Diseño Conceptual de las Esclusas Post Panamax

TRIPLE LIFT LOCK SYSTEM

TASK 2 -DESIGN CRITERIA

Tractebel Development Engineering
TECHNUM
COYNE ET BELLIER
CNR PROJETS

in association with

IMDC
SBE
FIRA AOSO

contents

A GENERAL DESIGN CRITERIA
B SPECIFIC DESIGN CRITERIA

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TASK 2

PART A: GENERAL DESIGN CRITERIA

R2-A

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1	**General**	**1-1**
1.1	**SCOPE**	**1-1**
1.2	**DATA**	**1-3**
1.2.1	*TERMS OF REFERENCE (TOR)*	*1-3*
1.2.2	*DOCUMENTS FURNISHED BY ACP (TILL THE DATE OF 31-03-02)*	*1-3*
1.2.3	*DOCUMENTS FURNISHED BY ACP AFTER 31-03-02*	*1-5*
1.3	**STANDARDS**	**1-6**
1.4	**DIMENSIONS**	**1-7**
1.4.1	*SHIP DIMENSIONS*	*1-7*
1.4.2	*LOCK DIMENSIONS*	*1-8*
1.5	**MAIN LEVELS**	**1-9**
1.6	**OTHER REQUIREMENTS IMPOSED BY THE TOR**	**1-9**
1.6.1	*GENERAL REQUIREMENTS*	*1-9*
1.6.2	*REPAIR AND MAINTENANCE*	*1-9*
2	**Hydraulic Design Criteria**	**2-1**
2.1	**DOCUMENTS FURNISHED BY ACP**	**2-1**
2.2	**SPECIFIC DESIGN TOOLS**	**2-1**
2.3	**PROPERTIES OF WATER**	**2-2**
2.4	**WATER SAVING BASIN DIMENSIONS**	**2-2**
2.5	**LOSS COEFFICIENTS**	**2-3**
2.6	**WATER VELOCITIES IN THE CONDUITS**	**2-4**
2.7	**VALVES**	**2-4**
2.7.1	*DIMENSIONS*	*2-4*
2.7.2	*RATIO VALVE AREA / CULVERT OR PIPE AREA*	*2-4*
2.7.3	*OPENING AND CLOSING TIME OF VALVES*	*2-5*
2.7.4	*GUARANTY OF SERVICEABILITY*	*2-5*
2.8	**CAVITATION AND AIR DEMAND**	**2-5**
2.9	**VORTEX AND FLOATING DEBRIS**	**2-6**
2.10	**SENSITIVITY ANALYSIS**	**2-6**
2.10.1	*BASINS*	*2-6*
2.10.2	*REMAINING HEAD BEFORE EQUALIZATION*	*2-7*

3	**Structural Design Criteria**	**3-1**
3.1	**GENERAL**	**3-1**
3.2	**MATERIAL PROPERTIES**	**3-1**
3.2.1	*CONCRETE*	*3-1*
3.2.2	*REINFORCING STEEL*	*3-2*
3.2.3	*STRUCTURAL STEEL*	*3-2*
3.2.4	*SOIL AND ROCK*	*3-2*
3.2.5	*GROUT ANCHORS*	*3-3*
3.2.6	*SHEET PILING*	*3-3*
3.2.7	*PILES*	*3-3*
3.3	**LIMIT STATES**	**3-3**
3.4	**LOADS**	**3-4**
3.4.1	*DEAD LOADS*	*3-4*
3.4.2	*HYDROSTATIC LOADS*	*3-4*
3.4.3	*EARTH*	*3-5*
3.4.4	*WIND*	*3-5*
3.4.5	*TEMPERATURE AND SHRINKAGE*	*3-5*
3.4.6	*WAVES*	*3-5*
3.4.7	*COLLISION*	*3-6*
3.4.8	*SEISMIC LOADS*	*3-6*
3.5	**SAFETY FACTORS**	**3-6**
3.5.1	*SEMI PROBABILISTIC APPROACH – PARTIAL SAFETY FACTORS*	*3-6*
3.5.2	*GLOBAL SAFETY FACTORS*	*3-7*
3.6	**LOAD COMBINATIONS**	**3-7**
4	**Geotechnical & Seismic Parameters**	**4-1**
4.1	**GEOLOGY – GEOTECHNICS**	**4-1**
4.1.1	*EXECUTIVE SUMMARY*	*4-1*
4.1.2	*GEOLOGY*	*4-2*
4.1.3	*GEOTECHNICS*	*4-8*
4.1.4	*ENGINEERING GEOLOGY*	*4-13*
4.1.5	*CONCLUSIONS*	*4-18*
4.1.6	*REFERENCES - DOCUMENTATION*	*4-19*
4.2	**SEISMIC HAZARD ANALYSIS**	**4-22**
4.2.1	*EXECUTIVE SUMMARY*	*4-22*
4.2.2	*INTRODUCTION*	*4-23*
4.2.3	*TECHNICAL DOCUMENTS CONSULTED*	*4-24*
4.2.4	*GEODYNAMIC FRAMEWORK*	*4-25*
4.2.5	*CRITICAL SEISMOGENIC SOURCES AND SEISMICITY*	*4-27*
4.2.6	*SEISMIC HAZARD ASSESSMENT*	*4-32*
4.2.7	*CONCLUSIONS*	*4-41*

5 Electrical and Power Requirements 5-1

5.1 DESIGN CRITERIA . 5-1

5.2 STANDARDS - REFERENCES 5-1

5.2.1 ELECTROMAGNETIC COMPATIBILITY: 5-1
5.2.2 MV SWITCHGEAR 5-2
5.2.3 TRANSFORMERS 5-2
5.2.4 LOW VOLTAGE SWITCHGEAR 400V AC 5-2
5.2.5 CABLES 5-3
5.2.6 DIESEL EMERGENCY SET(S) 5-3
5.2.7 UPS – RECTIFIER - CHARGER 5-4
5.2.8 BASIC DESIGN OF THE ELECTRICAL INSTALLATIONS 5-4

1 General

1.1 SCOPE

Task 2 of this conceptual design study for two lock configurations covers the development of the design criteria. For each project feature identified in task 4 proper design criteria have to be developed.
These design criteria will be based on:

- Properly recognized standards (International)
- The terms of reference (ACP)
- The data furnished by ACP
- Visits and meetings in Panama 13-15 February 2002

This draft final report on design criteria deals especially with the configuration of a 3-lift lock concept, as this is the first configuration that will be studied. Specific design criteria for the single lift configuration different from those for the triple lift configuration will be the subject of a separate report which will be part of the final report of the second configuration.

The design criteria have to be considered as applicable for detailed design purposes. As this actual study is a concept design, which precedes all other engineering activities as indicated in the project schedule below (ACP-origin), the design criteria will be implemented as far as this is required on this actual concept design level.

CONCEPTUAL DESIGN STUDY – 2002
(actual contract)

RECONNAISSANCE OR PREFEASIBILITY

FEASIBILITY

CONSULTATION & DECISION

FINAL DESIGN

BUILDING

COMMISSIONING - 2010

1.2 DATA

1.2.1 TERMS OF REFERENCE (TOR)

Licitación N°. SAA-109422
Pliego de Cargas para "Diseño conceptual de las Esclusas Post Panamax" by ACP

1.2.2 DOCUMENTS FURNISHED BY ACP (TILL THE DATE OF 31-03-02)

1. **ACP reports on seismic activity:**
   - Report: Seismicity Evaluation Tabasara Hydel Project western Panama, ACRES Canada dec. 81
   - Report: Excerpts from the Star and Herald on the Sept. 1882 Earthquake, from Engineering & Construction Bureau, Luis Carlos Fernandez.
   - Report: El terremoto de San Blas del 7 de Septiembre 1882; E Camacho y V. Viquez, Universidad de Panama , Junio 1993
   - Technical Report 2-17: Spectral Strong Motion Attenuation in Central America, NORSAR , August 1994
   - Technical Report 2-18: Seismic Hazard for Panama, Update, NORSAR, July 1994
   - Report: Seismicity of Panama during the interval 1904-83, Seismological Dept. Upsala Sweden, A. Vergara Munoz, 1987
   - Report: El terremoto de la Vieja del 2 de Mayo 1621, un sismo intraplaca, V. Viquez y E. Camacho, Universidad de Panama, Mayo 1993
   - Report. Historical Seismicity of the North Panama Deformed Belt, E Camacho y V. Viquez, Universidad de Panama ,
   - Report: Seismicidad Historica del Extremo Occidental del Cinturon Deformado del Norte de Panama, E Camacho y V. Viquez, Universidad de Panama , Abril 1992
2. **Geotechnical information**
   - Plan view
   - 1 set of 4 Maps en color: "Mapa geologico Republica de Panama", scale 1/250 000 Ministerio de comercio e industrios, hojas 3, 4, 5 y 6
   - Geotechnical Logs hardcopies with digital files
   - Drawing with Logs showing Alignments P1 and P2:
     Alineamiento P1 y P2 Sector Pacifico, scale 1/5000 Ubicacion de sondeos, February 2002

- 3 geological longitudinal profiles
  - Profile 1 (East)
  - Profile 2 (Center)
  - Profile 3 (West)

3. Preliminary locks profile drawing

- New Panama canal post-panamax locks profile diagram, 06/02/2002
- 2 drawings (digital) showing 1-step and 3-step lock profiles

4. Ground Survey Information – digital topographic map in X,Y,Z coordinates

5. Moffatt & Nichol study on water saving basins Draft final Report
   Mofatt & Nichol study on water saving basins – appendices A - J

6. Harza Lock Alignment Study Report

7. Data of size & type of gates (see n° 12 and PIANC Bulletin)

8. Data on the water management (resources)
   (See questions 48 & 102 of the pre bid conference)

9. Inventory of disposal sites for excavated soil and rock, including capacity and restrictions.

10. Data on existing road access (drawing) = item n°4

11. Pacific & Gatún Lake elevations:

a) Tidal Data (1991-1999) every 15 minutes (Temporal evolution of the ocean's level)

b) Gatun Elevations (1996-2000) at midnight

12. Presentation of ACP (Kick-off meeting 14/02/02)

13. Table of "Mareas Balboa"

14. Pilot Handbook

15. Real view with HARZA alignment

16. Texas A&M Report (vessel positioning systems)

17. Ship Squad Study Report

18. Emptying and filling system report:
    The third locks project of the Panama Canal, lock model tests – Design 3, August 1942

19. Data on wave propagation due to ship movement:
    Pressure Test Miraflores Locks (Pressure sensors in the chambers during the passage of boats)

20. **Lighting system / Electricity feeding**
    Plans of Electrical distribution, Single Line Diagrams (Electrical Location Plan and Diagram) for:

    a) Pedro Miguel Locks - June 19, 1996
    b) Miraflores Locks – July 31, 1995
    c) Gatun Locks - July 14, 1998

21. Temporal Evolution of the rain and wind
    "Balance hídrico superficial en la cuenca del canal de Panama", Preliminary Report, March of 2000.

22. Handbook of Lockage Procedure Locks Division

23. Coating type in use

- Specifications for Corrosion Control Coatings
- Plans of:
  a) Miraflores locks (10 sheets) – Corrosion Mitigation part Plan and Inspection Records - October 12, 1983
  b) All Locks (Pedro Miguel, Miraflores) – Intermediate Gates Cathodic Protection Outline of Gate Recess, UHM WHDP Hanger, Bill of Materials, and Sections. September 30, 1999
  c) All Locks – Cathodic Protection Mitre Gates, Water Compartments, 4 March 1967

24. Third locks construction effort

25. Data missing from the Moffatt & Nichol preliminary study on water saving basins

- Table of figure 11.15 for one month comparison of measured versus predicted tide level in the Pacific (page 15 of main report).
- Table is for the Pacific excedance on Appendix C
  "Finalized Percent Exceedance Distribution for Balboa Gage - Pacific Ocean Side"

26. Lock's Operation
    Comments about regular operation procedures by John Wong

27. Data about talus:
    "Slope criteria for conventional excavation2.dwg"

28. Description of rock
    Rock Data Tables "Definición, Criterio de Rocas y Escala de Dureza para Aplicación de Curvas"

29. Tests of Permeability Alignment P1 & P2

30. Information on the lighting system (existing high mast) at Panama Canal Locks

31. Indice of aerial orthophotos available

32. Data about loads

- 2 Autocad drawings showing the position of locomotives
  (typical section of either lateral and central walls),
- 1 Autocad drawing detailing the loads of the existing locomotives on the lock wall

33. Actual lockage times for vessel movement, filling/emptying, gate operation

1.2.3 DOCUMENTS FURNISHED BY ACP AFTER 31-03-02

34. Ortho photos (Corozal Lacona, Balboa Rio Cocoli, Clayton Esclusas 1-2-3)

35. Hard copy of third locks construction effort drawings (1939)

36. Summary of results of tests on Rock formations for boring TP1 (August 2001 and TP1C (January 2001)

37a. Report “Pump Saltwater to Gatun Lake”

37b. Report “Recycling Ponds”

37c. Report “Salinity Intrusion in the Panama Canal”

38a. General drawings showing Machinery Chambers Location (6124-6125-6126

38b. Drawings of Miraflores Locks (7065-7066-7067)

39. Plans of Lighting (g1 and g2)

40. Drawings miter gates (5023, 5063, 6210,6211, 6169)

41. Partial Hydraulic Model study of FILL/SPILL Valve LHL-898

42. Comprehensive Hydraulic Model Study of E/F Valves LHL-906

43. Drawings with cut slope profiles along new canal (6)

44. Bathymetric Survey Entrado Pacifico

45. ACP max tanker

1.3 STANDARDS

The design criteria will be based on:

- ROSA 2000 Recommendations pour le calcul aux états-limites des Ouvrages en Site Aquatique (Recommendations for the design of structures in aquatic site according to the approach of the Limit States)
- PIANC Final Report of the International Commission for the study of Locks.(Bull. 55, 1986)
- CARLIER: “Memento des pertes de charge”
- “INTERNAL FLOW SYSTEM” by Miller
- “MEMENTO DES PERTES DE CHARGES” by Idel’cik

The regulations of ROSA 2000 are based on the Eurocodes and completed with specific requirements for Maritime Structures.

The Eurocodes or European Standards for the design of structures are published in 9 separate volumes :

- Eurocodes 0 and 1 contain the basics for structural design and the loads to be applied on the structures.
  At this moment they are published together as one volume “Eurocode 1”.
- In addition there are six Eurocodes for the different building materials :
  Eurocode 2: Design of Concrete Structures
  Eurocode 3: Design of Steel Structures
  Eurocode 4: Design of Composite Steel - Concrete Structures
  Eurocode 5: Design of Timber Structures
  Eurocode 6: Design of Masonry Structures
  Eurocode 9: Design of Aluminium Alloy Structures
- Finally two other Eurocodes deal with geo-technical and seismic design :
  Eurocode 7: Geo-technical Design
  Eurocode 8: Design provisions for Earthquake Resistance

For items not included in the standards mentioned above, other suitable standards will be consulted as there are:

- BSI 6349 British Standard Code of practice for Maritime Structures
- DIN German Standards
- CUR (Dutch Recommendations)
- EAU 1996 (Recommendations of the Committee for Waterfront Structures)

1.4 DIMENSIONS

1.4.1 SHIP DIMENSIONS

According to the TOR, the main dimensions of the ships are:

- length : 385.60 m
- width : 54.90 m
- draft : 15.20 m

Maximum dead-weight:

- Bulk carrier :140 000 DWT
- Container ship: 105 000 DWT

1.4.2 LOCK DIMENSIONS

1.4.2.1 Length

The useful length of the lock chamber is given in the T.O.R. and is fixed at **426.80** m.
The chambers are equipped with two gates for reasons of :

- security
- maintenance

In the terms of reference a distance of 30.5 m between the gates is indicated as a value to be checked during design. This length will be treated as the net length.

Depending on the type of gates that will be chosen, this distance can eventually be reduced or increased.

1.4.2.2 Width

According to the TOR, the width between the lock walls is **61.00 m**.
Again, if another positioning system would be preferred, additional lock chamber width might be required.

1.4.2.3 Waterdepth

According to the TOR, the minimum waterdepth in the lock chamber is **18.30 m**. It is assumed that this depth has been determined taking into account all required margins (such as ship clearance, sedimentation, water fluctuation during emptying and filling, …).

1.4.2.4 Upper level of the structures

A minimal clearance or freeboard of 2.13 m is suggested in the T.O.R. above the maximum water level in the lock chamber. This value will be examined during the study in function of level fluctuations due to ship movement, and other requirements due to operational conditions. Minimal clearance for the lock gates will be determined separately

1.5 MAIN LEVELS

Gatun Lake	Maximum Level Minimum Level	+26.7m PLD +25.9 m PLD at first +23.9 m PLD later
Pacific Ocean	Maximum Level Mean Sea level Mean Low water spring Minimum Level	+3.60 m PLD +0.30 m PLD **-2.32 m PLD (design level)** –3.44 m PLD

1.6 OTHER REQUIREMENTS IMPOSED BY THE TOR

1.6.1 GENERAL REQUIREMENTS

- The design shall consider the future expansion of an additional lock structure (4th lane) of similar arrangement with water saving basins and recycling provisions.

- The engineering aspects shall be developed to a level that results in a baseline cost estimate within which the project can be designed and constructed within 25% of the estimated cost.
- Maintenance is an important consideration in the design of the system and shall consider a minimum of interruption of service.
- The possible use of the gates to serve as the maintenance closure for the dry chamber maintenance of the lock shall be evaluated.
- The alignment P1 is the alignment retained by ACP for the arrangement of the structures, and has to be optimized by the consultant.

1.6.2 REPAIR AND MAINTENANCE

The need of dry chamber maintenance for repair and maintenance of the lock gates and the filling and emptying system will be evaluated. After this evaluation and the approval of ACP, the design criteria for the structures will be adapted accordingly.

As maintenance work in a dry lock chamber causes interruption of service and leads to a higher cost of the structures, solutions will be examined to avoid this procedure.

Depending on the gate type, it may be possible that maintenance can be performed in a gate recess. The possibility to perform maintenance while the lock is in service will be one of the criteria in the multi-criteria analysis to decide on the choice of the gate type.

If the filling and emptying system is located either in the gates or in the lock walls, it may be less vulnerable to damage than a system located in the lock bottom floor.

2 Hydraulic Design Criteria

Note : **final** design and sizing of the hydraulic circuits and filling/emptying systems absolutely have to be determined by means of physical models studies which are not part of the scope of this conceptual design study.

2.1 DOCUMENTS FURNISHED BY ACP

For these documents, we refer to §1 General

2.2 SPECIFIC DESIGN TOOLS

- FLOWMASTER R
- Software CNR
- Experience on physical scale models of locks

FLOWMASTER2 - FLUID NETWORK SYSTEMS ANALYSIS

The emptying/filling systems will be studied with the software FLOWMASTER 2. Over 1000 companies across 47 countries have turned to FLOWMASTER2, the world number one in network fluid flow, to gain improved understanding of complex fluid systems.

FLOWMASTER2 is able to predict the pressures and flow rates, temperatures and heat flows that will occur in a system over a wide range of operating conditions, in steady state and transient simulation. With this information, it is possible to compare different emptying/filling locks designs and to optimise the systems.

FLOWMASTER2 presents an easy to use, windows based graphical user interface, and a comprehensive database of fluid components such as pumps, valves, pipes, reservoirs/tanks, expansion tanks, pressurised tanks, heat exchangers etc. The mouse is used to select and place components onto a drawing window, connecting them to form an easily understood schematic representation of the system.

Clicking on a component symbol prompts the user to insert appropriate data such as pipe lengths, or diameters.

To define the performance characteristics of the components (e.g. pressure loss and flowrate versus valve position), it is also possible to define other curves based on physical model results.

Results and input variables are displayed on the schematic network, and on graphs that can be re-formatted from the default settings to suit the user's individual tastes and requirements.

The mark FLOWMASTER is a Community Trade Mark of FLOWMASTER HOLDING BV.

Note : these design tools are not to be considered as design criteria, but help to understand how the Consultant will deal with the study.

2.3 PROPERTIES OF WATER

The following properties are partly issued from a presentation by ACP during the kick off meeting in February 2002

class	location	salinity ppt (g/l)	specific weight (kN / m3)	Dynamic viscosity (N x s / m2)	Kinematic viscosity (m2 / s)	Temperature (° C)
Salty	Pacific Ocean	35	10.034	9.92×10^{-4}	9.70×10^{-7}	24
Higher brackish	Pacific entrance	28	9.980	9.76×10^{-4}	9.59×10^{-7}	24
d°	Downstream Miraflores locks	25	9.9518	**9.72×10^{-4}**	**9.56×10^{-7}**	24
High brackish	Upper chamber Miraflores	18	**9.886**	9.62×10^{-4}	9.50×10^{-7}	24
Lower brackish	Miraflores lake	0.8	9.791	9.18×10^{-4}	9.20×10^{-7}	24
Fresh	Gatun lake	0	9.783	9.16×10^{-4}	9.19×10^{-7}	24

2.4 WATER SAVING BASIN DIMENSIONS

It is expected that the saving basins have the same area as the lock chambers for basic studies. Smaller ones save less water, larger ones save more water (but in a minor proportion), but on the

other hand they are more expensive and need more time to be filled and emptied due to the lower head. Some sensitivity studies will be undertaken for different ratio values basin / lock area.

The freeboard over the water saving basins is not yet determined, 0.80m seems a reasonable value. In the same way the minimum water depth in the basins will be determined during the conceptual design.

2.5 LOSS COEFFICIENTS

In addition to the linear friction head loss in the culverts, each hydraulic component of the filling/emptying system creates a singular head loss. The main components which can be distinguished are:

- Entrance
- Valve
- Expansions
- Contractions
- Bends
- Exit

All of these head losses (linear and singular) are integrated in the software “FLOWMASTER” used to design the filling/emptying system. The values are issued from following literature: "INTERNAL FLOW SYSTEM" by MILLER, and « MEMENTO DES PERTES DE CHARGES » by IDEL’CIK.

Loss coefficients are then adapted in order to verify the mathematical results in comparison with physical model results.

2.6 WATER VELOCITIES IN THE CONDUITS

There are many possible combinations of culvert and conduit sizes with valve opening rates which result in the same filling and emptying time. When the flow variation during the filling or the emptying of the lock has been computed for different solutions, the corresponding mean velocity in the different culverts and conduits can be determined.

In order to minimize the damage from erosion, the maximum velocity in the culverts/conduits is limited to 7 m/s.

At higher velocities, steel liners should be provided downstream the valves.

2.7 VALVES

2.7.1 DIMENSIONS

There are different sets of valves for the 3 rd lane of locks :

- Filling from reservoir to lock and emptying from lock to sea valves
- Filling and emptying from lock to lock valves
- Filling and emptying from lock to water saving basins (and reverse) valves

For an optimal maintenance, standardization of valve type and size is required.

2.7.2 RATIO VALVE AREA / CULVERT OR PIPE AREA

If the valve has a smaller area than the culvert or the conduit area, the transition angle between the valves and the culverts should not exceed 1 : 7, i.e. 1 unit of length in transversal direction and 7 units of length in longitudinal direction of the culvert.

2.7.3 OPENING AND CLOSING TIME OF VALVES

At the present time, the valves of the 1 st and 2 nd lines of locks operate with a rate of opening and closure of 1 minute, linearly from 0 to 100 %, without any problem : it is always possible to stop the manoeuvre. Due to the larger size of the valve for the third line of locks, it is expected to have longer times for the manoeuvre of these valves, especially for the opening times. In a first approach for a conceptual design study, we propose, based on the own civil engineering experience of the Consortium (dams, powerhouses and locks), opening times of 2 minutes and closure times of 1 minute. This assumption is conservative and the appropriate opening and closing time in association with the emptying and filling times will be investigated during the study.

2.7.4 GUARANTY OF SERVICEABILITY

Two valves on each culvert or conduit are suitable in order to be operational whatever the circumstances. Each valve is surrounded by bulkhead slots.

2.8 CAVITATION AND AIR DEMAND

Low pressures can cause serious and persistent cavitation, leading to erosion on the valve structures and boundary surfaces of the conduits/culverts.

Another problem is carrying away of large uncontrolled quantities of air into the system.

If it is whenever economically acceptable, the level of the filling conduit / culvert roofs at the valves zones can be fixed, so that negative pressures (calculated with the mathematical model) and cavitation do not occur.

If the level of conduit / culvert roofs is unacceptable because of larger excavation, air vents could be installed downstream of the valves.

Using a mathematical model gives a first answer on cavitation phenomena by calculating the cavitation parameter in the conduits/culverts section where the valve is installed and comparing it with the cavitation parameter of the valve.

The cavitation parameter generally used is the following :

$$s = \frac{P - pv}{\frac{1}{2} r V^2}$$

Where :

P: absolute pressure head of water at the contracted section (Pa)
pv: vapor pressure head of water (Pa)
ρ: density of water (kg/m³)
V: velocity at the contracted section (m/s)

2.9 VORTEX AND FLOATING DEBRIS

To avoid creation of vortex and aspiration of air or floating debris into the filling system, couples of values (head level, entrance velocity) can be determined considering theoretical formula and results of lock physical models.

These values should be specified on a future physical model.

2.10 SENSITIVITY ANALYSIS

2.10.1 BASINS

- Value of m

  m is the ratio value between the areas of the chambers and the saving basins :

  Area S_B = m * Area $_{chamber}$

  The value of 1 for m is generally adopted. Two other values are to be selected for a sensitivity analysis : m = 1.1 and 1.2

2.10.2 REMAINING HEAD BEFORE EQUALIZATION

- Value of D

D is the value of the head between the upper and lower structures when the valves or gates are closed before the equalization of the levels in order to save time.

In a first approach, and according to the literature, the value of 0.10 m for D has been chosen.

Other values should be examined in further studies, such as 0.20.

3 Structural Design Criteria

3.1 GENERAL

The structural design criteria for navigation locks cover stability, strength and serviceability.

3.2 MATERIAL PROPERTIES

Calculation will be done with the notations according to the Eurocode classification. If necessary the material classification can be translated into local standards.

3.2.1 CONCRETE

Reference is made to the European Standard **Eurocode 2** "Design of concrete structures" (EN1992).

An overview of the main characteristics of commonly used concrete types is given in the table below:

Strength class		**C20/25**	**C25/30**	**C30/37**
f_{ck} (N/mm²)		20	25	30
f_{ctm} (N/mm²)		2.2	2.6	2.9
E_{cm} (kN/mm²)	Secant - Modulus of Elasticity	29	30.5	32
Poisson's ratio ν		0.2	0.2	0.2
α m/m/°C	Coefficient of linear thermal expansion	$10*10^{-6}$	$10*10^{-6}$	$10*10^{-6}$

3.2.2 REINFORCING STEEL

Reference is made to the European Standard **Eurocode 2** "Design of concrete structures" (EN1992).
The main characteristics of the reinforcing steel that will be used are :

f_{yk} = 500 N/mm² (characteristic yield tensile strength)
E = 200 000 N/mm² (Modulus of elasticity)

3.2.3 STRUCTURAL STEEL

Reference is made to the European Standard **Eurocode 3** "Design of steel structures" (EN1993) and EN 10027-1.

An overview of the main characteristics of hot rolled steel is given in the table below:

Nominal steel grade	Thickness t (mm)			
	t ≤ 40 mm		40 mm < t ≤ 100 mm	
	f_y (N/mm²)	f_u (N/mm²)	f_y (N/mm²)	f_u (N/mm²)
S235 J2G3	235	360	215	340
S275 J2G3	275	430	255	410
S355 J2G3	355	510	335	490

f_y = yield strength
f_u = ultimate tensile strength

Modulus of elasticity	E = 210 000 N/mm²
Shear modulus	G = 81 000 N/mm²
Coefficient of linear thermal expansion	$\alpha = 12 * 10^{-6}$ (1/°C)
Density:	ρ = 7850 kg/m³
J2 Impact strength	27J Testing temperature –20°C
G3	Normalized or normalized rolled

3.2.4 SOIL AND ROCK

The soil and rock characteristics are elaborated in part A §4.

Reference is also made to the ROSA chapter "valeurs representatives des propriétés de base" (representative values of basic properties) and "valeurs representatives des résistances" (representative values of resistances).

3.2.5 GROUT ANCHORS

Reference is made the to ROSA Recommendations.

3.2.6 SHEET PILING

Reference is made to the ROSA Recommendations.

3.2.7 PILES

Reference is made to the ROSA Recommendations.

3.3 LIMIT STATES

The Limit states that are to be checked are:

- **External stability :**
  each part of the lock-system is considered as self-stabilizing, behaving like a rigid block

- **Global stability :**
  deep seated sliding

- **Hydraulic stability :**
  relates to the soils subjected to permanent or transitory hydraulic flows which can have destabilizing effects.

- **Strength (internal stability) :**
  the system consists of different parts to be examined: lock operating gates, lock walls, entrance walls, cofferdams, operating structures…

- **Serviceability :**
  displacements and deformations: the settlement of the structure is checked, in particular the relative settlements in case of foundations on compressible soil and/or piles.

3.4 LOADS

3.4.1 DEAD LOADS

Water	Fresh Saline	9.8 kN/m³ 10.0 kN/m³
Concrete	Reinforced Not reinforced:	25 kN/m³ 23 kN/m³
Steel		7,7 kN/m³
Soil/Rock		See part A§4

3.4.2 HYDROSTATIC LOADS

3.4.2.1 Main water levels

We refer to part A §1.5.: minimum and maximum water levels of the Gatun Lake and the Pacific.

3.4.2.2 Ground Water Level

Maximum ground water level: maximum level of Gatun Lake : +26.7m PLD
(and maximal equal to ground level)

Minimum ground water level: minimum level of Pacific : -3.44m PLD

The water levels behind the lock walls depend on the lock wall top levels, the existence of drains, the nature of the backfill, the type of lock wall, etc. They will be fixed in the lock wall design phase.

3.4.2.3 Free water level in the Lock chambers

The water level in the different lock chambers will be studied and determined during this conceptual design.

3.4.3 EARTH

The soil characteristics are elaborated in part A §4.

3.4.4 WIND

Wind pressure: $q = ½ . r . V^2$ Þ $q = 0.613 . V^2$
with :
ρ = 1,225 kg/m³: weight of air
V (m/s): velocity.

Based on the data furnished by ACP (nr. 11), we define the reference velocity V:

V = 25.5 m/s q = 0.4 kN/m² for a design period of 5 years (e.g. temporary constructions)

V = 33.8 m/s q = 0.7 kN/m² for a design period of 100 years

→ *ROSA 2000 actions climatiques §2*
→ *part A §1.2.2 nr.11*

3.4.5 TEMPERATURE AND SHRINKAGE

During future design phases, temperature and shrinkage loads shall be considered, especially with respect to expansion joint design and minimum steel reinforcement requirements. Temperature and shrinkage loads will not be addressed during conceptual design.

3.4.6 WAVES

The design wave heights depend on the reference return period and on the climatic conditions of the site. Data on wave height and wave period have to be communicated by ACP, although they appear to be negligible.
In the conceptual design, the wave impact will not be taken into account for the design of the gates.

3.4.7 COLLISION

The load case that must be considered will depend on the way the ships are moved in the set of locks (vessel positioning system). It is obvious that a collision of a ship with a gate will always lead to damage. The order of magnitude of the damage will also depend on the type of gate. This problem will be assessed during the study of the gates.

3.4.8 SEISMIC LOADS

See part A §4

3.5 SAFETY FACTORS

3.5.1 SEMI PROBABILISTIC APPROACH – PARTIAL SAFETY FACTORS

The ROSA-Standard is based on the semi-probabilistic approach considering Limit States. This approach is in accordance with the stipulations of the European Standard ENV1990 (Eurocode 0: “Basis of structural design”).

The goal of the semi-probabilistic approach is to limit the probability of possible phenomena to acceptable levels.

Therefore, the method resorts to the use of characteristic values and partial safety factors.

The partial safety factors as defined in the ROSA Standard are summarized as follows:

Uncertainty of loads	γ_f
Uncertainty of material properties Uncertainty of soil-structure interaction parameters	γ_M
Uncertainty of the resistance	γ_R
Every uncertainty in connection to the calculation model	γ_d

The partial safety factors depend on the expected service life of the structure.
The factors for a service life of 100 years will be applied.

→ *ROSA Présentation de l’ensemble* *§2.1.- §5.1.*

3.5.2 GLOBAL SAFETY FACTORS

The ROSA Standards make a difference between the conceptual design phases at one hand and the preliminary and detailed design at the other hand.

- **Preliminary design:** For the first dimensioning of structure, reflections are made on the project situation, the phenomena to be avoided, the favourable or unfavourable character of various parameters, the determination of their representative values, the models of calculation; once this analysis is concluded, the justification can be done with the traditional **global safety factors,** as used in well experienced methods.
- **Detailed design:** For the " fine " dimensioning of the structure, the ROSA recommendations implement the semi-probabilistic approach with **partial safety factors** for specific structures in aquatic site.
  As the number of loads and load combinations can be high (this is inherent to the semi-probabilistic approach), a software tool is recommended for the input/output processing of the calculation software. If not, it is legitimate to select a reduced number of variations of combinations of actions to carry out the checks, while trusting the experiment and preliminary calculations.
  The model coefficients (γd) as suggested in ROSA recommendations should not be used independently of other partial coefficients that are stipulated in ROSA
  ROSA mentions the interest to resort to finite element analysis for forces acting on the base of the structure, for the problems related to interaction soil - structure and for the control of deformations. Specific recommendations for the use and interpretation of finite element models are not included in ROSA.

→ *ROSA Présentation de l'ensemble* *§2.2.- §5.1.*

3.6 LOAD COMBINATIONS

There are 4 different design situations to be considered in preliminary and detailed design phase:

- **Sustained**
  Situation of long duration. They correspond in general to the normal exploitation of the work (the most unfavourable water levels are taken in account)
- **Transient**
  Situation of short duration: construction phase, maintenance, exceptional or special exploitation.
- **Accidental**
  Situation of very short duration: accidental actions or accidental configuration of the structure.
  - Seismic loads
  - Accidental scour
  - Collision of a vessel into a lock gate

- Consequences of a failure of the drainage (clogging, deformation, etc.)
- Breakdown of a hydraulic device upstream
- Breakdown of a hydraulic device downstream
- Accidental flood

- **Post-Accidental**
  Specific cases or transient situations after an accidental situation during which the data of the safety are supposed to have undergone a strong modification

The LOADS are subdivided in 3 load types :

- Permanent: G
- Variable: Q
- Accidental: A

The design values of the loads shall be combined using the following rules:

→ *ROSA presentation d'ensemble* *§5.2.5.2.*

Ultimate Limit State (ULS)	
Situation	Load Combination
Sustained or transient	**Fundamental load combination:** corresponding with a return period of 10 000 years $\gamma_d \cdot E\{\sum G_{Md} + \sum G_{md} + Q_{1d} + \sum y_{oi} \cdot Q_{id}\} + R_d$ et/ou X_d **Fatigue combinations:** *Exceptional*: unusual combinations (simplified method adapted for retaining walls or criterion of ultimate displacement)
Accidental	Accidental combination

Serviceability Limit State (SLS)	
Situation	Load Combination
Sustained or transient	**Rare Combination** Corresponding with a return period of 50 to 1000 years $\gamma_{d,serv} \cdot E\{\sum G_{Mk} + \sum G_{mk} + Q_{1k} + \sum y_{0i} \cdot Q_{ik}\} + R_k / \gamma_{R,serv}$ and/or $X_k / \gamma_{M,serv}$ **Frequent Combination** $\gamma_{d,serv} \cdot E\{\sum G_{Mk} + \sum G_{mk} + y_{11} \cdot Q_{1k} + \sum y_{2i} \cdot Q_{ik}\} + R_k / \gamma_{R,serv}$ and/or X_k / γ_{M} **Quasi-permanent Combination** $\gamma_{d,serv} \cdot E\{\sum G_{Mk} + \sum G_{mk} + \sum y_{2i} \cdot Q_{ik}\} + R_k / \gamma_{R,serv}$ and/or $X_k / \gamma_{M,serv}$
Accidental	

→ *ROSA 2000 présentation d'ensemble* *§5.2.5*

A	Accidental action
E	Effect of an action
G	Permanent action
G_d	Design value of a permanent action
G_M	Favourable permanent action
G_m	Unfavourable permanent action
Q	Variable action
Q_d	Design value of a variable action
R	Resistance parameter (or resistant force)
R_d	Design value of R
R_k	Characteristic value of R
X	Basic property of a material
X_d	Design value of X
X_k	Characteristic value of X
γ	Partial factor
γ_d	Partial model factor
$\gamma_{d,\ serv}$	Partial model factor for the rare combinations and the frequent or quasi-permanent combinations.
γ_f	Partial factor for the actions
γ_M	Partial factor for a material property
γ_R	Partial factor for a resistance action
$\gamma_{R,\ acc}$	Factor γ_R for accidental combinations
$\gamma_{R,\ serv}$	Factor γ_R for rare, frequent or quasi-permanent combinations
ψ_0	Factor applied to a characteristic value of a variable action in order to determine the value of the action in a combination
ψ_1	Factor applied to a characteristic value of a variable action in order to determine the frequent value of the action
ψ_2	Factor applied to a characteristic value of a variable action in order to determine the quasi permanent value of the action

4 Geotechnical & Seismic Parameters

4.1 GEOLOGY – GEOTECHNICS

4.1.1 EXECUTIVE SUMMARY

The geological context along the project area of the Post Panamax Locks alignment is complex and has been approached by successive campaigns of field survey and exploration by drilling and tests in laboratory. It can now be stated that the different rock formations, volcanic, igneous as well as sedimentary, which shall be excavated, are well identified. In addition, their geo-technical properties are properly determined from various tests and correlations.

Accordingly, the present analysis of all these available geological and geo-technical data allows to issue a rock mass classification into four types which differentiate the different conditions of excavation and foundation which shall be encountered along the currently proposed P1 alignment. This present analysis also recommends some of the geo-technical criteria required for the further excavation works of the channel and the construction of the locks basins. Providing some complementary cross-checking of stability analyses in a few sections of the future channel, with possible adverse geological conditions, it is proposed to adopt the criteria for cut slope which were determined in previous studies of the project and which result in four types of cut slope excavation regarding the rock mass types.

Application of the different rock mass criteria to the project is conclusively issued in the form of detailed longitudinal geological and geo-technical cross-sections, which were initially prepared by the A.C.P. engineers. These sections cover the total length of the P1 alignment and are subdivided into three parallel sections which correspond to the centre line of the channel alignment and its both lateral walls (east and west).

The present assessment of the local geological conditions can be considered as nearly complete except the detailed geological structure of the project area, which still requires a complementary stage of study to raise some uncertainties regarding the fault pattern. This analysis should consist in a detailed screening of the data basis, field survey and aerial photo interpretation. It could be

adequately reinforced by a campaign of geophysical survey, which would consist in several seismic refraction profiles to be purposely performed through the questionable and inferred boundaries between formations and through some of the inferred fault traces.

Construction of the New Post Panamax Locks requires sound and strong rock foundation mainly consisting of massive basalt within the area that was purposely selected by the A.C.P. engineers, between St.6+750 and St.8+000 of the proposed P1 alignment. Other components of the channel alignment shall be founded upon basalt, pyroclastics and sedimentary formations at a less extent. ***From the present assessment of the geological and geo-technical conditions, one could suggest to optimise the site of the locks by a slight shifting of the currently proposed alignment towards southeast. Such a shifting should allow the locks to be almost entirely founded in the local basalt mass (laccolithic rock mass) which is classified in the range of Rock Mass Type that undoubtedly corresponds to a very good rock mass foundation***

4.1.2 GEOLOGY

4.1.2.1 General Context

The formations which are exposed within the area of the new Post Panamax Locks P1 alignment are of heterogeneous nature. They consist of three main types of rocks:

a) Igneous, predominantly intrusive diabase and basalt which is present as dikes, flows, sills or laccolith,

b) Pyroclastics, mainly bedded tuffaceous agglomerate, locally identified as the Pedro Miguel formation,

c) Sedimentary, chiefly represented by La Boca formation and secondarily by the Cucaracha formation which is exposed at less extent along the northern section of the alignment.

The relationship between formations is rather complex due to the frequent disruption of sedimentary or pyroclastic bedding by faulting and magmatic intrusions, combined with short lateral variations of sedimentary facies.

Support documents for the assessment of the local geology consisted in several Drawings prepared by the A.C.P. engineers and completed by the Consultant, listed as follows:

- Geological Map (Ref. g) along the new Post Panamax Locks Alignment (designed as P1 alignment) which shows the distribution of the different formations and the faulting pattern,
- Geological cross-sections (Ref. h and i) which show the longitudinal sequence of foundation along three profiles corresponding to the centre line of the future channel and its both lateral walls. These sections are completed with the longitudinal distribution of the rock mass types with their relevant geo-technical criteria for excavation.

These Drawings are based on detailed field survey and numerous core holes and tests from successive campaigns of investigations of the project area. Survey of natural rock exposures and existing excavation walls permitted an almost complete in-situ visual inspection of each of the rock formations. The most recent boreholes (67) were carried out during the year 1999. They were denominated as TP1-, TP2- and TP1C-, and are concentrated along the new P1 alignment route. Around year 1942, about 400 boreholes, labelled by M-, were drilled in the surroundings of the present new alignment.

As a general approach of the local geology, it is stated that basalt is dominantly exposed in several locations as hills or dome-shaped structures, along the proposed locks alignment. It is also exposed, in some places, along ancient excavated surfaces, which generally show no adverse effects of weathering. Volcanic agglomerates and intercalated tuff strata are not directly exposed at the site of the new Miraflores Locks but forms an extensive part of the future northern approach channel foundation. Its exposures are connected to the presence of basalt masses and to the local absence of jointing. Sedimentary formations are also present within the project area but at a lesser extent than basalt and pyroclastics. Sandstones and silty shale of the La Boca formation and clay-shale of the Cucaracha formation are very few exposed, except in existing cut slopes, due to their high tendency to weather.

4.1.2.2 Lithology - Stratigraphy

4.1.2.2.1 Basalt

The basalt is the most extended rock type over the southern section of the new **P**ost **P**anamax Locks alignment. as it is shown on the geological map (see Dwg.1). Its natural exposures (Coccoli and Aguadulce hills) as well as the existing excavations present a hard, strong, generally dark greyish to blackish coloured, finely crystallised, and locally porphyritic igneous rock. The extended mass of basalt which shall constitute the almost unique rock foundation of the future Miraflores P.P. Locks is stated to be either a very thick sill or laccolith of diabase that cooled quite close to the surface and then developed a superficial columnar jointing (prism grade). Its petrography is typical dolerite rock mass with abundant plagioclases and ferromagnesian minerals. Other basalt exposures on sides and at North of the main Miraflores mass occur as dikes, sills and lava flows within and/or above the sedimentary and pyroclastic formations. These basaltic bodies shall be partially involved in the future excavations of the P1 channel alignment: between Station 5+700 and St.6+700, and around St.2+500.

4.1.2.2.2 Cucaracha

It is sedimentary formation composed with greenish-grey coloured clay shales with intercalated thin beds of poorly cemented sandstone. It is exposed in the northern part of the future channel alignment as spots of soft rock mass, which is weakened by local faulting.

4.1.2.2.3 Pedro Miguel

This formation is volcanic deposits composed of greyish (dark to light grey), fine to coarse textured agglomerates cemented in a dense tuffaceous matrix. Tuff are the finer volcanic products which are often found bedded within (inter-fingered) or in close association with coarse agglomerate horizons. Agglomerates contain sub-angular to sub-rounded pieces of dark grey to

black coloured basalt which range from 1 cm up to 50 cm (comparable to tuffaceous volcanic breccia). When fresh, the Pedro Miguel agglomerate formation is tightly cemented and hard. It is characterised by its massive unsystematic character of jointing.

4.1.2.2.4 La Boca

It corresponds to a sedimentary alternated sequence of dark, medium hard, compact, shales with intercalated medium hard to hard sandstone beds. It overlies the Pedro Miguel agglomerates with unconformity, as a marine phase of deposition. Prevailing beds of shales are laminated with well-developed schistosity cleavage parallel to the bedding. Sandstones are fine to very coarse-grained (up to micro-conglomeratic facies) and intercalated within shale strata in the form of thin or lenticular beds with cross bedding. The sequence is locally distorted and weakened by faulting and subsequent basaltic dikes or sills. It is medium hard to soft formation which is prone to weathering.

4.1.2.2.5 Soil Overburden

The soil overburden is developed over most the extent of the new P1 alignment as the result of the in-situ weathering of the formations present. These residual soils have a general high clay content.

The surficial overburden is also composed of fluvial, torrential and marine alluvium which constitute a minor portion of the materials involved in the excavation works of the future channel. Marine deposits (tidal fluctuations) shall be mainly encountered at the entrance of the south approach channel. They are composed of layered and saturated organic silts and soft sands which cumulate a total thickness not exceeding a few meters. The stream alluvial deposits are chiefly concentrated along the two existing streams and the downstream floodplain, which shall be partly crosscut by the P1 alignment: the **Rio Grande** around St.3+000-500 and **Rio Cocoli** around St.6+300. They are loose sand and poorly sorted gravel deposits with abundant silt layers.

In addition to alluvium and residual soil, quantities of artificial fill occupy some of the low areas along the P.P.P1 alignment, such as the spoil area between St.2+800 and St.3+700. Along this section, the spoil material from dredging and excavations partly overlies the floodplain material of the Rio Grande and is mixed with the Miraflores lake sediments.

4.1.2.3 Structure

Faulting is chiefly developed as linear planes or corridors transversal to the future P1 alignment. Several major faulted zones shall be encountered in the excavations. They were identified and partially explored at the different previous stages of geological study of the area. However, their trace and extension are still not completely defined so that some question marks are still pending on the actual tectonic pattern of the project area. Most of the fault traces which are drawn on the geological map (Dwg. Ref. g), attached to the present report are still inferred lines which require

complementary detailed survey. It exists several secondary and minor faults which were checked on the field or deducted from borehole logs but not plotted on the map. The local fault pattern, which is locally combined with basaltic intrusions, contributes to the complex geological structure of the Project area in which most of the contacts between the different rock, which shall constitute the foundations of the future channel and locks, are discordant. Based on the current approach of the local geological structure, the following section of the report tentatively describes the major discontinuities, which shall be encountered in the area of the new Miraflores locks and along the extension of the P1 alignment. In a second section, the joint pattern of the rock foundations shall be examined.

4.1.2.3.1 Faulting

At about St.7+700, a straight fault zone, formerly identified as the Aguadulce fault, shall cross the locks area. Its strike following N.045°E is almost normal to the locks alignment. This fault is reported to have a displacement of about 30 m along vertical walls (steeply dipping NW). It is a normal fault, which crosscuts the massive doleritic (diabase) laccolith over an inferred width of a few meters (5 m). Both walls of the fault shall be shallowly affected by tectonic influence whereas its filling is composed of densely fragmented and crushed rock with slickensided clayey material.

The fault corridor or zone identified as the Miraflores fault should be encountered between St.7+100 and St.7+500.i.e. in the middle part of the future lock basin. It is composed of several separate planes among which two segments appear to be dominant. The first one corresponds to the sharp discordant contact between the massive basaltic laccolith (diabase of Cocoli hill and Aguadulce hill) and the sandstone/siltstone sequence of la Boca formation, at the northern side. The second segment was identified in borehole (TP1C-28) inside of the La Boca formation. This faulted zone separates and up/down-drops several compartments or blocks of the Boca formation and probably Cucaracha shale. Along the individual fault planes, the sedimentary strata are distorted, sheared and crushed but the influence of the tectonic movement seems to be of low extent. The general dipping of the fault planes is nearly vertical and downthrow movements are reported to be eastwards.

The general strike of this faulted corridor is approximately N-S (from N.020°E to N010°W) and its extent is reported to be long. Towards South, it could go through Howard field, on the pacific shoreline, and to the North through the Miraflores Lake in direction of the Pedro Miguel stream.
The Cocoli Arm fault which was previously identified a major fault, cut the P1 alignment following an oblique direction about N020°E. Similarly to the Miraflores fault, it is bifurcated from South to North into two major fault planes between which one compartment of La Boca formation is down dropped. The two fault planes are distant of about 50 m along the east wall of the future channel, from about St.6+260 to St.6+310. Along the west wall, only one fault trace should be encountered around St.6+400 that should affect rock masses of basalt and diabase, in the form of a blocky zone of a few meters thick in a clayey matrix with shear and slicken sided figures. The extension of this fault zone towards North East is supposed to follow the discordant lithological contacts between Cucaracha and basalt. Consequently, its inferred trace progressively turns towards and goes through the Miraflores in straight direction of the Pedro Miguel stream.

Secondary fault traces shall be encountered along the P1 alignment, like the oblique one, which cuts the east wall of the P1 alignment around St.5+800 and the west one around St.6+000. Like drawn on the geological map (See Dwg.1). Such faults induced softening and slickensided movements within the rock and deep rock weathering. They shall affect the excavation works of the P1 alignment locally as long as their strike is generally almost perpendicular or slightly oblique to the direction of the alignment and their width is limited to a few meters.

Minor faults, which have not been mapped, shall be also encountered along the extent of the P1 alignment. They are of minor importance in term of extent and width which ranges less than a few centimetres. They will locally affect the rock stability of excavation walls by causing some excess break off.

4.1.2.3.2 Joint Pattern

The rock jointing which affects the local rock foundations along the P1 alignment has been plotted from:

- Previous and recent field survey which was cross-checked by a field visit during the Consultant's mission, during March 2002
- Oriented core drilling that was performed in three recent boreholes,

Transverse joint sets affect the basaltic rock mass and the pyroclastic exposures, whereas bedding is the dominant joint set of the local sedimentary formations. The columnar jointing of the basalt flows and upper horizon of dolerite laccolith is typical of transverse joints which are almost vertical and of low to medium persistence.

The development of vertical jointing within all of the formations, which compose the bedrock of the project, is directly connected to the major pattern of faulting which is identified as trending vertically. In addition, the dominant vertical joint set, striking almost N-S (N.150-180°E), derived from a noticeable influence of the local faulting. The second joint set could correspond to the conjugated direction E-W to the first set. The frequency of these two sets has been evaluated to 0.5 m to 1 m on basaltic and agglomerate outcrops, but it is closer in the vicinity of faulted zone.

Deviations in direction and variations in spacing of joint planes shall influence the size of possible rock wedges in wall excavations of the future channel. This shall lead to carry out a detailed plotting of joints and geological mapping during the excavation works.

4.1.2.4 Weathering

The depth of weathering varies considerably, depending on the rock mass type. As the weathering starts from the discontinuity planes, one can observe that the closely fractured or thinly bedded rock formations are more weathered than massive rocks in which the discontinuities are more than 1 m spaced.

La Boca sandstone and shale tend to weather rapidly where exposed surfaces are subject to alternate wetting and drying. Shale disintegrates into small pieces whereas spheroidal weathering separates sandstone and micro-conglomerate into blocks with well-rounded corners and edges. This creates at the exposed surfaces a superficial layer of softer rock material, which generates clayey residual soils. Weathering ranges in depth from 3 to 6 m thick and underlies deposits of soft silts and clays, which can be up to 15 m thick (like in borehole TP1C-28).

Cucaracha shale weathers and breaks down rapidly upon exposure to the air. The weathered Cucaracha material is known to act like bentonitic clay, and when excavated, it may generate slides of mudflow type.

Weathering of basalt masses, which are exposed over the project area, is superficial. It ranges from less than 1 m up to 6 m thick based on the logs of the recent boreholes (see Ref. c) and visual inspection. The residual soil, which is located above this fringe of weathered rock, is generally clayey reddish, similar to lato soil type, and may be locally more than 15 m thick.

Pedro Miguel agglomerates have weathered equivalents comparable to basalt.

4.1.3 GEOTECHNICS

4.1.3.1 Geotechnical Rock Parameters

This section deals with the estimation of the principal geo technical rock parameters relevant for the design of the excavation works and slopes of the channel. These parameters are chiefly related to the assessment and classification of the rock mass and the discontinuities. The relevant information are collected from the results of field and laboratory investigations and from geological and geo technical reports published at various times since the early 1943's (Ref. a) up to the present day. This available information is considered as sufficiently conclusive to issue criteria with respect to design and construction.

4.1.3.1.1 Rock Mass Parameters

Based on the description of the main geological features of the project area, the rock foundation of the P.P.L. alignment is subdivided into four rock mass types (RMT).

On the surface, a completely weathered rock mass up to 10-20 m deep is present. In these zones, a soil-like behaviour of the rock mass shall be considered for the design. The basic soil parameters are not herein analysed and only general recommendations are suggested.

The local rock mass is classified by using the Rock Mass Rating (RMR) which takes ressort to six parameters that can be measured on the field or obtained from boreholes data and laboratory tests. The general geo technical characteristics of the four selected rock masses are described hereinafter:

<u>RMT I – Very Good Rock Mass.</u>

Fresh, sound basalt and agglomerates which are massive or slightly jointed. Joints are generally closed and show rough surfaces. They are mostly free of clay filling. The spacing of the joints is greater than 1.50 m. The strength of the rock mass is high and the deformability is low. The rock is not sensitive to water.

RMT II – Good Rock Mass.

This rock mass type consists of several rock masses. The first is predominantly massive fresh basalt which has medium to closely spaced columnar jointing with a moderate degree of opening. The second is jointed agglomerate with medium bedded tuff intercalations. The third is the consolidated intact La Boca sedimentary formation. It is massive or slightly jointed shale/siltstone with medium bedded sandstone or micro-conglomerate intercalations. The rock masses can be occasionally slightly weathered and might contain some clay fillings or slickensides. The rock masses strength and deformability are moderate. They are not sensitive to water except the fillings.

RMT III – Fair Rock Mass.

It is predominantly the moderately weathered La Boca formation with thinly bedded sandstone/shale interbeds which has medium to very closely spaced joints or which contains shear zones and slickensides. It is also the closely jointed Pedro Miguel agglomerate which is moderately to highly weathered. The joints are smooth, with a high degree of opening and filled with clay. The soft Cucaracha sedimentary formation also belongs to this fair rock mass class. The strength of the rock mass class is moderate to low and the deformability is moderate to high. In weathered zones the influence of water is moderate to high.

RMT IV – Poor Rock Mass.

This rock mass consists of soft, weathered sedimentary and volcanic formations. It is dominant shale/mudstone/siltstone or tuff with thin sandstone or agglomerate intercalation's and very closely spaced jointing and bedding. Joints are showing a nearly complete degree of opening with some clay fillings. It also corresponds to mylonitised fault zones or frequent slickensides. The strength is low and the deformability high. This rock mass is very sensitive to water.

The interpretation of all available borehole logs and the geological information gained from outcrops and existing excavation results in the relationship between RMT and RMR as shown in the following Table 4.1.2a. RMT II and I are more or less equal to average values of RMR while RMT III is near RMR 35 and RMT IV near RMR 20. Most parts of the locks structures belong to RMT I and II while RMT III and IV shall be more frequently encountered along the northern approach channel.

Table 4.1.2a: Assignment of Rock mass Types to Ratings according to RMR Classification

Rock Mass Classification with RMR System	Rock Mass Type I	Rock Mass Type II	Rock Mass Type III	Rock Mass Type IV
Expected Mean Rating RMR	87	65	41	22
Range of Rating RMR	> 80	58 – 79	35 – 57	15 – 34

4.1.3.1.2 Basic Rock Mass Data

In laboratory the density of different representative rock samples were determined. The results are listed in Table 4.I.2b. The average density for each of the main formations is derived from more than 280 identifications and is about 2.40 tons/m³ for fresh Pedro Miguel agglomerates and 2.75 tons/m³ for sound basalt. Therefore unit weight of 24 kN/m³ and 28 kN/m³ shall be respectively used for all stability calculations. For the stratified La Boca formation, the mean unit weight value of 22 kN/m³ is representative of every component, from sandstone and conglomerate down to shale beds; this value decreases to 21 for the Cucaracha formation.

Table 4.1.2b: Laboratory tested Density of Samples belonging to different RMT

Material / Rock / Rock Mass	Density (tons/ m³)	Supposed RMT
Fresh Basalt	2.75	I
Basalt, jointed ,weathered	2.60	II
Pedro Miguel Agglomerate, fresh	2.40	I-II
Pedro Miguel Agglomerate, weathered	2.30	III
La Boca shale/sandstone formation, fresh	2.20	III

The uni-axial compression strength (UCS) was tested by point load tests and unconfined compression tests. Point load tests were performed only on fresh rock material which were taken from dry core samples in the area of the foreseen P1 alignment. The samples were representative of most of the geological conditions throughout the project area so that the results can be extrapolated to all the extent of the alignment. The results of the tests of both successive campaigns of tests (August 2001 and February 2002) are shown in Table 4.1.2c in which are indicated the variation coefficient values.

Table 4.1.2c: Uni-axial Compressive Strengths of Various Rock Types with Assignment according to RMT

Rock / Rock Mass	Nb of samples	Mean UCS value (MPa)	Variation Coeff. (x/s)	Rock Mass Type	Assigned UCS Value (MPa)
Basalt, fresh (1st series)	10	98	0.44	I	100
Basalt, fresh (2nd series)	38	96	0.45	I	100
Basalt, weathered (2nd series)	34	35	0,29	II	40
Ped.M. Agglom., fresh (1st s.)	14	47	0.25	I-II	50
Ped.M. Agglom., fresh (2nd s.)	156	35	0.28	II	40
Ped.M. Agglom., weath. (2nd.)	13	12	0.42	III	15
La Boca sdt/slt., fresh (1st)	16	16.5	>1	III	20
La Boca sdt/slt., fresh (2nd)	17	18	0.39	II-III	20
La Boca sdt/slt., weath (2nd)	21	6	0.60	IV	10

4.1.3.1.3 Tensile Strength (TS)

Usually, when no Brasilian test results are available, it is reasonable to set the tensile strength of rock to about 3 to 10% of the unconfined compression strength values, depending upon the nature of the rock. Therefore, one can assume that the following TS values were used in previous stability calculations and can be still recommended without any additional safety factor:

- RMT I: 10 MPa for basalt and 5 MPa for agglomerates
- RMT II: 5 and 4 MPa for basalt and agglomerate;
- RMT III: 1.5 MPa for both agglomerates and La Boca,

4.1.3.1.4 Rock Mass Hardness (RH) and Rock Classification

The apparent hardness and strength of the exposed basalt is confirmed by the full core recovery and high RQD values which is measured on rock cores from numerous boreholes. The high values of results of the previous and recent rock mechanics tests, which were performed on representative core samples, did confirm the very rock quality of the sound and fresh basalt. It can be classified in the range of very hard and strong rock foundation in close correspondence with the class RH5 of the rock classification, which was prepared for the previous design of the new channel alignment and other excavation, works in the vicinity of the project area.

Fresh agglomerates are similarly classified as RH5 but more frequently in class RH4, which characterises the hard rock but slightly, weathered or with intercalations of medium hard material such as tuffaceous layers. RH3 characterises the medium hard material which gather most of the sedimentary formations of the project area as well as the weathered and jointed basalt and volcanic agglomerates. It is in close correspondence with the Rock Mass Type III.

In La Boca sedimentary sequence, the sandstone beds that have a high degree of calcareous cementation act as the most competent rock mass regarding the load-bearing properties. However, these beds represent a relatively low part of the total sequence of the formation, which must be considered accordingly as a low to medium-hard rock, classified as RH3 that should correspond to RMT II.

From previous studies, it is mentioned that extreme variations in geotechnical characteristics were obtained from samples of the soft Cucaracha formation. These variations extend from medium hard sandstone beds to weak gouge-like materials from clay shale horizons. In this regard, this sedimentary formation is classified as RH2 that could correspond to a Rock Mass foundation type III to IV.

In accordance with these comments, the following Table 4.1.2d presents the assignment of the proposed Rock Mass Types to the hardness rock classes.

