54.4	35.9	21.9	121.1	304.2	251.9	201.5	306.2	329.9	399.7	285.6	153.8	2,466.1
Zanguenga	46.2	21.1	14.5	75.2	210.2	215.2	172.0	231.7	254.6	338.3	233.7	140.1	1,952.8
Cascadas	21.8	11.1	26.2	88.3	239.9	254.6	204.3	237.6	258.5	315.7	253.1	93.3	2,004.4
Average	65.3	30.6	30.0	113.8	276.9	268.1	255.2	284.0	269.0	353.6	345.7	201.4	2,493.6

Date Source : IRHE
Period : 1930~1987

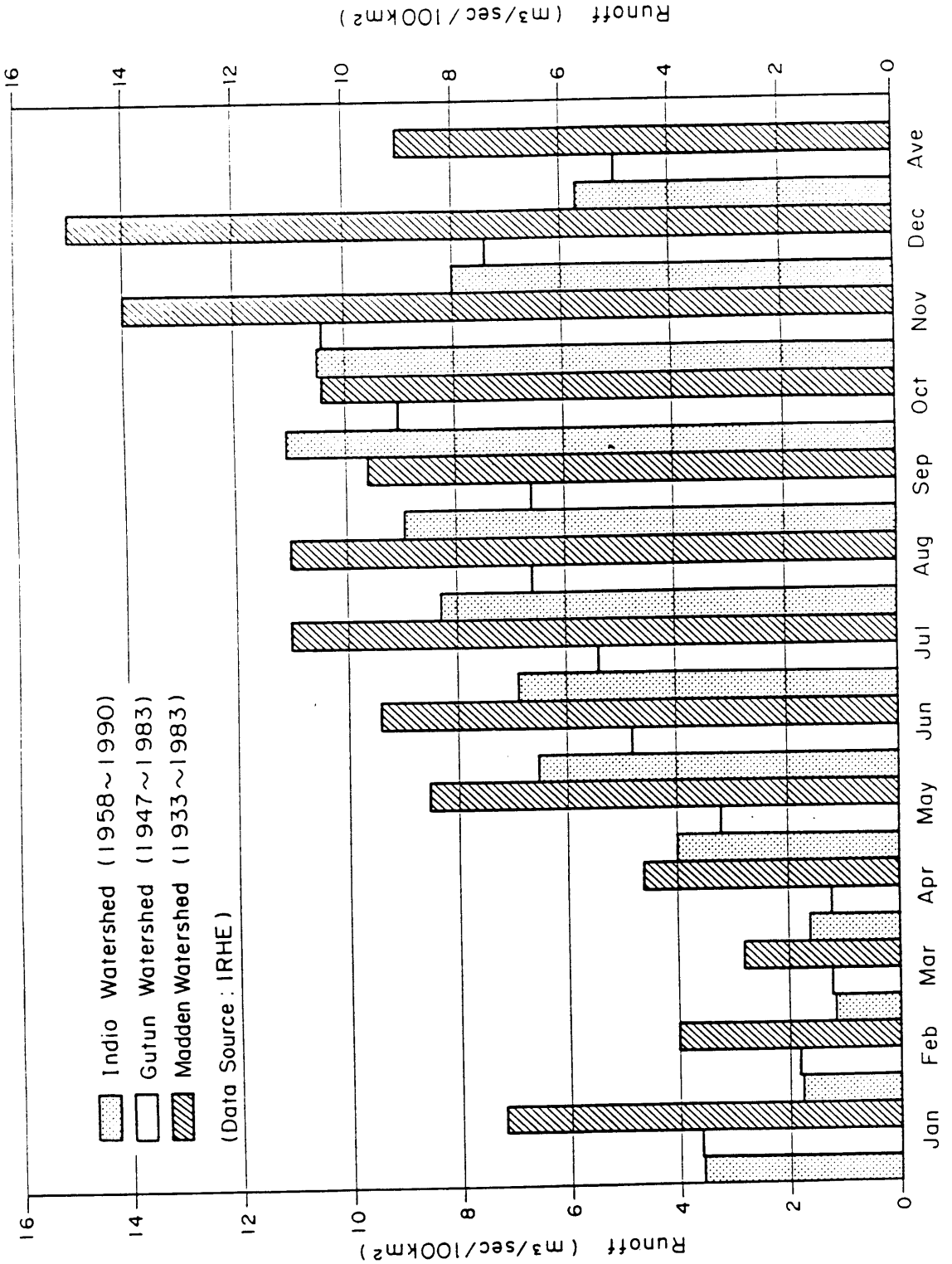


Fig. II.1.4-2 Specific Runoff

Table II.1.4-2 Average Watershed Yield

(Gatun Watershed Yield) (UNIT : m^3/s)

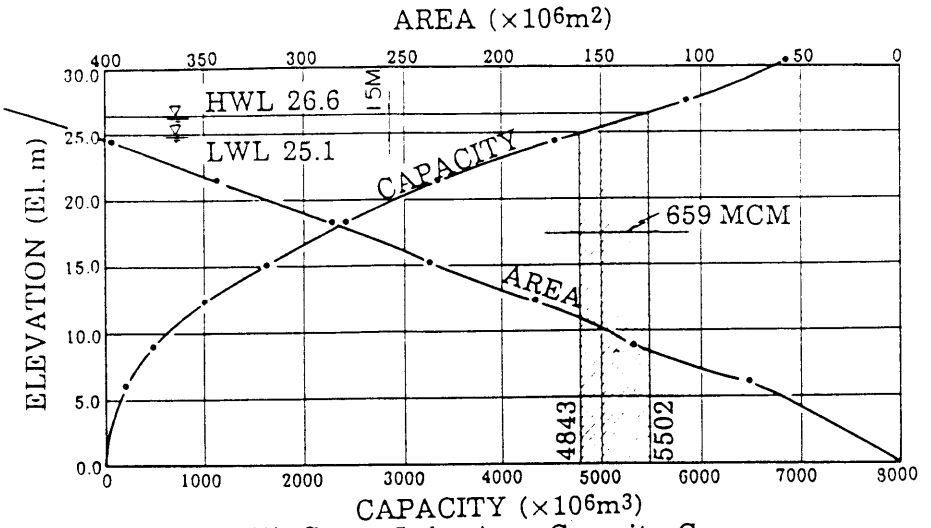
River	Station	Period of Record	Drainage Area	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Trinidad	Chorro	1947~1983	173km²	4.28	2.23	1.34	1.58	4.00	6.48	6.70	8.24	9.99	13.81	14.25	9.47
①			100km²	2.47	1.29	0.77	0.91	2.31	3.75	3.87	4.76	5.77	7.98	8.24	5.47
Gatun	Ciento	1943~1983	122km²	4.62	2.59	1.66	2.24	4.38	5.95	7.54	7.91	7.15	11.48	16.05	11.35
②			100km²	3.79	2.12	1.36	1.84	3.59	4.88	6.18	6.48	5.86	9.41	13.16	9.30
Ciri Grande	Los Cañones	1947~1983	186km²	7.94	3.69	2.42	2.43	6.47	10.68	11.26	13.16	15.23	17.85	18.42	13.24
③			100km²	4.27	1.98	1.30	1.31	3.48	5.74	6.05	7.08	8.19	9.60	9.90	7.12
①+②+③/3	Specific discharge		100km²	3.51	1.80	1.14	1.35	3.13	4.79	5.37	6.11	6.61	9.00	10.43	7.30

(Madden Watershed Yield) (UNIT : m^3/s)

River	Station	Period of Record	Drainage Area	Jan	Feb.	Mar	Apr.	May	Jun	Jul	Aug.	Sep.	Oct.	Nov.	Dec.
Chagres	Chico	1933~1986	414km²	25.90	15.33	10.99	15.72	27.09	29.21	32.72	34.78	33.34	38.83	49.16	48.46
①			100km²	6.26	3.70	2.65	3.80	6.54	7.06	7.90	8.40	8.05	9.38	11.87	11.71
Boquerón	Peluca	1933~1983	91km²	5.84	2.92	1.95	3.81	8.09	8.07	10.03	9.79	7.64	8.04	12.87	13.24
②			100km²	6.42	3.21	2.14	4.19	8.89	8.87	11.02	10.76	8.40	8.84	14.14	14.55
Pequeñi	Candelaria	1943~1983	135km²	12.25	6.48	4.56	7.65	15.23	16.40	18.64	18.29	16.30	17.23	22.14	25.40
③			100km²	9.07	4.80	3.38	5.67	11.28	12.15	13.81	13.55	12.07	12.76	16.40	18.81
①+②+③/3	Specific discharge		100km²	7.25	3.90	2.72	4.55	8.90	9.36	10.91	10.90	9.51	10.33	14.14	15.02

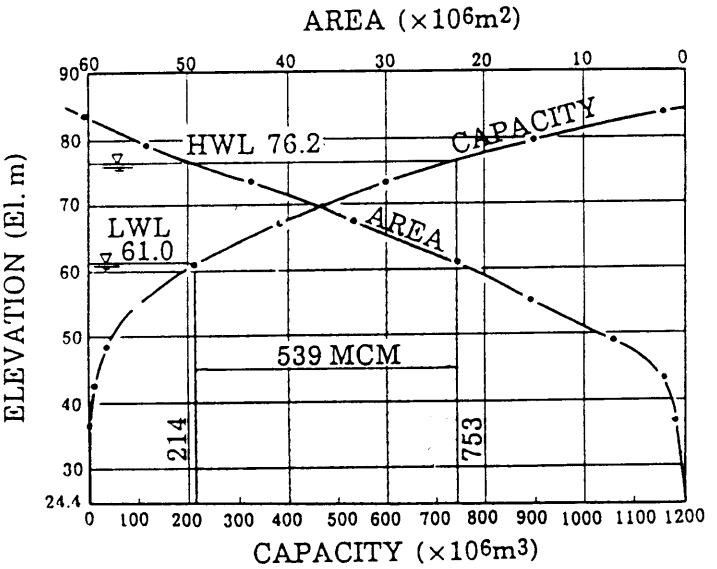
(Indio Watershed) (UNIT : m^3/s)

River	Station	Period of Record	Drainage Area	Jan	Feb.	Mar	Apr.	May	Jun	Jul	Aug.	Sep.	Oct.	Nov.	Dec.
Indio	Boca De Uracillo	1958~1990	376km²	13.39	6.75	4.45	6.38	15.01	24.46	25.54	31.32	33.36	41.92	39.43	29.96
	Specific discharge		100km²	3.56	1.80	1.18	1.70	3.99	6.51	6.79	8.33	8.87	11.15	10.49	7.97



(1) Gatun Lake Area Capacity Curve

(Data Source (1), (2): Improvement Program for the Panama Canal, 1969)
Appendix V (Water Supply of the Panama Canal)



(2) Madden Lake Area Capacity Curve

Fig. II.1.4-3 Gatun and Madden Area Capacity Curve

II.1 CHAPTER 5 - EXISTING POPULATION AND INFRASTRUCTURE

5.1 Population and Land Use

The population densities in the vicinity of both alignments are generally quite low with the exception of the urban development around Panama City (population 650,000 including San Miguelito) and Colon (54,000), and also corridor development along the Pan American Highway between Tacumen Airport to the east of Panama City and La Chorrera (44,000) to the west. Restrictions on development within the existing Panama Canal Zone have resulted in limited population growth in the Zone. With the exception of Colon, population densities between Gatun Lake and the Atlantic side are very low, due to limited access, more rugged terrain and the extremely high rainfall. The populations of the provinces of Colon and Panama in the vicinity of the alignments are shown in Table II.1.5-1(a) and (b) respectively.

The locations of the areas in the Table are shown on the attached Fig. II.1.5-1 and the population densities are indicated on Fig. II.1.5-2.

Within the urban areas described above there is commercial and light industrial development but elsewhere land use is mainly farming and grazing with some areas of tropical jungle still remaining.

The area lying west of the Canal Zone has undergone moderate development. The rolling hills on the Pacific side of the Continental Divide have been cleared and the land is now used for farming and grazing, although urban areas along the Pan American Highway corridor have been spreading. Between the Continental Divide and Lake Gatun the land is less developed but agricultural use predominates. Between Gatun Lake and the Atlantic coast the terrain is more rugged and rainfall is much higher. Much of this land is not developed with the exception of the narrow coastal corridor which is farmed.

Table II.1.5-1(a) Population Data - Province of Colon

District	Zone (Corregimiento)	Area km2	Population 1980	Population 1990	Growth %	Density per/km2
Donoso	76-Miguel de la Borda	313.6	1192	2218	86.1	7.1
	78-El Guasimo	283.2	1196	1992	66.6	7.0
	79-Gobea	48.9	522	610	16.9	12.5
	80-Rio Indio	35.2	800	963	20.4	27.4
	Others	1136.2	2057	3799	84.7	3.3
	Sub-total	1817.1	5767	9582	66.2	5.3
Chagres	69-Chagres	6.7	304	327	7.6	48.8
	70-Achiote	38	699	755	8.0	19.9
	71-El Guabo	53.9	909	1191	31.0	22.1
	72-La Encantada	135.7	1963	2532	29.0	18.7
	73-Palmas Bellas	75.1	1430	1628	13.8	21.7
	74-Pina	31.5	678	687	1.3	21.8
	75-Salud	104.7	1752	2085	19.0	19.9
	Sub-total	445.6	7735	9205	19.0	20.7
Colon	55&56-Colon City	2.9	59840	54654	-8.7	18846
	57-Buena Vista	114.5	5190	7547	45.4	65.9
	58-Cativa	21	10870	19101	75.7	909.6
	59-Ciricito	68.4	1753	2108	20.3	30.8
	60-Cristobal	728	3874	15178	291.8	20.8
	61-Escobal	94.4	1807	1964	8.7	20.8
	62-Limon	76.3	2700	3209	18.9	42.1
	63-Nueva Providencia	20.4	703	1253	78.2	61.4
	64-Puerto Pilon	100.6	8460	10241	21.1	101.8
	65-Sabanitas	13.8	9924	13729	38.3	994.9
	66-Salamanca	196.9	1733	2675	54.4	13.6
	67-San Juan	41.7	5839	8716	49.3	209.0
	68-Santa Rosa	25.5	460	533	15.9	20.9
	Sub-total	1504.4	113153	140908	24.5	93.7

Source: Panama National Census, 13 May 1991.
Note: For locations refer to Fig. II.1.5-1

Table II.1.5-1(b) Population Data - Province of Panama

District	Zone (Corregimiento)	Area km2	Population 1980	Population 1990	Growth %	Density per/km2
Capira	348-Ciri de Los Sotos	96.7	1886	2202	16.8	22.8
	351-La Trinidad	110.4	1626	2160	32.8	19.6
	Others	725.7	18069	23941	32.5	33.0
	Sub-total	932.8	21581	28303	31.1	30.3
La Chorrera	376&377-Central	23.4	37566	44444	18.3	1899.3
	378-Armador	65.5	1622	1925	18.7	29.4
	379-Arosemena	31.7	498	340	-31.7	10.7
	380-El Arado	60.5	1574	1696	7.8	28.0
	384-Herrera	86.3	686	715	4.2	8.3
	386-Iturralde	65.4	595	792	33.1	12.1
	387-La Represa	30.2	731	650	-11.1	21.5
	389-Mendoza	37.4	1120	851	-24.0	22.8
	392-Puerto Caimito	31.4	2233	3623	62.2	115.4
	Others	256.3	20349	34744	70.7	135.6
	Sub-total	688.1	66974	89780	34.1	130.5
Arraijan	332-Arraijan	53.4	16272	24665	51.6	461.9
	333-Juan D. Arosemena	48.3	8525	13418	57.4	277.8
	334-Nuevo Emperador	24.4	1926	2319	20.4	95.0
	335-Santa Clara	15.9	1169	1422	21.6	89.4
	336-Veracruz	13.8	5287	8224	55.6	595.9
	337-Vista Alegre	14.3	4007	11801	194.5	825.2
	Sub-total	170.1	37186	61849	66.3	363.6
San Miguelito	All	50	156611	243025	55.2	4860.5
Panama	394 to 404-Panama City	106.5	389172	413505	6.3	3882.7
	407-Ancon	664.5	6401	11518	79.9	17.3
	Others	1789.8	81534	159780	96.0	89.3
	Sub-total	2560.8	477107	584803	22.6	228.4

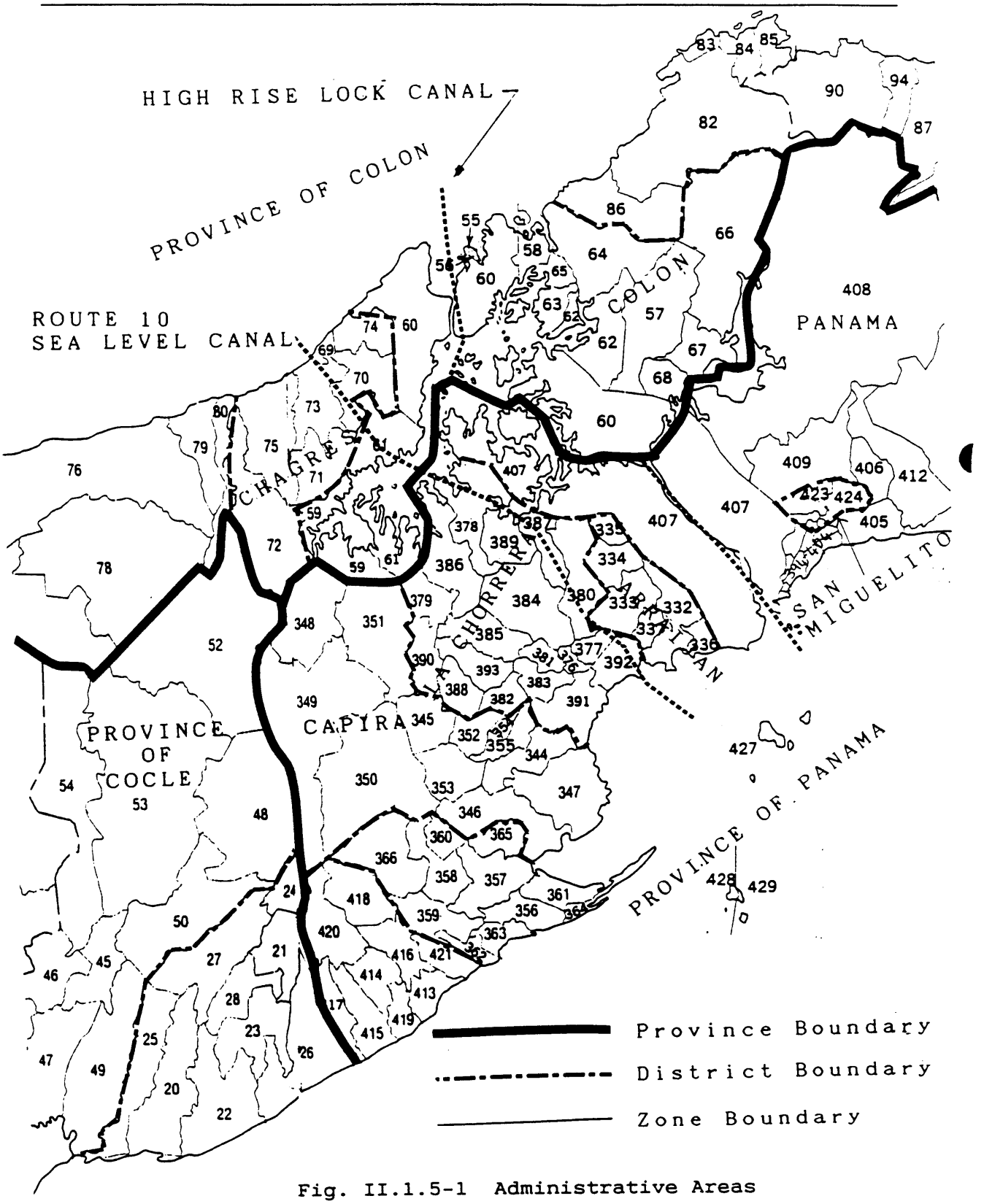
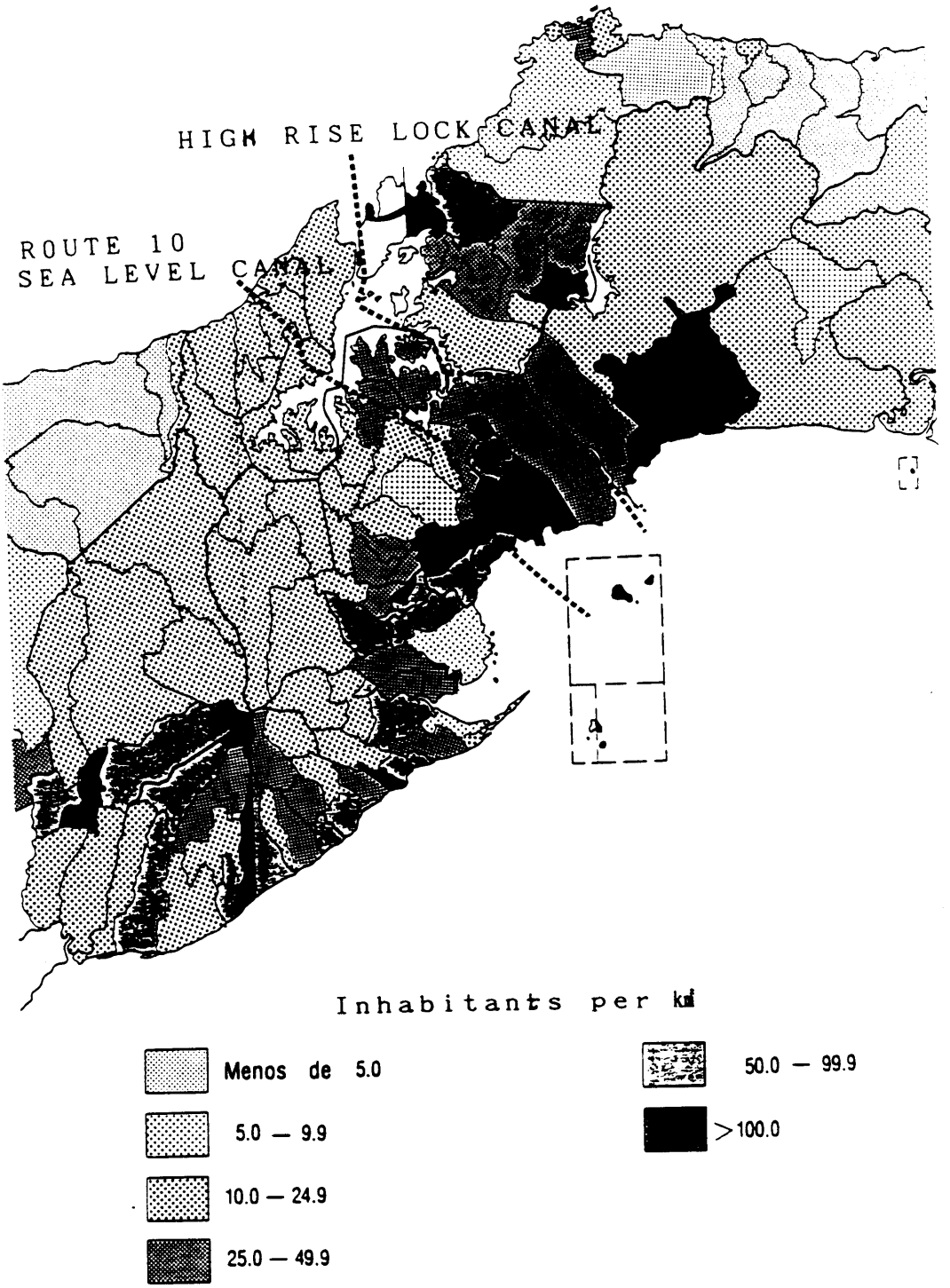


Fig. II.1.5-1 Administrative Areas



Source: Atlas of the Republic of Panama

Fig. II.1.5-2 Population Densities

5.2 Existing Road System

5.2.1 Road Network

The existing road network in the vicinity of the Canal Zone is shown in Fig. II.1.5-3. The roads have been classified according to function as follows:

1) National Highways and Autopista (Road Nos. 0-9)
2) Regional Roads (Nos. 10-99)
3) Local Roads (Nos. 100-999)
4) Minor Roads (Nos. 1000-9999)

The above functional classification is described in the "Estudio del Plan Nacional de Transporte, Informe Final, Tomo IV, Carreteras y Transport Vial, Ministerio de Obras Publicas, pages IV-34 to 36". The categories correspond approximately to the Panama road numbering system which allocates a 1, 2, 3 or 4 digit number according to the importance of the road, as shown above and described in pages IV-27 and 28 of the above study. The most important roads likely to be affected by the canal works are described below.

The Pan American Highway (National Route 1) is the only road which travels the full length of Panama. Near the Panama Canal Zone it is quite close to the Pacific Ocean side and mostly has one lane in each direction. Between Panama and Arraijan construction works are underway for widening to two lanes in each direction. There are many residential, industrial and commercial developments on this road and traffic is congested at times.

A section of autopista parallel to the Pan American Highway has been constructed between La Chorrera and Arraijan and the Route 10 Sea Level Canal alignment crosses both roads at a point where they are about 1 km apart. A further extension of the autopista to bypass the central area of Panama City (Van Dam Bypass) is proposed and will cross the existing canal by bridge just to the north of the Miraflores locks.

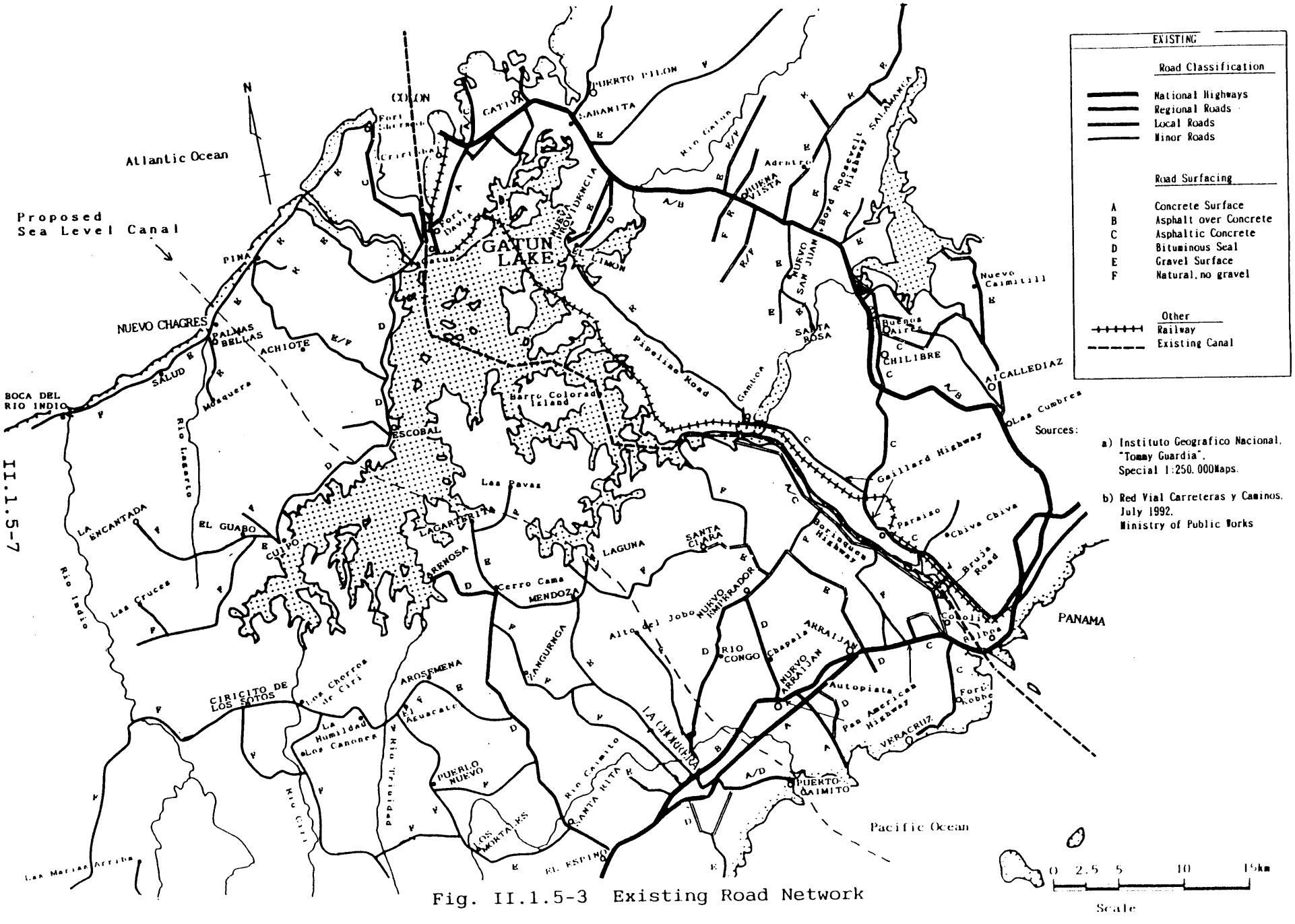


Fig. II.1.5-3 Existing Road Network

The Boyd Roosevelt Highway (National Route 3) crosses the Panamanian Isthmus between Panama City and Colon on an alignment to the east of the existing canal. This highway is about 90 km (56 miles) long. The majority is two lanes wide, while 7 km between Colon and Cativa and 15 km between Panama City and Isidro are four lanes wide. The alignment passes through hilly areas and there are sections where the surface and vertical alignment standards are poor. The sections closest to Panama and Colon carry high volumes of commuter traffic and are operating close to capacity.

The two-lane Gaillard Highway closely follows the eastern side of the canal between Panama and Gamboa. It is the main access road for operation and maintenance of the existing canal on the Pacific side of Gatun Lake, and also provides access to military and defense areas. On the opposite side of the canal, access is provided by the Borinquen Highway. This road is also hard surfaced and two-lanes wide. Traffic access to this road is controlled at a gate near Miraflores, and public access beyond the Gold Hill lookout is not permitted. The design and maintenance standards for both roads are good.

The maps also show Bruja Road between the Pan American Highway west of the canal and the Gaillard Highway on the east side of the canal. The road is only open to the public on the west side of the canal and crossing the canal over the Miraflores Bridge is not permitted under normal circumstances.

A coastal road follows the Atlantic Coastline from Colon to Miguel de la Borda at the mouth of the river of the same name. This road has two lanes as far west as Nuevo Chagres, but bridges are generally one lane only. The road has a gravel surface and is in need of maintenance. A second road from Gatun follows the north-west shore of Gatun Lake and connects to Escobal and Cuipo. This road is two lanes wide and has a bitumen seal, also in need of maintenance. The only access to both roads is from Gatun and Colon, via the movable one-lane bridge for light vehicles beside the lower locks at Gatun.

For the areas on the south side of Gatun Lake, the main road providing access to these farming and grazing areas is the road from El Espino to Cerro Cama, a sealed two-lane road in reasonable condition. The only other significant road in this area is the road between La Chorrera and Mendoza, a two-lane gravel road.

From La Chorrera to Nuevo Emperador via Rio Congo there is a two-lane sealed road in reasonable condition. The road from Nuevo Arraijan to Nuevo Emperador via Chapala is also a two-lane sealed road but is constructed to a higher standard and provides the best access fro Nuevo Emperador to the Pan American Highway.

5.2.2 Traffic Volumes

During the course of the Study, it has not been possible to obtain complete traffic data for the roads in the vicinity of the two canal alternatives. Some data is available, particularly for the highways, but is not published or readily accessible in a comprehensive form. The traffic data which was obtained is summarized in Table II.1.5-2 below, though there appear to be some inconsistencies in the data for the Escobal Road and the autopista.

Table II.1.5-2 Traffic Volume Data

Name of Road	Section	Date	Traffic*	Source
Pan Amer. H'way	Bridge of the Americas	4/92	30562	2
Pan Amer. H'way	East of Arraijan	7/90	16064	3
Pan Amer. H'way	Arraijan to La Chorrera	1992	8114	1
Pan Amer. H'way	El Espino to La Chorrera	1987	6258	4
Autopista	Arraijan	7/90	12922	3
Autopista	Arraijan to La Chorrera	1992	5501	1
Nuevo Emperador	N. Arraijan to N. Emper.	1992	1592	1
Port Caimito	From La Chorrera	1987	233	4
Cerro Cama Road	El Espino to Santa Rita	1987	769	4
Boyd Roosevelt Highway	Near Sabanita	11/92	10562	2
Escobal Road	Gatun to Escobal	1992	132	1
Escobal Road	Escobal to Cuipo	1987	382	4

* In vehicles per day

Sources (full citation of references 1, 2 and 3 unavailable):

1. Departamento de Control de Carga y Aforos, Registro de Afores.
2. Ministerio de Obras Publicas, Direccion de Programacion y Desarrollo Institucional, Departamento de Planification y Programacion.
3. Ministerio de Obras Publicas, 1990
4. Estudio del Plan Nacional de Transporte, Informe Final, Tomo IV, Carreteras y Transport Vial, Ministerio de Obras Publicas, pages IV-157, IV-169 (undated, about 1987).

5.2.3 Existing Bridges over the Canal

There are three bridges across the existing canal, as described below.

The Pan American Highway crosses via the Bridge of the Americas. This bridge is an arch truss steel bridge and has a main span of 1128 feet (334m) and a vertical clearance of about 209 feet (64m) at high water level. The maximum canal bottom width with the full vertical clearance would be about 800 feet (244m). The approach grades are 5%. Four lanes are provided over the three central spans but the approaches are 3 lanes only, two of which are for the climbing traffic.

The Miraflores swing bridge is located immediately downstream of the existing Miraflores locks. It is a substantial bridge with 4 lanes and a total length of 10,827 feet (3300m). The bridge and the approach roads are normally kept closed to public access.

At Gatun Locks there is a movable bridge beside the lower locks. It is closed when ships are passing through, remaining closed for about 20 minutes to 1 hour for each ship. The bridge has one lane only and vehicles are limited to a maximum weight of 20 tons and a maximum width of 14 feet.

5.3 Port Facilities

Existing port and harbor facilities are available at Cristobal and Limon Bay on the Atlantic Coast and at the Port of Balboa and Balboa Harbor on the Pacific coast. These facilities are well developed to serve shipping using the existing canal and include essential services such as bunkering and repair facilities.

5.4 Other Infrastructure

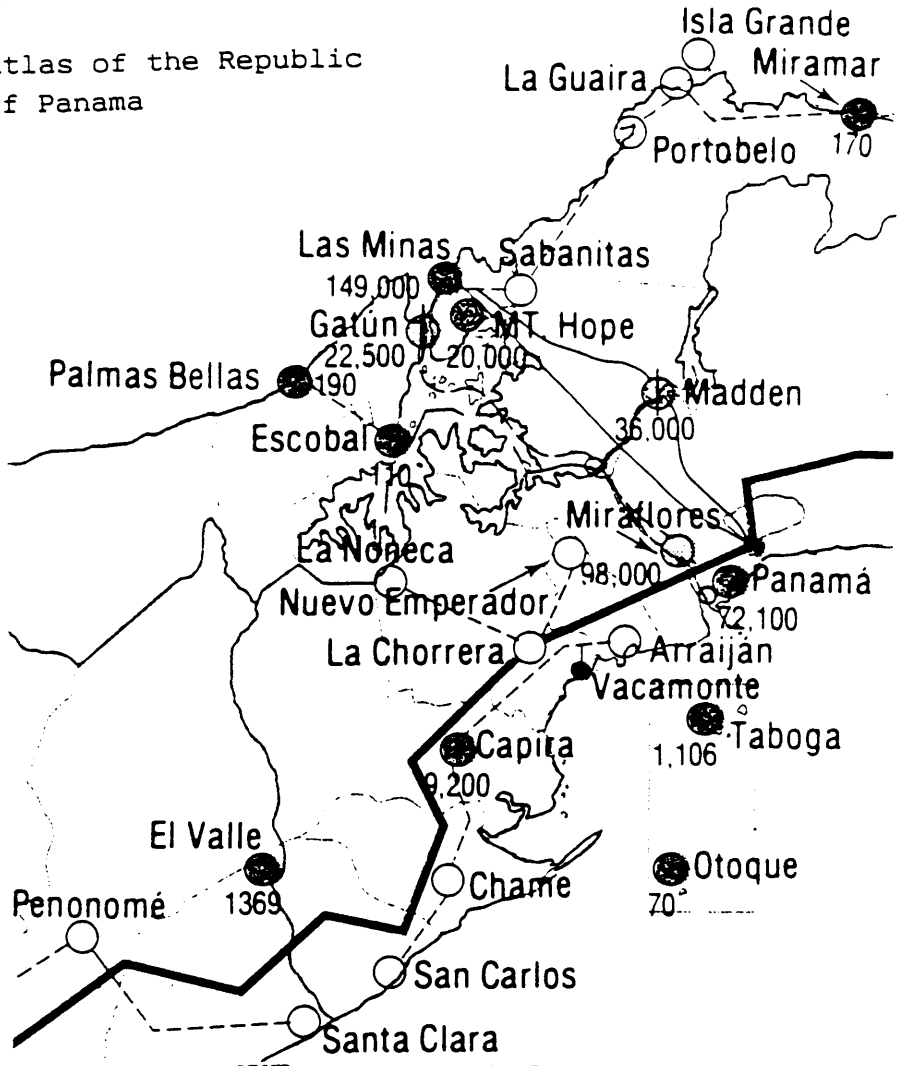
Panama City and Colon are linked by a railway line which follows a route similar to the existing canal. The line crosses Gatun Lake on an embankment about 8 km long. Passenger services no longer run but there are two or three cargo trains each day.

The major electricity distribution lines and power stations in Panama are shown in Fig. II.1.5-4. Production in Panama is divided between hydroelectric and thermal power stations. The PCC operate their own hydroelectric power stations at Madden(36,000 kW) and Gatun (22,500 kW) dams, and operate a thermal power stations at Miraflores (98,000 kW). Power generated by the PCC is in excess of canal requirements and the surplus is sold to the IRHE (Instituto de Recursos Hidraulicos y Electrificacion) national grid. The thermal power station at Mt. Hope (20,000 kW) was previously operated by PCC but has been transferred to IRHE. The other power stations in Panama are also operated by IRHE.

The PCC also operate water treatment plants on the Atlantic side at Mt. Hope and on the Pacific side at Balboa. These plants provide potable water for the Canal area and areas of the Republic of Panama. In 1991, water supplied by the PCC to Panama City and Colon Metropolitan areas amounted to approximately 216,000 cubic meters (57.1 million gallons) per day. (Source: PCC Annual Report, 1991)

The available maps show a pipeline across the isthmus adjacent to the existing canal, and crossing the canal near Gamboa. The pipeline was used to carry oil but is no longer used (Source: Status of Present Panama Canal, prepared by SINBOL Consortium, July 1991).

Source: Atlas of the Republic of Panama



	IRHE	C. de P.	Others
Thernal Power Station	● KW	○ KW	○
Hydroelectric Power Station	●	⊕	
Sub Station	●	○	
Transmission Lines			
230 KV	━━━		
115 KV	———	———	
Minor 115 KV	- - - - - - -	— — — —	

IRHE - Instituro de Recursos Hidraulicos y Electrificacion

C. de P. - Canal de Panama

Fig. II.1.5-4 Power Stations and Electricity Distribution Lines

PART-II Section-2 CONSTRUCTION PLANS

II.2 Chapter 1. General Construction Plans

The preparation of construction plans is based upon a High Rise Lock Canal for the passage of max. 200,000 DWT vessels at 85' lake level and a Sea Level Canal Route 10 for the passage of max. 250,000 DWT vessels.

In view of the huge volume of construction to be carried out, the construction plans must take into account the economy of scale by employing the largest construction equipment that can possibly be made available.

The items to be examined in the construction of canal alternatives include, among other things, the following:

1.1 Canal Excavation

The material to be excavated at the proposed site is complicated and diversified. In addition, topographic conditions also differ greatly zone by zone. Therefore, from the points of view of practicality and economy, it will be necessary to vary the excavation method for each zone and use a combination of several types of excavation method depending on the zone.

One of the key points in the selection of excavation method at each zone is as to which method should be applied, dry excavation or dredging work.

In areas where the need for underwater blasting is anticipated, efforts should be made to keep it to a minimum because dredging combined with underwater blasting can be very expensive. In zones where the possibility of having to use underwater blasting is small, dredging should be employed as much as possible.

In zones where application of the dry excavation method is inevitable, it should be done to prevent inflow from rivers and streams in the neighborhood and make dewatering of the excavation area more complete to assist the

excavation work.

1.2 Aggregate for Concrete

On the Pacific Side, as the outcrop of basalt is exposed in various places, aggregate may be obtained relatively easily. On the Atlantic Side, as there is no suitable material available for concrete aggregate in the vicinity of the construction site, new quarry sites need to be developed at locations away from the lock construction site in order to obtain source rock, which will then be transported to the construction site.

1.3 Arrangement of Construction Equipment and Materials

For the smooth execution of the Project, it is necessary that huge volumes of construction equipment and materials be procured efficiently and transported to the construction site. Because of the huge volumes of cement, fuel, lubricants and equipment that need to be supplied, logistic bases with wharves should be constructed both on Atlantic and Pacific sides for the unloading, temporary storage and transportation of main construction equipment and materials imported from abroad. Transportation will be via an access road which is to be constructed for that purpose.

II.2 Chapter 2. Equipment for Excavation

2.1 Dry Excavation

Because the canal excavation volume in this project is so huge, the construction cost and schedule will be greatly affected by how the canal excavation work is carried out. Therefore, it is necessary to employ the maximum size of heavy equipment in very large numbers to ensure that the whole project is on schedule.

(1) A study has been made to compare various combinations of equipment and dredging options.

(2) Basis of Excavation

a. The excavation study is to be carried out from the existing ground level down to the canal bed.

b. Areas for placing excavated materials are to be located along the Canal within a certain distance ranging from 2.0 to 20 km.

c. Temporary plugs (non-excavated in-situ earth intervals) will be provided approximately every 2 km in order to prevent rainfall from flowing into excavation areas. These plugs are to be removed one by one after completion of the dry excavation work between plugs.

d. Comparison of Earthwork Distance

It is considered that as the size of equipment employed becomes larger, costs will decrease in proportion. In this case, the cost comparison has been made for earthwork equipment in relation to respective distance. The haul distance is taken to be the distance from the excavation area to the placing area.

(3) Comparison of Earthwork Equipment

Comparison of earthwork equipment is as follows:

a. Hauling System by Dump Truck
- 240 ton class dump truck (max. capacity available)
- 190 ton class dump truck
- 150 ton class dump truck (Phase-1)

b. Conveying System by Conveyor Belt
- 17,000 t/hour capacity, belt width = 2.8m

c. Hauling System by Wheel Tractor Scraper
- 550 Hp, 47 ton class, 33.6 m^3 capacity

Table II.2.2.1 - 1 Comparisons for Earthwork System

System	Dump Truck	Conveyor	Scraper
Constructability	Weathered/soft rock to be excavated by ripper Hard rock to be blasted.	Large size boulders more than 600 mm to be crushed by crusher, or by blasting	Difficult for rock excavation
Availability of large scale Heavy Equipment	Up to 240 ton (160 m^3) dump truck Up to 1340 Hp 30 m^3 class Power Shovel is most suitable for combination with a 240 ton dump truck	Up to 2.8 m wide belt conveyor Loading/unloading facilities necessary for conveyor belt system	Up to 34 m^3 scraper

(4) Cost Comparison

Costs worked out for five (5) systems are shown in Table II. 2.2.1 - 2(a) - (e). Those costs are also illustrated in Fig. II. 2.2.1 - 5 showing the trend of cost for each system versus hauling distance.

(5) Employment of Earthwork System

An earthwork system utilizing dump trucks is the most suitable for the Canal Dry Excavation work.

Hourly working capacity is mainly decided by loading equipment (shovel). Allowing shovel to achieve a full loading capacity, the required numbers of bulldozer and dump truck are to be properly provided.