Table 4.1.2d: Assigned RMT to Rock Hardness Classification (RH)

Material / Rock	RH	RMT
Basalt and Agglomerates, fresh, massive	5	I
Basalt and Agglomerates, slightly jointed and weathered and La Boca, fresh,	4	I
Basalt and Agglomerates, jointed and weathered and sedimentary formations, fresh to slightly weathered	3	II
Agglomerate, highly weathered and sedimentary formations, weathered and jointed	2	III
La Boca shale/sandstone, highly weathered, Cucaracha shale weathered and faulted zones	1-OC5	IV

4.1.3.1.5 Deformation Behaviour

Based upon former field bearing tests as well as former and recent laboratory tests and taking into account the experience gained from studies and construction of the currently existing locks (Miraflores, new Pedro Miguel..) which are founded on the same formations which shall be encountered along the new Panamax Locks alignment, some values of deformation modulus can be proposed for the different rock mass types, as follows:

- RMT I = 12 GPa; RMT II = 8 GPa; RMT III = 6 GPa; and RMT IV = 2,5 GPa

4.1.4 ENGINEERING GEOLOGY

4.1.4.1 Shear Strength and Criteria for Rock Slopes

From previous stability analyses and experience on past and recent excavation works along and in the close vicinity of the present channel, values of the shear strength parameters (friction angle and cohesion) have been determined for the different rock masses as well as for the discontinuities. These values were duly taken into account for the assessment of the stability slope criteria which have been recommended and applied (see Fig 4.1.2.1) to the excavation works recently undertaken in the surroundings of the project area. This proposed criteria estimate does constitute a sound data basis that, in any case, shall be possibly refined by further local geological investigations and detailed calculations, prior to the construction design stage.

Fig. 4.1.2.2. shows the four slope excavation types which are recommended in regard with the different Rock Mass Types. Both types are in close correspondence: RMT I with cut slopes type 1, RMT II with type 2, RMT III with type 3 and RMT 4 with type 4.

The rock slopes have a moderate height which may raises up to 55 m. The location of all slopes in basaltic masses is generally favourable with respect to the local discontinuity system since dipping of discontinuities is steep compared to inclination of the slopes and its general transverse strike. In sedimentary and volcanic formations, bedding can become an adverse feature when dipping in conformity with excavation slopes.

4.1.4.1.1 Cut Slope In Basalt

Excavations of the predominant basalt are generally stable and show no adverse effects of weathering. The massive sound basaltic rock (or diabase) stands on slopes approaching the vertical. However, where jointing is well exposed and the columns of basalt are inclined steeply, or where crushed or sheared seams of soft basalt intersect the cut faces at acute angle, rock falls occur and reinforcement by concreting is required.

4.1.4.1.2 In Pedro Miguel Volcanics

The Pedro Miguel agglomerate ranges as Rock Mass Type I and II. Vertical cuts in such a rock, when fresh, can be safely envisaged like 1V/0.2H was already adopted within the area of concern. When it contains sandy tuff facies its hardness ranges RH3 that may require a type 3 cut slope profile, the more if affected by jointing planar and undulating.

4.1.4.1.3 In Sedimentary Formations

When excavated, the Cucaracha clay-shales rapidly exhibit features of incipient breakdown. Cut slopes require immediate coating to seal the natural moisture and prevent disintegration. Cut in the weathered Cucaracha material generates risk of landslide that must be prevented by adequate drainage pattern.

From previous experience, slope cuts in both weathered Cucaracha and clay from weathered Cucaracha were carried out following the slope gradients of 1V/1.5H for cuts up to 6 m and 1V/3H for cuts above 25 m with a gradual transition between these limits. In La Boca sandstone/shale alternation, slope excavations were carried out following 1V/0.3H for cuts up to 15m and 1V/2.8H for cuts up to 100m

4.1.4.1.4 In Soil Overburden

Geo-technical characteristics of the soil overburden were confirmed by additional standard penetration tests (65) which were performed in residual soil from all the different rock formations present along the P1 alignment. Soil samples were selected for identification and geo-technical tests in laboratory. These soils from sedimentary formations as well as from volcanic and magmatic are predominantly fine sandy or silty soils as elastic silts (MH) and silty sand (SM). A slight differentiation can be observed between soils from sedimentary strata which generate a higher content of clay (soil type CH) than the volcanic and basaltic formations (soil type SC) in which the content of coarse particles is more abundant (soil type GM-GC). The previously used values of soil mechanics parameters are adopted for the stability criteria of slopes which shall be preferably cut following type 4 and type 5.

4.1.4.1.5 In Spoil Material

Excavation in spoil areas shall be locally required, where material from dredging or excavation was deposited in convenient topographic lows during the time of original-construction and maintenance activities. This material which is made up largely of various-sized fragments of clay-shale, sandstone, agglomerate and other rock of the different formations intermingled with basalt from the dikes and sills is loose. It has weathered considerably since its deposition and, because of its fragmented nature, has slight shearing strength. Type 5 slope cut shall be applied.

4.1.4.2 Detailed Characterization of Rock Foundations along the P1 Alignment

This detailed rock mass characterisation of the future foundations of the channel, along the P1 alignment, is shown on the geological cross sections (Ref. h and i). The longitudinal rock mass distribution is plotted along the three sections which correspond to the centre line and both lateral walls of the future channel and locks alignment. Regarding this distribution and the local incidence of lithology and discontinuities, corresponding cut slope criteria are recommended that should assure the "global" stability of the slopes. As a matter of fact, a further detailed assessment of the local rock mass stability shall be undertaken at the early stage of the excavation works. In this respect, detailed wedge stability calculations shall be performed for large slopes whereas local weak zones of fault with extremely low shear strength shall be taken into due consideration. At this further stage of detailed design, the rock support and drainage system shall be studied. These measures are only suggested in the present study, which chiefly focus on the geological features of the project study. Some of the major geological findings are briefly described in the following sections as support comments of the longitudinal cross-sections.

4.1.4.2.1 South Approach Channel and Wall

It will rest upon La Boca formation and intrusive diabase. Both rock are contrasted in term of strength. The entrance of the approach channel shall be excavated in soil and underlying soft rock while the whole extent of the approach wall shall rest in massive diabase, classified as RMT I. This rock mass is discordant and probably in irregular intrusive contact with the sedimentary formation. Along this contact, sheared, slickensided and chloritised basalt could be encountered in zones combined with distorted sandstone and shale strata of la Boca. The cut slope types 4 and 5 are recommended for the entrance of the south channel where soft rock and soils are prevailaing as well as in the area of the intrusive contact zone. Type I shall be adopted for most the extent of the approach wall which shall be excavated in upper fringe of columnar jointed basalt down into massive dolerite rock mass type.

4.1.4.2.2 Locks

The foundation rock throughout most of the extent of the lock walls and channel shall be basalt. Where this rock is undisturbed by faulting, it is strong RMT I and of excellent foundation quality. Local, thin, brecciated zones shall be not uncommon, however, and several sections of closely fractured rock shall be of foundation pertinence. The most noteworthy of these is the identified Aguadulce fault, which shall transect the mid-portion of the lock extent. The weakness induced by its presence is reflected by the local over thickness of residual soil and rock weathering.

4.1.4.2.3 North Channel

Because the geological information of the North Section covers only part of this section, it was not possible to prepare detailed geological sections of the east and west walls of the P1 alignment, due to the too large spacing of the TP1C- borehole series (100 and 700 m long distance between boreholes). Notwithstanding this lack of accuracy, geological cross-sections were prepared combining the information from the TP1C- borehole logs and the data relevant to lithology, structure and geo-technics that allow to get a preliminary assessment of the geological features of the northern section of the channel from St.1+00 to about St.6+00. These sections show the following geological characteristics:

- The distribution of the rock formations along the profile corresponding to the eastern wall is as follows: a) Culebra, b) Cucaracha, c) Pedro Miguel, d) Basalt and Diabase, e) Pedro Miguel, f) La Boca, g) Pedro Miguel and h) Basalt and Diabase.

- The profile of the West Wall is made up of the following formations: a) Cucaracha, b) Pedro Miguel, c) Cucaracha, d) Pedro Miguel, f) Cucaracha, Pedro Miguel, g) La Boca, h) Pedro Miguel and i) Basalt and Diabase.

Along the geological profile of the east wall, the Pedro Miguel Formation (Tpa) predominates, with a length of 2,341 m, which represents 47 % of the total length of this profile. The rock mass that constitutes this formation is made up mainly of agglomerates, sandstone and tuff, classified as RMT I to RMT III. The recommended excavation slope criteria is Type 2 and 3. In this profile, according to the logs of boreholes TP1C-5, TP1C-10 and TP1C-11, there are three fault zones located in the Pedro Miguel formation between Stations 3+763 and 4+700.(see **Table 4.1.3a**). These fault zones are not shown on the geologic map (Ref. g).

Similarly to the profile of the east wall, the Pedro Miguel (Tpa) formation shall predominate along the west wall, with a length of 2,610 m, which represents about 52% of the total length of this profile. The rock mass that makes up this profile is constituted mainly of agglomerates, sandstone, tuff and basalt. These rocks are classified in term of rock mechanics under the RMT II to IV. The corresponding slope excavation criteria are type 1 to 3. According to the log of borehole TP1C-4, a fault should be encountered around St; 3+500 m (see **Table 4.1.3b**).

In both profiles, basalt and diabase (Tb) shall present the best rock mass characteristics. This one is classified RMT I that allow to recommend a slope excavation type 1. A long the east wall profile, these basaltic rock masses shall be encountered from St. 2+306 to 2+576, and from St. 4+844 to 6+000, that represents a cumulated length of approximately 1,426 meters. Along the west wall, the basalt rock mass type is forecast between St. 5+002 and St. 6+000 with a length of about 1000 m.

TABLE 4.1.3a Alignment P1 - North Channel

EAST WALL

FORMATION	SYMBOL	STATION (m)		LENGTH (m)	PREDOMINANT TYPE OF ROCK	ROCK CLASSIFICATION	EXCAVATION CRITERIA	OBSERVATION
		FROM	TO					
Culebra	Tcb	1000	1283	283	Calcareous Sandstone and siltstone	RH-2	3	
Cucaracha	Tca	1283	1934	651	Clay shale	RH-1	4	
Pedro Miguel	Tpa	1934	2306	372	Agglomerate and sandstone	RH-3 to RH-4	3	
Basalt and Diabase	Tb	2306	2576	270	Basalt and diabase	RH-4	1	
Pedro Miguel	Tpa	2576	3410	834	Agglomerate and sandstone	RH-3 to RH-4	3	
La Boca	TL	3410	3710	300	Sandstone,siltstone and tuff	RH-2 to RH-3	3	
Pedro Miguel	Tpa	3710	4844	1134	Agglomerate and tuff	RH-3 to RH-4	2	Three boreholes indicate **fault zone**(TP1C-5,TPIC-10 AND TP1C-11
Basalt and Diabase	Tb	4844	6000	1156	Basalt and diabase	RH-4 to RH-5	1	

TABLE 4.1.3b Alignment P1 - North Channel

WEST WALL

FORMATION	SYMBOL	STATION (m)		LENGTH (m)	PREDOMINANT TYPE OF ROCK	ROCK CLASSIFICATION	EXCAVATION CRITERIA	OBSERVATION
		FROM	TO					
Cucaracha	Tca	1000	1066	66	Clay shale	RH-1	4	
Pedro Miguel	Tpa	1066	1536	470	Agglomerate and sandstone	RH-3 to RH-4	3	
Cucaracha	Tca	1536	1847	311	Clay shale	RH-1	4	
Pedro Miguel	Tpa	1847	2216	369	Agglomerate and sandstone	RH-3 to RH-4	3	
Cucaracha	Tca	2216	2941	725	Clay shale	RH-1	4	
Pedro Miguel	Tpa	2941	3500	559	Agglomerate and sandstone	RH-3 to RH-4	3	
La Boca	TL	3500	3791	291	Sandstone,siltstone and tuff	RH-2 to RH-3	3	**fault zone** (from borehole TP1C-4)
Pedro Miguel	Tpa	3791	5002	1211	Agglomerate and Basalt	RH-4 to RH-5	1	
Basalt and Diabase	Tb	5002	6000	998	Basalt and diabase	RH-4	1	

4.1.5 CONCLUSIONS

The geological context of the project area is complex and has been approached by successive campaigns of field survey and exploration by drilling. The different rock types, volcanic, igneous and sedimentary, are now well identified and their geotechnical characteristics adequately determined from various tests and correlations. Accordingly, rock mass criteria of the different rock foundations have been defined in the present study and are recommended for the further excavation works and construction of the locks basins. Providing some complementary cross-checking of cut slope stability analysis in a few sections of the future channel, with possible adverse geological conditions, it is proposed to adopt the criteria for cut slope which were determined in previous studies of the project and which result in four types of cut slope excavation regarding the rock mass types. Application of these different criteria is issued in the present report in the form of detailed longitudinal geological cross-sections relevant to the Miraflores Lock area and its appurtenant approach ways (from St.6+000 to St.9+00), and in the form of description for the northern part of the alignment (St.1+000 to St.6+000).

From the present analysis of the local geology, it can be concluded that the assessment of the geological conditions of excavation and foundation of the P.P.L.P1 alignment is nearly complete. The assessment will be complete after a complementary analysis of the detailed geological structure of the project area, especially of the faulting pattern. This complementary stage of study requires screening of the data basis, field survey and aerial photo interpretation. It could be adequately reinforced by a campaign of geophysical survey, which would consist in several seismic refraction profiles to be purposely performed through the questionable and inferred boundaries between formations and through some of the inferred fault traces. Contrast of velocities should reflect contrasts of lithology and sequence soil/rock. In this respect such a survey is duly recommended to complete the geological mapping before the construction stage.

Construction of the New Post Panamax Locks requires sound and strong rock foundation that shall be mainly massive basalt within the area that was purposely selected between St.6+750 and St.8+000 of the proposed P1 alignment. Other components of the channel alignment shall be founded upon basalt, pyroclastics and sedimentary formations at a less extent. From the present assessment, it is suggested to optimise the site of the locks by a slight shifting of the currently proposed alignment towards southeast. Such a shifting should allow the locks to be almost entirely founded in the local basalt mass (laccolithic rock mass). The heavy concrete and massive machinery installations necessary for operation of the locks should be founded on stable rock mass type, almost uniformly classified as RMTI, which undoubtedly corresponds to a very good rock mass foundation

4.1.6 REFERENCES - DOCUMENTATION

a) Final Report, Part II – Design, *Modified Third Locks Project*, by Department of Operation and Maintenance, Special Engineering Division, December 1943.

b) Final Report, Volume IV, Engineering and Cost Estimates, *Commission for the Study of Alternatives to the Panama Canal,* by J.I.C.A., 1993

c) Geological Field Logs, Sondeos TP1C-1 – TP1C-34, *Tercer Juego de esclusas, alineamento P1, Pacifico*, by Bachy Fundaciunes, S.A., Octubre 2001.

d) Pruebas de Permeabilidad en 6 sondeos TP1C y 3 sondeos TP2C, *Tercer Juego de esclusas, alineamento P1, Pacifico*, by Bachy Fundaciones S.A., Octubre 2001.

e) Sumario de Perforacion de todos los Sondeos TP, TP1C y TP2C, *Tercer Juego de esclusas, alineamento P1, Pacifico,* by A.C.P. Engineering Division, Febrero 2002.

f) Summary of Results of Laboratory on Rock Formations *Third Set of Locks Project*, by A.C.P., Engineering Division, Geotechnical Branch, August 2001 and January 2002.

g) Mapa Geologico, scale 1/5,000, *Tercer Juego de Esclusas, Alineamiento P1*, by A.C.P., Engineering Division, Feb. 2002.

h) Secciones Geologicas Longitudinales, Perfiles 1 to 3, scale 1/2,000 H-1/400 V, St.6+000 to St.9+000; *Tercer Juego de Esclusas Alineamiento P1*, by A.C.P., Engineering Division, Feb. 2002.

i) Geological and geotechnical cross section of the northern channel from St.1+000 to St.6+000, scale1/2,000 H-1/400 V, *Third Set of Locks, P1 Alignment*, by the Consultant, March 2002

Fig.4.1.2.1

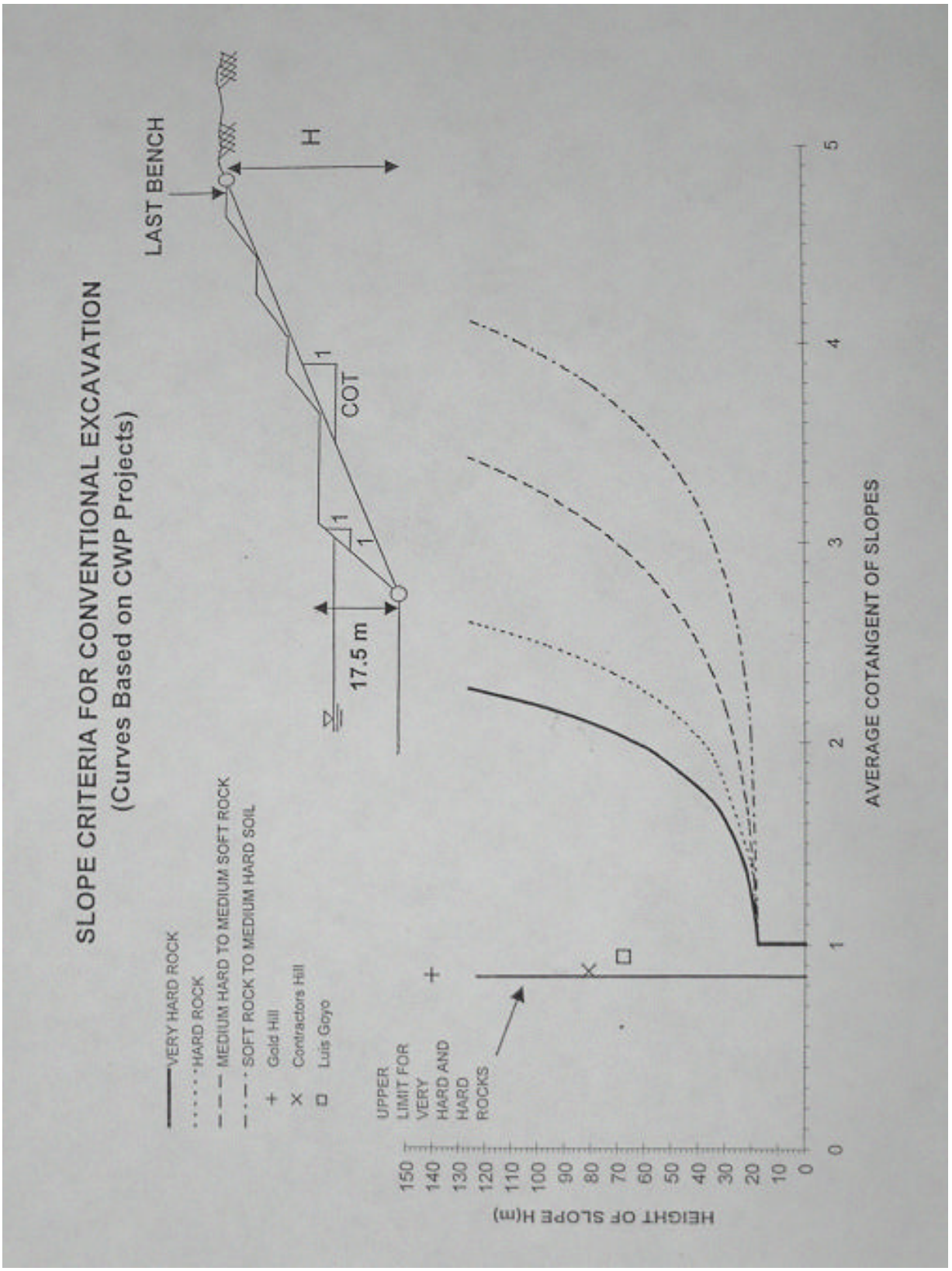
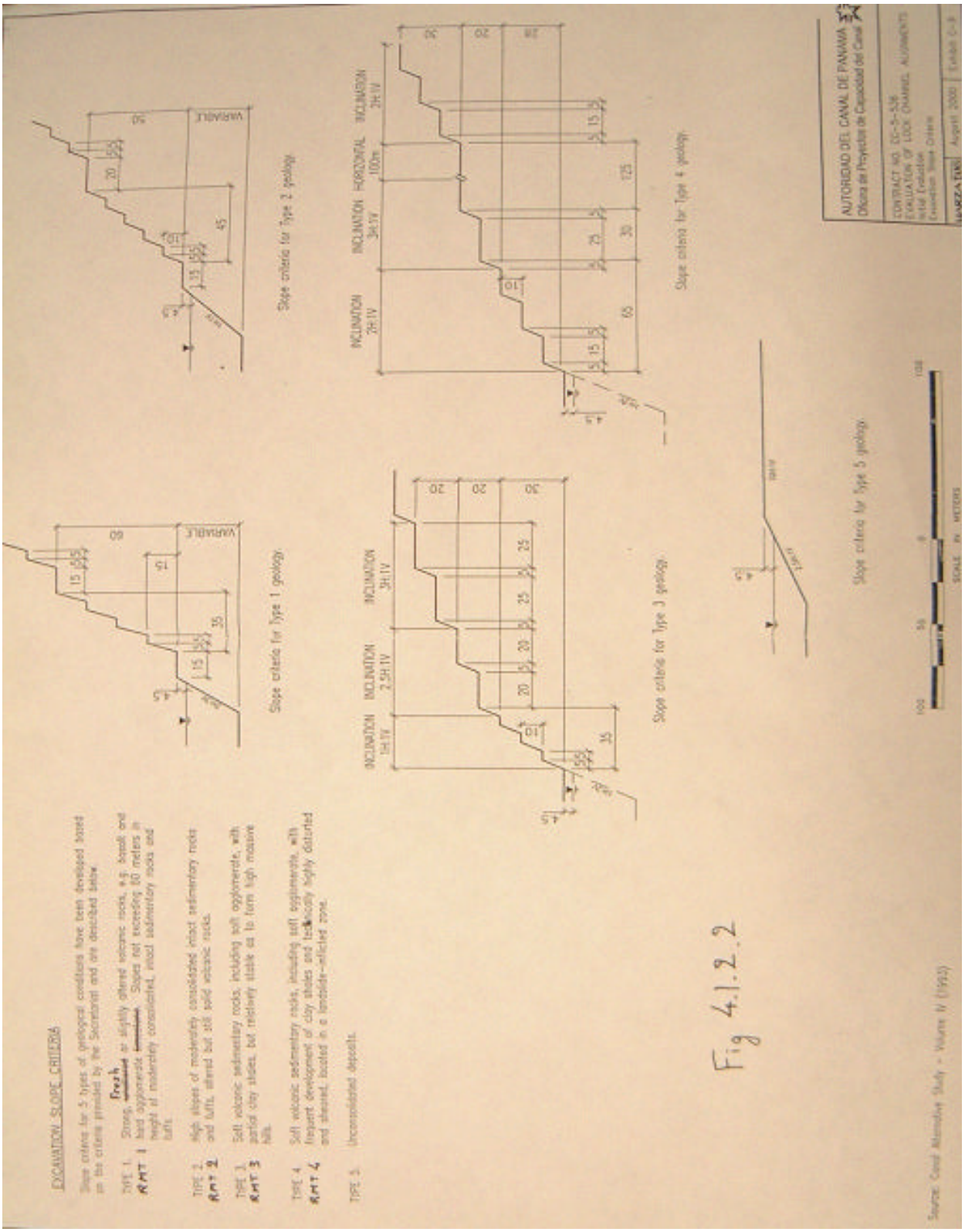

SLOPE CRITERIA FOR CONVENTIONAL EXCAVATION
(Curves Based on CWP Projects)
VERY HARD ROCK
HARD ROCK
MEDIUM HARD TO MEDIUM SOFT ROCK
SOFT ROCK TO MEDIUM HARD SOIL
Gold Hill
Contractors Hill
Luis Goyo
LAST BENCH
H
COT
1
1
1
17.5 m
UPPER LIMIT FOR VERY HARD AND HARD ROCKS
HEIGHT OF SLOPE H(m)
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
5
AVERAGE COTANGENT OF SLOPES

Fig.4.1.2.2.


EXCAVATION SLOPE CRITERIA
Slope criteria for 5 types of geological conditions have been developed based on the criteria provided by the Secretariat and are described below
TYPE 1. Strong, Fresh or slightly altered volcanic rocks, e.g. basalt and hard agglomerate. Slopes not exceeding 60 meters in height of moderately consolidated, intact sedimentary rocks and tuffs.
TYPE 2. High slopes of moderately consolidated intact sedimentary rocks and tuffs, altered but still solid volcanic rocks.
TYPE 3. Soft volcanic sedimentary rocks, including soft agglomerate, with partial clay shales, but relatively stable as to form high massive hills.
TYPE 4. Soft volcanic sedimentary rocks, including soft agglomerate, with frequent development of clay shales and basically highly disturbed and sheared, located in a landslide-affected zone.
TYPE 5. Unconsolidated deposits.
VARIABLE
Slope criteria for Type 1 geology
Slope criteria for Type 2 geology
INCLINATION 1H:1V
INCLINATION 2.5H:1V
INCLINATION 3H:1V
Slope criteria for Type 3 geology
INCLINATION 2H:1V
INCLINATION 3H:1V
HORIZONTAL 100m
INCLINATION 2H:1V
Slope criteria for Type 4 geology
Slope criteria for Type 5 geology
AUTORIDAD DEL CANAL DE PANAMA
Oficina de Proyectos de Capacidad del Canal
CONTRACT NO. CC-5-5A
EVALUATION OF LOCK CHAMBER ALIGNMENTS
Initial Evaluation
Excavation Slope Criteria
HARZA EIN
August 2002
SCALE IN METERS
Fig 4.1.2.2
Source: Canal Alternative Study - Volume IV (1993)

4.2 SEISMIC HAZARD ANALYSIS

4.2.1 EXECUTIVE SUMMARY

The Post Panamax Locks are located in Panama Block along the Pacific coast in a very complex tectonic zone where South America, Central America, Caribbean, Nazca and Cocos plates interfere giving rise to a significant seismicity.

A regional probabilistic seismic hazard carried out by Camacho et al. (1994) results for the Post Panamax Locks area in Peak Peal Ground Accelerations at bedrock of 0.17 g, 0.20 g and 0.24 g for mean return periods of 250, 500 and 1000 years respectively.

In order to validate/or implement this seismic hazard, an analysis of available scientific literature and technical reports has been conducted in March 2002. The main results are the following :

- the North Panama Deformed Belt which corresponds to the Caribbean plate subduction beneath the Panama Block actually controls the far-field seismic hazard.
- the NNW-SSE trending Pedro Miguel Fault which probably crosscut the Post Panamax route actually controls the near-field seismic hazard.
- Reference magnitudes (m^0) associated to design earthquakes as recommended by ICOLD and UBC, have been derived based on fault segmentation, kinematics, rates and historical seismicity.

Return periods->	**MCE** -	**MDE** 950 yrs	**OBE** 145 years	***SMDE*** *1000 years*	***SDE*** *475 years*
North Panama Deformed Belt	7.77	7.60	7.2	*7.60*	*7.48*
Pedro Miguel Fault	5.82	5.53	5.03	*5.54*	*5.38*
	ICOLD			*UBC*	

With MCE = Maximum Credible Earthquake
MDE = Maximum Design Earthquake
OBE = Operating Basis Earthquake
SMDE = Site Maximum Design Earthquake
SDE = Site Design Earthquake

- Peak Ground Accelerations (PGA) at site versus return periods have been derived throughout various attenuation laws. Results for the design earthquakes are reported hereafter:

	MCE	**MDE**	**OBE**	***SMDE***	***SMD***
North Panama Deformed Belt	0.21	0.16	0.13	*0.16*	*0.15*
Pedro Miguel Fault	0.21	0.18	0.13	*0.18*	*0.16*
Recommended PGA (in g) ***(for rock conditions)***	**0.21**	**0.18**	**0.13**	***0.18***	***0.16***
	ICOLD			*UBC*	

- It is emphasised here that the Pedro Miguel Fault seems to crosscut the Post-Panamax route near the planned lock site. According to the MCE associated to that fault, a maximum co-seismic displacement of 15-30 cm cannot be disregarded although the degree of activity seems very low.

Recommendations : in further stages of the design, it is highly recommended to ascertain (i) the actual presence of the Pedro-Miguel Fault on the site, and (ii) the degree of activity of this potential seismogenic source, throughout :

- a “marine” seismic investigations along the Panama Canal between existing Miraflores and Pedro Miguel Locks (as it was performed north of Pedro Miguel locks)
- an on-shore seismic investigations along the Post Panamax Canal route on the expected southern extension of the Pedro Miguel fault.
- if the Post Panamax Locks have to be designed in the close surroundings of that fault, a detailed seismotectonic analysis of this fault will be required including superficial geophysics and paleosismological analysis throughout trenching if necessary.

4.2.2 INTRODUCTION

The Post Panamax Locks are located in Panama Block along the Pacific coast in a very complex tectonic zone where South America, Central America, Caribbean, Nazca and Cocos plates interfere giving rise to a significant seismicity.

A regional probabilistic seismic hazard carried out by Camacho et al. (1994) results for the Post Panamax Locks area in Peak Peal Ground Accelerations at bedrock of 0.17 g, 0.20 g and 0.24 g for mean return periods of 250, 500 and 1000 years respectively.

In order to validate/or implement this seismic hazard, an analysis of available scientific literature and technical reports has been conducted taking into account that the reliability of a seismic hazard assessment strongly depends on both a good understanding of the geodynamics of a given region at different scales, from plate tectonics to fault scales and even in favourable cases to seismic scarp scale.

This analysis was addressed throughout both deskwork and a mission in Panama in March 2002. We present hereinafter the most relevant data, which have been taken into account for deriving PGA at the Post Panamax Locks site.

4.2.3 TECHNICAL DOCUMENTS CONSULTED

The following documents have been provided by ACP for conducting the expertise:

- 1 drawing from ACP: Alineamiento P1 y P2 Sector Pacifico, scale 1/5000 Ubicacion de sondeos, dated 15/01/01
- 1 set of 4 Maps en color: Mapa geologico Republica de Panama scale 1/250000 Ministerio de comercio e industrios, hojas 3, 4, 5 y 6.
- Report: Seismicity Evaluation Tabasara Hydel Project western Panama, ACRES Canada dec. 81
- Report: Excerpts from the Star and Herald on the Sept. 1882 Earthquake, from Engineering & Construction Bureau, Luis Carlos Fernandez.
- Report: El terremoto de San Blas del 7 de Septiembre 1882; E Camacho y V. Viquez, Universidad de Panama , Junio 1993
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Detailed discussions on seismotectonics and seismology of Panama took place at the Geosciences Institute of Panama with J. Toral (Director), E. Chichaco (Research Coordinator) and A. Tapia (Responsable of the seismological network).

Access to Landsat images and a regional Digital Elevation Model (30m) was possible thanks to R. Martinez (ACP-Environment division)

4.2.4 GEODYNAMIC FRAMEWORK

4.2.4.1 Instrumental seismicity and plates boundaries

Regional seismicity in Central America and Caribbean between 1977 and 1997 is reported in figure 1. Few features may be drawn:

- The eastward subduction of the pacific plates (Nazca and Cocos) beneath Central and South America characterised by shallow to deep seismicity.
- The westward subduction of the Lesser Antilles bounding to the East the Caribbean plate also characterised by shallow to deep seismicity.
- The northern transcurrent boundary of the Caribbean plate characterised by shallow seismicity extending in Guatemala throughout the Motagua and the Chixoy-Polochic fault systems.
- The relative complexity of the southern boundary of the Caribbean plates characterised by shallow seismicity distributed over Bocono fault system in Venezuela, Subandean thrusts in Colombia and Ecuador, and curved fold and thrust belt north of Colombia and Venezuela.
- The Panama Fracture Zone south of Costa Rica-Panama border between the Cocos and the Nazca oceanic Plate.
- The absence of deep seismicity between the southern end of the Central America subduction and the northern segment of the Colombia Subduction.
- The absence of seismicity within the Panama Gulf.
- Shallow and intermediate seismicity bounding to the North the Panama Isthmus.

It must be highlighted that the Isthmus of Panama located in a very complex tectonic area where plate boundaries are not always well defined and where surroundings plates strongly interfere.

4.2.4.2 Boundaries of the Panama Block

The Panamax Locks are located in a tectonic microplate called the Panama Block along the Pacific coast in a very complex tectonic zone where South America, Central America, Caribbean, Nazca and Cocos plates interfere giving rise to a significant regional seismicity. (Fig. 2). Surroundings plate boundaries accommodate mostly convergence:

- convergence between Cocos Plate and Central America is currently accommodated within Middle America subduction giving rise to large and damaging earthquakes with magnitude greater than 7.0 (for instance, 1803,1854, 1882, 1941 and 1983 for Costa Rica where the Cocos Ridge is currently subducting). Present-day convergence rate is about 90 mm/yr in a N30°E direction.

- Cocos and Nazca plate boundary is formed by the right-lateral Panama transform which subducted obliquely at a shallow angle within the South Panama Deformed Belt zone. The Panama Fracture Zone has experience large earthquakes (Ms>7.0) in 1927, 1934 and 1962. Relative movement between the two oceanic Cocos and Nazca plates is about 50-70 mm/yr. Two sub-parallel fracture zone (Balboa and Coiba) splay off from the Panama Fracture Zone to the NE.

- convergence between Nazca Plate and South America is currently accommodated within Colombia-Ecuador subduction and also generates major earthquakes with magnitude greater than 7.0 (1906, 1942, 1958 and 1979). It is about 50 mm/yr in a N70°E direction. It must be noted that close to Panama Isthmus, the Colombia-Ecuador subduction becomes less active. This feature may be interpreted as a seismic gap with a capability greater than 7.5 (Ms) earthquake.

- the link between the two above-mentioned subduction is formed by the N80°E trending South Panama Deformed Belt where the Nazca plate aseismically subducts in an oblique way (N70°E) consistently with the Nazca-South America convergence movement. It implies a significant left-lateral component along that plate boundary. South of the Azuero Peninsula, where the South Panama Deformed Belt trends progressively WNW-ESE, it seems that part of this lateral component of the movement is actually absorbed along a series of NW-SE trending left-lateral strike-slip faults located south of the Azuero and Sona Peninsulas and in the surroundings of the Coiba Islands. It must be noted that the Azuero area have been already affected by major earthquakes in 1516, 1803, 1845, 1883, 1913, 1943 and 1960.

- Caribbean plate currently converges toward both Panama Block and South America plate at about 15 mm/yr in a N105°E direction (although this direction is still under discussion). North of Panama this relative movement is accommodated along the North Panama Folded Belt. This arcuate wide shallow belt of folds and thrust faults extends offshore parallel to the Caribbean coast of Panama from the Gulf of Uraba to the Limon area in Costa Rica. Schweig et al. (1999) described a dipping zone of seismicity extending from depths of 30 to 60 km consistent with the presence of subducted Caribbean plate beneath Panama Block. This seismogenic structure has generated some of the largest historical earthquakes that affected Panama area : 1822 (Ms~7.5-7.7), 1904 (Ms~7.2), 1916 (Ms~7.2 and Ms~6.9), 1991 (Ms 7.5).

- the western boundary of the Panama Block is still debated although the western end of the North Panama Deformed Zone (near Puerto Limon in Costa Rica) coincides with both a NE trending zone of widespread faulting and a gravity lineament which extend from the Pacific coast (NW of the aseismic Cocos Ridge) across Costa Rica, to the Caribbean coast near Puerto Limon. Distribution and kinematics of the aftershock sequence of the 1991 (Mw 7.7) Limon earthquake apparently support the interpretation of such diffuse left–lateral western boundary of the Panama Block.

- The eastern boundary of the Panama Block is widespread and involves numerous faults of various kinematics both in Panama and in Colombia. For instance, Jaque River, the Sambu and the Sanson Hills left lateral faults seems to partially absorb the tangential component of the displacement of the Nazca plate relative to South America – Panama plates. However, it may also account for accommodating the E-W trending convergence between Panama Block and South America plate when assuming that these strike-slip faults act as a roughly N-S right-lateral bookshelf structure. Moreover thrust and reverse fault such as the N-S trending Pirre Hills, Ungia or Utria fault in Panama seem to accommodate this above-mentioned E-W trending convergence although, to our knowledge, no accurate analysis have never been performed along these faults.

It may be concluded that, even when widespread, each plate boundary above-mentioned is capable for generating major earthquake. The far-field (regional) seismic hazard in a given site will be thus controlled by its closeness to one of these plate boundaries. Within such complex boundary conditions, the Panama Block appears as a rather rigid block. The state of stress prevailing within the Panama Block and its regional variability is nearly unknown, giving rise to somewhat confusing assumptions on both the kinematics and the degrees of activity of intraplate faults within Panama.

4.2.5 CRITICAL SEISMOGENIC SOURCES AND SEISMICITY

As far as the above-mentioned plate boundaries are concerned, due to the closeness of the site of interest to the North Panama Deformed Belt (relative to the above-mentioned seismogenic sources) and because of its significant historical seismicity, the North Panama Deformed Belt is controlling the far-field seismic hazard. For that reason, although the seismic hazard generated at Post Panamax Locks site by other plate boundaries have been carefully checked, only the seismicity and the seismic hazard associated to the North Panama Deformed Belt seismogenic source are presented hereafter.

However, seismogenic sources may exist within the Panama Block although, to our knowledge, no clear evidence of surface rupture have been never found up to now, instead of detailed field investigations (see for instance Schweig et al., 1999, on the Rio Gatun fault or those assumed close to the Gatun dam). In order to estimate near-field seismic hazard, widespread seismicity and uncertainties on the seismogenic capability of the Pedro Miguel Fault crosscutting the Post-Panamax Locks site are discussed in a second chapter.

It must be noted that earthquake catalogue in Panama starts in 16th century although it is rather poor until 1900 because of uneven distribution of population and scarce written chronicles. However, large earthquakes with magnitude greater than 7.0 are recorded from the 18th century, although large uncertainties remain onto their locations (probably >50 km) especially for the offshore ones.

4.2.5.1 The North Panama Deformed Belt

4.2.5.1.1 The 1822, May 7 Earthquake

This earthquake is now assumed to have occurred near Costa Rica-Panama border. Damages reported at that time are similar to those observed during the 1991 Limon earthquake. Camacho and Viquez (1992) described the effects of this earthquake. Widespread liquefaction is reported along the coastal plain. Based on the regional intensity attenuation (Fig. 2 and 3), Camacho and Viquez (1992) derived a magnitude of about 7.6 (Ms). This earthquake is assumed to have occurred along a thrust of the western North Panama Deformed Belt segment.

4.2.5.1.2 The 1882, Sept. 7 Earthquake

This earthquake is the largest recorded in Panama since the 16th century. Camacho and Viquez (1993) described the effects of this earthquake, also revisited by Schweig et al. (1999). Widespread liquefaction is reported along the coastal plain and river valleys. A tsunami causing 68 victims affected the San Blas islands. Masonry buildings were destroyed or badly damaged in Colon and Panama. A magnitude ranging between 7.5 and 8.0 is generally ascribed to that earthquake. However, based on the regional intensity attenuation (Fig. 2 and 4), Camacho and Viquez (1993) derived a magnitude of 7.7 ±0.3 (Ms) which seems to prevail in the former seismic hazard analysis conducted in Panama. This earthquake is assumed to have occurred along a thrust of the North Panama Deformed Belt at a depth of about 30 km. Based on a slip-rate of 15 mm/yr, Camacho and Viquez (1993) derived a deterministic return period of about 300 years.

4.2.5.1.3 The 1904, Dec. 20 Earthquake

This earthquake affected the same area than in 1822 (Limon-Bocas del Toro) although the far field effects seem to have been more limited (Fig. 2 and 5). Magnitude ascribed to that event varies from 7.0 to 7.5. This earthquake is assumed to have occurred along a thrust of the western North Panama Deformed Belt segment.

4.2.5.1.4 The 1914, May 27 Earthquake

This earthquake occurred along the eastern segment of the North Panama Deformed Belt with an assumed depth of about 70 km beneath the Serrania de San Blas. This event has been strongly felt in Panama and Colon. A magnitude of about 6.5-7.2 is ascribed to that event.

4.2.5.1.5 The 1916, April 24 and 25 Earthquakes

These earthquakes are assumed to have occurred near Costa Rica-Panama border (Limon-Bocas del Toro) (Fig. 2). Extend of the damages reported are similar to that of the 1904 Earthquake but slightly shifted eastwards (Fig. 6). A tsunami occurred and debris and boats were washed 200 m inland at Bocas town. Magnitudes ascribed to these events vary from 7.0 to 7.3. This earthquake is assumed to have occurred along a thrust of the western North Panama Deformed Belt segment.

4.2.5.1.6 The 1991, April 22 Earthquake

The Limon earthquake badly affected an area of 8 000 km2 near Costa Rica-Panama border (Fig. 2). It ruptured the North Panama Deformed Belt which dips lanward at an angle of 30°. The rupture mechanism as analysed by time-histories is quite complex and reveals that several sub-events have likely occurred. Aftershock sequence suggests a 85 x 45 km2 rupture area. A magnitude 7.5 is given for that recorded event.

4.2.5.1.7 The 2000, Feb. 26 Earthquake

An event located 110 km Northeast of Panama City and about 60 km beneath the Serrania de San Blas in the subducted Caribbean plate was felt in Panama City.

4.2.5.1.8 North Panama Deformed Belt seismogenic capability

Although very few data are available on the geological structure of the North Panama Deformed Belt, some general features may be drawn from both the above-described seismicity and the temporary seismogical records made by Schweig et al (1999).

- The North Panama Deformed Belt corresponds to the Caribbean plate subduction beneath the Panama Block. A dip of the interface seems to range between 30° and 40° whereas deeper earthquakes seem to not exceed 60-70 km. This implies that the plate interface is at a depth of about 70 km beneath the Post Panamax Locks.

- Four main segments may be inferred from seismicity and strikes: (i) the NW-SW trending western segment (83°W-81.5°W) characterised by sustained significant seismicity with magnitude greater than 7.0, (ii) the NW-SE trending eastern segment (78.5°W-77°W) which also generated major earthquakes at depth of about 50-70 km, (iii) the E-W trending central East segment (80°W-78.5°W) which generated the major 1882 earthquake, and (iv) the NNE-SSW trending central West segment (81.5°W-80°W) characterised by a seismic quiescence associated by some authors to the absence of subduction.

- As far as the Central West segment is concerned when considering a N105°E convergence of Caribbean plate relative to South America plate, it is quite impossible that thrusting does not actually occur along that NNE-SSW trending segment of the North Panama Deformed Belt implying to consider this segment as a seismic gap.

- Thus, North Panama Deformed Belt segments seem to have a rather constant length of about 150 km, a dip of about 30°-40°, and a maximum depth of about 70 km. These features are consistent with 7.5-8.0 (Mw) magnitude earthquakes (see below)

4.2.5.2 Active faulting and seismicity within the Panama Block

4.2.5.2.1 General pattern of seismicity

Shallow seismicity in Panama Block is not really well-constrained. Since magnitudes are moderate, uncertainties on locations may be significant, and blast events have not been discriminated from the earthquake catalogue. During the USGS temporary (6 months) seismological network, Schweig et al. (2002) only recorded 3 earthquakes at shallow crustal depth, two among them being associated to the Gatun Fault (see below). In the surroundings of Panama City, few features may be drawn (Terral, 2002, pers. comm.): (i) the Panama Block appears relatively aseismic, (ii) earthquakes recorded are always with magnitude lesser than 5.6 (Ms) and (iii) focal depths are generally about 15 km.

Among these moderate to low magnitude-earthquakes, Viquez and Camacho (1992) discussed the 1621, May 2 earthquake with respect to the 1971 (5.6 Ms), January 20 one. We reported here their main conclusions:

- This earthquake is assumed to have occurred offshore between Panama City and Las Perlas islands. A magnitude between 5.6 and 6.0 is ascribed to that event.
- The damage area is similar that observed in 1971.
- It is assumed that this two earthquakes occurred on the same NW-SE trending fault within the Panama Bay.

This fault would be the southern extension of the so-called " Canal Discontinuity" proposed by Case (1974) based on gravity anomalies in Central Panama. To our knowledge, neither geological nor geological evidence supports that inference. Cowan (2001) mentioned that marine geophysical and geological studies in the Gulf of Panama evidenced that no deformation took place along the assumed fault trace since at least 3-5 millions years.

Very recently, on 2002, March 17th , a magnitude 5.2 (Ms) earthquake occurred at 40 km East of Panama City. Focal depth is assumed to be about 15 km, although no fault is known in that place.

This brief overview of the seismicity pattern suggests that the Panama block is an area of widespread moderate to low seismicity with probably a conservative upper threshold of about 5.5-6.0 (Mw). As a consequence, as far as seismic hazard analysis is concerned, taking into account the poor knowledge on the seismological characteristics, one can assume that such an earthquake may occur everywhere within the Panama Block. Of course, in the surroundings of identified or inferred active faults, this threshold has to be increased considering the seismogenic characteristics of the structure.

4.2.5.2.2 The Pedro Miguel Fault

The Post Panamax Locks are located in the close surrounding of the southern extension of the so-called "Pedro Miguel Fault" (Geological Map of the Panama Canal, scale 1;100 000, by Steward) (Fig. 7). This fault seems to vertically offset the Miocene "Las cascadas Formation" underlying by both the Late Oligocene "Caraba Formation" and the Oligocene "Panama Formation".

It must be noted that a secondary N-S trending fault running near the existing Miraflores locks is also mapped over less than 5 km north of the Canal.

The NNW-SSE trending Pedro Miguel Fault is mapped over about 15 km north of the Panama Canal, although its southern extension remains unknown. Tentative extrapolation based on crushed material found during borehole investigations performed in 1942 remains questionable in the present knowledge. According to the state of stress deduced from the fault kinematics assumed by Schweig et al. (1999), the Pedro Miguel fault is expected to be extensive with a right-lateral component.

Analysis of Landsat satellite images and DEM (30 m) shows that the mapped fault displays a narrow furrow morphological signature. This morphology does not suggest sustained normal faulting (i.e. with rate greater than 1 mm/yr). A strike-slip kinematics may be inferred but small wavelength morphological features, generally associated to such a kinematics, are not observed, neither on aerial photo nor in the field along the Post Panamax canal route.

Thus, under the present knowledge, since the Pedro Miguel seems to crosscut the Post-Panamax route near the planned lock site, one cannot disregard the possibility for a shallow earthquake to be generated along that fault. This uncertainties has to be considered for deriving the seismic hazard at the Post Panamax Locks site:

- the limited extension of the fault suggests that its seismogenic capability should not exceed 6.0 (Mw)
- its smooth morphological signature attests of a slow rate (<1 mm/yr),
- the focal depth of such an earthquake should be consistent with those regionally observed (Torral, 2002, pers. comm.) i.e. about 15 km.

In further stages of the design, it is highly recommended to ascertain (i) the actual presence of the Pedro-Miguel Fault on the site, and (ii) the degree of activity of this potential seismogenic source, throughout :

- a “marine” seismic investigations along the Panama Canal between existing Miraflores and Pedro Miguel Locks (as it was performed north of Pedro Miguel locks)
- an on-shore seismic investigations along the Post Panamax Canal route on the expected southern extension of the Pedro Miguel fault.
- If the Post Panamax Locks have to be designed in the close surroundings of that fault, a detailed seismo-tectonic analysis of this fault will be required including superficial geophysics and paleo-seismological analysis throughout trenching if necessary.

Thus, in the present knowledge, it does not seem too conservative to assume that the Pedro Miguel Fault may represent a critical seismogenic source in the near-field. Partial reactivation (about 10 km long) seems reasonable for deriving its maximum seismogenic capability. This feature is consistent with 5.6-6.0 (Mw) magnitude earthquakes recorded in the Panama Block.

4.2.5.2.3 Active faulting in the Panama Block

Other intraplate faults are reported in the surroundings of the Panama Canal, but their seismogenic capabilities and their significant distances to the Post Panamax Locks site make them not critical for the present seismic hazard analysis. They are just mentioned hereafter:

- The ENE trending Rio Gatun fault bounds the Northwest margin of the Madden Basin and the Mountain front of the Sierra Maestra. It separates middle/late Tertiary sediments of Madden basin from older (probably Cretaceous-Paleocene) sedimentary and igneous rocks. It has been mapped from the Gatun Lake to Boqueron River at the north end of the Madden basin. Morphological signature suggest normal faulting although no late Quaternary offsets have been documented on alluvial terraces or fans (Schweig, 1999). Rate <1 mm/yr is inferred. Seismicity is weakly connected to that structure. Recent seismic hazard analysis assumed a seismognenic capability of 6.8 (Mw) along that the Rio Gatun Fault. Its shortest distance to the Post Panamax Locks site is about 40 km.

- The N-S trending, east facing Azote and Limon normal faults are located west and south of the western termination of the Rio Gatun Fault (Geological Map of the Panama Canal, scale 1;100 000, by Steward). As far as we know, no recent faulting has ever been evidenced along those two faults. Taking into account their lengths, it seems reasonable to deduce that both their seismogenic capability and their degree of activity are comparable to, and even lower than, the Pedro Miguel Fault. The shortest distance to the Post Panamax Locks site is about 20 km.

4.2.6 SEISMIC HAZARD ASSESSMENT

4.2.6.1 Introduction

The seismic hazard assessment has been carried out in several steps:

- Determination of the period of return of design earthquakes related to the “acceptable” risks considering the service-life.

- Determination of the magnitude of the reference earthquakes likely to occur on each identified active faults or/and seismogenic sources during the time-window related to the acceptable risks.

- Estimates of the Peak Ground Accelerations (PGA) at site induced by each reference earthquake on each identified active faults.

- Estimates of the acceptable ground motions at site.

4.2.6.2 Design Earthquakes

We adopt here the recommendations of the "International Commission on Large Dams" (ICOLD) ([1]) suggesting 3 design earthquakes : the **Maximum Credible Earthquake (MCE),** the **Maximum Design Earthquake (MDE),** and the **Operating Basis Earthquake (OBE)**. However in order to apply if necessary other codes such as the Uniform Building Codes, the **Site Maximum Design Earthquake (SMDE)** and the **Site Design Earthquake (SDE)** are also provided. These reference earthquake are expressed in terms of ground motions at the dam site, generally the Peak Ground Acceleration. The attention is drawn to the fact that in this case the "dam" is equivalent with the "lock structure".

- The **Maximum Design Earthquake (MDE)** will produce the maximum level of ground motion for which the dam should be designed or analysed. The dam should stay "fail safe". For dams whose failure would present a great social hazard, the MDE will normally be characterised by a level of motion equal to that induced by the most severe combination of maximum magnitude and minimum distance to the site independent of the return period. Should the failure of the dam present no hazard to life, a lower level of ground motion may be acceptable, based on economic considerations. It is commonly accepted that the probability of occurring of the MDE should be about 10% during the service life (about 100 years) which corresponds to a 950-years-return period.

- The **Operating Basis Earthquake (OBE)** represents the level of ground motion at the dam site at which only minor damage is acceptable. The dam, appurtenant structures and equipment should remain functional and damage easily repairable, from the occurrence of earthquake shaking not exceeding the OBE. It is derived taking account both the design period of the structures and the past seismicity identified either in catalogues or dated in the field. Consequently, it is highly dependent on knowledge of the seismogenic behaviour of critical active faults in time. Typically, the probability of occurring of the OBE should be about 50% during the service life (about 100 years) which corresponds to a 145-years-return period

- With each of these design earthquakes generating features the notion of **Maximum Credible Earthquake (MCE)** is linked. This is the largest reasonably conceivable earthquake that appears possible along a recognised fault or within a geographically defined tectonic province, under the presently known or presumed tectonic framework. The MCE is generally defined as an upper bound of expected magnitude.

In UBC code, two levels of earthquakes are required :

- the Site Maximum Design Earthquake (SMDE) associated to a mean-return period of about 1000 years (i.e. about 5%-probability of occurring during a 50-years-service life).

- the Site Design Earthquake (SDE) associated to a mean-return period of 475 years (i.e. about 10%-probability of occurring during a 50-years-service life).

Table 1 shows the return periods associated to each above-mentioned design earthquakes.

(2) *ICOLD, 1989 - Selecting Seismic Parameters for Large Dams - Guide-lines.*

Table 1 : Return period associated to design earthquakes

Design Earthquakes	**Mean return Period (in years)**	**Codes**
MCE	-	ICOLD
MDE	950	ICOLD
OBE	145	ICOLD
SMDE	*1000*	*UBC*
SDE	*475*	*UBC*

4.2.6.3 Estimate of the Maximum Credible Earthquake (MCE)

A maximum magnitude (Maximum Credible Earthquake, MCE) has to be assigned to each identified critical seismogenic source, e.g. faults capable of representing a threat to the site of interest. This is the largest reasonable conceivable earthquake that appears possible along a recognised fault, under the presently known or presumed tectonic framework. The capability of these faults should have been ascertained through historical and instrumental seismic data and geological criteria such as the rupture length-magnitude, the rupture area, the slip-vector-magnitude, the slip-rate-magnitude and seismic moment-magnitude relationships.

Recently, the use of seismic moment has provided a physically meaningful measurement of the size of a faulting event. Seismic moment (Mo), in dyne-cm, is expressed by the equation:

$$Mo = \mu . Af . D$$

in which μ is the shear modulus of the material along the fault plane and is typically equal to $3x10^{11}$ dyne/cm^2 for crustal rocks, Af is the area, in square centimetres, of the fault plane undergoing slip, and D, in centimetres, is the average displacement over the slip surface.

Seismic moment provides a basic link between the dimensions of the fault and the seismic waves radiated due to rupturing along the fault. Seismic moment is therefore more reliable than the other methods. Kanamori (1977*[2]*) and Hanks and Kanamori (1979*[3]*) introduced a moment-magnitude scale, Mw, in which magnitude is calculated from seismic moment using the following formula:

$$\text{Log}\, Mo = c\, Mw + d$$

The moment-magnitude is different from other magnitude scales because it is directly related to average slip and ruptured fault area, while the other magnitude scales reflect the amplitude of a particular type of seismic wave.

The MCE on each identified active fault segment may be derived using a scale transfer from the fault-area (Wells et Coppersmith (1994 *[4]*). These authors compiled worldwide surface breaks related to earthquakes and derived relationships between the seismic moment and the fault-area according to the kinematics of the fault broken.

[2] Bull. Seism. Soc. Am., 69, 1645-1670.
[3] J.G.R., 84, 2348-2350.
[4] Bull. Seism. Soc. Am., 84, 974-1002.

For reverse faults, the relationship is the following :

$$Mw = 4{,}33 + 0{,}9 \times \mathrm{Log}(S)$$

For normal faults, the relationship is the following :

$$Mw = 3{,}93 + 1{,}02 \times \mathrm{Log}(S)$$

with S, the fault area in km^2 and a standard deviation of 0.25.

In the present knowledge, the Maximum Credible earthquake to be taken into account in the Post Panamax Locks is the following :

- **For the North Panama Deformed Zone**, taking into account both the historical seismicity (in particular the 1822, 1882, 1904, 1914, 1916 and the 1991 earthquakes), the segmentation and the dip of Caribbean-Panama interface above-described, and assuming a seismic rupture of about 130 km long over the half of the seismogenic layer, it seems that the seismogenic capability of this seismogenic source range between 7.7 and 8.0 (Mw) corresponding to a rupture area of 6600 ±600 km^2 with a co-seismic displacement of about 3.5-5.5 m

- **For the Pedro Miguel Fault**, taking into account (i) the widespread historical seismicity within the Panama Blocks, and in particular the 1621, 1971, and the March 17th , 2002 earthquakes, (ii) the length of this fault as reported on the 1:100,000 geological map (Steward), and (iii) the length of the morphological signature of this fault observed both on TMLandsat image and on the DEM provided by ACP, it seems that the seismogenic capability of this seismogenic source cannot exceed a magnitude (Mw) of 5.8 ±0.4 corresponding to a rupture area of 75 ±25 km^2 with a co-seismic displacement of about 15-30 cm.

4.2.6.4 Recurrence of Seismicity of Critical Seismogenic Source

The seismic parameters that characterise each area are defined by the frequency distribution law or recurrence curve for the different sizes of earthquakes in each area. This distribution was defined by Gutenberg and Richter (1944) and assumes that the number of earthquakes (N) decreases exponentially with its magnitudes (M), according to the following formula:

$$\log 10\ N(M) = a - bM$$

where N(M) is the number of earthquakes greater than M, "a" the annual earthquake rate of a magnitude greater than 0 in the region, and "b" is the value which defines the proportion of major earthquakes compared with small earthquakes.

This equation assumes that all earthquakes are independent of space and time, i.e. it has the properties of a Poisson model.

According to the detailed analysis of the earthquake catalogue carried out by Camacho et al. (1994) for updating the seismic hazard for Panama, the b-value must be taken as equal to 1 for all the Panama territory.

4.2.6.5 Estimate of the reference magnitudes

The seismic moment, M_O, is the most physically meaningful parameter to describe the size of an earthquake in terms of static fault parameters

$$Mo = mA_rD$$

where μ is the rigidity or shear modules (usually taken to be about 3 x 10^{11} dyne/cm²), A_r is the rupture area on the fault plane undergoing slip during the earthquake, and D is the average displacement over the slip surface. The total seismic moment rate $M_O{}^T$ or the rate of seismic energy release along a fault is estimated by:

$$M_o^T = mA_fS$$

where S is the average slip rate along the fault, and Af the total fault plane area. The seismic moment rate provides an important link between geologic and seismicity data. Seismic moment is translated to earthquake magnitude according to an expression of the form.

$$\log M_o = c.m + d$$

The moment magnitude, m, is considered to be equivalent to the local magnitude in the magnitude range $3 < M_L < 7$ and to the surface wave magnitude in the magnitude range $5 < M_S < 7½$.

Once the fault slip rate is used to constrain the seismic moment rate on the fault, a model must be assumed for the manner in which the rate of moment release is distributed to earthquakes of various magnitudes. Several authors have developed relationships between earthquake recurrence parameters and fault or crustal deformation rates, assuming an exponential magnitude distribution.

The link between fault slip rate and earthquake rates is made through the use of seismic moment.

The total rate of seismic moment can be related to earthquake occurrence rate by the expression

$$\dot{M}_T^o = \int_{-\infty}^{Mo} n(m).Mo(m).dm \quad (1)$$

where n(m) is the density function for earthquake occurrence rate and may be expressed according to Youngs and Coppersmith (1985[5]) as:

$$n(m) = \frac{N(m^o)\, b \exp(-b\left(m - m^o\right)}{1 - \exp\left(-b\left(m^u - m^o\right)\right)} \text{ for } m \leq m^u$$

$N(m^o)$ is the normalised number of events per unit time, m^u is the upper bound magnitude ($n(m) = 0$ for $m > m^u$), m^o is some arbitrary reference magnitude, β=b.Ln10 with b from the Gutenberg-Richter exponential frequency magnitude relationship log N(m) = a - b.m, when N(m) is the cumulative number of earthquake of magnitude greater than m, and a and b are constants.

Integrating equation *(1)*, we obtained we obtained the reference magnitude corresponding to a given return period as function of the upper bound magnitude m^u, the associated upper bound seismic moment, the slip-rate assumed along the fault, the chosen period of return, the b-value from the Gutenberg-Richter exponential frequency magnitude relationship, and the total fault plane area.

$$mA_fS = bN\left(m^o\right) M_o^u \exp\left(-b\left(m^u - m^o\right)\right) / (c - b)\left(1 - \exp\left(-b\left(m^u - m^o\right)\right)\right) \quad (2)$$

which is equivalent to the relationship developed by Anderson (1979) and the type 2 relationship presented by Anderson and Luco (1983). The term M_o^u is the moment for the upper bound magnitude m^u. Assuming the slip rate, S, on a fault is known, equation *(2)* provides a constraint on the three parameters of the recurrence relationship $N(m^o)$, b, and m^u.

The constraint imposed by fault slip rate allows the development of fault-specific recurrence relationships in regions where the historical seismicity data are only sufficient to establish the regional recurrence rate for small to moderate size earthquakes. For each fault in the region, estimates of the upper bound magnitude, m^u, can be made using fault characteristics. The historical seismicity data can be used to determine a regional b-value. Assuming the individual

[5] *Bull. Seism. Soc. Am., 75, 939-964.*

faults have all a b-value equal to the regional b-value, the earthquake activity rate for each fault, $N(m^o)$, can be computed from the estimated slip rate for the fault using equation *(2)*.

Hereinafter (Table 2), we estimate the reference magnitudes along each identified active faults taking into account the period of return of the reference earthquakes ($N(m_o) = 1/Tm$), the slip-rate along the faults (s) the maximum seismogenic capability of the faults and the b-value.

$$m^o = m^u - \frac{1}{b.\ln 10} \ln\left[1 + \frac{N\left(m^o\right) \times b \times M_o^u}{\left(1.5 - b\right) m\, Af\, S}\right] \qquad (3)$$

where M_o^u is the seismic moment corresponding to the maximum credible magnitude along a given fault.

The parameters selected for estimating the magnitude associated to the design return period are :

- North Panama Deformed Belt

$$\begin{aligned} m^u &= 7.77 \pm 0.3 \text{ (Mw)} \\ b &= 1 \\ Af &= 6600 \pm 600 \text{ km}^2 \\ S &= 10\text{-}15 \text{ mm/yr} \end{aligned} \qquad ([6])$$

- Fault in the near-fied such as the Pedro Miguel Fault

$$\begin{aligned} m^u &= 5.8 \pm 0.4 \text{ (Mw)} \\ b &= 1 \\ Af &= 75 \pm 25 \text{ km}^2 \\ S &= 0.5\text{-}1 \text{ mm/yr} \end{aligned}$$

Curves of magnitude versus mean return periods on each identified fault are reported on figure 8.

<u>Table 2</u> : Estimate of the reference magnitudes along the identified seismogenic sources

Seismogenic Sources	**MCE**	s	**MDE**	s	**OBE**	s	**SMDE**	s	**SDE**	s
North Panama Deformed Belt	7.77	0.30	7.60	0.14	7.2	0.07	7.60	0.15	7.48	0.10
Pedro Miguel Fault	5.82	0.40	5.53	0.18	5.03	0.13	5.54	0.19	5.38	0.15

([6]) According to the above-mentioned relationships (3), uncertainty lesser than 10% on the estimate magnitude is obtained when selecting b=1

4.2.6.6 Attenuation Laws

The **Peak Ground Accelerations (PGA)** (peak zero period acceleration) at the bed-rock at sites corresponding to the reference earthquakes (MCE, MDE and OBE, and SMDE and SDE) are inferred from attenuation laws of common practice in Engineering. This solution is statistically more representative than the derivation of a reference ground motion from only one accelerogram.

4.2.6.6.1 Campbell, 1997

Campbell (1997) summarises the results of several years work developing empirical relations for horizontal and vertical PGA, PGV and SA in active tectonic regions. These latest versions were developed using much larger data set than earlier versions. Campbell's suite of attenuation relationships are designed to be used to estimate ground motion from earthquakes of M>5.0 at sites within 60 km. The 84-percentile PGA value is conservatively adopted in the present analysis.

4.2.6.6.2 Sadigh, 1987

The Sadigh relationship (1987) based on worldwide data instead of exclusive Californian data, specific for shallow depth earthquakes. This predictive equation for randomly oriented horizontal component of peak acceleration has been developed for both rock sites. Equation was derived for strike-slip earthquakes. The 84-percentile PGA value is conservatively adopted in the present analysis.

4.2.6.6.3 Dahle, 1995

Dahle (1995) derived new spectral strong motion attenuation models for Central America. Empirical relations for horizontal PGA and SV are developed for rock and soil conditions. . The 84-percentile PGA value is conservatively adopted in the present analysis.

4.2.6.6.4 Joyner, 1999

Joyner (1999) derived an attenuation law for magnitude-7.7 subduction-earthquakes based on the 1985 Valparaiso Chilean earthquake. For that reason, this law is used for the North Panama Deformed Belt MCE. However, since this law is quite specific and rather conservative with respect those available in the scientific literature, it is recommended to only consider the median value.

4.2.6.7 Peak Ground Acceleration at site

Taking into account the North Panama Deformed Belt geometry, the closest distance to the Post Panamax Lock site is 60 km.

Taking into account the Pedro Miguel Fault and the general pattern of the seismicity in Panama Blocks, it has been assumed a focal depth of 15 km for earthquakes associated to that source

Curves of constant risk on each identified fault are reported in figure 9 and 10. In the table 3, Peak Ground Acceleration at site associated with the MCE, MDE, OBE together with SMDE and SDE design earthquakes are reported.

Recommended PGA for Post Panamax Locks are reported in table 4.

Table 3 : Estimate of the reference PGA (in g) generated at site (rock)

Faults	Laws	MCE	MDE	OBE	SMDE	SMD
North Panama Deformed Belt	Joyner 99	0.21	-	-	-	-
	Sadigh 87	0.18	0.13	0.10	0.13	0.12
	Campbell 97	0.11	0.10	0.07	0.10	0.09
	Dahle 95	0.21	0.19	0.17	0.19	0.18
	Recommended	**0.21**	**0.16**	**0.13**	**0.16**	**0.15**
Pedro Miguel Fault	Sadigh 87	0.22	0.19	0.13	0.19	0.17
	Campbell 97	0.20	0.16	0.11	0.16	0.14
	Dahle 95	0.18	0.17	0.14	0.17	0.16
	Recommended	**0.21**	**0.18**	**0.13**	**0.18**	**0.16**

Table 4 : Recommended PGA (in g) for Post-Panamax Locks

Post Panamax Locks	MCE	MDE	OBE	*SMDE*	*SMD*
Recommended PGA (in g) ***(for rock conditions)***	**0.21**	**0.18**	**0.13**	***0.18***	***0.16***
	ICOLD			*U B C*	

4.2.7 CONCLUSIONS

The Post Panamax Locks are located in Panama Block along the Pacific coast in a very complex tectonic zone where South America, Central America, Caribbean, Nazca and Cocos plates interfere giving rise to a significant seismicity.

A regional probabilistic seismic hazard carried out by Camacho et al. (1994) results for the Post Panamax Locks area in Peak Peal Ground Accelerations at bedrock of 0.17 g, 0.20 g and 0.24 g for mean return periods of 250, 500 and 1000 years respectively.

In order to validate/or implement this seismic hazard, an analysis of available scientific literature and technical reports has been conducted in March 2002. The main results are the following :

- the North Panama Deformed Belt which corresponds to the Caribbean plate subduction beneath the Panama Block actually controls the far-field seismic hazard.

- the NNW-SSE trending Pedro Miguel Fault which probably crosscut the Post Panamax route actually controls the near-field seismic hazard.

- Reference magnitudes (m^0) associated to design earthquakes as recommended by ICOLD and UBC, have been derived based on fault segmentation, kinematics, rates and historical seismicity.

Return periods->	**MCE** -	**MDE** 950 yrs	**OBE** 145 years	***SMDE*** *1000 years*	***SDE*** *475 years*
North Panama Deformed Belt	7.77	7.60	7.2	*7.60*	*7.48*
Pedro Miguel Fault	5.82	5.53	5.03	*5.54*	*5.38*
	ICOLD			*UBC*	

- Peak Ground Accelerations (PGA) at site versus return periods have been derived throughout various attenuation laws. Results for the design earthquakes are reported hereafter:

	MCE	**MDE**	**OBE**	***SMDE***	***SMD***
North Panama Deformed Belt	0.21	0.16	0.13	*0.16*	*0.15*
Pedro Miguel Fault	0.21	0.18	0.13	*0.18*	*0.16*
Recommended PGA (in g) ***(for rock conditions)***	**0.21**	**0.18**	**0.13**	***0.18***	***0.16***
	ICOLD			*UBC*	

- ***It is emphasised that the Pedro Miguel Fault seems to crosscut the Post-Panamax route near the planned lock site. According to the MCE associated to that fault, a maximum co-seismic displacement of 15-30 cm cannot be disregarded although the degree of activity seems very low.***

Recommendations : in further stages of the design, it is highly recommended to ascertain (i) the actual presence of the Pedro-Miguel Fault on the site, and (ii) the degree of activity of this potential seismogenic source, throughout :

- a "marine" seismic investigations along the Panama Canal between existing Miraflores and Pedro Miguel Locks (as it was performed north of Pedro Miguel locks)
- an on-shore seismic investigations along the Post Panamax Canal route on the expected southern extension of the Pedro Miguel fault.
- if the Post Panamax Locks have to be designed in the close surroundings of that fault, a detailed seismotectonic analysis of this fault will be required including superficial geophysics and paleosismological analysis throughout trenching if necessary.

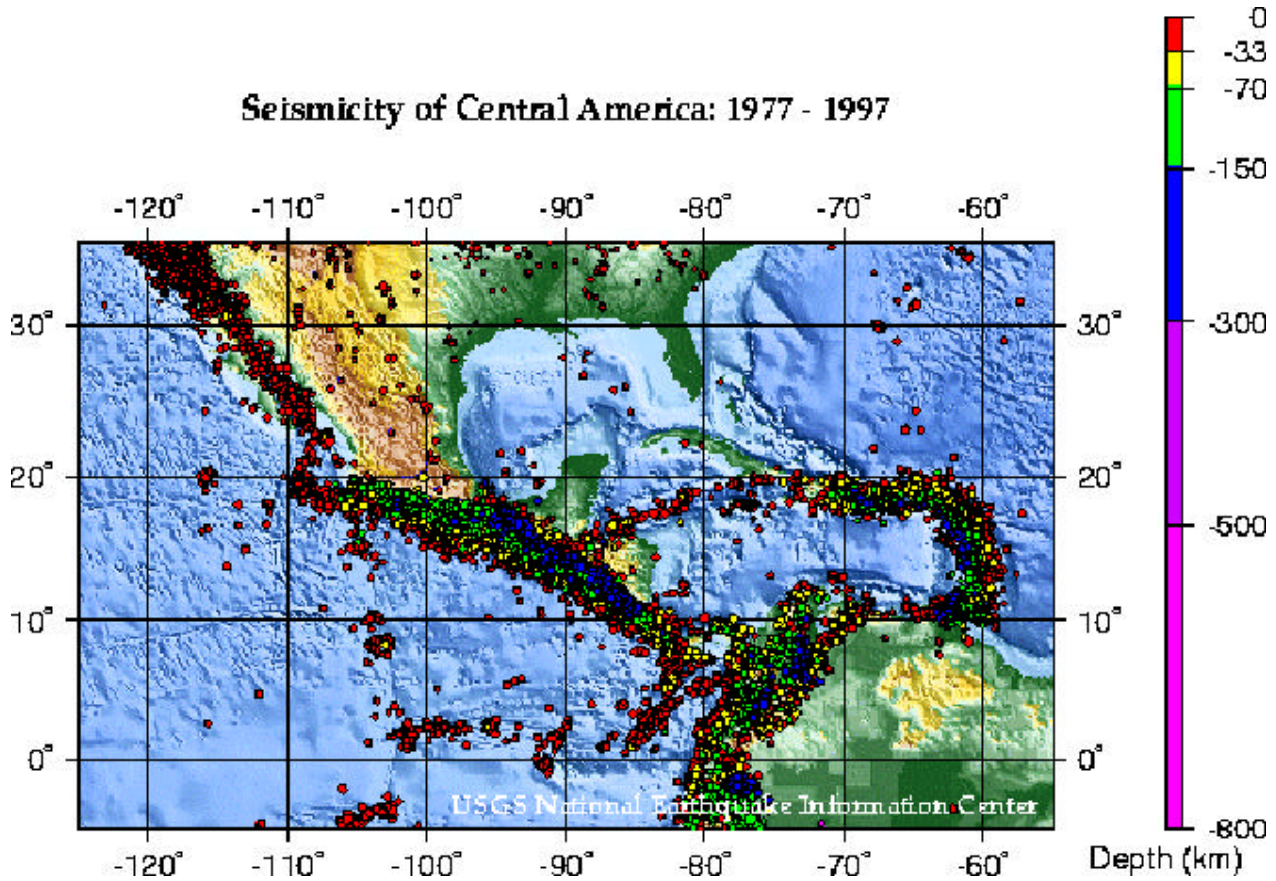


Figure 1: Seismicity of Central America 1977-1997

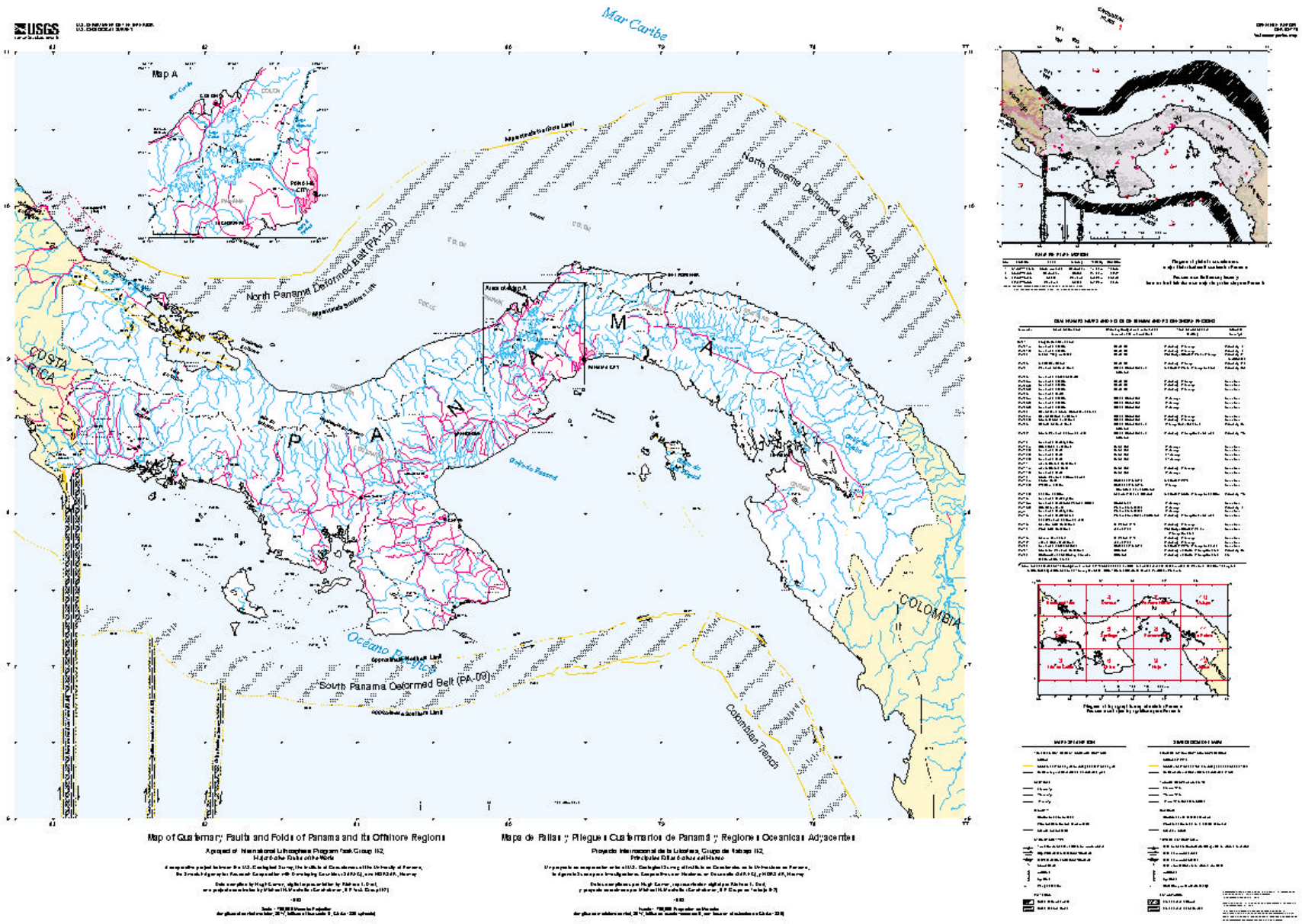


Figure 2 : Seismotectonic map of Guatemala

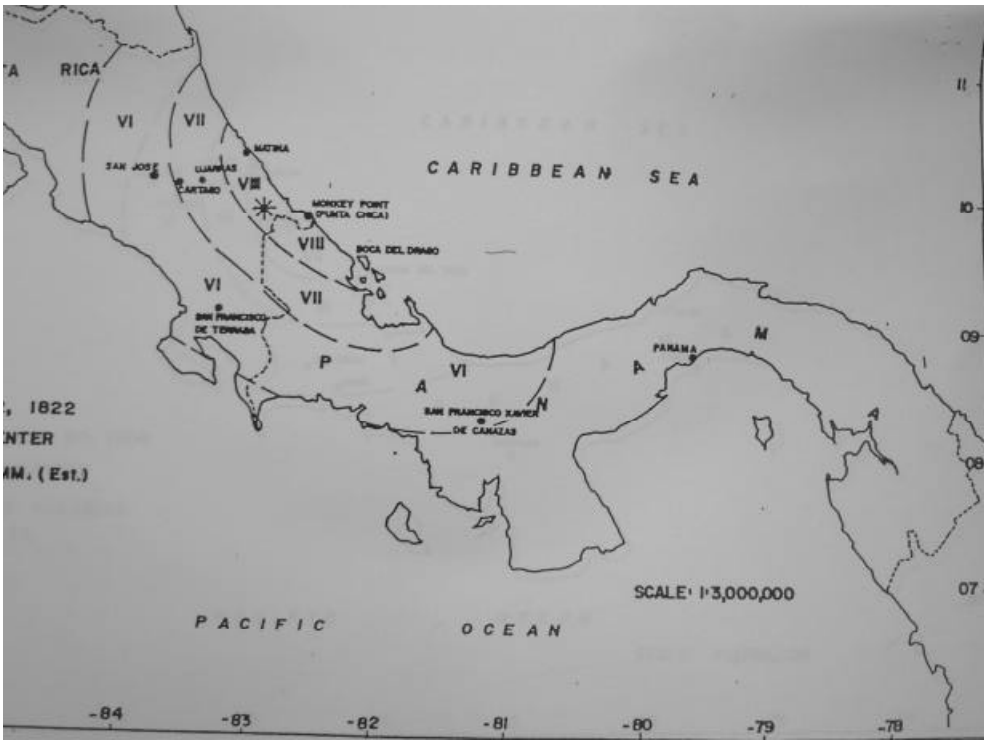


Figure 3: Macroseismic map of the 1822 earthquake. Star represent the assumed epicenter

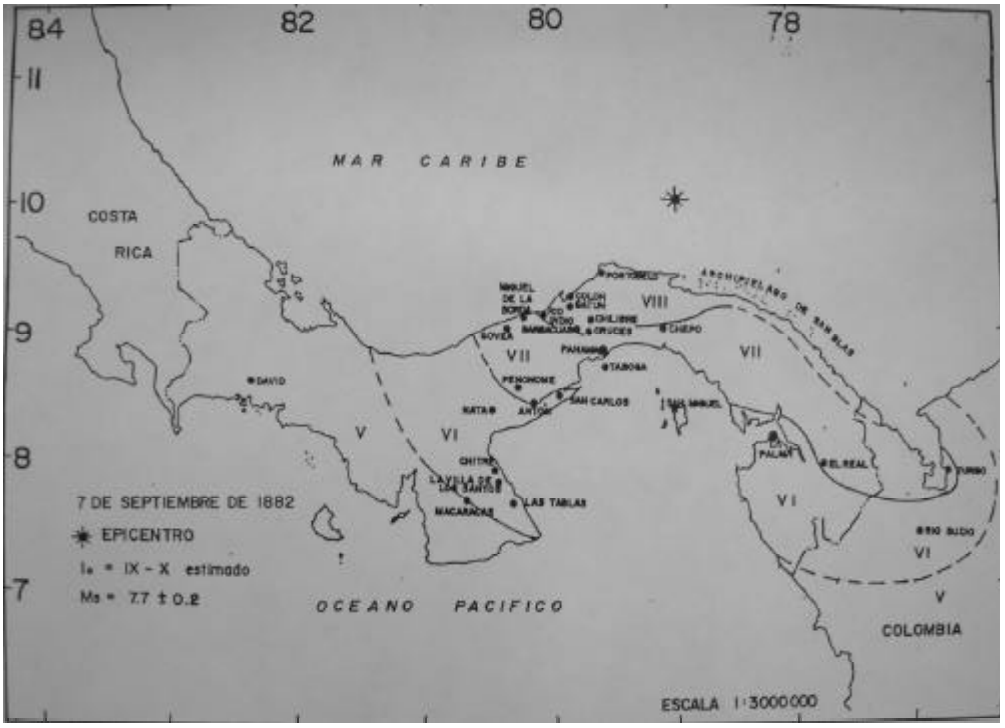


Figure 4: Macroseismic map of the 1882 earthquake. Star represent the assumed epicenter.

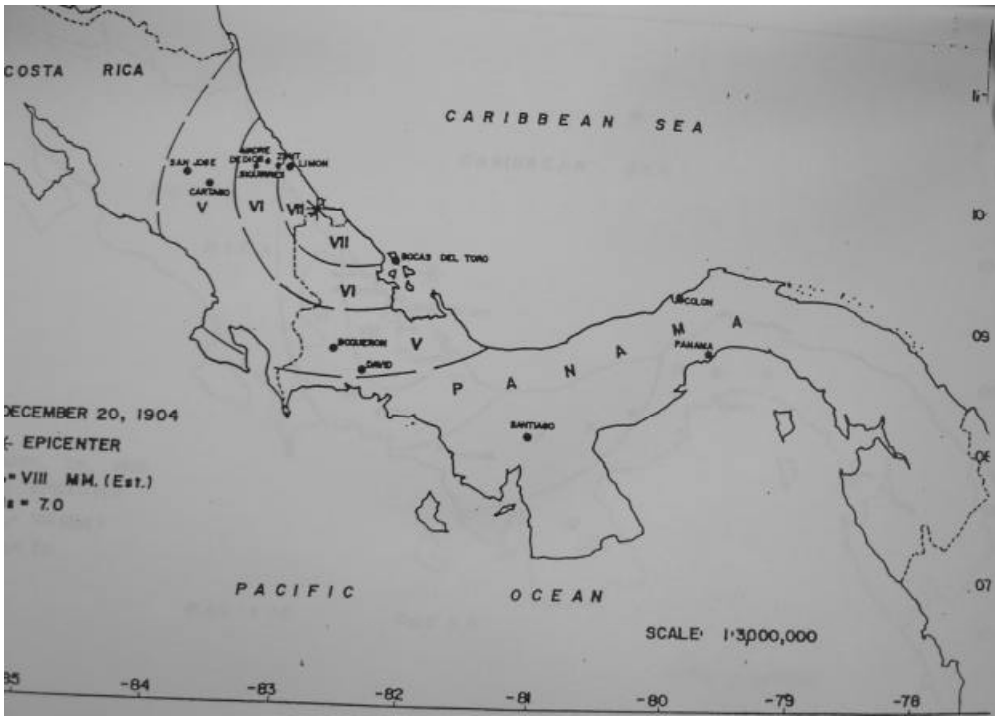


Figure 5: Macroseismic map of the 1904 earthquake. Star represent the assumed epicenter.

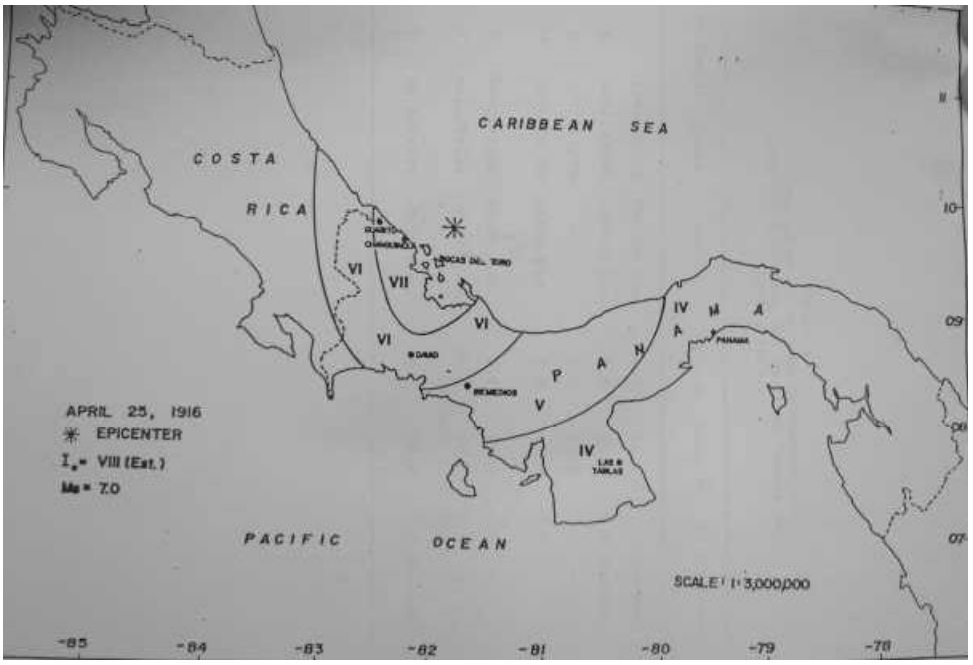


Figure 6: Macroseismic map of the 1916 earthquake. Star represent the assumed epicenter.

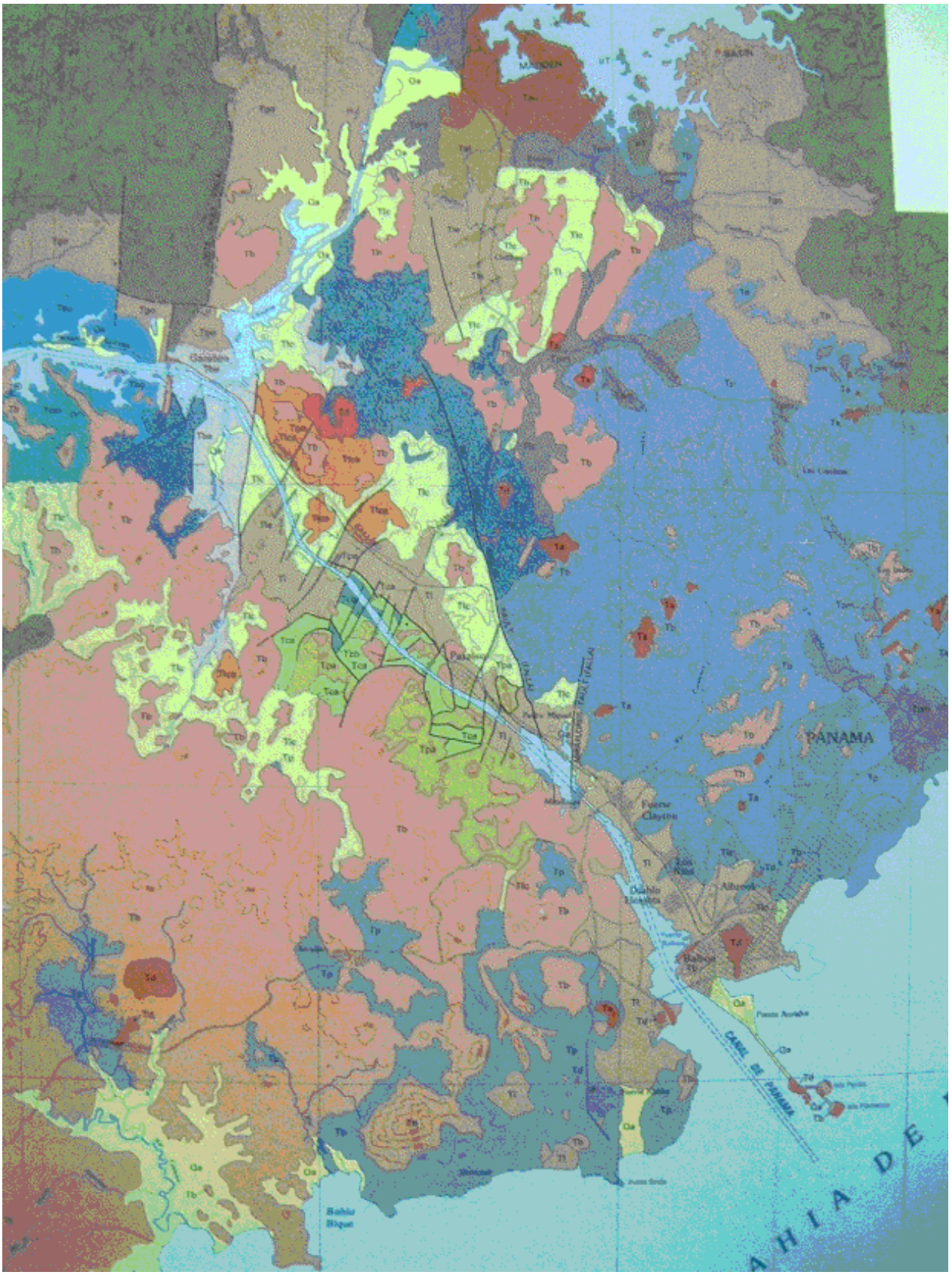
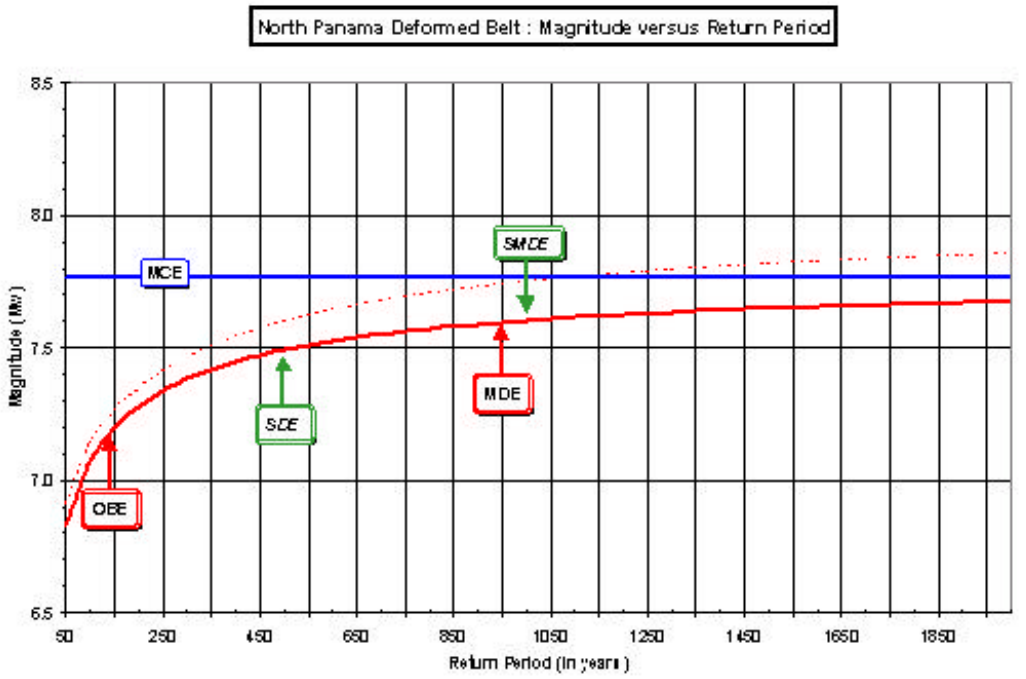


Figure 7: Extract of the Geological Map of the Panama Canal, showing the Pedro Miguel Fault relative to the Post Panamax Locks site (West of Miraflores existing locks).



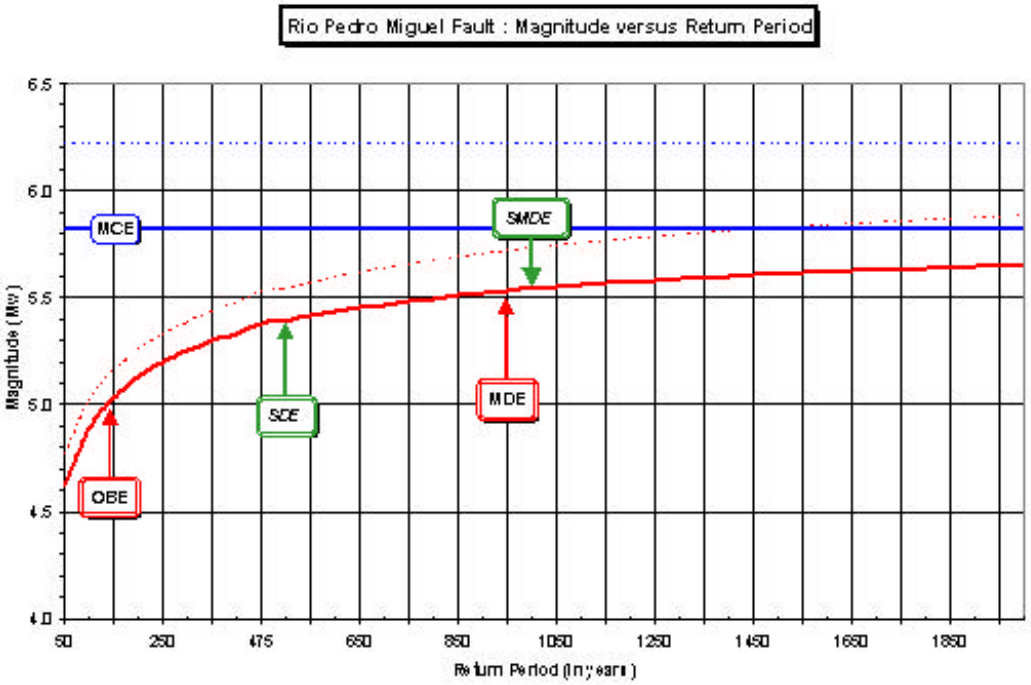
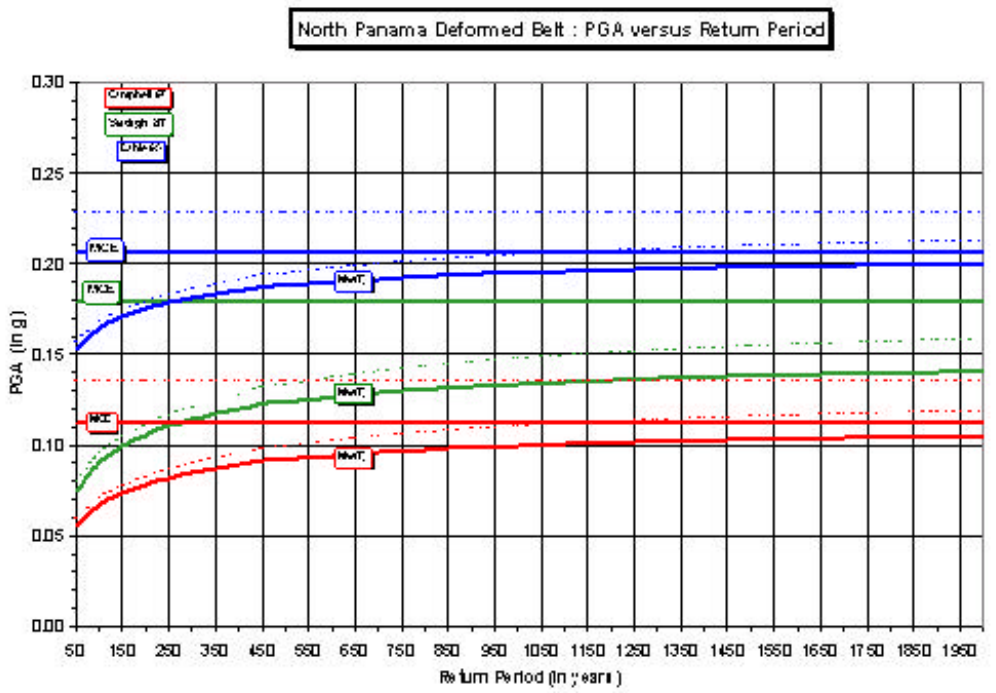


Figure 8: Curves of magnitude versus return periods for critical seismogenic sources



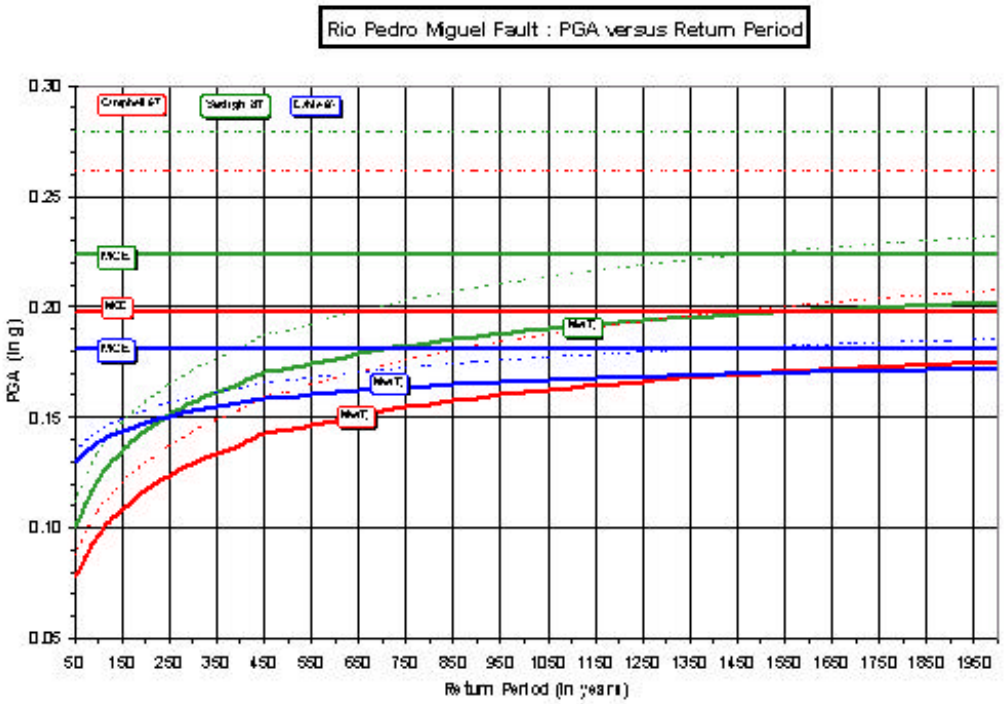


Figure 9: Curves of PGA versus return periods for critical seismogenic sources

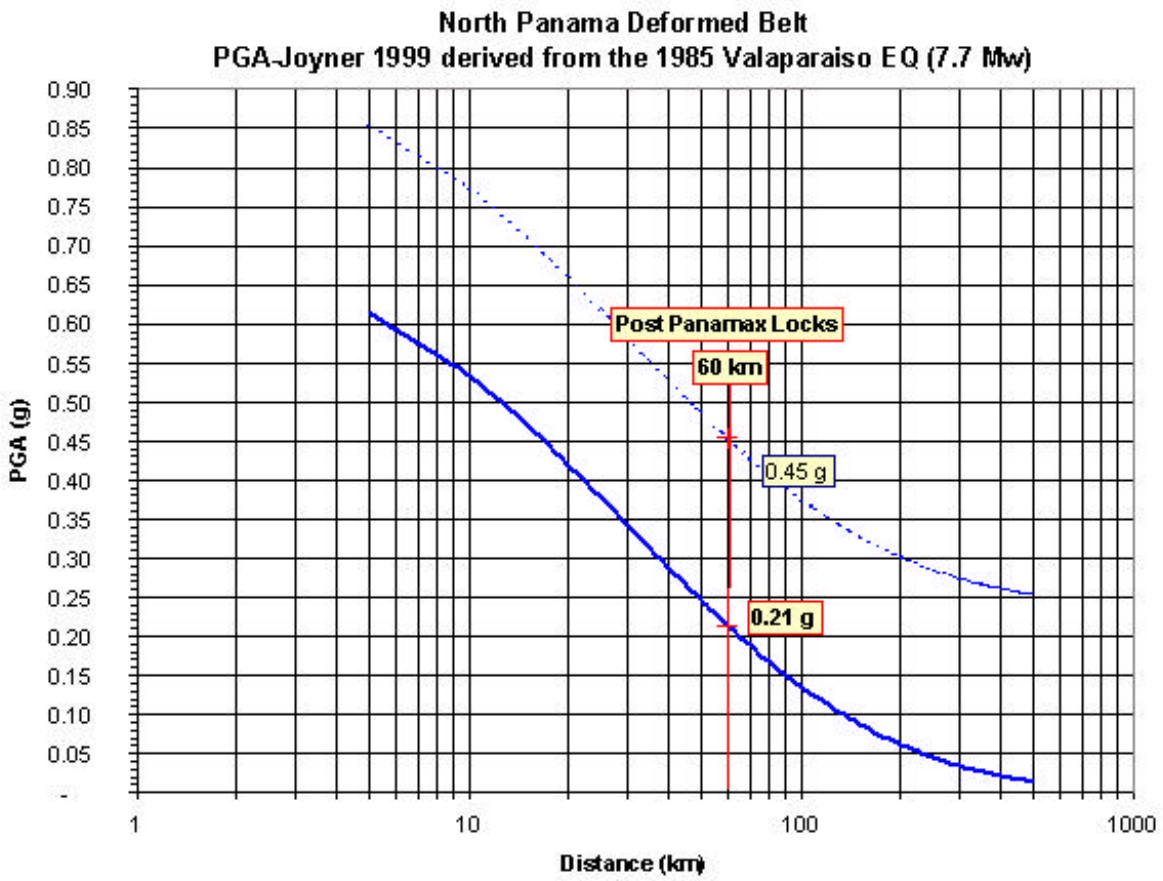


Figure 10: Curve of PGA derived from the 1985 Valparaiso earthquake. PGA expected for the MCE at Post-Panamaxcks.

5 Electrical and Power Requirements

5.1 DESIGN CRITERIA .

The design of electrical and power equipment shall be based on following criteria:

- Reliability of operation,
- Use of Uninterruptible Power Supply (UPS),
- Create provisions for emergency operation in case of failure
- Redundancy of important equipment,
- Special requirements for operation in a humid tropical environment,
- Suitable dimensioning for operating loads (acceptable lifetime),
- Allow a quick and easy maintenance (easy replacement and serviceability),
- Provide rapid and efficient operation,
- Know the statement of the different technical installations,
- Use of medium voltage of 6.900 V or 11.000 V or 15.000 V as existing.

5.2 STANDARDS - REFERENCES

5.2.1 ELECTROMAGNETIC COMPATIBILITY:

Limits for disturbances in supply systems (harmonic currents emissions, voltage fluctuations and flicker in low voltage supply systems) according to IEC 61003-2 and IEC 61000 3-3.

5.2.2 MV SWITCHGEAR

Design according to :

- IEC 60694 : Common specifications for high-voltage switchgear and controlgear standards,
- IEC 60298 : AC metal-enclosed switchgear and controlgear for rated voltages above 1kV and up to and including 52kV,
- IEC 60056 : AC High Voltage alternating current circuit breakers,
- IEC 60470 : AC High Voltage contactors and contactor-based motor starters
- IEC 60265-1 : High Voltage switches (> 1kV, < 52kV)
- IEC 60282-2 : High Voltage fuses.

Features :
Disconnecting switchgear with SF6 or vacuum insulation
Easy maintenance in the medium voltage section
Sealed enclosure providing protection against dirt, animals and humidity.
Modern digital control and protection equipment with full automation and communication capability.

5.2.3 TRANSFORMERS

Use of mineral oil transformers or dry dielectric transformers

Design according to :
IEC 60076 Power transformers
IEC 726 for dry type power transformers
Transformers shall be placed in a ventilated protection enclosure.

5.2.4 LOW VOLTAGE SWITCHGEAR 400V AC

Design according to :
IEC 60439 -1 / EN60439-1 : low voltage assemblies

Features :
Sealed enclosure providing protection against dirt, animals and humidity. Modern digital control and protection equipment with full automation and communication capability.

5.2.5 CABLES

- Common

The choice of the outer sheath of the cable shall be made according to humidity condition. (PVC, ...)

- Medium Voltage (up to 15 kV) :

Design according to NBN C33-323 or equivalent standards.

- Low voltage (up to 1 kV) :

- Design according to IEC 502 NAD
- Safety cables : For more security, the cables supplying safety equipment, shall be fire resistant with insulation integrity 3 h under impact of fire and mechanical chock according to IEC 60331.
- For the control room, we propose to use halogen free material. Design according to IEC 61034-1 + 2 : Smoke density and IEC 60754-2 : Test of gases evolved during the combustion of materials taken from electric cables.

- Data cables :

Cables with metallic screen shall be used.

- Telecommunications cables :

Choice shall be made according to the specification of the national telecommunication operator of PANAMA.

5.2.6 DIESEL EMERGENCY SET(S)

Emergency set shall be mainly used in case of failure of the normal power supply.

Design according to:
- IEC 60034-2 -2A -4 -6 -8 -11 (rotating electrical machines),
- EN 60034-5 (degrees of protection),
- ISO 3046-1 -3 -6: AC combustion engine.

The use of a diesel UPS system shall be decided according to the total electrical power to be secured.

5.2.7 UPS – RECTIFIER - CHARGER

Static UPS shall be used for all control and protection circuits.
Design according to IEC standards as:
IEC 60146
IEC 62040

Redresser shall be used for the production of DC current necessary for the electrical auxiliary, the information network
DC voltage: 110 V, 48 V or 24 V.

Type of batteries: lead acid standby batteries.

5.2.8 BASIC DESIGN OF THE ELECTRICAL INSTALLATIONS

The neutral point of the transformers shall be connected to earth.
Al metallic parts of electrical equipment shall be connected to the neutral conductor.

Power sources available :
- 2 different Medium Voltage sources
- 1 or 2 Diesel set.

Two different electrical substations shall be available.
Uninterruptible Power Supply shall be used for safety equipment and control equipment.

Dimensioning of the installations according to:
NF C 15-100
IEC 60909

Diseño Conceptual de las Esclusas Post Panamax

TASK 2

PART B: SPECIFIC DESIGN CRITERIA

R2-B

Tractebel Development Engineering
TECHNUM
COYNE ET BELLIER
CNR PROJETS

in association with

IMDC
SBE
FITA AOSO

1	**Lock Walls and Entrance Walls**	**1-1**
1.1	**SCOPE**	**1-1**
1.2	**DESIGN CODES**	**1-1**
1.3	**IMPOSED BY THE TERMS OF REFERENCE**	**1-1**
1.4	**LIMIT STATES**	**1-2**
1.4.1	*EXTERNAL STABILITY*	*1-2*
1.4.2	*GLOBAL STABILITY (DEEP SEATED SLIDING)*	*1-4*
1.4.3	*HYDRAULIC STABILITY*	*1-4*
1.4.4	*INTERNAL STABILITY*	*1-6*
1.4.5	*SERVICEABILITY*	*1-6*
1.5	**LOADS**	**1-6**
1.5.1	*DEAD LOADS - PERMANENT*	*1-6*
1.5.2	*HYDROSTATIC LOADS - VARIABLE*	*1-7*
1.5.3	*EARTH - PERMANENT*	*1-7*
1.5.4	*WIND - VARIABLE*	*1-7*
1.5.5	*TEMPERATURE AND SHRINKAGE - VARIABLE*	*1-7*
1.5.6	*SEISMIC LOADS - ACCIDENTAL*	*1-7*
1.5.7	*WAVES – VARIABLE*	*1-7*
1.5.8	*MAINTENANCE LOAD CASES – VARIABLE*	*1-8*
1.5.9	*SHIP COLLISION - ACCIDENTAL*	*1-8*
1.5.10	*LOCOMOTIVES*	*1-8*
1.5.11	*LIVE LOAD (OTHER THAN LOCOMOTIVES) - VARIABLE*	*1-9*
1.6	**LOAD COMBINATIONS**	**1-10**
1.7	**MODEL FACTORS**	**1-10**
2	**Filling and Emptying System**	**2-1**
2.1	**REQUIREMENTS BY THE TERMS OF REFERENCE**	**2-1**
2.2	**BIBLIOGRAPHY**	**2-1**
2.3	**DIMENSIONS**	**2-2**
2.4	**CONFIGURATION AND WATER SAVING PERFORMANCE**	**2-4**
2.4.1	*GENERAL ASSUMPTIONS*	*2-4*
2.4.2	*CONFIGURATION 1*	*2-5*
2.4.3	*CONFIGURATION 2*	*2-6*
2.4.4	*FILLING / EMPTYING RATES*	*2-6*
2.5	**MAIN LEVELS**	**2-7**
2.5.1	*LEVELS AND FLUCTUATIONS IN THE LAKE GATUN*	*2-7*
2.5.2	*LEVELS AND FLUCTUATIONS IN THE PACIFIC OCEAN*	*2-8*
2.6	**SCHEDULING**	**2-8**
2.6.1	*PRESENT SCHEDULING*	*2-8*

2.6.2 FUTURE SCHEDULING 2-9
2.7 MOVEMENTS OF LOCOMOTIVES 2-10
2.8 ARRANGEMENT OF THE STRUCTURES 2-10
2.8.1 MAINTENANCE OF THE SAVING BASINS 2-10
2.8.2 TRANSVERSAL WALLS 2-10
2.8.3 ARRANGEMENT OF THE SAVING BASINS 2-10

3 LOCK OPERATING GATES 3-1
3.1 SCOPE 3-1
3.2 DESIGN CODES 3-1
3.3 IMPOSED BY THE TERMS OF REFERENCE 3-1
3.4 DIMENSIONS OF THE GATES 3-2
3.4.1 LENGTH 3-2
3.4.2 HEIGHT 3-2
3.4.3 WIDTH 3-3
3.5 NUMBER OF GATES REQUIRED BETWEEN THE LOCK CHAMBERS 3-3
3.6 MAINTENANCE 3-4
3.6.1 EXCHANGEABILITY 3-4
3.6.2 AIR CHAMBERS AND POSSIBLE NECCESARY COUNTER-WEIGHTS 3-4
3.6.3 DRY LOCK CHAMBER 3-5
3.7 MATERIAL 3-5
3.8 LIMIT STATES 3-5
3.9 LOAD CASES 3-6
3.9.1 DEAD LOADS – PERMANENT 3-6
3.9.2 WIND LOADS – VARIABLE 3-6
3.9.3 TEMPERATURE LOADS – VARIABLE 3-6
3.9.4 HYDRAULIC LOADS – VARIABLE/EXCEPTIONAL 3-7
3.9.5 MAINTENANCE LOAD CASES – VARIABLE 3-8
3.9.6 SHIP COLLISION – ACCIDENTAL 3-8
3.9.7 OBJECT BLOCKING GATE MOVEMENT / PREVENTING NORMAL UNIFORM SUPPORT – ACCIDENTAL 3-8
3.9.8 TRAFFIC LOAD – VARIABLE 3-9
3.9.9 PRESSURES WITHIN AIR CHAMBERS 3-9
3.10 CYCLIC LOADING 3-9
3.11 LOAD COMBINATIONS 3-9

4 Culvert and Conduit Valves 4-1
4.1 LOAD CASES 4-1
4.2 DESIGN STANDARDS FOR CALCULATION 4-1

4.3 MATERIAL 4-2

4.4 CONTROL VALVES 4-3

4.4.1 USE OF HYDRAULIC CYLINDERS 4-3
4.4.2 OIL SYSTEM 4-4

4.5 EASE OF MAINTENANCE 4-6

4.6 TRASHRACK RAKE AT THE CULVERT INLET 4-6

5 OPERATING MACHINERY 5-1

6 Lighting 6-1

6.1 GENERAL 6-1

6.2 STANDARDS AND REFERENCE DOCUMENTS 6-1

6.2.1 CLASS OF PROTECTION 6-2
6.2.2 AUXILIARY DEVICES FOR HIGH-PRESSURE SODIUM VAPOUR LAMPS 6-2
6.2.3 LIGHT SOURCES 6-2
6.2.4 ELECTRICAL CONNECTIONS FOR THE LIGHT FIXTURES 6-3
6.2.5 MISCELLANEOUS CHARACTERISTICS 6-4

7 OPERATING STRUCTURES 7-1

7.1 GENERAL 7-1

7.1.1 NETWORK BETWEEN LEVEL 0 AND 1. 7-2
7.1.2 NETWORK BETWEEN LEVELS 1 AND 2 AND 3 7-2
7.1.3 FIELD EQUIPMENT : 7-2
7.1.4 OPERATOR CONSOLES : 7-3
7.1.5 CONTROL/COMMAND PANEL: 7-3

7.2 STANDARDS 7-3

1 Lock Walls and Entrance Walls

1.1 SCOPE

This chapter stipulates the specific design criteria for lock walls and entrance walls.
The specific design criteria must be used in addition tot the general design criteria as developed in chapters:

- Part A - §1 General
- Part A - §3 Structural design criteria

1.2 DESIGN CODES

The design codes that will be used as a basis for the calculation and evaluation of the lock walls and entrance walls:

- ROSA 2000
- Eurocode 2 : EN 1992 – "Design of concrete structures"

As the locks are founded in a rock formation, the shape of the lock chamber wall will differ from the shape of a backfilled wall, and may depend mainly on the rock excavation slope. Reference is also made to :

- "PIANC : The Final Report of the International Commission for the Study of Locks § 1.1.10".

1.3 IMPOSED BY THE TERMS OF REFERENCE

- A vessel positioning system similar to the existing system with locomotives will be used to handle the ships into, through and out of the locks. Space shall be provided for a merry-go-around arrangement of the system. The space has neen fixed at 15m width for a double track and 7.5m for a single track.

- Locks will have to be dewatered for maintenance. During meeting M3 it was decided that the dry lock chamber condition is not a basic design criterion. This will be treated in the conceptual design of the lock walls and the gates.
- The ships are not allowed to rub against the walls. Wall protection or armor shall generally not be provided but used only at the most collision exposed or damaged areas as indicated at the existing lock walls.
- Ships and boats do not tie-up to the walls and mooring posts shall only be provided at the top of the lock walls.

1.4 LIMIT STATES

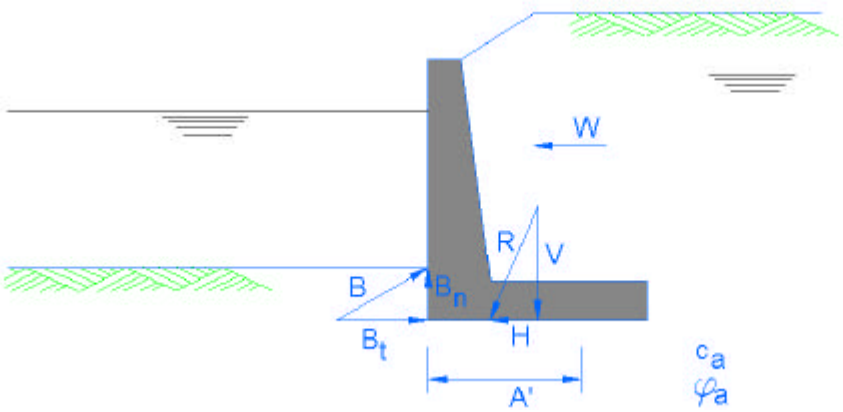
1.4.1 EXTERNAL STABILITY

1.4.1.1 Sliding along the base

The considered failure surface is located at the base of the footing. When the structure is founded on a ballast base, an additional verification has to be performed for the sliding surface between the ballast and the sub-grade.

The general formula is written as follows:

$$g_d \cdot (H + W - B_n) \pounds (V - U - B_t) \cdot tg j_a + A' \cdot c_a$$



in which:

- H and V are the horizontal and vertical component of the resulting forces of the footing on the sub-grade, taking in account the horizontal and vertical components of the earth

pressure of the backfill, as well as all external forces exerted on the wall (such as forces induced by the locomotives).

- A' is the compressed surface of the footing and has to be determined with the composed flexion model of Navier.
- c_a and φ_a are the cohesion and friction parameters at the interface on the sliding surface (short or long term according to the situation)
- The passive resistance B can be considered for an embedded foundation or when the structure is subjected to collision forces pointing to the backfill. Bn is the normal component (pointing upwards), Bt is the tangent component.
- Hydraulic upward pressure U under the footing reduces the effect of V
- W is the horizontal resulting force of hydraulic pressures

+	*ROSA quais-poids*	*§6.1.1*

1.4.1.2 Overturning – decompression of the sub-grade

The compressed surface of the ground under the foundation (A') must remain higher than a given fraction of the total surface (A) of the foundation. Calculation must be carried out with the model of the composed flexion (Navier).

$$A' / A > C\ (\%)$$

Another criterion refers to Mstab (resp. Munstab): moment of the stabilizing external forces (resp. destabilising) referred to the lower front-edge of the structure.

$$g_d \cdot M_{unstab} \pounds M_{stab}$$

This condition is given on a purely informative basis, the check being covered by that of the decompression of the sub-grade (Navier).

+	*ROSA quais poids*	*§6.1.2*
+	*ROSA quais poids*	*§6.1.4*

1.4.1.3 Bearing Capacity of Soil (Rock)

The reference pressure q_{ref} must remain lower than the ultimate bearing capacity q_u of the soil or rock, which takes in account the eccentricity of the load, its angle of inclination, the inclination of the footing and of the slope behind the structure. The criterion is written:

$$g_d \cdot q_{ref} < q_u$$

+	*ROSA quais poids*	*§6.1.3*

1.4.1.4 Flotation

Concrete hydraulic structures should be designed to have a minimum flotation safety factor.

$$g_d \cdot U \pounds PP + FR_V$$

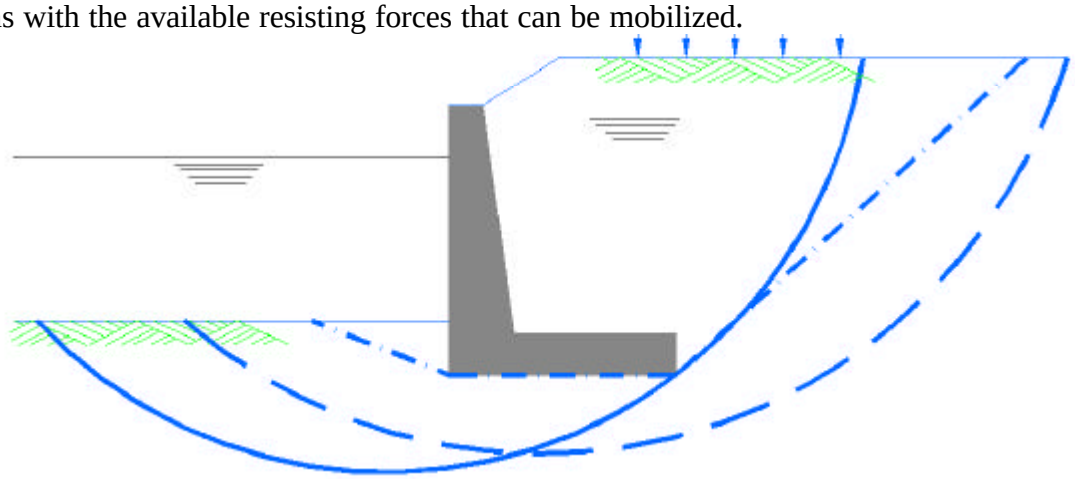
Where:

- U is the vertical resultant of the upward pressures
- PP is the dead load of the structure including the water content.
- FRv is the vertical component of the friction forces of the soil (rock) against the structure.

+	*ROSA écluses*	*§6.1.4*

1.4.2 GLOBAL STABILITY (DEEP SEATED SLIDING)

The purpose of the global stability analysis is to assess the critical sliding surface. The most unfavourable sliding surface can be different according to the exerted actions, the safety requirements adopted for the various parameters, the properties of the soil (study of short or long-term behaviour)...
One compares, for each potential sliding surface, the driving efforts resulting from the external actions with the available resisting forces that can be mobilized.



+	*ROSA Talus et Pentes*	*§4.1 §5.1*

1.4.3 HYDRAULIC STABILITY

The hydraulic stability relates to soils subjected to permanent or transitory hydraulic flows that can have destabilizing effects.

+	*ROSA Rideaux de soutènement*	*§5.1.2.*

1.4.3.1 Phenomena of “Renard”

When the level of water is different on the two sides of a structure, hydrodynamic effects are added to the hydrostatic effects because of the flow of water under the structure. This effect increases the effective stresses on the side with high water level and decreases the effective stresses on the side with low water level.
Especially in the construction phase and during maintenance, one can fear the appearance of hydraulic instabilities or phenomena of “Renard ".
One can distinguish:

- **Boiling**
  Hydraulic flows directed upwards can destabilize the soil and reduce the effective stresses. Boiling is a state that is characterized by a strong reduction in the shear strength of the soil under the effect of a gradient of flow that is opposed to the forces of gravity. When the soil is a fine sand and when the ascending vertical gradient cancels the effect of gravity, total liquefaction occurs. This effect is also called “liquid Renard”

- **Uplifting of the ground**
  The soil at the foot of a structure can be lifted up under the effect of the exerted stresses on the side of the back surface; this phenomenon is accelerated by the presence of hydraulic gradients. The phenomenon begins with a localised boiling which becomes particularly serious when the flows are accompanied by the movement of solid particles

- **Progressive erosion**
  The progressive movement of the fine particles of the soil in the direction of the flow can lead to the formation of channels or “pipes”, characterized locally by more significant voids, with an increased rate of the flow and consequently of the hydrodynamic driving forces. This phenomenon is also called “solid Renard”.