Combinations of main equipments are as follows:

Loading Equipment:	30 m^3 capacity Power Shovel
Hauling Equipment:	240 ton class Dump Truck
Dozing Equipment:	98 ton class Bulldozer

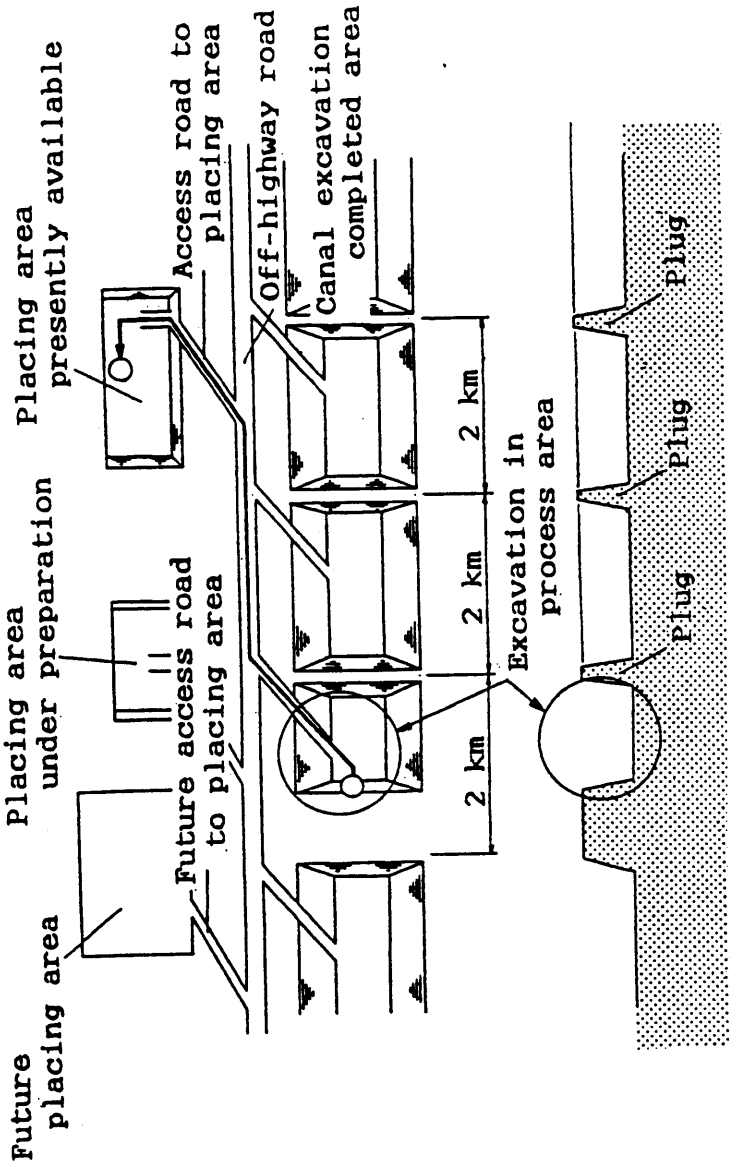


Fig. II.2.2.1 - 1 Standard Excavation Plan

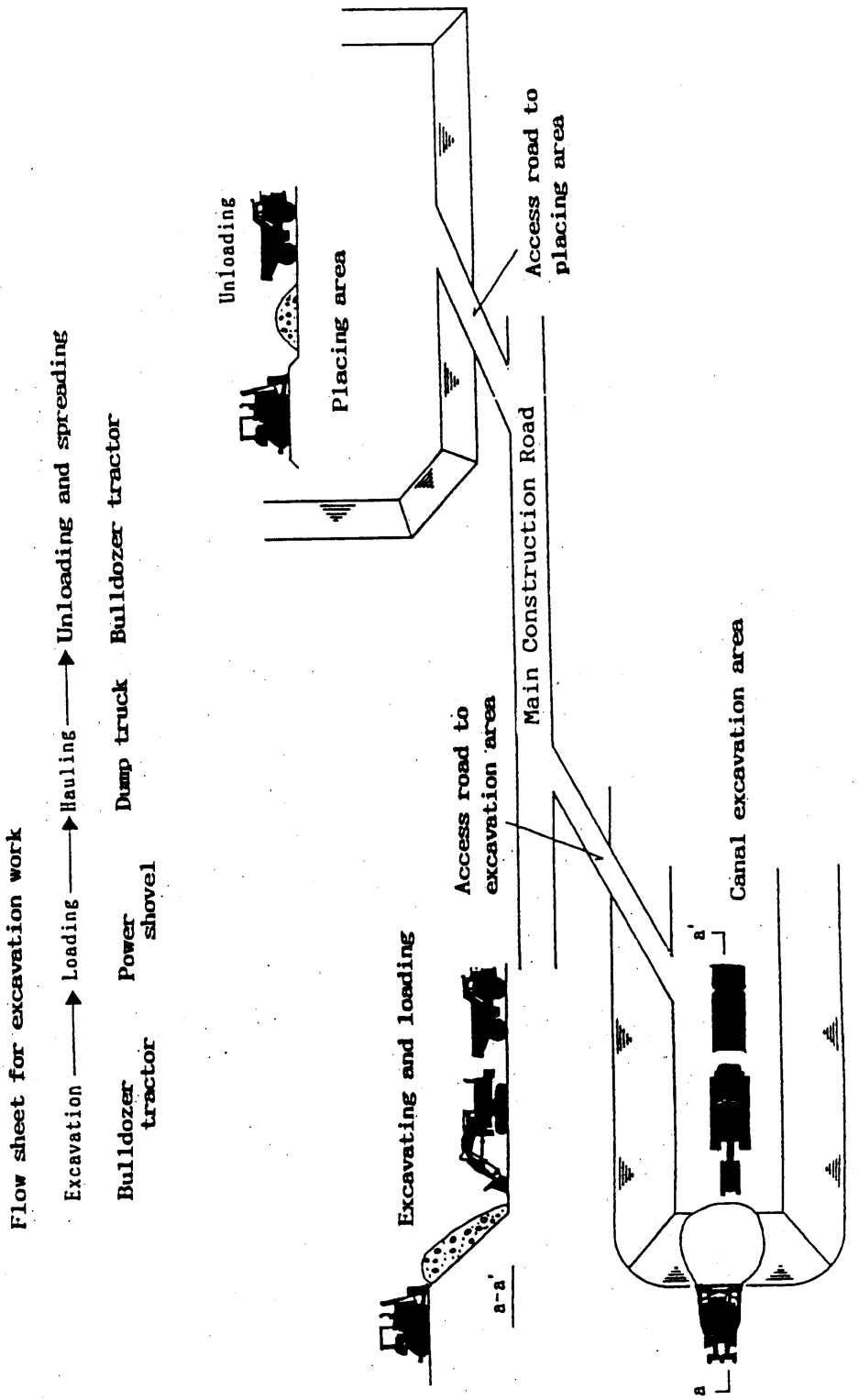


Fig. II.2.2.1 - 2 Earthwork System using Dump Truck

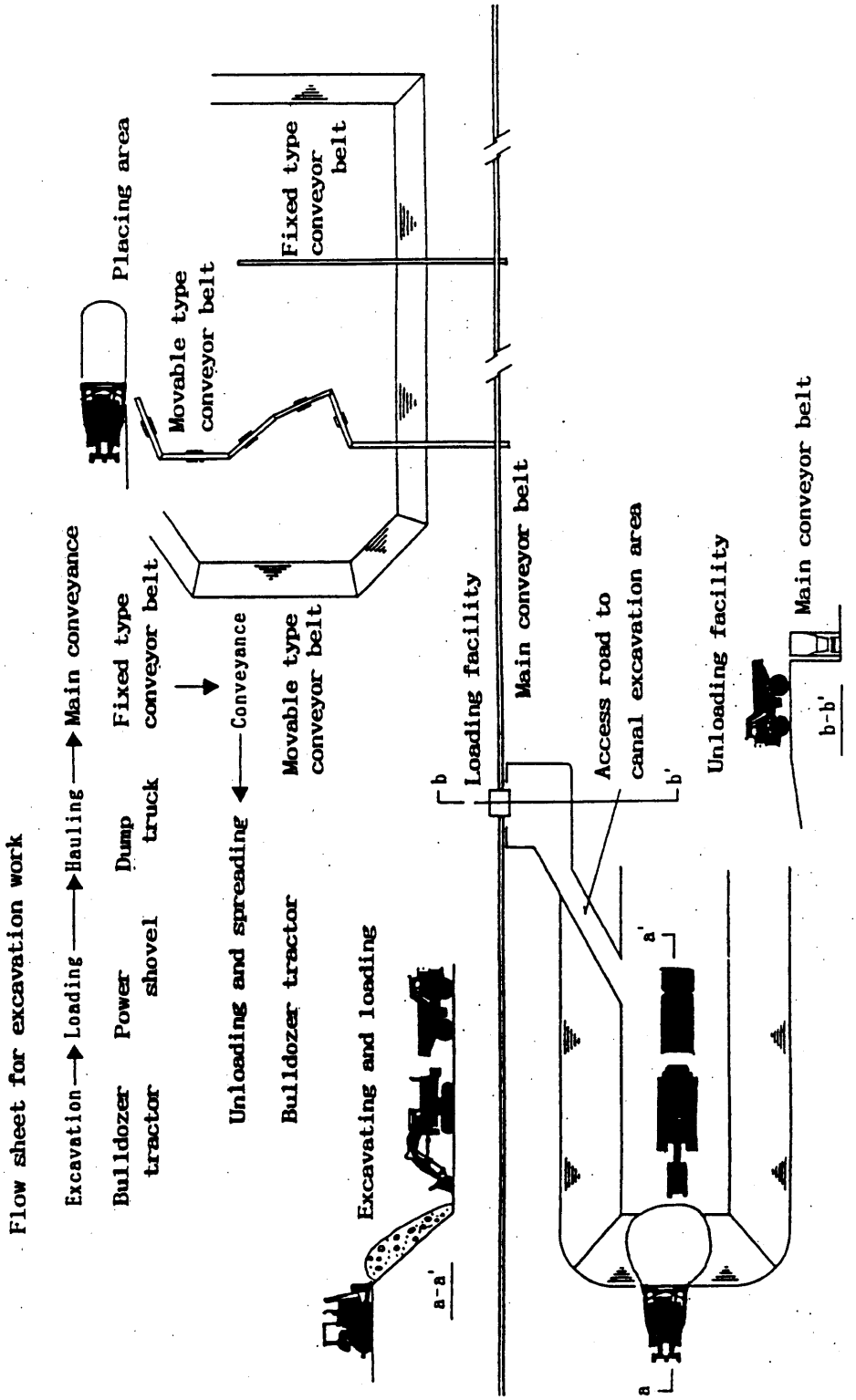


Fig. II.2.2.1 - 3

Earthwork System using Conveyor Belt

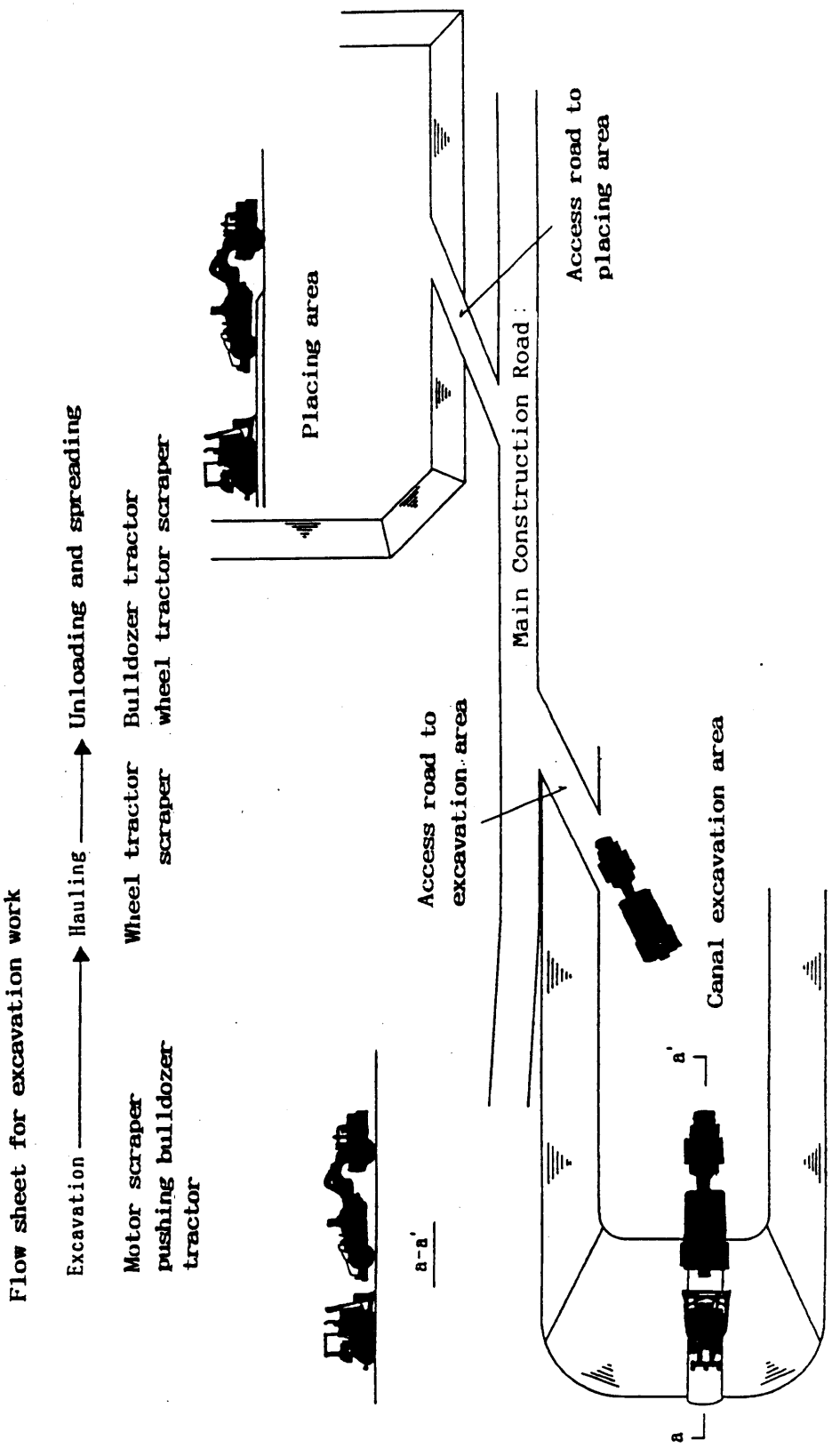


Fig. II.2.2.1 - 4 Earthwork System using Motor Scraper

CASE-1 MAIN EQUIPMENT :SHOVEL(30M3)+ DUMP TRUCK(240T)

Table II.2.2.1-2(a)

(FILE : PANAM-13)

PRODUCTIVITY 2.629 M3/Hr (COMMON EXCAVATION)

HAULAGE L (KM)	① EXCAVATION BULLDOZER (98TON) EQUIP. NO.	① U. COST ($/Hr)	② LOADING SHOVEL (30M3) EQUIP. NO.	② U. COST ($/Hr)	③ HAULING DUMP TRUCK (240TON) EQUIP. NO.	③ U. COST ($/Hr)	④ GRADING BULLDOZER (98TON) EQUIP. NO.	④ U. COST ($/Hr)	UNIT COST EXCAVATION BY ① ($/H)	UNIT COST EXCAVATION BY ① ($/M3)	UNIT COST LOADING BY ② ($/H)	UNIT COST LOADING BY ② ($/M3)	UNIT COST HAULING BY ③ ($/H)	UNIT COST HAULING BY ③ ($/M3)	UNIT COST GRADING BY ④ ($/H)	UNIT COST GRADING BY ④ ($/M3)	TOTAL UNIT COST (U$/H)	TOTAL UNIT COST ($/M3)
0.5	5.23	158.80	1	1065.71	3.31	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	892.57	0.34	420.82	0.16	3,209.63	1.22
1.0	5.23	158.80	1	1065.71	4.77	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	1,286.28	0.49	420.82	0.16	3,603.33	1.37
2.0	5.23	158.80	1	1065.71	7.69	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	2,073.69	0.79	420.82	0.16	4,390.74	1.67
3.0	5.23	158.80	1	1065.71	10.62	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	2,863.79	1.09	420.82	0.16	5,180.84	1.97
4.0	5.23	158.80	1	1065.71	13.55	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	3,653.89	1.39	420.82	0.16	5,970.95	2.27
5.0	5.23	158.80	1	1065.71	16.47	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	4,441.30	1.69	420.82	0.16	6,758.35	2.57
6.0	5.23	158.80	1	1065.71	19.40	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	5,231.40	1.99	420.82	0.16	7,548.46	2.87
7.0	5.23	158.80	1	1065.71	22.32	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	6,018.81	2.29	420.82	0.16	8,335.87	3.17
8.0	5.23	158.80	1	1065.71	25.25	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	6,808.92	2.59	420.82	0.16	9,125.97	3.47
9.0	5.23	158.80	1	1065.71	28.18	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	7,599.02	2.89	420.82	0.16	9,916.07	3.77
10.0	5.23	158.80	1	1065.71	31.10	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	8,386.43	3.19	420.82	0.16	10,703.48	4.07
11.0	5.23	158.80	1	1065.71	34.03	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	9,176.53	3.49	420.82	0.16	11,493.58	4.37
12.0	5.23	158.80	1	1065.71	36.95	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	9,963.94	3.79	420.82	0.16	12,280.99	4.67
13.0	5.23	158.80	1	1065.71	39.88	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	10,754.04	4.09	420.82	0.16	13,071.09	4.97
14.0	5.23	158.80	1	1065.71	42.81	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	11,544.14	4.39	420.82	0.16	13,861.20	5.27
15.0	5.23	158.80	1	1065.71	45.73	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	12,331.55	4.69	420.82	0.16	14,648.61	5.57
16.0	5.23	158.80	1	1065.71	48.66	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	13,121.66	4.99	420.82	0.16	15,438.71	5.87
17.0	5.23	158.80	1	1065.71	51.58	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	13,909.06	5.29	420.82	0.16	16,226.12	6.17
18.0	5.23	158.80	1	1065.71	54.47	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	14,688.38	5.59	420.82	0.16	17,005.43	6.47
19.0	5.23	158.80	1	1065.71	57.44	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	15,489.27	5.89	420.82	0.16	17,806.32	6.77
20.0	5.23	158.80	1	1065.71	60.36	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	16,276.68	6.19	420.82	0.16	18,593.73	7.07
25.0	5.23	158.80	1	1065.71	74.99	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	20,221.80	7.69	420.82	0.16	22,538.86	8.57
30.0	5.23	158.80	1	1065.71	89.62	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	24,166.93	9.19	420.82	0.16	26,483.98	10.07
35.0	5.23	158.80	1	1065.71	104.25	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	28,112.06	10.69	420.82	0.16	30,429.11	11.57
40.0	5.23	158.80	1	1065.71	118.88	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	32,057.18	12.19	420.82	0.16	34,374.23	13.08
45.0	5.23	158.80	1	1065.71	133.42	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	35,978.04	13.69	420.82	0.16	38,295.09	14.57
50.0	5.23	158.80	1	1065.71	148.14	269.66	2.65	158.80	830.52	0.32	1065.71	0.41	39,947.43	15.19	420.82	0.16	42,264.49	16.08

CASE-2 MAIN EQUIPMENT :SHOVEL(18M3)+ DUMP TRUCK(195T)

Table II.2.2.1-2(b)

(FILE : PANAM-13)

PRODUCTIVITY 1,578 M3/Hr (COMMON EXCAVATION)

HAULAGE L	① EXCAVATION BULLDOZER (98TON)		② LOADING SHOVEL (18M3)		③ HAULING DUMP TRUCK (195TON)		④ GRADING BULLDOZER (65TON)		UNIT COST EXCAVATION BY ①		UNIT COST LOADING BY ②		UNIT COST HAULING BY ③		UNIT COST GRADING BY ④		TOTAL UNIT COST	
	U. COST		U. COST		U. COST		U. COST											
(KM)	EQUIP. NO.	($/Hr)	EQUIP. NO.	($/Hr)	EQUIP. NO.	($/Hr)	EQUIP. NO.	($/Hr)	($/H)	($/M3)	($/H)	($/M3)	($/H)	($/M3)	($/H)	($/M3)	(U$/H)	($/M3)
0.5	3.17	158.80	1	605.21	2.80	220.42	1.85	100.82	503.40	0.32	605.21	0.38	617.18	0.39	186.52	0.12	1,912.30	1.21
1.0	3.17	158.80	1	605.21	3.92	220.42	1.85	100.82	503.40	0.32	605.21	0.38	864.05	0.55	186.52	0.12	2,159.17	1.37
2.0	3.17	158.80	1	605.21	6.17	220.42	1.85	100.82	503.40	0.32	605.21	0.38	1,359.99	0.86	186.52	0.12	2,655.11	1.68
3.0	3.17	158.80	1	605.21	8.41	220.42	1.85	100.82	503.40	0.32	605.21	0.38	1,853.73	1.17	186.52	0.12	3,148.86	2.00
4.0	3.17	158.80	1	605.21	10.66	220.42	1.85	100.82	503.40	0.32	605.21	0.38	2,349.68	1.49	186.52	0.12	3,644.80	2.31
5.0	3.17	158.80	1	605.21	12.91	220.42	1.85	100.82	503.40	0.32	605.21	0.38	2,845.62	1.80	186.52	0.12	4,140.75	2.62
														0.00				
6.0	3.17	158.80	1	605.21	15.15	220.42	1.85	100.82	503.40	0.32	605.21	0.38	3,339.36	2.12	186.52	0.12	4,634.49	2.94
7.0	3.17	158.80	1	605.21	17.40	220.42	1.85	100.82	503.40	0.32	605.21	0.38	3,835.31	2.43	186.52	0.12	5,130.43	3.25
8.0	3.17	158.80	1	605.21	19.65	220.42	1.85	100.82	503.40	0.32	605.21	0.38	4,331.25	2.74	186.52	0.12	5,626.38	3.57
9.0	3.17	158.80	1	605.21	21.89	220.42	1.85	100.82	503.40	0.32	605.21	0.38	4,824.99	3.06	186.52	0.12	6,120.12	3.88
10.0	3.17	158.80	1	605.21	24.14	220.42	1.85	100.82	503.40	0.32	605.21	0.38	5,320.94	3.37	186.52	0.12	6,616.06	4.19
														0.00				
11.0	3.17	158.80	1	605.21	26.39	220.42	1.85	100.82	503.40	0.32	605.21	0.38	5,816.88	3.69	186.52	0.12	7,112.01	4.51
12.0	3.17	158.80	1	605.21	28.63	220.42	1.85	100.82	503.40	0.32	605.21	0.38	6,310.62	4.00	186.52	0.12	7,605.75	4.82
13.0	3.17	158.80	1	605.21	30.88	220.42	1.85	100.82	503.40	0.32	605.21	0.38	6,806.57	4.31	186.52	0.12	8,101.69	5.13
14.0	3.17	158.80	1	605.21	33.13	220.42	1.85	100.82	503.40	0.32	605.21	0.38	7,302.51	4.63	186.52	0.12	8,597.64	5.45
15.0	3.17	158.80	1	605.21	35.37	220.42	1.85	100.82	503.40	0.32	605.21	0.38	7,796.26	4.94	186.52	0.12	9,091.38	5.76
16.0	3.17	158.80	1	605.21	37.62	220.42	1.85	100.82	503.40	0.32	605.21	0.38	8,292.20	5.25	186.52	0.12	9,587.32	6.08
17.0	3.17	158.80	1	605.21	39.87	220.42	1.85	100.82	503.40	0.32	605.21	0.38	8,788.15	5.57	186.52	0.12	10,083.27	6.39
18.0	3.17	158.80	1	605.21	42.08	220.42	1.85	100.82	503.40	0.32	605.21	0.38	9,275.27	5.88	186.52	0.12	10,570.40	6.70
19.0	3.17	158.80	1	605.21	44.36	220.42	1.85	100.82	503.40	0.32	605.21	0.38	9,777.83	6.20	186.52	0.12	11,072.95	7.02
20.0	3.17	158.80	1	605.21	46.61	220.42	1.85	100.82	503.40	0.32	605.21	0.38	10,273.78	6.51	186.52	0.12	11,568.90	7.33
25.0	3.17	158.80	1	605.21	57.84	220.42	1.85	100.82	503.40	0.32	605.21	0.38	12,749.09	8.08	186.52	0.12	14,044.22	8.90
30.0	3.17	158.80	1	605.21	69.08	220.42	1.85	100.82	503.40	0.32	605.21	0.38	15,226.61	9.65	186.52	0.12	16,521.74	10.47
35.0	3.17	158.80	1	605.21	80.31	220.42	1.85	100.82	503.40	0.32	605.21	0.38	17,701.93	11.22	186.52	0.12	18,997.05	12.04
40.0	3.17	158.80	1	605.21	91.55	220.42	1.85	100.82	503.40	0.32	605.21	0.38	20,179.45	12.79	186.52	0.12	21,474.57	13.61
45.0	3.17	158.80	1	605.21	102.68	220.42	1.85	100.82	503.40	0.32	605.21	0.38	22,632.73	14.34	186.52	0.12	23,927.85	15.16
50.0	3.17	158.80	1	605.21	114.01	220.42	1.85	100.82	503.40	0.32	605.21	0.38	25,130.08	15.93	186.52	0.12	26,425.21	16.75

CASE-3 MAIN EQUIPMENT :SHOVEL(15M3)+ DUMP TRUCK(150T)

Table II.2.2.1-2(c)

(FILE : PANAM-13)

PRODUCTIVITY 1,315 M3/Hr (COMMON EXCAVATION)

HAULAGE L	① EXCAVATION BULLDOZER (98TON)		② LOADING SHOVEL (15M3)		③ HAULING DUMP TRUCK (150TON)		④ GRADING BULLDOZER (65TON)		UNIT COST EXCAVATION BY ①		UNIT COST LOADING BY ②		UNIT COST HAULING BY ③		UNIT COST GRADING BY ④		TOTAL UNIT COST	
	U. COST		U. COST		U. COST		U. COST											
(KM)	EQUIP. NO.	(U$/Hr)	EQUIP. NO.	($/Hr)	EQUIP. NO.	($/Hr)	EQUIP. NO.	($/Hr)	($/H)	($/M3)	($/H)	($/M3)	($/H)	($/M3)	($/H)	($/M3)	(U$/H)	($/M3)
0.5	2.64	158.80	1	500.62	2.89	179.43	1.54	100.82	419.23	0.32	500.62	0.38	518.55	0.39	155.26	0.12	1,593.67	1.21
1.0	2.64	158.80	1	500.62	4.08	179.43	1.54	100.82	419.23	0.32	500.62	0.38	732.07	0.56	155.26	0.12	1,807.19	1.37
2.0	2.64	158.80	1	500.62	6.64	179.43	1.54	100.82	419.23	0.32	500.62	0.38	1,191.42	0.91	155.26	0.12	2,266.53	1.72
3.0	2.64	158.80	1	500.62	8.84	179.43	1.54	100.82	419.23	0.32	500.62	0.38	1,586.16	1.21	155.26	0.12	2,661.28	2.02
4.0	2.64	158.80	1	500.62	11.22	179.43	1.54	100.82	419.23	0.32	500.62	0.38	2,013.20	1.53	155.26	0.12	3,088.32	2.35
5.0	2.64	158.80	1	500.62	13.60	179.43	1.54	100.82	419.23	0.32	500.62	0.38	2,440.25	1.86	155.26	0.12	3,515.36	2.67
6.0	2.64	158.80	1	500.62	15.98	179.43	1.54	100.82	419.23	0.32	500.62	0.38	2,867.29	2.18	155.26	0.12	3,942.41	3.00
7.0	2.64	158.80	1	500.62	18.35	179.43	1.54	100.82	419.23	0.32	500.62	0.38	3,292.54	2.50	155.26	0.12	4,367.66	3.32
8.0	2.64	158.80	1	500.62	20.73	179.43	1.54	100.82	419.23	0.32	500.62	0.38	3,719.58	2.83	155.26	0.12	4,794.70	3.65
9.0	2.64	158.80	1	500.62	23.11	179.43	1.54	100.82	419.23	0.32	500.62	0.38	4,146.63	3.15	155.26	0.12	5,221.74	3.97
10.0	2.64	158.80	1	500.62	25.49	179.43	1.54	100.82	419.23	0.32	500.62	0.38	4,573.67	3.48	155.26	0.12	5,648.79	4.30
11.0	2.64	158.80	1	500.62	27.87	179.43	1.54	100.82	419.23	0.32	500.62	0.38	5,000.71	3.80	155.26	0.12	6,075.83	4.62
12.0	2.64	158.80	1	500.62	30.25	179.43	1.54	100.82	419.23	0.32	500.62	0.38	5,427.76	4.13	155.26	0.12	6,502.87	4.95
13.0	2.64	158.80	1	500.62	32.62	179.43	1.54	100.82	419.23	0.32	500.62	0.38	5,853.01	4.45	155.26	0.12	6,928.12	5.27
14.0	2.64	158.80	1	500.62	35.00	179.43	1.54	100.82	419.23	0.32	500.62	0.38	6,280.05	4.78	155.26	0.12	7,355.16	5.59
15.0	2.64	158.80	1	500.62	37.38	179.43	1.54	100.82	419.23	0.32	500.62	0.38	6,707.09	5.10	155.26	0.12	7,782.21	5.92
	2.64	158.80																
16.0	2.64	158.80	1	500.62	39.76	179.43	1.54	100.82	419.23	0.32	500.62	0.38	7,134.14	5.43	155.26	0.12	8,209.25	6.24
17.0	2.64	158.80	1	500.62	42.14	179.43	1.54	100.82	419.23	0.32	500.62	0.38	7,561.18	5.75	155.26	0.12	8,636.30	6.57
18.0	2.64	158.80	1	500.62	44.56	179.43	1.54	100.82	419.23	0.32	500.62	0.38	7,995.40	6.08	155.26	0.12	9,070.52	6.90
19.0	2.64	158.80	1	500.62	46.89	179.43	1.54	100.82	419.23	0.32	500.62	0.38	8,413.47	6.40	155.26	0.12	9,488.59	7.22
20.0	2.64	158.80	1	500.62	49.27	179.43	1.54	100.82	419.23	0.32	500.62	0.38	8,840.52	6.72	155.26	0.12	9,915.63	7.54
25.0	2.64	158.80	1	500.62	61.17	179.43	1.54	100.82	419.23	0.32	500.62	0.38	10,975.73	8.35	155.26	0.12	12,050.85	9.16
30.0	2.64	158.80	1	500.62	73.06	179.43	1.54	100.82	419.23	0.32	500.62	0.38	13,109.16	9.97	155.26	0.12	14,184.27	10.79
35.0	2.64	158.80	1	500.62	84.95	179.43	1.54	100.82	419.23	0.32	500.62	0.38	15,242.58	11.59	155.26	0.12	16,317.69	12.41
40.0	2.64	158.80	1	500.62	96.84	179.43	1.54	100.82	419.23	0.32	500.62	0.38	17,376.00	13.21	155.26	0.12	18,451.12	14.03
								100.82										
45.0	2.64	158.80	1	500.62	108.85	179.43	1.54	100.82	419.23	0.32	500.62	0.38	19,530.96	14.85	155.26	0.12	20,606.07	15.67
								100.82										
50.0	2.64	158.80	1	500.62	120.62	179.43	1.54	100.82	419.23	0.32	500.62	0.38	21,642.85	16.46	155.26	0.12	22,717.96	17.28

CASE-4 MAIN EQUIPMENT :MOTOR SCRAPER(34M3)

Table II.2.2.1-2(d)

PRODUCTIVITY 2,958 M3/Hr

(FILE : PANAM-13)

(COMMON EXCAVATION)

HAULAGE L	① PUSHING BULLDOZER (98TON)		② LOADING M. SCRAPER (34M3)		③ HAULING DUMP TRUCK (150TON)		④ GRADING BULLDOZER (65TON)		UNIT COST PUSHING BY ①		UNIT COST EXCAVATION HAULING BY ②		UNIT COST HAULING BY ③		UNIT COST GRADING BY ④		TOTAL UNIT COST	
	U. COST		U. COST		U. COST		U. COST											
(KM)	EQUIP. NO.	($/Hr)	EQUIP. NO.	($/Hr)	EQUIP. NO.	($/Hr)	EQUIP. NO.	($/Hr)	($/H)	($/M3)	($/H)	($/M3)	($/H)	($/M3)	($/H)	($/M3)	($/H)	($/M3)
0.2	4.20	158.80	8.47	262.42	-	179.43	3.45	100.82	666.96	0.23	2222.6974	0.75	-	-	347.83	0.12	3,237.49	1.09
0.5	4.20	158.80	14.79	262.42	-	179.43	3.45	100.82	666.96	0.23	3881.1918	1.31	-	-	347.83	0.12	4,895.98	1.66
1.0	4.20	158.80	25.28	262.42	-	179.43	3.45	100.82	666.96	0.23	6633.9776	2.24	-	-	347.83	0.12	7,648.77	2.59
2.0	4.20	158.80	46.20	262.42	-	179.43	3.45	100.82	666.96	0.23	12123.804	4.10	-	-	347.83	0.12	13,138.59	4.44
3.0	4.20	158.80	67.20	262.42	-	179.43	3.45	100.82	666.96	0.23	17634.624	5.96	-	-	347.83	0.12	18,649.41	6.30
4.0	4.20	158.80	89.60	262.42	-	179.43	3.45	100.82	666.96	0.23	23512.832	7.95	-	-	347.83	0.12	24,527.62	8.29
5.0	4.20	158.80	109.50	262.42	-	179.43	3.45	100.82	666.96	0.23	28734.99	9.71	-	-	347.83	0.12	29,749.78	10.06

CASE-BELT CONVEYOR SYSTEM
MAIN BELT W1=2.8M BRANCH BELT W2=2.0M

Table II.2.2.1-2(e)

TOTAL VOLUME 241,800,000 M3 (COMMON EXCAVATION)

HAULAGE L	① CONVEYING BELT CONVEYOR		② EXCAVATION BULLDOZER (98TON)	③ LOADING SHOVEL (30M3)	④ HAULING DUMP TRUCK (240TON)	⑤ GRADING BULLDOZER (98TON)	TOTAL UNIT COST	REMARKS
(KM)	($)	($/M3)	($/M3)	($/M3)	($/M3)	($/M3)	($/M3)	
0.5								
1.0	546,468,000	2.26	0.32	0.41	0.34	0.16	3.49	Hour rates for
2.0	606,918,000	2.51	0.32	0.41	0.34	0.16	3.74	D.Truck,Bulldozer and Shovel
3.0	669,786,000	2.77	0.32	0.41	0.34	0.16	4.00	are referred to Table
4.0	720,564,000	2.98	0.32	0.41	0.34	0.16	4.21	II.2.2.1-2a.
5.0	773,760,000	3.20	0.32	0.41	0.34	0.16	4.43	
10.0	1,080,846,000	4.47	0.32	0.41	0.34	0.16	5.70	
20.0	1,675,674,000	6.93	0.32	0.41	0.34	0.16	8.16	

NOTE :

* HAULING DISTANCE FOR DUMP TRUCK : 0.5KM

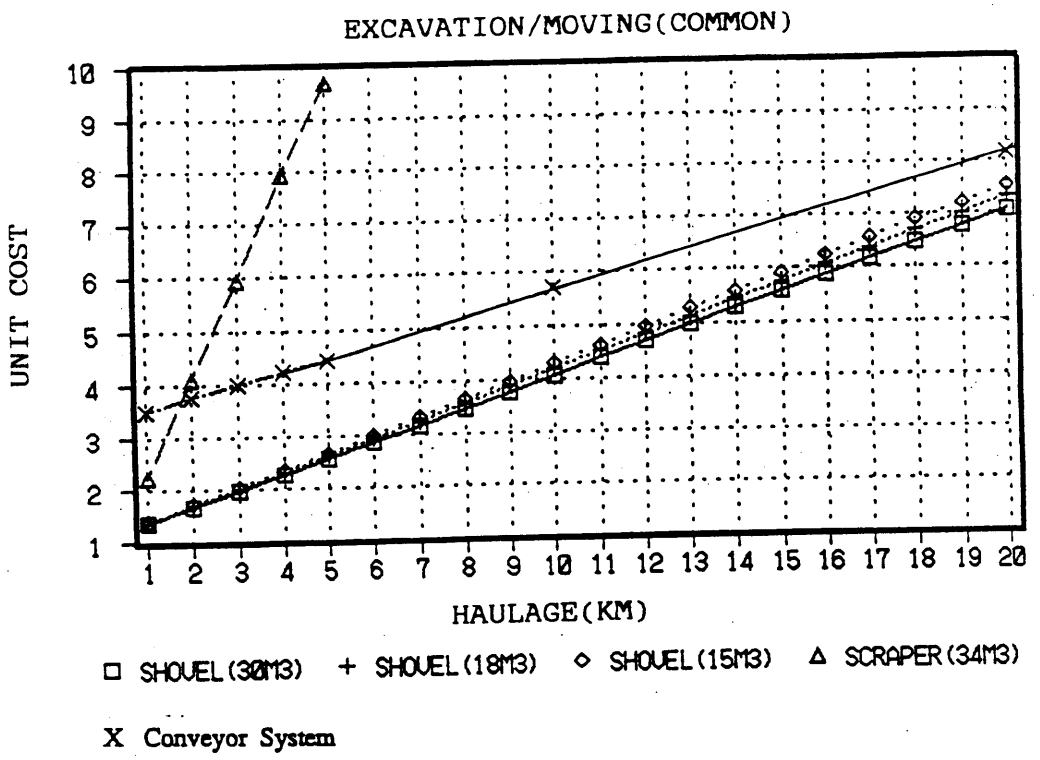


Fig. II.2.2.1 - 5 Unit Cost ($/m3)

2.2 Dredging

(1) Combination of Dredging Equipment

The combination of dredging equipment is to be decided depending on the nature of material to be dredged. Unit dredge groups have their own production rates and numbers of units will be decided by considering volume of excavation per year v.s. production rate per year.

1) Clayey Material

Dredging of clay and silt is to be carried out using the Grab Dredger ($50m^3$) - the largest size available. Dredged materials are to be loaded onto large split barges ($5,000m^3$) in order to minimize the construction period, reduce the number of vessels required and minimize interference to canal traffic. It is not considered using a cutter suction system with a view to preventing seawater pollution. The dredged materials are to be transported by $5,000m^3$ split barges and 3,200 HP pusher tugs.
A sufficient number of barges and tugs are to be provided in consideration of the distance to the dumping ground and maintaining maximum production for the Grab Dredger.
To economize the operation, one split barge will be moored to the Dredger without a pusher tug and it will be arranged so that other barges are back at the Dredger before the split barge is filled up.

Table II. 2.2.2 - 1 Unit Dredge Group for Grab Dredger

Grab Dredger	$50m^3$	3,000 HP	1 unit
Anchor Boat		600 HP	1 unit
Passenger Boat		180 HP	0.5 unit*
Split Barge	$5,000m^3$		depending on hauling distance
Pusher tug		3,200 HP	ditto

* The passenger boat serves two unit dredge groups.

The Grab Dredger has a production of 5,745,600 m^3/year.

If sufficient water depth is available in the operation areas, the Trailing Suction Hopper Dredger can be employed for dredging clayey materials.

Table II.2.2.2 - 2 Unit Dredge Group for Trailing Suction Hopper Dredger

Trailing Dredger	$10,000m^3$	19,000 HP	1 unit
Survey Boat		420 HP	1 unit
Passenger Boat		180 HP	1 unit

The dredging production of the Trailing Suction Hopper Dredger depends on its hauling distance.

2) Loose Sand and Gravel

The Cutter Suction Dredger (27,500 HP) is the most suitable for such materials. In the case where the placing area is within 2 km of the dredging site, dredged materials are directly disposed of through floating, submerged or land pipelines.
Where the distance to the placing area is more than 2 km, dredged materials are loaded onto $5,000m^3$ split barges using a floating pipeline and loading barge. The arrangement of barges and pusher tugs is the same as 1) above.

Table II. 2.2.2 - 3 Unit Dredge Group for Cutter Suction Dredger

Cutter Suction Dredger		27,500 HP	1 unit
Anchor Boat		400 HP	1 unit
Landing Barge		250 HP	1 unit
Passenger Boat		180 HP	0.5 unit*
Survey/Work Boat		600 HP	0.5 unit*
Crane Barge		80 t	0.5 unit*
Split Barge	$5,000m^3$		depending on hauling distance
Pusher Tug		3,200 HP	ditto

* Each passenger boat, survey/work boat and crane barge serves two groups.
The Cutter Suction Dredger has a production of 9,336,800 m^3/year.

If sufficient water depth is available in the operation areas, the Trailing Suction Hopper Dredger can be employed for dredging loose sand and gravel. The Arrangement of the Dredging group is shown in Table II.2.2.2 - 2.

3) Weathered Rock

Dredging of weathered rock is carried out using the Cutter Suction Dredger (27,500 HP) with the 5,000 HP cutter - the largest size available. Dredged materials are loaded onto the $5,000m^3$ split barge using the floating pipeline and loading barge as in the case of 2) above.
The arrangement of barges and pusher tugs is the same as case 1) above.

Boulders that cannot be broken into smaller pieces are removed by the Grab Dredger later.

Table II. 2.2.2 - 4 Unit Dredge Group for Cutter Suction Dredger (Weathered Rock)

Cutter Suction Dredger		27,500 HP	1 unit
Anchor Boat		600 HP	1 unit
Loading Barge		250 HP	1 unit
Passenger Boat		180 HP	0.5 unit*
Survey/Work Boat		600 HP	0.5 unit*
Crane Barge		30 t	0.5 unit*
Split Barge	5,000m^3		depending on hauling distance
Pusher Tug		3,200 HP	ditto

* Each passenger boat, survey/work boat and crane barge serves for two unit dredge groups.

The Cutter Suction Dredger has a production of 7,182,000 m^3/year.

If boulders are encountered during the above dredging operation, the unit dredge group for Grab Dredger in Table II.2.2.2 - 7 will be employed.

4) Medium Hard Rock

The Cutter Suction Dredger with the biggest rated capacity (27,500 HP) and 5,000m^3 split barges are employed for dredging medium hard rock.
As cutting harder materials requires more frequent repair/maintenance, the average operation hours per day are reduced to 18 hours.
Boulders cannot be broken into smaller pieces using the cutter, so they are removed using the Grab Dredger.
Blasted materials cannot be dredged by the Cutter Suction Dredger.

Table II. 2.2.2 - 5 Unit Dredge Group for Cutter Suction Dredger (Medium hard rock)

Cutter Suction Dredger		27,500 HP	1 Unit
Anchor Boat		600 HP	1 Unit
Loading Barge		250 HP	1 Unit
Passenger Boat		180 HP	0.5 Unit
Survey/Work Boat		600 HP	0.5 Unit
Crane Barge		30 t	0.5 Unit
Split Barge	5,000m^3		depends on distance
Pusher Tug		3,200 HP	depends on distance

The Cutter Suction Dredger has a production of 3,591,000 m^3/year.

If boulders are encountered during the above dredging operation, the unit dredge group for Grab Dredger in Table II.2.2.2-7 is employed.

5) Hard Rock

Blasting is carried out prior to dredging. The Grab Dredger (50m^3) is employed to dredge blasted materials.
Blasted materials are loaded into split barges (5,000m^3).
In case the hourly production of the Grab Dredger is rather low, it is more economical to use 1,000m^3 split barges.
An 800 HP pusher/tug boat is used together with the 1,000m^3 split barges.

Table II. 2.2.2 - 6 Unit Drilling Group for Hard Rock Excavation

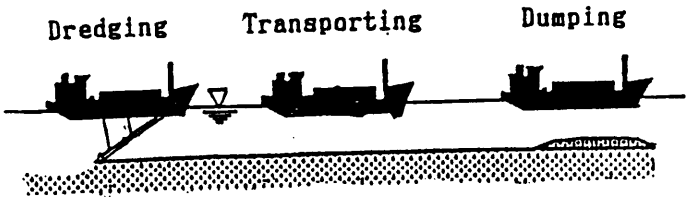
*SEP L35.0 x B20.0 x H3.4m with 8 sets of OD-Drilling Machine	1,500 ton	1 Unit
Floating Crane	500 ton	1 Unit
Tug Boat	500 HP	1 Unit
Anchor Boat	600 Hp	1 Unit
Mixing Barge L30.0 x B12.0 x H.2.3m		1 Unit
Diver Boat	100 HP	1 Unit
Warning Boat	800 HP	3 Units
Passenger Boat	180 HP	1 Unit

*SEP: Self elevated platform

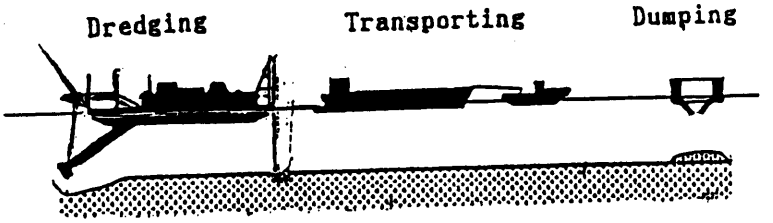
Table II. 2.2.2 - 7 Unit Dredge Group for Grab Dredger

Grab Dredger	$50m^3$	3,000 HP	1 Unit
Anchor Boat		600 HP	1 Unit
Passenger Boat		180 HP	0.5 Unit
Split Barge	$1,000m^3$		depend on distance
Pusher Tug		800 HP	depend on distance

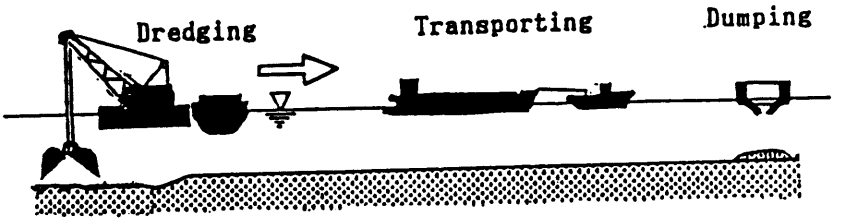
The above dredging fleet (grab dredger, $50m^3$) gives a production of 1,388,520 m^3/year.



Trailing hopper suction dredging

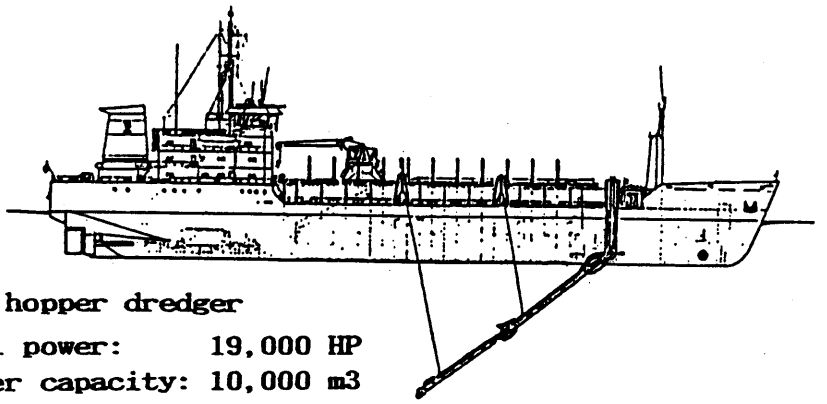


Cutter suction dredging



Grab dredging

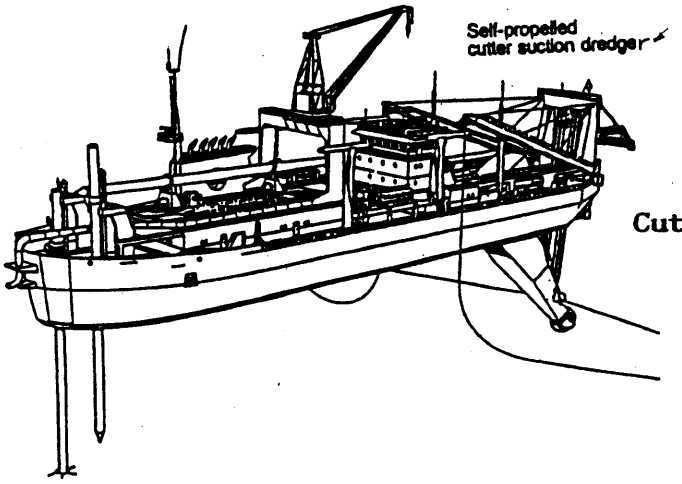
Fig. II.2.2.2 - 1 Typical dredging Procedure



Trailing hopper dredger

Total power: 19,000 HP

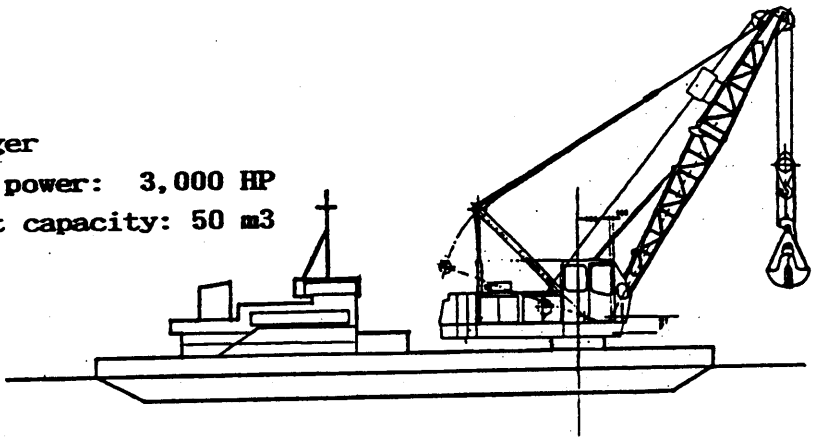
Hopper capacity: 10,000 m3



Cutter suction dredger

Total power: 27,500 HP

Cutter power: 6,000 HP



Grab dredger

Total power: 3,000 HP

Bucket capacity: 50 m3

Fig. II.2.2.2 - 2 Types of Dredgers

II.2. Chapter 3. Basic Conditions of Construction Plans

3.1 Working Days and Hours

The available working days (per annum) varies with the type of work, local meteorology, and local traditional holidays. The types of main works are classified as dry excavation, dredging, concrete works and others along with available working days.

(1) Dry Excavation (Earth & Rock Moving)

Available working days per annum are worked out to be 249 days (68%), on the assumption that the works cannot be carried out when rainfall equals and/or exceeds 15 mm/day.

Operator hours per day

Two shift operation is applied for the works
Operator hours: 10 hours/shift x 2 shifts
= 20 hours

Equipment
Operating hours: 9 hours/shift x 2 shifts
= 18 hours

(2) Dredging (Earth & Rock removal under Water)

Canal Excavation

It is generally held that the dredging works are not seriously affected by rainfall; available working days for canal excavation are assumed to be 266 days per annum (72%).

Operator hours per day

Two shift operation is applied for the Canal excavation
Operator hours: 12 hours/shift x 2 shifts
= 24 hours/day

Equipment
operating hours: 10 hours x 2 shifts
= 20 hours/day

Offshore Excavation

Based on the marine conditions on the Pacific and Atlantic Ocean sides. (Refer to Ocean Wave Statistics of (National Physical Laboratory)

Percentages of operable days for both trailer suction hopper dredger and cutter/grab dredger are estimated as follows.

Atlantic Ocean side	Pacific Ocean side	
164 days (44.8%)	266 days (72.0%)	for Excavation
164 days (44.8%)	178 days (48.9%)	for Breakwater

Operator hours per day

One shift operation is applied for the offshore works
Operator hours: 12 hours/shift x 1 shift
= 12 hours/day
Equipment
Operating hours: 10 hours x 1 shift
= 10 hours/day

(3) Concrete Works for the Lock

Available working days: 249 days per annum

Operator hours per day

Two shift operation is applied for the works
Operator hours: 10 hours/shift x 2 shifts
= 20 hours
Equipment
Operating hours: 9 hours/shift x 2 shifts
= 18 hours

(4) Data for available working days per annum

1) Earthwork, etc.
On the assumption that earth/rock moving by means of heavy duty equipment ceases when rainfall is more than 15mm/day

Table II.2.3.1-1 Available working days per annum on the Pacific Side (Totumen *)

(RAINFALL >= 15mm) (FILE:¥PANAM-06)

		JAN (DAY)	FEB (DAY)	MAR (DAY)	APR (DAY)	MAY (DAY)	JUN (DAY)	JUL (DAY)	AUG (DAY)	SEPT (DAY)	OCT (DAY)	NOV (DAY)	DEC (DAY)	TOTAL (DAYS)
1982		2	0	0	1	5	4	4	3	7	4	3	0	33
1983		0	0	0	2	6	7	4	5	4	8	6	3	45
1984		0	2	0	0	4	4	8	5	10	7	9	0	49
1985		0	0	0	0	2	3	5	2	7	7	2	2	30
1986		0	0	0	1	4	6	4	5	3	7	6	1	37
1987		0	0	0	2	5	4	5	5	5	9	6	2	43
1988		0	0	0	1	6	5	3	5	7	14	6	3	50
1989		0	0	0	0	2	5	1	6	6	3	11	4	38
1990		0	1	0	0	5	2	6	4	3	8	3	2	34
1991		0	0	0	2	3	5	2	3	7	5	5	0	32
TOTAL		2	3	0	9	42	45	42	43	59	72	57	17	391
AVE.		0.2	0.3	0	0.9	4.2	4.5	4.2	4.3	5.9	7.2	5.7	1.7	39.1
N.A.W.D*	①	0	0	0	1	4	5	4	4	6	7	6	2	39
P.HOLIDAY		2	1	1	0	1	0	0	1	0	0	3	2	11
SUNDAY		4	4	5	4	5	4	5	4	4	5	4	4	52
E. MAINT.*		1	1	1	1	1	1	1	1	1	1	1	1	12
SUBTOTAL	②	7	6	7	5	7	5	6	6	5	6	8	7	75
A.N.A.W.D*	③	7	6	7	6	10	9	9	9	10	12	12	9	106
A.W.D*	④	24	22	24	24	21	21	22	22	20	19	18	22	259
TOTAL DAYS	⑤	31	28	31	30	31	30	31	31	30	31	30	31	365

* Rainfall records presented by 'INSTITUTO DE RECURSOS Y ELECTRIFICATION DEPARTAMENTO DE HIDROMETEOROLOGIA SECCTION DE CLIMATOLOGIA')

* N.A.W.D : Non available working days modified due to rainfall

* A.N.A.W.D : Average nonavailale working days = {(⑤-②)/⑤}*①+②

* A.W.D : Available working days

* E. MAINT. : Equipment maintenance

Table II. 2. 3. 1-2 Available working days per annum on the Atlantic Side (San Pedro)

(RAINFALL >= 15mm)

		JAN (DAY)	FEB (DAY)	MAR (DAY)	APR (DAY)	MAY (DAY)	JUN (DAY)	JUL (DAY)	AUG (DAY)	SEPT (DAY)	OCT (DAY)	NOV (DAY)	DEC (DAY)	TOTAL (DAYS)
1982		5	0	0	2	2	8	10	4	10	8	2	3	54
1983		0	0	0	2	7	6	4	6	15	7	10	12	69
1984		3	3	0	0	6	15	4	10	6	7	7	0	61
1985		4	0	0	1	8	7	6	7	4	11	6	2	56
1986		0	0	0	1	4	12	7	4	4	10	3	0	45
1987		0	0	0	6	12	6	8	11	9	13	9	6	80
1988		0	0	0	1	6	8	11	8	11	12	13	5	75
1989		0	0	0	0	1	4	15	9	6	17	12	2	66
1990		0	0	0	1	9	7	10	11	13	12	13	4	80
1991		0	0	0	0	7	8	5	5	14	9	17	0	65
TOTAL		12	3	0	14	62	81	80	75	92	106	92	34	651
AVE.		1. 2	0. 3	0	1. 4	6. 2	8. 1	8	7. 5	9. 2	10. 6	9. 2	3. 4	65. 1
N. A. W. D*	①	1	0	0	1	6	8	8	8	9	11	9	3	64
P. HOLIDAY		2	1	1	0	1	0	0	1	0	0	3	2	11
SUNDAY		4	4	5	4	5	4	5	4	4	5	4	4	52
E. MAINT. *		1	1	1	1	1	1	1	1	1	1	1	1	12
SUB TOTAL	②	7	6	7	5	7	5	6	6	5	6	8	7	75
A. N. A. W. D*	③	8. 0	6. 0	7. 0	6. 0	12. 0	12. 0	12. 0	12. 0	13. 0	15. 0	15. 0	9. 0	127
A. W. D*	④	23	22	24	24	19	18	19	19	17	16	15	22	238
TOTAL DAYS	⑤	31	28	31	30	31	30	31	31	30	31	30	31	365

Available working days per annum are assumed on the basis of Meteorological data recorded for over past 10 years both on the Pacific Side and the Atlantic Side

On the Pacific Side	259 days per annum	(71%)
On the Atlantic Side	238 days per annum	(65%)
Average available working days	249 days per annum	(68%)

2) Dredging Inland Canal

Available working days per annum for dredging Canal

Dredging works are generally not so seriously affected by rainfall.
Work can proceed regardless of rainfall per annum.

Taking the past records(Suez Canal, etc.) regarding a period of annual operating days for dredging works into consideration, the available working days are obtained in the following manner :

Periodical Maintenance	--->	15 % * 365 days	=	55 days
Loss days due to	--->	12 % * 365 days	=	44 days

: Repair of mechanical defects

: Shifting dredging vessels to the operating spots

: Other incidents

Therefore, the average working days per annum are as given below
365 - (55 + 44) = 266 days (72 %)

3.2 Load Factor, Unit Weight

(1) Dry Excavation

Material types affect the performance and production of earth/rock moving operations. to understand how materials affect the performance, it is necessary to consider the density, swell, void, and amount of shrinkage.

Characteristics of materials to be taken into consideration in the cost estimation of earth/rock moving are given in the table below.

Table II.2.3.2-1 Load Factor and Unit Weight

(FILE:PANAMA¥FINAL¥PANAM-07)

Materials	State Materials	Weight (Ton/m3)	Swell (L)*	Load Factor (F=1/L)	Shrinkage (C)*	Remarks
Earth	Natural Bed	1.80 *	1.20 *	0.83 *	0.90 *	
Clay	Natural Bed	1.80 *	1.30 *	0.77 *	0.90 *	
			(1.2)	(0.83)		
Sand	Natural Bed	1.80 *	1.20 *	0.83 *	0.95 *	
Soft Rock	Natural Bed	2.20 *	1.50 *	0.67 *	1.20 *	
Hard Rock	Natural Bed	2.50 *	1.65 *	0.61 *	1.40 *	
			(1.7)	(0.59)		

Note :

L : Ratio of bulk in loosened state or haul unit (m3) to natural bed state(m3)

C : Ratio of bulk in compacted state(m3) to natural bed state(m3)

* Weight, swell and shrinkage factor are obtained from the Standard for Civil Works Estimation, Ministry of Construction, Japan'

Load factor = 1 / L (Once L is given, Load factor is defined)

Bankyards(m3) = haul unit capacity(m3) x load factor

For cost estimation Load Factors recomended by Ministry of Construction, Japan are followed except for Clay and Hard Rock, Load factor for Clay and Hard rock are somewhat modified for the estimate due to our experience.
Modified figures for the Load Factor are shown in parenthesis.

(2) Dredging

Table II.2.3.2-2 Load Factor and Unit Weight

(FILE:PANAM-07)

Materials	State Materials	Weight (Ton/m3)	Load Factor To Barge By Cutter	Load Factor To Barge By Grab	Remarks
Earth	Natural Bed	1.80 *	0.65	0.75	
Clay	Natural Bed	1.80 *	0.33	0.75	
Sand	Natural Bed	1.80 *	0.80	0.80	
Soft Rock	Natural Bed	2.20 *	0.80	0.80	
Hard Rock	Natural Bed	2.50 *	-	0.70	

Volume of loaded material in natural bed state is calculated as;

Barge Hopper Capacity x Load Factor

3.3 Dewatering System

(1) Bench Cut Method

The bench cut method (20m bench) is used as a standard and the lowest bench (Canal excavation) below the datum level is to be done by blasting in one pass. Bench slope finishing is carried out at every bench. The access road for access to each bench is provided. The open drainage ditch is provided at each berm.

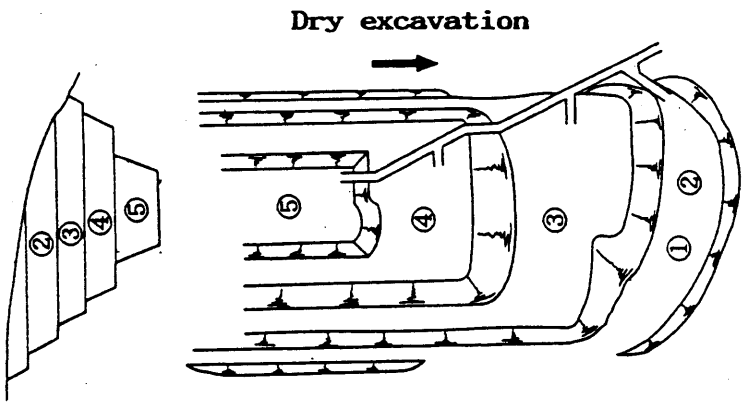


Fig. II. 2.3.3 - 1 Layout of Bench

(2) Dewatering System for Canal Excavation

1) Prior to commencing excavation of the Canal, open ditches will be excavated along the top of each slope. Any flow along these ditches will be led outside the construction area.

2) Open ditches for dewatering will also be provided along every 20m small step on the slope, then excavation of the Canal will start. At the deepest part of excavation, pumps for dewatering will be provided, pumping up water into the open ditches.

3) The open ditches on the lowest step on slopes shall be dug down to the water level of the Canal.

4) To divert water from streams and marshes that cross the canal or that flow into the excavation area, bypass ditches will be excavated. Refer to Fig. II. 2.3.3 - 2.

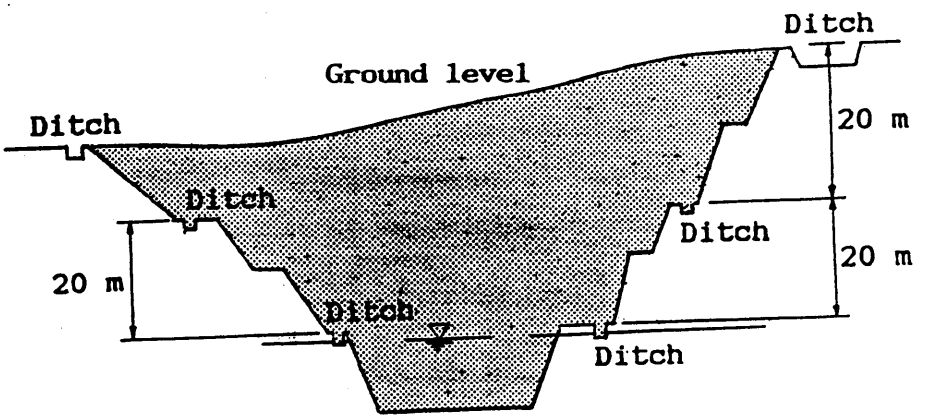


Fig. II. 2.3.3 - 2 Section of Canal and location of Ditches

(3) Specification of Pumps

Case	High Rise Lock Canal		Sea Level Canal	
Type	Fixed Location Pump	Portable Pump	Fixed Location Pump	Portable Pump
Specification				
Bore	300 mm	200 mm	300 mm	200 mm
Discharge Vol.	12 m^3/min	4 m^3/min	12 m^3/min	4 m^3/min
Head	30 - 40m	20m	40 - 150m	20m
Output	95 kW	22 kW	95 kW - 350 kW	22 kW

The volume of water to be pumped includes leakage and spring water.

Number of portable pumps will be 3 sets per one set of fixed location pump.
Generators are used as power sources.

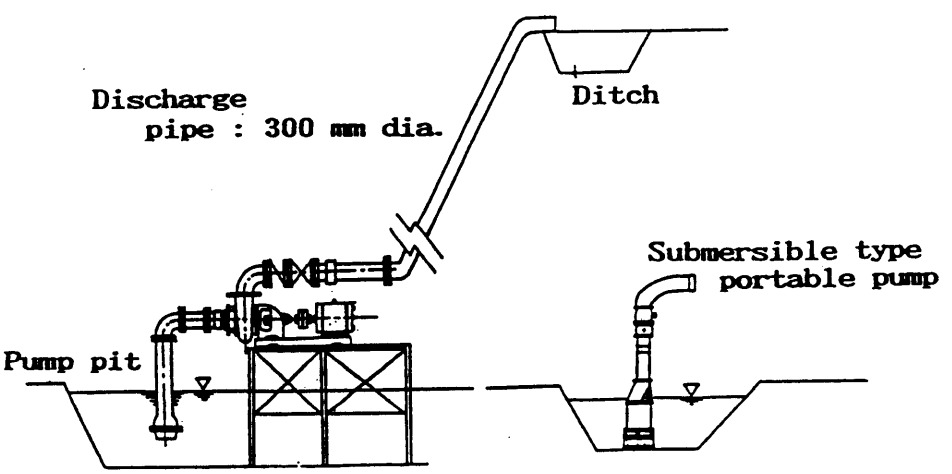


Fig. II. 2.3.3 - 3 Installation of Dewatering Pump

3.4 Placing Areas

(1) Construction and Type of Works

1) Dry Excavation

Placing Area → Clearing and Grubbing of Area → Banking / Dewatering → Surface Drainage

→ Revegetation of slope → ultimately used as a stock farm

2) Dredging

Placing Area → Dumping in Gatun Lake (Dumping to be to level - 4m below water level)

(2) Placing Areas for Dry Excavation

Materials from the dry excavation will be used for banking at designated placing areas. After completion of the banking, vegetation and drainage of surface water will be made.

1) Clearing and Grubbing

Prior to banking at the placing area, clearing and grubbing will be executed.

2) Along small rivers and streams which flow into the placing area, drainage pipes will be installed. Refer to Fig. II. 2.3.4 - 1.

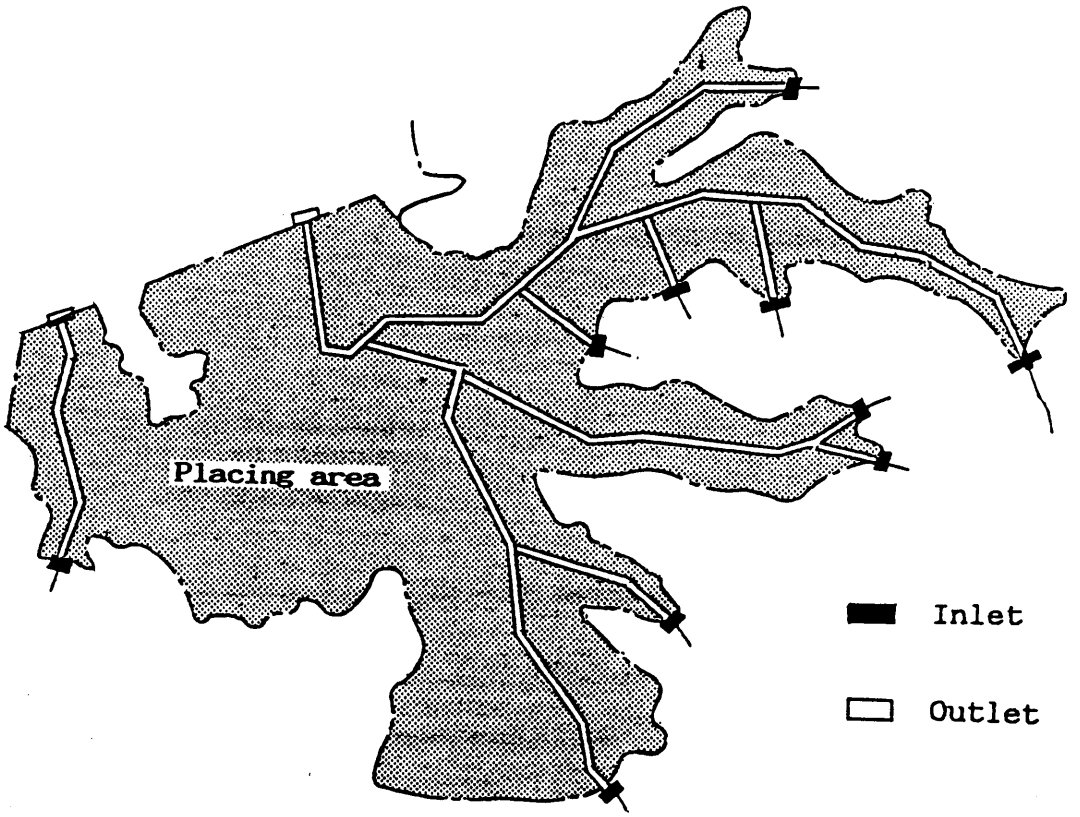


Fig. II. 2.3.4 - 1 General Layout for Drainage

3) Banking and Drainage

a. Banking will be carried out in such way that compaction is done by bulldozer layer by layer.

b. As for drainage during construction, temporary sedimentation ponds will be provided. Rainfall on the surface of the banking will be collected and led to the sedimentation pond. Clean water at the surface of sedimentation pond will be discharged to existing rivers. Refer to Fig. II. 2.3.4 - 2.

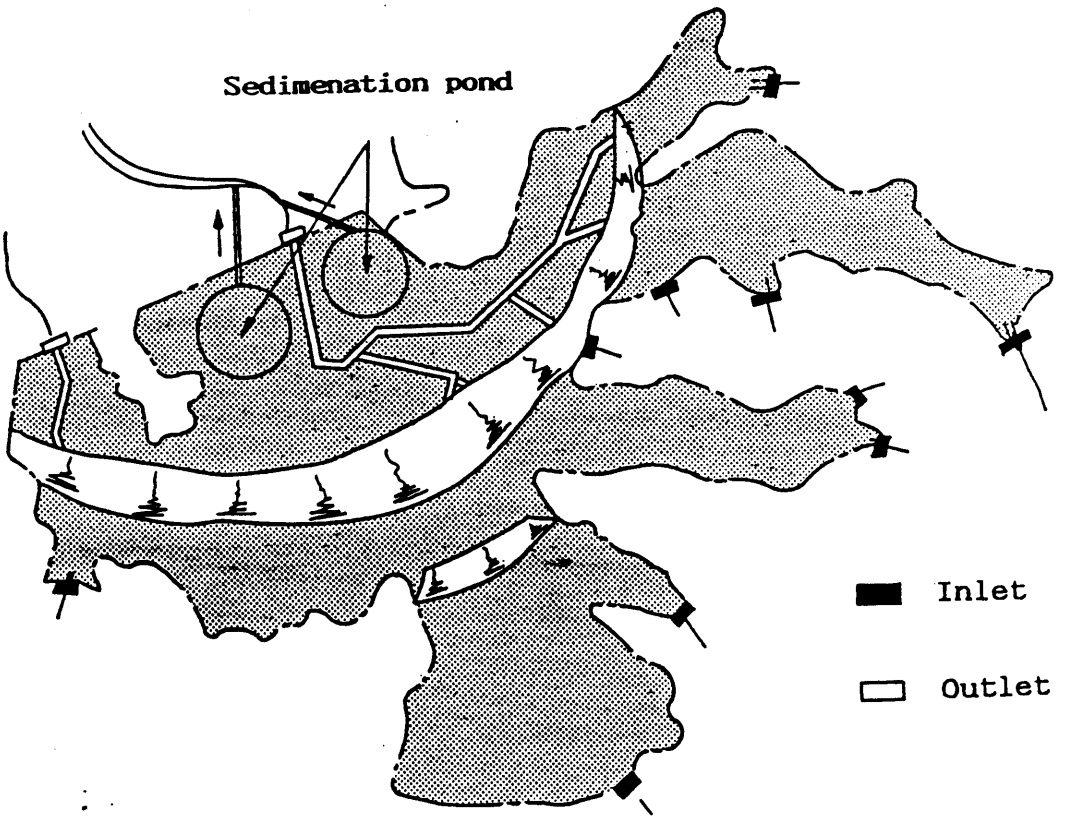


Fig. II. 2.3.4 - 2 General Layout for Sedimentation Pond

c. The surface of placing areas will be covered with top soil, and surface drainage using open ditches will be provided.

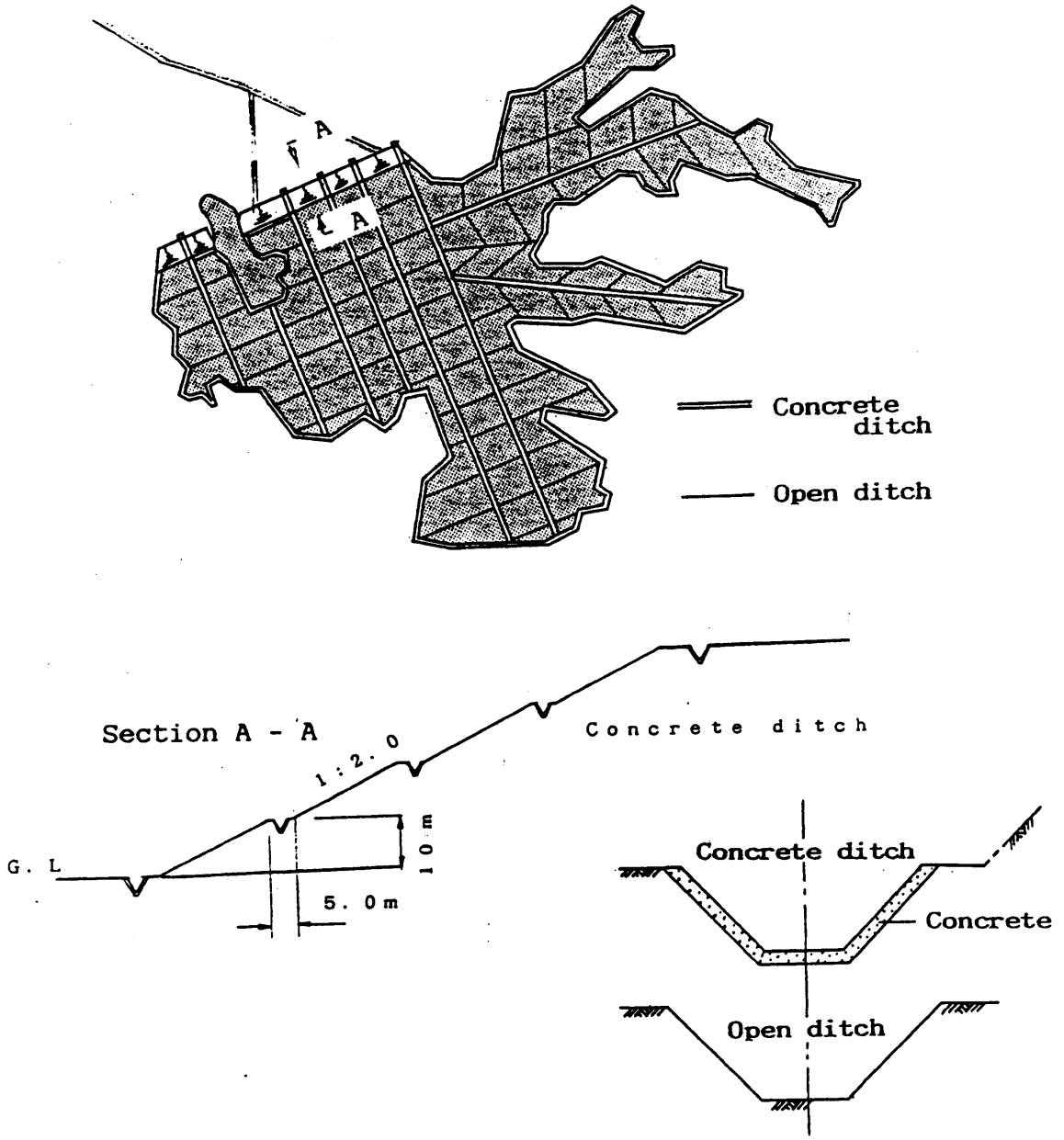


Fig. II. 2.3.4 - 3 General Plan and Section of Ditch

4) Vegetation

A mix incorporating seeds of vegetation will be sprayed on the surface of banking slope.

5) Utilization after Completion of Placing Area

Placing areas will be utilized mainly as cattle farms.

(3) Placing Area for Dredging

Excavated materials from the Atlantic and Gatun Lake will be disposed into water of about 5m or more depth so as not to interfere with the navigation of vessels.

3.5 Drilling and Blasting Guidelines

In order to find the most efficient/highly productive rock excavation method, these guidelines were prepared based on the experience and the data available. The most feasible method, of those methods available at present, is the Bench Cut using large scale drilling equipment. The guidelines describe the selection of the equipment, explosives, standard drilling and blasting calculations, estimated daily production and consumption of explosives.

(1) Drilling Equipment

Generally speaking, since drilling work is one of the major factors in achieving productivity in hard rock excavation, the selection of drilling equipment is considered the most important matter.

The performances of several types of drilling machine were studied to achieve the best possible productivity, and the most suitable equipment were selected as follows;

Preparation work: (Bench forming, Road work, Secondary blasting)

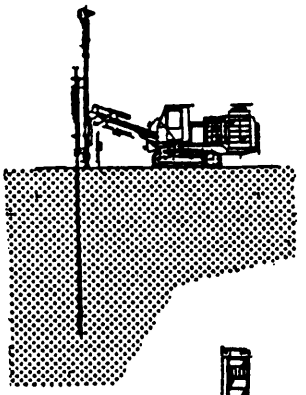
- Crawler drill of the 15 ton class equipped with a hydraulic rock drill for 75-100mm holes.

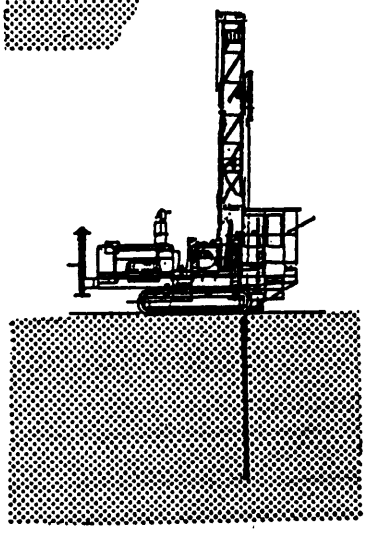
Bench cut: (Standard excavation, Quarrying work)

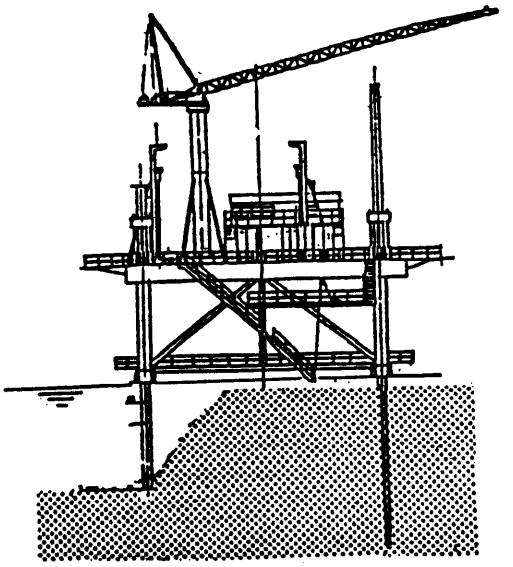
- Blast hole drill of the 32 ton class equipped with a down-the-hole drill for 225 mm holes.

Underwater excavation:

- Overburden drill installed on a self-elevated platform for 100 mm holes.







Left above: 15 ton Crawler Drill
Left below: 32 ton Blast Hole Drill
Above: Self-elevated Platform
equipped with Overburden Drill

Fig. II. 2.3.5 - 1

Illustration of Drilling Equipment

Table II. 2.3.5 - 1 Drilling Equipment

Equipment	Weight	Drill Hammer	Hole dia.	Drill Steel
	ton		mm	
Hydraulic Crawler Drill	15	Hydraulic Drill	76 - 127	3.7m x 6
Blast Hole Drill	32	Down Hole Drill	200 - 250	9.1m x 5
SEP with O.D. Machine	1,200	Overburden Drill	75 - 100	9.1m x 5

(2) Drilling for Standard Bench Cut

1. Bench Height: 20 m
2. Drill: 32 ton class Blast Hole Drill
3. Drill Hole Diameter: 225 mm
4. Drilling Cycle Time per Hole: 183 min.
   - Drilling: 22.5/0.131 = 172 "
   - Rod Extension: 1 min x 3 = 3 "
   - Extraction: 1 min x 3 = 3 "
   - Machine Shift: 5 min x 1 = 5 "
5. Average Drill Speed: 22.5m/183 min = 0.123 m/min
6. Average Work Cycle Time per day (2 shift x 10 hr): 1,200 min
   - Preparation: 30 min
   - Drilling: 840 min (7 hr x 2 shift)
   - Charging: 180 min
   - Rest: 120 min
   - Miscell: 30 min
7. Drilling Length per Day: 840 x 0.123 = 103.3 m
8. No. of Holes drilled per Day: 103.3/22.5 = 4.6 Holes
9. Max. Burden: 9.0 m, Spacing: 11.3 m, Height: 20.0 m
10. Blasting Vol. per Hole: 2,034 m^3
11. Blasting Vol. per 1 day: 9,356 m^3

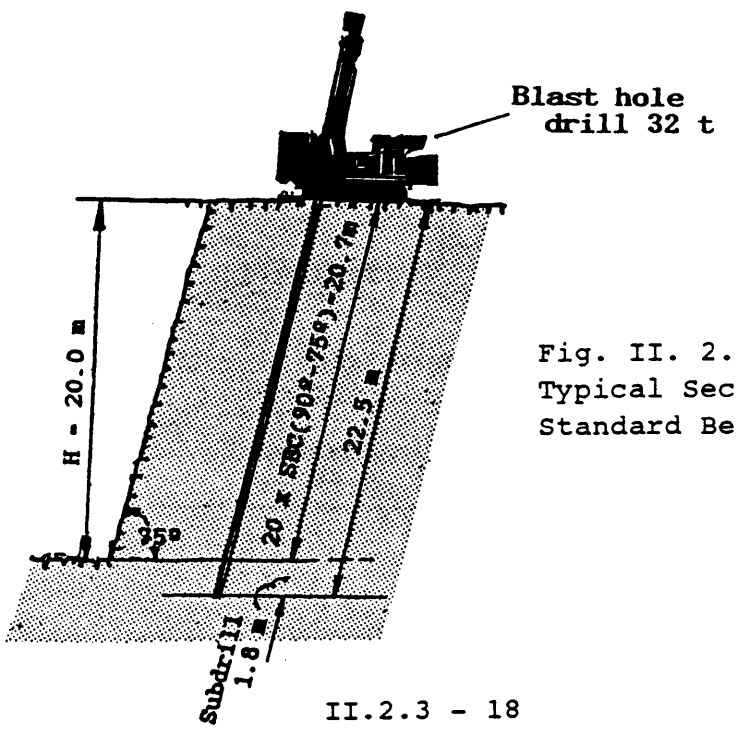


Fig. II. 2.3.5 - 2
Typical Section of
Standard Bench Cut

(3) Blasting Design

1) Explosives

The selection of explosives is one of the most important factors for estimating unit cost of the rock excavation. The table below shows several types of the explosives commonly used for the rock excavation work. AN/FO should be used for column charges and slurry explosives for bottom charges for dry excavation. Where AN/FO cannot be used, owing to wet conditions, slurry explosives and/or gelatine dynamite should be mainly used, as used for underwater excavation.

Table II. 2.3.5 - 2 Characteristics of Explosives

Type of Explosives	Specific Gravity	Strength per Unit of Weight	Detonating Speed	Water Resistance
	Kg/l		mm/sec	
Gelatine	1.40	1.00	6,000	Very good
Slurry	1.35	0.85	5,000	Good
AN/FO	0.85	0.90	2,500	No good

Table II. 2.3.5 - 3 Application of Explosives

Type of Work	Column Charge	Bottom Charge
Dry Excavation	AN/FO	Slurry
Dredging	Slurry/Gelatine	Gelatine

2) Calculation of the Charge for Bench Cut

Calculation of the charge is aimed to give the minimum charge required for loosening the rock. In practical application we must take into account the influence of faulty drilling, throw and swell, fragmentation and the effect on surrounding rock. The calculations given here are based on the formula as proposed by Dr. Langefors. (cf. "The modern technique of rock blasting", U. Langefors and "Swedish blasting technique" by R. Gustafsson.)

The blasting formula is:

$$V = d/33 \sqrt{p.s/\bar{c}.f\ (E/V)}$$

where; V: Max. burden : m
E: Hole spacing: m
d: Drill hole diameter: mm
p: Loading density of explosives: Kg/l
(Gelatine: 1.40, Slurry: 1.35, AN/FO: 0.85)
s: Coeff of strength per unit of weight of explosives
(Gelatine: 1.00, Slurry: 0.85, AN/FO: 0.90)
c: Rock factor: Kg/m^3 (usually c=0.40)

$\bar{c} = c + 0.05 = 0.45$ (cf. Langefors)

f:Degree of fixation: 0.9 at drilling angle of 75°

Inclination	Fixation factor
Vertical	1.0
3 : 1	0.9
2 : 1	0.85
Free	0.75

3) Blasting for Standard Bench Cut

a. Burden and Spacing

Theoretical burden for bottom charge (slurry):
V1 = 225/33/1.35x0.85/0.45x0.9x1.25 = 10.3 m
E = 12.9 m V.E = 132.9 m^2
Theoretical burden for column charge (AN/FO):
V2 = 225/33/0.85x0.90/0.45x0.9x1.25 = 8.4 m
E = 10.5 m V.E = 88.2 m^2
Ratio of bottom charge to column charge: 3:7
Average burden x spacing:
(132.9 x 0.3) + (88.2 x 0.7) = 101.6 m^2
Maximum burden: 9.0 M (40 x hole dia.)
Hole spacing: 11.3 m

b. Charge Length

Stemming length:	24 x Hole dia. = 5.4 m
Subdrill length:	1/3 x Stem L. = 1.8 m
Hole depth:	22.5 m
Length of charge:	Hole depth - stem L = 17.1 m

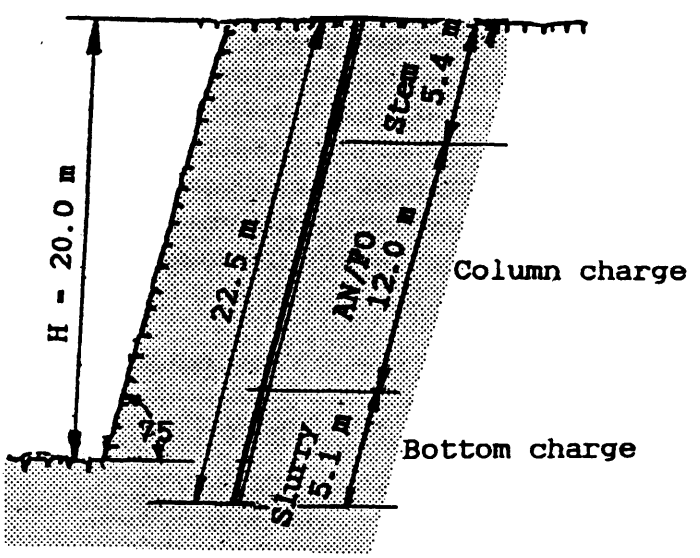


Fig. II. 2.3.5 - 3 Typical Section of Bench Cut Charge

c. Specific Charge

Quantity of explosives charged per meter:
Bottom(slurry): $1.35 \times 10 \times 2.25^2 \times 0.785 = 53.6$ Kg/m
Column(AN/FO): $0.85 \times 10 \times 2.25^2 \times 0.785 = 33.8$ Kg/m

Quantity of explosives per hole:
Bottom(slurry): $17.1 \times 0.3 \times 53.6 = 275.0$ Kg
Column(AN/FO): $17.1 \times 0.7 \times 33.8 = 404.6$ Kg

Specific charge:
$(275.0 + 404.6) \div 2{,}034 = 0.334$ Kg/m^3

4) Secondary Blasting

Secondary blasting has to be carried out from time to time to break up boulders which are too large for mucking or crushing, and this makes up a considerable part of the blasting costs. The definition of a boulder varies from one case to another and depends on the mucking/ loading equipment used and bucket volume as well as the dimension of the crushing plant input opening if the rock is to be crushed. The cost of secondary blasting will add 20-40% to that of primary blasting based on practical experience and data available. There are two different methods of secondary blasting, i.e. Plaster shots and Charge in a drilled hole. The application of the method depends mainly on the condition of the boulders produced by primary blasting. (cf. Table II. 2.3.5 - 4

Table II. 2.3.5 - 4 Method of Secondary Blasting

Description	Plaster shots	Charge in drilled hole
Application	Thin boulders highly cracked	Thick, round, less cracked
Specific charge	1.0 Kg/m^3	0.06 - 0.1 Kg/m^3
Explosives	High detonating speed explosives	Any type of explosives

5) Underwater Blasting

The specific charges for underwater blasting are generally increased from double to quadruple compared with normal bench cut blasting. According to "Swedish Blasting Technique" page: 261, the basic calculation is described as follows:

1. Normal bench blasting needs a specific charge of about 0.45kg/m^3 to ensure good fragmentation. Since misfire can be expected in one or two drill holes, the charge is double in the case of underwater blasting to 0.90kg/m^3. This implies that if a drill hole does not detonate but the surrounding holes do, the specific charge is still 0.45kg/m^3. Vertical holes imply a poorer break-out possibility in the bottom section. The blasting bottom shows a tendency to rise, particularly in the case of long rounds. In the case of vertical drill holes, the specific charge should be increased by about 10% to 1.0kg/m^3.

2. The water pressure above is compensated for by increasing the specific charge by 0.01 x water depth.

3. If the rock is covered with clay, the specific charge can be increased by 0.02 x thickness of clay. For rock sections, the normal ratio is 0.03 x bench height.

4. Because of the specific charge needed, hole spacing is calculated with respect to possible charge concentrations per meter drill hole. When a mechanical charger is being used, charge concentrations can be calculated as in the case of dry land blasting.

6) Fragmentation

Fragmentation concerns the size of the stones obtained after the rock has been blasted. The following factors influence fragmentation.

- Number of boulders per unit of volume
- Crushing capacity
- Loading/mucking capacity

The composition of the rock exerts a great effect on fragmentation since its strength characteristics vary.

The blasting technique factor with the greatest influence on fragmentation are:

- The specific charge
- The specific drilling

If the specific charge is increased and the same hole spacing maintained, the rock is broken up more, and it is often decided to increase the specific charge when the intention is to improve fragmentation.

Specific drilling is of great importance for fragmentation. The same specific charge increases fragmentation with burden. In the case of small burden, the charge is better distributed in the rock and possibility of boulder formation is limited by the hole spacing.

Generally, large diameter drill holes with concentrated charges provide a larger proportion of crushed material but also fractions with larger sizes. Fragmentation is smaller when small diameter holes are used for blasting.

In order to obtain better fragmentation and/or large size fragmentation, special methods are employed.

Basically, the charge in the relationship between spacing and burden from the normal E/V = 1.25 to considerably larger values will improve the fragmentation. With constant specific drilling and specific charge, the fragmentation increase when the ratio E/V is increased. The improvement will continue all the way up to values of E/V = 8.0.

In order to obtain good fragmentation, such as for quarrying concrete aggregates, the values of E/V should be increased gradually to the value mentioned above, and employment of millisecond firing would be recommended. (Fig.II. 2.3.5 - 4)

Sometimes large size fragmentation is desired, such as for armor stones for breakwater construction. In this case the values of E/V should be given less than 1, and instantaneous as well as simultaneous firing of one row would be recommended.

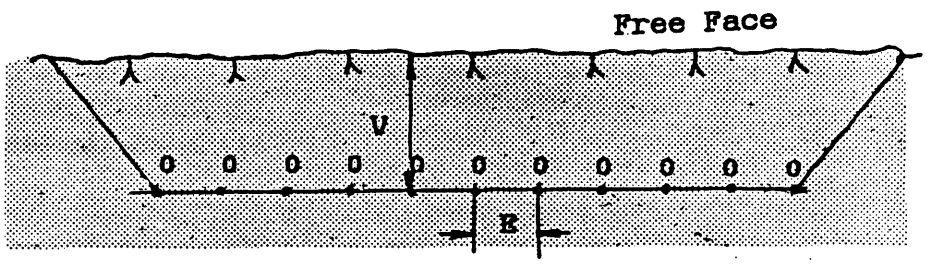


Fig. II. 2.3.5 - 4 Wide Space Hole Blasting Patterns
(Figures show detonating delays in millisecond)

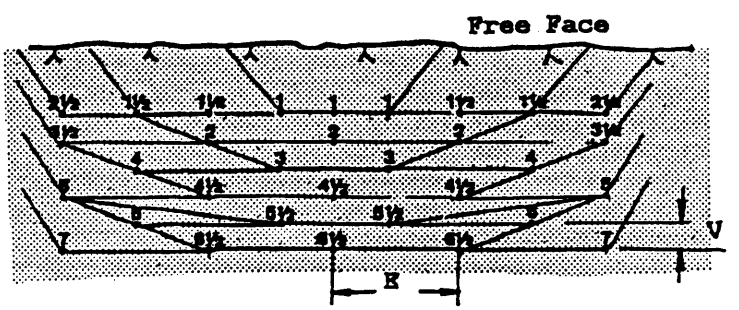


Fig. II. 2.3.5 - 5 Large Size Fragmentation Patterns

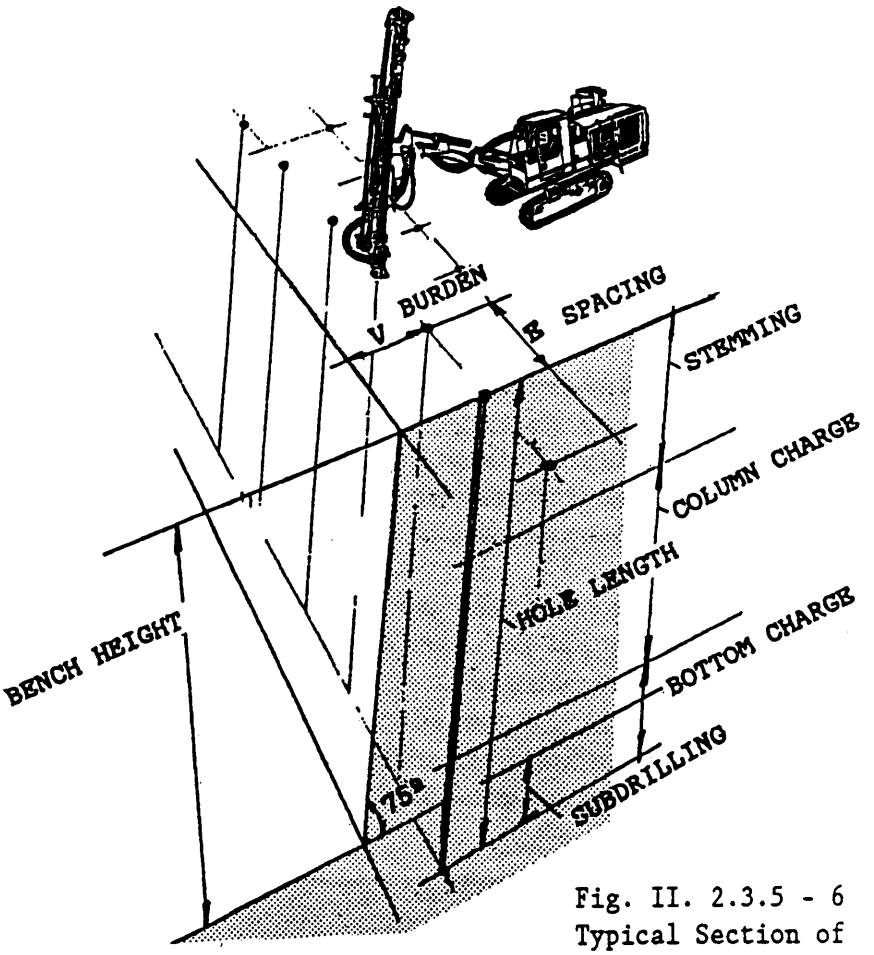


Fig. II. 2.3.5 - 6
Typical Section of
Bench Cut Burden and Spacing

Table II. 2.3.5 - 5 Burden and Spacing Ratio

Burden and Spacing	V	E	E/V
	m	m	
Standard Bench Cut for (Canal Excavation Quarrying for Rockfill)	9.0	11.3	1.25
Wide Space Hole Blasting (Concrete Aggregate)	3.6	18.8	8.00
Large Size Fragmentation (Armor stone)	15.0	6.8	0.45
Underwater Blasting			

7) Ground Vibration from Blasting

In order to plot vibrations from blasts which consist of widely varying quantities of explosives at many distances, it is useful to normalize or "scale" the data. This is done by dividing the distance by some function of the weight of explosives detonating at any instant of time. For deep underground point charges, and for the calculation of water or air pressures, the best choice has been to normalize distance by the cube root of the charge weight per delay. However, because of the greater spatial distribution of most land-based down-hole blasting operations, it has proved more useful to use square root scaling. The true distance divided by the square (or cube) root of the charge weight per delay is known as the normalized distances or scaled distance. For ground vibrations, it is customary to prepare a plot of the vibration data on a log-log graph of peak particle velocity versus square-root scaled distance.

It is a simple matter to express an equation for a straight line on a log-log graph, regardless of the source of the data or what it represents. Any value on the vertical axis can be expressed as a function of the value of the horizontal axis raised to a power which represents the slope of the line, and that product multiplied by the y-axis intercept at unity (see formula below). It should be recognized that the resulting formula is a data-processing formula from analytical geometry. It is not a seismic wave equation.

For blasting operations, it is useful to modify this data-processing formula to include certain additional factors related to bast design, such as confinement, spatial and timing distributions of the detonation process, and the like. These factors are used to modify the trend line equations. If blasting is done repeatedly with the same design at the same site, it suffices to consider all of these parameters to fall under the single factor "H", as a general "site response factor".

The general equation including "K factors", takes the form:

$$PPV = H\ (D/W^{1/2})^{-B}\ (k_1,\ k_2,\ k_3\ .\ .\ .\ )$$

where PPV = the peak particle velocity of the vibration, expressed in inches per second (ips)

D = the distance seismic path), in feet

W = the maximum charge weight per delay, in pounds

$(D/W^{1/2})$ = the scaled distance

H = the intercept of the attenuation slope at unity scaled distance

B = the slope of the line

K factors = as described above and on the graph of the prediction curves by *Oriard (see graph).

* Lewis L. Oriard, Inc., Huntington Beach Cal. U.S.A.