+	*ROSA Ecluses*		*§5.1.2. §*
+	*ROSA Gabions de Palplanches*		*§5.1.2.*
+	*ROSA Rideaux de soutènement*	*§7.3.1.*	*§6..1.2.*
+	*Barrages mobiles*		*§6..1.2.*

1.4.3.2 Resistance of the protection upstream and downstream

The upstream and downstream parts of the lock structures, as well as the areas surrounding the in- and outlet of the filling and emptying culverts, must be protected from the erosive effect of the water flows

+	*ROSA Barrages mobiles*	*§6..1.2..*
+	*ROSA Ecluses*	*§5.1.2.*

1.4.4 INTERNAL STABILITY

Reference is made to Part A § 3.

1.4.5 SERVICEABILITY

Limit State Load combination	Criteria
Serviceability	
Global Settlement	
quasi-permanent	< 0.05m ([1]) To be defined in function of type of doors, valves…
Differential Settlement	
quasi-permanent	The maximum value of angular distortion (settlement/length of structure) which can be tolerated without cracking of reinforced concrete retaining walls is 0.002 to 0.003 radian (Duncan and Buchignani 1976) ([2])
Deformations	
quasi-permanent	To be determined during study
Concrete cracking	
quasi-permanent	w < 0.3 mm

+	*ROSA quais poids*	*§6.4.2.*
+	*ROSA parties en béton*	

1.5 LOADS

1.5.1 DEAD LOADS - PERMANENT

Reference is made to the general design criteria (part A §3 : Structural Design Criteria)

[1] *Ontwerp van schutsluizen –Rijkswaterstaat.*
(Design of Locks – Dutch Government)
[2] *USACE EM 1110-2-2502*

1.5.2 HYDROSTATIC LOADS - VARIABLE

Reference is made to the general design criteria (part A §3 : Structural Design Criteria)

1.5.3 EARTH - PERMANENT

See Part A §4.

1.5.4 WIND - VARIABLE

Wind loads will not be included in the design of the walls (except where major portions are not backfilled). However, wind loads may need to be considered for the construction phases, prior to backfill placement. For the wind pressure, reference is made to part A §3.

Construction load conditions will not be considered for conceptual design purposes.

1.5.5 TEMPERATURE AND SHRINKAGE - VARIABLE

During future design phases, temperature and shrinkage loads shall be considered, especially with respect to expansion joint design and minimum steel reinforcement requirements. Temperature and shrinkage loads will not be addressed during conceptual design.

1.5.6 SEISMIC LOADS - ACCIDENTAL

See part A §4

1.5.7 WAVES – VARIABLE

Wave solicitation is usually not considered for the design of lock walls.
In the detailed design, the impact of the wave loads on the entrance wall at the Pacific side may need to be checked.

Wave loads will not be considered in the conceptual design, because the magnitude of the waves is rather small.

1.5.8 MAINTENANCE LOAD CASES – VARIABLE

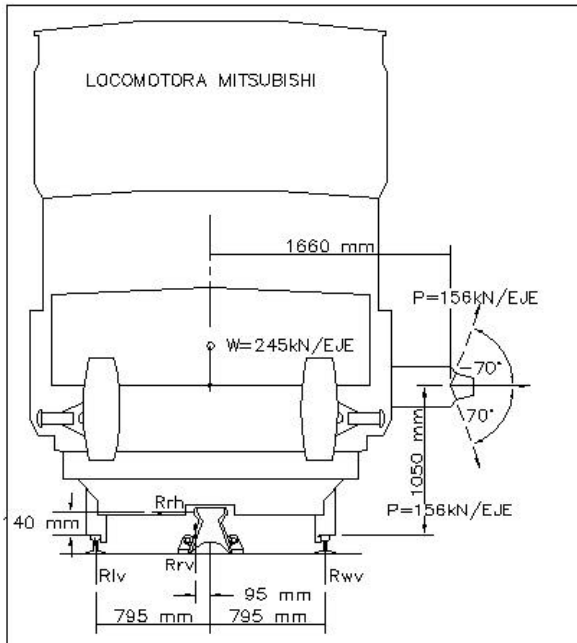
The dry lock chamber condition has been discussed during meeting M3. It will not been retained as a basic load case. The alternative procedures will be treated and explained in the conceptual design of lock walls and lock gates.

1.5.9 SHIP COLLISION - ACCIDENTAL

As stipulated in the Terms of Reference, the ships are not permitted to rub against the walls.
As the ships move parallel to the lock walls, a collision is not considered as a feasible load case. Nevertheless, as the ship impact is always directed in the opposite direction of the soil pressure acting on the back side off the wall, the global stability of the lock wall will never be affected.
At the other hand, a ship collision with the lock gate must be considered as realistic. The impact energy of a Post Panamax vessel is proportional with the square of its approach velocity, and it is impossible to design a lock gate to resist such a load case. The lock gate will either be subjected to local deformation of the steel structure, which will have to be repaired or partly replaced, or will be completely damaged, in which case it will have to be floated off and replaced. In this case the second lock gate will still be operational.

1.5.10 LOCOMOTIVES

A locomotive system similar to the existing system will be used to handle the ships into, through and out of the locks. Space shall be provided for a merry-go-around system.
The disposition of the locomotives at the existing locks and the forces exerted on the lock walls are given in the table below.



ANGULO	Ph	Pv	Rrh	Rrv	Rl	Rw
−70.0	53	−146	53	167	265	0
−65.0	66	−141	66	132	236	0
−60.0	78	−135	78	94	204	0
−55.0	89	−127	89	54	171	0
−50.0	100	−119	100	11	136	0
−45.0	110	−110	110	0	119	15
−40.0	119	−100	119	0	109	36
−35.0	127	−89	127	0	98	57
−30.0	135	−76	135	0	88	79
−25.0	141	−66	141	0	78	101
−20.0	146	−53	146	0	68	124
−15.0	150	−40	150	0	58	146
−10.0	153	−27	153	0	49	168
−5.0	155	−13	155	0	41	190
0.0	156	0	156	0	33	211
5.0	155	13	155	0	26	232
10.0	153	27	153	0	20	250
15.0	150	40	150	0	14	271
20.0	146	53	146	0	10	288
25.0	141	66	141	0	6	305
30.0	135	76	135	0	3	320
35.0	127	89	127	0	1	333
40.0	119	100	119	1	0	346
45.0	110	110	110	1	0	356
50.0	100	119	100	0	0	364
55.0	89	127	89	0	2	370
60.0	78	135	78	0	4	375
65.0	66	141	66	0	7	378
70.0	53	146	53	0	12	379

Ph ES EL COMPONENTE HORIZONTAL DE P
Pv ES EL COMPONENTE VERTICAL DE P
EL PESO TOTAL DE LA LOCOMOTORAE ES DE 495kN
LA CAPACIDAD TOTAL DE REMOLQUE DE LA LOCOMOTORA ES DE 315kN
LAS CARGAS POR EJE SON LA MITAD DE LA CARGA TOTAL
SE ASUME QUE P VARIA ENTRE 70° Y −70°
SE ASUME QUE LA CREMALLERA RESISTE TODA LA CARGA LATERAL
Y CUALQUIERA CARGA VERTICAL QUE SE REQUIERA PARA EQUILIBRIO

It will be assumed that the forces are proportional to the DWT-size and the overall dimensions of the vessel.

The maximum allowable vessel dimensions at the existing locks are:

- Width: 32.30m
- Length: 289.56m
- Draft: 12.04m

The Dead Weight Tonnage of these vessels is approximately:

- Container Ships: 50000DWT
- Bulk Carrier: 45000DWT

The loads acting on the lock walls due to the new locomotives will be assessed in the conceptual study of the lock walls.

1.5.11 LIVE LOAD (OTHER THAN LOCOMOTIVES) - VARIABLE

Behind the walls a uniform live load of 10kN/m² will be applied.

1.6 LOAD COMBINATIONS

In accordance to the General Design Criteria Part A §3, the Limit States are classified as follows:

Limit State	Category	Load Combination
External stability		
Sliding	ULS	fundamental
Overturning – decompression of the subgrade	ULS	fundamental / accidental
	SLS	rare / quasi-permanent
Bearing Capacity of Soil	ULS	fundamental / accidental
	SLS	rare
Flotation	ULS	fundamental / accidental
Global Stability		
Overall-sliding	ULS	fundamental / accidental
Hydraulic Stability		
Phenomenons of "Renard"	ULS	fundamental / accidental
Resistance of the rock fill protections	ULS	fundamental / accidental
	SLS	rare / frequent
Strength (internal stability)		
Strength of metal structures	*Eurocode 3*	
Strength of reinforced concrete	*Eurocode 2*	
Serviceability		
Global Settlement	SLS	quasi-permanent
Differential Settlement	SLS	quasi-permanent
Deformation		

+ *ROSA 2000 écluses* *§5.2*

1.7 MODEL FACTORS

As stipulated in part A §3, the semi-probabilistic approach (according to the ROSA-standard) takes resort to 4 types of partial safety factors.

Uncertainty of loads	γ_f
Uncertainty of material properties Uncertainty of soil-structure interaction parameters	γ_M
Uncertainty of the resistance	γ_R
Every uncertainty in connection to the calculation model (actions, load effects, resistance)	γ_d

For the factors γ_f, γ_M, γ_R reference is made to ROSA - valeurs representatives des propriétes de base des materiaux – valeurs representatives des resistances- situations et combinaisons

d'actions. (representative values of the basic properties of the materials – representative values of the resistance – situations and combinations of actions).

The factor γ_d (according to ROSA-écluses (locks)) is included in the tables below :

Model factors for sustained and transient situations

Limit State **Load combination**	**Model**	**Value for g_d or $g_{d,\,serv}$**
	Stability	
Sliding at the base		
fundamental		1,10
Overturning – decompression of the subgrade		
fundamental		10 %
rare	Navier	75 %
quasi-permanent		90 %
Bearing Capacity of Soil		
fundamental	the design value of the ultimate bearing capacity is determined by the application of a coefficient γ_R (laboratory tests or methods of investigation on site)	1,40
	the design value of the ultimate bearing capacity is determined by multiplication of the basic soil properties with the coefficients γ_m (laboratory tests only)	1,20
rare		1,00
Flotation		
fundamental		1,25
	Global Stability	
Overall – sliding		
fundamental	Method of slices general method rupture analysis	1,25 (normal safety level) 1,10 (low safety level)
	Hydraulic Stability	
boiling		
fundamental		1,00
Uplifting of the ground		
fundamental		1,25
Progressive erosion		
fundamental	Lane	1,00
Rock Filling protections upstream and downstream		
fundamental		1,00
rare		1,25
frequent		1,50
	Strength (Internal stability)	
Steel Structures	Eurocode 3	
Concrete	Eurocode 2	
	Serviceability	

Global Settlement		
quasi-permanent		-
Differential Settlement		
quasi-permanent		-

Model factors for Accidental Situations

Limit State **Load combination**	**Model**	**Value for $g_{d,acc}$**
Stability		
Sliding		1,00
Overturning – decompression of the sub-grade		10 %
Bearing Capacity of Soil		1,00
Flotation		1,1
Global Stability		
Overall-sliding		1,00 (general) 1,10 (seismic)
Hydraulic Stability		
Phenomena of "Renard" (boiling, uplifting of the ground, progressive erosion)		1,00
Rock fill protections upstream and downstream		1,00
Strength		
Steel Structures	*Eurocode 3*	
Concrete	*Eurocode 2*	

2 Filling and Emptying System

2.1 REQUIREMENTS BY THE TERMS OF REFERENCE

- It is required to keep the new filling and emptying design times within an 8-10 minutes range without any water savings basins for one lock chamber of a triple lift lock configuration and be suitably proportioned for either the entire triple lift lock configuration, or the single lift configuration.
- The three-lift lock system has to be designed to save approximately 60% of the lock water. The single lift lock system has to be designed to save approximately 75% of the lock water.
- The upper and lower levels of the lock configurations shall also have provisions for an intake and outlet for recycling of spilled lockage water in addition to the upstream lock water intake for the locks from Gatun Lake and downstream discharge of water to the sea.

2.2 BIBLIOGRAPHY

For the conceptual design of the filling and emptying systems, the following models and documents will be used in addition to those mentioned in the General Design Criteria

Physical models of locks studied in the CNR laboratory:

- Lock of Saint-Gilles and Péage de Roussillon - Rhône river - 1971
- Lock of Niffer -Saône-Rhin project - 1979
- Lock of Lappérière -Saône-Rhin project - 1994

Literature

- Hydraulique générale et appliquée : M. CARLIER
- Quelques problèmes posés par le remplissage des écluses de navigation by P ALMERAS (la Houille Blanche n°5 – 1948)
- Investigations of navigation lock on Yougoslavian side of Djerdap hydropower and navigation system by K DJONIN (A.I.P.C.N. – 24th congress in Leningrad 1977)

- Mixing in Inland and coastal waters by FISCHER
- Influence du mouillage dans le sas d'une écluse et de la présence de bassins d'epargne sur la durée d'éclusage by J MICHEL (Session SHF de novembre1980)
- Ondes et marnages dans les biefs dus au remplissage et à la vidange des écluses by J MICHEL (Session SHF de novembre 1980)
- Evolution dans la conception des écluses, notamment pour leur circuit hydraulique by A. PETITJEAN (Session SHF de novembre 1980)
- Locks on the wide-gauge Rhône Waterway by R. PINATEL (la Houille Blanche – 1981)
- The filling of high lift lock and its problems by Daniel Pinto Da Silva (A.I.P.C.N. – Bulletin n°52 - 1986)
- Hydraulic design experiences and practice of Zong Muwei navigation locks of China by Ding Xingrui (A.I.P.C.N. – Bulletin n°69 - 1986)
- Emptying / filling system of Donzere Mondragon lock on Rhône river – la Houille Blanche (numéro spécial)
- Caractéristiques du fonctionnement hydraulique de l'écluse de Bollène by E. PARISET (Neyrpic)
- Conception et entretien des aménagements fluviaux – Les écluses (Ecole Nationale des Ponts et Chaussées)
- Hydraulique générale by Armando LANCASTRE
- Internal Flow Systems by D.S. Miller
- Traité de génie civil - Hydraulique fluviale by Walter H. GRAF and M.S. Altinakar (Presses polytechniques et universitaires Romandes)
- Mécanique des fluides appliquée by VIOLLET CHABARD ESPOSITO and LAURENCE (Presses de l'école nationale des ponts et chaussée)
- La cavitation – Mécanismes physiques et industriels (collection Grenoble Science)

2.3 DIMENSIONS

Locks

- The general dimensions of the locks are given in part A – General design Criteria.

In order to determine the bottom levels of the locks and basins, and to calculate the fluctuations of the water levels in the lock chambers, new software has been developed for this purpose.
The Consultant proposes a distance between the gates of 10 m in case of rolling gates or at least 40 m in case of mitre gates. These values will be further optimised during the conceptional design.

In addition, the following assumptions are made :

- Gate recesses

A gate recess of 70 m x 20 m in case of rolling gates and 61 m x 4 m in case of mitre gates (2 gates per side of the chamber) is proposed

- Water inside the gates

  According to the European standards, 75 % of the volume of the gates is filled with water. This volume has to be included to calculate the water consumption of a lockage.

- General operating way ; both gates on each side of each lock (order of opening and closing) are always operated in the same way and for each direction (up or down-lockage) in order to have the same areas of the chambers and a minimum consumption of water

- Distance of nearly 8.20 m between the 2 nd gate at the upper silld of the chamber and the front wall (case of rolling gates) and 3.20 m (case of mitre gates)

Consequently, depending of the use of the gates, the length of each chambers will vary, from 435.00 m and up to 500.00 m (in the case of rolling gates) or from 496.00 and up to 577.00 m (in the case of mitre gates)

These maximum lengths are used in case of a failure of a gate or when a collision occurred (a rather exceptional event), or for maintenance reasons, which will be kept at a minimum (design criteria).

It can be concluded that average lengths of 467.50 m or 536.50 m depending on the types of gates can be adopted for the design of the saving basins and the chambers. The difference is only +/- 7 to 8 % and accordingly largely sufficient for a conceptual design and without any influence for the design of the levels of the structures.

In fact, to calculate the bottom levels of the structures and water level fluctuations, the length of the chambers, the recesses, the volumes of water in the gates are not essential because the area of the saving basins is adjusted to the entire area of the chamber. This will only affect the total volume of water consumption, while the percentage of water saving will be constant..

The case of the single lift lock configuration is by evidence easier to design : the different levels in the lock chamber depend only on the water levels in the Pacific Ocean and Gatun lake

2.4 CONFIGURATION AND WATER SAVING PERFORMANCE

2.4.1 GENERAL ASSUMPTIONS

The first calculations show that water levels in the saving basins and in the chambers are very fluctuating, due to the high range of variations of the levels, the upstream level in the lake Gatun and especially the tidal variation in the Pacific Ocean :

- the extreme head between the highest water level in the lake Gatun and the lowest in the Pacific Ocean is H_{max} = 26.67 + 3.44 = 30.11 m,

- the lowest head is H_{min} = 23.90 – 3.60 = 20.30 m

It appears immediately without any calculations that this great difference (near 10 m) would lead to fluctuating levels in the basins and the chambers. Therefore basins with high walls and locks with high chambers (i.e. uneconomic structures) would have to be designed. This has already been experienced by the Consultant during the design of another project in France (a two step lock configuration)

With these considerations, the following assumptions are made :

- The level of the Pacific Ocean fluctuates similarly during the dry and the wet seasons and for a whole day (four tides), so the daily mean consumption of water is not affected, the daily mean consumption varies only with the level of the lake Gatun.

NB : whether the highest and lowest tides correspond with the dry or the wet season, mean sea level is the same all year long.

- During the wet season, the level of the lake Gatun is high, allowing higher water consumption, but at the same period, water saving is not needed in general..

  During the dry season, even with the use of the basins, the level of the lake decreases, leading to less consumptions of water.

  To conclude in a preliminary approach, the design of the chambers and basins can be done with levels in the lake Gatun of +26.67 and +23.90m PLDThe first value will give the upper levels of the structures and the maximum consumptions of water, the second value will set the lowest levels of the structures

- The value of 60 % of saving water does not need to be reached for each single lockage, but has to be calculated as an average on a certain number of lockages, for example the number of lockages of one day. During this period, the value of the water level in the Pacific Ocean fluctuates, with four tides each day, two high tides and two low tides. The mean sea level in the Pacific Ocean at + 0.3048 m can be adopted.

- After calculating the levels according to these values, every combination of levels has to be checked by computations to be sure that these assumptions lead to acceptable lockages and arrangements of the structures.

Note : At this stage, we do not consider water losses through the structures.

2.4.2 CONFIGURATION 1

In the first configuration the total head has to be broken down by means of a 3 lift lock system, the TOR requiring for savings of at least 60 % of the water

This requirement can be summarized as follows :

- Scheduling

  For the three lift lock system, the best way to save the maximum of water is the scheduling of a "convoy"(see 2.6.1).

- Operating levels :

  As defined before, the operating levels of the lake Gatun are respectively +26.67 and +23.90m PLDand the Pacific Ocean level varies as following :

  - -2.32 to +2.40m PLD to set the levels of the chamber bottoms
  - - 3.44 to +3.60m PLD to set the bottom levels of the water saving basins

  According to the assumptions made in chapter 2.3 and the sizes of the chambers, the consumptions and the savings required are estimated at

- three lift configuration (case of rolling gates) :

basic mean consumption : (467.50*61+2*70*20+2*15*61*0.75)*1/3*(26.14-0.305) = 281,500 m3
basic mean saving water : 0.60 * 281 500 = 168,900 m3
basic water consumption : 273 270 – 163 960 = 112,600 m3

NB 1 : a loss of water occurs during the change of lockage direction
NB 2 : the same calculation can be made the same way in case of mitre gates

2.4.3 CONFIGURATION 2

In the second configuration the total head has to be stepped by means of a single lock system, the TOR requiring for water savings of at least 75 %.

This requirement can be summarized as follows :

- Scheduling

  For the single lift configuration, there is only one scheduling possible : alternate traffic

- Operating levels :

  As defined before, the operating levels of the lake Gatun are respectively +26.67 and +23.90m PLD and the Pacific Ocean level varies as following :

  - -2.32 to +2.40m PLD (leading to -2.32m PLD for the bottom level of the lowest chamber)
  - -3.44 to +3.60m PLD to set the bottom levels of the water saving basins

  These values are retained for the saving analysis. According to the sizes of the chambers, the consumptions and the savings required are (case of rolling gates) :

  basic mean consumption : = (467.50*61+2*70*20+2*15*61*0.75)*(26.14-0.305) = 844,500 m3
  basic mean saving water : 0.75 * 844 500 = 633,375 m3
  basic water consumption : 844 500 – 633 375 = 211,125 m3

NB 1 : No loss of water occurs at midday (due to alternate lockage system)

NB 2 : The same calculation can be made in case of mitre gates

2.4.4 FILLING / EMPTYING RATES

In the actual existing locks situation the filling and emptying times of the lock chambers are 8 minutes without using any saving basin. It corresponds to a rate of 1.25 m / minute rise or fall for the ships. These values have been confirmed during the kick off meeting in February 2002.

The filling rate of the basins can be higher because :

- there is no water turbulence limitation in the basin, as there is in a chamber with a ship

- emptying the chamber causes less turbulence compared to the filling.

The Consultant will calculate and check the rates of the following stages :

- Filling from the upstream reservoir (from lake Gatun)
- Emptying to the lower reservoir (to the sea)
- Filling (= emptying) lock to lock
- Lock to saving basins
- Saving basins to lock

2.5 MAIN LEVELS

2.5.1 LEVELS AND FLUCTUATIONS IN THE LAKE GATUN

Lake Gatun area : ***418.25 km 2***

With the previous values, a maximum of 15 lockages a day (8 in one way, 7 in the other way), a total area of the lake of 418.25 km², of which 219 km² being affected to the Pacific side, the daily fluctuations of the water levels are :

- three lift configuration : dz = $\frac{112\ 600*15}{219*10^6}*10^{+2} = 0.77\text{ cm}$

- single lift configuration : dz = $\frac{211\ 125*14}{219*10^6}*10^{+2} = 1.35\text{ cm}$

Even with the loss of water due to the turn around of ships in the middle of the day (1st configuration), the water level fluctuations of the lake Gatun are very small.

Consequently a constant value for the lake Gatun level during a whole day (or several consecutive days) can be adopted. The fluctuations of head are only due to the tidal fluctuations of the Pacific Ocean level.

The design of the bottom levels of the locks and of the saving basins has to be made for the two extreme values of the Gatun lake level : +26.67 m and +23.90 m PLD.

NB : Neither the inflows in the lake Gatun nor on the opposite the evaporation are taken into account in the above rough calculations

2.5.2 LEVELS AND FLUCTUATIONS IN THE PACIFIC OCEAN

The level in the Pacific Ocean fluctuates due to the tides. ACP provided data to the Consultant. The main data used by the Consultant are the fluctuations of the level from January the first 0h00 to February the first 23h00.

Other data are also available and will be checked to be sure that all configurations of the tides will be taken into account. In a first approach, a whole year seems to be sufficient.

It appears immediately that the Pacific level follows a sinusoïdal function.

A first approach has been made to correlate a sinusoïdal function to the real level during 96 hours (4 days) beginning on 1.January 1991. Then, a global coefficient has been applied to reach the extreme values of the level, i.e. –3.44 m and + 3.60 m PLD

The general equation of the function is the following one :

$$Z\ Pacific = 3.52 * \sin\left(\frac{2p}{12.47} * (t)\right) + Z\ mean$$

With Z mean = +0.08

According to this equation, when choosing the level of the Pacific Ocean at t_0 for the beginning of the cycles, the determination of the level of the Pacific at any time is possible, and especially before the equalisation of the levels in the lower chamber and the Pacific Ocean.

To undertake all the combinations, another coefficient will be chosen for the tides of lower range (from 09/01/91 to 13/01/91 for example)

In this way the software can run with any level of the Pacific Ocean and whatever the beginning of the simulation may be.

2.6 SCHEDULING

2.6.1 PRESENT SCHEDULING

At the present time, the operating system of the locks named ‘’convoy’’ is used, i.e. 12 hours in one way, 12 hours in the other way. This is mainly due to the fact that this system allows to maximise the number of ship passages (this is possible with multi-lift locks), and secondly because it is impossible for the ships to cross each other in the ‘’Gaillard Cut’’.

In the future, the lack of water will be an increasing problem, so the system ''convoy'' will continue to be used for each lane.

Since March 2002, two way traffic is possible in the ''Gaillard Cut'' widened for the Panamax ships, and so for from the entire canal from the Pacific side to the Atlantic side. The system ''convoy'' can then be used two ways using simultaneously the two lanes, one up and one down.

2.6.2 FUTURE SCHEDULING

In the future, the same system (''convoy'') is going to be adopted by ACP for the multi-lift locks, with the following modifications or additions :

- The Post Panamax ships will sail between ''Cucarracha junction'' and Gamboa from 6 am to 6 pm (day light) for the following reasons :
  - Probably the number of Post Panamax should be less (at least at the beginning) than the Panamax and the Panamax +, consequently: there is no reason for night shift lockages.
  - The passage in the "Gaillard Cut'' is supposed to be more dangerous during the night

- Both traffic systems (in the existing and future locks) must coexist, with an increasing traffic, so the Post Panamax traffic has to insert itself in the existing one.
  - The future operating system could then be the following :
    - Panamax the whole day in two ways, using the present locks (1st and 2nd lanes)
    - Post Panamax sailing in ''convoys'' 6hours / 6 hours, with a turn around at midday, with a Panamax + first in the early morning and probably another last one Panamax + in the evening, using the 3rd lane.
    - Crossing of two Post Panamax will remain impossible in the channel, the only possible crossing will be between a Post Panamax and a Panamax +
  - A minimum of 7 lockages a day is expected in the third lane as explained during the kick off meeting (at the beginning), and a maximum of 14 or 15 both ways (14.7 in the ACP presentation). Obviously in the distant future, the third lane will be probably opened all day long

The alternate system will be obviously used in case of a single lift lock.

2.7 MOVEMENTS OF LOCOMOTIVES

The movements and displacement times of the locomotives are not examined in this conceptual design study..

2.8 ARRANGEMENT OF THE STRUCTURES

2.8.1 MAINTENANCE OF THE SAVING BASINS

For the maintenance of the saving basins, at least 2 conduits per basin will be foreseen. An access for the operating crews and equipment (lorries, trucks, cranes) is also necessary on each side of the basins for maintenance.

Concrete thin cantilever walls will separate longitudinally the basins.

2.8.2 TRANSVERSAL WALLS

Some transversal walls are needed in the basins to avoid the flushing of water. These walls must not be continuous, in order not to share the basins. So, in case of a failure of one of the two valves, operating is always possible.

2.8.3 ARRANGEMENT OF THE SAVING BASINS

A location for the fourth lane of locks must be provided close to the third lane, as such saving basins on both sides of the third lane are not retained in case of the 3 lift lock system
Nevertheless, stacked basins on both sides are possibly expected for the single lift lock system because of overlapping caused by the high range of the tides.

3 LOCK OPERATING GATES

3.1 SCOPE

This chapter stipulates the specific design criteria for the gates.
The specific design criteria must be used in addition to the general design criteria as stipulated in the chapters:

- Part A - §1 General
- Part A - §3 Structural Design Criteria

3.2 DESIGN CODES

The design codes that will be used during calculation and evaluation of the lock gates:

- ROSA 2000
- Eurocode 1 : "Basis of design and actions on structures"
- Eurocode 3 : "Design of steel structures"
- PIANC – Final report of the International Commission for the study of Locks. (Bull. 55, 1986)

3.3 IMPOSED BY THE TERMS OF REFERENCE

- The possibility to use some of the gates to serve as the closure for the dry lock chamber maintenance will be evaluated.
- The gates shall be designed for the full range of operating loads to include normal operation, maintenance loads and temporary loads from overfilling of the lock chambers.
- In case of rolling gates, a slot bulkhead shall be provided and mechanisms provided to support the gates for maintenance in the storage slot.

3.4 DIMENSIONS OF THE GATES

3.4.1 LENGTH

The length of the gates equals the width of the locks plus the necessary allowances for the vertical bearings (ca. 2 m). The requested width of the lock, mentioned in the terms of reference, is set at 61 m, measured face to face of the lock walls. The width of the lock is based on the width of the design ship, being 54.9 m, increased with 10 %.

3.4.2 HEIGHT

Water levels

See part A §1.4

Single-lift configuration

For the single-lift solution, the net height of the lock gates can be determined:

- Pacific entrance gate

Level of the sill – Pacific Entrance	-2.32 – 18.3 = -20.62 m
Maximum water level Gatun Lake	26.7 m
Net height	20.62 + 26.67 = **47.29 m**

- Gatun lake gate

Level of the sill – Gatun Lake	+23.92 – 18.3 = +5.62 m
Maximum water level Gatun Lake	26.67 m
Net height	26.67 – 5.62 = **21.05 m**

This net height has to be increased with the necessary clearance (freeboard) and the necessary height required for the seals near the sill. Both values will be determined during conceptual design.

Triple-lift configuration

For the triple lift solution, the water levels mentioned above will be again the basis of the design. The net height of the gate, side Gatun Lake, will be the same as determined for the single lift solution.

How the difference in water levels between Pacific entrance and Gatun Lake are to be divided along the triple-lift solution will be examined during the conceptual design. The conceptual design of the filling and emptying system, and more specific the system of water saving basins, will result in the optimum floor and sill levels.

3.4.3 WIDTH

The width of the lock gates will be determined such that:

The overall deformations of the structure and material tensions (stresses) in the skin plating under different loading conditions are expected to remain "moderate", avoiding excessive material consumption as a result of the structural design work;
The lock gate has sufficient stability (or: limited instability) against rolling over in floating conditions (by means of air chambers) during maneuvering operations for mounting and maintenance;
The dimensions of the concrete structure around the lock gates should remain reasonable. In particular the length of the lock should exceed the "useful" length of the lock chamber (426.8 m/1400 ft) by a minimum.

3.5 NUMBER OF GATES REQUIRED BETWEEN THE LOCK CHAMBERS

Double sets of gates at the upstream and downstream locations of each lock chamber are required.

Spacing between the gates, set in the terms of reference at 30.5 m, needs to be optimized, depending on the selected type of gates.

If rolling gates will be recommended to use, this spacing could be reduced. This reduction is possible due to the lateral movement of the operating gates.

If miter gates will be recommended to use, this spacing has to be increased. The lock having a minimum width of 61 m, will require miter gates wider than 30.5 m due to the inclined position of the gates to constitute a 3-hinges arc. The distance between the gates will therefore also exceed this value.

3.6 MAINTENANCE

3.6.1 EXCHANGEABILITY

When major maintenance of the lock gates is necessary, floating of the gates must be possible. This way the gates can be floated off to a dry dock and can be replaced.
A procedure must be determined for the removal of a lock gate. Provisions have to be made so that the removal of a lock gate can be done without major preparations and work.

Exchangeability for each pair of lock gates will be made possible. Further advice will be given for the exchangeability of the different pairs of lock gates in the triple-lift configuration.

3.6.2 AIR CHAMBERS AND POSSIBLE NECCESARY COUNTER-WEIGHTS

In order to facilitate transport operations, reduce wearing of moving parts, and to keep power of the gearing installation to a minimum for opening and closing operations, air chambers will be built in the lock gates to relieve some of the weight of the gates.

These air chambers will be divided in watertight compartments by longitudinal and transversal inner walls, such that the operability of the gates is guaranteed over a large range of accidental ship collision events. Moreover the longitudinal walls bring additional stability against rolling over in floating conditions by limiting the width of the ballast water free surface in the compartments.
Furthermore, the air chambers will be positioned so that during operations at high water levels, a given minimum pressure of approximately 10 % of the total weight in the supports or the linkages should remain. The precise percentage will be examined in the conceptual design.

If necessary, counter-weights will be determined in size and position to adjust (lower) the center of gravity, in order to guarantee the rolling stability with relatively low positioned air chambers. The additional weight in its turn has to be compensated in additional volume of the air chambers, disadvantageously lowering their average position, which can be accounted for a priori.

The necessary volume and the positioning of the air chambers is determined such that the lock gate(including the eventually permanent ballast or counter-weights):

In floating conditions has sufficient stability (or: limited instability) against rolling over;
Can still be brought afloat (lifted) with a safety margin for possible additional weight (e.g.: sediments, equipment, compartments filled with water after a ship ramming event, etc)..

In extreme conditions, possible instability during floating may be inevitable. In that case, the recommendations for the necessary cranes and/or floating equipment during maneuvering operations for mounting and maintenance will be worked out.

3.6.3 DRY LOCK CHAMBER

The option of emptying the lock chambers will be evaluated. This is required in the T.O.R. for necessary maintenance work to the gate. This leads to an extreme load case to be applied to the gates.

Conceptual design will search for possible alternatives for the maintenance work. If all major repairs can be done, without a dry lock chamber, the steel structure can be kept much lighter. In case of rolling gates we recommend that a dry lock chamber is not necessary. This proposal was accepted by ACP during meeting M3.

In case of rolling gates the lock gate recesses can be emptied in order to perform maintenance and repair of the gates in dry conditions. See part B §3.3 – sub 3.

3.7 MATERIAL

See part A part §3.2.3

3.8 LIMIT STATES

Limit states to be considered :

- **Loss of static equilibrium**
  - Check if the structure is not moved from the normal supports.
  - Hydrostatic instability (gates with air chambers)

- **Internal instability**

  Structural resistance:
  - Resistance of separate elements
  - Global and local instabilities (buckling)
  - Local failure of the structure

Fatigue:
- Each detail exposed to a cyclic load, has to be verified for fatigue failure

- **Internal stability of connections**
- **Serviceability**

  Verification of deformations, vibrations, displacements

3.9 LOAD CASES

Following load cases will be considered for the design of the lock gates.

3.9.1 DEAD LOADS – PERMANENT

- Self weight of the structure
  See part A §3.5.1

- Accumulation of material
  Material will accumulate onto the lock gates air chambers. A supplementary dead load of 2.5 kN/m² (m² determined in plan view) will be taken in account.

3.9.2 WIND LOADS – VARIABLE

Especially during construction phase, the gate will be exposed to wind pressure.

Reference is made to part A §3 for the applicable wind pressures.

3.9.3 TEMPERATURE LOADS – VARIABLE

The reference temperature during erection is assumed at t= 25°C.

A global thermal expansion as well as a global shrinkage will occur by varying temperature.
The global temperature variation is assumed at:

∗ ΔT = +25°C (t = +50 °C)

∗ ΔT = -15°C (t = +10 °C)

For large steel structures, a local variation in temperature may occur. A difference of 25°C between different parts of the structure will be taken in account.

3.9.4 HYDRAULIC LOADS – VARIABLE/EXCEPTIONAL

- Hydrostatic loads (variable)
  Based on the occurring water retaining levels on both sides of the gates, different load cases can be determined. This will be done for the single-lift and triple-lift structure. The precise load case can only be determined when the levels of the different sills are known. These will be determined during the study in accordance with the filling and emptying system and the water-saving basins.
  To cover possible variations due to waves induced by ship movement, a supplement on the theoretical water level will be taken in account. A realistic value for this additional height will be deduced from the study report provided by ACP concerning the wave propagation due to ship movement.

+ *Design criteria Part A §1.2.2. (19) Data on wave propagation due to ship movement: Pressure Test Miraflores Locks (Pressure sensors in the chambers during the passage of boats)*

- overfilling of the locks (variable)
  Due to inertia in the filling system, the water level in the lock chamber will exceed the expected water level. The impact of this effect will be determined, once design of the filling system including the water saving basins is known.

- waves (variable)
  The wave loads are mentioned in Part A- §3 Structural design criteria

- dry maintenance conditions
  Depending on the evaluation of the necessity of a dry lock chamber as mentioned in part B §3.6.3, the option of dry chamber lock maintenance will be taken into consideration. It was decided to do this for the outer lock doors.

- emergency closure (accidental)
  Although an emergency closure is not integrated in the specific scope of work, the possible use of the gate selected for emergency closures under flowing water conditions will be evaluated. Considerations will be formulated for possible solutions enabling to operate the gates under these extreme loading conditions
  Due to the specific situation of the locks, connecting Pacific Ocean and Gatun Lake, an emergency closure can be considered as extremely rare. During the meetings with ACP it was decided that the risk will be minimized by closing two lock gates at the same time when a ship enters the lock chamber.

- gate movement with difference in water level
  To reduce the operating time of the gates, the possibility of gate movement with a slight

difference in water level will be examined. This will be determined by gate type, mechanism, seals…

3.9.5 MAINTENANCE LOAD CASES – VARIABLE

According to the type of gate selected, the maintenance procedures will lead to specific situations. These temporary maintenance situations will be translated into load cases.

For every type of gate it is necessary that the gate can be positioned vertically outside of the lock onto build-in supports, taken into account the self-weight of the structure. Local failure of these supports must be checked and excluded.

3.9.6 SHIP COLLISION – ACCIDENTAL

Based on the known problems with ship collisions in the major locks in Europe, a realistic accidental loading will be taken into account. Although local failure is inevitable, the lay out concept of the structural framework in combination with the lay out concept of the air chambers is determined to safeguard the "global" structure and functioning of the lock gates in case of an accidental ship ramming.

3.9.7 OBJECT BLOCKING GATE MOVEMENT / PREVENTING NORMAL UNIFORM SUPPORT – ACCIDENTAL

Once the type of gate and the type of movement of this gate is determined, accidental loading caused by an object blocking the gate will be examined.

Following situations may be considered:

- An object is situated between gate and gate stop. This causes a single point support where the object is located. This situation will be checked with all normal water levels, but not in the case of a dry lock chamber.
- An object is blocking the movement of the gate. Because of the blocking element the operating system will increase the force onto the gate. Checking will be done with an increase of 25 % of the normal working force onto the gate.

In general the gate structure will be designed in a way that possible damage occurs only locally without affecting the global functioning of the gates.

3.9.8 TRAFFIC LOAD – VARIABLE

No traffic load on the gates will be considered, except at one location (Gatun Lake side). Appropriate concentrated loads due to maintenance equipment will be considered, according to the needs of ACP.

3.9.9 PRESSURES WITHIN AIR CHAMBERS

Ballasting water in the air chambers will be removed through pumping. No other internal pressure in the air chamber is considered as a loading case.

3.10 CYCLIC LOADING

For the lock structure, the expected service life time is set at 100 years.

The loading of the lock gates is cyclic, based on its purpose. For the design of the lock gates, the amount of lock movements will be set at 25/day. This gives a total amount of 25 x 365 x 100 = 912500 loading cycles.

This amount will be the basis of a fatigue design of the joints and links.

3.11 LOAD COMBINATIONS

For the combinations that will be used, we refer to the general design criteria and Eurocode 1.

4 Culvert and Conduit Valves

4.1 LOAD CASES

Load cases shall be considered on the base of criteria given in DIN 19704. Attention shall be paid to all possible conditions of operation in order to ensure reliability and safety of each item.

4.2 DESIGN STANDARDS FOR CALCULATION

The gate shall be designed on best international practice for the valves, using DIN 19704 "Hydraulic steel structures - Criteria for Design and Calculation".

DIN 19704 : Hydraulic Steel Structures

Part 1 : Criteria for design and calculation
Part 2 : Design and manufacturing
Part 3 : Electrical equipment

The Standard 19704 as well as 19705 are used by a number of European countries . The valves must be designed for the loads listed in the Standard, appropriate to the individual site. These are mainly :

- Dead weight,
- Hydrodynamic pressures,
- Wind forces,
- Impact,
- Friction,
- Earthquake acceleration
- Closing and opening pressures,
- Temperature effects.

Calculations are by the method of allowable stresses and pressures and cover three distinct loading conditions :

- standard operating case,
- special operating case,
- exceptional conditions.

Allowable stresses for operating devices are limited to the material yield point, these are :

- 40% in the standard case,
- 50% in the special case,
- 80% in the exceptional case.

1. Normal Loads

Normal loads are those appearing in DIN 19704 for Normal Operation case NB.

2. Special Loads

Special loads are those appearing in DIN 19704 for special operation case BB combined with the particular earthquake by taking into account Operation Basis Earthquake (OBE - still to be defined) for horizontal and vertical acceleration. Both seismic conditions are concomitants.

The overpressure is calculated by Westergaard formula

$$p = 0{,}875\ (H.Y)^{0,5}\ W\ a$$

where

H : head of water at the toe of the civil works holding the gate
Y : head of water on the part of the valve examined
W : specific weight of water
a : horizontal seismic acceleration factor

3. Exceptional loads

Exceptional loads are those appearing in DIN 19704 for exceptional operation case AL combined with exceptional earthquake by taking into account: Maximum Design Earthquake (MDE – still to be defined) f or horizontal and vertical acceleration. Both seismic conditions are concomitants.

The overpressure is also calculated by Westergaard formula…

Both seismic conditions are concomitants.

The overpressures are combined with the maximal static head relative to the maximum water elevation.

4.3 MATERIAL

Steel P275N according to EN 10028

- min.yield point for a thickness <16mm : 275N/mm²

- min.yield point for a thickness >50mm <70mm : 255N/mm²

4.4 CONTROL VALVES

The type and size of the control valves shall first depend on the design of the lock culverts and water saving basins conduits.

A comparison of existing valve types shall be carried out. Following criteria shall be taken into account for the choice of the type of control valves:

- Assessment of suitability for the new locks,
- Reliability,
- Economical criteria,
- Compilation of a complete list of the various large types of lock gates currently in use
- Experience on various type of valves,
- Analysis of various types of valves,
- Easy operation and maintenance conditions,
- Influence on hydraulic design of the lock(s),
- Influence on civil works.

We propose to systematically install as basic solution with two valves working in parallel on each culvert instead of one (width of valve divided by two, same head). This layout is widely applied to locks of similar big size. This design has been tested satisfactorily on Berendrecht and Zeebruges locks in Antwerp.

Each control valve shall be equipped upstream and downstream with one auxiliary sliding bulkhead gate for maintenance purpose. Experience has shown that a mobile crane shall be used for handling the four auxiliary bulkhead gates per culvert or conduit flow control point.

Smaller control valves shall be easier to move and to maintain. This redundancy shall provide greater reliability for about the same overall cost.

On the conduits this solution will not be retained, because the WSB are linked with the culverts by means of several conduits.

4.4.1 USE OF HYDRAULIC CYLINDERS

STANDARDS:
ISO – CEN – CETOP :
for hydraulic transmission and hydraulic cylinders
see list in annex 1

We clearly recommend oil hydraulic servomotors to operate the control valves. The usual recommended working pressure shall be 160 bar (16Mpa).

The design of the oil hydraulic servomotor has improved a lot in terms of size, stroke and force, quick-opening / closing time, synchronising,.. and is widely used successfully for valve operating.

In order to reduce the effects of fluctuating hydraulic conditions during opening and closing of the gates, it is easy to pressurise both sides of the piston. Emergency closure by gravity is also very secure for vertical lift gates (such as fixed-wheel roller gates), flap and radial gates.

Design of hydraulic cylinders shall be based on ISO Standards (3320 to 22, 4393 to 95, 5597, 6020,..). Ceramic coating on the piston rods (as « ceramax ») is recommended..

4.4.2 OIL SYSTEM

The oil system of each valve shall consist mainly of two main pumps (one working, one in standby), an oil tank, an oil distribution system, and any control warning, alarm and protection devices required to ensure safe operation. All this equipment, except one tank on top of the servomotor will be accommodated in a metallic cubicle located near the hydraulic cylinder. Accumulators (N2 /oil) shall be provided for security in case of failure.

Equipment shall be composed of standard items designed and manufactured according to ISO / DIN / SAE Standards

01) Pumps

Two pumps for normal operation are proposed to produce the pressure required for the valve upward travel.

Pump discharge values shall depend on valve hoisting rates. The motors shall be designed for the maximum load and hoisting rate requirements.

The pumps shall be of the helical gear type with direct motor drive.

The pump deliveries shall be connected up and provided with all the necessary control and non-return valves. Each pump shall be provided with a relief valve discharging into the oil sump and with manual valves enabling each pump and circuit to be isolated.

The motors shall not start at the same time and the system shall be designed to let the oil discharges return to the tank during the start up procedure.

An emergency hand pump shall be provided for control purposes in the event of a power supply failure.

02) Oil tanks

The capacity of the oil tanks shall be sufficient to ensure satisfactory system operation. The main oil tank shall be provided with an opening allowing inside access and a line mesh filter for the oil returning from the servomotor, which shall be able to be easily dismantled for cleaning.

The oil tanks shall also be provided with an oil level indicator and the feed and drain pipes.

The air entering the tanks shall flow through an air dryer.

03) Pipes and valves

A valve shall be provided for manual emergency closure.

All pipes shall be dimensioned for a maximum oil flow velocity not exceeding 2 m/s over the full range of servomotor travel.

Seamless steel pipes shall be used with welded joints and steel bolted flange connection. All pipes shall be thoroughly cleaned; for transport,. All valves shall have steel bodies, bronze slides and seatings. All gate valves in the pressure circuit shall be of cast steel with the smallest possible clearances in order to reduce vibration to a minimum when working partly under pressure.

Whenever possible use wide-radius pipe bends rather than joints. Pipe-joints shall be welded whenever possible, flanges being used only when necessary for dismantling and reassembling for repairs. As far as possible, the maximum amount of pipe work shall be carried out in the factory, respecting erection, transport and handling limitations in order to reduce work at the site to a minimum. Pipe work shall be tested at a pressure of 50 % above the maximum operating pressure.

AVOID CONTROL VALVE VIBRATION AND CAVITATION EROSION

The filling and emptying system must be appropriate with respect to the effects of agitation and movements of the liquid mass within the lock while keeping the filling and emptying time within restricted limits. Quite frequently the submergence of the culverts is not adequate to maintain the pressure gradient above the culverts at the valves during lock filling and emptying. Model tests on high lift locks have indicated that low pressures below the culvert valves can be relieved by venting with air or water, lowering the culvert or arbitrarily increasing the head loss in the hydraulic system.

Moreover, special attention shall be paid to the bottom edge shape and to the sealing type and location to avoid abnormal vibration.

General recommendations about vibration analysis of the radial and emergency lift type valves :

In order to avoid significant vibrations, an hydrodynamic analysis based on wetted gate side geometry shall be required during detailed design. The upstream and downstream flow pattern for different lip levels must be taken into account. Two cases must be considered : the first one for various fixed positions for which stabilized flow pattern must be obtained (no pulsating flow) and the second one during opening and closing operation (resonance problem).

Particular attention must be paid to the detailed shape and angle of the lower gate edge to maintain for situations a stable flow line starting from the bottom edge (this item is one of the dynamic conditions characterising onset of gate vibration (also valid for vertical lift gates). As a first approach, refer for example to the criteria of Berezinski and G.Wickert. Several papers were presented on the subject during the symposium on practical experiences with flow-induced vibrations (IAHR – Chairmen Pr.Naudascher and Pr. Rockwell).

Any use of above Berezinski criteria requires the value of the eigenfrequency of the steel structure (Sh Strouhal number and damping effect of the water).

It is usual to ask the valve supplier to proceed to the hydrodynamic analysis and to recommend the best overall layout of the valves taking into account the guides of the rolling valve on the hydrodynamic flow pattern.

Physical model studies were often used in the past to improve the valve design. But before any accurate study (during the detailed design study) of the vibration problems needs before rough preliminary mathematical studies.

4.5 EASE OF MAINTENANCE

For maintenance purpose, the space between the bulkhead gates shall be emptied by the use of powerful submersible pumps.

Removable wear parts (such as seals) shall be located on the valve leaf and on bulkhead gates.

4.6 TRASHRACK RAKE AT THE CULVERT INLET

As mentioned during the kick-off meeting, the choice of the trashrack rake to be installed at the culvert inlet shall depend on :

- kind of screenings disposal,
- the nature of the water,
- water flows and relevant water levels,,
- inlet cross-section (w x h),

The equipment for price estimate shall be of the straight bar screen type of vertical design with grab cleaner. Bar spacing shall be adapted to the size of the debris. A head loss of 1m due to obstruction shall be considered .

ANNEX 1

NORMES ISO POUR L'HYDRAULIQUE, LA PNEUMATIQUE ET LES COMPRESSEURS

Catalogue 1997

Voici la publication de la liste actualisée des normes couvrant le matériel et les techniques hydrauliques et pneumatiques au 30 avril 1997.
Rappelons que l'ISO est une norme mondiale définie par divers comités nationaux (IBN), européens (CEN) et mondiaux (ISO).
FIMOP participe aux travaux des commissions via le CETOP (Comité Européen des Transmissions Oléo-Hydrauliques et Pneumatiques).
Toutes les normes font l'objet de cahiers détaillés et peuvent être obtenus contre paiement et selon un tarif (A) (B) (C), (etc.)
à l'IBN (Institut Belge de Normalisation);
tél. 02/738.01.11 - fax 02/733.42.64.

Transmissions hydrauliques et pneumatiques en général

ISO 1219-1 1991 40 p. (R)
Transmissions hydrauliques et pneumatiques - Symboles graphiques et schémas de circuit - Partie 1: Symboles graphiques. Edition bilingue.

ISO 1219-2 1995 21 p. (L)
Transmissions hydrauliques et pneumatiques - Symboles graphiques et schémas de circuit - Partie 2: Schémas de circuit. Edition bilingue.

ISO 2944 1974 1 p. (A)
Transmissions hydrauliques et pneumatiques - Gamme de pressions nominales.

ISO 3938 1986 7 p. (D)
Transmissions hydrauliques - Analyse de la pollution - Méthode de présentation des résultats d'analyse.

ISO 4021 1992 6 p. (C)
Transmissions hydrauliques - Analyse de la pollution par particules - Prélèvement des échantillons de fluide dans les circuits en fonctionnement.

ISO 4402 1991 6 p. (C)
Transmissions hydrauliques - Etalonnage des compteurs automatiques de particules en suspension dans les liquides - Méthode utilisant une fine poussière d'essai (ACFTD).

ISO 4405 1991 4 p. (B)
Transmissions hydrauliques - Pollution des fluides - Détermination de la pollution particulaire par la méthode gravimétrique.

ISO 4406 1987 4 p. (B)
Transmissions hydrauliques - Fluides - Méthode de codification du niveau de pollution par particules solides.

ISO 4407 1991 6 p. (C)
Transmissions hydrauliques - Pollution des fluides - Détermination de la pollution particulaire solide - Méthode de comptage au microscope.

ISO 4413 1979 17 p. (J)
Transmissions hydrauliques - Règles générales pour l'installation et l'utilisation d'équipements dans les systèmes de transmission et de commande.

ISO 4414 1982 25 p. (M)
Transmissions pneumatiques - Règles générales pour l'installation et l'utilisation d'équipements dans les systèmes de transmission et de commande.

ISO 5598 1985 90 p. (X)
Transmissions hydrauliques et pneumatiques - Vocabulaire. Edition bilingue.

ISO 5784-1 1988 21 p. (L)
Transmissions hydrauliques et pneumatiques - Logique par les fluides - Partie 1: Symboles pour fonctions logiques binaires et connexes.

ISO 5784-2 1989 5 p. (C)
Transmissions hydrauliques et pneumatiques - Logique par les fluides - Partie 2: Symboles pour alimentation et échappement, et règles d'emploi des symboles pour les fonctions logiques.

ISO 5784-3 1989 10 p. (E)
Transmissions hydrauliques et pneumatiques - Logique par les fluides - Partie 3: Symboles pour opérateurs séquentiels et fonctions connexes.

ISO 6072 1986 14 p. (G)
Transmissions hydrauliques - Compatibilité des fluides avec les caoutchoucs.

ISO 6358 1989 14 p. (G)
Transmissions pneumatiques - Eléments traversés par un fluide compressible - Détermination des caractéristiques de débit.

ISO 8778 1990 1 p. (A)
Transmissions pneumatiques - Atmosphère normale de référence.

ISO 9110-1 1990 4 p. (B)
Transmissions hydrauliques - Techniques de mesurage - Partie 1: Principes généraux de mesurage.

ISO 9110-2 1990 6 p. (C)
Transmissions hydrauliques - Techniques de mesurage - Partie 2: Mesurage de la pression moyenne dans un conduit formé en régime permanent.

ISO/TR 10949 1996 7 p. (D)
Transmissions hydrauliques - Méthodes de nettoyage et d'évaluation du niveau de propreté des composants.

Pompes et moteurs

ISO 3019-1 1975 8 p. (D)
Transmissions hydrauliques - Pompes volumétriques et moteurs - Dimensions et code d'identification des flasques de montage et des bouts d'arbres - Partie 1: Conversion en unités métriques de la série en inches.

ISO 3019-2 1986 13 p. (G)
Transmissions hydrauliques - Pompes volumétriques et moteurs - Dimensions et code d'identification des flasques de montage et des bouts d'arbres - Partie 2: Flasques à deux et quatre trous et bouts d'arbres - Série métrique.

ISO 3019-3 1988 5 p. (C)
Transmissions hydrauliques - Pompes volumétriques et moteurs - Dimensions et code d'identification des flasques de montage et des bouts d'arbres - Partie 3: Flasques polygonaux (y compris les flasques circulaires).

ISO 3662 1976 2 p. (A)
Transmissions hydrauliques - Pompes et moteurs - Cylindrées géométriques.

ISO 4391 1983 10 p. (E)
Transmissions hydrauliques - Pompes, moteurs et variateurs - Définitions des grandeurs et lettres utilisées comme symboles.

ISO 4392-1 1989 7 p. (D)
Transmissions hydrauliques - Détermination des caractéristiques des moteurs - Partie 1: Essai à pression constante et basse vitesse constante.

ISO 4392-2 1989 10 p. (E)
Transmissions hydrauliques - Détermination des caractéristiques des moteurs - Partie 2: Essai de démarrage.

ISO 4392-3 1993 10 p. (E)
Transmissions hydrauliques - Détermination des caractéristiques des moteurs - Partie 3: Essai à débit constant et couple constant.

ISO 4409 1986 15 p. (H)
Transmissions hydrauliques - Pompes, moteurs et variateurs volumétriques - Détermination du fonctionnement en régime permanent.

ISO 4412-1 1991 20 p. (K)
Transmissions hydrauliques - Code d'essai pour la détermination du niveau de bruit aérien - Partie 1: Pompes.

ISO 4412-2 1991 20 p. (K)
Transmissions hydrauliques - Code d'essai pour la détermination du niveau de bruit aérien - Partie 2: Moteurs.

ISO 4412-3 1991 9 p. (E)
Transmissions hydrauliques - Code d'essai pour la détermination du niveau de bruit aérien - Partie 3: Pompes - Méthode employant un étalage des microphones en parallélépipède.

ISO 8426 1988 9 p. (E)
Transmissions hydrauliques - Pompes et moteurs volumétriques - Détermination de la cylindrée mesurée.

ISO 9632 1992 8 p. (D)
Transmissions hydrauliques - Pompes à cylindrée fixe - Dégradation de l'écoulement due à la pollution par ACFTD.

ISO 10767-1 1996 22 p. (L)
Transmissions hydrauliques - Détermination des niveaux d'onde de pression engendrés dans les circuits et composants - Partie 1: Méthode de précision pour les pompes.

Vérins

ISO 3320 1987 2 p. (A)
Transmissions hydrauliques et pneumatiques - Alésages des vérins et diamètres des tiges de piston - Série métrique.

ISO 3321 1975 2 p. (A)
Transmissions hydrauliques et pneumatiques - Alésages des vérins et diamètres des tiges de piston - Série en inches.

ISO 3322 1985 1 p. (A)
Transmissions hydrauliques et pneumatiques - Vérins - Pressions nominales.

ISO 4393 1978 2 p. (A)
Transmissions hydrauliques et pneumatiques - Vérins - Série de base de courses de piston.

ISO 4394-1 1980 9 p. (F)
Transmissions hydrauliques et pneumatiques - Tubes pour vérins - Partie 1: Caractéristiques des tubes en acier à alésage de finition spéciale.

ISO 4395 1978 2 p. (A)
Transmissions hydrauliques et pneumatiques - Vérins - Dimensions et types de filetage des tiges de piston.

ISO 5597 1987 9 p. (E)
Transmissions hydrauliques - Vérins - Logements de joints d'étanchéité pour pistons et tiges de piston - Dimensions et tolérances.

ISO 6020-1 1981 6 p. (D)
Transmissions hydrauliques - Vérins 160 bar (16.000 kPa) à simple tige - Dimensions d'interchangeabilité - Partie 1: Série moyenne.

ISO 6020-2 1991 16 p. (H)
Transmissions hydrauliques - Dimensions d'interchangeabilité des vérins 16 MPa (160 bar) à simple tige - Partie 2: Série compacte.

ISO 6020-3 1994 12 p. (F)
Transmissions hydrauliques - Dimensions d'interchangeabilité des vérins 16 MPa (160 bar) à simple tige - Partie 3: Série compacte, alésages de 250 à 500 mm.

ISO 6022 1981 5 p. (C)
Transmissions hydrauliques - Vérins 250 bar (25.000 kPa) à simple tige - Dimensions d'interchangeabilité.

ISO 6099 1986 45 p. (S)
Transmissions hydrauliques et pneumatiques - Vérins - Code d'identification des dimensions de montage et des modes de fixation.

ISO 6195 1986 6 p. (C)
Transmissions hydrauliques et pneumatiques - Vérins - Logements de joints racleurs pour tiges de piston à mouvement linéaire - Dimensions et tolérances.

ISO 6430 1992 16 p. (H)
Transmissions pneumatiques - Vérins 1.000 kPa (10 bar) à simple tige, à fixations intégrées, de diamètres d'alésage 32 à 250 mm - Dimensions d'interchangeabilité.

ISO 6431 1992 11 p. (F)
Transmissions pneumatiques - Vérins 1.000 kPa (10 bar) à simple tige, à fixations détachables, de diamètres d'alésage 32 à 320 mm - Dimensions d'interchangeabilité.

ISO 6432 1985 5 p. (C)
Transmissions pneumatiques - Vérins à simple tige - Série 10 bar (1.000 kPa) - Alésages de 8 à 25 mm - Dimensions de montage.

ISO 6537 1982 6 p. (C)
Transmissions pneumatiques - Tubes pour vérins - Caractéristiques des tubes en métaux non ferreux.

ISO 6547 1981 3 p. (B)
Transmissions hydrauliques - Vérins - Logements de joints d'étanchéité à bagues de guidage pour pistons - Dimensions et tolérances.

ISO 6981 1982 4 p. (B)
Transmissions hydrauliques - Vérins - Tenons simples d'extrémité de tige de piston - Dimensions d'interchangeabilité.

ISO 6982 1982 4 p. (B)
Transmissions hydrauliques - Vérins - Tenons à rotule d'extrémité de tige de piston - Dimensions d'interchangeabilité.

ISO 7180 1986 2 p. (A)
Transmissions pneumatiques - Vérins - Dimensions d'alésage et de filetage des orifices.

ISO 7181 1991 3 p. (B)
Transmissions hydrauliques - Vérins - Rapports entre surfaces d'alésage et de tige.

ISO 7425-1 1988 5 p. (C)
Transmissions hydrauliques - Logements pour joints en élastomère renforcé par des matières plastiques - Partie 1: Logements de joints de piston.

ISO 7425-2 1989 3 p. (B)
Transmissions hydrauliques - Logements pour joints en élastomère renforcé par des matières plastiques - Dimensions et tolérances - Partie 2: Logements de joints de tige.

ISO 8131 1992 4 p. (B)
Transmissions hydrauliques - Vérins 16 MPa (160 bar) à simple tige, série compacte - Tolérances.

ISO 8132 1986 8 p. (D)
Transmissions hydrauliques - Vérins 160 bar (16 MPa) série moyenne et 250 bar (25 MPa), à simple tige - Dimensions d'interchangeabilité des accessoires.

ISO 8133 1991 12 p. (F)
Transmissions hydrauliques - Vérins 16 MPa (160 bar) à simple tige, série compacte - Dimensions d'interchangeabilité des accessoires.

ISO 8135 1986 2 p. (A)
Transmissions hydrauliques - Vérins 160 bar (16 MPa) série moyenne et 250 bar (25 MPa), à simple tige - Tolérances.

ISO 8136 1986 2 p. (A)
Transmissions hydrauliques - Vérins 160 bar (16 MPa) série moyenne à simple tige - Dimensions des orifices.

ISO 8137 1986 2 p. (A)
Transmissions hydrauliques - Vérins 250 bar (25 MPa) à simple tige - Dimensions des orifices.

ISO 8138 1986 2 p. (A)
Transmissions hydrauliques - Vérins 160 bar (16 MPa) série compacte à simple tige - Dimensions des orifices.

ISO 8139 1991 4 p. (B)
Transmissions pneumatiques - Vérins 1.000 kPa (10 bar) - Tenons à rotule d'extrémité de tige de piston - Dimensions d'interchangeabilité.