Table II.2.3.5 - 6

TYPICAL RANGE OF BLASTING VIBRATIONS

FOR VARIOUS CHARGE WEIGHTS AND DISTANCES

LBS/DELAY (lbs)	DISTANCE (ft)	AVERAGE RESPONSE (ips)	TYPICAL UPPER RANGE RESPONSE (ips)
100	500	0.306	0.463
100	1000	0.101	0.153
100	2000	0.033	0.050
100	3000	0.017	0.026
100	4000	0.011	0.017
100	5000	0.008	0.012
200	500	0.533	0.806
200	1000	0.176	0.266
200	2000	0.058	0.088
200	3000	0.030	0.046
200	4000	0.019	0.029
200	5000	0.013	0.020
500	500	1.109	1.677
500	1000	0.366	0.553
500	2000	0.121	0.183
500	3000	0.063	0.095
500	4000	0.040	0.060
500	5000	0.028	0.042
1000	500	1.931	2.921
1000	1000	0.637	0.963
1000	2000	0.210	0.318
1000	3000	0.110	0.166
1000	4000	0.069	0.105
1000	5000	0.049	0.073
5000	500	6.998	10.584
5000	1000	2.308	3.491
5000	2000	0.761	1.152
5000	3000	0.398	0.602
5000	4000	0.251	0.380
5000	5000	0.176	0.266

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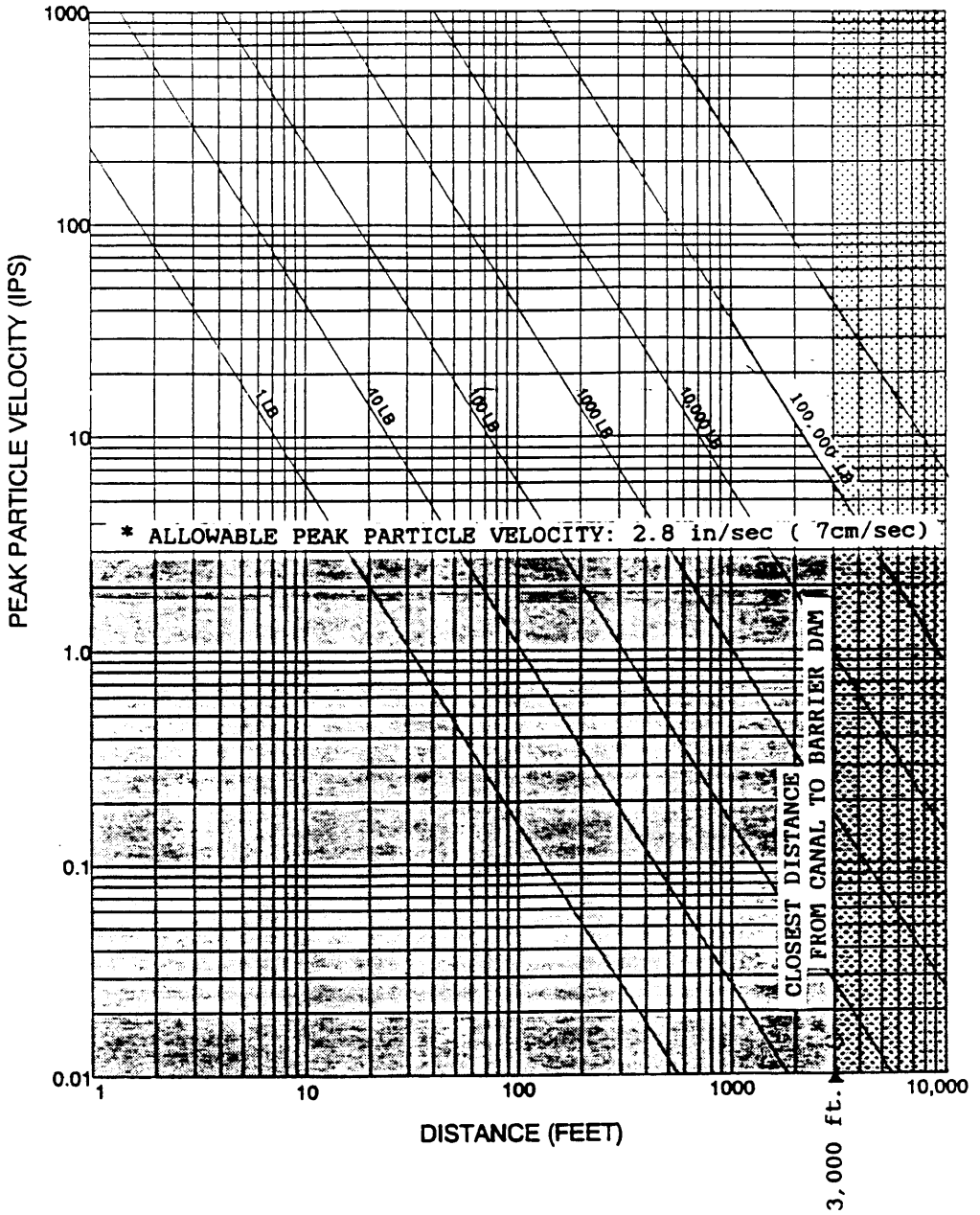


Fig. II.2.3.5-7

PEAK PARTICLE VELOCITY VERSUS DISTANCE
FOR VARIOUS CHARGE WEIGHTS PER DELAY

* SWEDISH BLASTING TECHINIQUE, P. 209,
R. GUSTAFSSON

LEWIS L. ORIARD, INC.

3.6 Aggregate Production

Generally, the best sources of concrete aggregate are considered to be from river sand, river gravel, sea sand, sea gravel or quarry. For this construction plan, the selection of the concrete aggregate was studied from geological data available as well as the report of site investigation, and concluded that all the aggregates are to be taken from prospective quarries located near the construction site.

(1) Required Quantity of Concrete Aggregate

Quantity of aggregate for $1m^3$ of concrete planned will be as follows:

Coarse Aggregate	1.094 ton/m^3
Fine Aggregate	0.730 ton/m^3
Total	1.824 ton/m^3

(2) Sources of Aggregates

Prospective quarries for each construction site are considered as follows:

Table II. 2.3.6 - 1 Sources of aggregates

Case	Atlantic Side Lock, tidal Gate	Pacific Side Lock Tidal Gate
High Rise	Quarry H-1 (Granite)	Excavated Material from Canal (Basalt)
Sea Level Canal	Quarry H-1 (Granite)	Quarry S-2 (Basalt)

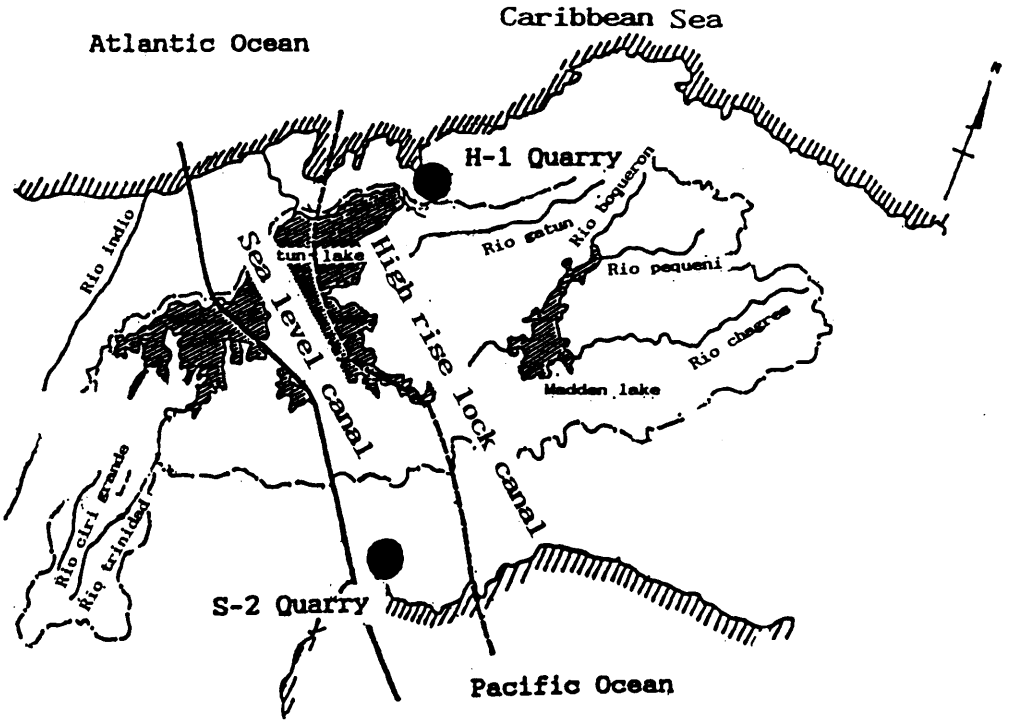


Fig. II. 2.3.6 - 1 Location of Quarries

(3) Crushing Plant

A crushing plant will be installed adjacent to the Quarry. The produced aggregates are to be stored and transported to each concrete mixing plant as scheduled.
The flow chart for the crushing plant is as shown in Fig. II. 2.3.6 - 2.
The crushing plant capacity required will be approximately 20% more than that of aggregate demands.

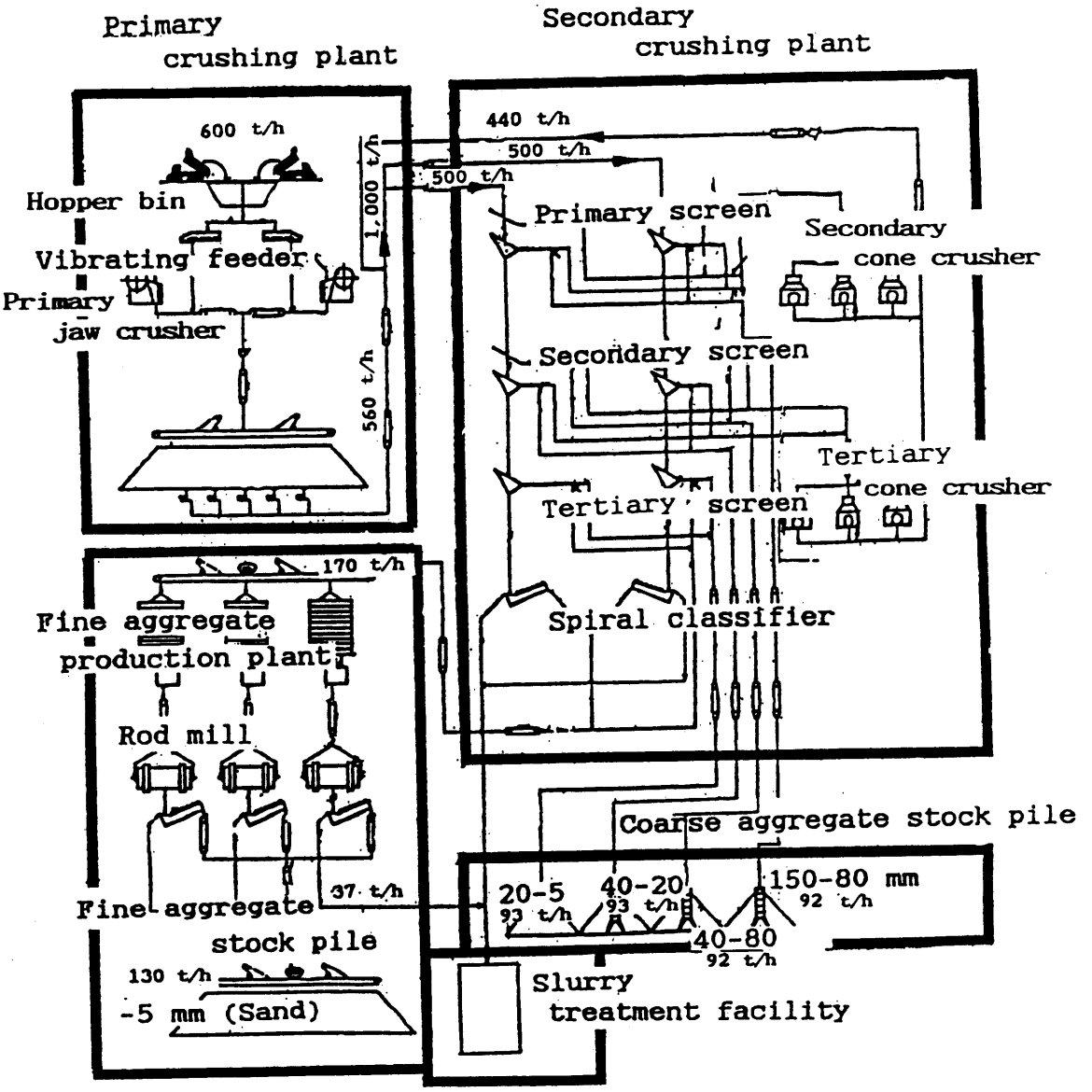


Fig. II.2.3.6 - 2 Flow Chart of 600 t/h Crushing Plant

3.7 Concrete Production

(1) Installation of Concrete Production Plants

Concretes produced by concrete production plants installed near the structure are delivered to the sites.

(2) Concrete Materials

Proportion of materials used for concrete (per/m^3)

Cement	0.295 ton/m^3
Coarse Aggregate	1.094 ton/m^3
Fine Aggregate	0.730 ton/m^3
AE Agent	0.800 l/m^3

(3) Supply of Aggregate and Cement

Aggregate: Stock yard at aggregate production plant ——> Concrete Production Plant

Cement : Cement Silo located at Temporary wharf ——> Concrete Production Plant

(4) Requirements of Concrete Production Plant

1) Capacity of each concrete production plant follows the scheduled concrete placing plan.

2) Each Concrete Production Plant has facility for cement and aggregate storages.

3) A water supply facility is required at each concrete production plant.

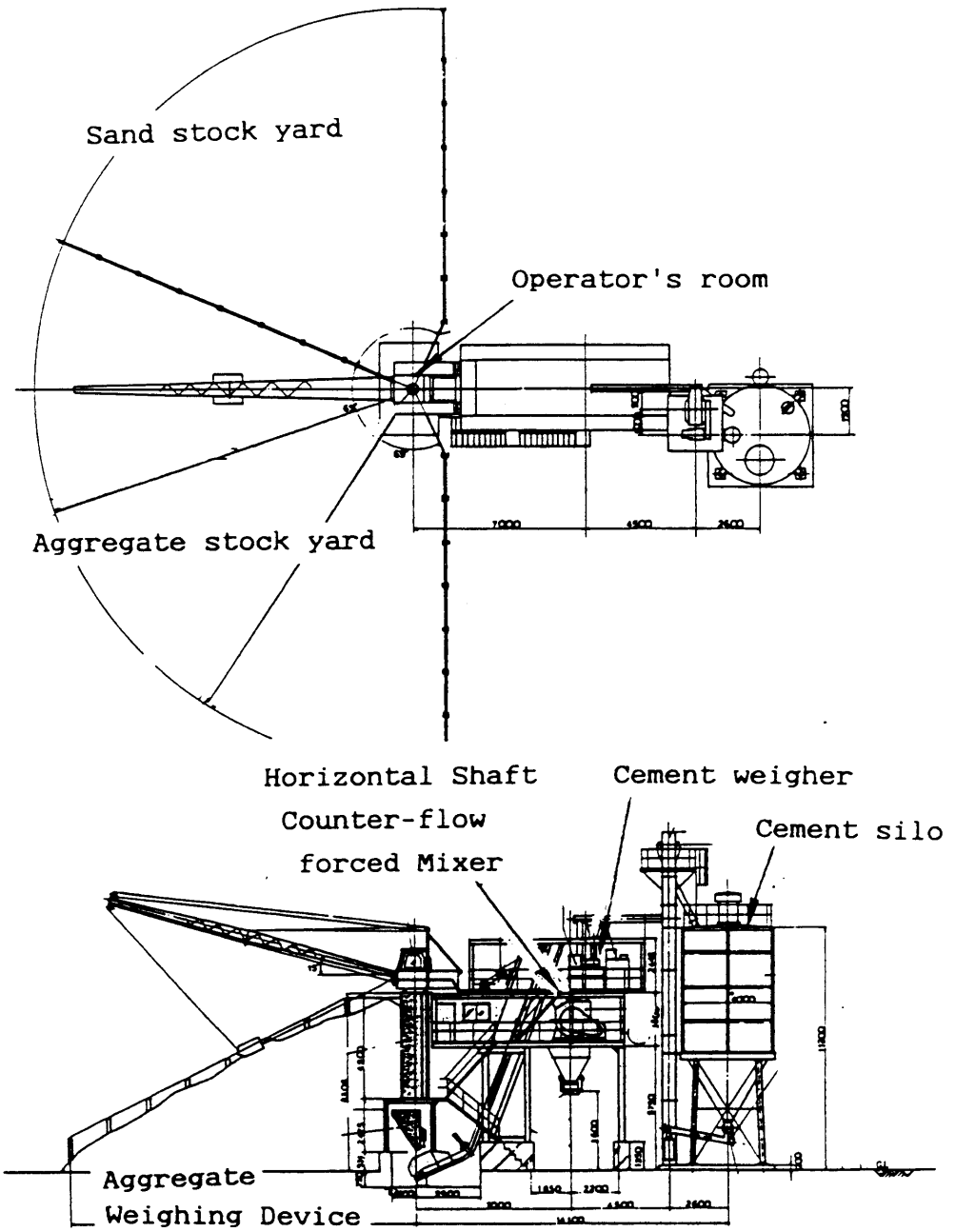


Fig. II. 2.3.7-1 Layout of Concrete Production Plant

II.2 CHAPTER 4 - FABRICATION OF GATE LEAVES

4.1 Introduction

This chapter covers gate leaves and related facilities not only for both the high rise and the tidal locks but also for the tidal regulating structures. Also it partially refers to small gates and valves for the new and the barrier dams. Major components included in the related facilities are as follows.

Table II.2.4-1 Major Facilities Related with Gate Leaves

Name	High Rise Lock	Tidal Lock	Tidal Regulating Structure
Gate Frames	yes	yes	yes
Gate Hoists	yes	yes	yes
Locomotives	yes	yes	
Electric Power and Control Equipments	yes	yes	yes
Emergency Power Plant	yes	yes	yes

4.2 Material Source and Place of Fabrication

It is anticipated that there is a possibility of economically procuring the gate leaves and related facilities or their materials from developing areas such as China, India, Brazil etc. where industry has been developed to a certain level, yet labor cost is still comparatively low. This point shall be much considered in realizing economical design of gate leaves. Table II.2.4-2 defines classification zones into which materials and fabrication methods of gate leaves are classified according to technical capability and cost competitiveness of the de veloping countries. A fabrication method or material which belongs the zone I in the table shall be employed

in the gate leaf design as much as possible whereas those which belong to the zone II or III shall be employed only when they are economically advantageous or a canal alternative to be studied can not be feasible without them.

The gates for dams (=the new dams or the barrier dams in the high rise lock canal or the sea level canal alternative) are all classified into the Zone I with respect to either material or fabrication method whereas lock gates and tidal gates can not be classified quantitatively into any zone at this stage of study. Table II.2.4-3 shows classifications of cost components in cost estimates. No.1 on the table was a classification applied to estimation of the lock gates and the tidal gates (=gate leaves for the locks of the both canal alternatives and for the tidal gate structures) and includes three cost components. The No.2 is a minimum requirement for cost classification which shall be applied for full study of economical gate leaf design including material source and place of fabrication. The Zone I thru Zone III in this calssification correspond to those defined on previous table and the supervision means cost for supervising fabrication or installation carried out by the firms in the under developed area. Break down of the costs in the Calssification No.1 into the No.2 will be possible only after detail design on the gate leaves is made.

Table II.2.4-2 Classification Zone for Cost Competitiveness and Technical Capability

		Technically Capable ?	
		Yes	No
Costly Competitive ?	Yes	Zone Ⅰ	Zone Ⅲ
	No	Zone Ⅱ	

Table II.2.4-3 Classification of Cost Component

Classification Number	1	2	
To be Applied for	Lock and Tidal Gates	Study on Design Optimization Including Supply Origins	
Cost Components	Fabrication	Shop Design	
		Material & Parts	Zone Ⅰ
			Zone Ⅱ
			Zone Ⅲ
		Fabrication	Direct Cost
			Supervision
	Transportation	Transportation	
	Installation	Direct Cost	
		Supervision	

4.3 Constructability

Constructability of all major facilities included in the preliminary designs made has been reviewed and it has been concluded that possibility of all their shop fabrications is positive at least from the feasibility study point of view. Nevertheless the same effort has to be made for the more detail portions of designs at detail design stage.

4.4 Fabrication Procedure

(1) Gate leaves

In general, usual procedures and technics adopted in ship building industry or bridge fabrication industry will be applicable to the fabrication of all gate leaves. Specific points having considered in their preliminary designs are as follows.

1) The gate leaf blocks listed on Table II.2.4-4 are fabricated to a block including majority of accessories either at open yards or dry docks of shop and shipped to the site as a whole.

Table II.2.4-4 Approx. Maximum Block Weight in Ton of Gate Leaves When Launched at Shop

Gate Name	High Rise Lock	Tidal Lock	Tidal Reg. Structure	Barrier Dam
Operating Gate	2100	800	7300	
Emergency Gate	3000	1800	7300	
Maintenance Gate	6700			
Culvert Maint. G.	30	25		
Spillway Slide G.				17

2) The gate leaf blocks for the Tidal Regulating Structures in Table II.2.4.3-4 can, as an option, be fabricated into a several vertically

sliced blocks, then bolted together to a block having weight shown on the table.

3) An Operating Gates and An Emergency Gates for Tidal Lock and Tidal Regulating Structure are consist of a pair of blocks which are supposed to be joined each other at the site. Their good matings should be confirmed at their shop fabrication stages.

4) The gates listed on Table II.2.4-5 are fabricated to blocks and preliminarily assembled to a body including majority of accessories, then dis-assembled for their shipments.

Table II.2.4-5 Gate Leaves to be Shipped after Dis-assembling at Shop

Gate Name	High Rise Lock	Tidal Lock	New Dam	Barrier Dam
Culvert Gate	yes	yes		
Outlet Pipes			yes	
Spillway Rad. G.				yes

5) Intensive welding tests will be necessary to establish welding procedures and to provide enough number of qualified welders in order to meet needs for fabrication of steel structures which will be made from massive thichnessed high tensile strength steel up to 100 Kg/mm^2 in ultimate strength.

6) Special attention shall be paid to keep the major structures being welded at the acceptable position which will be specified in the welding procedures being established. This requirement will have a more important meaning at the later stages of shop fabrications because the weight of a block to be welded may become more than a thousand tons.

(2) Gate Frames and Gate Hoists
All gate frames and hoists are fabricated to blocks and preliminarily assembled to a body including majority of accessories, then dis-assembled for their shipments.

(3) Locomotives, Electric Power Equipments, Emergency Power Plant
Locomotives, Electric Power and Control Equipments and Emergency Power Plant shall be fabricated at shop to as big block as acceptable for their site handling.

4.5 Transportation Procedures

Followings are classifications for ocean transportation procedures. ( ) shows abbreviation of each procedure.

(1) Being towed by tugs;(Towed)

(2) Being towed on a sink barge; (On barge)

(3) On board as a whole with small protection work; (On board)

(4) On board after packed; (Packed)

Application of the procedures classified above mainly depends upon dimension, shape, nature of material used for the product to be transported. Table II.2.4-6 shows assumptions made for cost estimation of this study.

Table II.2.4-6 Transportation Precedure

Name of Product	Trans. Procedure
(1) Ope. Gate Leaf for Tidal Reg. Structure	Towed
(2) Eme. Gate Leaf for Tidal Reg. Structure	Towed
(3) Ope. Gate Leaf for Locks	On barge
(4) Eme, Gate Leaf for Locks	On barge
(5) Maintenace Gate Leaf for H. R. Lock	Towed
(6) Culvert Maintenance Gate & Spillway Slide Gate	On board
(7) Locomotive	On board
(8) All other Fabricated Material	Packed

Table II.2.4-7 gives typical dimensional examples of products to be shipped in case of the 200,000DWT high rise lock canal alternative. The product number in the table corresponds to the numbers in Table II.2.4-6.

Table II.2.4-7 Dimensions of Products when Shipped (200,000DWT H.R.L.C. Option)

Product Name	Length (m)	Width (m)	Height (m)	Weight (m. ton)	T. Procedure
(3) Ope. Gate Leaf	37	35	4.8	2,100	On barge
(4) Eme. Gate Leaf	60	25	5.5	3,000	On barge
(5) Maint. Gate Leaf	60	9.2	30	6,700	Towed
(6) Culv. M. G. Leaf	9	2.5	1.0	30	On board
(7) Locomotive	15	3.3	4.5	80	On board

4.6 Possibility of Fabricating Gate Leaves in Panama and Immediate Region

Possibility of gate leaf fabrications at shops in Panama or immediate regions was studied. It's result is shown in Table II.2.4-8 in three steps grading system with respect to technical ability and cost competitiveness. The immediate region includes Colombia, Ecuador, Peru, Costa Rica, Nicaragua, El Salvador, Guatemala and Honduras.

Table II.2.4-8 Possibility of Local Fabrication

Gate Name	Technical Ability	Competitiveness
Tidal Gate	x	
Lock		
1) Operating Gate	x	
2) Culvert Gate	x	
3) Emergency Gate	x	
4) Maintenance Gate	x	
5) Culvert Maintenance Gate	▮	?
Dam		
1) Crest Gate*	o	?
2) Outlet Valve**	▮	?
3) Outlet Pipe*	o	?

Note 1. The local fabrication means shop fabrication of gate leaves in Panama or immediate regions.
2. o; positive, ▮; needs more study, x; negative, ?; needs study on an inquiry base.
3. * shows for the new dam and ** shows for the barrier dams.

4.7 Work Schedule

Gate leaves which will be fabricated at shop in one piece may be so heavy that handling to move to storage areas at shop yards would be almost impossible and also their long storage times either at the shop side or at site are not recommendable. Therefore their fabrication schedules at shop should exactly correlate to the total work schedule at site. This synchronization between the work schedules at shop and at site will be really possible only when detailed schedules for preceding events such as design, material procurements at shop and water filling to the canal at installation site are duly established.

PART II SECTION 3 COST ESTIMATES

II.3 CHAPTER 1 - ESTIMATE BASES AND ASSUMPTIONS

1.1 Type of Contract

The selection of contractors and the type of contract will be based on international competitive tender in accordance with the FIDIC, 4th Edition, 1987.

1.2 Estimate Date and Currency

All estimates are based on the 1990 index and in US dollars. Cost increases during the construction period, and in the case of delays in commencement, are not included.

1.3 Payment

It is assumed that progressive payments will be made during the construction work, such that no payment/receipt of interest is due to the contractors, and no allowance is made for retention.

1.4 Taxes and Custom Duties

Taxes and duties are not included in the cost estimate for Phase 2.

1.5 Social Benefit and Overtime Premium

Overtime premiums and allowances for social benefits, amounting to 67%, are added to the man-day/hour cost in Panama as of 1990, according to the Panamanian Labor Regulations.

1.6 Gate Leaves and Related Items

Way of cost estimation for gate leaves and related items is either one of or combination of following methods.

(1) Aggregation of major cost components.
(2) Estimation based upon established unit prices.
(3) Quotation from commercial suppliers.
(4) Adjustment of the cost in the Deep Draft Lock Plan based on another cost components.

Cost estimation of gates for dams is based on unit prices ($/ton) which have been established according to records of resent international biddings for similar kinds of gates. Other gate leaves and related facilities as explained at II.2-Chapter 4.1 are estimated based on the assumption that they are fabricated in Japan. Detail discussion is given for back ground of these assumptions at II.2-Chapter 4.2.

1.7 General Conditions

The general conditions for the Alternative Panama Canal Project is based on the assumption that the work will be divided into 10-20 work sections (contracts). The amount of one section will be more or less US$500 million. Considering the scale of the project, items in the general conditions for the project are calculated by adding the costs of required items.

Items of the general conditions are considered to be as follows:

(1) Field Office Expenses

- Salaries/Wages of Managers, Engineers, Technicians, Clerks, Watchmen, Safety Personnel and others.
- Office Facilities and Engineering Supplies.
- Utilities* (Water, Power, etc.) *consumption
- Communications (Telephone, etc.)

- Traveling Expenses and Labor Transportation
- Welfare Expenses
- Reports and Photographic Records
- Legal Expenses
- Insurance (Construction, Third Party and Marine and Construction Equipment)
- Premium for bonds (Performance and Advance Payment)
- Vehicles

(2) Home Office Expenses

- Home Office Overheads
- Risks and Profits
- Subcontract, Procurement Arrangements

1.8 Supporting Construction

In order to carry out the construction work, support works and facilities will be required during the construction period. The cost of the support work and facilities is calculated by summing up the required items. Cost items to be included for supporting construction work are as follows:

(1) Construction Village and Camps

- Housing (family quarters, bachelor quarters)
- Public services (mess, school, police, fire, post office, etc.)
- Warehouses
- Churches
- Recreation facilities
- Medical office and hospital
- Supermarket
- Utilities (water supply, sewerage system, power plant, telephone, garbage collection and incineration, etc.)

(2) Main Haul Road

(3) Access Road Connecting Quarry Sites

(4) Logistic Base Facilities

- Temporary wharves
- Cement storage
- Fuel, lubricant storage
- Aggregate stockyards
- General cargo warehouses

1.9 Indirect Costs

Indirect costs to be borne by the client are categorized as follows:

(1) Administrative Costs

Client's costs for salaries, travel and expenses for engineering and construction activities.

(2) Headquarters Costs

Engineering, design and head quarters support, including salaries, travel and other expenses, of all home office assigned personnel.

(3) Construction Supervision

Salaries, travel and other expenses for construction management activities.

(4) Land Acquisition and Land Rights

Costs are to be calculated in collaboration with local consultants.

(5) Compensation and Relocation of the Local Population

Include the development of new infrastructure to service the relocated communities as costs for relocation of the displaced local population.

Recommendations of local consultants are to be applied in cost calculation.

(6) Contingency

A physical contingency allowing for uncertainties in scope, quantities and prices.

II.3 Chapter 2. Unit Price

2.1 Materials

Table II. 3.2.1 - 1

As of 1990

Description	Specification	Unit	CIF Panama ($/Unit)	Total On Site ($)	Import From	Remarks
Gasoline		L		0.50	Local	
Fuel	For Diesel Engine	L	0.17	0.19	US	
Heavy Oil	Fuel for M. Craft	L	0.09	0.11	US	
Lubricant Oil	For Engine	L	1.54	1.57	US	
Lubricant Oil	For Gear Mission	L	1.54	1.57	US	
Lubricant Oil	For Open Gear	L	-	-	US	
Lubricant Oil	For Marine Craft	L	0.15	1.57	US	
Hydraulic Oil		L	1.10	1.13	US	
Grease		KG	13.45	13.48	US	
Explosives						
AN-FO	Ammonium Nitrate	KG	1.23	1.35	US	
Dynamite		KG	4.58	4.63	US	
Slurry Explosive		KG	4.58	4.63	US	
Detonator		NO	0.97	0.98	US	
Detonating Code	500M/Coil	M	0.97	0.98	US	
Detonating Code	Under Water Use	M	0.97	0.98	US	
LEG Wire	Copper 0.42MM	M	0.06	0.07	US	
Steel Products						
Shaped Steel	L-Shape	Ton	490.20	507.80	Countries of Middle/South America Brazil, Argentine, Venezuela, Mexico, etc.	
Shaped Steel	H-Shape	Ton	498.40	517.60		
Sheet Steel	T=12MM	Ton	462.20	478.20		
Re-Bars	Round	Ton	462.10	478.10		
Re-Bars	Deformed	Ton	462.10	478.10		

Table II. 3.2.1 - 2

As of 1990

Description	Specification	Unit	CIF Panama ($/Unit)	Total On Site ($)	Import From	Remarks
Sheet Pile						
Steel Pipe Pile	Steel, Dia500	Ton	671.00	709.22	Countries of Middle/South America	
Steel Pipe Pile	Dia200x16MM	Ton	1062.00	1,152.13		
Steel Tube (Pipe)	Dewatering, D=400MM	Ton	639.10	675.99		
Steel Tube (Pipe)	Pipeline, D=100MM	Ton	545.30	563.88		
Steel Tube (Pipe)	Seamless, 25MM	Ton	512.80	528.98		
Annealed Wire		Ton	746.00	762.00		
Cement	Ordinary Portland	Ton	93.10	103.15	US	Bulk
Cement	Moderate Heat	Ton	93.10	103.15	US	Bulk
Admixture	Water Reducing	L	2.25	2.27	US	Bulk
Sand	For Concrete	M3	-	-	to be produced at site	
Crushed Stone	For Concrete,<40MM	Ton	-	-	to be produced at site	
Crushed Stone	For Concrete,<40MM	Ton	-	-	to be produced at site	
Ready Mixed concrete		M3	-	-	to be produced at site	
Asphalt Premixture	For Road					
Straight Asphalt	Needle Penet.80-100	Ton	309.26	325.58	US	
Emulsion	MC 30	Ton	303.26	303.26	US	
Lumber		M3		185.00	Local	
Electric Wire						
Copper (No Cover)	Dia.5MM or 22SQMM	M		6.73	US	
Cabtyre, (3P)	High Voltage,100MM2	M		36.39	US	4 Cores
Cabtyre, (2P)	Low Voltage,200MM2	M		27.93	US	3 Cores

Table II. 3.2.1 - 3

As of 1990

Description	Specification	Unit	CIF Panama ($/Unit)	Total On Site ($)	Import From	Remarks
ROD						
ROD (ø165x10.7)	1.8 Ton	No.		5,750.24	US	
ROD (ø225x10.7)	3.34 Ton	No.		9,656.11	US	
BIT						
BIT (ø165)		No.		2,062.56	US	for drilling
BIT (ø225)		No.		4,546.56	US	for drilling

2.2 Manpower

Table II. 3.2.2-1 Manpower (Staff & Labor) Rates - As of 1990

Occupation/Trade	Hourly Rate ($/Hr)	Social Charges ($/Hr)	Total Hourly ($/Hr)	Daily Rate ($/Day)
General Foreman	3.60	2.41	6.01	48.08
Foreman	2.92	1.96	4.88	39.04
Common Worker	1.35	0.90	2.25	18.00
Plasterer, Bar Bender, Plumber Carpenter, Electrician	1.70	1.14	2.84	22.72
Rigger	1.45	0.97	2.42	19.36
Painter	1.50	1.01	2.51	20.08
Watchman	1.00	0.67	1.67	13.36
Land Surveyor	2.00	1.34	3.34	26.72
Chainman	1.45	0.97	2.42	19.36
Survey Assistant	1.35	0.90	2.25	18.00
Operator (Heavy Duty Equipment)	2.50	1.68	4.18	33.44
Operator	1.80	1.21	3.01	24.08
Driver(Heavy)	1.80	1.21	3.01	24.08
Driver (Vehicle)	1.30	0.87	2.17	17.36
Mechanic	2.10	1.41	3.51	28.08
Steel Assembler	2.40	1.61	4.01	32.08
Steel Welder	3.50	2.35	5.85	46.80
Steel Pipe Worker	3.00	2.01	5.01	40.08
Marine Works Expatriate		10,000.00	(Monthly Rate)	
Marine Works Senior Crew	5.70	3.82	9.52	95.20
Marine Works Crew	3.50	2.35	5.85	58.50
Marine Works Deck Hand	1.80	1.21	3.01	30.10
Marine Works Drilling Crew	4.00	2.68	6.68	80.16
Marine Works Diver		3,000.00	(Monthly Rate)	
Marine Works Diver Assistant		1,500.00	(Monthly Rate)	

Note: 1) Hourly rates are mainly based on CAPAC (Camara Panamena de la Con struccion 15 de Octubre de 1990, No.48)

2) Cost estimate assumes that skilled, qualified Panamanian craftsmen are available in sufficient quantity to support all construction activities. No cost allowance is included for training or importing craftsmen from outside Panama.

3) Note that labor wage rates in 1990 for manual craftsmen in Panama decreased following the liberation of Panama (Operation Just Cause) and the above rates reflect that decline.

2.3 Equipment

Table II.3.2.3-1 Equipment Costs for Earthworks (1)

(FILE:PANAM-04)

EQUIPMENT	HORSE POWER (HP/KW)	PRICE ON SITE ($)	DEPRECIATION IN LIFETIME 90% ($)	DEPRECIATION OF EQUIP ($/hr)	FUEL & L. OIL ($/hr)	PARTS REPAIR COST ($/hr)	CONSUMABLE COST ($/hr)	MANAGE. COST ($/hr)	EQUIP. COST ($/hr)	LABOR COST TOTAL ($/hr)	TOTAL EQUIP. COST ($/hr)
DUMP TRUCK								(7%/ANNUM)			
DUMP TRUCK(150TON)	1,308	1,078,061	970,255	37.28	14.01	57.99	45.84	17.40	172.51	6.92	179.43
DUMP TRUCK(195TON)	1,825	1,323,692	1,191,323	45.77	19.54	71.20	55.62	21.36	213.49	6.92	220.42
DUMP TRUCK(240TON)	2,190	1,631,506	1,468,356	56.41	23.45	87.76	68.79	26.33	262.73	6.92	269.66
BULLDOZER								(7%/ANNUM)			
					0.00						
BULLDOZER(46TON)	375	600,012	540,011	30.00	10.04	33.33	-	14.00	87.37	8.22	95.60
BULLDOZER(63TON)	527	608,944	548,050	30.45	14.11	33.83	-	14.21	92.59	8.22	100.82
BULLDOZER(98TON)	781	1,006,084	905,476	50.30	20.91	55.89	-	23.48	150.58	8.22	158.80
POWER SHOVEL								(7%/ANNUM)			
POWER SHOVEL(30M3)	3,590	7,536,966	6,783,270	260.61	96.11	579.14	-	121.62	1,057.49	8.22	1,065.71
POWER SHOVEL(18M3)	1,523	4,360,608	3,924,547	150.78	40.77	335.07	-	70.36	596.99	8.22	605.21
POWER SHOVEL(15M3)	1,684	3,506,832	3,156,149	121.26	45.08	269.47	-	56.59	492.40	8.22	500.62
MOTOR SCRAPER(34M3	550	1,325,968	1,193,371	49.72	14.72	110.50	56.05	23.20	254.20	8.22	262.42
WHEEL LOADER(20M3)	1,336	2,005,140	1,804,626	75.19	35.77	116.97	85.21	35.09	348.22	8.22	356.45
WHEEL LOADER(10M3)	735	1,105,005	994,505	66.30	19.68	103.13	47.23	30.94	267.29	8.22	275.51
CRAWLER DRILL(15T)	290	392,000	352,800	17.43	7.76	17.43	-	8.13	50.75	10.59	61.34
BLAST HOLE DRILL	600	734,760	661,284	32.67	16.06	32.67	-	15.24	96.64	10.59	107.23
BLAST HOLE DRILL	600	734,760	661,284	32.67	16.06	32.67	-	15.24	96.64	10.59	107.23
DRILL HAMMER											
DRILL HAMMER, φ165	-	16,640	14,976	1.25	0.00	1.94	-	0.35	3.53	7.37	10.90
DRILL HAMMER, φ165	-	19,760	17,784	1.48	0.00	2.30	-	0.41	4.19	7.37	11.57

Table II.3.2.3-2 Equipment Costs for Earthworks (2)

EQUIPMENT	HORSE POWER (HP/KW)	PRICE ON SITE ($)	DEPRECIATION IN LIFETIME 90% ($)	DEPRECIATION OF EQUIP ($/hr)	FUEL & L. OIL ($/hr)	PARTS REPAIR COST ($/hr)	CONSUMABLE COST ($/hr)	MANAGE. COST ($/hr)	EQUIP. COST ($/hr)	LABOR COST TOTAL ($/hr)	TOTAL EQUIP. COST ($/hr)
TIRE ROLLER(20-30T	100	59,528	53,575	3.57	2.68	2.78	0.00	1.95	10.97	8.22	19.19
MOTOR GRADER(4.9M)	280	327,096	294,386	19.63	7.50	15.26	0.00	10.69	53.08	8.22	61.30
VIB. ROLLER(15T)	177	115,432	103,889	6.93	4.74	6.16	0.00	3.23	21.05	8.22	29.27
COMPACTOR(32.4T)	315	546,529	491,876	32.79	8.43	29.15	0.00	15.30	85.68	8.22	93.90
FOR GENERAL USE											
BULLDOZER											
BULLDOZER(32TON)	339	557,360	501,624	22.80	9.08	17.73	0.00	10.65	60.26	8.22	68.48
BULLDOZER(24TON)	203	289,166	260,249	21.69	5.43	16.87	0.00	5.53	49.52	8.22	57.74
BULLDOZER(16TON)	142	195,800	176,220	14.69	3.80	11.42	0.00	3.74	33.65	8.22	41.87
BACKHOE											
BACKHOE(2.0 M3,STR	375	698,754	628,879	52.41	10.04	40.76	0.00	20.38	123.58	8.22	131.80
BACKHOE(1.5 M3,STR	276	427,940	385,146	32.10	7.39	24.96	0.00	12.48	76.93	8.22	85.15
BACKHOE(1.1 M3,STR	222	322,022	289,820	24.15	5.94	18.78	0.00	9.39	58.27	8.22	66.49
BACKHOE(0.8 M3,STR	148	246,244	221,620	18.47	3.96	14.36	0.00	7.18	43.98	8.22	52.20
BACKHOE(0.6 M3,STR	128	185,394	166,855	13.90	3.43	10.81	0.00	5.41	33.55	8.22	41.77
POWER SHOVEL											
P. SHOVEL(3.6 M3 S	500	946,194	851,575	32.75	13.39	72.78	0.00	15.30	134.22	8.22	142.44
P. SHOVEL(6.0 M3 S	800	2,010,746	1,809,671	69.60	21.42	154.67	0.00	32.51	278.20	8.22	286.42
DUMP TRUCK											
DUMP TRUCK(32.0T)	456	430,505	387,455	19.37	4.88	25.83	19.46	9.04	78.59	6.92	85.51
DUMP TRUCK(77.0T)	920	896,155	806,540	33.61	9.85	44.81	40.51	15.68	144.45	6.92	151.38

Table II.3.2.3-3 Equipment/Vesselels Cost for Canal Dredging

(FILE:¥EQPCOS¥PLCOST)

EQUIPMENT	HORSE POWER (HP/KW)	PRICE ON SITE PANAMA ($)	DEPRECIATION IN LIFETIME 90% ($)	DEPRECIATI OF EQUIP. ($/hr)	F & O COST ($/hr)	PART REPAIR COST ($/hr)	CONSUMABLE COST ($/hr)	MANAGE. COST ($/hr)	EQUIP. COST PER YEAR ($/hr)	LABOR COST TOTAL ($/hr)	TOTAL EQUIP. COST ($/hr)
CUTTER SUCTION(WEATHERED ROCK)	27,500	40,771,429	36,694,286	574.79	828.83	880.39	1304.54	536.47	4125.01	271.40	4396.41
CUTTER SUCTION(HARD ROCK)	27,500	40,771,429	36,694,286	718.48	828.83	1,100.49	1681.09	670.58	4999.47	339.25	5338.72
CUTTER SUCTION(LOOSE MATERIAL)	18,000	27,485,714	24,737,143	387.49	542.51	577.14	368.55	361.65	2237.34	271.40	2508.74
ANCHOR BOAT	600	1,785,714	1,607,143	23.98	9.04	38.63	1.68	26.11	99.43	20.03	119.46
SURVEY/WORK BOAT	420	1,342,857	1,208,571	18.03	6.33	29.05	1.26	19.63	74.30	20.03	94.33
CRANE BARGE(30 t)	390	461,429	415,286	6.51	5.88	7.95	0.43	6.07	26.84	24.89	51.73
PASSENGER BOAT(LAUNCH)	180	131,429	118,286	2.06	2.71	3.55	0.12	1.92	10.36	12.41	22.77
LOADING PONTOON	350	1,214,286	1,092,857	17.12	5.27	20.92	1.14	15.98	60.43	38.61	99.05
SPLIT BARGE(5,000m3)	-	3,850,000	3,465,000	72.37	-	133.25	30.75	39.80	276.17	27.83	304.01
PUSHER BOAT	3,200	3,646,429	3,281,786	51.41	0.00	58.47	23.06	37.70	294.64	269.77	440.42
TRAILER DREDGER(LOOSE MATERIAL	19,000	82,557,143	74,301,429	831.34	572.64	1,078.91	299.38	905.23	3687.50	422.47	4109.97
TRAILER DREDGER(LOOSE MATERIAL	16,000	68,658,714	66,485,714	743.89	482.23	892.30	251.73	752.84	3122.98	422.47	3545.46
GRAB DREDGER(CLAY & ROCK)	3,000	13,314,286	11,982,857	312.84	90.42	317.87	489.60	194.65	1405.37	262.53	1667.90
BLASTING BARGE(HARD ROCK)	7,180	16,202,057	14,581,851	425.75	216.40	3,609.39	3621.59	849.78	8525.57	744.73	9270.30
FLOATING CRANE	1,000	6,714,286	6,042,857	94.66	30.14	94.66	9.47	88.35	317.26	176.45	493.71
TUG BOAT	500	507,143	456,429	6.13	19.38	9.19	0.92	6.67	42.29	18.03	60.31
ANCHOR BOAT	600	1,785,714	1,607,143	21.58	18.08	34.76	1.68	23.50	99.60	18.03	117.63
MIXING BARGE	350	285,714	257,143	4.03	10.55	4.92	0.49	2.95	22.95	73.70	96.64
DIVER BOAT	105	107,143	96,429	1.29	3.16	1.44	0.14	1.41	7.45	38.42	45.87
WARNING BOAT	600	594,286	534,857	7.18	18.08	10.77	75.23	7.82	119.09	18.03	137.11
SPLIT BARGE(1,500 m3)	-	2,652,857	2,387,571	41.55	-	99.10	30.75	30.47	201.88	30.93	232.81
PUSHER BOAT	1,300	2,088,571	1,879,714	32.72	0.00	31.08	23.06	23.99	161.23	125.06	255.29

Table II.3.2.3-4 Equipment/Vesselels Costs for Barrier Dam

(FILE:¥EQPCOS¥PCOSTBAL)

EQUIPMENT	HORSE POWER (HP/KW)	PRICE ON SITE PANAMA ($)	DREPRECIATION OF EQUIPMENT ($/hr)	F & O COST PER HOUR ($/hr)	PART REPAIR COST ($/hr)	CONSUMABLE COST ($/hr)	MANAGE. COST ($/hr)	EQUIP. COST PER HOUR ($/hr)	LABOR COST TOTAL ($/hr)	TOTAL EQUIP. COST ($/hr)
BACKHOE BARGE(700t)	250	431,429	145.97	7.53	15.19	489.60	11.35	535.84	180.36	716.20
BACKHOE 8.0 m3	872	219,143	74.15	26.28	20.71	489.60	5.77	552.95	22.33	575.29
CRANE BARGE(700t)	300	511,000	14.41	9.04	17.46	17.22	13.45	71.58	180.36	251.94
CRANE 50t x 2 units	207	886,857	42.87	12.48	85.29	38.55	23.34	405.03	44.66	899.39
STONE BARGE(2,000t)	3000	5,085,390	1720.62	90.42	205.58	202.74	133.83	775.95	180.36	956.31
STONE BARGE(1,000t)	2420	4,018,214	1359.55	72.94	165.83	163.54	105.74	621.35	180.36	801.71
SPLIT BARGE(5,000m3)	-	3,850,000	1302.63	-	266.50	30.75	79.61	521.59	55.67	577.26
PUSHER BOAT	3,200	3,646,429	1233.75	124.00	116.95	23.06	75.40	442.22	539.55	981.77
FLAT TOP BARGE(2000t)	-	976,857	330.52	-	70.75	30.75	20.20	158.42	41.56	199.98
TUG BOAT(1,300 HP)	1,300	1,204,286	407.47	50.38	43.64	23.06	24.90	175.93	116.77	292.70
FLAT TOP BARGE(1000t)	-	575,714	194.79	-	37.24	30.75	11.90	101.53	41.56	143.09
TUG BOAT(800HP)	800	854,286	289.04	31.00	29.93	23.06	17.66	125.74	116.77	242.51
CUTTER SUCTION(18,000m3)	18,000	27,485,714	4649.84	542.51	577.14	351.00	361.65	2219.79	271.40	2491.19
ANCHOR BOAT	600	1,785,714	604.19	18.08	69.53	1.59	46.99	179.35	36.05	215.41
SURVEY/WORK BOAT	420	1,342,857	454.35	12.66	52.29	1.20	35.34	133.93	36.05	169.99
WORK BOAT	390	461,429	156.12	11.75	15.90	0.41	12.14	53.22	49.78	103.00
LOADING PONTOON	350	1,214,286	410.85	10.55	41.85	1.08	31.95	119.67	77.22	196.89
PASSENGER BOAT(LAUNCH)	180	131,429	44.47	5.43	6.38	0.12	3.46	19.09	22.33	41.42
CRANE 70t	207	559,857	189.43	6.24	54.29	24.54	14.73	126.86	22.33	149.19
WHEEL LOADER(988B)	380	516,800	174.86	10.90	45.33	23.36	13.60	122.34	23.72	146.05

Table II.3.2.3-5 Equipment/Vesselels Costs for Breakwater on the Atlantic Side

(FILE:¥EQPCOS¥PCOSTATL)

EQUIPMENT	HORSE POWER (HP/KW)	PRICE ON SITE PANAMA ($)	DEPRECIATION IN LIFETIME 90% ($)	DREPRECIA-TION OF EQUIPMENT ($/hr)	F & O COST PER HOUR ($/hr)	PART REPAIR COST ($/hr)	CONSUMA-ble COST ($/hr)	MANAGE. COST ($/hr)	EQUIP. COST PER HOUR ($/hr)	LABOR COST TOTAL ($/hr)	TOTAL EQUIP. COST ($/hr)
BACKHOE BARGE(700t)	250	431,429	388,286	236.76	7.53	15.19	489.60	18.41	550.47	292.54	843.01
BACKHOE 8.0 m3	872	219,143	197,229	120.26	26.28	20.71	489.60	9.35	563.13	36.22	599.35
CRANE BARGE(700t)	300	511,000	459,900	23.37	9.04	17.46	17.22	21.81	88.91	292.54	381.44
CRANE 50t x 2 units	207	886,857	798,171	69.53	12.48	85.29	38.55	37.85	487.39	72.44	1,119.65
STONE BARGE(2,000t)	3,000	5,085,390	4,576,851	2,790.76	90.42	205.58	202.74	217.06	948.36	292.54	1,240.90
STONE BARGE(1,000t)	2,420	4,018,214	3,616,393	2,205.12	72.94	165.83	163.54	171.51	757.58	292.54	1,050.12
SPLIT BARGE(5,000m3)	-	3,850,000	3,465,000	2,112.80	-	432.25	30.75	129.12	826.87	90.29	917.17
PUSHER BOAT	3,200	3,646,429	3,281,786	2,001.09	124.00	189.69	23.06	122.29	625.80	875.12	1,500.92
FLAT TOP BARGE(2000t)	-	936,857	843,171	514.13	-	114.75	30.75	31.42	234.05	67.40	301.45
TUG BOAT(1,300 HP)	1,300	1,204,286	1,083,857	660.89	50.38	70.78	23.06	40.39	239.68	189.40	429.07
FLAT TOP BARGE(1000t)	-	575,714	518,143	315.94	-	60.39	30.75	19.31	145.56	67.40	212.96
TUG BOAT(800HP)	800	854,286	768,857	468.82	31.00	48.54	23.06	28.65	170.33	189.40	359.72
CUTTER SUCTION(18,000m3)	18,000	27,485,714	24,737,143	7,541.81	542.51	936.10	351.00	586.59	3,044.68	440.20	3,484.87
ANCHOR BOAT	600	1,785,714	1,607,143	979.97	18.08	112.77	1.59	76.22	278.66	58.48	337.14
SURVEY/WORK BOAT	420	1,342,857	1,208,571	736.93	12.66	84.81	1.20	57.32	208.61	58.48	267.09
WORK BOAT	390	461,429	415,286	253.22	11.75	25.79	0.41	19.70	78.75	80.74	159.49
LOADING PONTOON	350	1,214,286	1,092,857	666.38	10.55	67.87	1.08	51.83	186.86	125.25	312.12
PASSENGER BOAT(LAUNCH)	180	131,429	118,286	72.13	5.43	10.35	0.12	5.61	27.51	36.22	63.73
CRANE 70t	207	559,857	503,871	307.24	6.24	54.29	24.54	23.90	152.85	36.22	189.07
WHEEL LOADER(988B)	380	516,800	465,120	283.61	10.90	73.53	23.36	22.06	177.12	38.47	215.58

Table II.3.2.3-6 Equipment/Vesselels Costs for Breakwater on the Pacific Side

(FILE:¥EQPCOS¥PCOSTPAC)

EQUIPMENT	HORSE POWER (HP/KW)	PRICE ON SITE PANAMA ($)	DEPRECIATION IN LIFETIME 90% ($)	DREPRECIA-TION OF EQUIPMENT ($/hr)	F & O COST PER HOUR (U$/hr)	PART REPAIR COST ($/hr)	CONSUMA-BLE COST ($/hr)	MANAGE. COST ($/hr)	EQUIP. COST PER HOUR ($/hr)	LABOR COST TOTAL ($/hr)	TOTAL EQUIP. COST ($/hr)
BACKHOE BARGE(700t)	250	431,429	388,286	218	7.53	15.19	489.60	16.97	547.47	269.53	817.00
BACKHOE 8.0 m3	872	219,143	197,229	111	26.28	20.71	489.60	8.62	561.04	33.37	594.41
CRANE BARGE(700t)	300	511,000	459,900	22	9.04	17.46	17.22	20.10	85.35	269.53	354.88
CRANE 50t x 2 units	207	886,857	798,171	64	12.48	85.29	38.55	34.88	470.50	66.74	1,074.47
STONE BARGE(2,000t)	3000	5,085,390	4,576,851	2,571	90.42	205.58	202.74	199.99	913.00	269.53	1,182.52
STONE BARGE(1,000t)	2420	4,018,214	3,616,393	2,032	72.94	165.83	163.54	158.02	729.64	269.53	999.17
SPLIT BARGE(5,000m3)	-	3,850,000	3,465,000	1,947	-	398.25	30.75	118.96	764.26	83.19	847.45
PUSHER BOAT	3,200	3,646,429	3,281,786	1,844	124.00	174.77	23.06	112.67	588.14	806.29	1,394.44
FLAT TOP BARGE(2000t)	-	976,857	879,171	494	-	105.72	30.75	30.18	221.54	62.10	283.64
TUG BOAT(1,300 HP)	1,300	1,204,286	1,083,857	609	50.38	65.21	23.06	37.21	226.60	174.50	401.10
FLAT TOP BARGE(1000t)	-	575,714	518,143	291	-	55.64	30.75	17.79	136.53	62.10	198.63
TUG BOAT(800HP)	800	854,286	768,857	432	31.00	44.73	23.06	26.40	161.18	174.50	335.68
CUTTER SUCTION(18,000m3)	18,000	27,485,714	24,737,143	6,949	542.51	862.47	351.00	540.45	2875.48	405.57	3,281.06
ANCHOR BOAT	600	1,785,714	1,607,143	903	18.08	103.90	1.59	70.22	258.29	53.88	312.17
SURVEY/WORK BOAT	420	1,342,857	1,208,571	679	12.66	78.14	1.20	52.81	193.30	53.88	247.18
WORK BOAT	390	461,429	415,286	233	11.75	23.76	0.41	18.15	73.52	74.39	147.90
LOADING PONTOON	350	1,214,286	1,092,857	614	10.55	62.53	1.08	47.75	173.08	115.40	288.48
PASSENGER BOAT(LAUNCH)	180	131,429	118,286	66	5.43	9.54	0.12	5.17	25.79	33.37	59.16
CRANE 70t	207	559,857	503,871	283	6.24	54.29	24.54	22.02	147.52	33.37	180.89
WHEEL LOADER(988B)	380	516,800	465,120	261	10.90	67.75	23.36	20.32	165.88	35.44	201.32

II.3 CHAPTER 3 - OPERATION AND MAINTENANCE COSTS

3.1 Introduction

In the Phase 1 Final Report, the Operation and Maintenance (O & M) costs for each of the study cases under consideration were calculated by applying a cost ratio to the known costs associated with the operation and maintenance of the existing Panama Canal as presented in the 1990 PCC Annual Report. A similar method is employed for the Phase 2 Report and the results are allocated separately for the existing canal and the new facility for each of the alternatives studied.

3.2 Previous Studies

The IOCS Commission Report considered the experiences of various major world waterways, including the existing Panama Canal, as a basis for determining the general operation and maintenance costs and requirements for various sea-level and lock canals. Appendix 8 of Annex V - Study of Engineering Feasibility presents and discusses the characteristics of the Panama, Suez, Cape Cod, St Lawrence, Kiel and North Sea - Amsterdam Canals and based on this information, recommendations are made for a new interoceanic canal. The appendix is general in nature and cost estimates for individual routes are given in the appendices for each specific route.

Chapter II of Annex III - Study of Canal Finance summarises the basic estimates of construction costs, operating expenses and revenues from projected interoceanic canal traffic for the most favourable of the canal routes considered. Those relevant to the current study are the Route 10 sea level canal, the Route 15 lock canal and the continued operation of the existing lock canal. The estimated annual cost of operating and maintaining the existing lock canal from 1971 to 2000 is given, based on

information provided by the Panama Canal Company, which appears to be limited to the costs involved with depreciation and operation of planned improvements and the cost of additional pilotage to cope with the increase in annual transits. The cost of operation and maintenance of a sea level canal on Route 10 and the additional cost of operation and maintenance of new deep draft locks alongside the existing facilities (Route 15) are stated without giving a detailed breakdown or derivation.

The costs associated with each of the case studies are described in different sections of Annex V - The Study of Engineering Feasibility. In Chapter 9 - Cost Elements, the O & M costs are described as those continuing costs incurred in the operation and maintenance of the facilities directly involved in the operation of the canal. The O & M costs are subdivided into those which vary only slightly from year to year (fixed costs) and those which vary with the number of transits (variable costs). The total O & M cost would be the sum of the fixed costs, the variable costs and the costs of major replacements. Attention is drawn to the fact that O & M costs are strongly dependent on the length of the canal. In Appendix 10 : Routes 10 and 14, Section M : Operation and Maintenance Requirements further defines the basic components of annual operation and maintenance costs as personnel and nonpersonnel costs, some of which are fixed and others, primarily the costs for marine pilots and tugs, which vary significantly with the number of transits.

3.3 O & M Costs for Existing Canal - PCC Annual Reports

The Panama Canal Commission (PCC) in their Annual Reports present a breakdown of operating revenues and expenses as a Statement of Operations. Operating revenues are obtained from two principal sources - revenue from tolls (approx 70%) and non-toll revenue, mainly from navigation fees and sales of electric power. Operating expenses are divided between payments made to the Republic of Panama

and the actual costs associated with operating and maintaining the canal and its supporting services. The operating expenses were previously sub-divided into 10 divisions and in the Phase 1 Report this itemised breakdown from the 1990 PCC Annual Report was used as a base from which to estimate O & M costs for each of the 36 alternatives.

However in the 1991 Report, the sub-headings were revised and depreciation included as a separate item for the first time. The list of operating expenses as given in the 1991 Report follows :

Item 1 - Maintenance of channels, dams and spillways
Item 2 - Navigation service and control
Item 3 - Locks operation
Item 4 - General repair, engineering and maintenance services
Item 5 - Supply and transportation services
Item 6 - Utilities
Item 7 - Administrative and general
Item 8 - Depreciation
Item 9 - Fire and facility protection
Item 10- Interest on interest-bearing investment
Item 11- Other

The 1991 Annual Report provided data for the fiscal years ended September 30th 1990 and 1991 and the operating expenses are given in Table II.3.3-1 on the next page.

A more detailed breakdown of Items 1, 3 and 4, including actual expenditure for 1991 and anticipated expenditure for 1992 and 1993, was provided by PCC (Ref: CAS letter dated Jan 8th 1993) and this information is presented in Table II.3.3-2.

Based on the additional information provided by PCC and descriptions included in the Annual Reports, the itemised operation and maintenance costs are described below. For those items where information was not available, assumptions are included and explained.

Table II.3.3-1 PCC Annual Report 1991, Operating Expenses

PCC Annual Report - Financial Statement 1990 and 1991

Item No	Operating Expenses: ('000 US dollars)	1990 Actual		1991 Actual	
1	Channels, Dams and Spillways	29,482	7.6%	35,628	8.6%
2	Navigation Service and Control	78,912	20.3%	86,221	20.8%
3	Locks Operation	48,294	12.4%	53,778	13.0%
4	Repair, Engineering and Maintenance	23,074	5.9%	25,775	6.2%
5	Supply and Transportation	19,829	5.1%	23,851	5.7%
6	Utilities	29,854	7.7%	31,772	7.7%
7	Administrative and General	77,697	20.0%	78,576	18.9%
8	Depreciation	23,393	6.0%	22,685	5.5%
9	Fire and Facility Protection	12,699	3.3%	13,351	3.2%
10	Interest on Investment	15,113	3.9%	11,544	2.8%
11	Other	30,853	7.9%	32,059	7.7%
	Total Operating Expenses	389,200	100.0%	415,240	100.0%

Table II.3.3-2 Detailed Breakdown of Operating Expenses 1991

Operating Expenses: ('000 US dollars)	1991 Actual		1992 Estimate		1993 Estimate	
Item 1 - Maintenance of channels, dams and spillways						
Dams and Spillways *	503		748		647	
Dredging Operations	7,114	15.2%	8,162	17.2%	9,584	19.0%
Other Support Equipment Operations	6,797	14.5%	7,263	15.3%	7,653	15.2%
Drilling and Blasting Operations	2,218	4.7%	2,400	5.1%	2,378	4.7%
Meteorological and Hydrographic Surveys	3,116	6.7%	3,096	6.5%	3,333	6.6%
Maintenance, Repairs and Overhaul	9,946	21.3%	9,154	19.3%	10,286	20.4%
Supervision and General	2,626	5.6%	2,855	6.0%	2,867	5.7%
Other Channel Maintenance *	3,873	8.3%	3,256	6.9%	2,963	5.9%
Depreciation and Other Nonfund	11,037	23.6%	11,247	23.7%	11,331	22.5%
Total Operating Expenses	46,727		47,433		50,395	
* Dams and Spillways included in Other Channel Maintenance						
Item 3 - Lock operations						
Lockage Operations	14,123	24.8%	12,863	23.4%	13,557	23.2%
Locks Maintenance and Repair	26,445	46.4%	26,854	48.9%	29,369	50.3%
Supervision and General	2,919	5.1%	2,405	4.4%	2,569	4.4%
Accrual for Locks Overhaul	10,361	18.2%	9,385	17.1%	9,443	16.2%
Depreciation	3,123	5.5%	3,429	6.2%	3,499	6.0%
Total Operating Expenses	56,971		54,936		58,437	
Item 4 - General repair, engineering and maintenance services						
General Maintenance	17,935	28.1%	17,115	28.1%	17,978	28.2%
Vessel Repair Services	25,045	39.2%	22,513	37.0%	23,516	36.9%
Electrical Services	7,914	12.4%	8,285	13.6%	8,662	13.6%
Grounds and Sanitation Services	5,924	9.3%	5,909	9.7%	6,199	9.7%
Engineering Services	3,468	5.4%	3,502	5.7%	3,636	5.7%
Contract and Inspection Services	2,081	3.3%	2,027	3.3%	2,129	3.3%
Depreciation and Other Nonfund	1,454	2.3%	1,559	2.6%	1,654	2.6%
Total Operating Expenses	63,821		60,910		63,774	

3.3.1 Item 1 - Maintenance of channels, dams and spillways

The bulk of this cost is spent on the dredging and other support equipment operations and the maintenance, repairs and overhaul of the same. A significant proportion of this item (approx 23%) is allocated to depreciation of property, plant and equipment and other nonfund expenditure. However, for the purpose of estimating O & M costs, all expense associated with depreciation is considered under Item 8 of the list of operating expenses.

3.3.2 Item 2 - Navigation service and control

No detailed breakdown of this item has been provided by PCC, even though it constitutes the largest single item of the operating expenses listed in the 1991 Annual Report. It is assumed that this item includes the fixed costs of the navigation systems of lights, buoys, radar, radios and other means of communication as well as the transit related costs of tugs, pilots and launches. A proportion of this item is assumed to be the personnel costs of the pilots, tugboat and launch crews and traffic management staff.

3.3.3 Item 3 - Locks operation

From the information provided by PCC, it can be seen that the cost of operating the existing locks only amounts to 25% of this item, whereas the expenditure on maintenance, repair and accrual (method of averaging costs) for major overhaul of the locks totals approximately 65%. The Marine Bureau of the PCC which is responsible for the operation and maintenance of the locks and related facilities produces an overhaul schedule for the lock miter gates and associated valves. The schedule is tentative and is updated based on the condition of components found during periodic inspections. Typical life cycles for overhaul of the lock gates are 25 to 30 years and for the rising stem valves approximately 12 years. The cylindrical valves are replaced every 35 to 40 years. Two spare gates are stored floating in Gatun Lake and are used to replace the gates that are being overhauled - the spare

gates were themselves overhauled in 1988.

3.3.4 Item 4 - General repair, engineering and maintenance services

The principal costs in this category are for vessel repair (assumed to include tugboats, pilot launches and dredgers), general maintenance and electrical services. It is assumed that the costs of general maintenance and vessel repairs are transit related. The reason for the significant difference between the sum stated in the Annual Report for this item and the total given in the detailed information provided by PCC is not known, but is possibly caused by the inclusion of additional categories in the supplementary information. For reasons of consistency, the value stated in the Annual Report has been adopted for the operation and maintenance costs analysis.

3.3.5 Item 5 - Supply and transportation

The Panama Canal Commission operate and maintain a fleet of over 950 vehicles to meet the supply and transportation needs of operating the canal. The motor pool includes trucks, passenger vehicles and special purpose vehicles and is supported by fully equipped workshops for overhauls, repairs and maintenance. The Motor Transportation Division provides services ranging from transisthmian pilot trips to heavy equipment operation.

3.3.6 Item 6 - Utilities

Operation of the existing canal requires a range of utilities and the PCC facilities include electric power generation and distribution, communications and water purification and distribution. Both electricity and water are supplied by PCC to the Republic of Panama and revenues obtained are included in the Financial Statement under ' other revenues '.

3.3.7 Item 7 - Administrative and general

The costs associated with the administration and general

running of the Panama Canal constitute approximately 20% of the total operating expenses. No detailed information has been provided, but it is assumed that all the overheads and costs associated with an organisation of over 8,000 employees are included in this category.

3.3.8 Item 8 - Depreciation

Prior to the 1991 Annual Report, depreciation was not listed as a separate item and it is believed that an allowance for depreciation was made under the items to which it applied. This assumption is substantiated by the fact that the two different methods of presentation for the 1990 data (Table II.3.3-1 from the 1991 PCC Annual Report and Table II.3.3-3 based on the 1990 Annual Report) result in the same total figure for O&M costs. The Annual Report states that property, plant and equipment are depreciated over their estimated service lives at rates computed using the straight line method. In such a meaning, the depreciation allowance is considered to reflect the maintenance cost of major items and to be part of the regular maintenance program.

3.3.9 Item 9 - Fire and facility protection

Fire and facility protection is listed as a separate item for the first time in the 1991 Annual Report, presumably because of the completion of the improved fire protection system at Miraflores Locks. Installation of similar systems is scheduled for Pedro Miguel Locks in FY 1993 and Gatun Locks in FY 1995/6. Emergency and stand-by services are provided, especially at the locks for those vessels designated by the Marine Bureau as carrying dangerous cargoes.

3.3.10 Item 10 - Interest on interest-bearing investment

Interest is currently paid on interest-bearing capital including U.S. Government capital. This item is considered to be a transfer item and is not included in the analysis of O & M costs.

3.3.11 Item 11 - Other

No details are available of the costs covered by this category but it is assumed to include all the miscellaneous operation and maintenance costs which cannot be allocated to other categories.

3.4 Phase 1 Report - Operation and Maintenance Costs

In YEC's Phase 1 Final Report, the itemised breakdown of O & M costs given in the 1990 PCC Annual Report was used as the base from which to estimate the operation and maintenance costs for each of the 36 alternatives then under consideration. In the 1990 Annual Report, the operating expenses were subdivided into 10 items similar to those described in the previous section. YEC classified each of the items into groups according to whether the cost depends on the length of the canal, the size of the ship or the number of lifts of the lock. The total O & M cost of operating both the existing canal and the alternative was then calculated by applying a cost ratio to the 1990 costs. Cost ratios were estimated based on the limited data available and YEC's interpretation of likely changes.

SINBOL accepted the principle of YEC's analysis, but requested an incremental approach to separate the costs for the existing canal from those for each of the new alternatives. Comments were also made on the choice of cost ratios and SINBOL suggested changes to certain costs. These changes were incorporated in a restudy of the operation and maintenance costs and forwarded to CAS under letter dated May 13th 1992.

The routes chosen for detailed study in the Phase 2 Study correspond to Route I (IOCS Route 15) for the High Rise Lock Canal with design ship size of 150,000 DWT and 200,000 DWT, and to Route 10 for the Sea Level Canal with design ship size of 250,000 DWT. The incremental cost ratios and estimated operation and maintenance costs for

these routes as presented in the Phase 1 Report are given in Table II.3.3-3 a) & b) , together with the adjustments suggested by SINBOL.

Note : The Phase 1 O & M costs shown in Table II.3.3-3 were based on the breakdown of operating expenses given in the 1990 PCC Annual Report. The Phase 2 O & M costs are based on the slightly different breakdown given in the 1991 PCC Annual Report as described in the preceding section (see Tables II.3.3-4 & 5).

Table II.3.3-3 Phase 1 Report O & M Costs

a) Phase 1 - Operation and Maintenance Cost Ratios

O & M Expense	Status Quo 1990	Existing Canal	YEC Phase 1 Final Report HRL 150,000	HRL 250,000	SLC 250,000	SINBOL Adj'ment HRL	SLC
1	1.0	1.0	0.3	0.4	1.2	0.2	1.0
2	1.0	1.0	0.3	0.4	1.2	1.0	1.0
3	1.0	1.0	0.8	1.0	0.1	0.9	0.1
4	1.0	1.0	0.1	0.1	0.3	0.8	0.3
5	1.0	1.0	0.1	0.1	0.3	1.0	1.0
6	1.0	1.0	0.1	0.1	0.3	1.0	0.3
7	1.0	1.0	0.1	0.1	0.3	0.3	0.3
8	1.0	1.0	0.1	0.1	0.3	0.3	0.3
9	1.0	1.0	0.1	0.1	0.3	0.0	0.0
10	1.0	1.0	0.1	0.1	0.3	0.3	0.3

b) Phase 1 - Operation and Maintenance Costs ('000 US$ 1990)

1	40,236	40,200	12,100	16,100	48,300	8,000	40,200
2	82,747	82,700	24,800	33,100	99,300	82,700	82,700
3	51,318	51,300	41,100	51,300	5,100	46,200	5,100
4	24,485	24,500	2,400	2,400	7,300	19,600	7,400
5	22,162	22,200	2,200	2,200	6,600	22,200	22,200
6	33,615	33,600	3,400	3,400	10,100	33,600	10,100
7	4,895	4,900	500	500	1,500	1,500	1,500
8	75,051	75,100	7,500	7,500	22,500	22,500	22,500
9	15,113	15,100	1,500	1,500	4,500	0	0
10	39,578	39,600	4,000	4,000	11,900	11,900	11,900
Total	389,200	389,200	99,500	122,000	217,100	248,200	203,600
TOTAL	389,200	389,200	488,700	511,200	606,300	637,400	592,800

Expense	Description
1	Maintenance of channels and harbours
2	Navigation service and control
3	Locks operation
4	General repair, engineering and maintenance services
5	Supply and logistics
6	Utilities
7	Housing operations
8	General and administrative
9	Interest on interest-bearing investment
10	Other
Total	Incremental sub-total for each alternative
TOTAL	Total O & M cost for existing canal plus new alternative

3.5 High Rise Lock Canal - O & M Cost Factors

As described in previous sections, the O & M costs of operating the existing canal in 1990 (as presented in the 1991 PCC Annual Report) are used as a base from which to estimate the operation and maintenance costs of the Phase 2 Study alternatives. The cost ratios to be applied are described below and are generally similar to those adopted in Phase 1, but take SINBOL's comments into consideration. Certain of the cost items will vary with the number of ship transits for both the existing canal and the new alternatives.

3.5.1 Item 1 - Maintenance of channels, dams and spillways

As the major proportion of this item is dedicated to dredging and support operations (and maintenance and repair of the same), this item is strongly dependent on both the length and cross-section of the canal and therefore the maximum size of ship under consideration is of importance. The high rise lock alternatives involve the lengthening and deepening of both ocean canal approaches, the overall effect of which is to increase the total length of the existing canal by approximately 10% for the 150,000 DWT ship case and 14% for the 200,000 DWT ship case. The increase in width and depth of the canal will also increase the costs of maintaining the channel and again this increase will depend on the maximum size of ship. For these reasons, it is proposed that the incremental cost ratio for the 150,000 DWT and 200,000 DWT cases be 0.2 and 0.3 respectively for Year 2020. Lockage water demands in Year 2060 require the construction of new dams. To allow for any costs associated with the operation and maintenance of these dams and for slope monitoring equipment, the ratios have been increased to 0.3 and 0.4 respectively for Year 2060.

3.5.2 Item 2 - Navigation service and control

The operation and maintenance costs associated with

navigation service and control are divided between the fixed costs of the navigation control system and the variable costs of tugs and pilots which will depend on the number of ship transits through the canal. As a relatively low proportion of transits have been allocated to the new locks and the navigation service is shared with the existing canal, the incremental cost ratio will be taken as 0.2 for both the 150,000 DWT and 200,000 DWT cases in Year 2020, and as 0.4 for both cases in Year 2060.

3.5.3 Item 3 - Locks operation

The O & M costs of lock operation are also strongly dependent on the number of ship transits. From the additional information pro-vided by PCC, it can be seen that approximately 25% of this cost is spent on lockage operations, while nearly 50% is dedicated to locks maintenance and repair. The breakdown for 1991 also shows an accrual item for lock overhauls which is the method adopted for averaging the cost of major overhauls over several years. The high rise lock canal alternatives in Phase 1 considered the case of new two lane locks alongside the existing locks. However, the Phase 2 Study only considers the case of single lane locks and so the cost ratios are lower than those in the Phase 1 Report. The incremental cost ratios will also vary with size of ship as larger locks will incur higher O & M costs because of the larger gates, valves, mules etc. Close proximity to the existing locks will allow the sharing of some common facilities and this fact is allowed for in assessing estimated values for increased costs. Water demand requirement in Year 2060 necessitates the provision of new pumping stations for both the 150,000 DWT and 200,000 DWT ship cases. The values chosen for the incremental cost ratios include the anticipated O&M costs for such pumping stations. The following values for incremental cost ratio have been adopted :-

	Year 2020	Year 2060
150,000 DWT	0.5	0.7
200,000 DWT	0.6	0.8

3.5.4 Item 4 - General repair, engineering and maintenance services

The major portion of this expense is considered to be spent on repairs to support vessels and general maintenance associated with lock operation. This assumption is based on the additional information provided by PCC and the comments made by SINBOL on YEC's Phase 1 Report. For these reasons, the O & M costs in this category are again strongly dependent on the number of ship transits, but the size of ship is of comparatively minor importance. The incremental cost ratios for both the 150,000 DWT and 200,000 DWT ship cases have been chosen as 0.6 for Year 2020 and 0.8 for Year 2060.

3.5.5 Item 5 - Supply and transportation services

Whilst many of these services could be shared with the existing canal, the increased number of transits will obviously increase the O & M cost. Also included under this item is an allowance for the maintenance of the new canal crossings - two new bridges are planned for Year 2020 and a further two for Year 2060. For these reasons, incremental cost ratios of 0.4 for Year 2020 and 0.8 for Year 2060 have been assumed.

3.5.6 Items 6, 7, 9 and 11 - Utilities, Administrative and general, Fire and facility protection and Other.

All of these expenses are common to the existing canal but the additional locks and increased transits will of necessity lead to increased O & M costs. Incremental cost ratios of 0.2 for Year 2020 and 0.3 for Year 2060 have been assumed for all these items for both the 150,000 DWT and 200,000 DWT ship cases.

Electrical power. A separate item for the anticipated costs of electrical power has been included in the O & M costs section of the comparative summation table which is presented in the executive summary. Currently, the power requirements of the existing canal are met by hydro-electric power generation as part of canal operation.

With the larger locks and increased transits, lockage water requirements will take precedence and power generation will no longer be possible. The conceptual design of the electricity supply and distribution system outlined in Part IV, Section 5, Chapter 5 (Item 5.2.8 - Electric Power Control) assumes that a suitable commercial power supply is available at the lock sites. The electric power demands for locomotives, lighting and lock gate operation have been calculated and the estimated costs included in the cost summation table as the separate item "electrical power" in Section E - O & M Cost.

3.5.7 Item 8 - Depreciation

The incremental cost ratio approach is not thought to be suitable as a method to estimate the depreciation allowance. Based on the figures given for depreciation in the 1991 Annual Report, the allowance has been calculated as 6% of the total annual O & M cost for all cases under consideration. As explained previously, this allowance represents a maintenance cost of major items as part of the regular maintenance program. An accrual item is also included within Item 3 - locks operation, to allow for the cost of major overhauls of the locks and gates.

3.5.8 Item 10 - Interest on interest-bearing investment.

As stated in SINBOL's comments on the Phase 1 Report, for the purpose of economic analysis, interest is considered to be a transfer item and no further consideration has been given.

The above proposed incremental cost ratios for the High Rise Lock Canal are summarised in Table II.3.3-4 for Year 2020 and Table II.3.3-5 for Year 2060 at the end of the chapter. The resulting O & M costs are calculated in the same tables - all figures are in thousand US dollars at 1990 prices. A sub-total for each alternative is presented and the combined total for the existing canal and new alternative calculated.

3.6 Sea Level Canal - O & M Cost Factors

The incremental cost ratios to be applied to the 1990 operation and maintenance costs to estimate the O & M costs for the Sea Level Canal alternatives are described below. Attention is again drawn to those items that are considered to be transit related.

3.6.1 Item 1 - Maintenance of channels, dams and spillways

The sea level canal alternatives all consider a maximum ship size of 250,000 DWT but differ in the method adopted for maintaining a fresh water barrier between the two oceans. Associated with the three cases are varying distances of single or dual lane canal depending on the type of control structure considered. The total length of the sea level alternatives is also affected by the type of structure (because the canal bed level changes). The length of canal that is two lane is approximately 50% of the existing canal length for each of the three sea level cases and therefore an incremental cost of less than 1.0 is appropriate. Maintenance of the new dams, barrier dams and the facilities for the diversion scheme are also included in this item. There might be some opportunity to share dredging and maintenance equipment and facilities with the existing canal but because of the importance of maintaining both canals, separate operations are preferable. For this reason, the value of incremental cost ratio for all cases has been chosen as 0.8.

3.6.2 Item 2 - Navigation service and control

The fixed O & M costs of a completely separate navigation control system for the sea level canal would be approximately equal to those of the existing canal. The variable costs of pilots and tugs however will depend on the relative number of transits and, as the allocation to the sea level canal is a lot lower than to the existing canal, an incremental cost of less than 1.0 is valid. As the major proportion of this item is assumed to be the

variable costs of tugs, launches, pilots and other personnel, cost ratios of 0.6 for Year 2020 and 0.8 for Year 2060 have been chosen.

3.6.3 Item 3 - Locks operation

All three sea level canal cases will have significantly lower O & M costs for this category than either the existing canal or the high rise lock canal alternatives. It is also assumed that the costs associated with operating and maintaining the tidal gates will be considerably less than those for the tidal lock.The effect of increasing the number of transits has been estimated and the following cost ratios have been adopted :

	Year 2020	Year 2060
Case a) - Tidal lock at Pacific end :	0.3	0.4
Case b) - Tidal gates at both ends :	0.1	0.2
Case c) - Tidal lock at Pacific / tidal gate at Atlantic :	0.4	0.5

3.6.4 Item 4 - General repair, engineering and maintenance services

As the major portion of this expense is considered to be spent on repairs to support vessels and general maintenance associated with lock or gate operation, the cost is again considered to depend on the number of ship transits. The incremental cost ratio for Year 2020 is estimated as 0.2 and for Year 2060 as 0.4.

3.6.5 Item 5 - Supply and transportation services

Whilst some of these services could be shared with the existing canal, some duplication will be necessary for the new canal and this will obviously increase the O & M cost. Also included under this item is an allowance for the maintenance of the new canal crossings - three new bridges are planned for Year 2020 and a further four for Year 2060. For these reasons, incremental cost ratios of 0.5 for Year 2020 and 0.9 for Year 2060 have been assumed.

3.6.6 Items 6, 7, 9 and 11 - Utilities, Administrative and general, Fire and facility protection and Other.

While some of the expenses covered by these categories can be considered as common to the existing canal, others will increase by the nature of having two separate canals. Duplication of some fire and facility protection services will be necessary for the sea level canal whereas the administration of the new and existing canals could be handled by a single organisation. Costs of such items as utilities and general administration will increase with increasing number of transits. For these reasons and based on comments made by SINBOL on the Phase 1 Report, values of 0.3 for Year 2020 and 0.4 for Year 2060 have been chosen as incremental cost ratios.

Electrical power. As explained in section 3.5.6 for the high rise lock canal, the electric power demands for locomotives, lighting and lock gate operation have been calculated and the estimated costs included in the cost summation table presented in the executive summary.

3.6.7 Item 8 - Depreciation

As explained in section 3.5.7, the incremental cost ratio approach is not thought to be suitable as a method to estimate the depreciation allowance. Based on the figures given for depreciation in the 1991 Annual Report, the allowance has been calculated as 6% of the total annual O & M cost for all cases under consideration.

3.6.8 Item 10 - Interest on interest-bearing investment.

As stated in SINBOL's comments on the Phase 1 Report, for the purpose of economic analysis, interest is considered to be a transfer item and no further consideration has been given.

The above proposed incremental cost ratios for the three Sea Level Canal alternatives are summarised in Table II.3.3-4 for Year 2020 and Table II.3.3-5 for Year 2060 at the end of the chapter. The resulting O & M costs are calculated in the same tables - all figures are in thousand US dollars at 1990 prices. A sub-total for each alternative is presented and the combined total for the existing canal and new alternative calculated.

Table II.3.3-4 Year 2020 O & M Costs and Ratios

a) Year 2020 - Operation and Maintenance Cost Ratios

O & M Expense	Status Quo 1990	Existing Canal 2020	HRL 150,000	HRL 200,000	SLC 250,000 Locks	SLC 250,000 Gates	SLC 250,000 Lock/Gate
Transits	12,000	16,200	2,000	2,000	2,100	2,100	2,100
1	1.0	1.0	0.2	0.3	0.8	0.8	0.8
2	1.0	1.3	0.2	0.2	0.6	0.6	0.6
3	1.0	1.3	0.5	0.6	0.3	0.1	0.4
4	1.0	1.3	0.6	0.6	0.2	0.2	0.2
5	1.0	1.0	0.4	0.4	0.5	0.5	0.5
6	1.0	1.0	0.2	0.2	0.3	0.3	0.3
7	1.0	1.0	0.2	0.2	0.3	0.3	0.3
8	1.0	***	***	***	***	***	***
9	1.0	1.0	0.2	0.2	0.3	0.3	0.3
10	1.0	0.0	0.0	0.0	0.0	0.0	0.0
11	1.0	1.0	0.2	0.2	0.3	0.3	0.3

b) Year 2020 - Operation and Maintenance Costs ('000 US$ 1990)

1	29,482	29,500	5,900	8,800	23,600	23,600	23,600
2	78,912	102,600	15,800	15,800	47,300	47,300	47,300
3	48,294	62,800	24,100	29,000	14,500	4,800	19,300
4	23,074	30,000	13,800	13,800	4,600	4,600	4,600
5	19,829	19,800	7,900	7,900	9,900	9,900	9,900
6	29,854	29,900	6,000	6,000	9,000	9,000	9,000
7	77,697	77,700	15,500	15,500	23,300	23,300	23,300
8	23,393	25,300	6,200	6,700	9,300	8,700	9,600
9	12,699	12,700	2,500	2,500	3,800	3,800	3,800
10	15,113	0	0	0	0	0	0
11	30,853	30,900	6,200	6,200	9,300	9,300	9,300
Total	389,200	421,200	103,900	112,200	154,600	144,300	159,700
TOTAL	389,200	421,200	525,100	533,400	575,800	565,500	580,900

Expense	Description
1	Maintenance of channels, dams and spillways
2	Navigation service and control
3	Locks operation
4	General repair, engineering and maintenance services
5	Supply and transportation (incl. canal crossings)
6	Utilities
7	Administrative and general
8	Depreciation - *** Estimated as 6% of O&M costs
9	Fire and facility protection
10	Interest on interest-bearing investment
11	Other
Total	Incremental sub-total for each alternative
TOTAL	Total O & M cost for existing canal plus new alternative

Table II.3.3-5 Year 2060 O & M Costs and Ratios

a) Year 2060 - Operation and Maintenance Cost Ratios

O & M Expense	Status Quo 1990	Existing Canal 2060	HRL 150,000	HRL 200,000	SLC 250,000 Locks	SLC 250,000 Gates	SLC 250,000 Lock/Gate
Transits	12,000	20,000	4,600	4,600	5,100	5,100	5,100
1	1.0	1.0	0.3	0.4	0.8	0.8	0.8
2	1.0	1.6	0.4	0.4	0.8	0.8	0.8
3	1.0	1.6	0.7	0.8	0.4	0.2	0.5
4	1.0	1.6	0.8	0.8	0.4	0.4	0.4
5	1.0	1.0	0.8	0.8	0.9	0.9	0.9
6	1.0	1.0	0.3	0.3	0.4	0.4	0.4
7	1.0	1.0	0.3	0.3	0.4	0.4	0.4
8	1.0	***	***	***	***	***	***
9	1.0	1.0	0.3	0.3	0.4	0.4	0.4
10	1.0	0.0	0.0	0.0	0.0	0.0	0.0
11	1.0	1.0	0.3	0.3	0.4	0.4	0.4

b) Year 2060 - Operation and Maintenance Costs ('000 US$ 1990)

1	29,482	29,500	8,800	11,800	23,600	23,600	23,600
2	78,912	126,300	31,600	31,600	63,100	63,100	63,100
3	48,294	77,300	33,800	38,600	19,300	9,700	24,100
4	23,074	36,900	18,500	18,500	9,200	9,200	9,200
5	19,829	19,800	15,900	15,900	17,800	17,800	17,800
6	29,854	29,900	9,000	9,000	11,900	11,900	11,900
7	77,697	77,700	23,300	23,300	31,100	31,100	31,100
8	23,393	28,200	9,800	10,300	12,400	11,700	12,700
9	12,699	12,700	3,800	3,800	5,100	5,100	5,100
10	15,113	0	0	0	0	0	0
11	30,853	30,900	9,300	9,300	12,300	12,300	12,300
Total	389,200	469,200	163,800	172,100	205,800	195,500	210,900
TOTAL	389,200	469,200	633,000	641,300	675,000	664,700	680,100

Expense	Description
1	Maintenance of channels, dams and spillways
2	Navigation service and control
3	Locks operation
4	General repair, engineering and maintenance services
5	Supply and transportation (inc. canal crossings)
6	Utilities
7	Administrative and general
8	Depreciation - *** Estimated as 6% of O&M costs
9	Fire and facility protection
10	Interest on interest-bearing investment
11	Other
Total	Incremental sub-total for each alternative
TOTAL	Total O & M cost for existing canal plus new alternative

APPENDIX

Appendix 1

Terms of Reference

(Revised on August 11, 1992)

Terms of Reference

Engineering and Cost Estimates - Phase 2

1. Introduction

These Terms of Reference (TOR) describe the technical requirements for Phase 2 Engineering and Cost Estimates (ECE) for the following Conceptual Alternatives which survived the prescreening process by applying the prescreening criteria developed for this purpose during Phase 1:

- High Rise Lock Canal
- Sea Level Canal, Route 10

Complete features of the above routes including engineering, preliminary design, construction planning, and cost requirements will be included in this Phase 2 - TOR.

The High Rise Lock Canal represents a new study case derived from the results of Phase 1 work and generally characterized as follows:

Design Ship Size 200,000 DWT.

Alignment Route I from Phase 1 with minor refinements.

Navigation Channel cross section to be 2 lanes throughout the Canal.

Third set of larger new locks having 1 lane with 2 lifts, with an option for a second additional future lane.

Gatun Lake Level of 85 feet and 90 feet.

The Sea Level Canal is to be a one lane canal with double lane approaches closely represented by study case S-3 from Phase 1 work operating in combination with the Status Quo with cases (a) of Pacific entrance locks, (b) a pair of tidal gates and (c) a combination of Pacific entrance locks and one tidal gate towards the Atlantic entrance.

2. Objectives

The purpose of this Phase 2 Study is to provide CAS with complete engineering and cost estimate information, data, analysis, and conclusions to enable CAS to select and recommend to the Board the best and most viable alternative to the present Panama Canal. The Phase 2 Report should include as a minimum the following:

- Engineering and preliminary design of the previously described two Conceptual Alternatives and their variations as well as the appurtenant facilities as complete as possible to provide the basis to make appropriate recommendations.
- Estimated Costs for construction of the two Conceptual Alternatives including variations using unit cost/prices and/or lump sum prices based on realistic evaluations of present and improved technology, economy of scale, potential innovations, anticipated improvement in excavation and hauling equipment, and a well planned construction sequence and schedule. Estimated costs are to be based upon a comparable level of detail for both alternatives. Where this can not be reasonably achieved, the uncertainty in cost estimates should be noted.

3. Scope of Services

The scope of services for the Phase 2 Engineering and Cost Estimate Study may be summarized in Terms of four major activities: Engineering, Preliminary Design, Construction Planning, and Cost Estimating.

3.1 Engineering:

The Phase 2 Engineering specific scope of work will include, but not limited to the following:

1. <u>Base Map Preparation</u>

   Preparation of a topographic base map to be used consistently throughout the Study will use the basic 1:50,000 scale quadrangle maps with 20 meter contour interval from the Phase 1 Study. The quadrangles may be reduced 50 percent in

linear dimension and assembled to form an entire coverage of the watershed boundary area contributing surface runoff to both Gatun lake and the proposed Route 10 Sea Level Canal Alignment.

2. Canal Alignment and Profile Drawings

Prepare Canal alignment and profile drawings using Route I as defined in YEC Phase 1 Final Report for the High Rise Lock Canal with minor refinement in Gatun Lake, and Route 10 Sea Level Canal with minor refinement to improve navigation.

3. Study of Gatun Lake level

The design and cost estimate for the High Rise Lock Canal developed in Phase 1, where Gatun Lake level is raised to El. 90 ft., should be further evaluated to include all cost items in order to provide the basis for a comparison with an 85 ft. level for Gatun Lake level for a High Rise Lock Canal.

4. Establish Canal and Lock Dimensions

Canal and Lock dimensions are to be refined for the Phase 2 Study.

a. Design ship size dimensions for each of the following DWT Ship Displacements: 150, 200, and 250 thousand are to be delivered to ECE contractor at Inception Report meeting.

b. For a 200,000 DWT High Rise Canal System and a 250,000 DWT Sea Level Canal System lock chamber dimensions are to be given to ECE contractor one month after Inception Report meeting.

c. For the Sea Level case the tidal gates should provide the same channel clearance as the navigation channel. The gates (navigation and emergency gates) should be in a revetted transition section of the channel.

d. Dimensions for High Rise Locks are to provide for single lockages for each 200,000 DWT ship. Dimensions for Sea Level Locks will consider single lockages for a 250,000 DWT ship, considering phased construction, three parallel lock chambers are designed.

e. The basic canal navigation channel section is to be sized using PIANC ship to channel ratios to be confirmed by CAS.

f. Determine Canal cross-sections, locations, and cross-sections at bends, minimum bend-to-bend distances of waterways using information included in ICOS (1970) report and PIANC guidelines as a basis.

g. The Study should include cross-checks between items developed above for consistency and adjust data as required to satisfy this consistency. Reconcile canal cross-section with PIANC discussion identified in the Phase 1 Report.

5. Water Management Strategy

This Phase 2 Study should include refinement and optimization of the water balance studies to support the transit demands using revised lock sizes to determine the total volume of water required.

a. Develop a water saving plan, recycling requirements, pumping stations, dams, etc. Consideration should be given to allowing storage for currently spilled (not leakage) water which will provide for additional estimated lockage/transits, and/or a fresh water barrier in the Sea Level Canal Alternative. In addition, the Water Simulation Model should be repeated to remove assumptions made in Phase 1 (no limitation on dam capacity, cumulative spill taken in storage,...,etc.).

b. Provide plans for the development of new dams to the extent needed and practical at the following sites: (1) Ciri Grande (2) Trinidad (3) Indio, and (4) Gatun as identified in the Phase 1 report. To the extent added water requirements are still needed, develop new required pumping capacity and pump systems to meet any added water demands.

c. Study for recycling ponds should be performed to minimize water loss.

d. For the fourth set of locks, incorporate water transference between chambers.

6. Seismicity Analysis of Panama

Conduct a complete but brief analysis of seismic events in the Republic of Panama based on both historical and instrumental records of earthquakes, and select an appropriate design earthquake and its seismic coefficient to be used in

the design. Provide this information to CAS before use in design in the study for concurrence prior to its use.

7. Geologic Information and Subsurface conditions

Review all information used as a basis for development of the Geologic Profiles illustrated on Figures 3.3.11 (1) for Route I and (2) for Route 10. Consider the need for any additional study or field work to confirm the large difference in amount of weathered rock for Route 10 compared with hard rock for Route I. Make appropriate recommendations for any study or investigation, and associated costs, needed to confirm these basic interpretations.

8. Canal Bank Slopes

Review Canal bank slope type(s) used in Phase 1 Engineering and Cost Estimate Report. Conduct a sensitivity analysis to determine the impact of conservatively selected bank slopes on the estimate. Identify risks associated with the revised selection of bank slope criteria, including monitoring requirements, if any.

9. Excavated Material Placement Plan

A Phase 2 Placement Plan for Excavated Materials should approximate the actual locations and haul distances to be used for conveying and placing excavated materials from both the dry and underwater excavation. This study must consider the following elements: clearing, stockpiling and re-using topsoil, temporary and permanent placement of spoil, consolidation of spoilage versus pumping rate, reforestation, revegetation, and restoration of the newly formed land surfaces. Both temporary and permanent soil erosion measures must be included in the plan.

10. Transportation Plan

Phase 2 should include a study to determine the appropriate level of transportation needs including roads, highways, and number of bridge crossings which will be required to provide vehicular and pedestrian access to both sides of the Canal. Both temporary roads for construction as well as permanent roads to service the canal and local residents should be considered. A plan to provide adequate permanent road access to the canal for safety inspection and maintenance as well as access to local residents affected by construction of the canal alternatives must

be developed.

11. Tidal Locks and Tidal Gates

A combination of tidal locks and/or tidal gates are to be studied for use in restricting tidal currents as well as helping to create a fresh water barrier within the Route 10 Sea Level Canal.

Perform a study of the tidal gates and/or tidal locks for the Sea Level Canal and determine the type and dimensions of each considering the requirements for the design ship size.

Conduct an engineering analysis of the tidal gates to verify stability and adequacy considering, as a minimum:

a. Deflection, twist, and deformation due to height to length ratio of these gates.

b. Check gate structural stability including its supports using varying hydrostatic pressures and develop design criteria for the most critical load combination.

c. Check gate mechanical stability especially during gate closure.

Should the information listed above be available from gate manufacturers, YEC should request a copy of their analysis, or correspondence be provided in an appendix. Incorporate emergency gates for the Route 10 Sea Level Canal.

A Study is to be made concerning the means of fabrication, construction, transportation, and installation of lock leafs and gates. Gate types to be selected for use in Phase 2 are those resulting from optimization studies performed during Phase 1.

Experience gained from the three tidal ship locks operating in Belgium are to be used as a basis for design and cost estimating for the Pacific Locks variation of the Route 10/Status Quo option.

12. Freshwater Barrier

For the sea level canal option, as the result of other component study, a freshwater barrier may be required somewhere in the Canal; therefore, Phase 2 should include as a minimum the following studies:

a. Review the engineering feasibility of a freshwater barrier concept using natural runoff flowing into the Canal.

b. Determine physical configuration to realize a concept for a freshwater barrier. A conceptual design of the freshwater barrier is to be given to ECE contractor during the Inception Report discussion.

c. Determine the compatibility of a freshwater barrier with tidal gates and/or tidal locks and provide recommendations.

d. Develop the design of such a recommended barrier concept to allow for the appropriate level cost estimate.

13. Environmental Requirements

This Phase 2 Study is to incorporate environmental requirements to the extent practicable. Ecologically sensitive areas are being identified by other component studies and will be provided to YEC during the Inception Report Review as a series of overly maps. Some of these key requirements are identified below:

a. Provide approximate locations of all land-based and marine-based spoil disposal areas. Selection of these locations should avoid and be outside ecologically sensitive areas.

b. Provide locations of all marine structures and avoid ecologically sensitive marine areas to the extent possible.

c. A study to determine the feasibility of constructing a freshwater barrier in the sea-level canal (item 13) should not inhibit the proposal of other concepts to control marine species transport.

d. This Study should avoid location of temporary construction facilities such as labor camps, housing, batch plants, processing plants, etc., in ecologically sensitive areas.

14. Excavation Studies

a. The volume of wet excavation calculated for Route 10, Sea Level Canal for Phase 1 is to be re-evaluated. It is not conceivable that the ratio of wet excavation to total excavation is in the range of 3 to 8 percent as reported in Phase 1. Dewatering to allow excavation in the dry is very difficult if

not nearly impossible. It would require massive pumping stations, energy, and point discharge locations requiring some environmental mitigation. This has to be accounted for in the cost estimates.

b. Third set of locks' excavation which had been already completed should be accounted for.

15. Analysis to Support Feasibility Cost Estimates

All engineering analysis conducted for Phase 1, and needed to directly confirm Phase 2 work are to be refined to a level consistent with the new Phase 2 Engineering Studies in order to provide a suitable basis for the Phase 2 Cost Estimate. An example is the need for design of additional dams and reservoirs as well as a revised size of pumping recirculation system.

16. Operation and Maintenance Costs

The degree of accuracy of Operating and Maintenance Cost is very important for Phase 2 Work. Selection of a lock canal or sea-level canal may very well depend upon these cost figures. PCC standard breakdown of O&M costs, included in their annual reports, should be used as a base plus additional operating and maintenance costs required for either High Rise Lock or Route 10 Sea Level Canal should be added. Such added items, which are to be separately listed and costed, may include, for example, O&M costs for pumping facilities.