ISO 8140 1991 4 p. (B)
Transmissions pneumatiques - Vérins 1.000 kPa (10 bar) - Chapes d'extrémité de tige de piston - Dimensions d'interchangeabilité.

ISO 10099 1990 3 p. (B)
Transmissions pneumatiques - Vérins - Essais de réception.

ISO 10100 1990 2 p. (A)
Transmissions hydrauliques - Vérins - Essais de réception.

ISO 10766 1996 7 p. (D)
Transmissions hydrauliques - Vérins - Dimensions de logements de dispositifs de guidage à section rectangulaire pour pistons et tiges de piston.

Tuyauterie, raccord, robinets et filtres

ISO 1436 1991 7 p. (B)
Tuyaux et flexibles en caoutchouc - Type hydraulique à armature de fils métalliques - Spécifications.

ISO 2941 1974 2 p. (A)
Transmissions hydrauliques - Eléments filtrants - Vérification de la résistance à l'écrasement ou à l'éclatement.

ISO 2942 1974 4 p. (B)
Transmissions hydrauliques - Eléments filtrants - Vérification de la conformité de fabrication et détermination du point de première bulle.

ISO 2943 1974 2 p. (A)
Transmissions hydrauliques - Eléments filtrants - Vérification de la compatibilité des matériaux avec les fluides.

ISO 3723 1976 1 p. (A)
Transmissions hydrauliques - Eléments filtrants - Méthode de détermination de la résistance à la déformation axiale.

ISO 3724 1976 2 p. (A)
Transmissions hydrauliques - Eléments filtrants - Vérification des caractéristiques d'un filtre par un essai de fatigue due au débit.

ISO 3862 1991 6 p. (C)
Tuyaux et flexibles en caoutchouc - Type hydraulique à revêtement de caoutchouc et armature spiralée de fil métallique - Spécifications.

ISO 3949 1991 4 p. (B)
Tuyaux et flexibles en plastique - Type hydraulique en thermoplastiques à armature textile - Spécifications.

ISO 3968 1981 7 p. (D)
Transmissions hydrauliques - Filtres - Evaluation de la perte de charge en fonction du débit.

ISO 4079 1991 4 p. (B)
Tuyaux et flexibles en caoutchouc - Type hydraulique à armature textile - Spécifications.

ISO 4397 1993 2 p. (A)
Transmissions hydrauliques et pneumatiques - raccords et éléments associés - Diamètres extérieurs nominaux des tubes et diamètres intérieurs nominaux des tuyaux flexibles.

ISO 4399 1995 2 p. (A)
Transmissions hydrauliques et pneumatiques - Raccords et composants associés - Pressions nominales.

ISO 4401 1994 16 p. (J)
Transmissions hydrauliques - Distributeurs à quatre orifices - Plan de pose.

ISO 4411 1986 11 p. (F)
Transmissions hydrauliques - Appareils de distribution - Détermination des caractéristiques pression différentielle/débit.

ISO 4572 1981 28 p. (N)
Transmissions hydrauliques - Filtres - Evaluation du rendement par la méthode de filtration en circuit fermé.

ISO 5599-1 1989 0 p. (D)
Transmissions pneumatiques - distributeurs à cinq orifices principaux - Partie 1: Plans de pose sans connecteur électrique.

ISO 5599-2 1990 12 p. (F)
Transmissions pneumatiques - Distributeurs à cinq orifices principaux - Partie 2: Plans de pose avec connecteur électrique facultatif.

ISO 5599-3 1990 9 p. (E)
Transmissions pneumatiques - Distributeurs à cinq orifices principaux - Partie 3: Codification de l'information sur les fonctions des distributeurs.

ISO 5781 1987 17 p. (J)
Transmissions hydrauliques - Réducteurs de pression (à l'exception des limiteurs de pression), soupapes de séquence, soupapes de décharge, soupapes d'étranglement, clapets de non-retour - Plan de pose.

ISO 5782-1 1990 5 p. (C)
Transmissions pneumatiques - Filtres pour air comprimé - Partie 1: Principales caractéristiques à inclure dans la documentation commerciale et exigences particulières.

ISO 5783 1995 4 p. (B)
Transmissions hydrauliques - Code pour l'identification des plans de pose et des logements de cartouche.

ISO 6149-1 1993 6 p. (C)
Raccordements pour transmissions hydrauliques et pneumatiques et applications générales - Orifices et éléments mâles à filetage ISO 261 et joint torique - Partie 1: Orifices à joint torique dans un logement tronconique.

ISO 6149-2 1993 10 p. (E)
Raccordements pour transmissions hydrauliques et pneumatiques et applications générales - Orifices et éléments mâles à filetage ISO 261 et joint torique - Partie 2: Eléments mâles de série lourde (série S) - Dimensions, conception, méthodes d'essai et prescriptions.

ISO 6149-3 1993 10 p. (E)
Raccordements pour transmission hydrauliques et pneumatiques et applications générales - Orifices et éléments mâles à filetage ISO 261 et joint torique - Partie 3: Eléments mâles de série légère (série L) - Dimensions, conception, méthodes d'essai et prescriptions.

ISO 6150 1988 13 p. (G)
Transmissions pneumatiques - Raccords rapides cylindriques pour pressions maximales d'utilisation 10 bar, 16 bar et 25 bar (1 MPa, 1,6 MPa et 2,5 MPa) - Dimensions de raccordement de la partie mâle, spécifications, conseils d'utilisation et essais.

ISO 6162 1994 13 p. (G)
Transmissions hydrauliques - Brides de raccordement fendues à quatre vis pour des pressions d'utilisation de 2,5 à 40 MPa (25 à 400 bar) - Type I série métrique et type II série en inches.

ISO 6164 1994 11 p. (F)
Transmissions hydrauliques - Brides de raccordement carrées monobloc à quatre vis pour des pressions d'utilisation de 25 MPa et 40 MPa (250 et 400 bar).

ISO 6263 1987 17 p. (J)
Transmissions hydrauliques - Régulateurs de débit - Plan de pose.

ISO 6264 1987 25 p. (M)
Transmissions hydrauliques - Limiteurs de pression - Plan de pose.

ISO 6403 1988 53 p. (V)
Transmissions hydrauliques - Régulateurs de débit et de pression - Méthodes d'essai.

ISO 6404 1985 23 p. (L)
Transmissions hydrauliques - Servodistributeurs - Méthodes d'essai.

ISO 6605 1986 6 p. (C)
Transmissions hydrauliques - Flexibles de raccordement - Méthode d'essai.

ISO 7241-1 1987 5 p. (C)
Transmissions hydrauliques - Raccords rapides - Partie 1: Dimensions et spécifications.

ISO 7241-2 1988 15 p. (H)
Transmissions hydrauliques - Raccords rapides - Partie 2: Méthodes d'essai.

ISO 7368 1989 17 p. (J)
Transmissions hydrauliques - Distributeurs à cartouche, à bride, à deux orifices - Logements.

ISO 7744 1986 5 p. (C)
Transmissions hydrauliques - Filtres - Spécification des conditions d'emploi.

ISO 7790 1986 2 p. (A)
Transmissions hydrauliques - Appareils empilables et distributeurs à quatre orifices, de tailles 03 et 05 - Dimensions de montage.

ISO 8434-1 1994 28 p. (N)
Raccords de tubes métalliques pour transmissions hydrauliques et pneumatiques et applications générales - Partie 1: Raccords à compression à 24 degrés.

ISO 8434-2 1994 28 p. (N)
Raccords de tubes métalliques pour transmissions hydrauliques et pneumatiques et applications générales - Partie 2: Raccords évasés à 37 degrés.

ISO 8434-3 1995 29 p. (N)
Raccords de tubes métalliques pour transmissions hydrauliques et pneumatiques et applications générales - Partie 3: Raccords à joints faciaux toriques.

ISO 8434-4 1995 25 p. (M)
Raccords de tubes métalliques pour transmissions hydrauliques et pneumatiques et applications générales - Partie 4: Raccords à cône à 24 degrés, à embout à souder à joint torique.

ISO 8434-5 1995 7 p. (D)
Raccords de tubes métalliques pour transmissions hydrauliques et pneumatiques et applications générales - Partie 5: Méthodes d'essai pour raccords filetés pour transmissions hydrauliques.

ISO 9974-1 1996 4 p. (B)
Raccordements pour applications générales et transmissions hydrauliques et pneumatiques - Orifices et éléments mâles à filetage ISO 261 et joint en élastomère ou étanchéité métal sur métal - Partie 1: Orifices filetés.

ISO 9974-2 1996 8 p. (D)
Raccordements pour applications générales et transmissions hydrauliques et pneumatiques - Orifices et éléments mâles à filetage ISO 261 et joint en élastomère ou étanchéité métal sur métal - Partie 2: Eléments mâles avec joint en élastomère (type E).

ISO 9974-3 1996 7 p. (D)
Raccordements pour applications générales et transmissions hydrauliques et pneumatiques - orifices et éléments mâles à filetage ISO 261 et joint en élastomère ou étanchéité métal sur métal - Partie 3: Eléments mâles avec étanchéité métal sur métal (type B).

ISO 10372 1992 7 p. (D)
Transmissions hydrauliques - Servodistributeurs à quatre et cinq orifices - Plans de pose.

ISO 10763 1994 3 p. (B)
Transmissions hydrauliques - Tubes de précision en acier, soudés ou non, à extrémités lisses - Dimensions et pressions nominales de travail.

ISO 11170 1995 4 p. (B)
Transmissions hydrauliques - Eléments filtrants - Procédure d'homologation.

ISO/TR 11340 1994 2 p. (B)
Caoutchouc et produits en caoutchouc - Flexibles hydrauliques - Classification des fuites externes des installations hydrauliques.

ISO 11926-1 1995 4 p. (B)
Raccordements pour applications générales et transmissions hydrauliques et pneumatiques - Orifices et éléments mâles à filetage ISO 725 et joint torique - Partie 1: Orifices à joint torique dans un logement tronconique.

ISO 11926-2 1995 11 p. (F)
Raccordements pour applications générales et transmissions hydrauliques et pneumatiques - Orifices et éléments mâles à filetage ISO 725 et joint torique - Partie 2: Eléments mâles de série lourde (série S).

ISO 11926-3 1995 11 p. (F)
Raccordements pour applications générales et transmissions hydrauliques et pneumatiques - Orifices et éléments mâles à filetage ISO 725 et joint torique - Partie 3: Eléments mâles de série légère (série L).

Autres éléments pour transmissions hydrauliques et pneumatiques

ISO 3601-1 1988 4 p. (B)
Systèmes de fluides - Joints d'étanchéité - Joints toriques - Partie 1: Diamètres intérieurs, sections, tolérances et code d'identification dimensionnelle.

ISO 3601-3 1987 7 p. (D)
Systèmes de fluides - Joints d'étanchéité - Joints toriques - Partie 3. Critères de qualité.

ISO 3722 1976 2 p. (A)
Transmissions hydrauliques - Flacons de prélèvement - homologation et contrôle des méthodes de nettoyage.

ISO 3939 1977 4 p. (B)
Transmissions hydrauliques et pneumatiques - Garnitures à lèvres multiples - Méthodes de mesurage des hauteurs d'empilage.

ISO 4400 1984 4 p. (B)
Transmissions hydrauliques et pneumatiques - Connecteurs électriques à trois broches avec contact de sécurité - Caractéristiques et exigences.

ISO 5596 1982 4 p. (B)
Transmissions hydrauliques - Accumulateurs hydropneumatiques avec séparateur - Gammes de pressions et de volumes, grandeurs caractéristiques et identification.

ISO 6194-1 1982 7 p. (D)
Bagues d'étanchéité à lèvres pour arbres tournants - Partie 1: Dimensions nominales et tolérances.

ISO 6194-2 1991 18 p. (J)
Bagues d'étanchéité à lèvre pour arbres tournants - Partie 2: Vocabulaire. Edition bilingue.

ISO 6194-3 1988 7 p. (D)
Bagues d'étanchéité à lèvre pour arbres tournants - Partie 3: Stockage, manipulation et montage.

ISO 6194-4 1988 9 p. (B)
Bagues d'étanchéité à lèvre pour arbres tournants - Partie 4: Méthodes d'essai de performance.

ISO 6194-5 1990 7 p. (D)
Bagues d'étanchéité à lèvre pour arbres tournants - Partie 5: Identification des imperfections visuelles.

ISO 6301-1 1989 5 p. (C)
Transmissions pneumatiques - Lubrificateurs pour air comprimé - Partie 1: Principales caractéristiques à inclure dans la documentation commerciale et exigences particulières.

ISO 6952 1994 5 p. (C)
Transmissions hydrauliques et pneumatiques - Connecteurs électriques à deux broches avec contact de sécurité - Caractéristiques et exigences.

ISO 6953-1 1990 5 p. (C)
Transmissions pneumatiques - Réducteurs de pression pour air comprimé - Partie 1: Principales caractéristiques à inclure dans la documentation commerciales et exigences particulières.

ISO 9461 1992 3 p. (B)
Transmissions hydrauliques - Identification des orifices des appareils, embases, organes de commande et solénoïdes.

ISO 10945 1994 2 p. (A)
Transmissions hydrauliques - Accumulateurs hydropneumatiques - Dimensions des orifices gaz.

Compresseurs et machines pneumatiques

ISO 1217 1996 58 p. (U)
Compresseurs volumétriques - Essais de réception.

ISO 2151 1972 7 p. (D)
Mesure du bruit aérien émis par des groupes moto-compresseurs destinés à être utilisés à l'extérieur (A remplacer par la future ISO 3989 du TC 43).

ISO 3857-1 1977 6 p. (C)
Compresseurs, outils et machines pneumatiques - Vocabulaire - Partie 1: Généralités. Edition bilingue.

ISO 3857-2 1977 4 p. (B)
Compresseurs, outils et machines pneumatiques - Vocabulaire - Partie 2: Compresseurs. Edition bilingue.

ISO 3857-3 1989 7 p. (D)
Compresseurs, outils et machines pneumatiques - Vocabulaire - Partie 3: Outils et machines pneumatiques. Edition bilingue.

ISO 5388 1981 20 p. (K)
Compresseurs d'air fixes - Règles de sécurité et code d'exploitation.

ISO 5389 1992 172 p. (XC)
Turbocompresseurs - Code d'essais des performances.

ISO 5390 1977 6 p. (C)
Compresseurs - Classification. Edition bilingue.

ISO 5391 1988 13 p. (G)
Machines portatives pneumatiques et machines pneumatiques - Vocabulaire. Edition bilingue.

ISO 5941 1979 2 p. (A)
Compresseurs, outils et machines pneumatiques - Pressions préférentielles.

ISO 7183 1986 13 p. (C)
Sécheurs à air comprimé - Spécifications et essais.

ISO 7183-2 1996 6 p. (C)
Sécheurs d'air comprimé - Partie 2: Caractéristiques nominales de fonctionnement.

ISO 8010 1988 56 p. (U)
Compresseurs pour l'industrie de procédé - Types à vis et connexes - Spécifications et feuilles de données pour la conception et la construction.

ISO 8011 1988 100 p. (X)
Compresseurs pour l'industrie de procédé - Turbocompresseurs - Spécifications et feuilles de données pour la conception et la construction.

ISO 8012 1989 70 p. (V)
Compresseurs pour l'industrie de procédé - Types alternatifs - Spécifications et feuilles de données pour la conception et la construction.

ISO 8573-1 1991 8 p. (D)
Air comprimé pour usage général - Partie 1: Polluants et classes de qualité.

ISO 8573-2 1996 23 p. (L)
Air comprimé pour usage général - Partie 2: Méthodes d'essai pour mesurer les aérosols d'huile.

ISO 11123 1994 8 p. (D)
Machines portatives pneumatiques et machines pneumatiques - Interfaces électroniques.

5 OPERATING MACHINERY

The type of gate operating machinery shall depend on the proposed lock gate alternative. Closure device used for lock gates can be indeed very different as for example:

- miter gates that are usually operated by hydraulic cylinders,
- rolling gates that are typically operated by a motorised winch.

The design criteria of hydraulic cylinders system has been described above with the «Culvert and conduit valves ».

Regarding the design of the winch system for rolling gates, we propose to refer by comparison to existing similar lock equipment (in particular to Berendrecht and Zeebruges locks) and proceed if necessary to some design improvement based on past operating and maintenance experience. Operating experience has shown limitations when applying some theroretical calculation.

The design of winches shall be based on ISO / SIS standards called:

- ISO Standards Handbook, Cranes in 2 volumes:
  - Volume 1 :
    - Terminology and graphic symbols,
    - Information to be provided,
    - Use, operation and maintenance,
  - Volume 2 :
    - Design requirements,
    - Accessories.

The operating machinery shall be in accordance with the final recommended type of control gates and valves.

Gear boxes shall be used in case the lock gate is of the rolling type. High factors of safety shall be chosen to ensure long life and avoid “fatigue” phenomenon on the gear teeth. Standards are not been found regarding the gearbox as a whole. The design shall be based on existing equipment of big locks with rolling gates.

Gears shall be dip-lubricated by means of an oil bath which is an integral part of the gearbox casing. Bearings shall be equipped with standard roller bearings easy to remove. The casing shall be horizontally split by flange and sealing system.

On the two redundant electrical motors (one operating and the other in standby) shall be connected to the main gearboxes on the high speed side, they shall be working with AC.

Experience with DC motors and use of thyristors controlled by electronics is considered by Antwerp operators as not enough reliable and requiring substantial maintenance. A third smaller AC motor shall be used for emergency closing of the lock gate at low speed.

Diesel emergency sets shall provide AC emergency supply.

Submerged and top supports of the rolling gates as well as guiding railways shall be proposed similar to the ones experienced for large locks in the port of Antwerp. The best experienced material quality for bearing areas, wheel axles,.. shall be applied to the present design.

In the case of miter gates, the systems in use shall be similar to the existing locks on the Panama Canal after recent implementation of new hydraulic cylinders. ll be reviewed and are likely to serve as a basis for the design of the moving parts including lubrication systems.

6 Lighting

6.1 GENERAL

The equipment, auxiliary devices, fittings and software, as well as their execution, assembly and installation requirements shall at all times be consistent with sound engineering practice and shall be designed in a way that they satisfy with following requirements :

- not compromise the safety of individuals or the integrity of goods,
- ensure the perfect operation of the lock(s) they are related to, and present a perfect visual aspect,
- reduce to the lowest possible the number of potential breakdowns and incidents, by excluding for instance any component that may incur premature ageing,
- permit easy access, i.e. in suitable conditions of safety and comfort, to the components, controls and connections,
- ensure optimal conditions of operation,
- allow modifications and extensions.

Any proposed type of equipment or material incorporated in the supplied installations shall be based on first class existing products. We intend to propose in agreement with ACP the most suitable locations of the lamps (in particular in the lock walls and above lock gates)

6.2 STANDARDS AND REFERENCE DOCUMENTS

The installations, equipment and fittings shall be compliant in particular with the :

- General Safety Regulations in Panama,
- General Regulations on Electrical Installations,
- DIN 67 500 : Beleuchtung von Schleusenanlagen -Anforderungen, berechnung und Messung (in German language),
- IEC 129 : Guide for lighting exterior work areas-4.4 Harbours,
- IEC Publication 88 : Guide for the lighting of road tunnel and underpasses,
- NBN C 72-662 : High-pressure sodium vapour lamps,
- IEC 922, NBN C71-922 : Ballast for discharge lamps (except fluorescent tubes),

- IEC 923, NBN C71-923 : Auxiliaries for lamps - Ballast for discharge lamps,
- IEC 921, NBN C71-926 : Auxiliaries for lamps – Starting devices,
- IEC 927, NBN C71-927 : Starting devices,
- L18-001: Code of good practice for public lighting
- L18-002 : Recommendations for particular cases of public lighting
- C71-598 : Particular rules for light fixtures (electrical safety).

6.2.1 CLASS OF PROTECTION

The protection classes of the electrical equipment shall be defined according to the equipment location, so as to meet the water criteria (AD), solid particle criteria (AE) and mechanical load criteria (AG).

6.2.2 AUXILIARY DEVICES FOR HIGH-PRESSURE SODIUM VAPOUR LAMPS

Ballast, starting device, condenser

The auxiliary devices shall be in accordance with IEC 922/923 and VDE 0712 standards.

Ballast shall be of the electromagnetic type.

The starting device shall be of the electronic self-blocking type to avoid flashing (pumping) in the event of a lamp defect, and shall be compatible with the selected type of ballast.

The compensation condenser shall be equipped with a discharge resistance.

6.2.3 LIGHT SOURCES

High-pressure sodium vapour lamps

The tube-type HP sodium vapour lamps shall be compliant to the requirements of NBN C72-662 and shall satisfy to following conditions:

The flux and operational life values specified in the tables below shall be considered as a minimum design criteria.

After a 12-hour operation cycle the characteristics shall be as follows:

Rated wattage of the lamp (W)	Colour temperature (K)	Colour rendition index	Flux (Lm)	Power absorbed with auxiliary devices (W)	Average useful life (h)	Operational life with 15 % failure (h)
50	1950	25	3.900	64	9.000	6.000
70	1950	25	6.500	86	10.000	7.000
100	1950	25	9.700	120	12.000	8.000
150	1950	25	15.500	176	12.000	8.000
250	1950	25	29.000	285	15.000	10.000
400	1950	25	53.000	442	15.000	10.000
1000	1950	25	130.000	1056	15.000	6.000

The average useful life of the lamp shall be that of the number of hours of operation in a laboratory at standardised ageing conditions at the end of which 50 % of the lamps of the tested batch have failed.
Any lamp that in the course of this test produces less than 70 % of the rated luminous flux is considered to have failed.

6.2.4 ELECTRICAL CONNECTIONS FOR THE LIGHT FIXTURES

The internal wiring of the light fixture shall be done with flexible 1.5 mm² wires that withstand a temperature of 105°C. Each conductor shall be clearly identified by means of markings at either ends and by using wires of different colours. The wires shall be suited to the rated voltage and the operating current, and shall withstand the effects of direct outdoor exposure : mechanical and thermal loads, UV radiation.

The electrical links to the electrical compartment shall be arranged by a system of plug-in/out terminals with polarising slot, designed so that the connection to earth is made before connection of the active parts.

Each light fixture is equipped with a terminal block compatible with the intended number of lighting duties.

As the basic solution, the light fixtures are connected by the so-called «branch-off » method. A connection box of IP657 class of protection shall be placed next to the light fixture. In this box, the connection shall be made between the feeder cables (incoming/outgoing) of the lighting circuit and the derivation to the light fixture.

The terminal blocks shall be suitable for the following connections:

- feeder : 2 cables of 5 x 25 mm²
- derivation : 1 or 2 cables of 3 x 2.5 mm²

Alternatively, the light fixtures can be connected by the « thru» method.
A connection box similar to that for the other method is fitted to the light fixture.

Terminal blocks shall be of the clamp type.
It shall be forbidden to connect more than one wire per terminal.

With the ambient operating temperature, and at rated voltage, the temperatures of all the insulating parts in all the light fixture connection boxes shall not exceed 100°C.

All wiring shall withstand a test voltage of 2000 V held one minute.

6.2.5 MISCELLANEOUS CHARACTERISTICS

6.2.5.1 Protection against corrosion

The necessary measures shall be taken to prevent different metals to be in direct contact. Any nut, bolt and fixing device used for mounting the light fixture components shall be of stainless steel.

Bolting used in order to mount light fixtures shall be 15-micron cadmium-plated and provided with an isolating washer.

Parts made of extruded aluminium shall be protected against corrosion by means of :

- either, anode oxidation
. to 15-micron depth for the outer surfaces,
. to 10-micron depth for the inner surfaces;

  - or, two coats of RAL 7030-colour powdered paint, applied to a thickness of at least 100 microns,

  - or, by self-protection using an alloy having enough silicon content to allow only limited surface corrosion.

Welded aluminium parts shall be corrosion protected by coating with paint.

All painted components shall have a corrosion warranty of at least 10 years.

6.2.5.2 Plastics

Thermoplastic materials as may be used for the lamps or the auxiliary devices shall be free of any halogen compounds. Furthermore, the thermoplastic materials shall be self-extinguishing.

6.2.5.3 Fire resistance

The lamps and their auxiliary devices shall be designed, built and installed to provide ½ hour fire resistance consistent with the requirements of the NBN 713.020 standard.

7 OPERATING STRUCTURES

7.1 GENERAL

The lockmaster control room with operator console, control and monitoring panels must ideally overlook the lock(s) from the top floor of a single control building. It is indeed desirable that the operator has a clear view of the inside of the lock and of the approaches.

It can be easily realised for the 1 single lock configuration.

In the case of the three-locks configuration, it is for sure advisable to incorporate closed circuit television systems in order to allow full monitoring of all the locks.

Technical rooms shall be located on the first floor in order to get easy access from outside.

The design of the operating structures shall be based on following criteria :

- Reliability of operation,
- Visibility for control of locks and entrances,
- Use of modular and standard equipment according to the IEEE,
- Use of a similar architecture of existing and modern installations,
- Use of equipment with a long MTBF (Minimum Time Before Failure),
- Use of data network and field bus,
- Use of redundancy for automated elements and critical equipment,
- Software with easy maintenance,
- Possibility of integration of all functions of existing locks in one structure,
- Provide rapid and efficient operation,
- Know the statement of the different technical installations,
- Allow a quick and easy maintenance.

For safety reasons, electrical supply to the lock(s) equipment shall be supplied through an Uninterruptible Power Supply or by a converter with batteries for the DC circuit.

In particular, the heart of control systems is located near the field equipment. For the control of critical equipment, the system of 3 captors (of sensors) shall be used. The real information (position of gates,..) is defined according to the majority (2) of the captors. This system improves security of lock(s) operation. It also clearly provides basic information for replacement of any captor out of use.

All floor plans with definition, location and size of equipment shall be provided.

Location and space requirements of maintenance facilities shall also be considered for the layout.

The architecture shall be divided within different levels:

- level 0 : field equipment level,
- level 1 : local automatism level,
- level 2 : zone automated elements,
- level 3 : central supervision and control (in the control room)

7.1.1 NETWORK BETWEEN LEVEL 0 AND 1.

For field equipment (Input / Output), transmission shall be made by field bus. Serial liaison shall be of the type RS 485 - PROFIBUS DP, PROFIBUS FMS, MODBUS or by equivalent and standard bus. Coaxial cable, copper cable or optical fibre shall be used.

7.1.2 NETWORK BETWEEN LEVELS 1 AND 2 AND 3

Each automated element and operator console is connected to a transmission data network. The real type is to define after evaluation of the amount of I / O. But it would be equivalent in communication speed to a standard such Ethernet (IEEE 802.3). Optical fibre or coaxial cable as well as standard and modular equipment shall be used.

Special requirements shall be recommended for equipment that operate in a humid tropical environment.

7.1.3 FIELD EQUIPMENT :

The equipment for the data acquisition shall be chosen according to manufacturer specification.

Automated elements and controllers :
Redundancy through separate power supplying standby equipment,
Modern standardised rack mounted equipment.
Self diagnostic with failure mode display.

7.1.4 OPERATOR CONSOLES :

Dynamic graphic displays on high resolution displays monitor.
All actions initiated from keyboard (Mimic diagram display).
Control and alarm indicator.
Printing and logging of alarm and events and routine data on separate printers
Backup on hard disk

7.1.5 CONTROL/COMMAND PANEL :

Mimic diagram of the system that allows the commands of gates, valves with push-buttons shall be studied.

7.2 STANDARDS

Control equipment design shall be carried out according to following well known international standards organisations as:

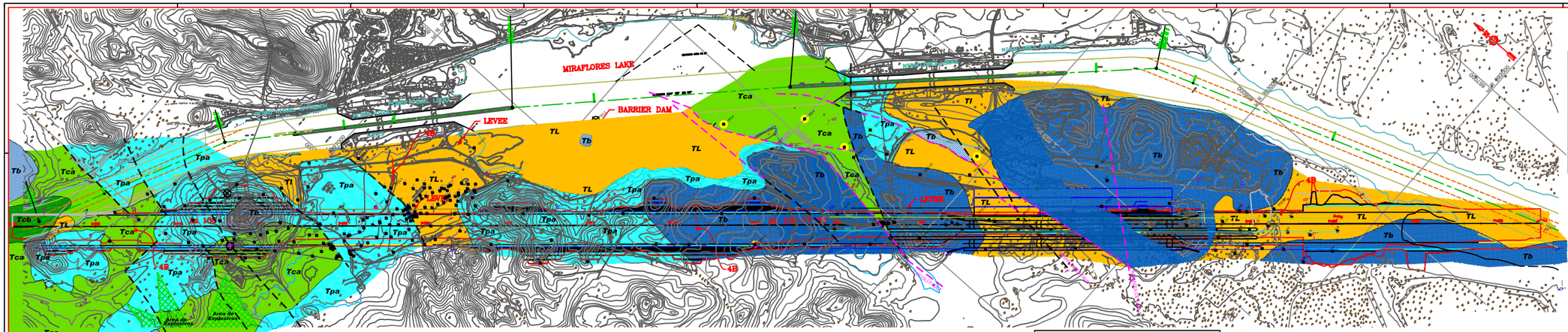
- IEC: International Electrotechnical Commission ;
- UIT: Union Internationale des Télécommunications ;
- ISO : International Standardisation Organisation ;
- CENELEC: European Standards ;
- ETSI: European Technical Standard Institute ;
- IEEE: Institution of Electrical and Electronical Engineers ;
- EIA: Electronic Industry Association.

Control design shall also take into account experienced practice of lock operators and manufacturers of the main equipment and in particular for gate and valve operating equipment.

DRAWING LIST
TASK 2

2/22/2006

x	Rev.	Title
D2- 0 -001	B	Mapa geologico con serie TP1, TP1C y TP2 y TYPC + Miraflores
D2- 0 -002	B	Pacifico Alineamiento P1 Seccion Longitudinal Mostrando valores de CR%, RQD%, UCS, y PLT, Perfil 1 Este
D2- 0 -003	B	Pacifico Alineamiento P1 Seccion Longitudinal Mostrando valores de CR%, RQD%, UCS, y PLT, Perfil 2 Centro
D2- 0 -004	B	Pacifico Alineamiento P1 Seccion Longitudinal Mostrando valores de CR%, RQD%, UCS, y PLT, Perfil 3 Oeste
D2- 0 -005	B	Alineamiento 1, Perfiles Geologicos
D2- 0 -006	D	Triple-lift configuration



MAPA GEOLOGICO SECTOR PACIFICO

ESCALA 1:5000

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GEOLOGICAL MAP REVISED BY CONSORCIO POST PANAMAX (CPP) MARCH/2002

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Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
MAPA GEOLOGICO CON SERIE TP1, TP1C Y TP2 Y TP2C
+MIRAFLORES

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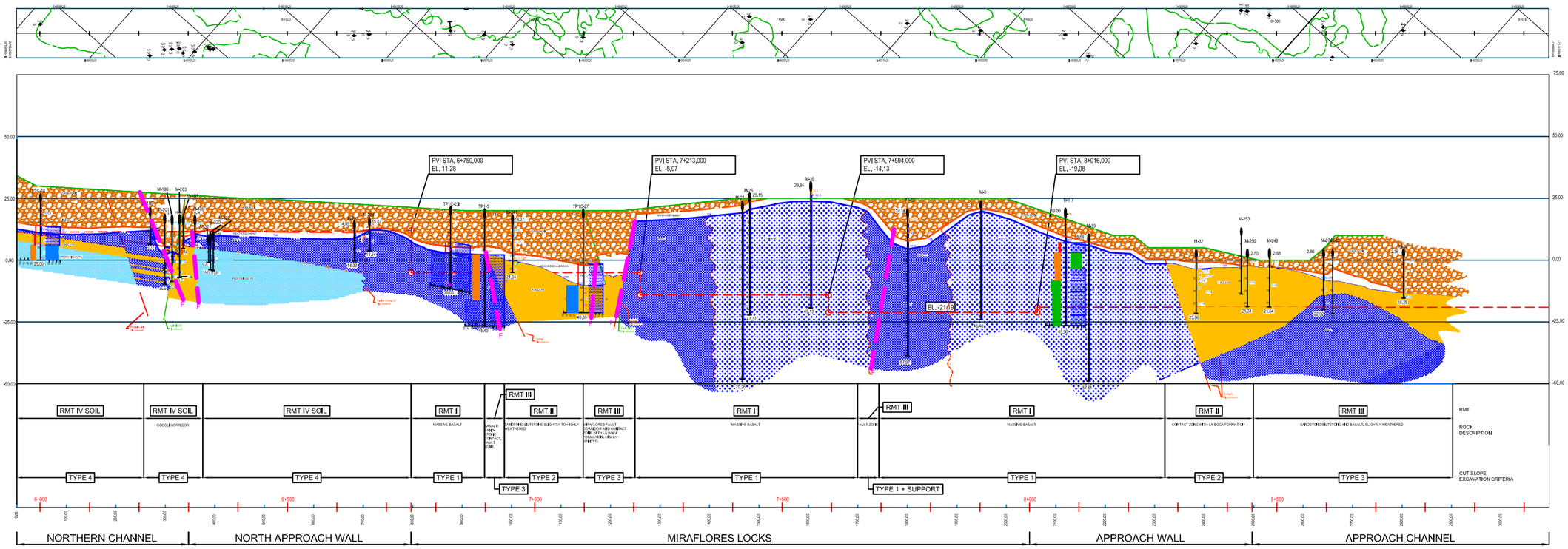

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PERFIL GEOLÓGICO DE PARED ESTE REVISADO POR CONSORCIO POST PANAMAX (CPP)
MARZO-2002

GEOLOGICAL PROFILE OF EAST WALL REVISED BY CONSORCIO POST PANAMAX (CPP)
MARCH-2002

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EXCAVATION SLOPE CRITERIA:

TYPE 1. STRONG, UNALTERED OR SLIGHTLY ALTERED VOLCANIC ROCKS, E.G. BASALT AND HARD AGGLOMERATE LIMESTONE. SLOPES NOT EXCEEDING 60 METERS IN HEIGHT OF MODERATELY CONSOLIDATED, INTACT SEDIMENTARY ROCKS AND TUFFS.

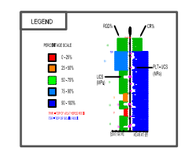
TYPE 2. HIGH SLOPES OF MODERATELY CONSOLIDATED INTACT SEDIMENTARY ROCKS AND TUFFS, ALTERED BUT STILL SOLID VOLCANIC ROCKS.

TYPE 3. SOFT VOLCANIC SEDIMENTARY ROCKS, INCLUDING SOFT AGGLOMERATE, WITH PARTIAL CLAY SHALES, BUT RELATIVELY STABLE AS TO FORM HIGH MASSIVE HILLS.

TYPE 4. SOFT VOLCANIC SEDIMENTARY ROCKS, INCLUDING SOFT AGGLOMERATE, WITH FREQUENT DEVELOPMENT OF CLAY SHALES AND TECHNICALLY HIGHLY DISTORTED AND SHEARED, LOCATED IN A LARGE SLIDE-AFFECTED ZONE.

TYPE 5. UNCONSOLIDATED DEPOSITS.

SOURCE: CANAL ALTERNATIVE STUDY - VOLUME IV (1993)



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Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
PACIFICO ALINEAMIENTO P1A ; PERFIL 1 (ESTE)
SECCION LONGITUDENAL MOSTRANDO
TRAZADO DE CAPAS, ROCA, UCS, y PLT

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A	01-03-02	DISEÑO INICIAL				
REV.	FECHA	DESCRIPCIÓN	DIB.	REV.	APR.	APP.

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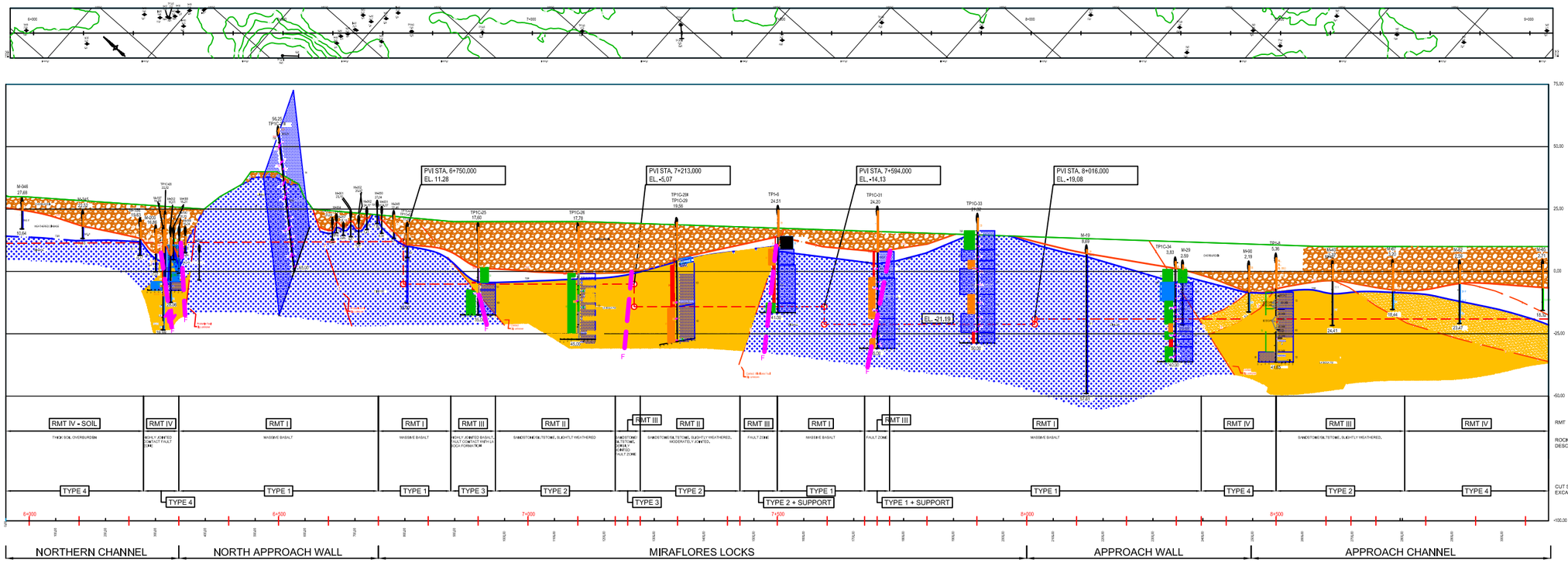

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GEOLOGICAL PROFILE OF CENTER WALL REVISED BY CONSORCIO POST PANAMAX (CPP)
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DESCRIPCIÓN DE LOS CRITERIOS:

TYPE 1: STRONG, UNALTERED OR SLIGHTLY ALTERED VOLCANIC ROCKS, (LIS, BASALT, AND HARD AGGLOMERATE LIMESTONE, SUPPORT NOT EXCEEDING 50 METERS IN HEIGHT OF MODERATELY COMPOSUDATED, INTACT SEDIMENTARY ROCKS AND TUFFS.).

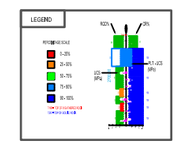
TYPE 2: HIGH SLOPES OF MODERATELY COMPOSUDATED INTACT SEDIMENTARY ROCKS AND TUFFS, ALTERED BUT STILL SOLID VOLCANIC ROCKS.

TYPE 3: SOFT VOLCANIC SEDIMENTARY ROCKS, INCLUDING SOFT AGGLOMERATE, WITH PARTIAL CLAY SHALES, BUT RELATIVELY STABLE AS TO FORM HIGH MASSIVE HILLS.

TYPE 4: SOFT VOLCANIC SEDIMENTARY ROCKS, INCLUDING SOFT AGGLOMERATE, WITH FREQUENT DEVELOPMENT OF CLAY SHALES AND TECTONICALLY HIGHLY DISTORTED AND SHEARED, LOCATED IN A LANDSLIDE-AFFLICTED ZONE.

TYPE 5: UNCONSOLIDATED DEPOSITS.

SOURCE: CANAL ALTERNATIVE STUDY - VOLUME II (1995)



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Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
PACIFICO ALINEAMIENTO P1A : PERFIL 2 (CENTRO)
SECCION LONGITUDENAL MOSTRANDO
VALORES DE CPS, RQD%, GCS, y PLT

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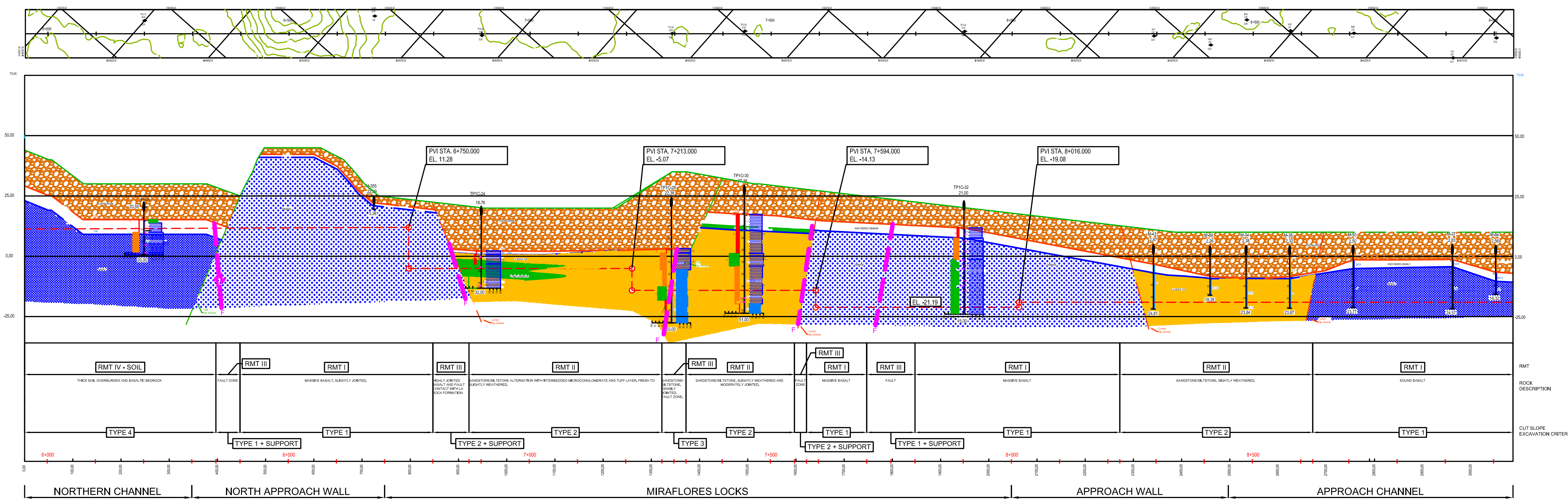

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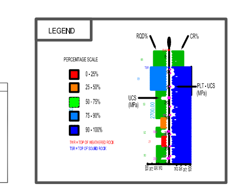
TYPE 5. UNCONSOLIDATED DEPOSITS.

SOURCE: CANAL ALTERNATIVE STUDY - VOLUME IV (1993)

LEYENDA

- BASALTO/BASALT
- DIABASA/DIABASE
- SUELO RESIDUAL/OVERBURDEN
- FORMACION LA BOCA/LA BOCA FORMATION
- ZONA DE FALLA/FAULT ZONE
- TWR INICIO DE ROCA METEORIZADA/TOP OF WEATHERED ROCK
- TSR INICIO DE ROCA SANA/TOP OF SOUND ROCK
- PISO DE ESCLUSAS PROPUESTO/PROPOSED LOCKS FLOOR
- NOMBRE DEL POZO/NAME OF BORING, ELEVACION/ELEVATION
- PROF. DEL POZO/LENGTH OF BORING
- CONTACTO/CONTACT
- FALLA/FAULT
- FALLA PROBABLE/PROBABLE FAULT



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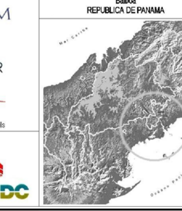

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Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
PACIFICO ALINEAMIENTO P1A : PERFIL 3 (OESTE)
SECCION LONGITUDENAL MOSTRANDO
VALORES DE CR%, RQD%, UCS, y PLT

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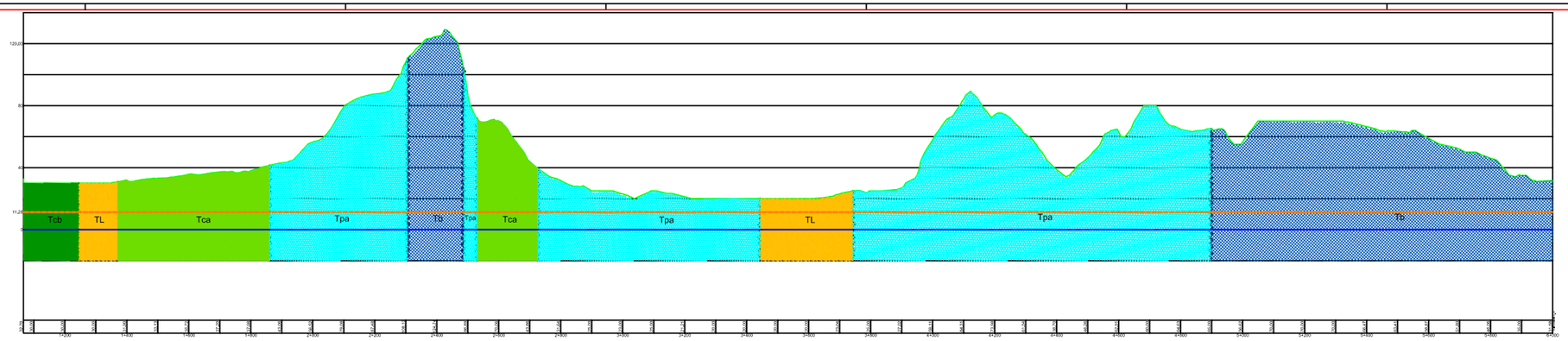

ALIGNMENT 1 - NORTH SECTION - CENTER LINE PROFILE
SCALE H: 1:2000 V: 1:800

Simbología / Symbols

Tb — ***Basaltos y Diabasa***
Basalto, intrusivo y extrusivo, Mioceno medio y superior
(Intrusive and extrusive basalt, middle and late Miocene)

Tpa — ***Pedro Miguel***
Formación Pedro Miguel, Mioceno inferior, Aglomerado, grano-fino a grueso duro
(Pedro Miguel Formation, early Miocene hard, fine to coarse-grained agglomerate)

TL — ***La Boca***
Formación La Boca, Mioceno inferior. Esquito arcilloso, lutita, areniscas, toba y caliza, terroso.
(La Boca Formation, early Miocene soft, Mudstone, siltstone, sandstone, tuff and limestone)

Tca — ***Cucaracha***
Formación Cucaracha, Mioceno inferior. Arcilla laminada bentonítica, arcilla laminada carbonífera y en la parte inferior, una capa delgada de lignítrita dura
(Cucaracha Formation, early Miocene, soft Bentonite clay shale, carbonaceous clay shale and in lower part, a thin sub lignite hard)

Tcb — ***Culebra***
Formación Culebra Mioceno inferior. Arenisca calcárea y lutita calcárea, suave
(Culebra Formation, early Miocene soft, Calcareous sandstone and siltstone)

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TRIPLE LIFT CONFIGURATION
ALINEAMIENTO P1A : PERFILES GEOLÓGICOS
ALIGNMENT P1A : GEOLOGICAL PROFILES

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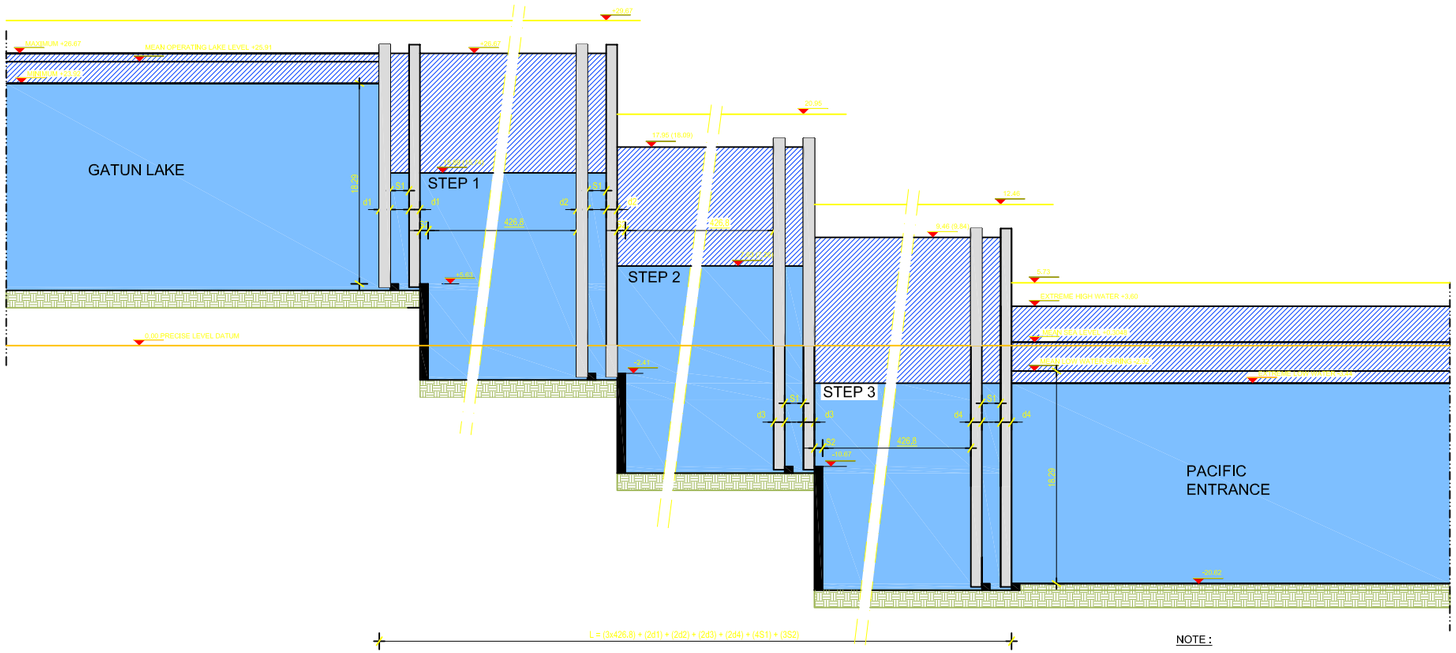

NOTE :

* WATER LEVELS AND SILL LEVELS ARE CALCULATED FOR MEAN LOW WATER SPRING -2.32 PLD
* WATER LEVELS SHOWN BETWEEN BRACKETS CORRESPOND WITH EXTREME LOW WATER SPRING (-3.44 PLD)

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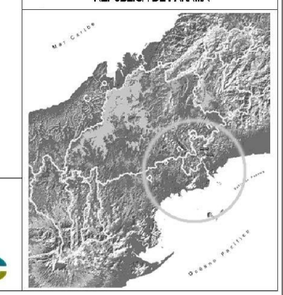

Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE-LIFT CONFIGURATION
LEVELS IN THE LOCK CHAMBERS

REV.	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
F					
E					
D	12/11/02	FINAL REPORT	BME	JDR	JDR
C	15/08/02	DRAFT FINAL REPORT	BME	JDR	JDR
B	05/06/02	LEVELS	BME	JDR	JDR
A	18/04/02	SECOND EDITION	JDV	TD	RT
–	28/03/02	FIRST EDITION	JDV	TD	RT

Contract	Scale:	Drawing n°:	Rev.:	Folio
SAA–97462	1/250	D2– 0 – 006	D	1 / 1

Diseño Conceptual de las Esclusas Post Panamax

TRIPLE LIFT LOCK SYSTEM

TASK 3 – ALIGNMENT OPTIMISATION

Tractebel Development Engineering
TECHNUM
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SBE
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contents

Executive Summary

1 Nautical Analisys

2 Geo-technical Optimization of the Alignment

3 Excavation Volumes

Executive Summary

The alignment optimization study for the third lock and by-pass canal at the Pacific Ocean side of the Panama Canal comprises three tasks :

- the nautical analysis of the access to the new locks,
- the analysis of the local geo-technical conditions,
- the optimization of the excavation volumes.

Each of these tasks is fully reported in this document and reference is also made to other parts of the draft final report of the three step lock configuration.

The results of the alignment study have been reached according to the following methodology :

1) the geo-technical analysis of the area performed by the consultant's expert showed out that the triple lift lock structure in the P1A alignment (drawing D3-0-001) as it was retained by ACP in the Terms of Reference, is partly situated in sound Basalt rock formations, and partly in the "La Boca" formation, which can be considered as a softer rock formation with bearing characteristics which are of an order of magnitude 5 times lower than the corresponding Basalt characteristics. Therefore it was suggested to **shift the entire lock structure to the south-east**, where the Basalt formation is sufficiently extended to locate the entire lock structure. This resulted in a new alignment "P1B" as shown on the drawing D3-0-002.
2) the P1B alignment seemed also to be interesting in order to minimize the excavation volumes, which may be considered as very important in the lock project as they represent an important part of the investment cost. Therefore the excavation volumes for both alignments were quantified using the software MX. The results show that the excavation volumes are indeed quite lower for the P1B alignment, **showing 62million cubic meters instead of 77million for P1A.**
3) as it is obvious that the nautical access for Post Panamax vessels to the locks has to be designed according to maximum safety for the ships as well as maximum performance of the locks, both alignments had to be analyzed. The nautical analysis was undertaken by the Hydraulics Laboratory at Borgerhout, using a fast time simulator model. Although this model only deals with tug boat assistance in a simplified way, the model can be considered as sufficiently useful to undertake the nautical analysis. The results of this study show that the P1A alignment can be accepted as a very reliable approach to the locks, while **the P1B alignment has to be rejected** because of the locks are too close to the existing approach channel to the Miraflores locks, which have to be kept in operation together with the new lock.

It was decided to retain P1A as the most suitable alignment, although it is clear that the geo-technical conditions are less favorable, and that 15millions of cubic meters more excavation will be required than with the P1B alignment.
It is recommended to undertake additional geo-technical and seismic investigation to locate exactly the different strata and faults in the lock area.

1	**Nautical Analysis**	**1-1**
1.1	**INTRODUCTION**	**1-1**
1.1.1	*PROBLEM DEFINITION*	*1-1*
1.1.2	*METHOD*	*1-2*
1.2	**DEVELOPMENT OF A MATHEMATICAL MODEL FOR A POST-PANAMAX CONTAINER SHIP**	**1-3**
1.2.1	*CHOICE OF THE DESIGN SHIP*	*1-3*
1.2.2	*SCALING OF A POST-PANAMAX CONTAINER SHIP*	*1-3*
1.2.3	*VALIDATION OF HYDRODYNAMIC COEFFICIENTS BASED ON CAPTIVE MODEL TESTS*	*1-4*
1.2.4	*TRIAL MANOEUVRES IN SHALLOW WATER*	*1-5*
1.2.5	*SUMMARY*	*1-5*
1.3	**FAST-TIME SIMULATION: DATA INPUT**	**1-6**
1.3.1	*FAST-TIME SIMULATION: CHARACTERISTICS OF THE AUTOPILOT*	*1-6*
1.3.2	*LAY-OUT OF THE CHANNEL ALIGNMENT*	*1-6*
1.3.3	*WATER DEPTH*	*1-7*
1.3.4	*BANK SUCTION*	*1-7*
1.3.5	*CURRENT*	*1-8*
1.3.6	*WIND*	*1-9*
1.3.7	*INVESTIGATED CONDITIONS*	*1-9*
1.4	**EVALUATION OF THE SIMULATION RUNS**	**1-12**
1.4.1	*DETERMINATION OF THE STOPPING LENGTH*	*1-12*
1.4.2	*EVALUATION METHOD*	*1-13*
1.4.3	*SAILING TO THE LOCK FROM THE PACIFIC OCEAN*	*1-13*
1.4.4	*LEAVING OF THE LOCK ENTRANCE FOR THE PACIFIC OCEAN*	*1-17*
1.4.5	*SAILING TO THE LOCK FROM GATÚN LAKE*	*1-18*
1.5	**EVALUATION OF CHANNEL ALIGNMENT P1B**	**1-21**
1.5.1	*INTRODUCTION*	*1-21*
1.5.2	*EVALUATION BASED ON THE CONCLUSIONS OF CHANNEL ALIGNMENT P1A*	*1-21*
1.5.3	*DESIGN RULES AND OTHER CONSIDERATIONS*	*1-22*
1.5.4	*CONCLUSION*	*1-22*
1.6	**GENERAL CONCLUSIONS**	**1-23**
TABLES		
FIGURES		
2	**Geo-technical analysis**	**2-1**
2.1	**INTRODUCTION.**	**2-1**
2.2	**GEO-TECHNICAL ANALYSIS.**	**2-1**

3	**Excavation Volumes**	**3-1**
3.1	**INTRODUCTION**	**3-1**
3.2	**DATA**	**3-2**
3.2.1	*SURVEY INFORMATION*	*3-2*
3.2.2	*CANAL/LOCK POSITION*	*3-2*
3.2.3	*GEOLOGICAL INFORMATION*	*3-3*
3.3	**SOFTWARE**	**3-4**
3.4	**METHODOLOGY**	**3-5**
3.4.1	*GENERAL*	*3-5*
3.4.2	*SUB-VOLUMES*	*3-5*
3.4.3	*CANAL SLOPES*	*3-6*
3.5	**RESULTS**	**3-9**
3.5.1	*P1A*	*3-9*
3.5.2	*P1B*	*3-11*
3.6	**COST ESTIMATE**	**3-12**

1 Nautical Analysis

1.1 INTRODUCTION

For the proposed post-panamax locks on the Pacific side of the Panama Canal, a by-pass of the San Pedro Miguel and Miraflores locks has been designed in an earlier study for the Panama Canal Authority (see fig 6). The chosen alignment P1A needs to be validated on its nautical merits as part of a global evaluation and comparison with an alternative alignment P1B that is designed during this conceptual study.

1.1.1 PROBLEM DEFINITION

Aim is the evaluation of the proposed lock channel alignment (channel dimensions and position of the locks) for four possible manoeuvres with the design ship (container ship or bulk carrier 385.7 m long, 54.9 m wide and 15.2 m draught):

- Entering from Port of Balboa and stop in front of the lock
- Entering from the Cucaracha Reach and stop in front of the lock
- Leaving the lock channel for the Pacific
- Leaving the lock channel for Gatún Lake

1.1.2 METHOD

In this phase of study a detailed analysis of the actual entering or leaving of the locks is not intended. The complications caused by the use of locomotives or by the density currents are out of scope. The proper entrance manoeuvres require a further modelling of a large set of forces and effects involved.

Therefore, an evaluation of the behaviour of the ship along its approach to the locks with respect to the required deceleration and the amount of control forces involved is performed with the use of fast-time simulation. The use of an autopilot is of course a significant simplification of the human capability for ship handling. The aim however is to stay on the safe side and show how little control is needed for the manoeuvres. Relying on tug assistance as only mean to manoeuvre the ship has not been considered.

The following steps were taken during this nautical study:

- Choice of design ship;
- Short literature study about next generation container ships;
- Deriving mathematical ship model for design ship;
- Validation of manoeuvring characteristics or ship forces with measurements from the towing tank;
- Selecting conditions;
- Preparing input data;
- Making fast-time simulations for the conditions;
- Evaluation of simulations with respect to
  - Engine use, rudder use
  - Position, speed and drift
  - Swept path
  - Stopping length
- Formulating conclusions and recommendations

1.2 DEVELOPMENT OF A MATHEMATICAL MODEL FOR A POST-PANAMAX CONTAINER SHIP

1.2.1 CHOICE OF THE DESIGN SHIP

The terms of reference mention a ship with dimensions LxBxT = 385.7m x 54.9m x 15.2m, a container ship and a bulk carrier.

The use of a container ship was proposed and accepted for the following reasons:

- With a service speed of 25 knots, a container ship will have a speed of 8.5 knots at Dead Slow in deep water and a speed of 7.5 knots at Dead Slow in shallow water (15% UKC).
- Therefore, the propulsion cannot be used continuously in a canal where a speed limit of 6 knots or 4 knots is imposed, or where speed has to be reduced in the approach of a lock.
- On the other hand bulk carriers have service speeds of about 14 knots, the corresponding speed at Dead Slow is around 4 knots in open water, even less with 15% UKC.
- Therefore, for the container ship it is more difficult to manoeuvre while keeping a low speed because without propeller jet the rudder generates only a small lift force.
- The most important external force is the wind. As the lateral surface of the container is much larger than that of a bulk carrier of the same length, the external forces will be more disturbing.
- At the meeting with ACP in Antwerp on April 24th 2002, it was agreed to simulate the container ship. However, it should have a single propeller arrangement.

1.2.2 SCALING OF A POST-PANAMAX CONTAINER SHIP

The development of a mathematical manoeuvring model for a post-panamax container ship is based on limited data delivered by the Panama Canal Authority (ACP):

- Main geometrical values of the container ship: length over all 385.7 m, beam 54.9 m, draught 15.2 m;
- Estimated Dead Weight Tonnage of 105 000 ton;

- Container capacity of 12 500 TEU's;
- Single engine – single propeller arrangement;
- Minimal under keel clearance of 15% of the ship's draught.

The mathematical model of the post-panamax container ship is obtained after scaling an existing mathematical model of a Panamax sized container ship with geometrical and full-scale trial data in deep water summarised in table 1. Mathematical manoeuvring models are available at under keel clearances of 20% and 100%. Under keel clearances in-between are obtained by interpolation between these values. This interpolation is linear with the factor $\frac{h}{h-T}$, h the water depth and T the ship's draught.

This Panamax sized ship, named CON_A, is scaled to a post-panamax sized ship, and named CON_PP, which general characteristics are summarised in table 2.

The rudder area is scaled to an area of 104.6 m^2. This means 1.78% of the lateral underwater area (L_{OA}.T).

Based on a literature study there's no certainty about the engine-propeller arrangement for this non-existing container ship [1 – 4].

The container ship is powered by a single-engine single-propeller arrangement with a minimum propeller rate of 30 rpm. The propeller is fixed. Depending on the required service speed and service margin (10% to 20%) the required power has been estimated to be about 100 000 kW (table 3).

At 20% service margin, the service speed has to be reached during a trial manoeuvre with 80% of the engine power. The estimated maximum continuous rating (MCR) of the engine is 94 rpm. At 20% service margin, the propeller rate is 87 rpm at sea full condition.

The characteristics of the largest existing container vessels according to TEU's are summarised in table 4.

1.2.3 VALIDATION OF HYDRODYNAMIC COEFFICIENTS BASED ON CAPTIVE MODEL TESTS

The hydrodynamic coefficients of the mathematical manoeuvring model are validated based on captive model tests executed in the Towing Tank for manoeuvres in Shallow Water (Co-operation Flanders Hydraulics – Ghent University, Antwerp, Belgium) with a ship model of a post-panamax container ship with characteristics summarised in table 5. This ship model is named D518. The scale factor is 1/75. Ship model length, beam and draught can be approximately scaled to the post-panamax container ship CON_PP with the scale factor of the draught, 1/98.25.

Captive model tests had been performed with ship model D518 in shallow water with 15% under keel clearance. These measurements have been used to validate forces predicted by the mathematical model obtained after scaling of ship CON_A to CON_PP.

Based on the measured forces and moment at lower speed ranges (2 to 5.8 knots) a few coefficients of the scaled mathematical model (originally for 20% UKC) have been improved to meet the experimental results at 15% UKC.

Figures 1, 2 and 3 compare forces and yawing moment measured during a multimodal captive model test with a drift angle of –5 deg at a full scale speed of 2 knots (varying propeller rate and rudder angle) with the forces and yawing moment predicted by the validated mathematical model.

Remark: The influence of squat is incorporated in the coefficients, as the ship model is free to sink during the captive model tests.

1.2.4 TRIAL MANOEUVRES IN SHALLOW WATER

The manoeuvrability of the post-panamax container ship CON_PP at an under keel clearance of 15% is illustrated based on full-scale trial manoeuvres in open, unrestricted water.

Acceleration manoeuvres for the post-panamax container ship CON_PP at an under keel clearance of 15% are summarised in table 6. The initial velocity is zero. At Dead Slow telegraph, the ship still reaches a speed value of 7.4 knots.

A crash stop manoeuvre starting at Dead Slow velocity is summarised in table 7.

Turning circle characteristics at half-ahead speed (0.89 rps and 6.85 m/s) for a starboard turn of 35 deg rudder angle are shown in table 8. The tactical diameter at 15% UKC is 9.87 times the overall length of the container ship (figure 4).

Due to the important resistance at 15% under keel clearance the overshoot angle during a 20/20 zigzag manoeuvre at Dead Slow velocity is restricted to 1.4 deg (table 9).

1.2.5 SUMMARY

The scaled and validated post-panamax container ship is a look-alike container ship with a container capacity of 12 500 TEU's. The developed mathematical model predicts the ship's manoeuvrability at an under keel clearance of 15% and 100% of the ship's draught. Only the mathematical model at 15% UKC is validated based on captive model tests at low speed ranges.

1.3 FAST-TIME SIMULATION: DATA INPUT

1.3.1 FAST-TIME SIMULATION: CHARACTERISTICS OF THE AUTOPILOT

During a fast-time simulation, the ship is steered by an autopilot instead of a human pilot. The autopilot used at Flanders Hydraulics is not a PID controller. In stead, the autopilot is a coarse version of the mathematical model, used to predict the ship's position after a short lapse of time for different rudder angles and propeller telegraphs. This method is used to include some of the real pilots experience in predicting effects of bank suction and other local effects. The optimum rudder command is the one that minimises the cost function of the ships predicted position (figure 5):

$$\text{Cost} = a\left(\text{Offset}_{\text{bow}}\right)^2 + b\left(\text{Offset}_{\text{amidships}}\right)^2 + c\left(\text{Offset}_{\text{stern}}\right)^2 \qquad (3\text{-}1)$$

The ship's position is evaluated based on the offsets at bow, amidships and stern according to the trajectory that has to be followed by the ship.

The weighted coefficients a, b and c in expression (3-1) are tuned during the validation of the autopilot. Depending on the value of coefficient b in comparison with the values a and c the lateral deviation (coefficient b) of the ship according to the trajectory is more or less important than the angular deviation (coefficients a and c) of the ship according to the trajectory course. The validated coefficients are a = 1, b = 5 and c = 1.

Rudder commands are tested every 10 seconds and telegraph commands are tested every 20 seconds and changed with discreet steps (5 or 10° rudder, a telegraph order) as to mimic a more realistic behaviour than a continuous stream of commands. The combined rudder and telegraph command that minimises the cost function is invoked during the next 10 seconds. The predicted position of the ship is the position after 100 seconds. For a ship travelling at a speed of 2.5 m/s this means a prediction distance of 250 m or 0.65 times the overall length.

Along the trajectory, speed limits can be imposed. Bow thruster and tugboats aren't steered automatically but a constant force and moment executed by bow thruster and one or more tugboats can be imposed during a chosen part of the trajectory.

1.3.2 LAY-OUT OF THE CHANNEL ALIGNMENT

The study will focus on the manoeuvres along the newly developed channel to the lock with post-panamax sizes, from the Pacific Ocean and from the side of Gatún Lake (figure 6). The lock alignment P1A will be evaluated first. This alignment corresponds to drawing D3–0–001.

The bottom width of the channel is 220 m or four times the ship's beam. The bank slope is 45 deg or 1:1. It should be noted that the bank slopes will most probably be flatter in practice. As far as the ship's movement is concerned it will be less difficult because bank suction decreases with flatter slopes.

The simulated manoeuvre sailing to the lock from the Pacific Ocean starts approximately 1.5 km for the Port of Balboa and stops in front of the guiding wall of the lock. The trajectory that has to be followed by the ship is illustrated in figure 7. The distance parameter along the trajectory is indicated on this figure. The manoeuvre to the lock consists of a zigzag manoeuvre with consecutively a turn to starboard (entering Port of Balboa) and a turn to port (entering the by-pass). The stopping length after the latest turning manoeuvre is approximately three times the overall ship length.

The manoeuvre from the lock to the Pacific Ocean starts at one ship length from the guiding wall and stops at the moment the ship leaves the Port of Balboa (figure 8).

Sailing to the lock from Gatún Lake starts in the Culebra Reach and stops in front of the lock. The trajectory that has to be followed is illustrated in figure 9. The manoeuvre consists of two consecutive turning manoeuvres, to port and to starboard and a long decelerating manoeuvre to the lock (more than 11 ship's lengths available).

1.3.3 WATER DEPTH

In the Culebra and Cucaracha Reach (Gatún Lake) the available under keel clearance will be 15% of the ship's draught. The water depth is consequently 17.48 m.

At the Pacific Ocean the water depth depends upon the tidal level. At low water the available under keel clearance is still 15% of the ship's draught. At maximum flood current (half tide) the water depth is increased with 3.42 m. The available under keel clearance at half tide is 37.5% of the ship's draught. At maximum ebb current (half tide – 0.6 m) the water depth is 20.3 m. The available under keel clearance is 33.6% of the ship's draught.

The channel bottom is supposed to be flat.

1.3.4 BANK SUCTION

Channel banks influence ship manoeuvrability in two ways:

- A ship travelling parallel to a bank undergoes a lateral attracting force to the bank and a bow away yawing moment; when the distance from fore and aft body of the ship to the starboard and port bank is the same, the influences of the starboard and port banks counteract.

- A ship, travelling in the middle of a channel with a certain drift angle, undergoes an increased lateral swaying force and yawing moment depending on the channel width and under keel clearance.

Bank suction coefficients for vertical banks are obtained according to Norrbin [5]:

$$Y = rC_B BTu^2 h_0 \left[0.0926 + 0.372\left(\frac{T}{h}\right)^2\right]$$
$$N = -rC_B LBTu^2 h_0 \left[0.0025 + 0.0755\left(\frac{T}{h}\right)^2\right] \qquad (3\text{-}2)$$

The bank slope of 45 deg is incorporated in the formulations (3-2) by measuring bank distances at half ship draught. Distances from the axis of the channel to the starboard and port banks used for this calculation are shown in figure 10 for the Pacific Ocean side and in figure 11 for the Gatún Lake side.

The channel width has also an influence on the linear hydrodynamic coefficients Y_β and N_β, which represent the effect of drift on the lateral force and yawing moment. An estimation of this influence of the restricted width is obtained based on the measurements for a Mariner type scale model described by Fujino in [6].

The ratios of the hydrodynamic coefficients $Y_{\beta W}/ Y_\beta$ and $N_{\beta W}/ N_\beta$ are known from these measurements for three water depth to draft ratios (h/T = 1.2, 1.5, 1.9) and for three canal width to beam ratios W/B.

The ratios at 15% under keel clearance are obtained after extrapolation (figure 12).

1.3.5 CURRENT

At the Pacific Ocean and in the Port of Balboa a tidal current occurs with a maximum of 0.75 knots at rising tide and a maximum of 0.5 knots at outgoing tide (pilot handbook of Panama Canal [7]).

The worst case while entering the lock from the Pacific Ocean is a flood current (figure 13). While leaving the lock for the Pacific Ocean the ebb current diminishes the ship velocity relative to the water.

The detailed current flow pattern is not known and is replaced by a simplified uniform flow pattern with strongly decreasing flow velocities in the by-pass.

1.3.6 WIND

Due to the height of the container rows placed on deck, the estimated lateral wind area of the post-panamax container ship of 10000 m² is impressive (table 2). At low ship's speed wind forces will dominate the ship's motion.

The synoptic mean wind over a measuring period of 10 minutes is the wind summarised in the condition table. The fluctuations of wind speed and wind direction around this mean wind are simulated according to a Von Karman spectrum.

Based on information given by the Panama Canal Authority (ACP) the wind conditions for the simulation study are selected. Monthly averaged wind speeds (table 10) and monthly maximum wind speeds over a period of 20 seconds (table 11) are given for the period 1985 to 2001. Monthly averaged wind directions are summarised in table 12. Coded hourly averaged wind data were used to select the hourly maximum wind speed.

According to the definition for the wind direction given by the ACP (figure 14) the dominating wind direction is situated between North and Northwest (figure 15). Wind influences coming from Southwest to South have minor speed values. Wind from East is rather unusual based on the averaged wind directions.

The simulation will not be based on the maximum wind speeds. The maximum monthly averaged wind speed (6.8 mph) corresponds to the upper range of the 2 Beaufort wind class. Therefore, the *3 Beaufort wind* class (mean value 4.4 m/s or 9.8 mph) is used as minimum wind speed condition. The wind direction is varied from North to South over West.

The hourly year maximum is reached in 1987 and reaches 8.23 m/s. This is the lower range of the *5 Beaufort wind* class. This range is used as the maximum wind speed with a wind direction varying from North to West.

1.3.7 INVESTIGATED CONDITIONS

Fast-time simulation runs are executed for the following manoeuvres:

- Sailing from the Port of Balboa to the lock;
- Leaving the lock entrance for the Port of Balboa;

- Sailing to the lock from the Gatún Lake side.

The leaving of the lock entrance for Gatún Lake is not simulated as the manoeuvring through the bends can be evaluated based on the entrance manoeuvre to the lock from Gatún Lake.

1.3.7.1 Sailing to the lock from Port of Balboa

Speed limitations in the Panama Canal are necessary to reduce the ship's squat. In [7] the conclusions of a study of ship squat in the Panama Canal run as follows: "A steady speed of 6 knots or less with minimal engine speed changes should limit sinking to –2.0 feet. This speed limit may be more difficult for container ships to meet than other ships because of their tendency for higher speeds and could require careful ship handling by the pilots."

At 15% under keel clearance the ship velocity at Dead Slow telegraph in open, unrestricted water is 3.82 m/s or 7.4 knots. In restricted water (Panama Canal), this speed will be smaller. If the ship speed has to be limited to 6 knots in the Port of Balboa and the by-pass, the assistance of a tugboat at the stern is necessary. This tugboat is connected near the entrance of the Port of Balboa and can exert an additional longitudinal braking force with its 40 tonf thrust.

At distance parameter 9400 m (figure 7: three ship lengths to the guiding wall) the engine is stopped. If necessary, an additional thrust of 10 tonf is executed by the tugboat at the stern. To reduce the influence of the wind at low ship speed the bow thruster is used (at a propeller rate of 75% of the maximum rate of turn). One can interpret the requirement for the use of a bow thruster as the need to use a tug at the bow of the ship. The guiding wall starts at distance parameter 10545 m (figure 7).

Simulation runs are executed at low water and rising tide. The conditions are summarised in table 13.

Table 13. Conditions: sailing to the lock from Port of Balboa.

wind			tug assistance		stop engine	bow thruster	tide	current
direction	speed	Beaufort	thrust	from	from	from 9400 m		
Condition 1								
no	no	no	40 tonf	6600 m	9400 m	no	LW	no
no	no	no	40 tonf	6600 m	9400 m	no	+3.42 m, FLOOD	max.
Condition 2								
S	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
SW	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
N	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
NW	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
W	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
Condition 3								
N	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	+3.42 m, FLOOD	max.
NW	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
W	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
SW	4.4 m/s	3 Bf	40 tonf + 10 tonf	6600 m + 9400 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
S	4.4 m/s	3 Bf	40 tonf + 10 tonf	6600 m + 9400 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.

Condition 4								
N	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	LW	no
NNW	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	LW	no
NW	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	no	LW	no
W	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% P	LW	no
Condition 5								
N	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
NNW	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
W	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
NW	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.

1.3.7.2 Leaving of the lock entrance for Port of Balboa

The manoeuvre while leaving the lock entrance starts at one ship length from the guiding wall with a starting velocity of approximately 0.5 knot. During the simulation run no tug assistance is provided. By stopping the engine from time to time the container ship tries to handle its velocity to a maximum of 6 knots. The conditions are executed at outgoing tide (considered the worst condition) and are summarised in table 14.

Table 14. Conditions: leaving the lock entrance for Port of Balboa

wind			tug assistance		tide	current
direction	speed	Beaufort	thrust	from		
Condition 6						
N	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum
NW	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum
W	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum
SW	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum
S	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum

1.3.7.3 Sailing to the lock from Gatún Lake

The entrance of the by-pass from Gatún Lake starts in the Culebra Reach at a starting speed of 6 knots.
At 3 Bf wind no tug assistance is provided. A speed target of zero knots is imposed at a distance parameter of 8400 m. The guiding wall of the lock starts at distance parameter 8650 m (figure 9). The autopilot is reducing the ship's speed by stopping the engine from time to time.
At 5 Bf wind the engine is stopped at distance parameter 7500 m (three ship lengths from the guiding wall) and a tugboat at the stern assists the stopping manoeuvre. The bow thruster is not used.

Table 15. Conditions: sailing to the lock from Gatún Lake

wind			tug assistance		stop engine	bow thruster	current
direction	speed	Beaufort	thrust	from	from		
Condition 7							
no	no	no	40 tonf	7500 m	7500 m	no	no
no	no	no	no	no	no	no	no
Condition 8							
N	4.4 m/s	3 Bf	no	no	no	no	no
NW	4.4 m/s	3 Bf	no	no	no	no	no
W	4.4 m/s	3 Bf	no	no	no	no	no
S	4.4 m/s	3 Bf	no	no	no	no	no
SW	4.4 m/s	3 Bf	no	no	no	no	no
Condition 9							
N	9.4 m/s	5 Bf	40 tonf	7500 m	7500 m	no	no
NNW	9.4 m/s	5 Bf	40 tonf	7500 m	7500 m	no	no
NW	9.4 m/s	5 Bf	40 tonf	7500 m	7500 m	no	no

1.4 EVALUATION OF THE SIMULATION RUNS

1.4.1 DETERMINATION OF THE STOPPING LENGTH

The stopping length of the container ship takes part in deciding where the lock has to be positioned. Four stopping manoeuvres have been executed in an open, unrestricted area with 15% under keel clearance to determine the stopping length:

- Condition A: at a speed of 7.3 knots (corresponding to Dead Slow ahead) the ship is stopped by reversing engine to Dead Slow astern. The ship stops after 610 m, however the transfer is 31 m, considered to be too large in the approach of a lock as it has to be combined with other deviating influences.

- Condition B: at 4 knots, the engine is stopped and a tug pulls astern with a thrust of 40 tonf. The ship stops after 863 m, without transfer.

- Condition C starts at 5 knots, with the same tug assistance, the ship is stopped after 1189 m.

- Condition D starts at 6 knots, a stopping distance of 1517 m is needed.

The characteristics of the stopping manoeuvres are summarised in table 16.

Entering from the Pacific Ocean the available stopping length in the by-pass is about 1000 m. This means that the speed should be reduced or that additional tug assistance during the stopping manoeuvre is necessary (hence the extra 10 tonf in the simulations).

In the by-pass entering from Gatún Lake the available distance for stopping the ship is largely sufficient. Tug assistance is supposed to be kept minimal during the stopping manoeuvre at moderate wind conditions.

1.4.2 EVALUATION METHOD

The different simulation runs summarised in table 13, 14 and 15 are evaluated based on:

- The swept path;
- The lateral and angular deviation along the trajectory;
- The longitudinal ship velocity;
- The use of rudder and propeller along the trajectory.

Track plots for the simulation runs are shown in appendix A, in the order of tables 13, 14 and 15. Distances of one, two and three ship lengths are indicated on the track plots and on figure 16 and following.

1.4.3 SAILING TO THE LOCK FROM THE PACIFIC OCEAN

1.4.3.1 Test condition 1: Low water versus rising tide, no wind

At low water the hydrodynamic resistance of the under water body stops the turning manoeuvre approximately in front of the guiding wall. Lateral and angular deviations are small one ship length from the guiding wall, 2.5 m and 0.6 deg.

The lateral deviation and the angular deviation are larger during rising tide compared to the low water condition (figure 16a). Two factors explain this difference:

- Due to the larger under keel clearance at rising tide (37.5%) compared to low water (15%) the hydrodynamic resistance is smaller so the turning motion to port side at the entrance of the by-pass can not be arrested without additional tug assistance at the bow or use of the bow thruster.

- At half tide the flood current enhances the turning motion at the entrance of the by-pass. As the engine is stopped three ship lengths from the guiding wall, the ship is not able to reduce this motion by use of the rudder. The use of a bow thruster and an additional tugboat at the bow are necessary.

Swept path lines along the trajectory are shown on figure 17.

On figure 16b, the measured ship velocity is given as a function of the position amidships. The entering manoeuvre from the Pacific Ocean starts at 6 knots. The desired ship velocity entering the by-pass is 4 knots. This desired ship velocity is not reached. Three ship lengths from the guiding wall the ship velocity is approximately 5 knots at low water and rising tide. At low water, the ship is stopped in front of the lock. At rising tide due to the lower ship resistance, in front of the guiding wall the ship's velocity is 1.5 knots.

Rudder angle commands and propeller commands are indicated on figure 16c. A rudder command to port is positive. At low water, the telegraph is Dead Slow during an important period of the trajectory. Although a tugboat is connected at the stern from distance parameter 6600 m, at rising tide the engine has to be stopped from time to time to reduce the ship's speed. Engine and rudder commands are minimal during straight manoeuvres. At the entrance of the Port of Balboa and the entrance of the by-pass a telegraph of Dead Slow and an increasing rudder angle are needed. If the engine is stopped (for example from distance parameter 9400 m), the rudder angle from hard starboard to port is of no importance, as the rudder is no longer placed in the slipstream of the propeller. Rudder forces and moment are very small.

1.4.3.2 Test condition 2: Low water and 3 Bf wind condition

Entering the Port of Balboa the lateral deviation is maximal to port at a wind condition northwest (-16 m) (figure 18a). Entering the by-pass to the new lock, the lateral deviation is maximal to starboard at westerly and southerly wind (20 m). In front of the guiding wall the lateral deviation is maximum 5 m. When the midship is two ship lengths away from the guiding wall, the angular deviation is maximal at a westerly wind (-4 deg). One ship length away from the guiding wall the angular deviation is small. If the ship velocity becomes very small to zero, the angular deviation increases very quickly. Depending on the wind direction tugboats at the bow and the stern are necessary to keep up the ship in the wind. During the simulation runs at test condition 2 the bow thruster was not used.

Swept path lines along the trajectory are shown on figure 19.

The ship is stopped in front of the guiding wall for all wind directions (figure 18b). At three ship lengths away from the guiding wall, the ship's velocity is 4.5 to 5 knots.

Maximum rudder angles are needed at the entrance of the port of Balboa (starboard) and at the entrance of the by-pass (port) (figure 18c). The minimal telegraph is Dead Slow, sometimes for a very short period during the turning manoeuvres increased to Slow.

1.4.3.3 Test condition 3: Rising tide and 3 Bf wind condition

As the ship resistance decreases with increasing under keel clearance, the hydrodynamic resistance of the under water body cannot resist the influence of the wind. To reduce the turning manoeuvre to port at the entrance of the by-pass the bow thruster is used to starboard from distance parameter 9400 m (at this parameter the engine is stopped). The effective lateral force and yawing moment exerted by the bow thruster depend upon the ship's velocity (figure 20). At zero ship's velocity, the effectiveness is 100%. The bow thruster is not handled by the autopilot. From distance parameter 9400 m to the end of the manoeuvre in front of the guiding wall the propeller rate of the bow thruster is set to 75% of the maximum propeller rate of turn. At the end of the manoeuvre when the ship's speed is small, this setting is exaggerated.
Passing the Port of Balboa the lateral deviation is maximal at northerly wind (-17 m) (figure 21a). A three ship's length away from the guiding wall the lateral deviation is maximum 17 m (to starboard). Two ship lengths from the guiding wall the deviation is diminished to a value between 0 and –5 m. To stop the turning manoeuvre and keep up the ship in the wind at velocities lower than 2 knots additional tug assistance at the bow and the stern is necessary. The angular deviation is small until the ship enters the by-pass. Depending on the wind direction two ship lengths away from the guiding wall the angular deviation varies between –1 deg and –3.5 deg, decreasing to zero between one and two ship lengths from the guiding wall. Under the influence of the bow thruster (to starboard) the angular deviation increases very quickly to starboard if the ship's velocity becomes lower than 2 knots. The correction of the bow thruster should be smaller.

Swept path lines along the trajectory are shown on figure 22.

One ship's length away from the guiding wall the ship's velocity is decreased to 1 or 2 knots depending on the wind direction. The ship cannot be stopped in front of the guiding wall for all wind directions (figure 21b).

The engine varies continuously between Dead Slow and stopped propeller to reduce the ship's speed (figure 21c). At stopped engine the rudder angle is hard starboard or port. At Dead Slow the rudder angle is restricted.

1.4.3.4 Test condition 4: Low water and 5 Bf wind condition

Leaving the Port of Balboa the lateral deviation reaches a maximum value of –30 m at northerly wind condition (figure 23a). Three ship lengths away from the guiding wall the maximum starboard deviation is 15 m at north-westerly and westerly wind condition. The course angle of the lock corresponds approximately to a north-westerly direction. The lateral deviation when the ship is

stopped is small for a north-westerly (-4 m) and a north-north-westerly wind (+4 m). At zero ship's velocity and no tug assistance for the wind (only a tugboat at the stern to reduce the ship's speed) the lateral deviation increases very quickly at a northerly and westerly wind.

Two ship lengths away from the guiding wall the angular deviation is almost zero for a northerly and north-north-westerly wind. For a northwesterly and a westerly wind the angular deviation is approximately –2 deg.

Swept path lines along the trajectory are shown on figure 24.

To keep up the ship in the wind at a 5 Bf strong cross wind and zero ship velocity a lateral force of more than 40 tonf is needed. The tugboat that assists the stopping manoeuvre by delivering a longitudinal resistance force cannot exert this lateral force at the same time. Additional tugboats have to assist the approaching manoeuvre to the guiding wall.

For all wind directions the ship is stopped in front of the guiding wall thanks to the assistance of a tugboat at the stern (40 tonf thrust) (figure 23b).

Maximum rudder angles are necessary during a short period at the entrance of the Port of Balboa and the entrance of the by-pass (figure 23c). The engine telegraph is almost continuously Dead Slow.

1.4.3.5 Test condition 5: Rising tide and 5 Bf wind condition

Zero lateral deviation is reached for a north-westerly and a north-north-westerly wind condition between one and two ship lengths away from the guiding wall (figure 25a). For a westerly wind, the lateral deviation is at the starboard side of the prescribed trajectory during the whole run. The deviation is minimal (12 m) at two ship lengths away from the guiding wall. Tug assistance to keep the ship in the wind is necessary. For a northerly wind the lateral deviation is zero between two and three ship lengths away from the guiding wall, increasing to port side at decreasing velocity. Additional tug assistance is necessary as well.
Depending on the wind direction a small angular deviation is reached with only a correction by the bow thruster at a distance of one or two ship lengths away from the guiding wall. The setting of the bow thruster is too high for low ship's velocity in a northerly, a north-north-westerly and a north-westerly wind condition.

Swept path lines along the trajectory are shown on figure 26.

One ship length away from the guiding wall the ship's velocity is maximum 2 knots for all wind directions (figure 25b).

The rudder command depends upon the engine command. This means hard rudder at stopped engine (figure 25c). The engine is stopped during a longer period for the Port of Balboa. Passing the Port of Balboa with a tugboat connected at the stern the engine is nevertheless stopped from time to time to reduce the ship's speed.

1.4.3.6 Conclusion

Based on the simulation runs executed for the entrance manoeuvre from the Pacific Ocean the following conclusions can be made:

- At all tested wind and tidal conditions the container ship can pass the Port of Balboa and enter the by-pass on its own. The lateral deviation is restricted to a distance of approximately half a ship's beam. Tug assistance at the stern is necessary to help reducing the ship's speed to minimise squat effects.

- The stopping length of the container ship depends upon the ship's velocity, the telegraph setting and the availability of tugboats. From the moment the engine is stopped or Dead Slow astern has been given, the own manoeuvrability of the ship is small as the rudder is no longer in the slipstream of the propeller.

- At moderate wind conditions and low water, thanks to the hydrodynamic resistance, the turning manoeuvre during the entrance of the by-pass can be stopped with minimal additional assistance of tugboats or bow thruster. The ship is stopped in front of the guiding wall.

- At rising tide, the available hydrodynamic resistance is reduced compared to low water condition. Additional tug assistance (or use of a bow thruster if present) in the by-pass is essential to stop the turning manoeuvre to port and to stop the container ship in front of the guiding wall. (The same conclusion will apply if the draught of the ship is significantly smaller than that of the design ship.)

- If the container ship has to be stopped in front of the guiding wall, tugboats and bow thruster have to keep the ship in position. For a crosswind of 5 Beaufort, the estimated lateral tug force is more than 40 tonf.

1.4.4 LEAVING OF THE LOCK ENTRANCE FOR THE PACIFIC OCEAN

1.4.4.1 Test condition 6: Outgoing tide and 3 Bf wind condition

The simulation runs are executed without tug assistance. The ship is trying to maintain its velocity to a maximum of 6 knots along the trajectory to reduce the ship's squat.

The lateral deviation is small during the whole simulation run (figure 27a). The absolute value of the lateral deviation is no more than 10 m except for a westerly wind. This deviation is caused by the reduced manoeuvrability at stopped engine. From the moment the ship leaves the Port of Balboa the ship moves to starboard (30 m) until the engine is used to produce the necessary rudder force to reduce the lateral deviation.

The angular deviation is small. As described for the lateral deviation, leaving the Port of Balboa the angular deviation varies quickly from 3.5 deg starboard to –4 deg port for a westerly wind condition.

Swept path lines along the trajectory are shown on figure 28.

The starting velocity is small (0.5 knots) (figure 27b). Two thousand meters away from the guiding wall of the lock the ship's velocity is 6 knots. Leaving the Port of Balboa the velocity reaches 6 to 7 knots for all wind directions.

Turning manoeuvres to starboard and port can be executed with rudder angles of –20 (starboard) and 20 deg (port). No hard over rudder command is necessary (figure 27c). The telegraph varies between Dead Slow and stopped engine.

1.4.4.2 Conclusion

Based on the simulation runs executed for the manoeuvre from the lock to the Pacific Ocean the following conclusions can be made:

- If the ship starts in the centre line of the channel, the swept path can be kept small during the whole trajectory to the Pacific Ocean, even without tug assistance.
- If a lower velocity is wanted during the passing of the Port of Balboa, a tugboat at the stern will be necessary.
- A tugboat connected at the stern makes it possible to keep the engine to a minimal telegraph of Dead Slow during the turning manoeuvres so that the ship's swept path can be restricted.

1.4.5 SAILING TO THE LOCK FROM GATÚN LAKE

1.4.5.1 Test condition 7: no wind, with or without tug assistance

The lateral and angular deviations during the entrance of the lock from Gatún Lake without wind are very small (figure 29a). Swept path lines along the trajectory are narrow (figure 30).

The manoeuvre starts in the Culebra Reach with a speed of 6 knots. From the distance parameter 3800 m, the ship's velocity decreases to a minimum of 1.5 knots due to a stopped engine (figure

29b). If the manoeuvre is executed without tug assistance, the ship's velocity varies from 1.5 knots to 2.5 knots approaching the guiding wall.

If the engine is stopped three ship lengths away from the guiding wall (distance parameter 7500 m) and a tug is connected at the stern (thrust of 40 tonf), the ship's velocity is decreasing very quickly. The ship could be stopped in front of the guiding wall with less tug force.

A maximum rudder angle of 30 deg is necessary for the turn from the Culebra Reach to the Cucaracha Reach (port turn) (figure 29c). The next starboard turn can be executed with a maximum rudder angle of –25 deg. Hard over rudder commands are given during a stopped engine command.

1.4.5.2 Test condition 8: 3 Bf wind condition, without tug assistance

All the simulation runs are executed without tug assistance. The container ship is trying to approach the guiding wall at moderate wind condition on its own. Lateral and angular deviations are very small during the whole trajectory (figure 31a). Swept path lines are shown on figure 32.

The starting speed is again 6 knots (figure 31b). The velocity in front of the guiding wall varies between 2.5 and 3 knots. The minimum velocity reached during the approaching manoeuvre to the guiding wall varies approximately between 1.5 and 2.25 knots. This means that the entrance of the lock is possible with a minimum of tug assistance if the container ship has not to be stopped in front of the guiding wall (taking into account the available ship's speed to connect to the locomotives).

During the turning manoeuvre to port the maximum rudder angle is 35 deg for a very short time period (figure 31c). Entering the by-pass the maximum rudder angle is –25 deg to starboard. From distance parameter 6300 m, the rudder command varies continuously depending on the engine command. The engine is stopped directly after the starboard turn (distance parameter 3900 m). From distance parameter 6200 m, the engine varies continuously between Dead Slow and stopped engine.

1.4.5.3 Test condition 9: 5 Bf wind condition, with tug assistance

At 5 Bf wind condition, the engine is stopped at a distance of three ship lengths away from the guiding wall. During the stopping manoeuvre the container ship is assisted by a tugboat connected at the stern (40 tonf thrust). The bow thruster is not used.

The lateral deviation is small until the engine is stopped (figure 33a). At a northwesterly wind, (orientation of the lock) the lateral deviation increases to a maximum port deviation of –12 m.

For wind conditions coming from north and north-northwest the lateral deviation becomes respectively –40 m and –26 m. The angular deviation varies in the same way as the lateral deviation. The angular deviation is small until the engine is stopped and additional tug assistance is needed to keep up the ship in the wind at low ship's velocities. Swept path lines along the trajectory are shown on figure 34.

Three ship lengths away from the guiding wall (at the moment the engine is stopped) the ship's velocity varies between 3.5 knots (northwest) and 4 knots (north) (figure 33b). During the approaching manoeuvre, a minimum velocity is reached of 2.5 knots for a northwesterly wind and 3 knots for a northerly and north-north-westerly wind. The ship can be stopped properly with less tug force at the stern, but additional tug assistance is needed to keep the ship in the wind.

A maximum rudder angle of 35 deg is reached during the turn to port and –30 deg during the turn to starboard (figure 33c). From distance parameter 4900 m until the end of the manoeuvre, the rudder command varies continuously.

The engine is stopped at 4000 m. From distance parameter 5000 to 7500 m the engine varies between stopped engine and Dead Slow. From distance parameter 7500 m, the engine is stopped to reduce the ship's speed to zero.

1.4.5.4 Conclusion

Based on the simulation runs executed for the entrance manoeuvre from Gatún Lake the following conclusions can be made:

- The entrance manoeuvre from the Culebra Reach to the by-pass should not cause any difficulty at moderate and even strong wind conditions.
- The available stopping length is approximately eleven times the overall ship length so that stopping the engine from time to time can reduce the ship's speed. Even at strong wind conditions, tug assistance can be kept minimal if the ship has not to be stopped in front of the guiding wall.
- At zero ship's velocity depending on the assistance method (pushing or towing) tugboats at the stern and the bow or amidships are necessary to keep the container ship in the wind.

1.5 EVALUATION OF CHANNEL ALIGNMENT P1B

1.5.1 INTRODUCTION

The alignment P1B is sketched in figure 35. The by-pass connects to the Balboa reach more to the Miraflores side than alignment P1A. The distance between the two bends is larger than in alignment P1A.

The main nautical characteristic that makes the difference between both alignments is the distance between the lock quay wall and the existing Balboa reach.

Alignment	P1A (figure 8)	P1B (figure 35)
Distance between axis Balboa reach and front of guiding wall	1600 m	500 m
Distance between end of 2000 m radius and front of guiding	1250 m	100 m

1.5.2 EVALUATION BASED ON THE CONCLUSIONS OF CHANNEL ALIGNMENT P1A

The very short distance for Alignment P1B will lead to the following inconvenience for the navigation and the general traffic in the Panama channel (for the ships coming from the Pacific):

- With alignment P1A, it will be possible that the design ship can enter the by-pass on his own force under most normal conditions. A tugboat at the stern assists the design ship while passing the Port of Balboa to reduce the ship's speed. An additional tugboat at the bow has to connect at the entrance of the bypass to assist under heavy wind conditions. Thanks to the tug assistance, the design ship will be able to stop before the lock. With alignment P1B, this is impossible taking into account the necessary stopping length. Consequently, the post-panamax container ships have to come to a stop before being towed into the by-pass by tugs.

- The assistance of the tugboats required to enter the by-pass at slow speed will (Alignment P1B) be much more important compared to the assistance needed to stop the ship in a straight line (Alignment P1A). This procedure will also be much more time consuming reducing the number of ships that can be passed through the locks within a certain period of time.

- For Alignment P1A, for the manoeuvre to connect with the locomotives on the guiding wall the ship will be completely in the by-pass. For Alignment P1B the stern of the ship (and the stern tug) will still be inside the main channel hindering normal traffic to the Miraflores Lake.

- Queuing of ships to enter the locks in convoy can be organised in the by-pass for Alignment P1A. For Alignment P1B the waiting ships have to stay in the main channel or Balboa port hindering navigation to Miraflores Lake.

- The curvature from the main channel to the by-pass continues almost until the head of the guiding wall for Alignment P1B. For Alignment P1A, the ships have at least 1250m of straight channel in front of the lock, which will make the stopping manoeuvre easier.

For convoys going to the Pacific there is almost no difference.

1.5.3 DESIGN RULES AND OTHER CONSIDERATIONS

For the alignment of channels, the PIANC preliminary design rules [8] suggest that between two consecutive bends a straight section of at least five ship's lengths should be available. As the alignment of a ship in front of a lock seems to be harder and requires more precision than entering a next bend, it is reasonable to use the same order of magnitude for the distance from a bend to the lock entrance. This condition is respected more or less by alignment P1A, but not for alignment P1B where the bend ends in front of the lock.

Another example is the Dutch design rules for inland navigation [9], where the layout for locks is discussed. The distance in front of the lock considered part of the lock entrance is estimated to require at least 2.5 times the ship's length.

If one considers the shorter entrances for the larger locks on the River Scheldt (about 600 m between river and lock for Berendrecht lock or 800m for Zandvliet lock) to be an argument for alignment P1B, one has to bear in mind that in Antwerp ships are assisted into the lock by the tugs. No approach of a guiding wall is needed.

A further consideration is the density current generated by the exchange of freshwater and seawater in the lock, quoted in the pilot handbook to be of considerable influence near the lock. In the alignment P1B, the turning manoeuvre will be considerably influenced by this current.

1.5.4 CONCLUSION

From a nautical point of view, Alignment P1A is highly preferable to alignment P1B.

To make the alignment P1B acceptable from a nautical viewpoint the locks have to be shifted over 500 to 1000 m upstream.

If this alignment P1B has to be studied further on to make the new lock accessible from the Pacific Ocean for post-panamax container ships and bulk carriers, a detailed study based on real-time simulation runs executed by human pilots will be necessary. This will also require the modelling of density currents, tidal currents and freshwater discharges in the Balboa Reach.

1.6 GENERAL CONCLUSIONS

The design ship used during this simulation study is a look-alike post-panamax container ship with a container capacity of 12 500 TEU's. The mathematical model is obtained after scaling an existing Panamax sized container ship and executing a validation based on captive model tests at an under keel clearance of 15% of the ship's draught.

The evaluation of channel alignment P1A is based on the following conclusions:

- At all tested wind and tidal conditions the container ship can pass the Port of Balboa and enter or leave the by-pass at both sides (Pacific Ocean and Gatún Lake) on its own. The lateral deviation is small. Coming from the Pacific Ocean tug assistance at the stern is necessary to help reducing the ship's speed to minimise squat effects.

- The available distance between the axis of the Balboa reach or the Culebra reach and the front side of the guiding wall determines the entrance manoeuvre respectively from the Pacific Ocean or from Gatún Lake. Entering the by-pass from both sides should not cause any difficulty taking into account the stopping length of the container ship.

- The stopping length of the container ship depends upon the ship's velocity, the telegraph setting and the availability of tugboats. From the moment the engine is stopped or Dead Slow Astern has been given, the own manoeuvrability of the ship is small as the rudder is no longer in the slipstream of the propeller.

- At moderate wind conditions and low water, thanks to the hydrodynamic resistance, the turning manoeuvre during the entrance of the by-pass from the Pacific Ocean can be stopped with minimal additional assistance of tugboats or bow thruster. The ship is stopped in front of the guiding wall. At rising tide, the available hydrodynamic resistance is reduced compared to the low water condition. Additional tug assistance (or use of a bow thruster if present) in the by-pass is essential to stop the turning manoeuvre to port and to stop the container ship in front of the guiding wall. (The same conclusion will apply if the draught of the ship is significantly smaller than that of the design ship.)

- If the container ship has to be stopped in front of the guiding wall, tugboats and bow thruster have to keep the ship in position. For a crosswind of 5 Beaufort, the estimated lateral tug force is more than 40 tonf.

The evaluation of the alternative channel alignment P1B is based on the following nautical disadvantages:

- Due to the very short distance between the axis of the Balboa reach and the front side of the guiding wall (500 m), the design ship can not enter the by-pass without tug assistance. Additionally, taking into account the necessary stopping length the post-panamax containerships have to come to a stop before being towed into the by-pass by tugs.

- The curvature from the main channel to the by-pass continues almost until the head of the guiding wall for Alignment P1B. For Alignment P1A, the ships have at least 1250m of straight channel in front of the lock, which will make the stopping manoeuvre easier.

- The assistance of tugboats required to enter the by-pass at slow speed (Alignment P1B) will be much more important compared to the assistance needed to stop the ship in a straight line (Alignment P1A). This procedure will also be much more time consuming reducing the number of ships that can be passed through the locks within a certain period of time.

- During the connection of the locomotives with the ship the stern of the ship (and the stern tug) will still be inside the main channel hindering normal traffic to the Miraflores Lake. Additionally, queuing of ships to enter the locks in convoy cannot be organised out of the main channel.

- In Alignment P1B the turning manoeuvre into the by-pass from the Pacific Ocean will be considerably influenced by the density current generated by the exchange of freshwater and seawater in the lock.

- A straight section of at least five ship's lengths between two consecutive bends should be available according to the PIANC preliminary design rules. This same order of magnitude could be used concerning the distance from a bend to a lock as the ship's speed has to be minimised during the entrance of the lock.

From a nautical point of view Alignment P1A is highly preferable to Alignment P1B. A minimum length of approximately 2.5 times the ship's length between the bend of the by-pass and the lock is necessary.

September 2002

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TABLES

Table 1. Geometrical and full-scale trial data of container ship CON_A
Table 2. General characteristics of Post-panamax container ship CON_PP
Table 3. Estimated power according to the service speed and service margin
Table 4. Characteristics of largest existing container ships
Table 5. Characteristics of post-panamax container ship D518, modelled at scale 1/75
Table 6. Acceleration manoeuvres at an under keel clearance of 15%
Table 7. Crash-stop manoeuvre at an under keel clearance of 15%
Table 8. Turning circle manoeuvre at an under keel clearance of 15%
Table 9. 20/20 zigzag manoeuvre at an under keel clearance of 15%
Table 10. Monthly average wind speed (mph) (Balboa FAA)
Table 11. Monthly maximum wind speed (mph) (Balboa FAA)
Table 12. Monthly average wind direction (deg) (Balboa FAA)
Table 13. Conditions: sailing to lock from Port of Balboa.
Table 14. Conditions: leaving the lock entrance for Port of Balboa
Table 15. Conditions: sailing to the lock from Gatún Lake
Table 16. Characteristics of stopping manoeuvres at 15% under keel clearance in open water

Table 1. Geometrical and full-scale trial data of containership CON_A

Main dimensions		Engine	
Length over all	265.0 m	Type	Diesel
Beam	32.2 m	Power	23500 kW
Draft fully laden	13.5 m	max revs	96 rpm
Displacement	70.099 ton	Bow thruster	2500 kW
Dead weight tonnage	50.000 ton	Stern thruster	2500 kW

Telegraph	rpm	Speed	Crash stop		
		kn	Distance	Time	
SEA FULL	96	19.5	mile	min	sec
HARBOUR FULL	75	15.2			
HALF	60	12.2	1.16	5	44
SLOW	45	9.1			
DEAD SLOW	30	6.1			

Turning circles						
Telegraph	Rudder	Advance	Transfer	Time		Tact. diameter
		Mile	Mile	min	sec	Mile
SEA FULL	SB	0.47	0.27	2	39	0.45
	P	0.46	0.26	2	34	0.44
HALF	SB	0.46	0.25	4	19	0.44
	P	0.45	0.26	4	10	0.43

Table 2. General characteristics of Post-panamax containership CON_PP

Main dimensions		Engine	
Length over all, L_{OA}	385.7 m	Type	Diesel
Length between perpendiculars, L_{PP}	371.0 m	Power	>100 000 kW
Beam, B	54.9 m	Max revolutions	94 rpm
Design Draft, T	15.2 m	Estimated service speed	25 knots
Displacement	195 000 ton	Bow thruster	2500 kW
Dead weight tonnage (estimated)	105 000 ton		
Estimated frontal wind area	1812 m^2		
Estimated lateral wind area	10300 m^2		

Table 3. Estimated power according to the service speed and service margin

10% Service margin		20% Service margin	
Service speed knots	Power kW	Service speed knots	Power kW
23	88450	23	92248
24	96308	24	100445
25	104501	25	108989

Table 4. Characteristics of largest existing containerships

Ship name	Maersk S-klasse	Hamburg Express
built in	1997	2001
LOA (m)	347	320
Beam (m)	42.8	42.8
Depth (m)	*25*	24.5
Draft (m)	14.6	14.5
TEU's	>7000	7500
DWT (ton)	104 886	100 000
Speed (knots)	25.0	25.3
Engine Power (kW)	54 900	68 640
MCR (rpm)		94
Propeller D (m)		9.1

Table 5. Characteristics of post-panamax containership D518, modelled at scale 1/75

L_{OA}	301.05	m
L_{PP}	289.804	m
B	40.252	m
D	22.8	m
T_M	11.6	m
T_F	11.6	m
T_A	11.6	m
Block coefficient	0.5607	
Displacement	77765	ton
Number of propellers	1	
Diameter of propeller	8.145	m
Pitch ratio	0.9696	
Number of rudders	1	
Total rudder area	60.96	m^2
Scale	75	

Table 6. Acceleration manoeuvres at an under keel clearance of 15%

Telegraph	Propeller rate (rpm)	Ship speed (m/s)	Acceleration time (s)	Acceleration distance (m)
Harbour Full	65.5	8.36	1198	7736
Half	53.5	6.85	1278	6979
Slow	41.5	5.34	1500	6290
Dead slow	30	3.82	1846	5378

Table 7. Crash-stop manoeuvre at an under keel clearance of 15%

Initial speed	3.82 m/s
Initial propeller rate	30 rpm
Propeller rate at crash stop	-30 rpm
Time to stop	6 min 18 sec
Distance to stop	610.5 m
Transfer at stop	31.5 m
Course angle at stop	9.7°

Table 8. Turning circle manoeuvre at an under keel clearance of 15%

Turning circle						
Telegraph	Rudder	Advance (m)	Transfer (m)	Time to 90° turn		Tact. diameter (m)
HALF	SB, 35°	2340.4	1904.0	10 min	0 sec	3799.7 = 9.87L_{OA}

Table 9. 20/20 zigzag manoeuvre at an under keel clearance of 15%

Zigzag manoeuvre				
Telegraph	Rudder	Period	Overshoot angle	Overshoot time
DEAD SLOW	20°/ 20°	26 min 10 sec	1.4°	50 sec

Table 10. Monthly average wind speed (mph) (Balboa FAA)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1985	4.8	6.1	6.3	5.7	4.6	3.5	4.0	3.7	3.4	3.8	4.4	4.5
1986	6.1	5.3	6.3	5.4	5.3	4.4	5.2	4.6	4.3	3.5	3.7	5.1
1987	6.7	6.8	5.2	6.0	4.2	3.6	3.9	4.4	3.5	3.8	4.2	4.4
1988	5.3	6.0	6.1	5.2	3.9	3.6	4.1	3.4	4.2	3.8	4.1	3.8
1989	4.9	6.5	5.4	5.7	4.0	2.9	3.9	3.5	3.6	3.6	3.4	4.0
1990	4.6	5.7	6.2	5.8	4.2	4.1	3.7	3.5	3.6	3.3	3.6	3.6
1991	5.3	6.5	5.8	4.9	4.2	3.5	3.9	3.9	3.4	3.2	4.1	3.6
1992	5.8	5.9	6.7	5.8	5.4	3.8	3.9	4.5	3.7	3.7	3.2	3.4
1993	4.7	5.7	5.7	5.0	4.2	3.5	4.0	3.8	3.2	3.2	3.8	4.3
1994	5.2	5.7	5.3	4.9	3.6	3.7	4.1	3.3	3.7	2.5	3.6	3.5
1995	4.8	5.2	4.8	3.2	2.8	2.9	2.9	2.9	2.9	3.2	2.7	2.8
1996	3.1	3.8	4.2	3.6	2.1	2.5	2.6	2.7	2.5	3.4	4.1	3.3
1997	4.0	4.1	5.2	5.2	4.4	3.5	4.0	4.3	3.4	3.0	3.4	5.1
1998	5.5	4.8	5.8	4.8	3.9	3.4	3.6	3.6	3.5	4.0	2.9	2.9
1999	3.2	4.6	4.8	4.3	3.4	3.3	4.0	3.5	3.7	3.7	3.8	4.1
2000	3.8	4.2	4.2	4.6	3.9	3.5	3.9	3.7	3.4	4.1	3.3	4.3
2001	5.4	6.6	6.1	5.7	4.5	3.4	3.9	4.1	3.3	3.3	4.3	4.1
Monthly Maximum	6.7	6.8	6.7	6.0	5.4	4.4	5.2	4.6	4.3	4.1	4.4	5.1
Monthly Minimum	3.1	3.8	4.2	3.2	2.1	2.5	2.6	2.7	2.5	2.5	2.7	2.8
Monthly Average	4.9	5.4	5.5	5.0	4.0	3.5	3.9	3.7	3.5	3.5	3.7	3.9

Table 11. Monthly maximum wind speed (mph) (Balboa FAA)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1985	25.7	28.5	33.4	29.6	26.1	31.4	21.5	20.4	25.7	23.5	27.2	25.2
1986	25.0	25.7	30.5	27.9	27.9	26.1	33.8	46.0	23.5	19.5	24.1	24.6
1987	29.6	31.2	28.1	26.8	24.3	26.5	20.8	23.9	21.2	22.3	22.8	23.0
1988	26.5	25.2	29.9	25.4	23.2	25.0	23.5	22.3	27.0	24.1	20.1	25.7
1989	24.3	35.2	30.1	24.6	25.0	21.1	42.3	34.3	22.6	26.5	24.3	26.8
1990	25.0	24.6	30.8	27.9	33.2	50.9	36.6	23.9	24.6	21.9	21.9	23.5
1991	24.8	34.3	27.2	29.4	23.9	24.8	34.3	25.2	24.6	24.8	21.0	21.2
1992	26.8	27.4	27.7	25.4	27.0	31.4	27.2	32.3	28.5	38.3	33.0	20.6
1993	25.9	26.5	28.3	24.8	23.7	29.9	28.1	25.2	35.6	21.0	25.2	24.3
1994	25.2	28.1	33.4	28.3	24.1	25.9	23.0	22.1	24.1	23.2	25.0	23.5
1995	28.3	26.8	29.6	23.0	29.0	25.0	26.5	30.3	27.9	29.0	23.7	21.7
1996	25.0	23.7	30.8	23.9	21.2	21.7	23.7	21.5	19.7	23.2	23.9	21.2
1997	22.3	23.0	26.3	25.0	22.8	26.5	29.0	30.1	29.4	20.8	24.3	28.1
1998	27.0	25.2	30.1	30.5	24.8	30.1	23.2	25.9	22.1	27.7	23.5	23.0
1999	23.5	25.0	24.8	24.6	21.0	22.3	23.2	21.5	21.7	26.3	21.5	27.2
2000	33.4	25.2	25.0	26.5	27.0	21.9	25.4	33.3	28.1	23.1	25.1	24.8
2001	24.3	25.9	25.5	24.5	24.0	30.5	22.3	28.9	29.4	22.3	22.1	24.4
Monthly Maximum	33.4	35.2	33.4	30.5	33.2	50.9	42.3	46.0	35.6	38.3	33.0	28.1
Monthly Minimum	22.3	23.0	24.5	23.0	21.0	21.1	20.8	20.4	19.7	19.5	20.1	20.6
Monthly Average	25.9	27.1	28.7	26.5	25.2	27.7	27.3	27.5	25.6	24.6	24.0	24.0

Table 12. Monthly average wind direction (deg) (Balboa FAA)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	WD
1985	333.0	344.3	342.3	335.1	302.6	308.1	311.9	303.9	287.3	279.9	301.4	323.2	**318.08**
1986	341.6	338.0	343.3	335.3	272.9	289.6	316.5	314.6	297.5	269.6	299.7	306.4	**313.62**
1987	318.2	329.4	326.8	323.8	313.5	309.3	305.7	309.4	301.1	285.0	313.2	328.9	**315.65**
1988	340.2	344.7	349.5	333.3	313.3	175.6	277.5	108.1	90.0	113.8	261.6	293.9	**332.75**
1989	270.6	278.9	343.9	2.5	336.6	322.2	328.9	328.7	229.6	331.6	310.9	341.1	**317.75**
1990	354.6	343.6	343.4	338.7	293.1	311.0	304.4	301.1	305.6	218.8	298.7	336.1	**320.00**
1991	335.2	351.8	340.8	339.7	270.6	318.0	308.9	305.0	295.6	275.0	308.7	348.1	**320.75**
1992	336.2	336.8	345.5	334.6	316.2	305.0	313.9	318.5	269.7	297.8	285.5	323.9	**319.91**
1993	324.7	337.1	330.7	318.8	227.5	294.4	303.6	300.1	259.0	274.0	297.1	320.7	**304.67**
1994	327.9	337.1	333.1	339.8	274.6	302.4	302.1	295.6	278.5	204.8	259.4	332.9	**308.54**
1995	337.1	342.5	339.5	299.5	289.5	194.0	284.9	180.3	188.4	185.6	303.2	316.7	**295.16**
1996	331.9	346.1	342.4	329.2	202.7	297.0	301.2	337.2	279.0	216.3	278.3	335.1	**309.24**
1997	336.9	344.2	349.7	341.8	334.2	287.6	325.3	326.6	316.5	285.4	288.3	337.5	**326.52**
1998	341.5	336.8	346.8	335.4	328.7	307.5	314.8	309.9	219.8	206.7	308.5	327.7	**318.24**
1999	357.5	354.4	351.4	338.4	315.8	300.7	320.8	299.7	193.6	194.5	316.2	327.4	**320.27**
2000	351.8	353.4	354.7	354.5	321.3	302.9	311.6	299.0	184.5	301.7	296.3	313.0	**321.39**
2001	328.6	332.9	325.6	336.0	295.9	296.3	298.3	302.8	267.9	267.8	303.8	319.1	**311.03**
Monthly	**333.4**	**338.6**	**341.9**	**336.4**	**299.2**	**296.6**	**308.3**	**308.8**	**263.9**	**255.4**	**296.0**	**325.0**	

Table 13. Conditions: sailing to lock from Port of Balboa.

wind direction	wind speed	wind Beaufort	tug assistance thrust	tug assistance from	stop engine from	bow thruster from 9400 m	tide	current
Condition 1								
no	no	no	40 tonf	6600 m	9400 m	no	LW	no
no	no	no	40 tonf	6600 m	9400 m	no	+3.42 m, FLOOD	max.
Condition 2								
S	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
SW	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
N	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
NW	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
W	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	LW	no
Condition 3								
N	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	no	+3.42 m, FLOOD	max.
NW	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
W	4.4 m/s	3 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
SW	4.4 m/s	3 Bf	40 tonf + 10 tonf	6600 m + 9400 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
S	4.4 m/s	3 Bf	40 tonf + 10 tonf	6600 m + 9400 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
Condition 4								
N	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	LW	no
NNW	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	LW	no
NW	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	no	LW	no
W	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% P	LW	no
Condition 5								
N	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
NNW	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
W	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.
NW	9.4 m/s	5 Bf	40 tonf	6600 m	9400 m	yes, 75% SB	+3.42 m, FLOOD	max.