3.2 Preliminary Design:

Phase 2 preliminary design work should be developed to provide sufficient and complete information beyond that developed for Phase 1 in order to provide a strong basis for selection of the most promising and viable single alternative for further development. The design effort developed for Phase 1 should be upgraded, commensurate with a new design effort to be conducted specifically for Phase 2, to provide consistency. Design information should include but not be limited to the following:

1. Blasting Criteria Development

Formulate blasting criteria and specification requirements to yield aggregate sizes suitable for use as various construction materials, i.e., concrete aggregate, rock fill

dams, erosion control, breakwater, and jetties.

2. Excavated Material Placement Plan

In addition to locating land-based and marine-based disposal sites, this phase should also develop the design of disposal sites by providing design criteria to include site clearing & grabbing, provision for appropriate drainage, dikes, placement, compaction and consolidation, restoration, and permanent closure. Examples of such design effort may be found in U.S. Soil Conservation Service manuals and/or equivalent Panamanian and Japanese guidelines/standards.

3. Seismic Design of Structures

Phase 2 important hydraulic structures should be designed for an appropriate level of seismic coefficient recommended by the engineering study. The general criteria to be followed in the design is:

a. For safety related structures and systems, where potential failure caused by earthquakes may occur, the design earthquake full coefficient should be used. Safety related structures may be defined as those where failure may cause loss of life, failure of fresh water barrier or complete loss of navigation in the Canal.

b. For important structures and systems where interruption of Canal operation is temporary or will not cause loss of life, one-half the design earthquake load may be used.

c. For all other structures where their failure will not cause danger to public safety nor will it effect safe navigation in the Canal, no seismic design would be required.

4. High Rise Locks Design

For the high-rise canal alternative, a third set of new locks are to be designed as a single lane even though the excavation will be completed for a two lane canal. The single lane chamber walls including culverts should be designed to easily incorporate requirements for construction of a second lane of locks and gates in the future. The second lane excavation at the new larger locks should be closed by construction of a gravity type cofferdam, earth or rock fill dam, or other type

structure until future traffic demand requires construction of a second lane of larger locks. In addition, a second set of new locks will be designed and their cost estimated separately to illustrate future expansion requirements.

5. Supplementary Systems

This Phase 2 Study should include the design of supplementary systems (i.e. partial concepts referred to in Phase 1 Work) including improved navigation systems, improved lock structures, and lockage guidance systems.

6. Interface with Existing Canal Operation

The design should include details to articulate both temporary and permanent structural additions from the existing canal structures and systems. Special attention should be given to the interference with, and potential impact on, operation of the present canal.

7. Corrosion Protection System

All hydraulic structures and especially lock gates and tidal gates should include appropriate corrosion protection systems. It is required that the design should incorporate PCC maintenance experience and findings. This design feature should also be applied to pumps and pumping stations. In addition to corrosion protection, some hydraulic structures and especially pumps are sensitive to, and could be damaged by, electric storms; therefore, the design should include special features to minimize this damage.

8. Gate Leaf Design

The design of gate leafs including source of material, specification, and place of fabrication should provide for an economical design.

9. Water Optimization Provisions

Design of the water saving plan should be based on demand capacity, and revised lock sizes and should include optimization of the recycling ponds including multiple level recycling ponds in addition to one level.

10. Fresh Water Barrier - Sea Level Canal

Based on engineering feasibility results, provide the designs appropriate for a fresh water barrier as part of the sea level canal system. The scheme should include

safety features to allow for potential malfunction, interruption, and failure of the primary system.

11. Refinement of Phase 1 Designs

The designs developed in phase 1 for design ships of 150 thousands DWT for high rise lock canal are to be refined to the same level of design used for 200 thousand DWT design ships added for this Phase 2 Study.

12. Design for Gatun Lake Level Structure Modification

Phase 2 design should include all of the design and cost items omitted in phase 1 for the high rise lock canal option where Gatun lake is at El 90 ft. The design of these items should be sufficiently developed to allow for an appropriate level of costing.

13. Anchorage, Mooring, and Breakwaters

Phase 2 design effort should develop port, anchorage, mooring, and breakwater designs for both alternatives to be included in the cost estimate.

3.3 Construction Planning:

Phase 2 construction plans, procedures, and specifications should be developed in sufficient detail to allow for an appropriate level of cost estimate to be used to select the most viable canal alternative. Development of the construction plan for Phase 2 should be consistent for all alternatives studied. The Construction Planning effort of the Phase 1 Study should be upgraded commensurate with the level required for Phase 2 Study. The effort requiring development during Phase 2 should include but not be limited to the following:

1. Placement of Excavated Materials

Prepare construction plans and procedures to develop conservation, spoil placement, and restoration of the designated spoil disposal areas. Provide guidelines and criteria to implement acceptable restoration techniques including but not limited to;

a. Clearing and grubbing

b. Construction access

c. Placement and compaction

d. Surface runoff and drainage control

e. Erosion protection measures

f. Embankments for containment of dredged material

g. Restoration to include landscape, topography, restore original vegetative cover, and other requirements compatible with local and regional planning, if any

h. Final closure (cover) of the filled areas

2. Equipment Use Plan

Study and develop guidelines concerning the use of compatible construction equipment. Examples are balancing excavating equipment with hauling equipment and the possibility of using conveyors, rails, and off-road haul equipment. The most economically feasible balance should be adopted and used for cost estimation.

3. Lay Down Areas

Design construction lay down areas to consider the location of processing plants. Examples include the stockpiling of aggregate near where it is intended to be used, concrete aggregates near concrete batch plant, armor size stones near jetties and breakwater construction,..., etc. These plans will allow for minimizing construction material handling to preferably one step from supply source to the demand location.

4. Interface with Existing Canal

Construction planning, procedures, and schedules should be developed in sufficient detail to articulate new construction required for the canal alternative with the existing canal, structures, and systems. Temporary and permanent structures should be identified and procedures and schedules should be developed to allow for minimum interference with the existing canal navigation, operation, and maintenance. Where interference cannot be avoided, it should be identified and be included in the cost estimate.

5. Foundation Preparation

Provide details for foundation preparation underwater, especially for barrier dams. In addition, procedures should be developed addressing placement, compaction,..., etc. of materials in water for the construction of hydraulic structures.

6. Dredging & Navigation Planning

Develop construction procedures to allow for minimum interference between dredging equipment used in the new construction with navigation in the existing canal. Efficiency of construction should be balanced with navigation, and many years of PCC experience in this regard should be used. An estimate of navigation interruption affecting capacity due to construction, and reduced construction efficiency due to navigation should be estimated and included in the cost estimate.

7. Constructability Reviews

Constructability of structures and systems should be reviewed by Construction Specialists to provide input during the design stage and prior to the cost estimation stage.

8. Fabrication of Gates

A study should be conducted by Construction Specialists to determine the possibility of fabrication and construction of gate leafs in Panama or the immediate region in order to provide for the most economical project. Procedures should be developed concerning transportation, storage, and placement of these structures.

9. Excavation Methods

A study should be undertaken to refine the Phase 1 excavation methods and to maximize dry excavation and reduce dredging. Dewatering should be kept to a minimum. This is particularly important, in the sea level canal alternative.

10. Cofferdams

Removal of temporary cofferdams, constructed to expedite third lock excavation and construction for the high rise alternative, should not disturb the foundations and undermine stability of the existing and new structures. Construction methods and procedures should be developed to avoid this disturbance.

11. Fresh Water Barrier

Constructability of a fresh water barrier should be determined prior to cost estimation. Fresh water barrier will be discussed during the Inception Report meeting.

12. Marine Construction

Because marine construction is usually completed at a much higher cost than land-based construction, a construction study should be conducted to provide the proper economical balance.

13. Construction Schedule

Phase 2 should include a detailed construction schedule for estimation of project cash flow.

3.4 Feasibility Cost Estimate:

The cost estimate developed for phase 1 should be upgraded to provide consistent numbers meaningful for comparison and decision. The Cost estimate should include but not be limited to the following:

1. Culebra Cut Widening

Phase 2 High Rise costs are to exclude any costs or estimates associated with the Culebra Cut widening. The basis for the costs is to be the "widened" Culebra Cut.

2. Infrastructure Development

Infrastructure Costs should be provided for the Phase 2 estimate. These Costs should include bridges, highways, railroad, canal access and maintenance roads, pedestrian traffic...etc.

3. Cement

The unit cost of cement used in Phase 1, should be revised to include economy of scale based on bulk purchase.

4. Aggregates

The unit price of aggregates for use in concrete, erosion, breakwater, jetties, and others should be estimated based on refinement of blast design conducted by an engineering and design study and refinement of material handling, processing and

transportation.

5. High Rise Lock

The Construction Cost estimate for the high rise lock canal should be based on a single lock system as indicated in the engineering and design sections of this TOR. An incremental added cost item for the future construction of a second lane of large locks should be developed and identified. By-pass channel construction costs should be identified separately.

6. Overtime Rates

Cost estimates should incorporate adjustments to allow for overtime rates and shift differentials.

7. Demolition of Structures

Demolition Costs for the removal or modification of existing structures should be costed based on actual work to be done.

8. Operating and Maintenance of Canals

The cost for Operating and maintenance are very important elements in the evaluation of alternatives in Phase 2. Therefore, a detailed itemized breakdown should be provided. The itemized breakdown of O & M used by the PCC in their annual reports may be used as a guide. Special attention should be paid to:

a. O & M costs associated with monitoring, providing safety inspection, and potential remediation of excavation slopes for both Canal alternatives studies in Phase 2.

b. O & M costs associated with pumping stations, additional dams, recycling, and surge ponds for the high rise canal alternatives and, if necessary, for the Sea Level Canal.

c. O & M costs associated with safety of the barrier dams including inspection.

d. O & M costs associated with tidal gates or tidal locks for the sea level canal.

e. O & M costs associated with maintenance of shoreline structures.

f. O & M costs associated with moorage, anchorage, and dredging

requirements, hydrographic/bathymetric surveys and other work required.

g. O & M costs associated with infrastructure development including port facilities.

h. O & M costs associated with navigation aids, equipment, safety systems, mules, and tug fleet.

i. O & M costs should include "preventive maintenance" costs.

The above items (a) through (i) are a listing of examples which may be considered in the O&M Costs and is not complete. It should be noted that a single lane Route 10 Sea Level Canal will be operated in conjunction with the Status Quo.

9. Real Estate Costs

The Phase 2 cost estimate should include a realistic allowance for real estate needed for expanding Gatun Lake to a 90 foot elevation, the area required for developing spoil disposal sites, and right-of-way and separately the disposal areas for the Route-10 sea level canal.

Other potential real estate allowance includes access roads, railroad, new dams, locks, construction laydown areas including batch and processing plants. Each of the above reasons for real estate costs are to be itemized separately.

10. Third Set of Locks

Provide a cost breakdown for the third set of locks by excavation, structures, mechanical system, and added provision to allow for future expansion including damming the second lane canal excavation.

11. Dredging

Provide a breakdown of costs associated with marine dredging and dredge disposal for Canal approaches separate from actual Canal dredging; and dredging required to construct foundations for barrier dams, moorage, anchorage, jetties and breakwater.

12. Navigation

Cost estimates should include for the high rise;

a. Cost associated with interference of construction with navigation, and

b. Cost associated with construction inefficiency due to providing a priority

for navigation.

13. Relocation of Population

Estimated costs should include allowance for compensation and relocation of displaced population including the development of new infrastructure to service relocated communities.

14. Costs for Giant Scale Excavations

All unit costs for items used in Phase 1 should be reviewed and revised prior to estimating phase 2. Excavation, handling and disposal, in particular, should be subjected to rigorous scrutiny due to the high sensitivity of the total construction Cost to excavation Costs.

15. Other Cost Criteria

All other cost criteria used in Phase 1, for example, taxes, custom, and escalation, remain unchanged.

16. Base Year Estimate

1990 should be considered as the base year for all phase 2 estimates (similar to Phase 1).

17. Supply Sources

It is preferable to solicit the cost of major and large mechanical items directly from Vendors and Suppliers, where possible.

18. Construction Cost Schedule

An annual cost should be developed for each alternative using the construction schedule as a basis.

19. Total Construction Costs

Total construction costs shall include all infrastructure, housing, health facilities, camps, security, consumable, sanitary facilities, transportation allowance, and other items required to support construction. It should be noted that due to the planned construction schedule, some of these facilities may not be considered temporary.

20. Provisions for Climate Conditions

All estimates should incorporate the effects of local Panamanian climate conditions upon the construction schedule and working environment.

4.0 Reports

All reports published for Phase 1 will be required in Phase 2. These reports included as a minimum; Inception Report, Interim Report, Final Report, and Monthly Reports. All reports are to be submitted in hard copy and on computer disk formatted in DOS. The sources of all data acquired for this study are to be included in YEC reports.

All assumptions must be clearly stated; footnotes and citations must be identified in the text; all cases where comparability is compromised by insufficient information must be clearly identified in the text.

The reports will be submitted five(5) days in advance of any joint meetings between the client and the contractor.

5.0 Staffing

Appropriate levels of professional staff are required in order to provide all of the engineering, design, construction, and cost estimates included in this TOR within the allotted schedule. As a minimum, similar type professionals used for Phase 1 work as expected to continue for Phase 2 and additional staff will be needed to complete tasks identified in this TOR such as Blasting Specialist, Construction Specialists,..etc.

6.0 Schedule

As negotiated in the Contract for all work, reports, and meetings in Panama, USA, and Japan.

Appendix 2

Basic Engineering Criteria

APPENDIX 2 - BASIC ENGINEERING CRITERIA

This Appendix presents further detailed information relevant to some of the basic engineering criteria that were defined in Part II, Section 1, Chapter 1. In particular, additional information is presented regarding the canal cross section dimensions, anchorage areas, emergency tie-up stations, moorings, and transit and flush numbers.

2.1 Canal Cross-Section and Bank Alignment

2.1.1 Canal Cross-sections on a Curved Alignment

The previous paragraphs relate to the cross-sections of straight canals. Some widening may be necessary on curves of small radius. Many empirical formulae have been suggested but all relate to single lane canals. More recently radar tracking has been used in Germany to measure actual navigation lane widths but the method has been restricted to river and estuary channels. Lane widths measured have exceeded those proposed above for straight sections by up to 30% (R. Elbe in ref. (1) 2.1.2.3.4).

Increase in navigation lane width on curves was found to be very dependent upon curvature with increases of 1.5 to 2B at a radius of 15L (L = length of ship).

In contrast, the Suez Canal Authority uses an approach described by Ammar (2). This is based on the observation that safe navigation is possible if the ships bow is always a ship's length or more from the outer bank measured along an extension of the ship's centerline.

It is assumed that the ship turns about a point 3/4 of the ship's length behind the bow. In deep water the turning point is generally 1/3 L behind the bow. This distance increases to 0.5L in shallow water but could be moved further aft when the bank effect is included. The

geometry of the situation leads to the formula:

$$r/B = 3.063\ (L/B)^2\ B/W - W/(4B)$$

(where; r=radius of canal centerline, B=beam, w=required navigation lane width, and L=LBP is the length between ship perpendiculars). Considering a typical L/B ratio of 6.35 leads to the following:

Table A.2-1 Minimum Lane Widths on Curves

r/L	r/B	W/B
5	31.75	3.78
5.4	34.41	3.50
10	63.50	1.93
15	95.25	1.29

As the navigation width provided will be 3.5B, no widening would be needed unless the canals centerline radius were to be less than 5.5L. For a radius of 5L the widening would be 0.3B.

Application of this method to a two-lane canal would require modification to allow for the greater difficulty in assessing a ship's position when steering an eccentric course and possibly, for the different direction of the bank effect on a ship nearer to the inside bank. A widening of twice the single lane allowance plus 50% is proposed as a preliminary value. This would amount to 0.9B.

Noteworthy among the observations reported in (1) is the influence of an accurate knowledge of a ship's position on the width of navigation lane required. The accuracies being considered (from ±5m to ±20m) are difficult to achieve visually using leading marks on straight channels and much more difficult if the canal is curved. A portable electronic aid which can be taken aboard each vessel by the pilot and which gives the distance from the desired course to within ±1m would help to minimize errors

in navigation. Such an aid is already in use in Milford Haven, UK.

2.1.2 Bank Alignment

In a single lane canal the bank on the outside of the curve has the greatest influence upon a ship passing along the canal. It is important, therefore, to construct and maintain that bank as a smooth curve to avoid sudden changes in bank effect. The inner bank is relatively unimportant and changes in canal width should be made by adjusting the line of the inner bank. Transitions between widths of section should be implemented gradually. Ref (1) recommends that the change should take place over a distance equal to 10 times the increase in width.

The same rate of change in width should be applied where canal widths change, for example at the approaches to locks.

With two-lane operation the ship on the inside of a bend will be most influenced by the inside bank. That bank also should then be formed in a smooth curve. It follows that if widening is required, it should be arranged to take place in the straight part of the canal before the tangent point is reached.

Curves should preferably be separated by a straight section of canal at least 10 times the length of the largest vessel and where large course changes are required, a single curve should be adopted rather than a sequence of smaller curves at close intervals. Ref (1) 2.2.3.4 to 2.2.3.6.

2.1.3 Sea Approaches

Ships using the sea approaches will be subject to the effects of wind, tide, currents and waves and larger channel sections are normally adopted. For the main report, a canal width of 7.0B has been adopted throughout, including the canal approaches, and a sensitivity

analysis has been carried out for changes in canal width. It is recommended that consideration should be given to adopting the PIANC recommendations (1) for the sea approaches, as described below.

Channels depths should be greater to allow for a ship's response to waves. Diagrams are available to estimate vertical ship movements in deep water (3). In shallow water the small under keel clearance is known to increase the natural period of response as well as the added mass and hydraulic damping but the combined effect is not amenable to calculation. For large vessels, vertical movement will be small for waves with period shorter than 10 seconds. The wave climate in the Caribbean is mild but waves of 2 to 3 m height and 11-12 seconds period occur with a frequency of 0.1% (8). Under those conditions the heave of the design vessels would be less than the wave height; a figure of 1 m has been assumed. The channel depth below Chart Datum would then be the sum of:

Table A.2-2 Depths in Atlantic Approaches

	250,000 DWT	200,000 DWT	150,000 DWT
Design ship draught	21	19	18
Heave in waves	1	1	1
Squat at 10 knots	0.6	0.6	0.6
Under keel clearance	1.0	1.0	1.0
Siltation allowance	1.5	1.5	1.5
	25.1	23.1	22.1

Little allowance for heave need be made in the Pacific approaches. Depths would be 1m less than on the Atlantic side.

Widths would be 12.5B for two-way traffic as recommended by PIANC (1), so that the design channel dimensions would be as in Table A.2-3.

Table A.2-3 Dimensions of Approaches

Design ship size	250,000 DWT	200,000 DWT	150,000 DWT
Depth	25m	23m	22m
Bottom width	675m	625m	575m

References for Appendix 2.1

(1) Optimal layout and dimensions for the adjustment to large ships of maritime fairways in shallow seas, sea straits and maritime waterways. Supplement to Bulletin No. 35, Permanent International Association of Navigation Congress (PIANC) 1980.

(2) Ammar, A.A., et al: 'Design of Navigation Canal Cross-Section and Alignment'. PIANC 22nd Int. Nav. Congress, Section 2, Subject 3, Paris 1969.

(3) Maritime Structures. British Standard Code of Practice BS 6349, Part I: 1984.

2.2 Anchorages and Emergency Tie-Up Stations

2.2.1 Anchorages

Ships awaiting passage require safe anchorage close to the canal entrance and in sufficient, but not excessive, depths of water. The anchorage area will be determined by the number of vessels to be accommodated and the swinging radius of each ship about its anchor. The swinging radius will increase with the depth of water.

Predicted traffic includes vessels from less than 10,000 DWT to 250,000 DWT with draughts varying from 8m to 21m. These have been grouped into three categories based on overall length for the purposes of estimating the area required for anchorage.

Table A.2-4 Vessel Grouping for Anchorage

Group	Max Size DWT	Max Length (m)	Max Draught (m)	Min Water Depth (m)
A	40,000	200	11.5	14.5
B	100,000	300	16.5	21.0
C	250,000	330	21.0	26.5

Various grades of wrought iron or steel are allowed for ship's anchor chains but the diameter required for a particular ship will reduce as the strength of material is increased.

Conversely, as it is essential that the pull on an anchor is horizontal, the scope of chain (length of chain from anchor to ship) must be greater for stronger and lighter chain. The Admiralty Manual of Seamanship (Her Majesty's Stationery Office, London) recommends a scope of:

$50\sqrt{D}$ for steel chain
$70\sqrt{D}$ for H.T. steel chain
where D is the depth in feet

The scope required for a range of water depths for both grades of chain is set out in Table A.2-5.

Table A.2-5 Scope of Chain at Different Water Depths

Depth (m)	15	21	24	27.5	29
D (ft)	49.2	68.9	78.7	90.2	95.1
$\sqrt{D}$	7.0	8.3	8.87	9.5	9.75
$50\sqrt{D}$	351 (7.1)	415(6.0)	443(5.6)	475(5.3)	488(5.1)
$70\sqrt{D}$	491(10.0)	580(8.4)	621(7.9)	665(7.4)	682(7.2)

Figures in ( ) = ratio of scope to depth of water.

The quality of chain used varies with the size of ship. The use of high-tensile chain has a number of benefits including lower price and the need for smaller windlasses and ordinary quality is now seldom used for ships greater than 10,000 DWT. The recommended scopes given in Table A.2-5 assume the use of high tensile chain by all groups. The swinging radii listed in this table are equal to the scope plus the length overall of the largest ship in the group. A spacing between anchoring position of 2.5 times the swinging radius has been assumed. This will allow for errors in positioning the anchor and differences in the amount of chain which is veered.

Table A.2-6 Spacing of Anchoring Points

Group		A (m)	B (m)	C (m)
Draft		11.5	16.5	21
Min depth		15	21	27
Max depth	(Pacific)	20	26	32
Length		200	300	330
Radius	(Atlantic)	351	479	532
Radius	(Pacific)	373	497	549
Spacing	(Atlantic)	877	1196	1328
Spacing	(Pacific)	932	1243	1372

2.2.2 Emergency Tie-Up Stations

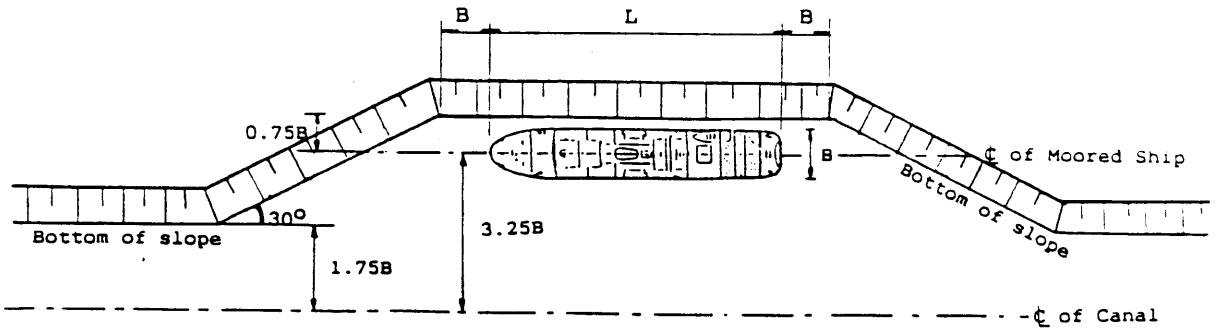
Provision must be made to enable a disabled ship to be removed from the navigation channel to minimize the disruption which it would otherwise cause to canal traffic.

Passing ships induce forces on a nearby moored vessel which may be large enough to break moorings if the separation of the vessels is small. An emergency tie-up station needs to be large enough to accommodate the design vessel without excessive capital cost yet far enough from the nearest navigation lane for mooring

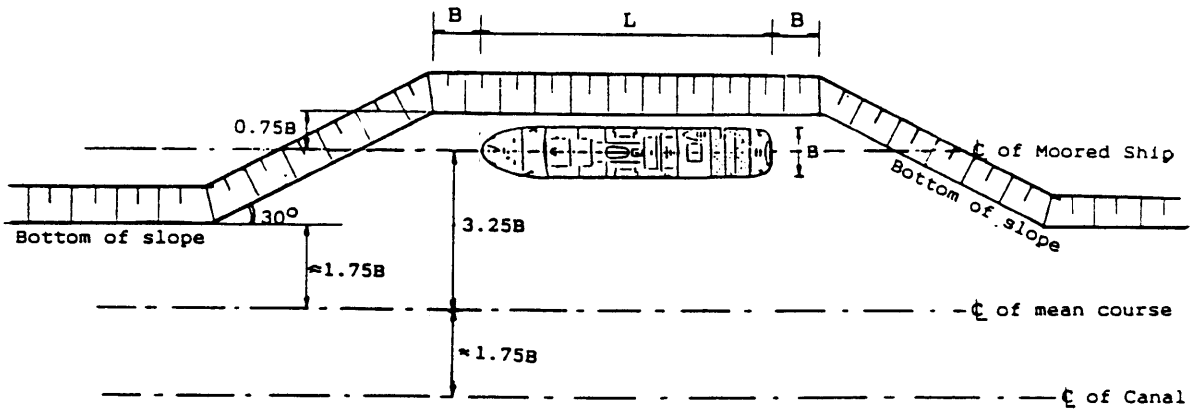
forces to remain within the capacity of the design ship's normal equipment.

The length of a tie-up station should be comparable with that provided at a quay used by the design vessel as mooring of a disabled vessel will be under the control of tugs. A length of (LOA +2B) is shown in Fig. A.2-1. The depth of water in the tie-up station is the same as in the canal.

Measurement in the field of the forces imposed on a moored ship by the passage alongside of another of comparable size has not been successful. Difficulties in the measurement of forces in the hawsers, and ill-defined friction between quay and ship are among the problems which were not overcome in the few known attempts at full scale experiments. Some model experiments have been made and use has been made of a report by NMI (1) in assessing the desirable separation of a moored disabled ship and a second design vessel passing in the canal. Tensions were measured in a spring and a breasting rope and were compared with a numerical model to predict these forces. Reasonable agreement was found. The ships modeled represented vessels of 250,000 and 200,000 DWT, the smaller being used as the passing vessel. Separation of the ships centerlines was varied from 1.9B to 2.6B and the speed of the passing ships varied from 2 to 4 knots.



(a) Single Lane Canal



(b) Two Lane Canal

Fig. A.2-1 Layout of Emergency Tie-Up Stations

Some extrapolation has been necessary to apply the results to the proposed tie-up stations and the assumption of the numerical model that force is proportional to the square of the speed has been used to estimate rope forces. The observed effect of separation is approximately linear, forces being inversely proportional to the separation of the ship's centerlines.

With these assumptions it is estimated that the maximum rope force induced by a passing ship would be about 50 tonnes if the moving ship is in the center of its navigation lane and the moored ship is as shown in Fig. A.2-1. This force might increase to about 190 tonnes were the passing ship to be at the nearest point in its navigation lane to the moored ship. For 95% of its time a ship would be within a lane width not exceeding 2.6B and would induce rope forces of about 130 tonnes if it were to pass the tie-up station when at the extremity of this narrower lane. The combined probability that a design ship would pass a lay-by while at its greatest deviation from its course will be low and the mooring structure should be designed for the forces induced in the ropes when the passing ship is on the centerline of its navigation lane.

Tensions in breasting ropes were measured in one rope of a pair so that breasting structures shuld be designed for twice the loads given above. Spring forces were estimated at about 70% of the breasting rope tension but should not be doubled as only one rope was used.

1. Simulation of the behaviour of a moored ship when passed by other ships. I W Dand, British National Maritime Institute, RIII, August 1981.

2.3 Calculation of Ship Transits and Lock Flushes

The annual numbers of ship transits and lock flushes are of particular importance in the Phase 2 Study since they define the lock water usage used in the water management analysis and the lock operation frequencies used in the lock gates fatigue design.

2.3.1 Calculation and Assignment of Ship Transits

This analysis has been based on the following sources of data on the number of ship transits :

- Vessel transit data and ship size distribution from the Phase 1 Final Report of the CTP Study (WEFA). The Phase 1 Report provided transit and ship data for canals designed for 150,000 and 250,000 DWT vessels.

- TAMS projected transit data and ship size distribution for a canal designed for 200,000 DWT maximum size vessels. (TAMS FAX dated Nov. 20 1992, received by YEC on Nov.25 1992).

  The information from both the above sources has been collated and summarized on Table A.2-7(a) and (b) for the years 2020 and 2060 respectively.

- CAS Table of assigned ship transits, i.e. the number of ships likely to use either the existing locks or the new alternative for 150,000 and 250,000 DWT High Rise Lock Canals and for 250,000 DWT Sea Level Canals (Refer to Table A.2-8. This Table is dated Nov.19 1992 and was received by the ECE Contractor on Nov.25 1992). For the 250,000 DWT Sea Level Canal both Tidal Gate and Tidal Lock alternatives, ship transits are provided for the existing canal only - allocation of ship transits to the Sea Level Canal is not stated explicitly.

Table A.2-7(a) Total Transit Projections for 2020

Canal DWT	Load and Bound Condition		Ship DWT Range 0 to 9999	10000 to 24999	25000 to 39999	40000 to 64999	65000 to 99999	100000 to 149999	150000 to 199999	1500000 to 249999	Total
150000	Ballast	North	141	233	317	213	90	83			1075
		South	585	839	202	101	29	2			1757
	Laden	North	1593	2805	1638	954	350	14			7353
		South	1212	2215	2130	1303	629	470			7958
	Total	North	1734	3038	1955	1167	440	96			8427
		South	1797	3054	2332	1404	658	472			9715
	Grand Total		3531	6092	4287	2571	1098	568			18142
200000	Ballast	North	141	233	317	213	90	82	7		1080
		South	585	839	202	101	29	1	1		1756
	Laden	North	1593	2805	1637	954	350	7	5		7351
		South	1212	2215	2130	1303	629	463	36		7987
	Total	North	1734	3038	1954	1167	440	89	12		8431
		South	1797	3054	2332	1404	658	464	37		9743
	Grand Total		3531	6092	4286	2571	1098	553	49		18174
250000	Ballast	North	141	233	317	213	90	82		15	1088
		South	585	839	202	101	29	1		1	1756
	Laden	North	1593	2805	1637	954	350	7		5	7351
		South	1212	2215	2130	1303	629	463		77	8028
	Total	North	1734	3038	1954	1167	440	89		20	8439
		South	1797	3054	2332	1404	658	464		78	9785
	Grand Total		3531	6092	4286	2571	1098	553		98	18224

Note: 1. Transit projections for 200,000 DWT vessels are from TAMS's fax dated 20 November 1992, Job No. 5419-15.
2. Transit projections for 150,000 and 250,000 DWT vessels are from the Final Report of the CTP Phase 1 Study (WEFA).
3. Totals are not necessarily equal to the sum of the elements. For reasons, consult above report.

Table A.2-7(b) Total Transit Projections for 2060

Canal DWT	Load and Bound Condition		Ship DWT Range 0 to 9999	10000 to 24999	25000 to 39999	40000 to 64999	65000 to 99999	100000 to 149999	150000 to 199999	150000 to 249999	Total
150000	Ballast	North	152	354	360	266	144	109			1384
		South	421	1315	231	151	45	1			2168
	Laden	North	1357	4525	1908	1345	456	59			9652
		South	1370	4011	2537	1781	941	657			11296
	Total	North	1509	4879	2268	1611	600	168			11036
		South	1791	5326	2768	1932	986	658			13464
	Grand Total		3300	10205	5036	3543	1586	826			24500
200000	Ballast	North	152	354	360	266	144	0	91		1367
		South	421	1315	231	151	45	0	1		2164
	Laden	North	1357	4525	1908	1345	456	49	8		9648
		South	1370	4011	2537	1781	941	45	508		11193
	Total	North	1509	4879	2268	1611	600	49	99		11015
		South	1791	5326	2768	1932	986	45	509		13357
	Grand Total		3300	10205	5036	3543	1586	94	608		24372
250000	Ballast	North	152	354	360	266	144	0		99	1374
		South	421	1315	231	151	45	0		1	2167
	Laden	North	1357	4525	1908	1345	456	49		8	9650
		South	1370	4011	2537	1781	941	45		549	11231
	Total	North	1509	4879	2268	1611	600	49		107	11024
		South	1791	5326	2768	1932	986	45		550	13399
	Grand Total		3300	10205	5036	3543	1586	94		657	24422

Note: 1. Transit projections for 200,000 DWT vessels are from TAMS's fax dated 20 November 1992, Job No. 5419-15.
2. Transit projections for 150,000 and 250,000 DWT vessels are from the Final Report of the CTP Phase 1 Study (WEFA).
3. Totals are not necessarily equal to the sum of the elements. For reasons, consult above report.

Table A.2-8 CAS Data for Assignment of Transits between New and Existing Facilities

November 19, 1992

Table: Number of Times each Lane of Locks Empties Water

1. Third Set of Locks 150,000 DWT

Third Set of Locks 150,000 DWT		2020		2060	
		Flushes	Ships	Flushes	Ships
Atlantic Gatun Lock	lane 1	5,107	7,930	5,568	9,916
	lane 2	5,358	8,219	5,661	10,101
	new	1,497	1,934	3,345	4,484
Pacific Miraflores	lane 1	5,207	7,505	6,204	9,653
	lane 2	6,061	8,643	6,722	10,371
	new	1,502	1,933	3,339	4,487
TAMS fax:Date, Page		11/6, 4		11/17, 2	

2. Third Set of Locks 250,000 DWT

Third Set of Looks 250,000 DWT		2020		2060	
		Flushes	Ships	Flushes	Ships
Atrantic Gatun Lock	Lane 1	5,068	7,897	5,571	10,000
	Lane 2	5,417	8,239	5,654	10,018
	New	1,578	2,014	3,433	4,561
Pacific Miraflores	Lane 1	5,125	7,428	6,253	9,603
	Lane 2	6,066	8,709	6,739	10,426
	New	1,569	2,013	3,425	4,560
TAMS fax: date, page		11/6, 2		11/17, 3	

3. Sea Level Canal (Tidal Gate) 250,000 DWT

Sea Level Tidal Gate 250,000 DWT		2020		2060	
		Flushes	Ships	Flushes	Ships
Atrantic Gatun Lock	Lane 1	5,061	7,865	5,647	9,506
	Lane 2	5,401	8,271	5,866	9,865
Pacific Miraflores	Lane 1	5,208	7,483	6,133	9,9253
	Lane 2	6,115	8,653	6,676	10,124
TAMS fax: date, page		11/16, 6		11/16 7	

4. Sea Level Canal (Tidal Locks) 250,000 DWT

Sea Level Tidal Lock 250,000 DWT		2020		2060	
		Flushes	Ships	Flushes	Ships
Atrantic Gatun Lock	Lane 1	5,172	7,934	5,680	9,604
	Lane 2	5,398	8,201	5,823	9,771
Pacific Miraflores	Lane 1	5,193	7,488	6,103	9,262
	Lane 2	6,120	8,648	6,647	10,115
TAMS fax: date, page		11/16, 8		11/16, 9	

Based upon TAMS's Phase I traffic and chambering analysis. Converted to flushes by Bastian.

Note that while Table A.2-8 includes suggested lock flush numbers as well as transit allocations, only the transit allocations have been used. The flush numbers have not been adopted for the Phase 2 Study. The derivation of the flush numbers which have been used is described in section 2.3.2.

Further refinement of the transit data from TAMS and CAS is necessary since the data provided does not include:

- The assignment of ship transits to existing and new locks for the 200,000 DWT High Rise Lock alternative.
- The number of ships likely to use the Sea Level Canal for either the Tidal Gate or Tidal Lock alternatives.

Derivation and rounding of the transit data for the above two cases are explained in (1) and (2) below.

(1) 200,000 DWT High Rise Lock Alternative

No information was provided on the assignment of the total number of transits for the 200,000 DWT case between the new locks and the existing locks. Therefore an assignment has been assumed after making comparisons with the assignments provided by CAS (Table A.2-8) for the 150,000 DWT and 250,000 DWT cases of the High Rise Lock Canal.

(a) Year 2020 Situation

Through Existing Locks
No. of Ships = Average of 150,000 DWT and 250,000 DWT
= (16,149 + 16,137) / 2
= 16,143 ;say 16,200

Through Third Set of Locks
No. of Ships = Balance of CAS 250,000 DWT transits for new locks minus (difference between TAMS transits for 200,000 DWT and 250,000 DWT)
= 2,014 - (98 - 49)
= 1,965 ;say 2,000

(b) Year 2060 Situation

Through Existing Locks
No. of Ships = Average of 150,000 DWT and 250,000 DWT
= (20,021 + 20,024) / 2
= 20,022 ;say 20,000

Through Third Set of Locks
No. of Ships = Balance of CAS 250,000 DWT transits for new locks minus (difference between TAMS transits for 200,000 DWT and 250,000 DWT)
= 4,561 - (657 - 608)
= 4,512 ;say 4,600

(2) 250,000 DWT Sea Level Canal

In the absence of assigned ship transit data for the 250,000 DWT Sea Level Canal (either Tidal Gate or Tidal Lock alternatives), it has been assumed that the number of ships to use the new Sea Level Canal will be equal to the balance of WEFA total number of ships for 250,000 DWT (Table A.2-7(a) and (b)) minus the number of transits assigned by CAS to the existing canal for the 250,000 DWT Sea Level Canal (Table A.2-8). The total number of transits estimated by CAS tidal gate and tidal lock variations are almost identical so the transit numbers assigned are the same. As there are slight differences in the data provided by CAS for the existing Atlantic and Pacific

locks for the tidal gate and tidal lock cases, average values for the transits through the existing locks have been used in the following analysis.

(a) Year 2020 Situation

Through Existing Locks
No. of Ships = Average value from CAS Table A.2-8 for tidal gate or tidal lock case.
= 16,136 ;say 16,200

Through Sea Level Canal (Tidal lock or gates)
No. of Ships = Balance of WEFA total (Table A.2-7) minus No. of Ships assigned by CAS to the existing locks (above).
= 18,224 - 16,136
= 2,088 ;say 2,100

(b) Year 2060 Situation

Through Existing Locks
No. of Ships = Average value from CAS Table A.2-8 for tidal lock case.
= Average value CAS Table of Nov 19th 1992
= 19,376 ;say 19,400

Through Sea Level Canal (Tidal lock or gates)
No. of Ships = Balance of WEFA total transits minus No. of Ships assigned by TAMS to the existing locks (above)
= 24,422 - 19,375
= 5,047 - say 5,100

(3) Discussion on Assignment of Ship Transits

In order to complete the Phase 2 Study, the assignment of ship transits has been estimated as described above based on information provided by CAS and WEFA. Accordingly, the results are summarized in Table A.2-9.

Table A.2-9 Adopted Number of Annual Ship Transits

Canal (DWT)	Assigned to:	A.D. 2020 Transits	A.D. 2060 Transits
H.R.L. (150,000)	Existing Locks	16200	20000
	3rd Set of Locks	2000	4600
	Total Transits	18200	24600
H.R.L. (200,000)	Existing Locks	16200	20000
	3rd Set of Locks	2000	4600
	Total Transits	18200	24600
S.L.C. (250,000)	Existing Locks	16200	19400
	Sea Level Canal	2100	5100
	Total Transits	18300	24500

In the case of the 250,000 DWT Sea Level Canal alternative, the data derived in (2) above results in significantly fewer ship transits being assigned to the Sea Level Canal than to the existing canal. This situation is similarly reflected in the cases where data has been provided, for example, far fewer ship transits are assigned to the 150,000 DWT High Rise Lock than to the existing locks.

To investigate this further, a comparison has been made between the number of ship transits assigned to each of the alternatives and the estimated capacity of each alternative as given in the OCCE Study (TAMS) Phase 1 Report. This comparison is also presented in Table A.2.10 overleaf where the assigned ship transits have been converted to a percentage of the estimated capacity.

Table A.2.10 Comparison of Assigned Annual Transits and Capacity

Alternative	Estimated Capacity **	Existing Locks		New Facility	
		Transits	% of Cap.*	Transits	% of Cap.
(1) Year 1990					
Status Quo	18,300 (1990)	12,000	65.6%		
(2) Year 2020					
150,000 HRL	11,800	16,200	88.5%	2,000	17.0%
200,000 HRL	11,500	16,200	88.5%	2,000	17.4%
250,000 HRL	11,200	16,200	88.5%	2,000	17.9%
250,000 SLC with gates	15,800	16,200	88.5%	2,100	13.3%
250,000 SLC with locks	11,700 (3 lanes)	16,200	88.5%	2,100	17.9%
(3) Year 2060					
150,000 HRL	11,800	20,000	109.3%	4,600	39.0%
200,000 HRL	11,500	20,000	109.3%	4,600	40.0%
250,000 HRL	11,200	20,000	109.3%	4,600	41.1%
250,000 SLC with gates	15,800	19,400	106.0%	5,100	32.3%
250,000 SLC with locks	11,700 (3 lanes)	19,400	106.0%	5,100	43.6%

Notes: * For simplicity, capacity of the existing locks has been assumed to be 18,300 in all cases.
** From OCCE Study (TAMS) Phase 1 Report (Tables XI-1&2).

For both 2020 and 2060, the existing locks would be operating at or near their capacity. Even allowing for future improvements to the existing facilities, the assignment of ship transits is heavily weighted towards the existing canal whilst the new alternative appears to be under utilized. In the case of the Sea Level Canal for example, the 3 lock lanes would not be essential in either Year 2020 or Year 2060 according to the above figures.

2.3.2 Number of Lock Flushes Adopted for Phase 2 Study

The annual number of lock flushes is shown in Table II.1.1-5. These numbers were obtained from the number of ship transits by assuming that each transit of the High Rise Lock Canal requires two lock flushes, while each transit of the Sea Level Canal requires one flush of the tidal lock. According to Table II.1.1-5, number of flushes for high rise lock with 3 lifts and alternative traffic is shown as 3 flushes per ship transit. The background of calculation of 3 flushes per ship transit is the explained in Fig. A.2-2 as an example.

2.3.3 Comparison with PCC Data for the Existing Locks

In the analysis above, it was assumed that an average value of two flushes per transit could be adopted. The validity of this assumption for the existing canal can be checked using PCC records for transits and water usage.

The data provided by PCC for Water Supply and Usage (PCC Annual Reports for 1990 and 1991 - Table 13) provides a breakdown of water usage for Madden and Gatun Lakes, including the annual volume of lockage water for Gatun and Pedro Miguel locks. This volume was compared to the total number of ship transits for Years 1989, 1990 and 1991 and the volume per transit used to estimate the number of lock flushes. The results are presented in Table A.2-11 below.

Table A.2-11 Volume of Lockage Water per Transit and Number of Flushes for Existing Locks

Year	1989	1990	1991
Lockage Water Used:			
1. In acre-feet x 10^3 *	2,006	1,970	2,123
2. In m^3 x 10^6	2,487	2,443	2,633
3. No. of Transits	12,075	12,052	12,763
4. Volume used per Transit = 2÷3 (m^3)	206,000	202,700	206,300
5. Volume of one Lock Flush (m^3)**	101,000	101,000	101,000
6. No. of Lock Flushes per transit = 4÷5	2.04	2.01	2.04

Note:

* One acre-foot is approx. 1240 cubic metres (m^3)

** Volume of one flush of existing locks is estimated from the actual dimensions between the lock gates, rather than the nominal 1000' available. Therefore the volume of one flush

≈ (length of 1071'+81'-7') x (width of 110') x (depth of say 85'/3)

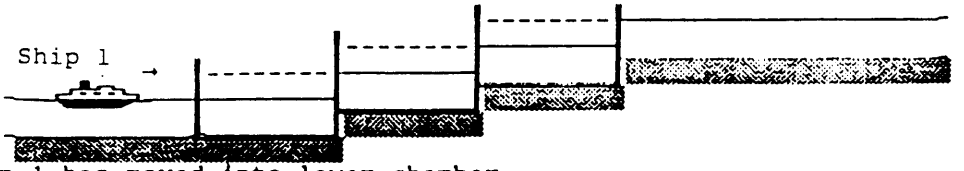
= 1145' x 110' x 28.33'

= 3.57 million ft^3 = 101,000 m^3

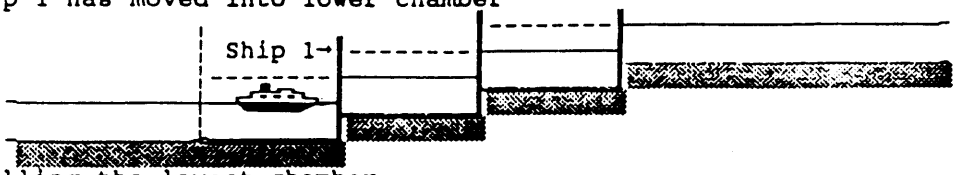
The results of this analysis therefore support the contention that two flushes per transit is a reasonable assumption for preliminary design.