Table 14. Conditions: leaving the lock entrance for Port of Balboa

wind direction	wind speed	wind Beaufort	tug assistance thrust	tug assistance from	tide	current
Condition 6						
N	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum
NW	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum
W	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum
SW	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum
S	4.4 m/s	3 Bf	no	no	+2.82m, EBB	maximum

Table 15. Conditions: sailing to the lock from Gatún Lake

wind			tug assistance		stop engine	bow thruster	current
direction	speed	Beaufort	thrust	from	from		
Condition 7							
no	no	no	40 tonf	7500 m	7500 m	no	no
no	no	no	no	no	no	no	no
Condition 8							
N	4.4 m/s	3 Bf	no	no	no	no	no
NW	4.4 m/s	3 Bf	no	no	no	no	no
W	4.4 m/s	3 Bf	no	no	no	no	no
S	4.4 m/s	3 Bf	no	no	no	no	no
SW	4.4 m/s	3 Bf	no	no	no	no	no
Condition 9							
N	9.4 m/s	5 Bf	40 tonf	7500 m	7500 m	no	no
NNW	9.4 m/s	5 Bf	40 tonf	7500 m	7500 m	no	no
NW	9.4 m/s	5 Bf	40 tonf	7500 m	7500 m	no	no

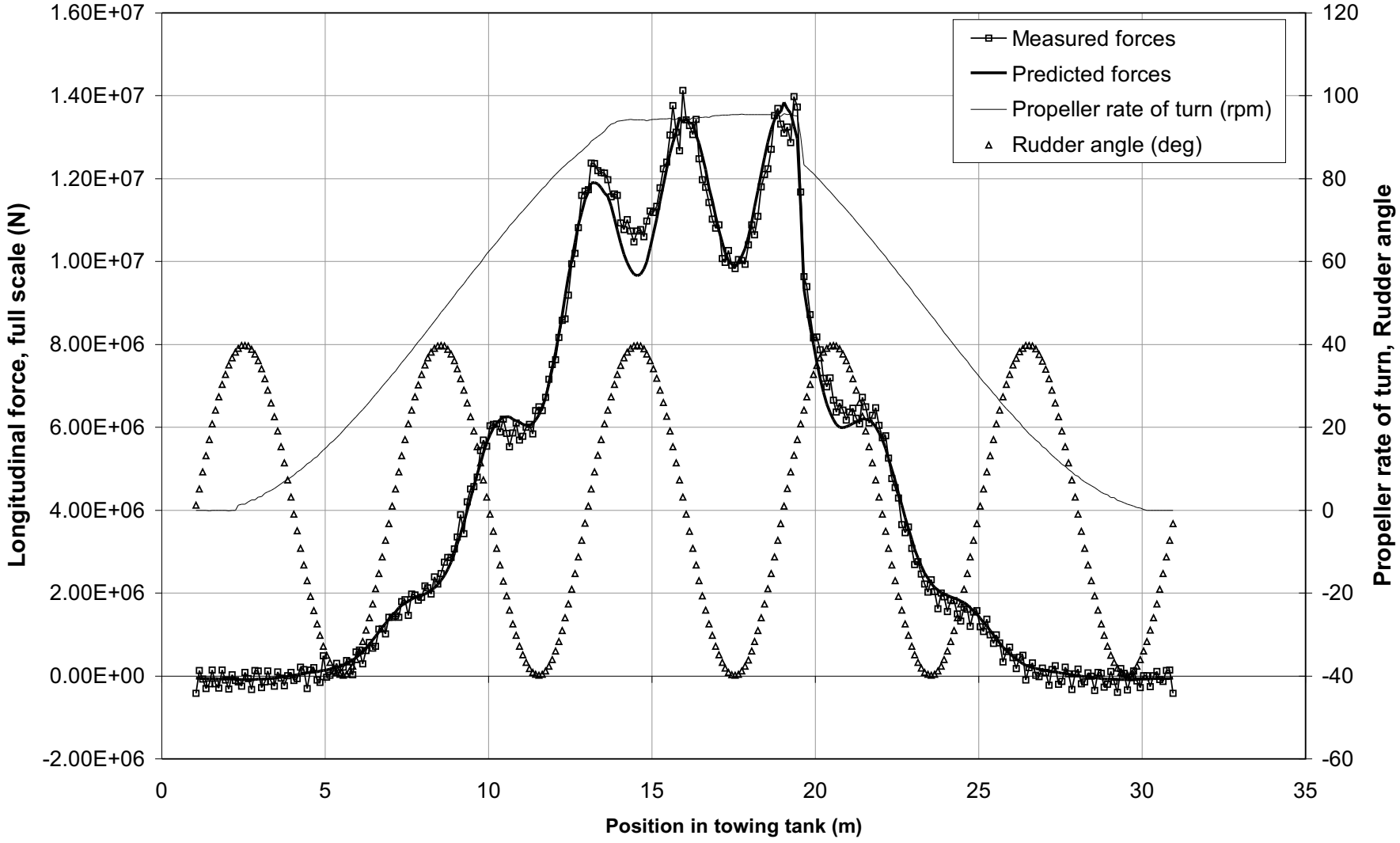
Table 16. Characteristics of stopping manoeuvres at 15% under keel clearance in open water

	Condition A	Condition B	Condition C	Condition D
Initial speed	3.82 m/s	2.06 m/s	2.57 m/s	3.09 m/s
Initial propeller rate	30 rpm	30 rpm	30 rpm	30 rpm
Propeller rate at stop	-30 rpm	0 rpm	0 rpm	0 rpm
Tug thrust at stop	0 tonf	-40 tonf	-40 tonf	-40 tonf
Time to stop	6 min 18 sec	15 min 18 sec	17 min 38 sec	19 min 34 sec
Distance to stop	610 m	863 m	1189 m	1517 m
Transfer at stop	31.5 m	0 m	0 m	0 m
Course angle at stop	9.7 deg	0 deg	0 deg	0 deg

FIGURES

Figure 1 Comparison of measured and modelled longitudinal force
Figure 2 Comparison of measured and modelled lateral force
Figure 3 Comparison of measured and modelled yawing moment
Figure 4 Turning circle manoeuvre at 15% under keel clearance
Figure 5 Definition of cost function parameters
Figure 6 Study area
Figure 7 Trajectory from the Pacific Ocean to the lock
Figure 8 Trajectory from the lock to the Pacific Ocean
Figure 9 Trajectory from Gatún Lake to the lock
Figure 10 Bank suction along the Pacific Ocean trajectory
Figure 11 Bank suction along the Gatún Lake trajectory
Figure 12 Influence of the channel width on coefficients Y_β and N_β
Figure 13 Current pattern from the Pacific Ocean to the lock
Figure 14 Definition of wind direction
Figure 15 Monthly average wind speeds and wind directions
Figure 16a Lateral and angular deviation for condition 1: no wind
Figure 16b Ship's velocity for condition 1: no wind
Figure 16c Rudder and engine command for condition 1: no wind
Figure 17 Swept path for condition 1: no wind
Figure 18a Lateral and angular deviation for condition 2: low water, 3 Beaufort wind
Figure 18b Ship's velocity for condition 2: low water, 3 Beaufort wind
Figure 18c Rudder and engine command for condition 2: low water, 3 Beaufort wind
Figure 19 Swept path for condition 2: low water, 3 Beaufort wind
Figure 20 Lateral force and yawing moment delivered by the bow thruster during a simulation run
Figure 21a Lateral and angular deviation for condition 3: rising tide, 3 Beaufort wind
Figure 21b Ship's velocity for condition 3: rising tide, 3 Beaufort wind
Figure 21c Rudder and engine command for condition 3: rising tide, 3 Beaufort wind
Figure 22 Swept path for condition 3: rising tide, 3 Beaufort wind
Figure 23a Lateral and angular deviation for condition 4: low water, 5 Beaufort wind
Figure 23b Ship's velocity for condition 4: low water, 5 Beaufort wind
Figure 23c Rudder and engine command for condition 4: low water, 5 Beaufort wind
Figure 24 Swept path for condition 4: low water, 5 Beaufort wind
Figure 25a Lateral and angular deviation for condition 5: rising tide, 5 Beaufort wind
Figure 25b Ship's velocity for condition 5: rising tide, 5 Beaufort wind
Figure 25c Rudder and engine command for condition 5: rising tide, 5 Beaufort wind
Figure 26 Swept path for condition 5: rising tide, 5 Beaufort wind
Figure 27a Lateral and angular deviation for condition 6: outgoing tide, 3 Beaufort wind
Figure 27b Ship's velocity for condition 6: outgoing tide, 3 Beaufort wind
Figure 27c Rudder and engine command for condition 6: outgoing tide, 3 Beaufort wind
Figure 28 Swept path for condition 6: outgoing tide, 3 Beaufort wind
Figure 29a Lateral and angular deviation for condition 7: no wind

Figure 29b Ship's velocity for condition 7: no wind
Figure 29c Rudder and engine command for condition 7: no wind
Figure 30 Swept path for condition 7: no wind
Figure 31a Lateral and angular deviation for condition 8: 3 Beaufort wind
Figure 31b Ship's velocity for condition 8: 3 Beaufort wind
Figure 31c Rudder and engine command for condition 8: 3 Beaufort wind
Figure 32 Swept path for condition 8: 3 Beaufort wind
Figure 33a Lateral and angular deviation for condition 9: 3 Beaufort wind
Figure 33b Ship's velocity for condition 9: 3 Beaufort wind
Figure 33c Rudder and engine command for condition 9: 3 Beaufort wind
Figure 34 Swept path for condition 9: 3 Beaufort wind
Figure 35 Alignment P1B



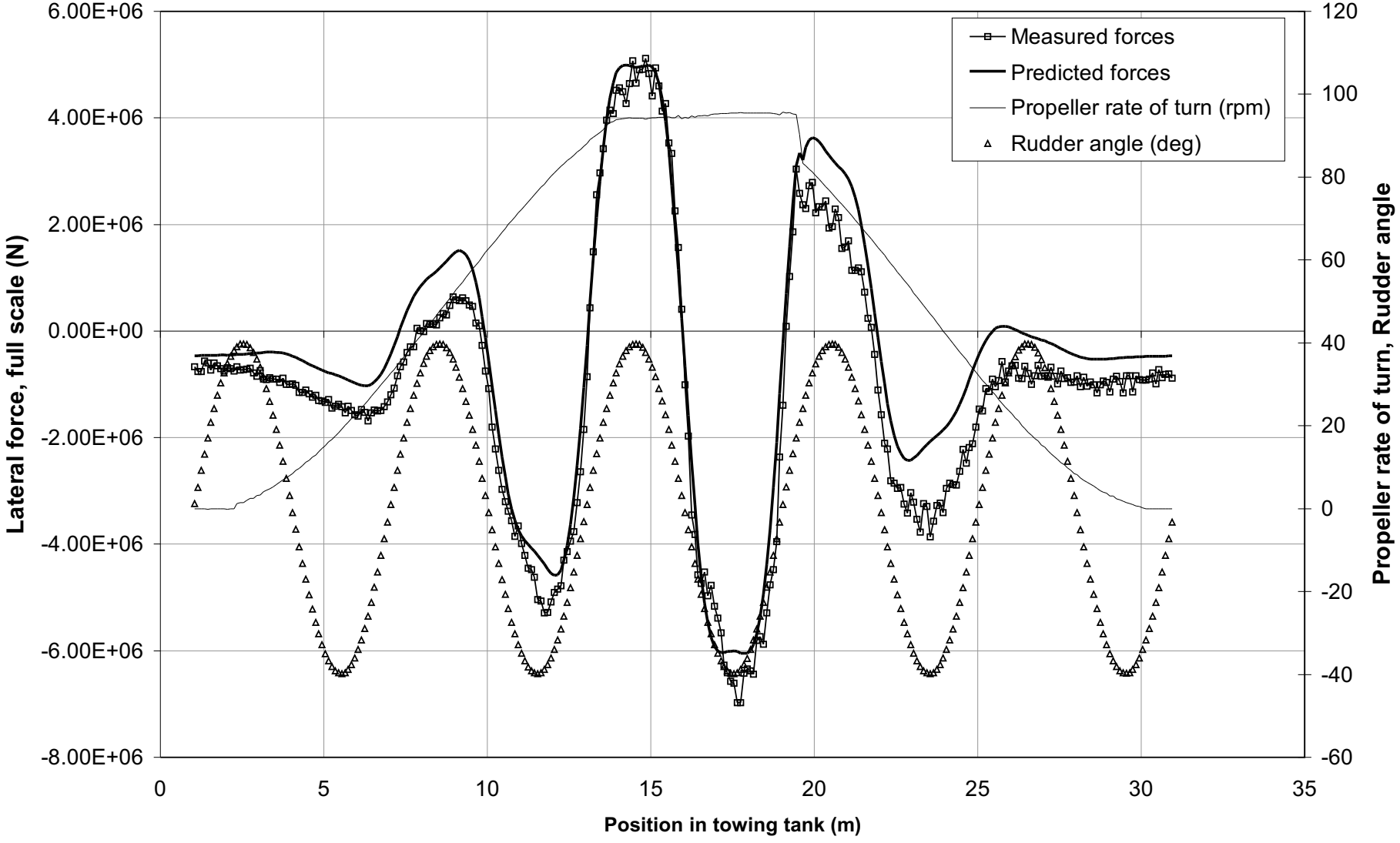
FLANDERS HYDRAULICS

MOD. 673 SIMULATION STUDY PANAMA CANAL

COMPARISON OF MEASURED AND MODELLED LONGITUDINAL FORCE

Figure 1

W.L. 02.0175



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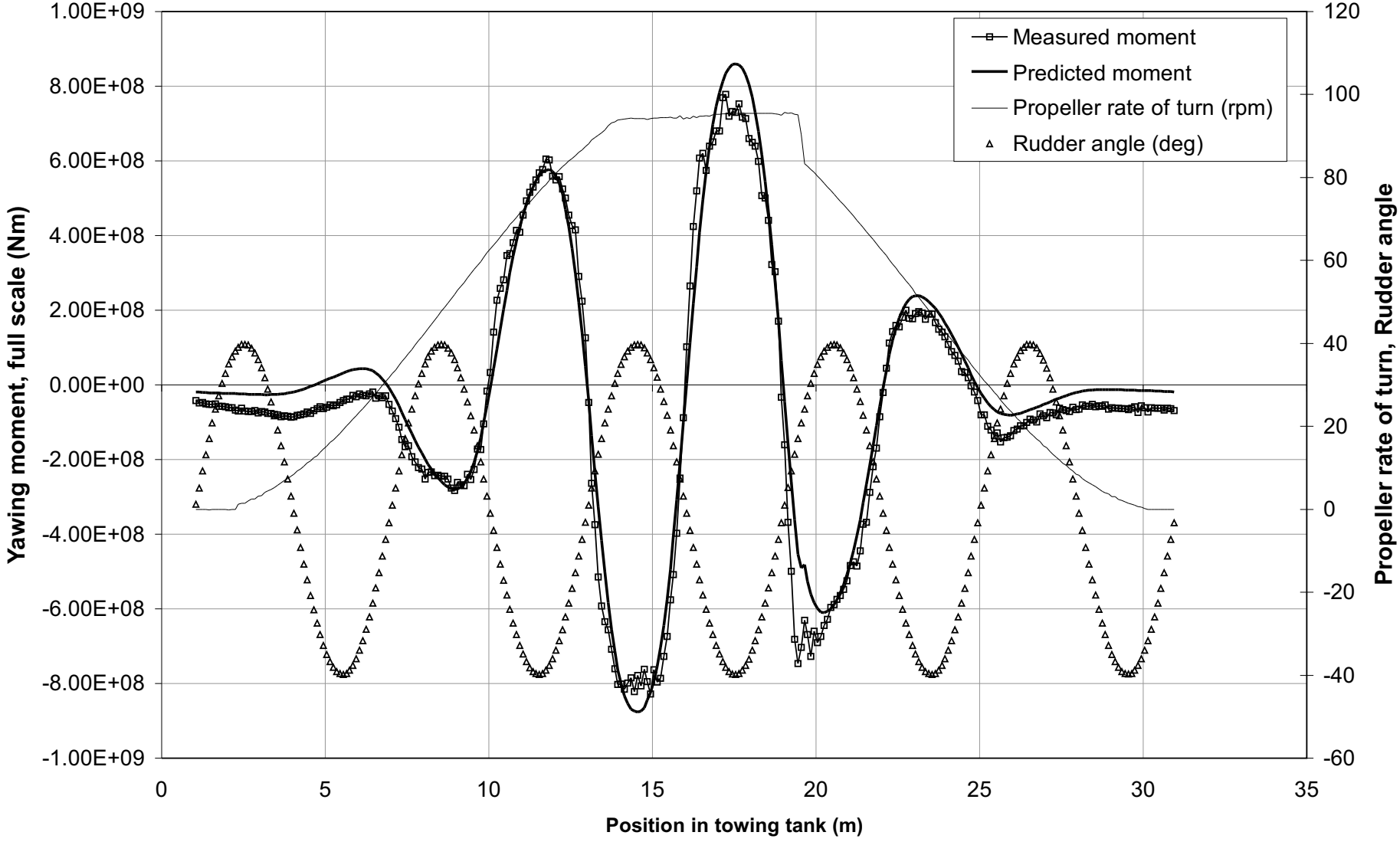
FLANDERS HYDRAULICS

MOD. 673 SIMULATION STUDY PANAMA CANAL

COMPARISON OF MEASURED AND MODELLED LATERAL FORCE

Figure 2



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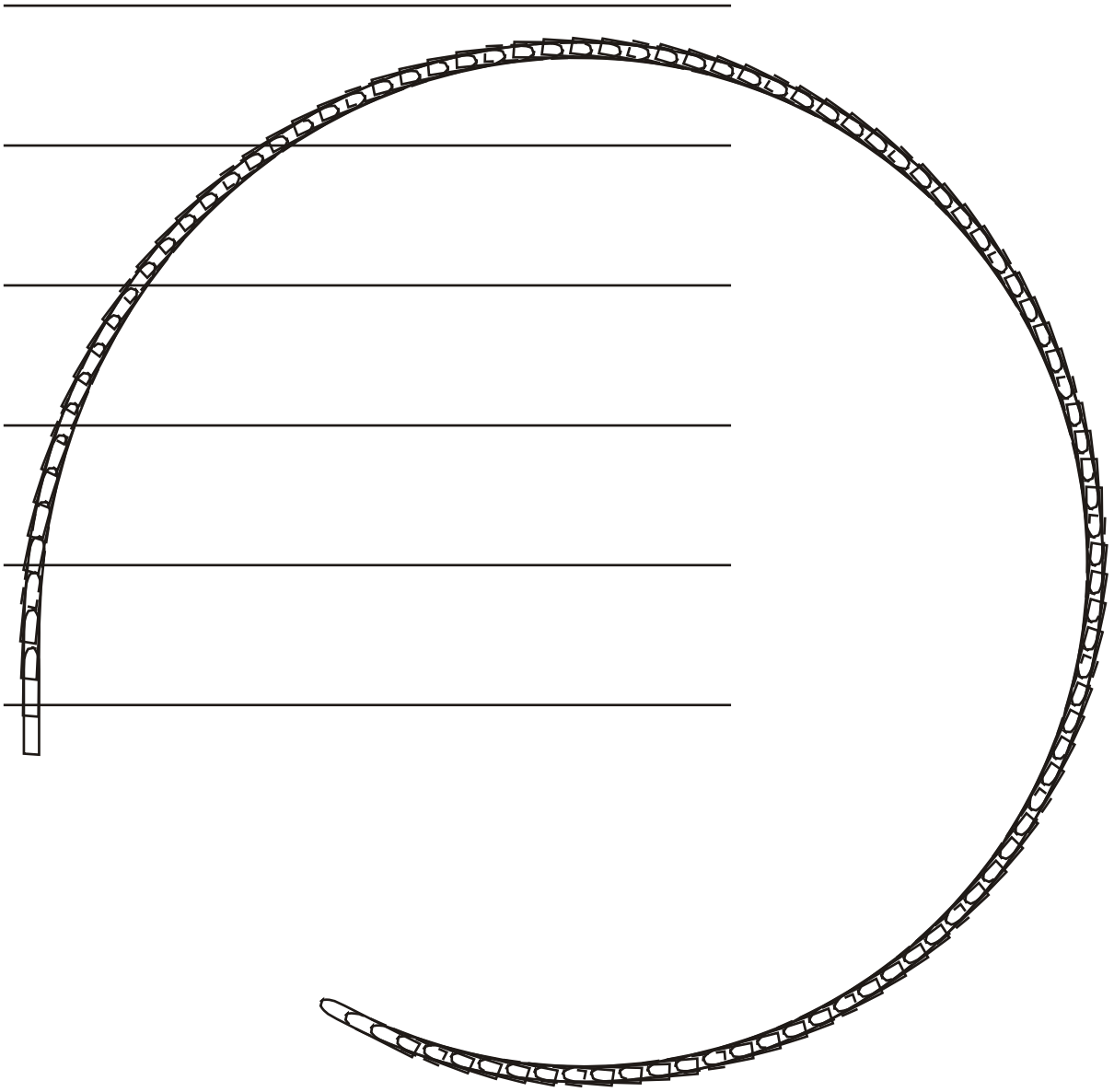
FLANDERS HYDRAULICS

MOD. 673 SIMULATION STUDY PANAMA CANAL

COMPARISON OF MEASURED AND MODELLED YAWING MOMENT

Figure 3

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 4
Plotinterval 20. s Scale 1/25000	TURNING CIRCLE MANOEUVRE AT 15% UNDER KEEL CLEARANCE	



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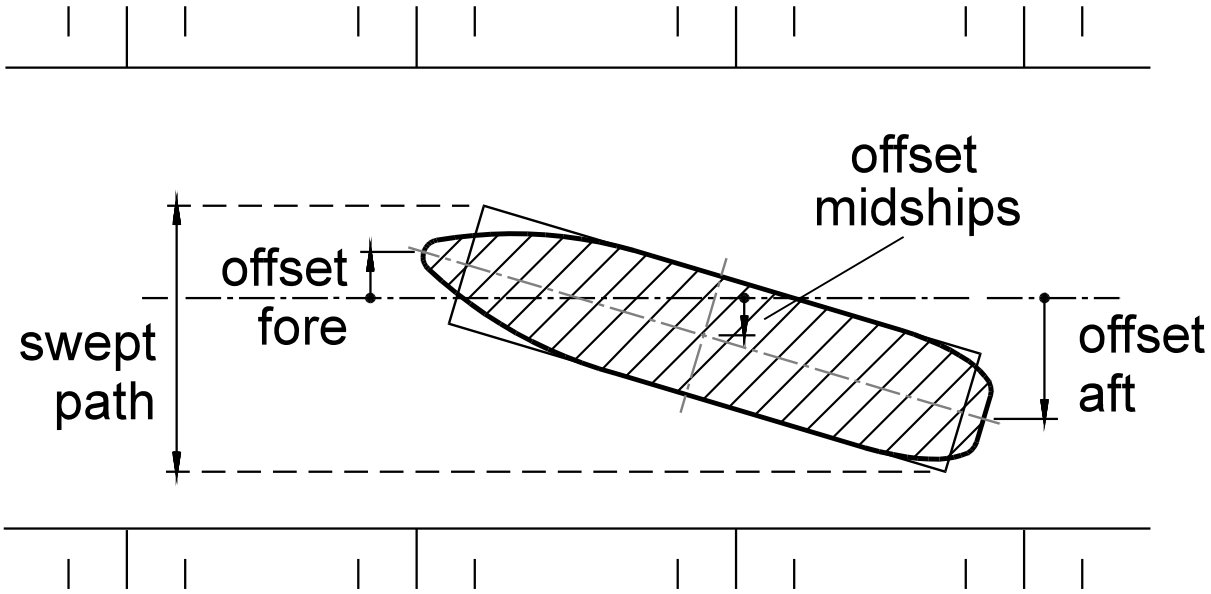
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 5
	DEFINITION OF COST FUNCTION PARAMETERS	



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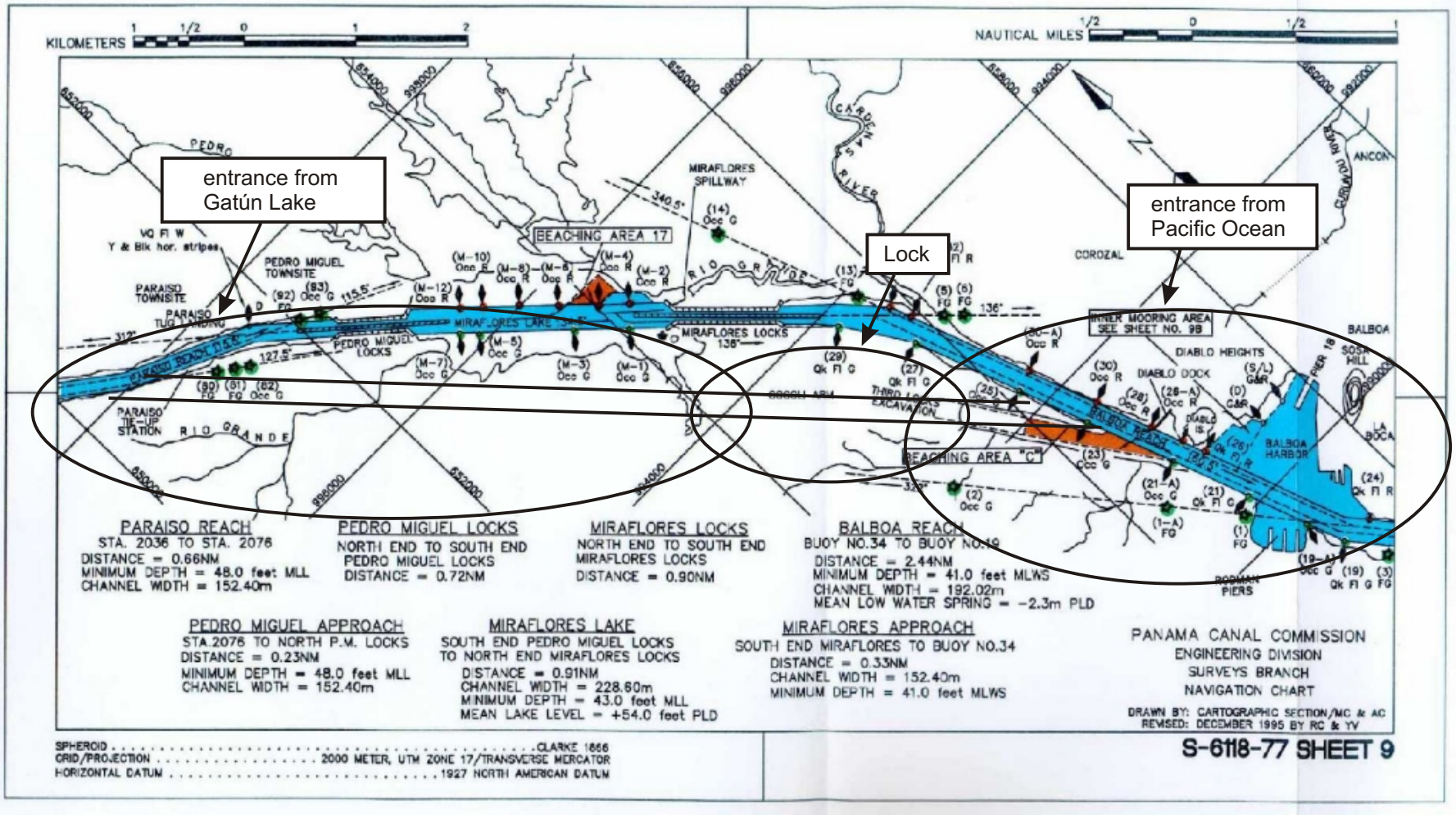


FLANDERS HYDRAULICS

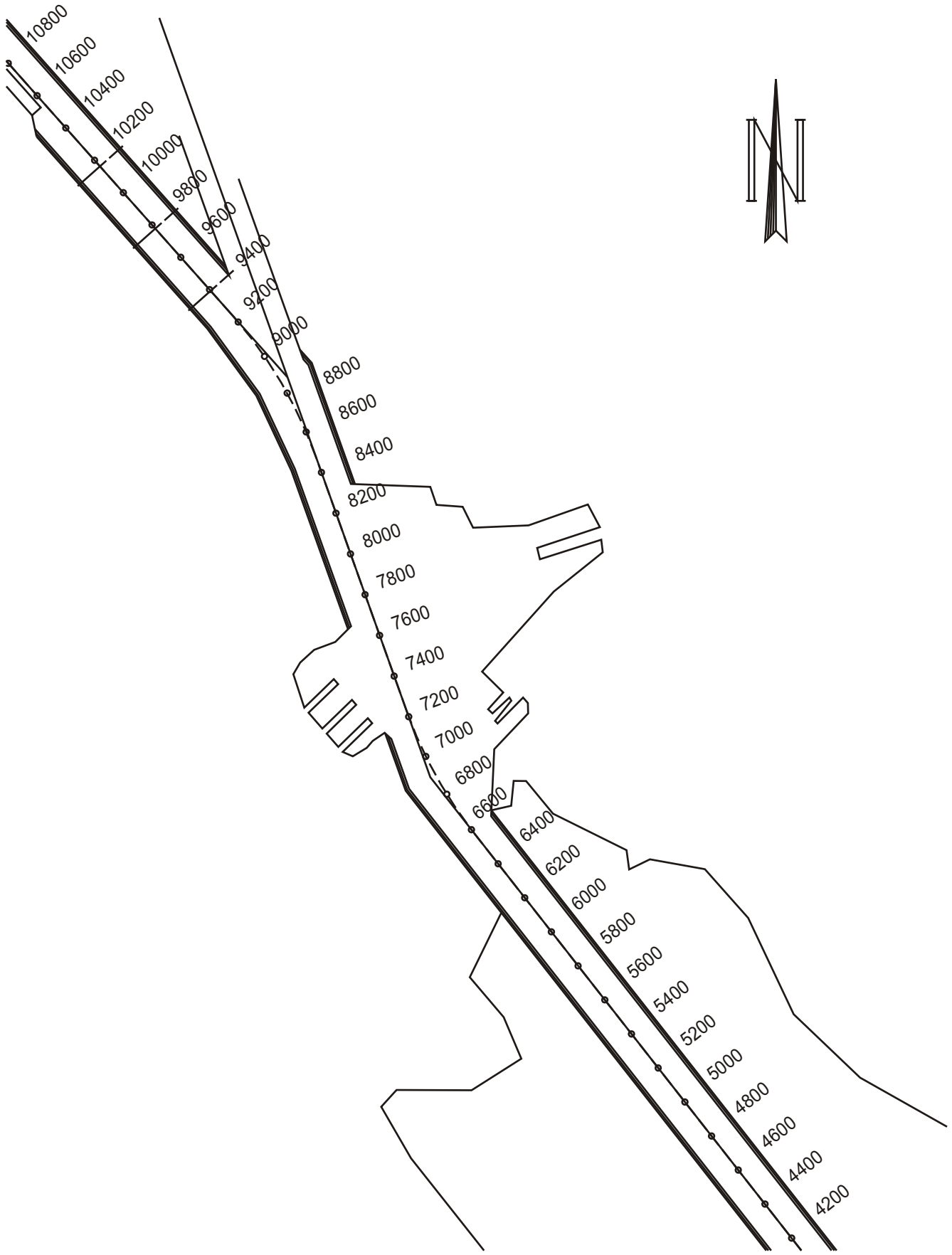
MOD. 673 SIMULATION STUDY PANAMA CANAL

STUDY AREA

Figure 6

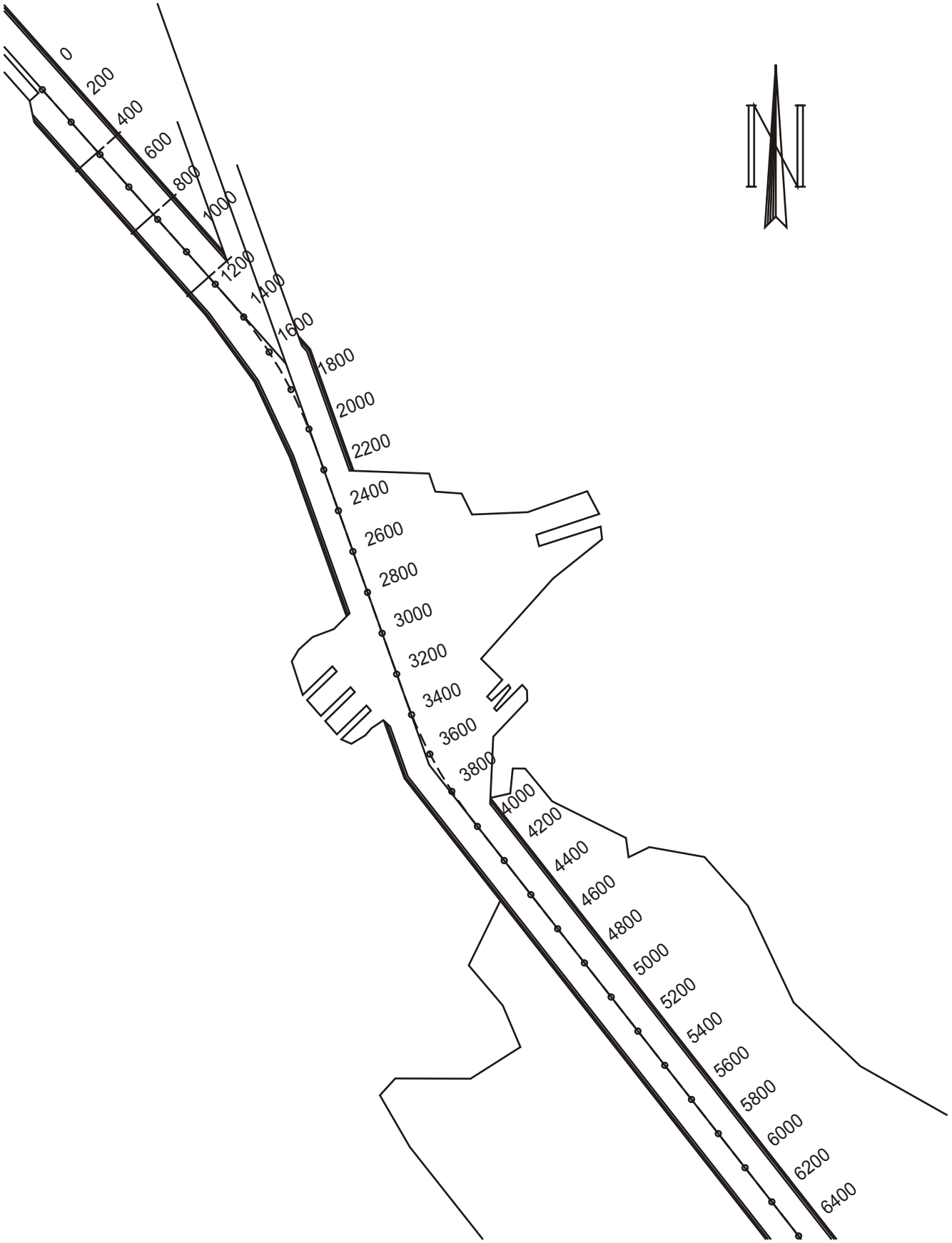
W.L. 02.0180

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 7
Distance in meter (m) Scale: 1/25000	TRAJECTORY FROM THE PACIFIC OCEAN TO THE LOCK	



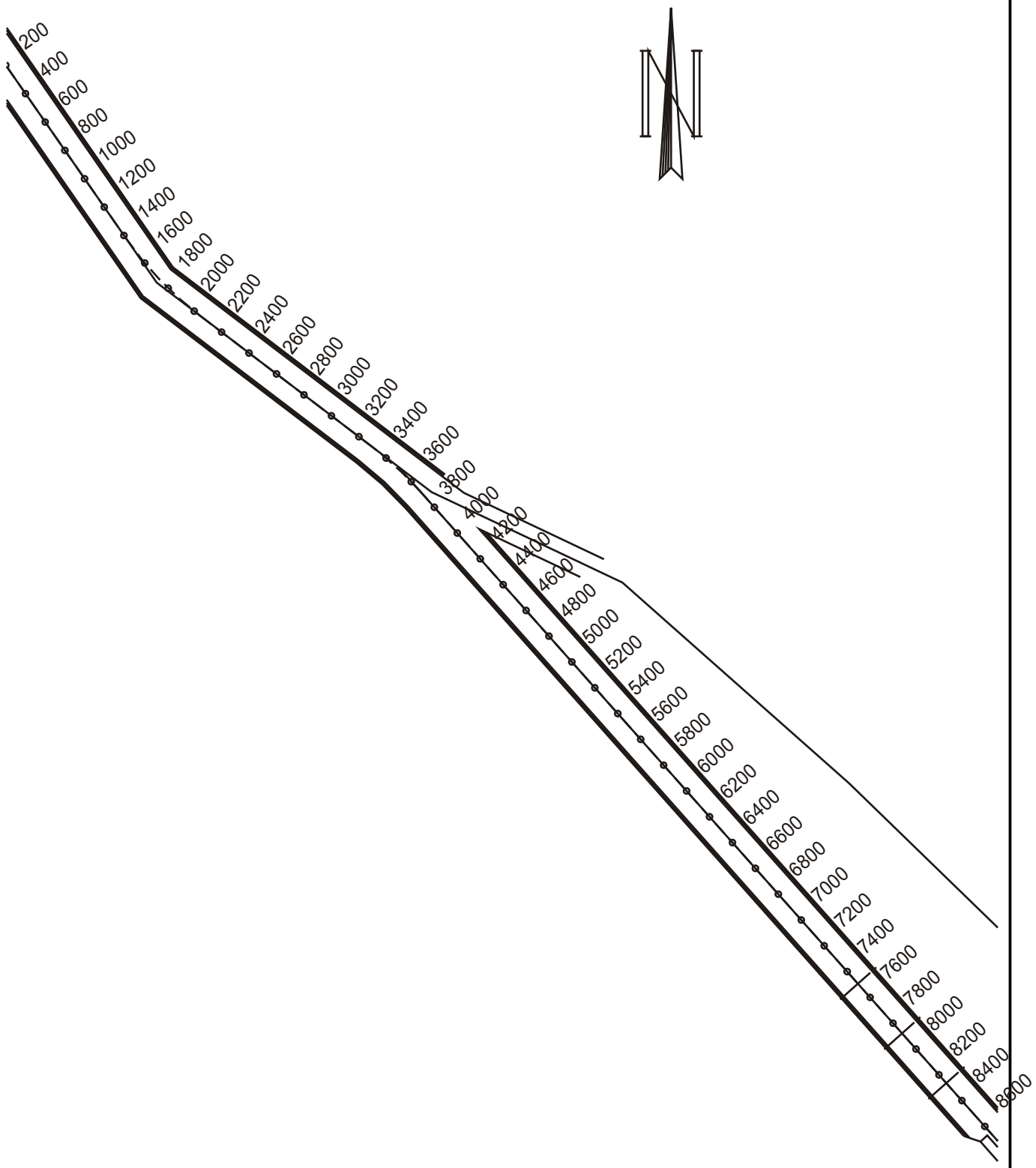
W.L. 02.0181

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 8
Distance in meter (m) Scale: 1/25000	TRAJECTORY FROM THE LOCK TO THE PACIFIC OCEAN	



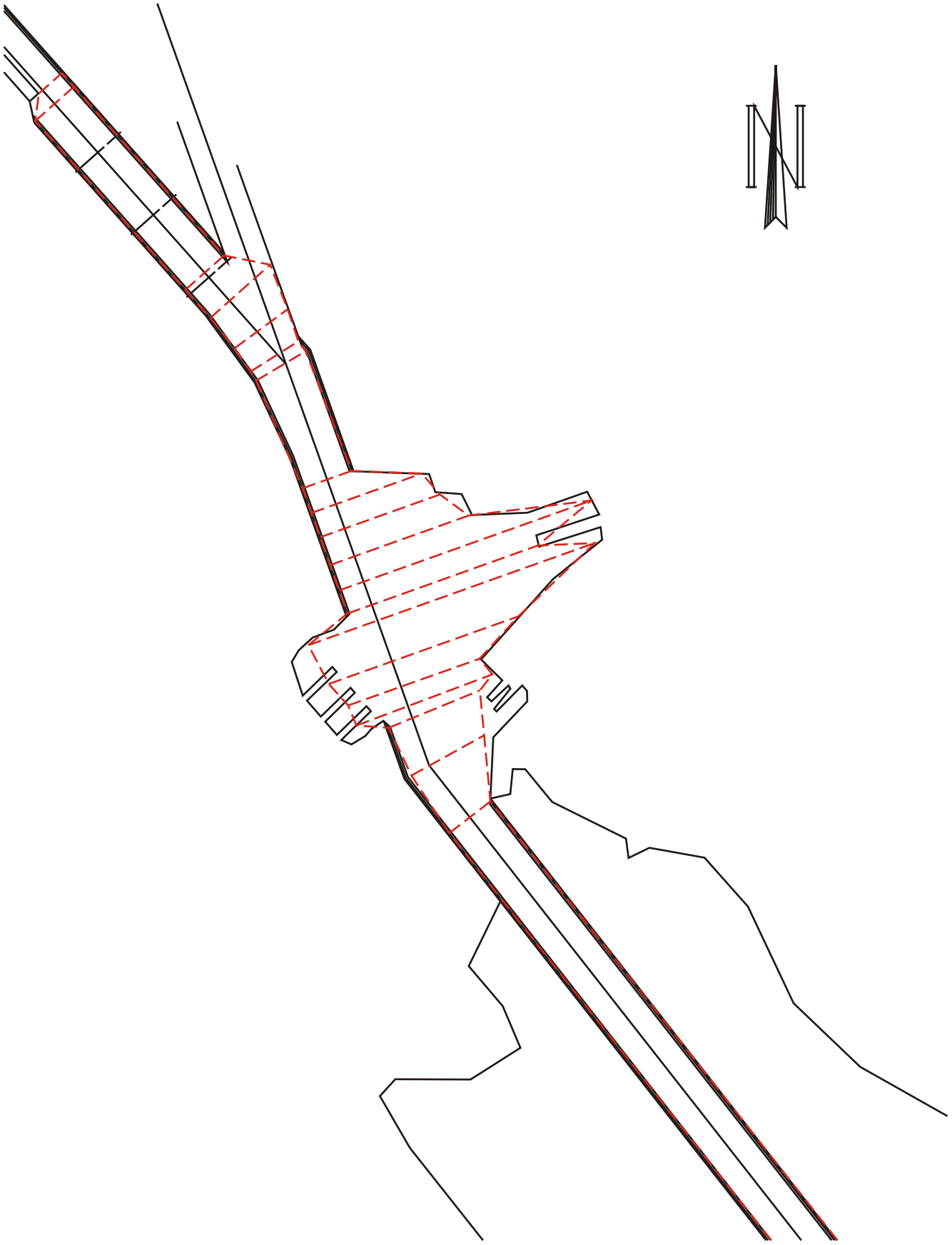
W.L. 02.0182

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 9
Distance in meter (m) Scale: 1/33000	TRAJECTORY FROM GATUN LAKE TO THE LOCK	



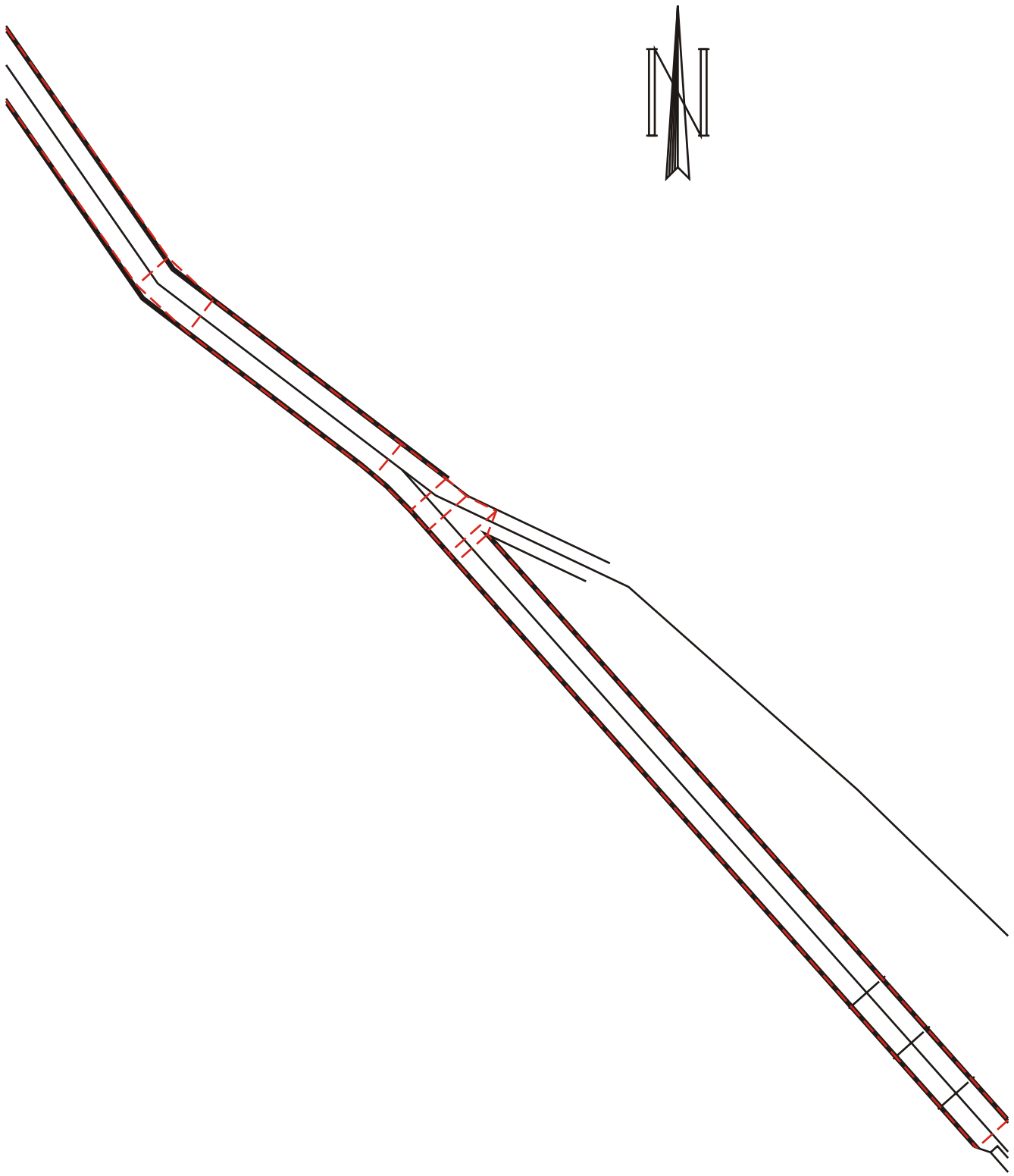
W.L. 02.0183

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 10
Scale: 1/25000	BANK SUCTION ALONG THE PACIFIC OCEAN TRAJECTORY	



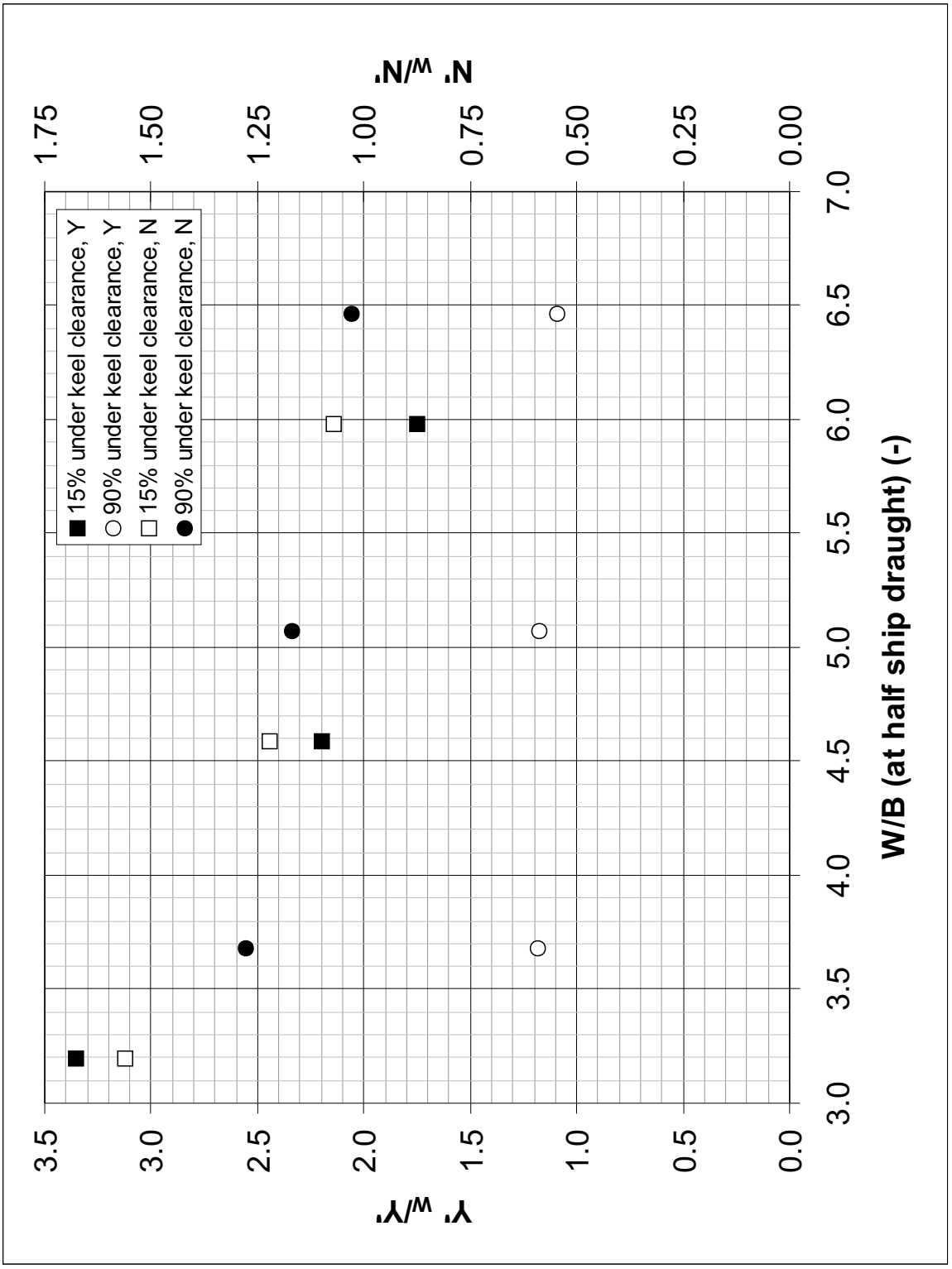
W.L. 02.0184

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 11
Scale: 1/33000	BANK SUCTION ALONG THE GATUN LAKE TRAJECTORY	

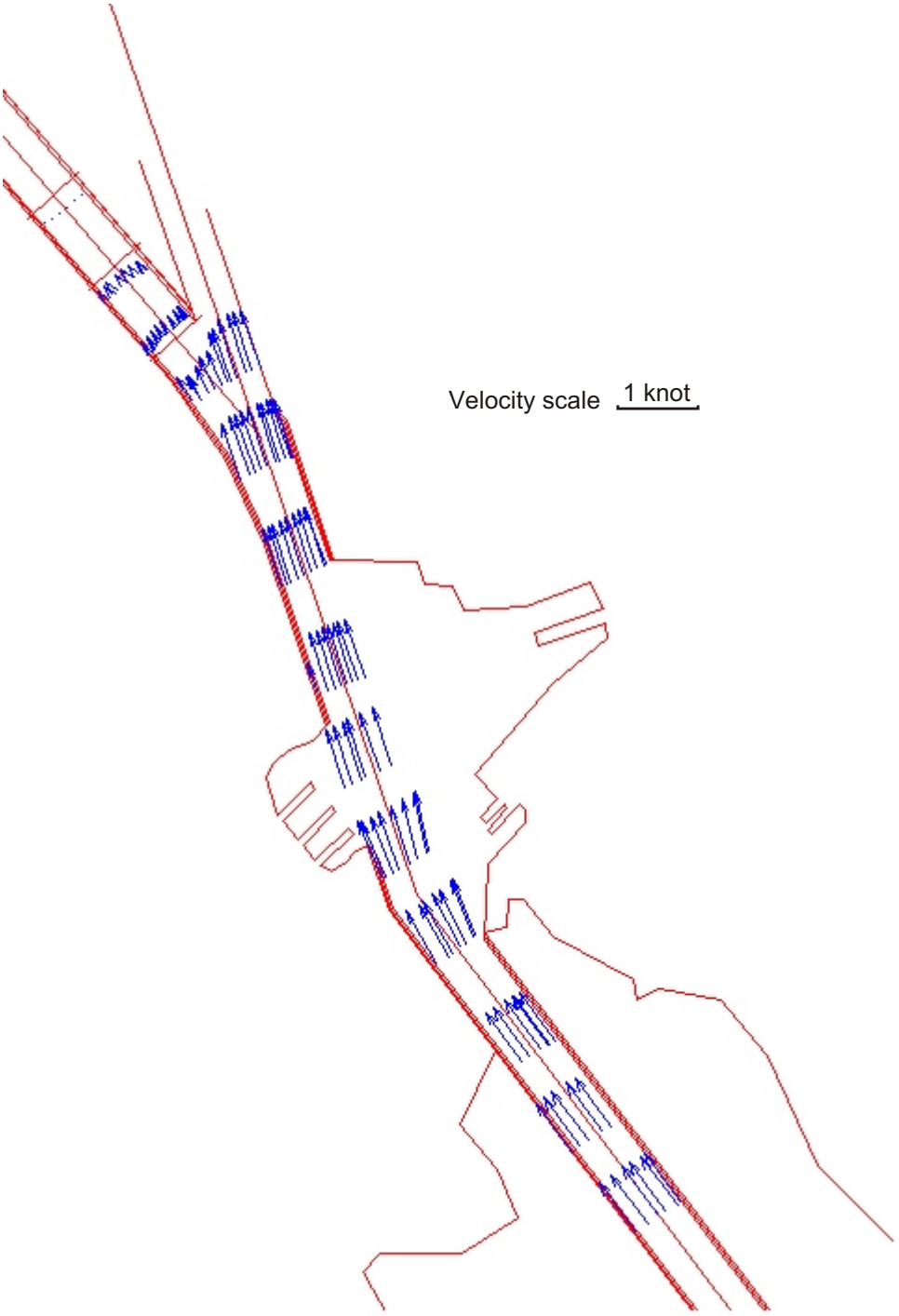


W.L. 02.0185

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 12
	INFLUENCE OF THE CHANNEL WIDTH ON COEFFICIENTS $Y_ß$ AND $N_ß$	



FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 13
	CURRENT PATTERN FROM THE PACIFIC OCEAN TO THE LOCK	

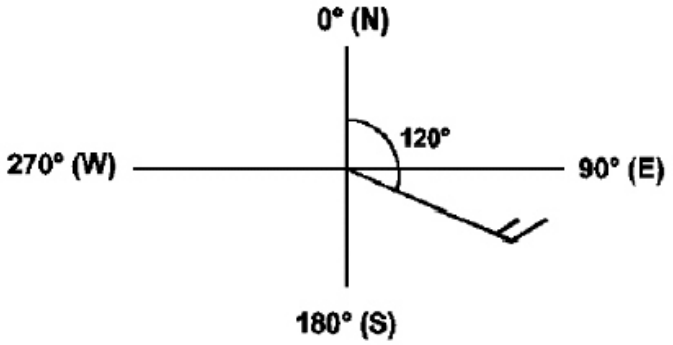


W.L. 02.0187

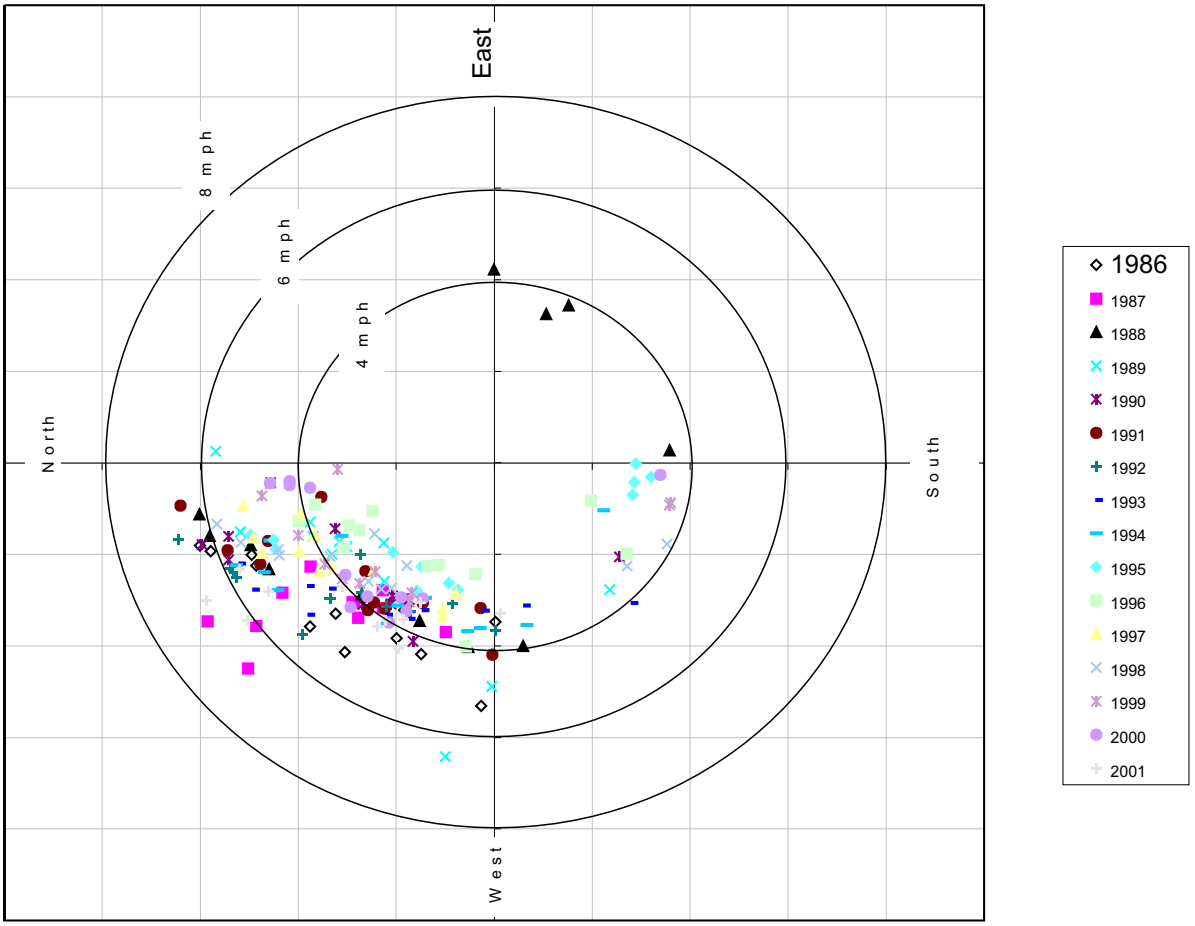
DEFINITION OF WIND DIRECTION

La dirección del viento:

Se representa en grados de 0 a 360 como se muestra en la siguiente figurra. En esta, 0 grados corresponde al Norte, 90 al Este, 180 al Sur, 270 al Oeste y 360 grados nuevamente al Norte. En la Fig se ha representado el viento con una dirección de 120 grados (aprox. del sureste), la punta de la flecha indica de donde viene el viento y las barbas como se verá a continuación la magnitud del viento, en este caso 15 nudos.