[3] Triple lifts (3) Alternate traffic

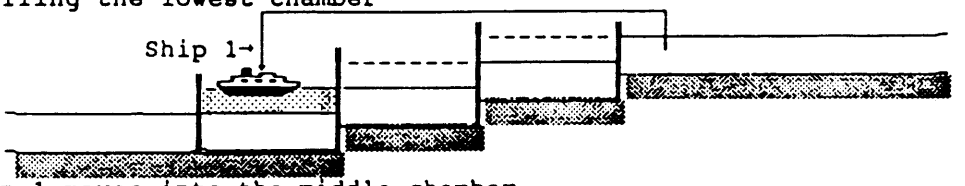
(1) Initial Stage (Ship 1 from Downstream)



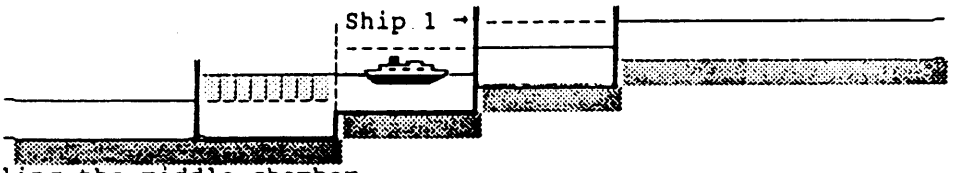
(2) Ship 1 has moved into lower chamber



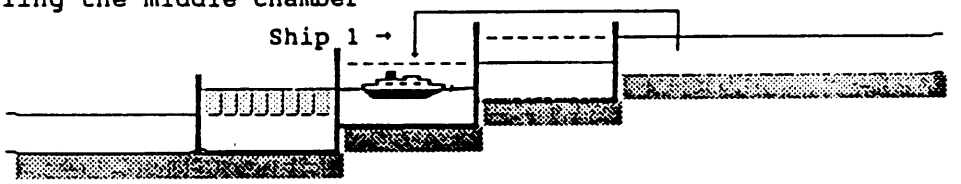
(3) Filling the lowest chamber



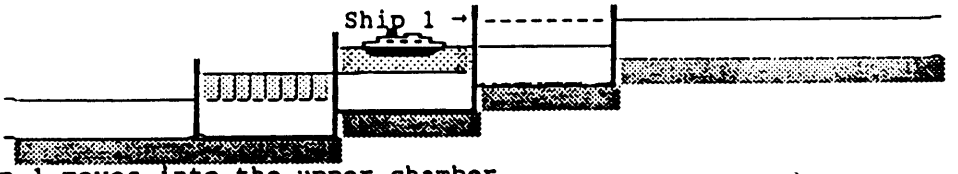
(4) Ship 1 moves into the middle chamber



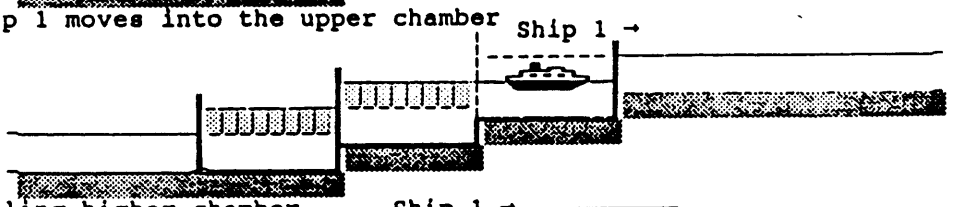
(5) Filling the middle chamber



(6)



(7) Ship 1 moves into the upper chamber



(8) Filling higher chamber

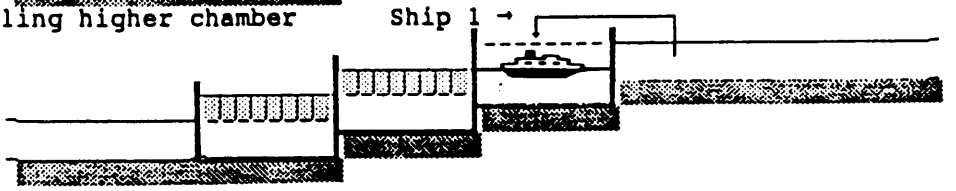


Fig. A.2-2(a) Number of Lock Flushes (by Dr. K. KOBUNE, CAS)

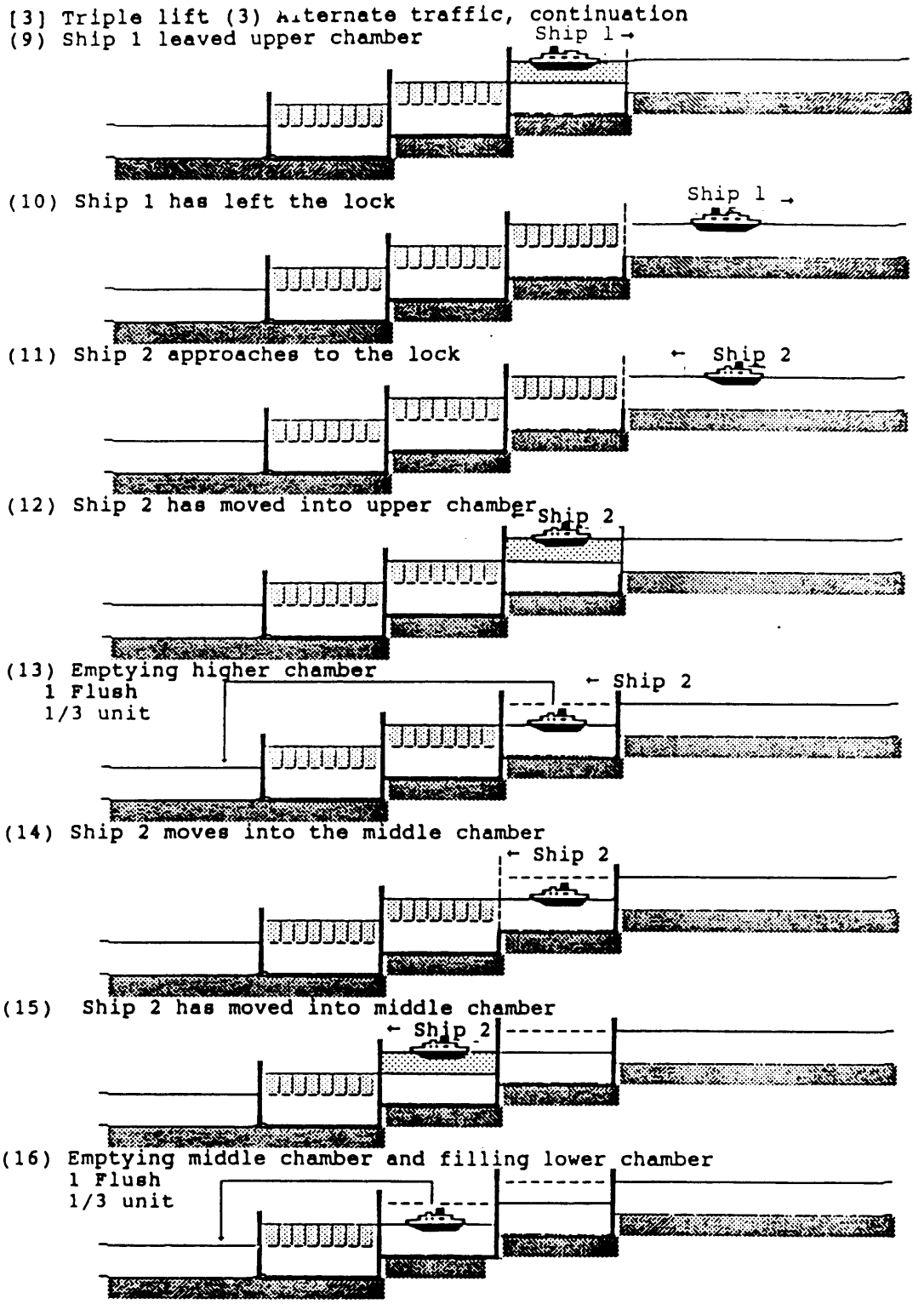
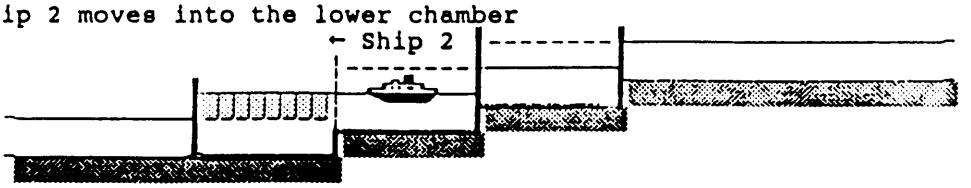


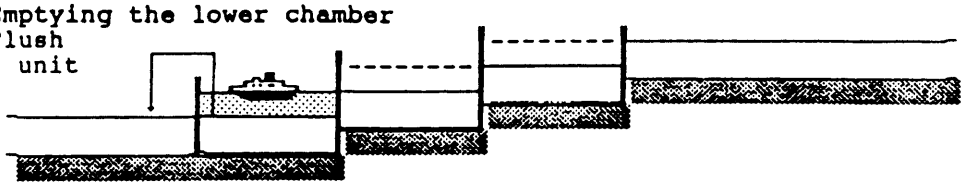
Fig. A.2-2(b) Number of Lock Flushes (by Dr. K. KOBUNE, CAS)

[3] Triple lift (3) Alternate traffic (continuation 2)

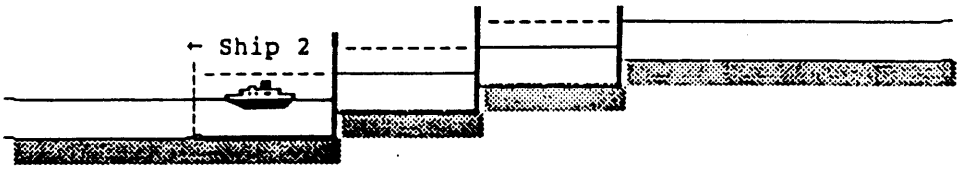
(17) Ship 2 moves into the lower chamber



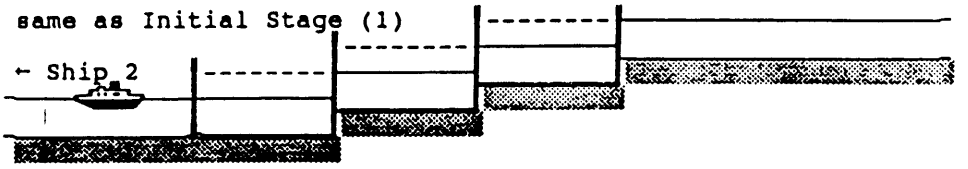
(18) Emptying the lower chamber
1 Flush
1/3 unit



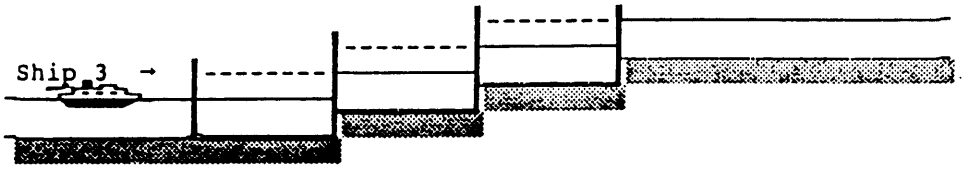
(19) Ship 2 leaves the lower chamber



(20) Ship 2 left the lower chamber
The same as Initial Stage (1)



(21) Ship 3 from Downstream (The same as (1) Initial stage)



Triple lift lock, Alternate Traffic operation
3 Flushes per 2 lockges, i.e. 1.5 Flushes per lockage
1 Flush discharges 1/3 unit

Fig. A.2-2(c) Number of Lock Flushes (by Dr. K. KOBUNE, CAS)

Appendix 3

Alternative to Tidal Lock and Tidal Gate

APPENDIX 3 - ALTERNATIVE TO TIDAL LOCK AND TIDAL GATE

3.1 Discussion on Basic Designs and Alternative Designs

3.1.1 Introduction

In addition to the results presented in IV.1 Chapter 5 "Tidal Lock Gates and Tidal Gates", study efforts were made on alternative proposals.

This sub-chapter intends to present brief discussions for comparison between torsion type tidal gate and the basic design for the Phase 2 study, triangular sliding tidal gate. The presentation in this sub-chapter also includes impact on the total cost of the Canal Alternatives under studied and a conclusion to this additional study.

CAS directed the elimination from consideration tug boat guided sea-level locks based upon the recommendations of an experienced pilot for reasons of safety, insurance and efficiency for capacity.

3.1.2 Triangular and Torsion Type Tidal Gates

The gate type recommended for the Tidal Gate in the Phase 1 study was a triangular leaf rolling gate. The best evaluation was given to a torsion type slide gate (=rolling gate in the Phase 1 study), but this recommendation was made based on working records of this type of gate at Zandvliet lock in the port of Antwerp, Belgium. Further study of the largest three tidal locks in Belgium disclosed the fact that the statement in the Phase 1 Final Report about the working record of a triangular type gate in Belgium was revised. In Phase 2, it was concluded that there is no definitive difference between the experience records supporting both gate types, and that a torsion type slide gate is recommendable for the Tidal Gate. In the Phase 2 study, the triangular leaf rolling type Tidal Gate was studied as a base for cost estimation, and a torsion type slide gate for the Tidal Gate was studied as an alternative proposal whose results

including estimates are presented in detail at 3.2 of this Appendix. Benefits of the torsion type Tidal Gate over the triangular leaf rolling type in addition to substantial cost savings for the gates and related items are listed below.

(1) Simplification on concrete structures and concrete volume reduction.

(2) Cost reduction for maintenance work because of mono-rail system on the canal floor.

(3) Possibility of leaf maintenance at the existing dry dock because of the smaller gate thickness.

3.1.3 Impact on Total Costs of Canal Alternatives under Studied

Table A.3-1 shows possible reduction amounts for total cost of each Canal Alternative under studied.

3.1.4 Conclusion

As a result of this additional study on the two alternative proposal for Tidal Gate, following two conclusions with recommendations can be derived.

(1) The concept of the torsion type Tidal Gate has been shown to be feasible and to have high potential for cost reductions for the Sea Level Canal Alternatives. It is recommended that further study of this proposal should be made during subsequent design stages for a sea level canal alternative.

(2) The study of the torsion type Tidal Gate for the Sea Level Canal Alternative has also indicated that the torsion type Operating Gate for High Rise Lock Canal Alternative as proposed in (3) of 4.6.1 in the Phase 1 Final Report is likely to be feasible, and probably advantageous. It is therefore also recommendable that comparative study effort should be made on this gate type during the next study step for the third set of locks for the high rise lock canal. It is anticipated that a substantial cost reduction rate could be achieved for the high rise lock, much more than use of this gate type as the Tidal Gate for the Sea Level Canal Alternative.

Table A.3-1 Reduction Amounts for Total Cost of Canal Alternatives under Studied

(UNIT;1000US$)

Canal Option	Basic Desing			Alternative Design			Reduction
	2020	2060	Total	2020	2060	Total	Amount
200,000DWT High Rise Lock	1,011,318		1,011,318				-
150,000DWT Hig Rise Lock	888,867		888,867				-
Sea Lelevel Case A	949,507	316,002	1,265,509				-
Sea Level Canal Case B	737,962		737,962	503,166		503,166	234,796
Sea Level Canal Case C	1,252,178	316,002	1,568,180	1,151,897	316,002	1,467,899	100,281

Note

1. The figures in the tabel are cost for Gate Leaves and Related Facilities only.
2. "Bacic Design" in the table column means the desing for cost estimates at the Phase 2 study.
3. "Alternative Design" in the table column means a Design proposed as an alternative.
4. Alternative Design is the torsion type tidal gate.

3.2 Torsion Type Tidal Gate

3.2.1 Introduction

The torsion type tidal gate resists against torsional moment with its whole section. This type of structure has various advantages over bending type structures.

(1) Leaf weight is much smaller than a triangular leaf rolling gate.

(2) Ability to withstand pressure on both sides of the leaf without a pressure balance apparatus.

(3) Reduced maintenance cost.

(4) Advantageous for fatigue designs due to lower tensile stresses.

The torsion type gate has almost 20 year's of working records. Records of 100m width x 13.5m height and 80m width x 12.0 high gates are introduced in TECHNICAL REVIEW June 1980 by Mitsubishi Heavy Industries, Ltd. in Japan.

3.2.2 Description of Gates

The height, width and weights of the Operating Gate and the Emergency Gate are shown on Table A.3-2. The width is nominal and equal to the canal channel width. The gate can also be used as a road bridge for vehicular traffic crossing the canal channel. No spare gate is required for the Operating Gate.

Fig. A.3-1 shows the general arrangement of the Tidal Gate. The gate is divided into two leaves wich meet at the center of the canal. The whole gate leaf is a box section which resists a torsional moment whose magnitude is equal to the couple of a hydraulic force and roller reaction forces on the gate leaf. Each leaf is supported at one end by a couple of forces, one at the top on the downstream side and another at the bottom on the upstream

side of the leaf. The hull (=skin plate) which is subject to hydraulic water pressure is supported by vertical auxiliary girders, which are supported by horizontal girders, which are finally supported by vertical diaphragms arranged at every 10 meters throughout the length of the gate leaf. Each diaphragm is supported by two rollers having a roundly concaved tread surface which will run on a round steel rail. Eventually, most of the hydraulic force on the gate leaf will be supported by the rollers arranged along the bottom of the gate leaf, and only a torsional moment will be carried to a supported leaf end. The inside of the leaf is separated into several compartments some of which are assigned as buoyant tanks used when the leaf is removed for maintenance work. The buoyant tank has a longitudinal bulkhead, but will have no bottom.

The Emergency Gate is exactly identical to the Operating Gate.

Table A.3-2 Dimensions and Weights of Operating Gate and Emergency Gate

Case	Side	Gate Name	Height in Meter	Width in Meter	Weight in Ton		
					Leaf	Frame	Hoist
(B)	Atlantic	Operating Gate	27.5	190	5950	1785	595
		Emergency Gate	27.5	190	5950	1785	595
	Pacific	Operating Gate	27.5	190	5950	1785	595
		Emergency Gate	27.5	190	5950	1785	595
(C)	Atlantic	Operating Gate	25.0	190	3699	1110	370
		Emergency Gate	25.0	190	3699	1110	370

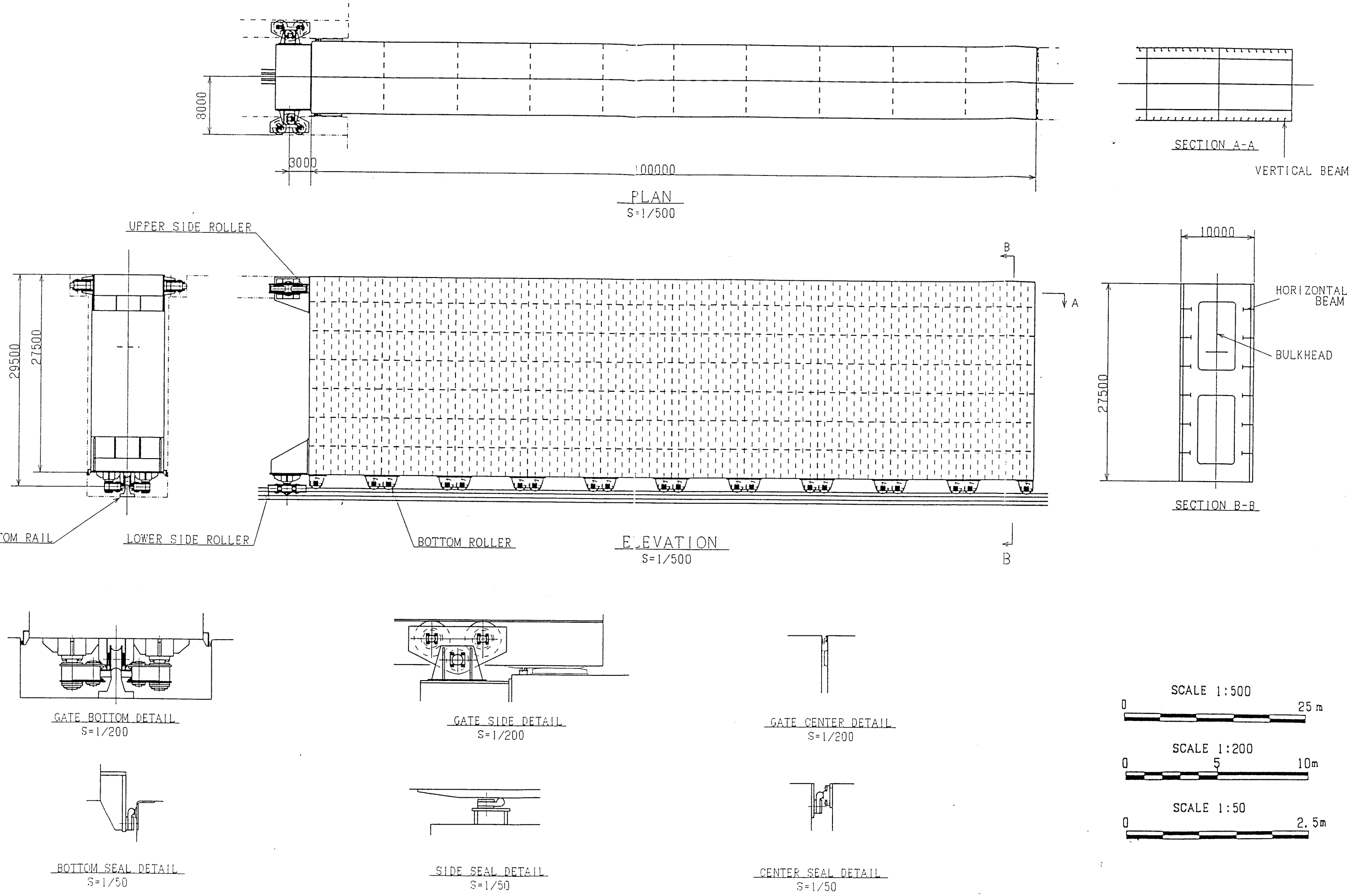


Fig.A.3-1 General Arrangement of Tidal Gate (Torsion Type)
Sea Level Canal 250000 DWT Canal Option

3.2.3 Design of Gates

Design heads in meters for the Atlantic and Pacific side of each gate are shown on Table A.3-3. For Case (b), the Atlantic side heads correspond to the minimum water elevations of the Atlantic ocean, and the sea side heads correspond to the maximum water elevations of the Pacific Ocean. For Case (c), the Atlantic side head corresponds to the minimum water elevations of the Atlantic and the Pacific side head is the maximum water elevation of the canal.

Table A.3-3 Design Heads in Meter of Operating Gate and Emergency Gate

Case	Side	Gate Name	Atlantic Side	Pacific Side
(b)	Atlantic	Operating Gate	23.0	27.3
		Emergency Gate	23.0	27.3
	Pacific	Operating Gate	23.0	27.3
		Emergency Gate	23.0	27.3
(c)	Atlantic	Operating Gate	23.0	24.4
		Emergency Gate	23.0	24.4

3.2.4 Installation and Removal of Gates

The Operating Gate and the Emergency Gate for the Tidal Regulating Structure fabricated at shop including all accessories are towed to Panama by tug boats. They are set in place using a floating crane according to the steps shown in Fig.III.1.5-4. It is anticipated that gate leaves fabricated and installed using these procedures would have bigger deviations from their drawing dimensions and from their relative drawing positions to the gate frames or the hoist mechanisms installed in concrete structures before hand than gates installed using a conventional block erection method with which dimensional adjustment of gate leaves is possible during block erec-

tion either at shop or at site. Hence, gate seal mechanisms, roller systems, support system etc. which interface on the leaf side to the equipment on concrete structures under water must be able to adapt to bigger dimensional deviations in gate leaves and embedded metals than ordinary gate designs. The proposed installation method is hereby named "Just-Set-In-Place Installation" versus "the conventional block erection installation ".

The Just-Set-In-Place Installation method also enables the gate leaves under water to be removed or reset for maintenance. It is worth while to note that all gate parts under water cannot be adjustable in drained conditions.

3.2.5 Machinery for Operating Gate and Emergency Gate

The machinery type adopted for the Operating Gate and for the Emergency Gate for the tidal regulating structure is a looped wire cable and hydraulic motor type as used in guiding small vessels at a tidal lock. A looped wire cable in a pulley system is driven by hydraulic oil motors with the assistance of a small hydraulic cylinder to give proper tension in the continuous cable. At least six cables are used to pull each leaf. The machinery is capable of opening or closing the canal in 10 minutes. Each leaf is equipped with two hydraulic power units, each of which is powered by two induction motors of 250 KW for Case (b) and 130 KW for Case (c). Time restrictions prevented the study of the possibility of a cost saving with a hydraulic pumping station for each Tidal Regulating Structure with the related controls and distribution system.

3.2.6 Result of Engineering Analysis

Engineering analysis was made for the Tidal Gate for Case (b).

(1) Structural Analysis of Gate Leaf

The whole gate leaf was analyzed using the computer program, NASTRAN, which was developed at NASA in the USA. This program is based upon a finite element method and a very effective analysis is possible for the number of elements adopted because of combining plate elements and beam elements. The number of plate elements in mesh was approximately 2500, and the beam elements numbered approximately 2000. The computer used was an IBM 9021. Load applied corresponds to the design heads. Fig. A.3-2 thru -4 show the results of the structural analysis.

Fig. A.3-2 shows scantlings and the principal stress of major structural members. The results are still at an intermediate stage, and the possibility of an improvement in weight distribution and leaf configuration can be seen.

Fig. A.3-3 shows a mesh of elements analyzed and deformations for the leaf. Displacement at the gate top at gate width center, which is a free end structurally, is equal to approximately 390 mm in a horizontal direction. No particular problem can be seen with this deflection including the sealing function.

Fig. A.3-4 shows principal stress distribution on the hull at the pressure side. This stress pattern is acceptable for such a torsion type gate.

(2) Stability when Totally Closed

Stability of the leaf totally closed and subject to various hydraulic conditions was analyzed and the conclusions below were obtained.

a. The leaf is stable throughout all hydraulic conditions.

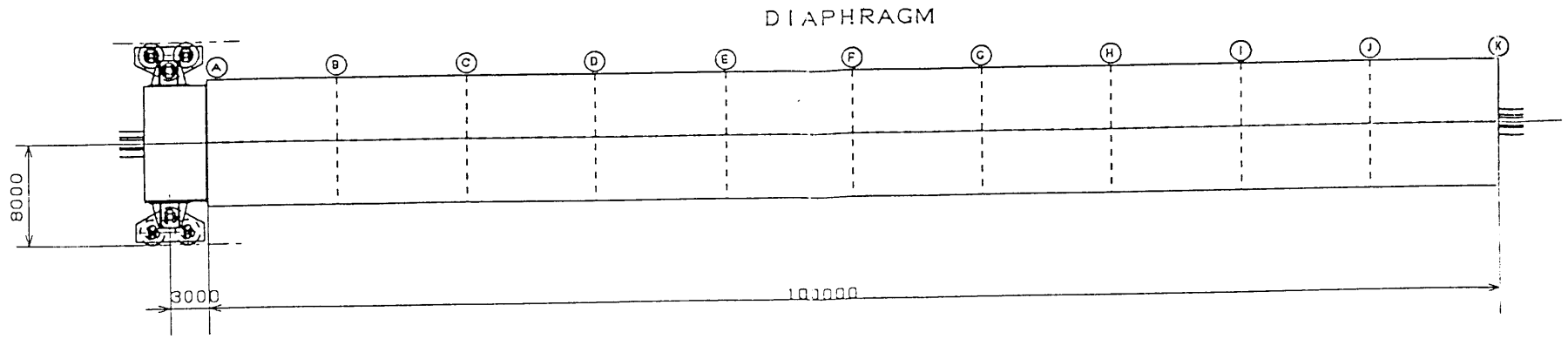
b. Wheel load; approximate values are 150 tons normal to the tread surface, and 600 tons parallel to the tread surface.

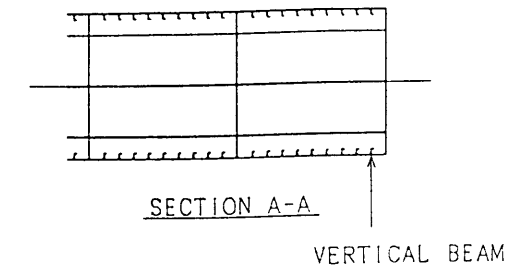
(3) Stability during Opening or Closure Operation

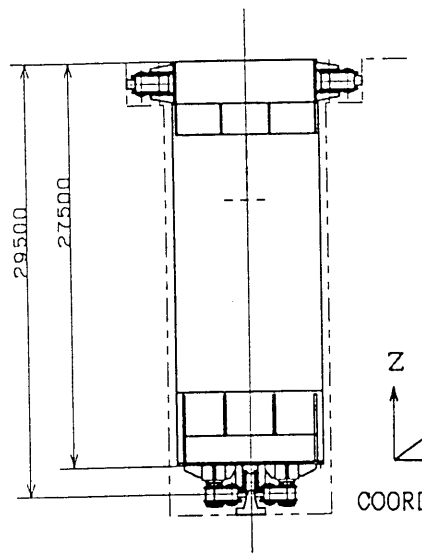
Stability of the leaf during moving operation while subject to a 0.9 m water head difference was analyzed and the conclusions are given below.

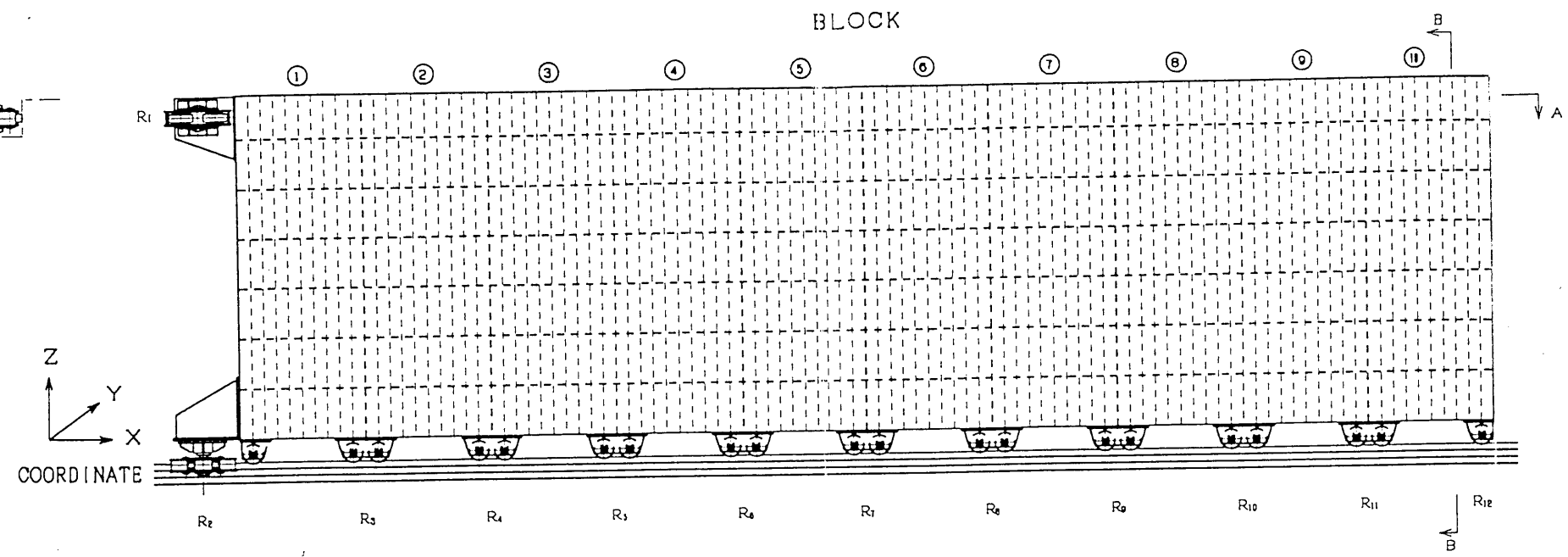
a. The leaf is stable throughout all hydraulic conditions.

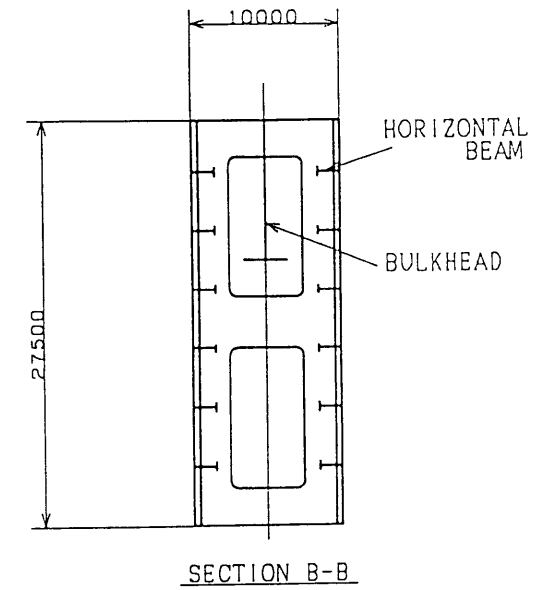
b. Wheel load during gate movement;
Approximate values are 150 tons normal to the tread surface and 150 tons parallel to the tread surface.











SKIN PLATE AND BEAM

+:TENTION
-:COMPRESSION

BLOCK		① ②	③ ④	⑤ ⑥	⑦ ⑧	⑨ ⑩
DIMENSION OF MEMBER(mm)	SKIN PLATE THICKNESS	34	16	14	12	12
	BULKHEAD THICKNESS	10	10	10	10	10
	HORIZONTAL BEAM SECTION	I 1500x721 x35/40	I 1500x699 x30/50	I 1000x546 x23/55	I 800x389 x10/36	I 600x244 x13/33
	VERTICAL BEAM SECTION	L 400x100x13/18				
MAXIMUM PRINCIPAL STRESS (kgf/mm²)	SKIN PLATE	+10.86 -13.08	+13.74 -14.44	+13.14 -14.18	+11.99 -13.28	+11.72 -12.77
	BULKHEAD	+4.54 -4.59	+4.40 -4.70	+4.44 -5.35	+4.76 -5.76	+5.16 -5.68
	HORIZONTAL BEAM	+13.15 -13.68	+16.85 -17.45	+6.19 -7.50	+9.41 -12.67	+10.52 -13.21
	VERTICAL BEAM	+9.38 -9.60	+12.10 -12.26	+10.70 -10.93	+8.81 -9.05	+9.48 -12.21
SHEARING STRESS	SKIN PLATE	10.82	12.65	11.43	10.24	9.53
	BULKHEAD	3.92	3.85	3.35	3.54	3.74
	HORIZONTAL BEAM	1.59	1.85	3.35	5.49	3.30

DIAPHRAGM

	A	B C	D E	F G	H I	J	K
THICKNESS (mm)	22	11	10	10	10	10	10
MAXIMUM PRINCIPAL STRESS (kgf/mm²)	+11.25 -12.46	+9.45 -12.70	+6.11 -9.44	+6.09 -9.14	+5.37 -8.33	+6.06 -8.61	+9.31 -10.40
SHEARING STRESS (kgf/mm²)	11.66	9.28	6.84	6.42	5.84	6.31	7.18

REACTION FORCE

	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12
Y-DIRECTION	-5192	+4036	+182	+326	-665	-928	-729	-929	-734	-717	-734	-1317
Z-DIRECTION	0	+137	+432	+312	+233	+218	+204	+187	+173	+164	+153	+79

Fig.A.3-2 Result of Stractural Analysis of Tidal Gate (Torsion Type)
Principal Stress Table

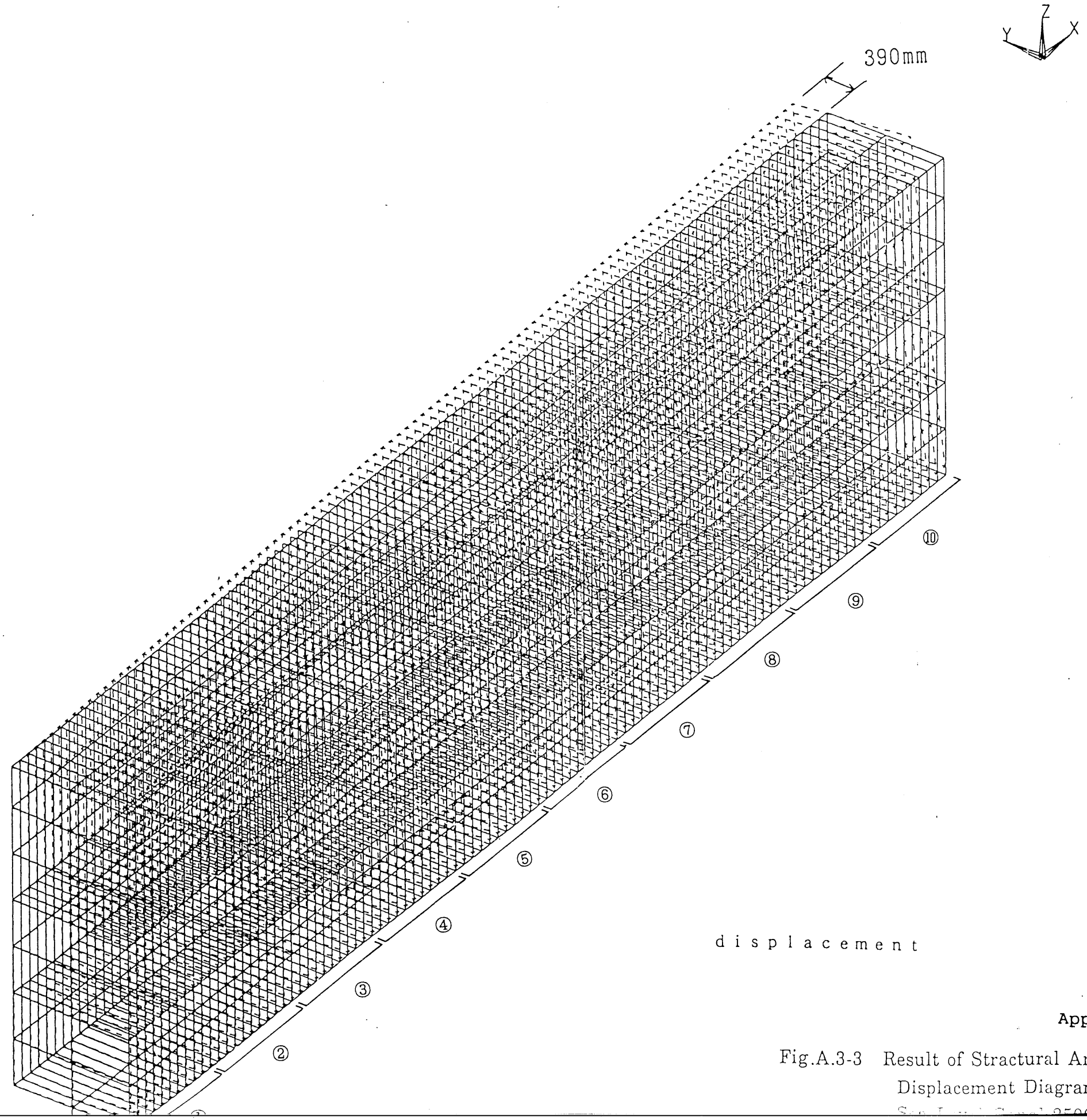
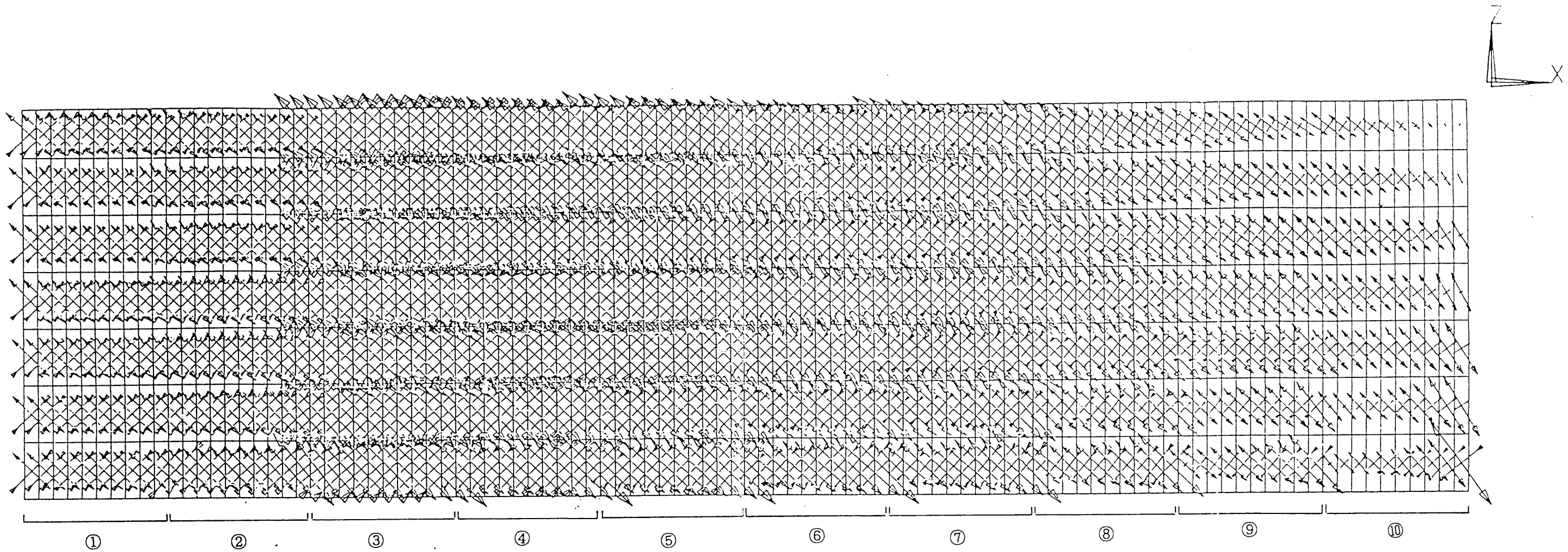
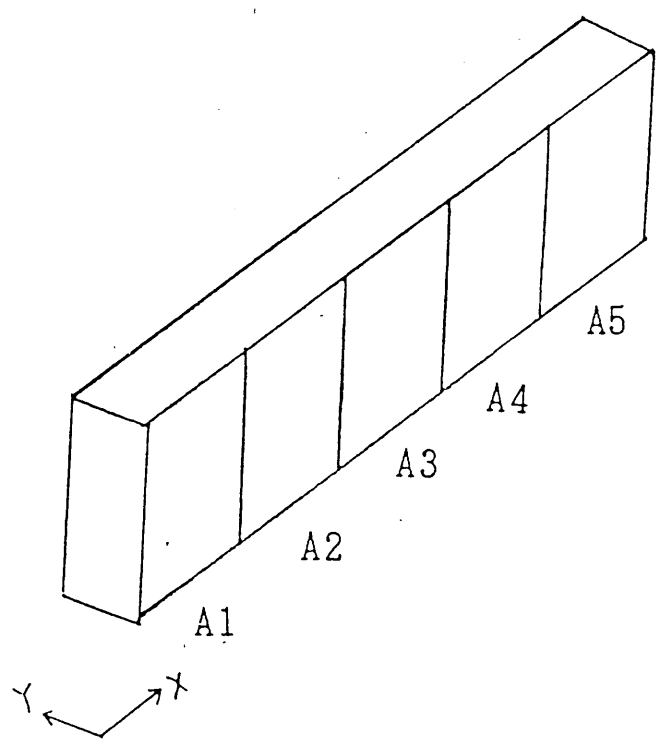


Fig.A.3-3 Result of Stractural Analysis of Tidal Gate (Torsion Type)
Displacement Diagram



PRINC	STRE	1.37E1
CASE NO.		1



principal stress A1－A5 (top)

Fig.A.3-4 Result of Stractural Analysis of Tidal Gate (Torsion Type)
Principal Stress Diagram
Sea Level Canal 250000 DWT Canal Option

3.2.7 Other Design Items

Design was made for the other all items included in IV.1-Chapter 5.3 although their results are not given.

3.2.8 Cost Estimates

Table A.3-4 shows cost estimates for Case (b) with the torsion type tidal gates. These are the costs for gates and related facilities, and the following cost reductions could be expected as well.

(1) Simplification of concrete structures and concrete volume reduction.

(2) Cost reduction for maintenance work

(3) Possibility of leaf maintenance at the existing dry dock.

Table A.3-5 shows cost estimates for Case (c) with torsion type tidal gates and tug boat guided tidal lock. These are the costs for gates and related facilities and other cost reductions listed above chapter could be expected.

Table A.3-4 Cost Estimate for Gate and Related Facilities
250000 DWT Sea Level Canal Case (b)
- Alternative (2020)

(1) ATLANTIC TIDAL GATE--TORSION TYPE TIDAL GATE

Item No.	Description	Unit	Q'ty	Unit Cost in 1000$	Amount in 1000$
1)	Gate Leaves				
1	Gate Leaves	ton	11900	9.60	114,240
2	Accessory	set	1	10,282	10,282
				Total	124,522
2)	Mechanical Equipments				
1	Gate Frames	ton	3570	11.00	39,270
2	Gate Hoists	ton	1190	15.60	18,564
3	Other Mechanical Component	set	1	37,356	37,356
				Total	95,190
3)	Electrical Power and Control				
1	Electric Power and Control	set	1	4,200	4,200
	Equipments (3600KW)				
2	EMERGENCY POWER PLANT	KW	3600	1.15	4,154
3	Miscellaneous El. Equipments	set	1	1,000	1,000
				Total	9,354
4)	Others	set	1	5,230	5,230
				Total	5,230
Grand Total					234,296

(2) PACIFIC TIDAL GATE

Item No.	Description	Unit	Q'ty	Unit Cost in 1000$	Amount in 1000$
1)	Gate Leaves				
1	Gate Leaves	ton	14400	9.60	138,240
2	Accessory	set	1	12,442	12,442
				Total	150,682
2)	Mechanical Equipments				
1	Gate Frames	ton	3570	11.00	39,270
2	Gate Hoists	ton	1190	15.60	18,564
3	Other Mechanical Component	set	1	45,000	45,000
				Total	102,834
3)	Electrical Power and Control				
1	Electric Power and Control	set	1	4,200	4,200
	Equipments (3600KW)				
2	EMERGENCY POWER PLANT	KW	3600	1.15	4,154
3	Miscellaneous El. Equipments	set	1	1,000	1,000
				Total	9,354
4)	Others	set	1	6,000	6,000
				Total	6,000
Grand Total					268,870

Table A.3-5 Cost Estimate for Gate and Related Facilities
250000 DWT Sea Level Canal Case (c)
- Alternative (2020)

(1) ATLANTIC TIDAL GATE--TORSION TYPE TIDAL GATE

Item No.	Description	Unit	Q'ty	Unit Cost in 1000$	Amount in 1000$
1)	Gate Leaves				
1	Gate Leaves	ton	9899	9.60	95,027
2	Accessory	set	1	8,552	8,552
				Total	103,579
2)	Mechanical Equipments				
1	Gate Frames	ton	2220	11.00	24,415
2	Gate Hoists	ton	740	15.60	11,542
3	Other Mechanical Component	set	1	31,000	31,000
				Total	66,957
3)	Electrical Power and Control				
1	Electric Power and Control Equipments (3600KW)	set	1	21,700	21,700
2	EMERGENCY POWER PLANT	KW	3600	1.15	4,154
3	Miscellaneous El. Equipments	set	1	2,000	2,000
				Total	27,854
4)	Others	set	1	4,000	4,000
				Total	4,000
Grand Total					202,390

(2) PACIFIC TIDAL LOCK---LOCOMOTIVE GUIDED

Item No.	Description	Unit	Q'ty	Unit Cost in 1000$	Amount in 1000$
1)	Gate Leaves	ton	32328	9.73	314,666
				Total	314,666
2)	Mechanical Equipments				
1	Gate Frames	ton	9628	11.11	106,920
2	Gate Hoists	ton	4510	15.00	67,648
3	Locomotives	unit	30	3,600	108,000
4	Rails and Racks for Loco-motives	ton	9042	8.14	73,558
5	Power Supply line for Loco.	set	1	12,000	12,000
6	Other Mechanical Component (Compressed Air System, Drainage Pump and Piping and Miscellaneous Mechanical Equipments)	set	1	94,400	94,400
				Total	462,526
2)	Electrical Power and Control				
1	Electric Power and Control Equipments (84000KW)	set	1	78,400	78,400
2	EMERGENCY POWER PLANT	KW	84000	0.77	64,615
3	Miscellaneous El. Equipments	set	1	16,000	16,000
				Total	159,015
4)	Others				
1	Local Control Buildings and	set	1	1,100	1,100
2	Their Electrical Equipments,	set	1	300	300
3	Cathodic Protection,	set	1	4,100	4,100
4	Monitoring & Surveillance Equipments and Communication System	set	1	4,800	4,800
5	Grounding System	set	1	1,100	1,100
6	and Other Electric works and Consumables	set	1	1,900	1,900
				Total	13,300
Grand Total					949,507

Appendix 4

Cost Estimates

4.1. Additional Subsurface Investigations for Sea Level Canal

(FILE:APPD1)

	WORK ITEM No. * Month * Year	Unit	Quantity	Unit Cost	Amount US$
1.	TEST BORING				
	1.1 Preparation	Nos	69	25,000	1,725,000
	1.2 Boring Work	Mo	6,600	400	2,640,000
	1.3 Travel Expense 10*4	Trips	40	5,000	200,000
	1.4 Site Transport	Mo	6	30,000	180,000
	1.5 Test Report	LS	1		55,000
2.	GEOLOGICAL SURVEY				
	2.1 Field Reconnaissance	LS	1		100,000
	2.2 Sismic Prospecting	Nos.	12	50,000	600,000
	2.3 Salaries/Wages 10*6	Mo	60	7,000	420,000
	2.4 Travel Expense 10*4	Trips	40	5,000	200,000
	2.5 Communications (TEL, FAX)	Mo	6	10,000	60,000
	2.6 Office Equipment & Supply	Mo	6	5,000	30,000
	2.7 Report/Documentation	LS	1		60,000
3.	GROUNDWATER SURVEY/ANALYSIS				
	3.1 Field Reconnaissance	LS	1		100,000
	3.2 Salaries/Wages 10*6	Mo	60	7,000	420,000
	3.3 Travel Expense 10*4	Trips	40	5,000	200,000
	3.4 Communications (TEL, FAX)	Mo	6	10,000	60,000
	3.5 Office Equipment & Supply	Mo	6	5,000	30,000
	3.6 Test Report	LS	1		50,000
4.	LABORATORY TEST				
	4.1 Soil Test	Nos	50	1,000	50,000
	4.2 Salaries/Wages 6*12*2	Mo	144	7,000	1,008,000
	4.3 Travel Expense 6*4	Trips	24	5,000	120,000
	4.4 Communications	Mo	12	10,000	120,000
	4.5 Office Equipment & Supply	Mo	12	5,000	60,000
	4.6 Test Report	LS	1		50,000
5.	AERIAL SURVEY				
	5.1 Aerial Survey	LS	1		500,000
	5.2 Salaries/Wages 4*12	Mo	48	7,000	336,000
	5.3 Travel Expense 4*8	Trips	32	5,000	160,000
	5.4 Communications	Mo	12	10,000	120,000
	5.5 Office Equipment & Supply	Mo	12	5,000	60,000
	5.6 Mapping/Report	LS	1		1,500,000
	TOTAL				11,214,000

Note: Number of geological boring and test: 69points 6,600 m
Aerial survey: 100km2 and 1/2,500scale mapping

4.2 Comparison of Studied Result
(Sea Level Canal Route 10<Case-b> with Nagano Plan)

File Name : APPD2 2-1

DESCRIPTION	SEA LEVEL CANAL, ROUTE 10 CASE b				NAGANO PLAN	
1. Canal						
(1) Route	Route 10, Sea Level				Route 10, Sea Level	
(2) Length	88.4km				98.00 km	
(3) Ship Size	250,000 DWT				300,000 DWT	
(4) Canal Cross Section Dimentions		Width	Depth		Width	Depth
	Atlantic	380m	-24m	2-lane	400m	-33.4m
	Inland	190m	-24m	1-lane	200m	-33.0m
	Pacific	380m	-24m	2-lane	400m	-35.8m
(5) Tidal Gate	2 Locations				N.A.	
2. Volume of Canal Excavation	Dry Excavation	1,033,572,000 m3			1,413,000,000 m3	
	Dredging	351,378,000 m3			387,000,000 m3	
	Total	1,384,950,000 m3			1,800,000,000 m3	
3. Construction Plans						
(1) Yearly Working Days		249 days			250 days	
(2) Work Schedule	Engineering/Disign	3.5 yrs.			2 yrs.	
	Canal Excavation etc.	11.5 yrs.			8 yrs.	
	Total	15 yrs.			10 yrs.	
(3) Main Construction Equipment	Shovel	30 m3 class			20 m3 class	
	Bulldozer	98 ton class			86 ton class	
	Dumptruck	240 ton class			120 ton class	
	Belt Conveyer	N.A.			w=3.2m, w=2.8m, w=1.8m	
	Crusher (for boulder)	N.A.			750 t/h class	
	Crawler Drill	φ225 mm			φ165 mm	
	Cutter Suction Dredger	27,500 HP, 18,000 HP			25,000 HP	
	Trailer Hopper Drodger	10,000m3 -19,000 HP			10,000m3 - 19,000 HP	
	Grub Dredger	50m3 - 3,000 HP			-	
		Spilt Barge 5,000 m3			Flat Barge 300t , 1,000t	
		Pusher Tug 3,200 HP			Tug Boat 500ps , 3,000 HP	

DESCRIPTION	SEA LEVEL CANAL ROUTE 10 (CASE-b)				NAGANO PLAN	
	COST OF CASE-b (the Year of 2,020)		$	COSTS AS PER NAGANO PLAN (Mill. of $)		(Mill. of $)
4. Cost Comparision	CONSTRUCTION COSTS					
	Canal Excavation	ls	7,373,056,330	7,373.00		
	Placing Area	ls	188,682,691	189.00		
	Lock Construction	ls	428,936,820	0.00		
	New Dams		-	-		
	Pump Station		-	-		
	New Permanent Roads	ls	19,419,590	0.00		
	New Canal Crossing	ls	309,088,760	0.00		
	Barrier Dams	ls	493,502,306	494.00		
	Breakwater	ls	402,015,840	402.00		
	Diversion Scheme	ls	75,285,156	75.00		
	Supporting Construction	ls	574,412,520	574.00		
	Subtotal		9,864,400,013	9,107.00		
	PLANT FORMING A PART OF PERMANENT WORKS					
	Gate Leaves and Related Facilities	ls	737,960,800	0.00		
	Navigation Support System	ls	477,046,333	0.00		
	Subtotal		1,215,007,133	0.00		
	CONSTRUCTION COSTS TOTAL		11,079,407,146	9,107.00		
	INDIRECT COSTS					
	Administrative Costs	ls	113,010,000	0.00		
	Headquarter's Costs	ls	191,425,600	0.00		
	Construction Supervision	ls	214,298,900	0.00		
	Land Aquisition & Rights	ls	27,337,175	0.00		
	Compensation Costs	ls	13,557,600	0.00		
	Contingency (10% of Const. Costs)	ls	1,107,941,000	0.00		
	INDIRECT COSTS TOTAL		1,667,570,275	0.00		
	GRAND TOTAL (PROJECT COSTS)		12,746,977,421	9,107.00		16,200 (*1)
	Total Project Costs / Total Excavation Volume			$ 6.58 /m3		$ 9.00/m3
	The Year Studied			1990	Updated to 1989	

Note: 1. Comparison of studied result with I.O.C.S. Route 10 is omitted from this table because of the insufficiency of data concerning construction planning on I.O.C.S. Route 10.
2. (*1) 'The Transisthmian alternatives for Increasing the Throughout Across Panama' by David F. Bastian May, 1990

4.3 High rise Lock Canal

(1) Costs of Second Set of New Locks (200,000DWT)

File Name : APPD3

Description		High Rise Lock Canal, 200,000DWT			
		Unit	Q'ty	Unit Cost	Amount
				$	$
I.	CONSTRUCTION COSTS				
A.	CIVIL WORKS				
(1)	Lock Construction				
1)	Atlantic Side				
1)-1	Removal of Coffer Dam	m3	500,000	3.52	1,760,000.00
1)-2	Dewatering	ls	1	13,849,450.00	13,849,450.00
1)-3	Backfill	m3	8,500,000	3.85	32,725,000.00
1)-4	Grouting(Consolidation)	m	226,000	86.65	19,582,900.00
1)-5	Concrete	m3	3,600,000	87.68	315,648,000.00
1)-6	Reinfocing Bar	ton	36,000	804.09	28,947,240.00
2)	Pacific Side				
2)-1	Removal of Coffer Dam	m3	500,000	3.52	1,760,000.00
2)-2	Dewatering	ls	1	13,849,450.00	13,849,450.00
2)-3	Backfill	m3	8,500,000	3.85	32,725,000.00
2)-4	Grouting(Consolidation)	m	226,000	86.65	19,582,900.00
2)-5	Concrete	m3	3,600,000	69.24	249,264,000.00
2)-6	Reinforcing Bar	ton	36,000	804.09	28,947,240.00
	Sub Total(1)				758,641,180.00
(2)	Supporting Construction	ls	1		63,289,000.00
(3)	General Conditions				
1)-1	Lock Construction((1)*15.8%)	ls	1		119,865,000.00
1)-2	Supporting Construction((2)*14.3%)	ls	1		9,050,000.00
	Sub Total(3)				128,915,000.00
	Total A				950,845,180.00
B.	PLANT FORMING A PART OF PERMANENT WORKS				
(1)	Atlantic Side				
	Gate Leaves, Facilities	ls	1	443,696,240.00	443,696,240.00
(2)	Pacific Side				
	Gate Leaves, Facilities	ls	1	567,541,110.00	567,541,110.00
	Total B				1,011,237,350.00
	Construction Costs, Total(A+B)				1,962,082,530.00
II.	INDIRECT COSTS (15.8%*{A+B})	ls	1		310,009,000.00
	PROJECT COSTS TOTAL				2,272,091,530.00

(2) Costs of Second Set of New Locks (150,000DWT)

File Name : APPD3

Description		High Rise Lock Canal, 150,000DWT			
		Unit	Q'ty	Unit Cost	Amount
				$	$
I.	CONSTRUCTION COSTS				
A.	CIVIL WORKS				
(1)	Lock Construction				
1)	Atlantic Side				
1)-1	Removal of Coffer Dam	m3	500,000	3.52	1,760,000.00
1)-2	Dewatering	ls	1	13,849,450.00	13,849,450.00
1)-3	Backfill	m3	7,000,000	3.85	26,950,000.00
1)-4	Grouting(Consolidation)	m	200,900	86.65	17,407,985.00
1)-5	Concrete	m3	3,200,000	87.68	280,576,000.00
1)-6	Reinfocing Bar	ton	32,000	804.09	25,730,880.00
2)	Pacific Side				
2)-1	Removal of Coffer Dam	m3	500,000	3.52	1,760,000.00
2)-2	Dewatering	ls	1	13,849,450.00	13,849,450.00
2)-3	Backfill	m3	7,000,000	3.85	26,950,000.00
2)-4	Grouting(Consolidation)	m	200,900	86.65	17,407,985.00
2)-5	Concrete	m3	3,200,000	69.24	221,568,000.00
2)-6	Reinforcing Bar	ton	32,000	804.09	25,730,880.00
	Sub Total(1)				673,540,630.00
(2)	Supporting Construction	ls	1		56,189,000.00
(3)	General Conditions				
1)-1	Lock Construction((1)*15.8%)	ls	1		106,419,000.00
1)-2	Supporting Construction((2)*14.3%)	ls	1		8,035,000.00
	Sub Total(3)				114,454,000.00
	Total A				844,183,630.00
B.	PLANT FORMING A PART OF PERMANENT WORKS				
(1)	Atlantic Side				
	Gate Leaves, Facilities	ls	1		391,793,210.00
(2)	Pacific Side				
	Gate Leaves, Facilities	ls	1		497,094,290.00
	Total B				888,887,500.00
	Construction Costs, Total(A+B)				1,733,071,130.00
II.	INDIRECT COSTS {15.8%*(A+B)}	ls	1		273,825,000.00
	PROJECT COSTS TOTAL				2,006,896,130.00

Appendix 5

Computer Outputs of Water Balance Simulation

A.5-1 (1)Water Balance Simulation of 150,000 DWT in 2020 (Average Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5497.9	26.59	218.2	172.9	11.6	60.5	.0	753.1	76.20	197.7	156.7	11.6	6.9	.0	0.	0.
FEB	5371.5	26.30	101.1	152.7	10.5	64.2	.0	686.4	74.86	96.0	145.1	10.5	7.2	.0	0.	0.
MAR	5197.6	25.91	70.9	161.1	11.6	72.1	.0	572.3	72.33	74.2	168.6	11.6	8.1	.0	0.	0.
APR	5071.1	25.62	81.2	128.7	11.3	67.7	.0	483.2	69.87	120.1	190.3	11.3	7.6	.0	0.	0.
MAY	5060.2	25.59	194.6	146.7	11.6	47.2	.0	526.0	71.05	242.7	182.9	11.6	5.3	.0	0.	0.
JUN	5125.1	25.74	288.2	171.8	11.3	40.2	.0	610.0	73.33	247.0	147.2	11.3	4.5	.0	0.	0.
JUL	5234.6	25.99	333.8	174.3	11.6	38.5	.0	736.3	75.86	297.5	155.3	11.6	4.3	.0	0.	0.
AUG	5495.8	26.59	379.8	184.9	11.6	41.1	.0	753.1	76.20	297.2	144.7	11.6	5.0	.0	0.	0.
SEP	5502.3	26.60	397.7	319.0	11.3	39.1	21.8	753.1	76.20	250.9	.0	11.3	5.0	234.7	0.	0.
OCT	5502.3	26.60	559.5	329.6	11.6	41.5	176.7	753.1	76.20	281.7	.0	11.6	5.3	264.7	0.	0.
NOV	5502.3	26.60	627.5	319.0	11.3	39.6	257.7	753.1	76.20	373.1	.0	11.3	5.2	356.6	0.	0.
DEC	5502.3	26.60	453.8	329.6	11.6	49.9	62.6	753.1	76.20	409.5	.0	11.6	6.8	391.1	0.	0.
			308.8	215.9	11.4	50.1	43.2			240.6	107.6	11.4	5.9	103.9		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-1 (2)Water Balance Simulation of 150,000 DWT in 2020 (Dry Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5422.2	26.42	141.8	149.9	11.6	60.3	.0	725.0	75.64	170.0	179.7	11.6	6.8	.0	0.	0.
FEB	5281.3	26.10	65.7	131.9	10.5	64.2	.0	624.1	73.61	82.6	165.8	10.5	7.2	.0	0.	0.
MAR	5105.4	25.70	46.1	138.2	11.6	72.1	.0	476.8	69.69	63.8	191.4	11.6	8.1	.0	0.	0.
APR	4971.3	25.39	52.8	107.9	11.3	67.7	.0	350.1	65.91	103.3	211.1	11.3	7.6	.0	0.	0.
MAY	4914.5	25.26	126.5	124.4	11.6	47.2	.0	336.7	65.43	208.7	205.2	11.6	5.3	.0	0.	0.
JUN	4900.9	25.23	187.3	149.5	11.3	40.2	.0	363.8	66.41	212.4	169.5	11.3	4.5	.0	0.	0.
JUL	4916.5	25.27	217.0	151.3	11.6	38.5	.0	425.3	68.27	255.8	178.3	11.6	4.3	.0	0.	0.
AUG	4948.8	25.34	246.9	162.0	11.6	41.1	.0	497.0	70.25	255.6	167.7	11.6	4.6	.0	0.	0.
SEP	4983.2	25.42	258.5	173.8	11.3	38.9	.0	552.1	71.77	215.8	145.1	11.3	4.4	.0	0.	0.
OCT	5096.8	25.68	363.7	197.8	11.6	40.6	.0	646.4	74.06	242.2	131.8	11.6	4.6	.0	0.	0.
NOV	5334.3	26.22	407.9	178.5	11.3	38.5	.0	753.1	76.20	320.9	140.5	11.3	4.5	.0	0.	0.
DEC	5502.3	26.60	295.0	150.2	11.6	49.4	70.8	753.1	76.20	352.2	179.4	11.6	6.1	.0	0.	0.
			200.8	151.3	11.4	49.9	5.9			206.9	172.1	11.4	5.7	.0		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-1 (3)Water Balance Simulation of 150,000 DWT in 2020 (Wet Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5502.3	26.60	342.6	269.8	11.6	60.7	.5	753.1	76.20	282.7	59.9	11.6	7.7	203.5	0.	0.
FEB	5426.6	26.43	158.7	159.6	10.5	64.2	.0	734.6	75.83	137.3	138.1	10.5	7.2	.0	0.	0.
MAR	5285.4	26.11	111.3	168.8	11.6	72.1	.0	660.1	74.33	106.1	160.9	11.6	8.1	.0	0.	0.
APR	5197.9	25.91	127.5	135.9	11.3	67.7	.0	629.9	73.72	171.7	183.0	11.3	7.6	.0	0.	0.
MAY	5321.7	26.19	305.5	154.3	11.6	47.2	.0	753.1	76.20	347.0	175.3	11.6	5.4	.0	0.	0.
JUN	5502.3	26.60	452.4	220.1	11.3	40.4	.2	753.1	76.20	353.2	98.9	11.3	5.2	237.9	0.	0.
JUL	5502.3	26.60	524.1	329.6	11.6	39.1	143.7	753.1	76.20	425.4	.0	11.6	5.3	408.4	0.	0.
AUG	5502.3	26.60	596.3	329.6	11.6	42.1	213.0	753.1	76.20	425.0	.0	11.6	5.7	407.7	0.	0.
SEP	5502.3	26.60	624.3	319.0	11.3	40.0	254.1	753.1	76.20	358.8	.0	11.3	5.2	342.4	0.	0.
OCT	5502.3	26.60	878.4	329.6	11.6	42.7	494.5	753.1	76.20	402.8	.0	11.6	5.6	385.6	0.	0.
NOV	5502.3	26.60	985.1	319.0	11.3	40.7	614.2	753.1	76.20	533.5	.0	11.3	5.5	516.7	0.	0.
DEC	5502.3	26.60	712.5	329.6	11.6	51.2	320.1	753.1	76.20	585.6	.0	11.6	7.3	566.8	0.	0.
			484.9	255.4	11.4	50.7	170.0			344.1	68.0	11.4	6.3	255.7		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-2 (1)Water Balance Simulation of 150,000 DWT in 2060 (Average Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5386.9	26.34	218.2	261.7	11.6	60.3	.0	695.3	75.04	197.7	237.1	11.6	6.8	.0	0.	0.
FEB	5182.2	25.87	101.1	231.0	10.5	64.2	.0	554.2	71.83	96.0	219.5	10.5	7.2	.0	0.	0.
MAR	4925.7	25.29	70.9	243.7	11.6	72.1	.0	353.6	66.04	74.2	255.0	11.6	8.1	.0	0.	0.
APR	4842.8	25.10	81.2	194.7	11.3	67.7	.0	214.2	61.00	120.1	287.9	11.3	7.6	.0	157.	0.
MAY	4842.8	25.10	194.6	221.9	11.6	47.2	.0	214.2	61.00	242.7	276.8	11.6	5.3	.0	294.	0.
JUN	4842.8	25.10	288.2	259.9	11.3	40.2	.0	214.2	61.00	247.0	222.7	11.3	4.5	.0	309.	0.
JUL	4862.8	25.15	333.8	263.7	11.6	38.5	.0	260.7	62.68	297.5	235.0	11.6	4.3	.0	309.	0.
AUG	4910.1	25.25	379.8	279.8	11.6	41.1	.0	322.8	64.92	297.2	218.9	11.6	4.6	.0	309.	0.
SEP	4961.7	25.37	397.7	295.9	11.3	38.9	.0	371.4	66.68	250.9	186.7	11.3	4.4	.0	309.	0.
OCT	5137.2	25.77	559.5	331.7	11.6	40.6	.0	469.9	69.50	281.7	167.0	11.6	4.6	.0	309.	0.
NOV	5412.3	26.40	627.5	302.7	11.3	38.5	.0	647.4	74.08	373.1	180.0	11.3	4.3	.0	309.	0.
DEC	5502.3	26.60	453.8	302.4	11.6	49.6	.2	753.1	76.20	409.5	196.3	11.6	5.9	90.0	309.	0.
			308.8	265.8	11.4	49.9	.0			240.6	223.6	11.4	5.6	7.5		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-2 (2)Water Balance Simulation of 150,000 DWT in 2060 (Dry Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5345.3	26.24	141.8	226.8	11.6	60.3	.0	632.8	73.78	170.0	271.9	11.6	6.8	.0	0.	0.
FEB	5136.7	25.77	65.7	199.6	10.5	64.2	.0	446.8	68.86	82.6	250.9	10.5	7.2	.0	0.	0.
MAR	4889.9	25.21	46.1	209.1	11.6	72.1	.0	214.2	61.00	63.8	289.6	11.6	8.1	.0	13.	0.
APR	4842.8	25.10	52.8	163.3	11.3	67.7	.0	214.2	61.00	103.3	319.4	11.3	7.6	.0	390.	0.
MAY	4842.8	25.10	126.5	188.2	11.6	47.2	.0	214.2	61.00	208.7	310.5	11.6	5.3	.0	629.	0.
JUN	4842.8	25.10	187.3	226.2	11.3	40.2	.0	214.2	61.00	212.4	256.5	11.3	4.5	.0	780.	0.
JUL	4842.8	25.10	217.0	228.9	11.6	38.5	.0	214.2	61.00	255.8	269.8	11.6	4.3	.0	872.	0.
AUG	4842.8	25.10	246.9	245.0	11.6	41.1	.0	214.2	61.00	255.6	253.7	11.6	4.6	.0	937.	0.
SEP	4842.8	25.10	258.5	263.0	11.3	38.9	.0	214.2	61.00	215.8	219.6	11.3	4.4	.0	1011.	0.
OCT	4854.8	25.13	363.7	299.3	11.6	40.6	.0	240.9	61.96	242.2	199.4	11.6	4.6	.0	1011.	0.
NOV	4942.9	25.33	407.9	270.1	11.3	38.5	.0	333.7	65.32	320.9	212.5	11.3	4.3	.0	1011.	0.
DEC	4949.5	25.34	295.0	227.3	11.6	49.4	.0	397.3	67.50	352.2	271.4	11.6	5.5	.0	1011.	0.
			200.8	228.9	11.4	49.9	.0			206.9	260.4	11.4	5.6	.0		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-2 (3)Water Balance Simulation of 150,000 DWT in 2060 (Wet Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5502.3	26.60	342.6	273.2	11.6	60.7	35.7	753.1	76.20	282.7	225.5	11.6	7.0	.0	0.	0.
FEB	5344.7	26.24	158.7	241.5	10.5	64.2	.0	663.7	74.40	137.3	209.0	10.5	7.2	.0	0.	0.
MAR	5116.9	25.72	111.3	255.3	11.6	72.1	.0	506.7	70.52	106.1	243.4	11.6	8.1	.0	0.	0.
APR	4959.8	25.37	127.5	205.7	11.3	67.7	.0	382.6	67.09	171.7	276.9	11.3	7.6	.0	0.	0.
MAY	4972.9	25.40	305.5	233.5	11.6	47.2	.0	447.5	68.88	347.0	265.2	11.6	5.3	.0	0.	0.
JUN	5102.8	25.69	452.4	271.0	11.3	40.2	.0	573.3	72.36	353.2	211.6	11.3	4.5	.0	0.	0.
JUL	5307.8	26.16	524.1	275.3	11.6	38.5	.0	753.1	76.20	425.4	223.4	11.6	4.3	.0	0.	0.
AUG	5502.3	26.60	596.3	348.8	11.6	41.2	.2	753.1	76.20	425.0	150.0	11.6	5.3	258.1	0.	0.
SEP	5502.3	26.60	624.3	482.6	11.3	39.4	91.1	753.1	76.20	358.8	.0	11.3	5.2	342.4	0.	0.
OCT	5502.3	26.60	878.4	498.7	11.6	42.1	326.0	753.1	76.20	402.8	.0	11.6	5.6	385.6	0.	0.
NOV	5502.3	26.60	985.1	482.6	11.3	40.2	451.0	753.1	76.20	533.5	.0	11.3	5.5	516.7	0.	0.
DEC	5502.3	26.60	712.5	498.7	11.6	50.4	151.8	753.1	76.20	585.6	.0	11.6	7.3	566.8	0.	0.
			484.9	338.9	11.4	50.3	88.0			344.1	150.4	11.4	6.1	172.5		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-3 (1)Water Balance Simulation of 200,000 DWT in 2020 (Average Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5482.1	26.55	218.2	181.3	11.6	60.5	.0	753.1	76.20	197.7	164.2	11.6	6.9	.0	0.	0.
FEB	5348.4	26.25	101.1	160.0	10.5	64.2	.0	679.4	74.72	96.0	152.1	10.5	7.2	.0	0.	0.
MAR	5166.7	25.84	70.9	168.8	11.6	72.1	.0	557.2	71.91	74.2	176.7	11.6	8.1	.0	0.	0.
APR	5034.0	25.53	81.2	134.9	11.3	67.7	.0	458.9	69.20	120.1	199.4	11.3	7.6	.0	0.	0.
MAY	5016.0	25.49	194.6	153.8	11.6	47.2	.0	492.9	70.14	242.7	191.8	11.6	5.3	.0	0.	0.
JUN	5072.7	25.62	288.2	180.0	11.3	40.2	.0	569.9	72.26	247.0	154.3	11.3	4.5	.0	0.	0.
JUL	5173.7	25.85	333.8	182.7	11.6	38.5	.0	688.6	74.90	297.5	162.8	11.6	4.3	.0	0.	0.
AUG	5371.6	26.30	379.8	193.8	11.6	41.1	.0	753.1	76.20	297.2	151.7	11.6	4.8	.0	0.	0.
SEP	5502.3	26.60	397.7	216.6	11.3	39.0	.0	753.1	76.20	250.9	117.7	11.3	4.7	117.2	0.	0.
OCT	5502.3	26.60	559.5	345.5	11.6	41.4	160.9	753.1	76.20	281.7	.0	11.6	5.3	264.7	0.	0.
NOV	5502.3	26.60	627.5	334.4	11.3	39.5	242.4	753.1	76.20	373.1	.0	11.3	5.2	356.6	0.	0.
DEC	5502.3	26.60	453.8	345.5	11.6	49.9	46.8	753.1	76.20	409.5	.0	11.6	6.8	391.1	0.	0.
			308.8	216.4	11.4	50.1	37.5			240.6	122.6	11.4	5.9	94.1		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-3 (2)Water Balance Simulation of 200,000 DWT in 2020 (Dry Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5415.0	26.40	141.8	157.1	11.6	60.3	.0	716.4	75.46	170.0	188.4	11.6	6.8	.0	0.	0.
FEB	5267.7	26.07	65.7	138.2	10.5	64.2	.0	607.4	73.27	82.6	173.8	10.5	7.2	.0	0.	0.
MAR	5085.1	25.65	46.1	144.9	11.6	72.1	.0	450.9	68.98	63.8	200.6	11.6	8.1	.0	0.	0.
APR	4945.8	25.33	52.8	113.1	11.3	67.7	.0	314.1	64.61	103.3	221.2	11.3	7.6	.0	0.	0.
MAY	4883.1	25.19	126.5	130.4	11.6	47.2	.0	290.7	63.77	208.7	215.1	11.6	5.3	.0	0.	0.
JUN	4862.3	25.14	187.3	156.7	11.3	40.2	.0	309.7	64.45	212.4	177.7	11.3	4.5	.0	0.	0.
JUL	4870.6	25.16	217.0	158.6	11.6	38.5	.0	362.6	66.36	255.8	186.9	11.6	4.3	.0	0.	0.
AUG	4895.0	25.22	246.9	169.8	11.6	41.1	.0	426.3	68.30	255.6	175.7	11.6	4.6	.0	0.	0.
SEP	4921.1	25.28	258.5	182.2	11.3	38.9	.0	474.3	69.62	215.8	152.1	11.3	4.4	.0	0.	0.
OCT	5025.2	25.51	363.7	207.4	11.6	40.6	.0	562.2	72.05	242.2	138.1	11.6	4.6	.0	0.	0.
NOV	5196.2	25.90	407.9	187.1	11.3	38.5	.0	720.3	75.54	320.9	147.2	11.3	4.3	.0	0.	0.
DEC	5386.4	26.34	295.0	157.5	11.6	49.4	.0	753.1	76.20	352.2	188.0	11.6	6.0	.0	0.	0.
			200.8	158.6	11.4	49.9	.0			206.9	180.4	11.4	5.6	.0		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-3 (3)Water Balance Simulation of 200,000 DWT in 2020 (Wet Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5502.3	26.60	342.6	269.8	11.6	60.7	.5	753.1	76.20	282.7	75.7	11.6	7.6	187.7	0.	0.
FEB	5418.9	26.41	158.7	167.3	10.5	64.2	.0	728.0	75.69	137.3	144.8	10.5	7.2	.0	0.	0.
MAR	5269.6	26.07	111.3	176.9	11.6	72.1	.0	645.7	74.04	106.1	168.6	11.6	8.1	.0	0.	0.
APR	5175.6	25.86	127.5	142.5	11.3	67.7	.0	606.7	73.26	171.7	191.9	11.3	7.6	.0	0.	0.
MAY	5260.5	26.05	305.5	161.8	11.6	47.2	.0	753.0	76.20	347.0	183.7	11.6	5.3	.0	0.	0.
JUN	5502.3	26.60	452.4	187.8	11.3	40.3	161.5	753.1	76.20	353.2	146.6	11.3	5.1	.0	0.	0.
JUL	5502.3	26.60	524.1	345.5	11.6	39.1	127.9	753.1	76.20	425.4	.0	11.6	5.3	408.4	0.	0.
AUG	5502.3	26.60	596.3	345.5	11.6	42.0	197.2	753.1	76.20	425.0	.0	11.6	5.7	407.7	0.	0.
SEP	5502.3	26.60	624.3	334.4	11.3	40.0	238.8	753.1	76.20	358.8	.0	11.3	5.2	342.4	0.	0.
OCT	5502.3	26.60	878.4	345.5	11.6	42.6	478.7	753.1	76.20	402.8	.0	11.6	5.6	385.6	0.	0.
NOV	5502.3	26.60	985.1	334.4	11.3	40.7	598.8	753.1	76.20	533.5	.0	11.3	5.5	516.7	0.	0.
DEC	5502.3	26.60	712.5	345.5	11.6	51.1	304.3	753.1	76.20	585.6	.0	11.6	7.3	566.8	0.	0.
			484.9	263.1	11.4	50.6	175.6			344.1	75.9	11.4	6.3	234.6		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-4 (1)Water Balance Simulation of 200,000 DWT in 2060 (Average Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5367.7	26.29	218.2	280.9	11.6	60.3	.0	677.9	74.69	197.7	254.5	11.6	6.8	.0	0.	0.
FEB	5146.1	25.79	101.1	247.9	10.5	64.2	.0	520.7	70.90	96.0	235.6	10.5	7.2	.0	0.	0.
MAR	4871.6	25.17	70.9	261.6	11.6	72.1	.0	301.4	64.15	74.2	273.7	11.6	8.1	.0	0.	0.
APR	4842.8	25.10	81.2	209.0	11.3	67.7	.0	214.2	61.00	120.1	309.0	11.3	7.6	.0	299.	0.
MAY	4842.8	25.10	194.6	238.2	11.6	47.2	.0	214.2	61.00	242.7	297.1	11.6	5.3	.0	472.	0.
JUN	4842.8	25.10	288.2	279.0	11.3	40.2	.0	214.2	61.00	247.0	239.1	11.3	4.5	.0	523.	0.
JUL	4843.4	25.10	333.8	283.1	11.6	38.5	.0	243.5	62.06	297.5	252.2	11.6	4.3	.0	523.	0.
AUG	4870.2	25.16	379.8	300.3	11.6	41.1	.0	289.5	63.72	297.2	235.0	11.6	4.6	.0	523.	0.
SEP	4900.1	25.23	397.7	317.6	11.3	38.9	.0	324.4	64.98	250.9	200.4	11.3	4.4	.0	523.	0.
OCT	5051.3	25.57	559.5	356.1	11.6	40.6	.0	410.6	67.86	281.7	179.3	11.6	4.6	.0	523.	0.
NOV	5304.1	26.15	627.5	324.9	11.3	38.5	.0	575.0	72.40	373.1	193.2	11.3	4.3	.0	523.	0.
DEC	5415.6	26.40	453.8	281.4	11.6	49.4	.0	713.4	75.40	409.5	253.9	11.6	5.5	.0	523.	0.
			308.8	281.7	11.4	49.9	.0			240.6	243.6	11.4	5.6	.0		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-4 (2)Water Balance Simulation of 200,000 DWT in 2060 (Dry Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5328.7	26.21	141.8	243.5	11.6	60.3	.0	612.9	73.38	170.0	291.8	11.6	6.8	.0	0.	0.
FEB	5105.4	25.70	65.7	214.2	10.5	64.2	.0	408.5	67.80	82.6	269.3	10.5	7.2	.0	0.	0.
MAR	4843.3	25.10	46.1	224.5	11.6	72.1	.0	214.2	61.00	63.8	310.8	11.6	8.1	.0	73.	0.
APR	4842.8	25.10	52.8	175.3	11.3	67.7	.0	214.2	61.00	103.3	342.8	11.3	7.6	.0	532.	0.
MAY	4842.8	25.10	126.5	202.0	11.6	47.2	.0	214.2	61.00	208.7	333.3	11.6	5.3	.0	808.	0.
JUN	4842.8	25.10	187.3	242.8	11.3	40.2	.0	214.2	61.00	212.4	275.3	11.3	4.5	.0	993.	0.
JUL	4842.8	25.10	217.0	245.7	11.6	38.5	.0	214.2	61.00	255.8	289.6	11.6	4.3	.0	1122.	0.
AUG	4842.8	25.10	246.9	263.0	11.6	41.1	.0	214.2	61.00	255.6	272.3	11.6	4.6	.0	1224.	0.
SEP	4842.8	25.10	258.5	282.3	11.3	38.9	.0	214.2	61.00	215.8	235.7	11.3	4.4	.0	1333.	0.
OCT	4842.8	25.10	363.7	321.3	11.6	40.6	.0	216.3	61.43	242.2	214.0	11.6	4.6	.0	1333.	0.
NOV	4911.0	25.26	407.9	289.9	11.3	38.5	.0	293.5	63.87	320.9	228.1	11.3	4.3	.0	1333.	0.
DEC	4900.9	25.23	295.0	244.0	11.6	49.4	.0	337.2	65.45	352.2	291.3	11.6	5.5	.0	1333.	0.
			200.8	245.7	11.4	49.9	.0			206.9	279.5	11.4	5.6	.0		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-4 (3)Water Balance Simulation of 200,000 DWT in 2060 (Wet Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	5501.5	26.60	342.6	293.3	11.6	60.5	.0	753.1	76.20	282.7	242.0	11.6	6.9	.0	0.	0.
FEB	5326.3	26.20	158.7	259.2	10.5	64.2	.0	648.4	74.10	137.3	224.3	10.5	7.2	.0	0.	0.
MAR	5079.7	25.64	111.3	274.1	11.6	72.1	.0	473.5	69.60	106.1	261.2	11.6	8.1	.0	0.	0.
APR	4907.4	25.25	127.5	220.8	11.3	67.7	.0	329.1	65.15	171.7	297.3	11.3	7.6	.0	0.	0.
MAY	4903.5	25.24	305.5	250.6	11.6	47.2	.0	374.5	66.79	347.0	284.7	11.6	5.3	.0	0.	0.
JUN	5013.5	25.49	452.4	290.9	11.3	40.2	.0	484.8	69.91	353.2	227.1	11.3	4.5	.0	0.	0.
JUL	5192.0	25.89	524.1	295.5	11.6	38.5	.0	654.4	74.22	425.4	239.8	11.6	4.3	.0	0.	0.
AUG	5502.3	26.60	596.3	312.6	11.6	41.1	7.9	753.1	76.20	425.0	222.8	11.6	4.9	.0	0.	0.
SEP	5502.3	26.60	624.3	518.1	11.3	39.3	55.8	753.1	76.20	358.8	.0	11.3	5.2	342.4	0.	0.
OCT	5502.3	26.60	878.4	535.3	11.6	42.0	289.5	753.1	76.20	402.8	.0	11.6	5.6	385.6	0.	0.
NOV	5502.3	26.60	985.1	518.1	11.3	40.1	415.7	753.1	76.20	533.5	.0	11.3	5.5	516.7	0.	0.
DEC	5502.3	26.60	712.5	535.3	11.6	50.2	115.3	753.1	76.20	585.6	.0	11.6	7.3	566.8	0.	0.
			484.9	358.6	11.4	50.3	73.7			344.1	166.6	11.4	6.0	151.0		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-5 (1)Water Balance Simulation of 65,000 DWT in 2020 (Average Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	4635.8	26.63	131.2	97.2	11.6	44.5	10.8	753.1	76.20	197.7	146.3	11.6	6.9	.0	0.	0.
FEB	4553.6	26.35	60.8	85.2	10.5	47.3	.0	696.8	75.07	96.0	134.7	10.5	7.2	.0	0.	0.
MAR	4442.7	26.02	42.6	88.9	11.6	53.1	.0	596.6	73.00	74.2	154.6	11.6	8.1	.0	0.	0.
APR	4362.3	25.78	48.8	68.1	11.3	49.8	.0	530.3	71.17	120.1	167.5	11.3	7.6	.0	0.	0.
MAY	4353.7	25.76	117.0	79.2	11.6	34.7	.0	591.8	72.87	242.7	164.3	11.6	5.3	.0	0.	0.
JUN	4389.0	25.86	173.3	97.2	11.3	29.6	.0	684.5	74.82	247.0	138.5	11.3	4.5	.0	0.	0.
JUL	4519.2	26.25	200.8	98.1	11.6	28.3	.0	753.1	76.20	297.5	145.4	11.6	4.5	.0	0.	0.
AUG	4635.8	26.92	228.5	105.8	11.6	30.2	107.0	753.1	76.20	297.2	137.7	11.6	5.1	.0	0.	0.
SEP	4635.8	26.60	239.2	199.0	11.3	28.7	.2	753.1	76.20	250.9	36.6	11.3	4.9	198.1	0.	0.
OCT	4635.8	26.60	336.5	243.5	11.6	30.1	51.3	753.1	76.20	281.7	.0	11.6	5.3	264.7	0.	0.
NOV	4635.8	26.60	377.4	235.6	11.3	28.6	101.9	753.1	76.20	373.1	.0	11.3	5.2	356.6	0.	0.
DEC	4635.8	26.60	273.0	224.5	11.6	36.4	.4	753.1	76.20	409.5	19.0	11.6	6.7	372.2	0.	0.
			185.8	135.2	11.4	36.8	22.6			240.6	103.7	11.4	5.9	99.3		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of the water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-5 (2)Water Balance Simulation of 65,000 DWT in 2020 (Dry Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	4583.8	26.44	85.3	81.4	11.6	44.4	.0	742.6	75.99	170.0	162.1	11.6	6.8	.0	0.	0.
FEB	4494.3	26.18	39.5	71.2	10.5	47.3	.0	658.7	74.30	82.6	148.8	10.5	7.2	.0	0.	0.
MAR	4383.6	25.84	27.7	73.7	11.6	53.1	.0	533.0	71.24	63.8	169.8	11.6	8.1	.0	0.	0.
APR	4298.9	25.59	31.8	55.4	11.3	49.8	.0	437.2	68.60	103.3	180.2	11.3	7.6	.0	0.	0.
MAY	4263.5	25.49	76.1	65.0	11.6	34.7	.0	450.5	68.97	208.7	178.5	11.6	5.3	.0	0.	0.
JUN	4253.7	25.46	112.7	81.7	11.3	29.6	.0	493.2	70.14	212.4	154.0	11.3	4.5	.0	0.	0.
JUL	4262.0	25.48	130.5	82.3	11.6	28.3	.0	571.9	72.32	255.8	161.2	11.6	4.3	.0	0.	0.
AUG	4279.2	25.53	148.5	89.5	11.6	30.2	.0	657.2	74.27	255.6	154.0	11.6	4.6	.0	0.	0.
SEP	4296.1	25.58	155.5	98.7	11.3	28.6	.0	720.4	75.54	215.8	137.0	11.3	4.4	.0	0.	0.
OCT	4423.0	25.96	218.7	115.5	11.6	29.9	.0	753.1	76.20	242.2	128.0	11.6	4.8	.0	0.	0.
NOV	4635.8	26.79	245.3	102.1	11.3	28.3	62.1	753.1	76.20	320.9	133.5	11.3	4.8	.0	0.	0.
DEC	4635.8	26.60	177.4	129.2	11.6	36.4	.2	753.1	76.20	352.2	114.3	11.6	6.3	220.0	0.	0.
			120.7	87.1	11.4	36.7	5.2			206.9	151.8	11.4	5.7	18.3		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of the water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-5 (3)Water Balance Simulation of 65,000 DWT in 2020 (Wet Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	4635.8	26.60	206.0	149.7	11.6	44.5	.2	753.1	76.20	282.7	93.8	11.6	7.6	169.7	0.	0.
FEB	4583.3	26.44	95.4	90.2	10.5	47.3	.0	743.0	76.00	137.3	129.8	10.5	7.2	.0	0.	0.
MAR	4491.3	26.17	66.9	94.2	11.6	53.1	.0	680.0	74.73	106.1	149.3	11.6	8.1	.0	0.	0.
APR	4434.2	26.00	76.7	72.8	11.3	49.8	.0	670.0	74.53	171.7	162.9	11.3	7.6	.0	0.	0.
MAY	4574.7	26.42	183.7	84.3	11.6	34.7	.0	753.1	76.20	347.0	159.2	11.6	5.6	.0	0.	0.
JUN	4635.8	26.60	272.1	170.0	11.3	29.7	.2	753.1	76.20	353.2	65.7	11.3	5.3	271.0	0.	0.
JUL	4635.8	26.60	315.2	243.5	11.6	28.4	31.7	753.1	76.20	425.4	.0	11.6	5.3	408.4	0.	0.
AUG	4635.8	26.60	358.7	243.5	11.6	30.5	73.1	753.1	76.20	425.0	.0	11.6	5.7	407.7	0.	0.
SEP	4635.8	26.60	375.5	235.6	11.3	28.9	99.7	753.1	76.20	358.8	.0	11.3	5.2	342.4	0.	0.
OCT	4635.8	26.60	528.3	243.5	11.6	30.6	242.6	753.1	76.20	402.8	.0	11.6	5.6	385.6	0.	0.
NOV	4635.8	26.60	592.5	235.6	11.3	29.1	316.6	753.1	76.20	533.5	.0	11.3	5.5	516.7	0.	0.
DEC	4635.8	26.60	428.5	243.5	11.6	36.9	136.5	753.1	76.20	585.6	.0	11.6	7.3	566.8	0.	0.
			291.6	175.5	11.4	37.0	75.0			344.1	63.4	11.4	6.3	255.7		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of the water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-6 (1)Water Balance Simulation of 65,000 DWT in 2060 (Average Year)

	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
MONTH	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	4598.6	26.49	131.2	116.4	11.6	44.4	.0	753.1	76.20	197.7	175.3	11.6	6.8	.0	0.	0.
FEB	4499.5	26.19	60.8	102.1	10.5	47.3	.0	670.1	74.53	96.0	161.3	10.5	7.2	.0	0.	0.
MAR	4371.0	25.81	42.6	106.4	11.6	53.1	.0	539.4	71.42	74.2	185.2	11.6	8.1	.0	0.	0.
APR	4277.2	25.53	48.8	81.6	11.3	49.8	.0	440.0	68.67	120.1	200.6	11.3	7.6	.0	0.	0.
MAY	4253.0	25.45	117.0	94.9	11.6	34.7	.0	469.0	69.47	242.7	196.8	11.6	5.3	.0	0.	0.
JUN	4269.1	25.50	173.3	116.4	11.3	29.6	.0	534.3	71.28	247.0	165.9	11.3	4.5	.0	0.	0.
JUL	4312.4	25.63	200.8	117.5	11.6	28.3	.0	641.7	73.96	297.5	174.1	11.6	4.3	.0	0.	0.
AUG	4376.9	25.82	228.5	126.8	11.6	30.2	.0	753.1	76.20	297.2	164.9	11.6	4.6	.0	0.	0.
SEP	4529.0	26.28	239.2	137.7	11.3	28.6	.0	753.1	76.20	250.9	144.5	11.3	4.6	.0	0.	0.
OCT	4635.8	26.60	336.5	188.1	11.6	30.0	.1	753.1	76.20	281.7	103.6	11.6	5.1	161.4	0.	0.
NOV	4635.8	26.60	377.4	282.2	11.3	28.5	55.4	753.1	76.20	373.1	.0	11.3	5.2	356.6	0.	0.
DEC	4635.8	26.60	273.0	224.5	11.6	36.4	.3	753.1	76.20	409.5	67.1	11.6	6.6	324.2	0.	0.
			185.8	141.2	11.4	36.7	4.7			240.6	144.9	11.4	5.8	70.2		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of the water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-6 (2)Water Balance Simulation of 65,000 DWT in 2060 (Dry Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	4567.7	26.40	85.3	97.4	11.6	44.4	.0	710.5	75.34	170.0	194.2	11.6	6.8	.0	0.	0.
FEB	4464.2	26.09	39.5	85.2	10.5	47.3	.0	597.2	73.02	82.6	178.2	10.5	7.2	.0	0.	0.
MAR	4338.9	25.71	27.7	88.3	11.6	53.1	.0	438.0	68.62	63.8	203.3	11.6	8.1	.0	0.	0.
APR	4243.2	25.42	31.8	66.4	11.3	49.8	.0	306.5	64.34	103.3	215.9	11.3	7.6	.0	0.	0.
MAY	4195.0	25.28	76.1	77.9	11.6	34.7	.0	284.5	63.54	208.7	213.7	11.6	5.3	.0	0.	0.
JUN	4169.0	25.20	112.7	97.8	11.3	29.6	.0	296.8	63.98	212.4	184.4	11.3	4.5	.0	0.	0.
JUL	4161.0	25.18	130.5	98.5	11.6	28.3	.0	343.5	65.67	255.8	193.1	11.6	4.3	.0	0.	0.
AUG	4160.5	25.18	148.5	107.2	11.6	30.2	.0	398.4	67.53	255.6	184.5	11.6	4.6	.0	0.	0.
SEP	4157.9	25.17	155.5	118.2	11.3	28.6	.0	434.6	68.52	215.8	164.1	11.3	4.4	.0	0.	0.
OCT	4196.7	25.29	218.7	138.4	11.6	29.9	.0	507.3	70.54	242.2	153.3	11.6	4.6	.0	0.	0.
NOV	4280.2	25.54	245.3	122.3	11.3	28.3	.0	652.7	74.18	320.9	160.0	11.3	4.3	.0	0.	0.
DEC	4352.5	25.75	177.4	97.7	11.6	36.3	.0	753.1	76.20	352.2	194.0	11.6	5.7	.0	0.	0.
			120.7	99.6	11.4	36.7	.0			206.9	186.5	11.4	5.6	.0		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of the water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

A.5-6 (3)Water Balance Simulation of 65,000 DWT in 2060 (Wet Year)

MONTH	****** GATUN DAM ******							****** MADDEN DAM ******							*NEW DAM*	*PUMP*
	VOL. ①	WL. ②	ROFF ③	DEM.L ④	DEM.E ⑤	EVP. ⑥	SPIL ⑦	VOL. ⑧	WL. ⑨	ROFF ⑩	DEM.L ⑪	DEM.E ⑫	EVP. ⑬	SPIL ⑭	⑮	⑯
JAN	4635.8	26.60	206.0	149.8	11.6	44.5	.1	753.1	76.20	282.7	141.9	11.6	7.4	121.8	0.	0.
FEB	4565.5	26.39	95.4	108.0	10.5	47.3	.0	717.3	75.48	137.3	155.4	10.5	7.2	.0	0.	0.
MAR	4454.9	26.06	66.9	112.8	11.6	53.1	.0	624.8	73.62	106.1	178.8	11.6	8.1	.0	0.	0.
APR	4383.3	25.84	76.7	87.2	11.3	49.8	.0	582.6	72.61	171.7	195.1	11.3	7.6	.0	0.	0.
MAY	4419.8	25.95	183.7	101.0	11.6	34.7	.0	722.0	75.58	347.0	190.7	11.6	5.3	.0	0.	0.
JUN	4635.8	26.72	272.1	122.8	11.3	29.6	38.8	753.1	76.20	353.2	159.4	11.3	5.0	.0	0.	0.
JUL	4635.8	26.60	315.2	274.9	11.6	28.4	.4	753.1	76.20	425.4	16.8	11.6	5.3	391.7	0.	0.
AUG	4635.8	26.60	358.7	291.7	11.6	30.4	25.0	753.1	76.20	425.0	.0	11.6	5.7	407.7	0.	0.
SEP	4635.8	26.60	375.5	282.2	11.3	28.8	53.2	753.1	76.20	358.8	.0	11.3	5.2	342.4	0.	0.
OCT	4635.8	26.60	528.3	291.7	11.6	30.5	194.6	753.1	76.20	402.8	.0	11.6	5.6	385.6	0.	0.
NOV	4635.8	26.60	592.5	282.2	11.3	29.0	270.0	753.1	76.20	533.5	.0	11.3	5.5	516.7	0.	0.
DEC	4635.8	26.60	428.5	291.7	11.6	36.7	88.5	753.1	76.20	585.6	.0	11.6	7.3	566.8	0.	0.
			291.6	199.7	11.4	36.9	55.9			344.1	86.5	11.4	6.3	227.7		

Note (1)Gatun and Madden dams are assumed to be full at beginning of January, because this corresponds to the end of the wet season.
(2)The numbers show the results of the water balance at end of each month.
(3)Numbers at column headings are explained as follows.

①Gatun Dam Capacity(MCM)
②Gatun Dam Level (EL.m)
③Runoff From Gatun Watershed(MCM)
④Lockage Water From Gatun Dam(MCM)
⑤Municipal Water From Gatun Dam(MCM)
⑥Evaporation From Gatun Dam(MCM)
⑦Spillage From Gatun Dam(MCM)
⑧Madden Dam Capacity(MCM)
⑨Madden Dam Water Level(EL.m)
⑩Runoff From Madden Watershed(MCM)
⑪Lockage Water From Madden Dam(MCM)
⑫Municipal Water From Madden Dam(MCM)
⑬Evaporation From Madden Dam(MCM)
⑭Spillage From Madden Dam(MCM)
⑮New Dam Capacity To Be Planned(MCM)
⑯Pump Capacity To Be Planned(MCM)

APPENDIX 5 - COMPUTER OUTPUTS OF WATER BALANCE SIMULATION

(1) High Rise Case (85ft)

A.5-1(1) 150,000 DWT Year 2020 (Average)
(2) 150,000 DWT Year 2020 (Dry)
(3) 150,000 DWT Year 2020 (Wet)

A.5-2(1) 150,000 DWT Year 2060 (Average)
(2) 150,000 DWT Year 2060 (Dry)
(3) 150,000 DWT Year 2060 (wet)

A.5-3(1) 200,000 DWT Year 2020 (Average)
(2) 200,000 DWT Year 2020 (Dry)
(3) 200,000 DWT Year 2020 (Wet)

A.5-4(1) 200,000 DWT Year 2060 (Average)
(2) 200,000 DWT Year 2060 (Dry)
(3) 200,000 DWT Year 2060 (Wet)

(2) Sea Level Case (SQ)

A.5-5(1) 65,000 DWT Year 2020 (Average)
(2) 65,000 DWT Year 2020 (Dry)
(3) 65,000 DWT Year 2020 (Wet)

A.5-6(1) 65,000 DWT Year 2060 (Average)
(2) 65,000 DWT Year 2060 (Dry)
(3) 65,000 DWT Year 2060 (Wet)

BASE MAPS

BASE MAPS