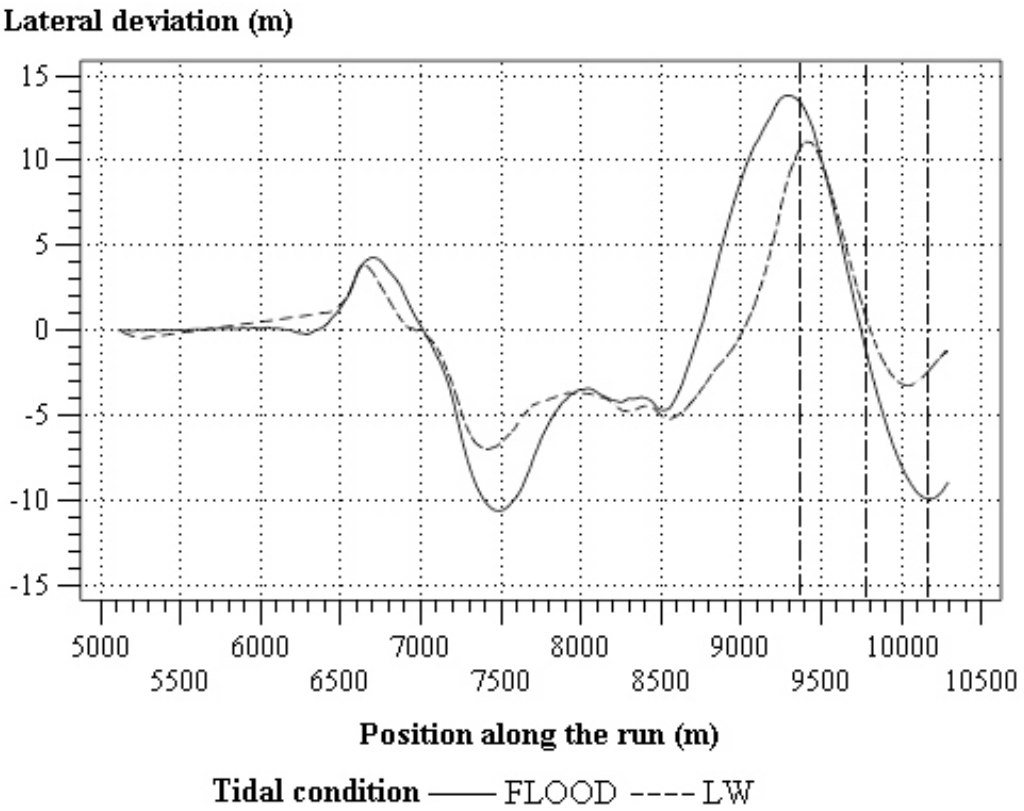


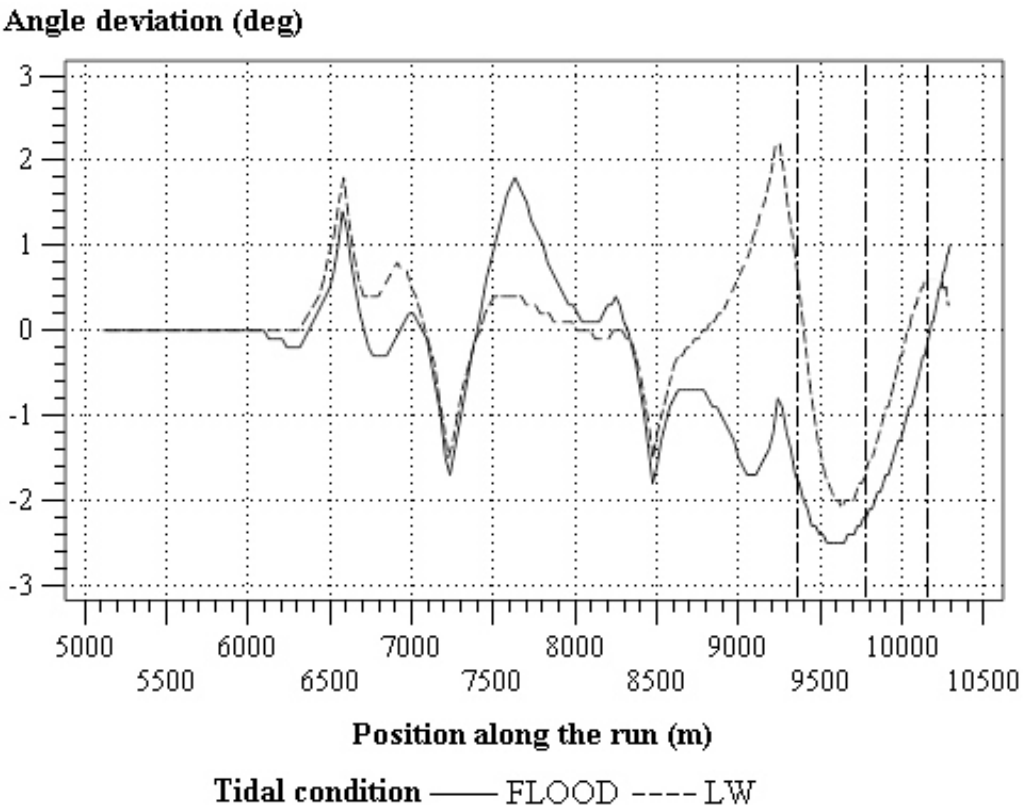
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 15
	MONTHLY AVERAGE WIND SPEEDS AND WIND DIRECTIONS	



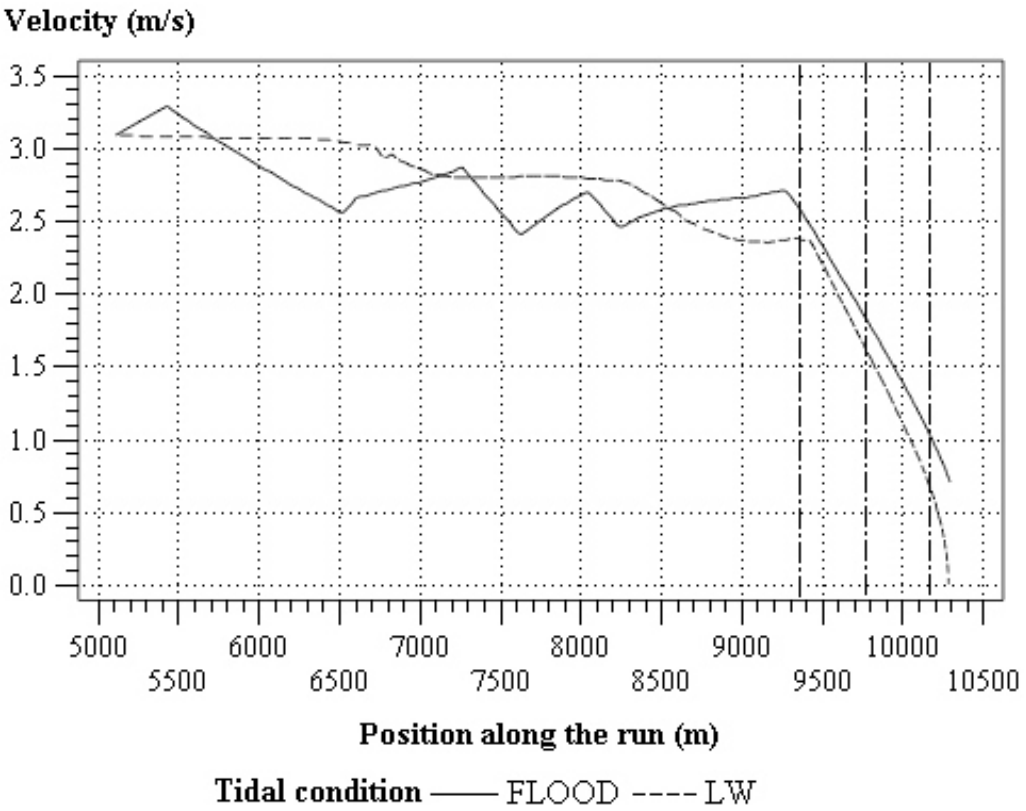
W.L. 02.0189

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 16a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 1: NO WIND	

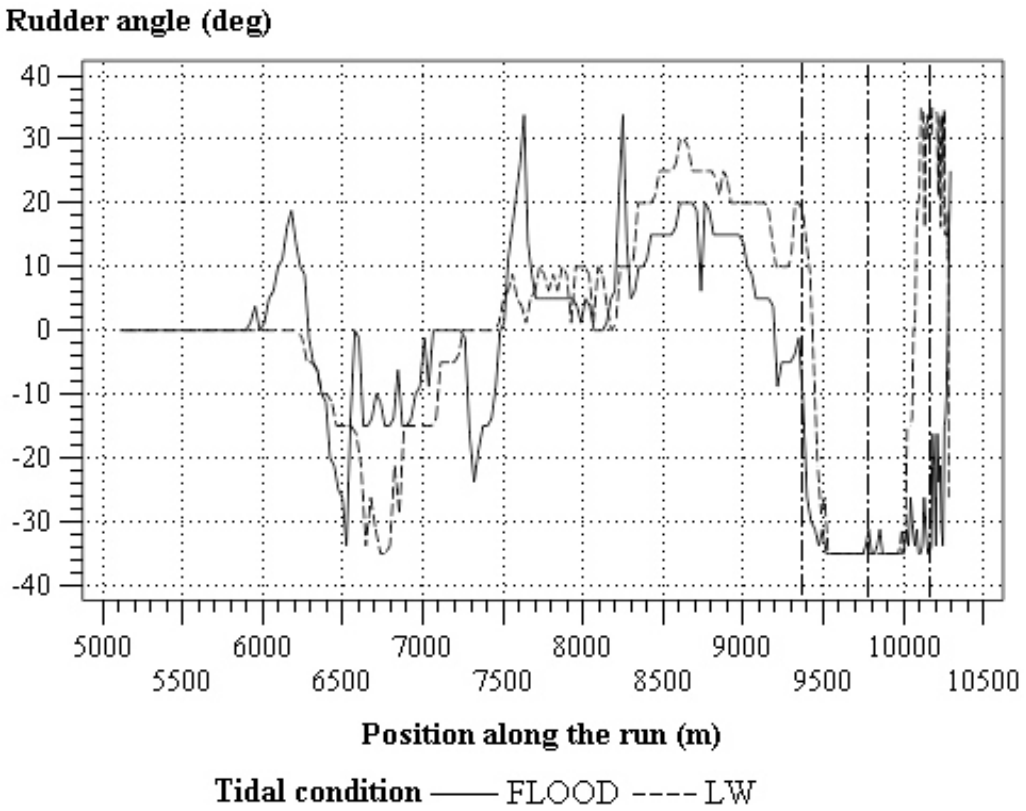


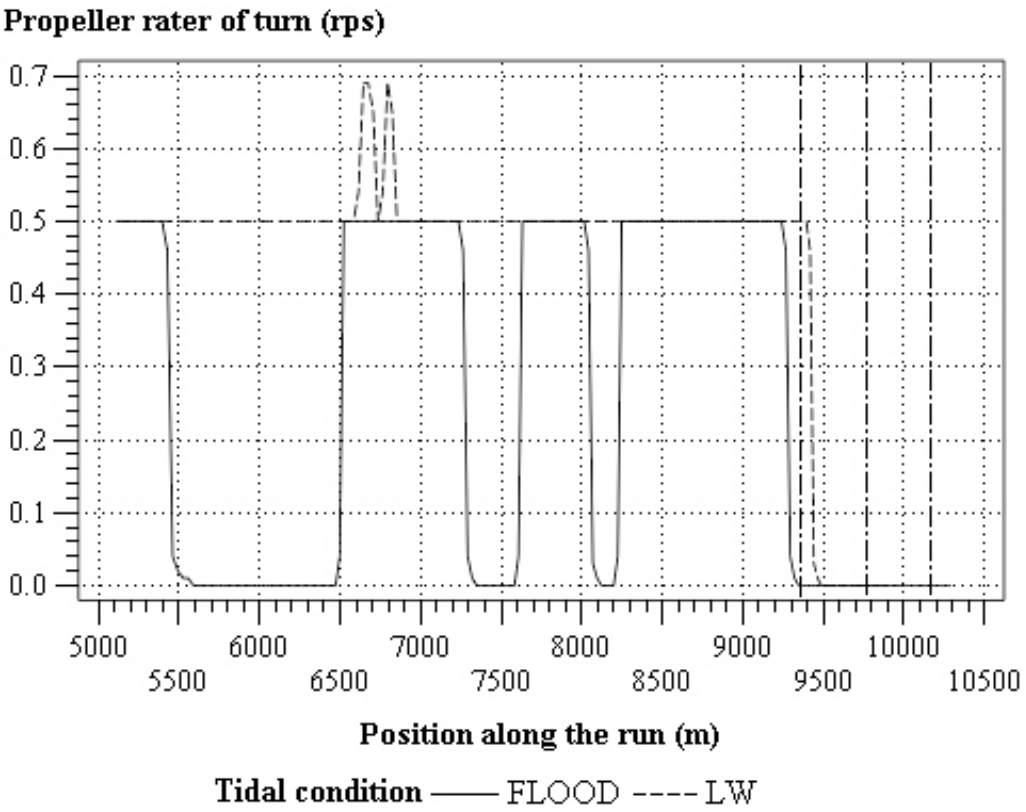


FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 16b
	SHIP'S VELOCITY FOR CONDITION 1: NO WIND	

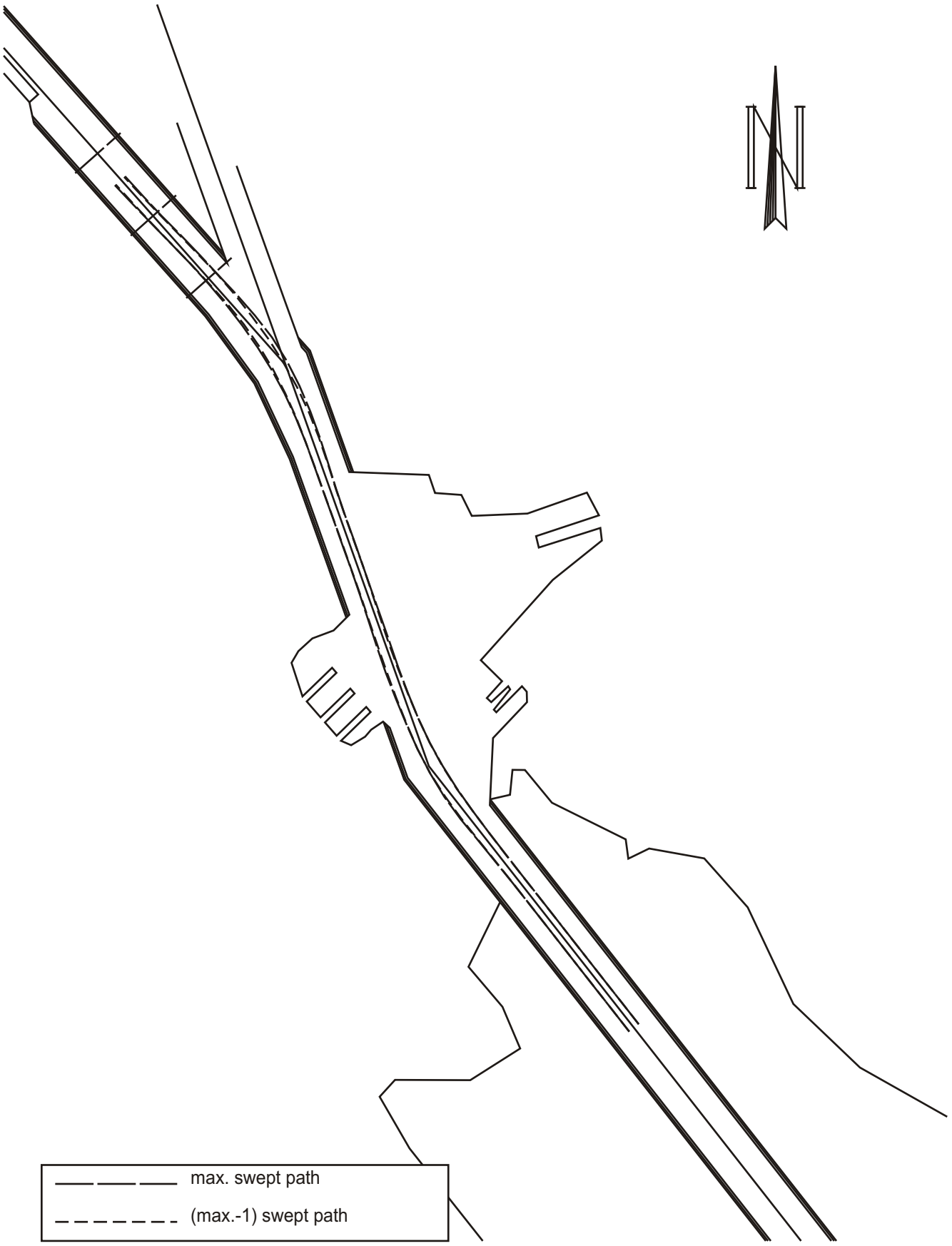


FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 16c
	RUDDER AND ENGINE COMMAND FOR CONDITION 1: NO WIND	



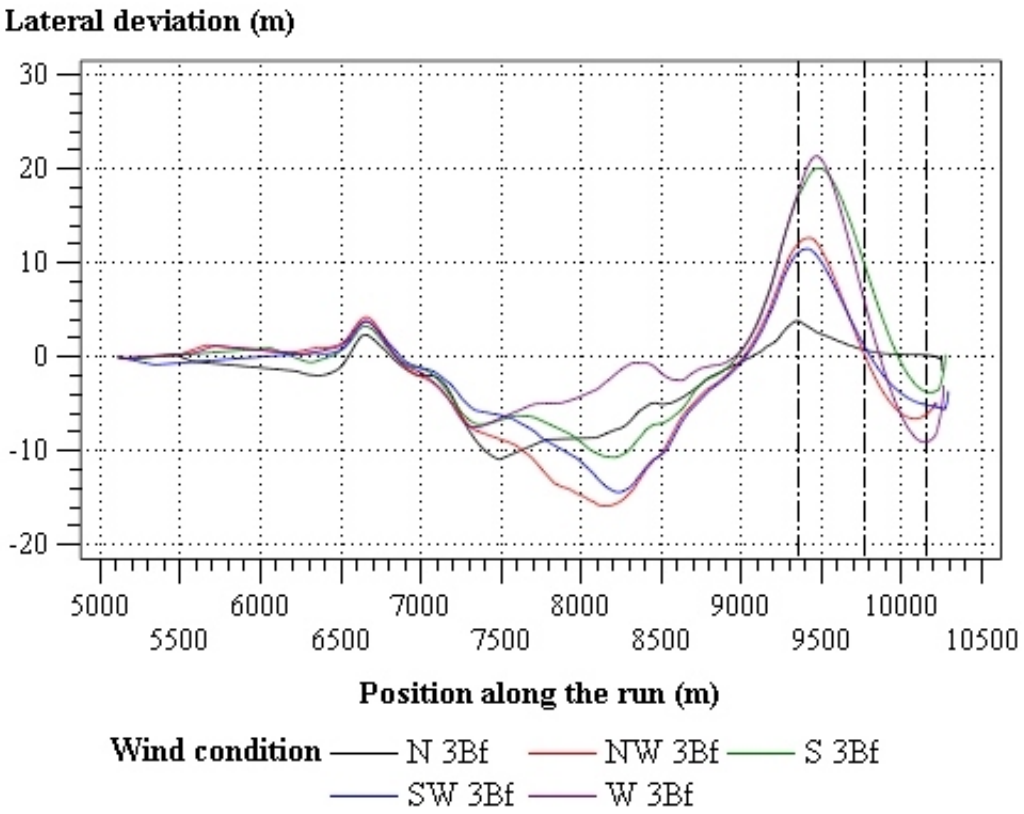


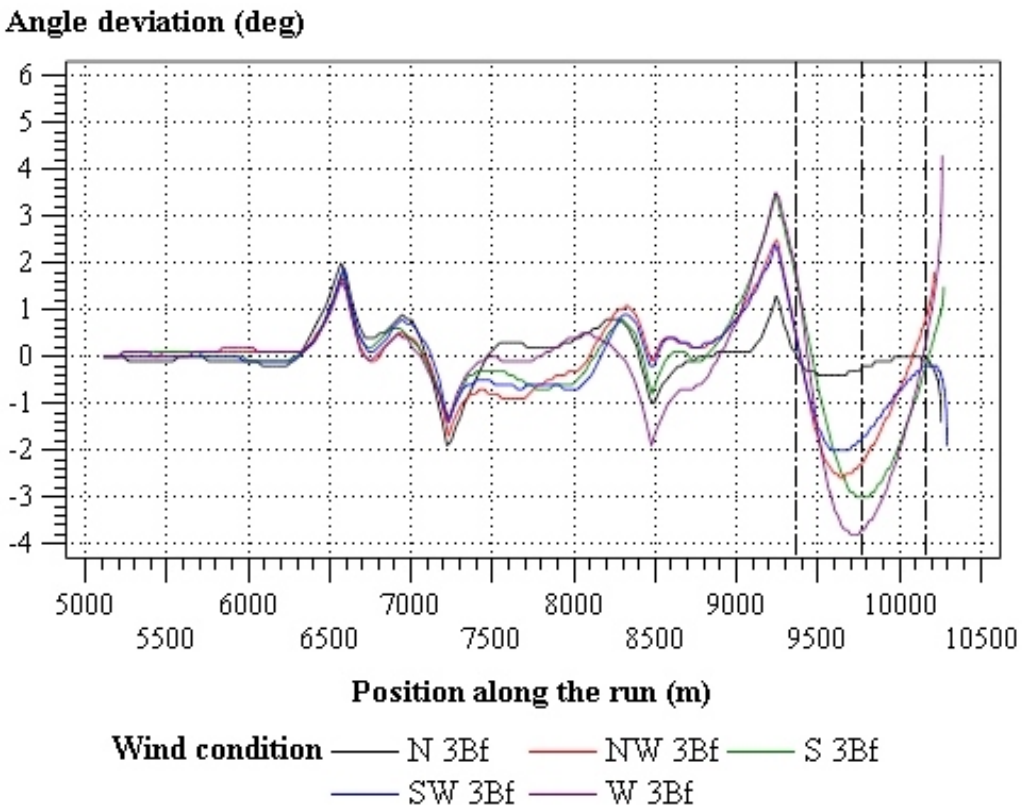
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 17
Scale: 1/25000	SWEPT PATH FOR CONDITION 1: NO WIND	



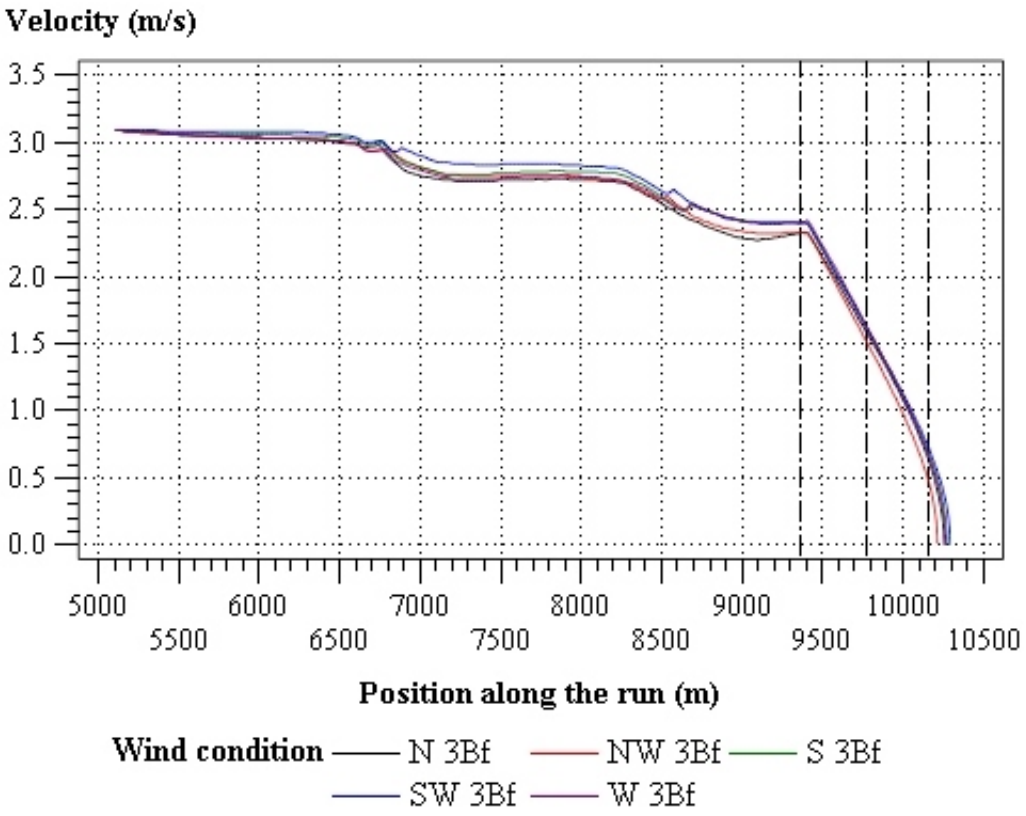
W.L. 02.0193

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 18a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 2: LOW WATER, 3 BEAUFORT WIND	



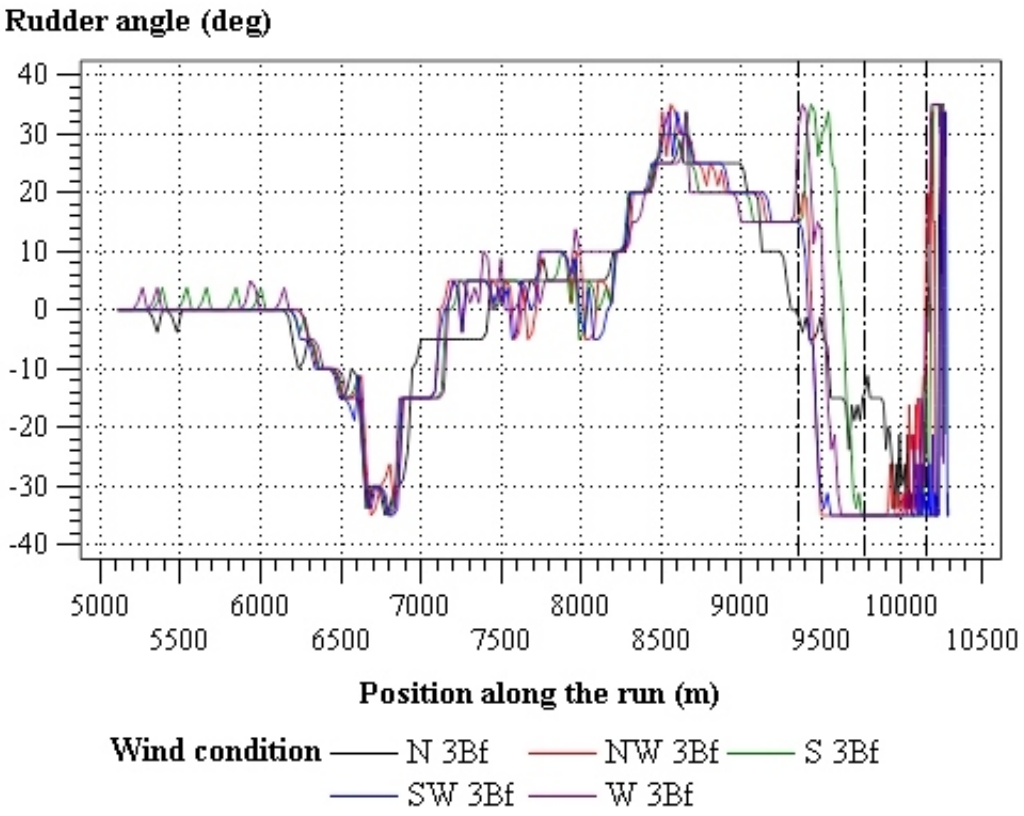


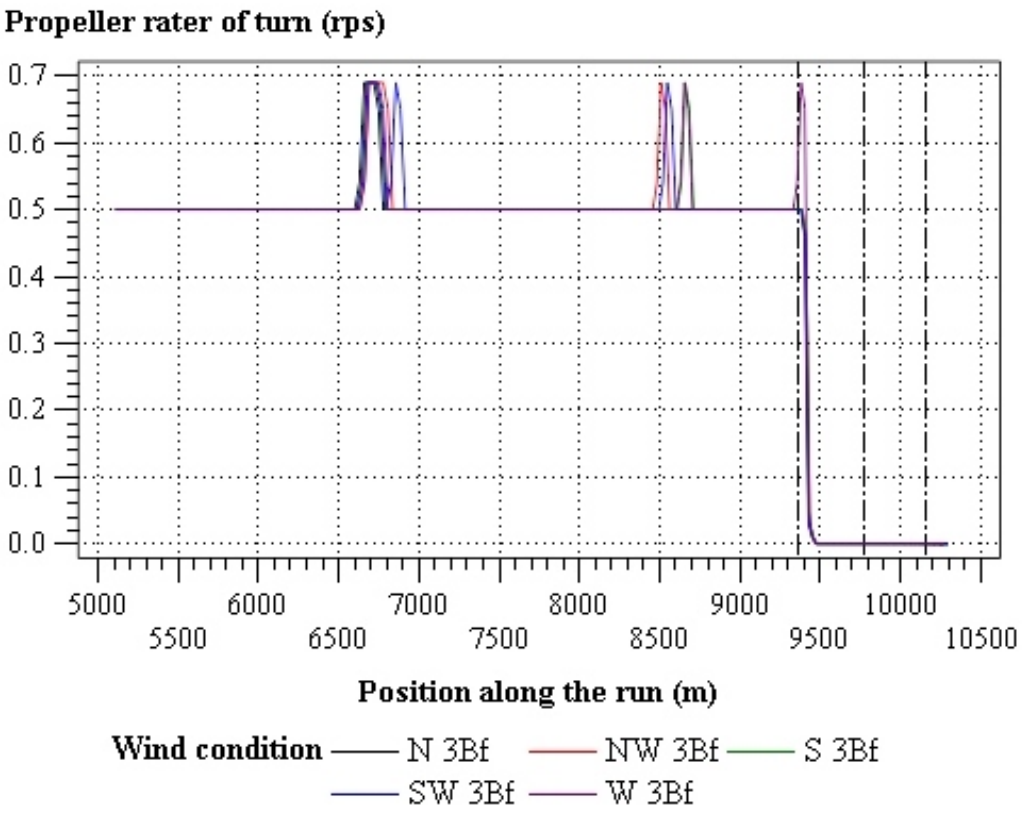
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 18b
	SHIP'S VELOCITY FOR CONDITION 2: LOW WATER, 3 BEAUFORT WIND	



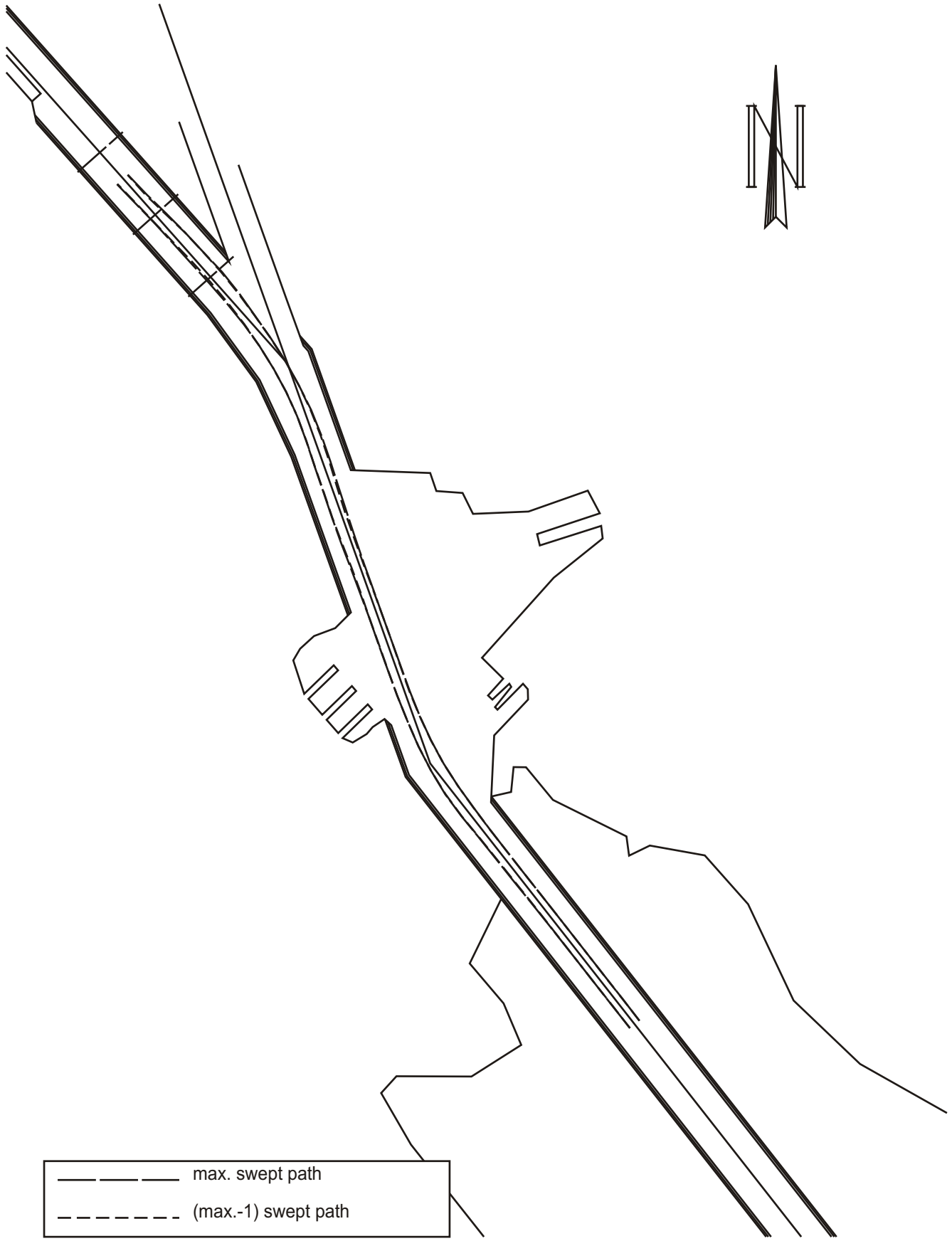
W.L. 02.0195

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 18c
	RUDDER AND ENGINE COMMAND FOR CONDITION 2: LOW WATER, 3 BEAUFORT WIND	



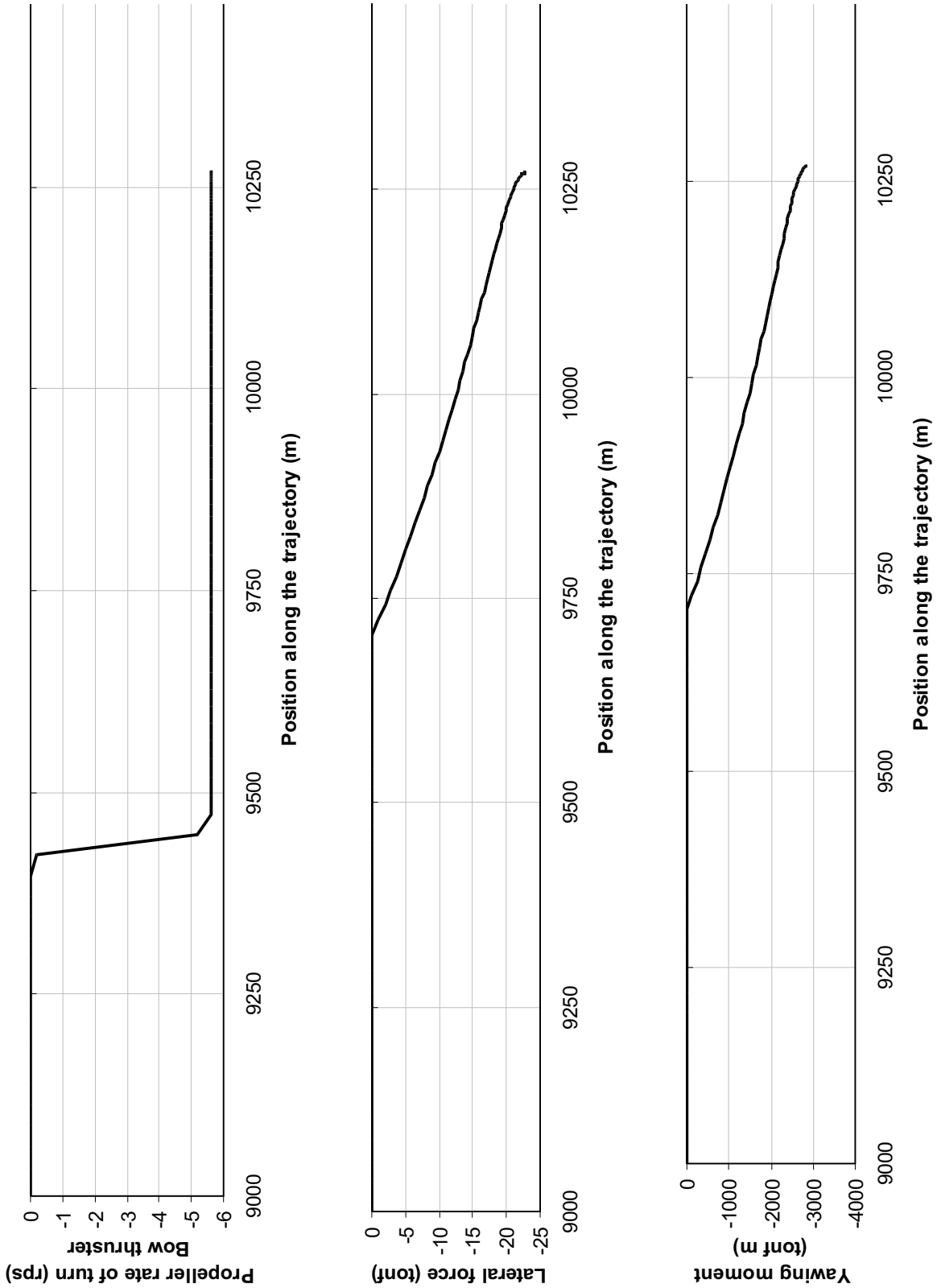


FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 19
Scale: 1/25000	SWEPT PATH FOR CONDITION 2: LOW WATER, 3 BEAUFORT WIND	



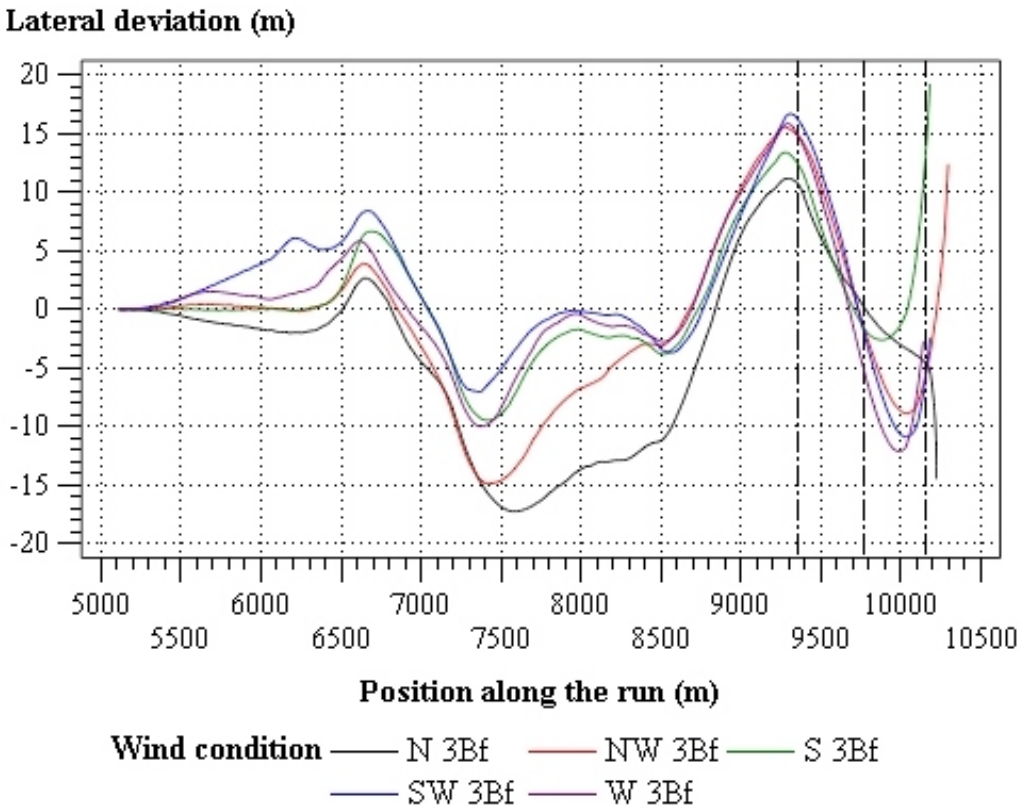
W.L. 02.0197

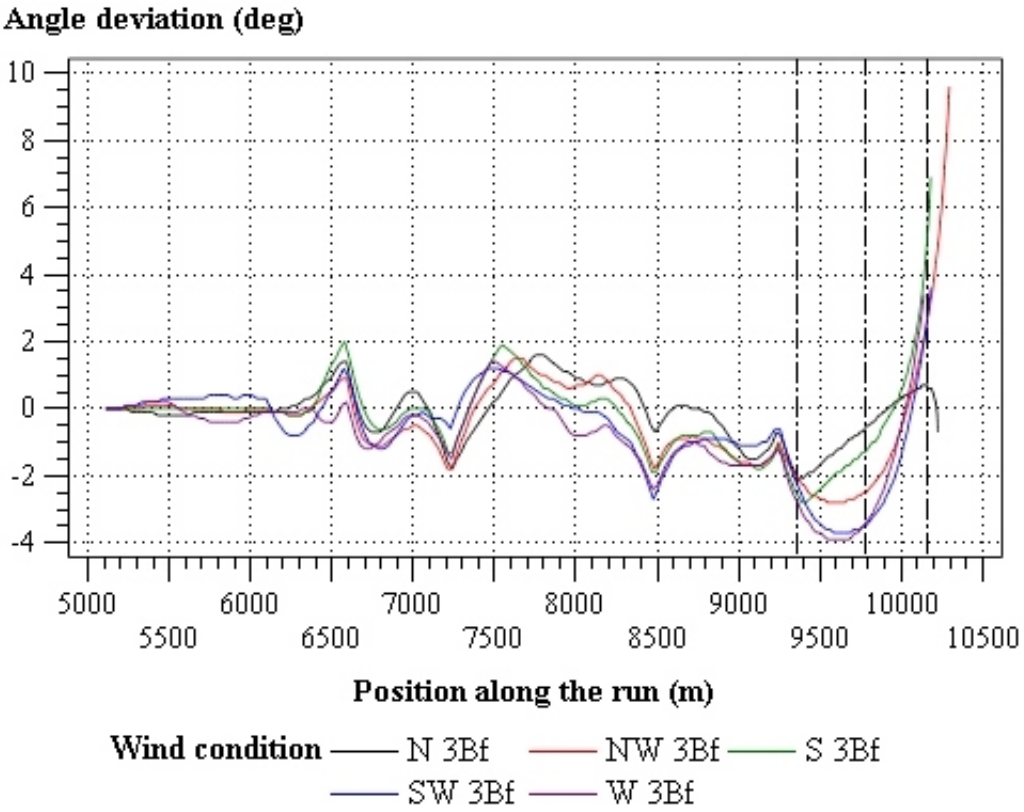
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 20
	LATERAL FORCE AND YAWING MOMENT DELIVERED BY THE BOW THRUSTER DURING A SIMULATION RUN	



W.L. 02.0198

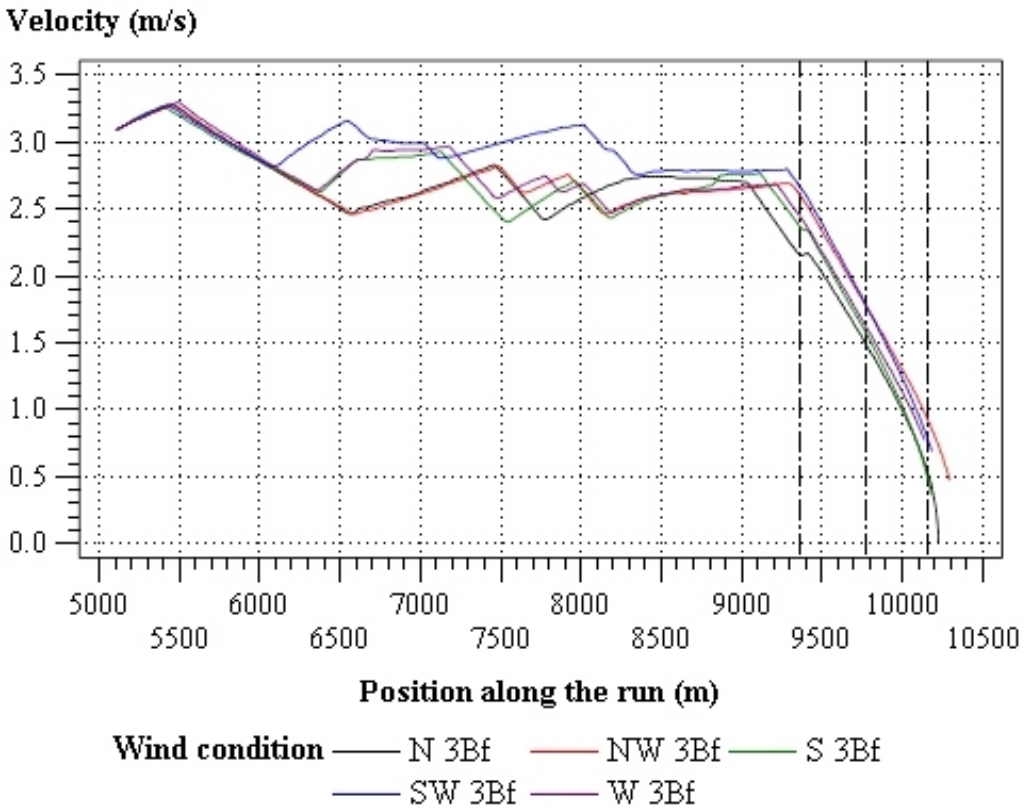
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 21a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 3: RISING TIDE, 3 BEAUFORT WIND	





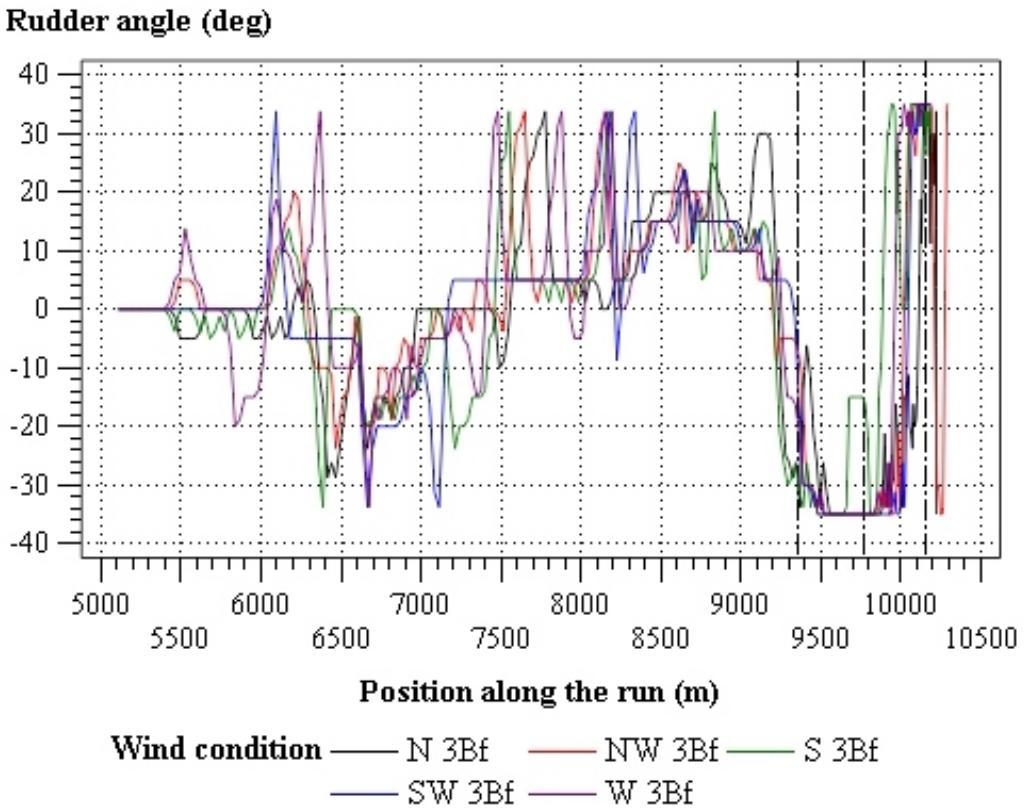
W.L. 02.0199

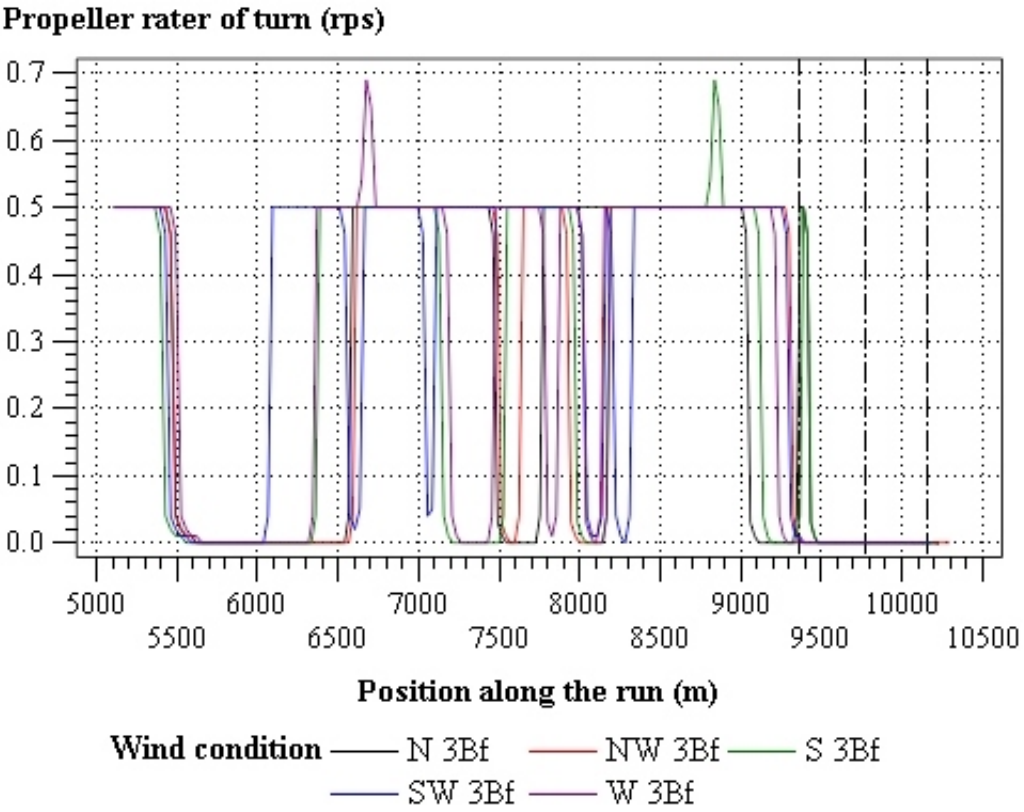
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 21b
	SHIP'S VELOCITY FOR CONDITION 3: RISING TIDE, 3 BEAUFORT WIND	



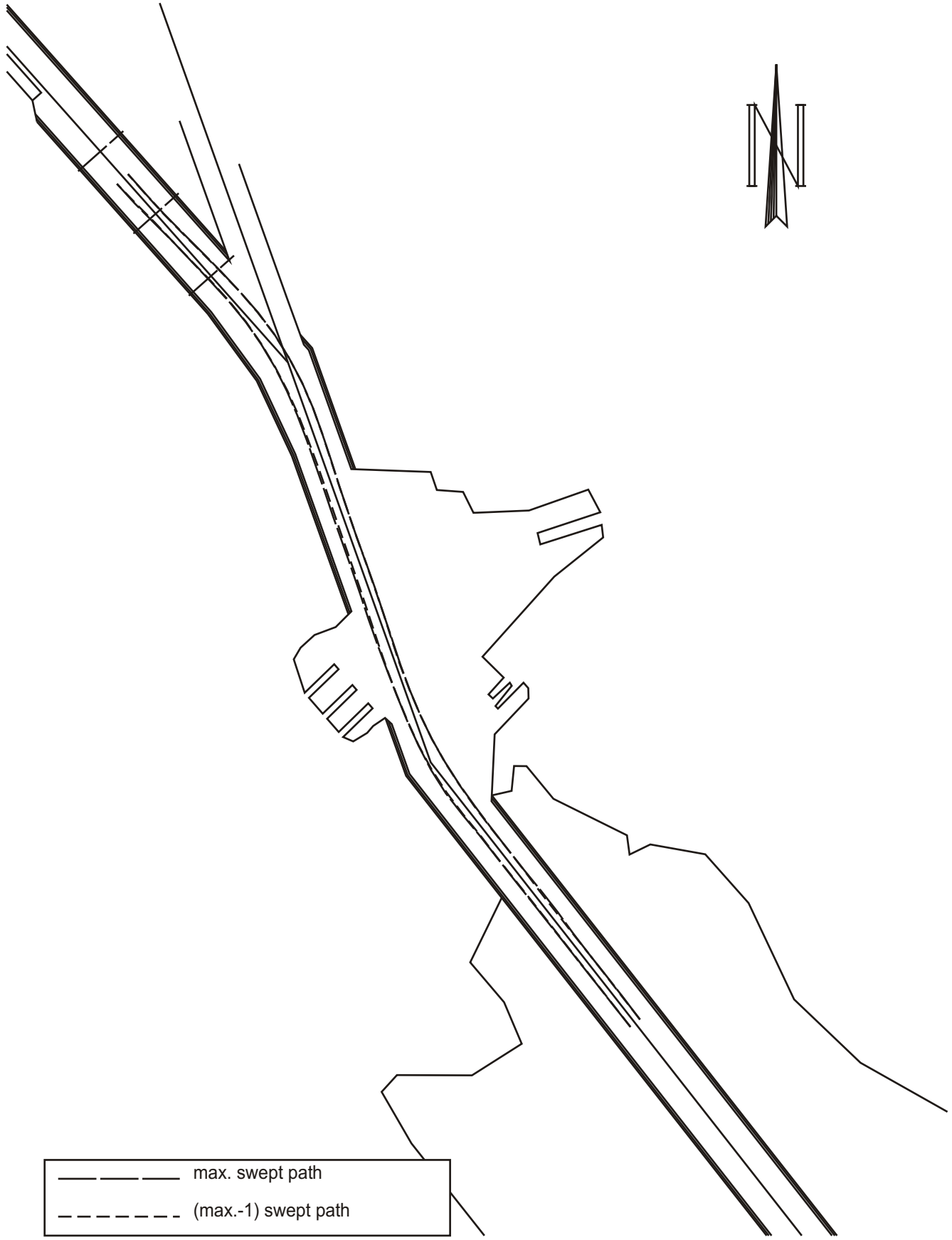
W.L. 02.0200

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 21c
	RUDDER AND ENGINE COMMAND FOR CONDITION 3: RISING TIDE, 3 BEAUFORT WIND	



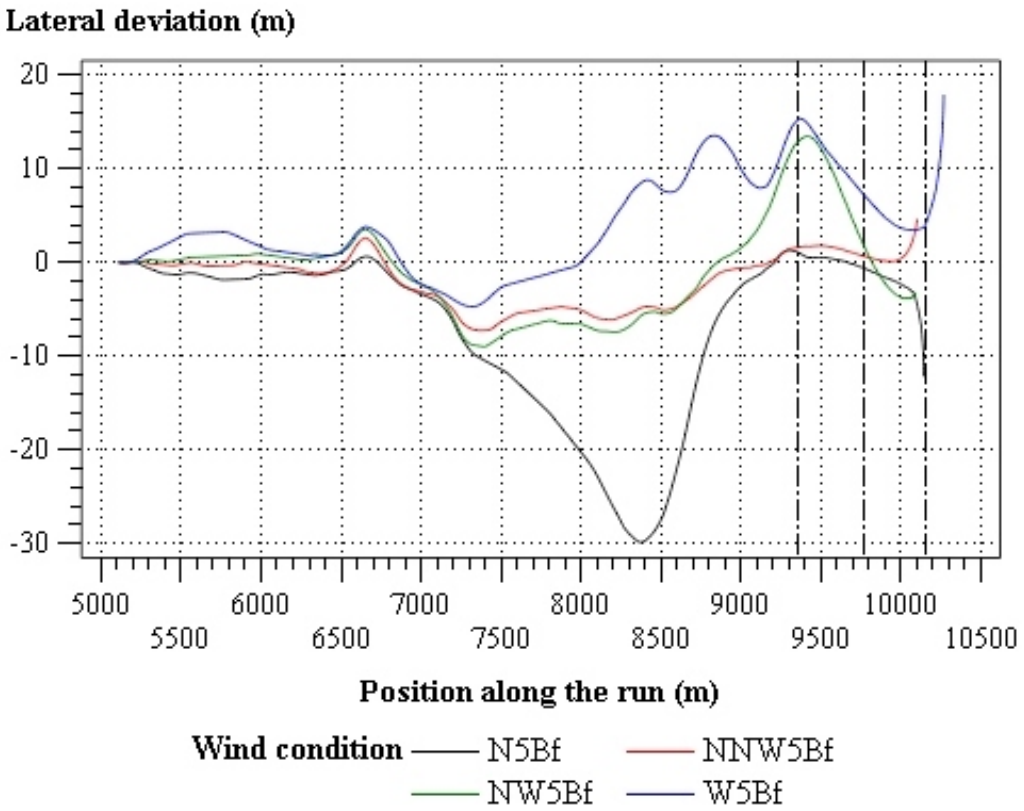


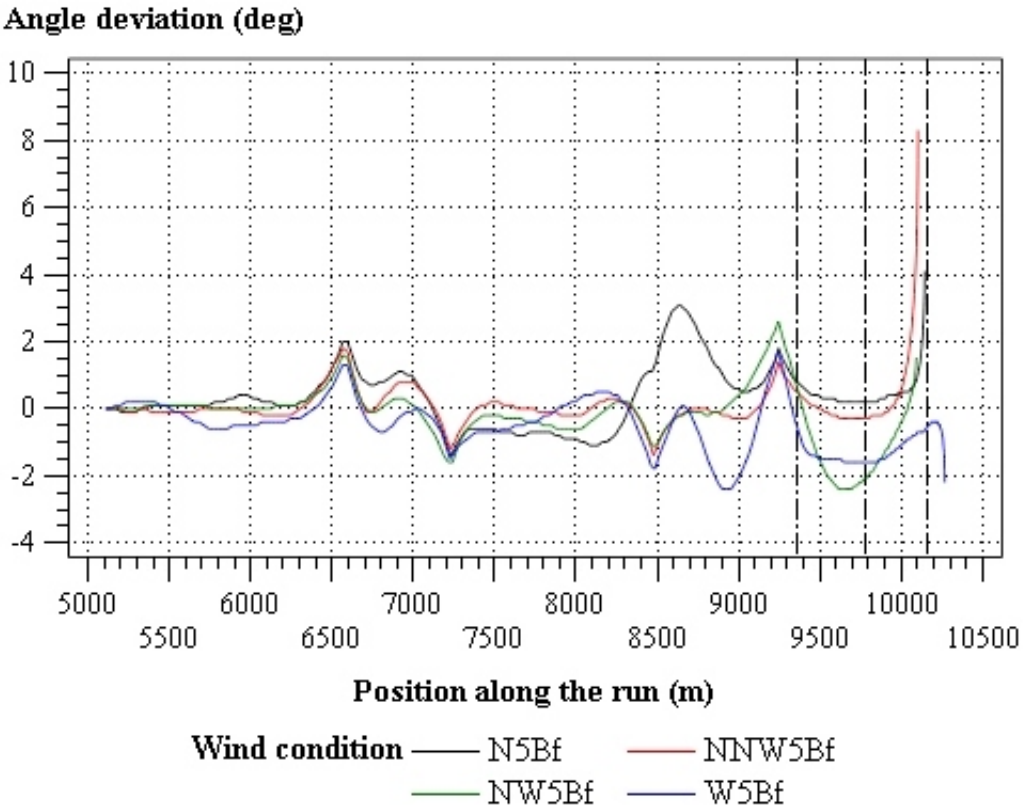
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 22
Scale: 1/25000	SWEPT PATH FOR CONDITION 3: RISING TIDE, 3 BEAUFORT WIND	



W.L. 02.0202

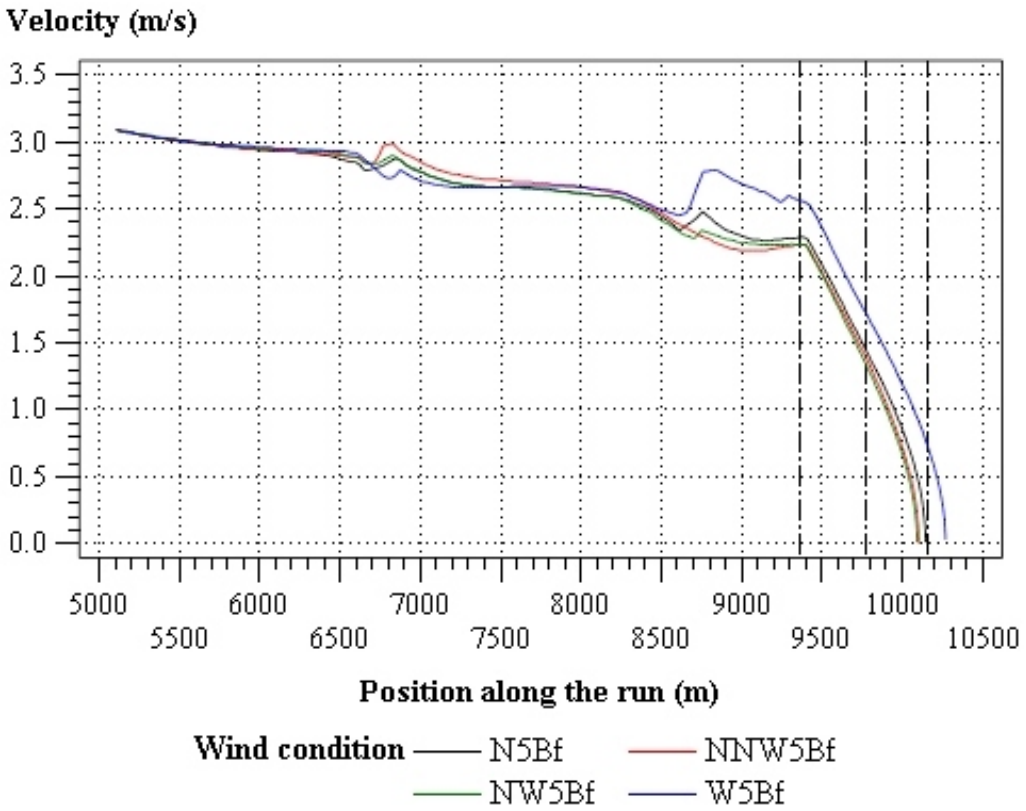
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 23a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 4: LOW WATER, 5 BEAUFORT WIND	





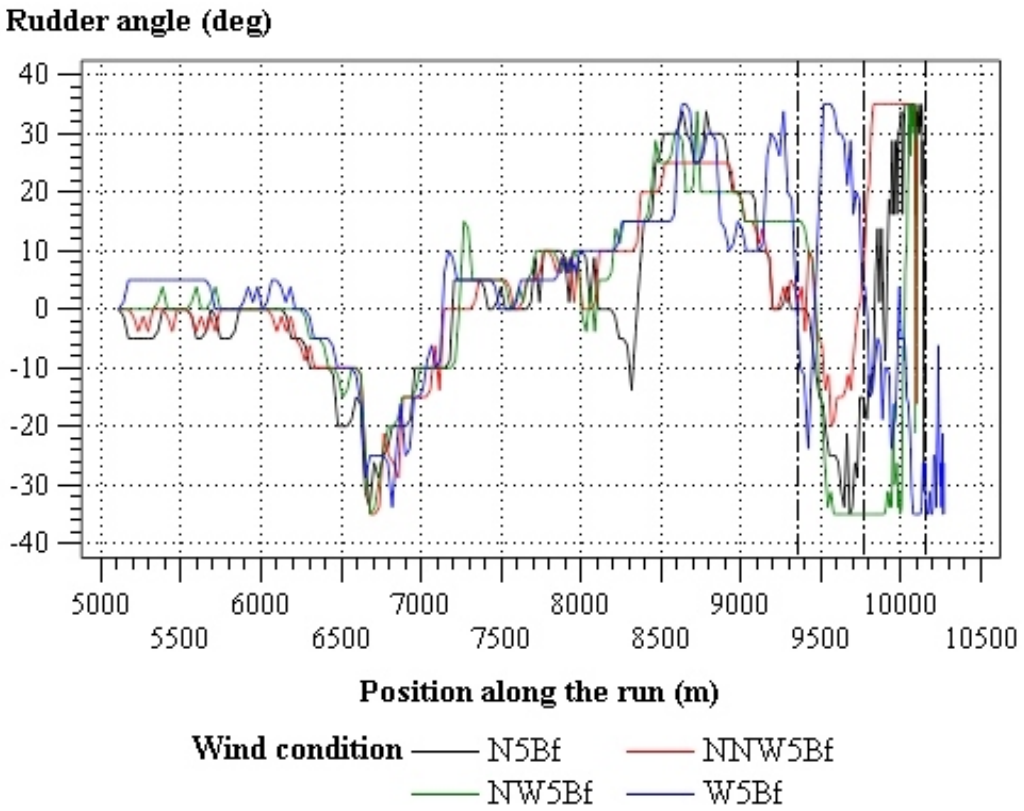
W.L. 02.0203

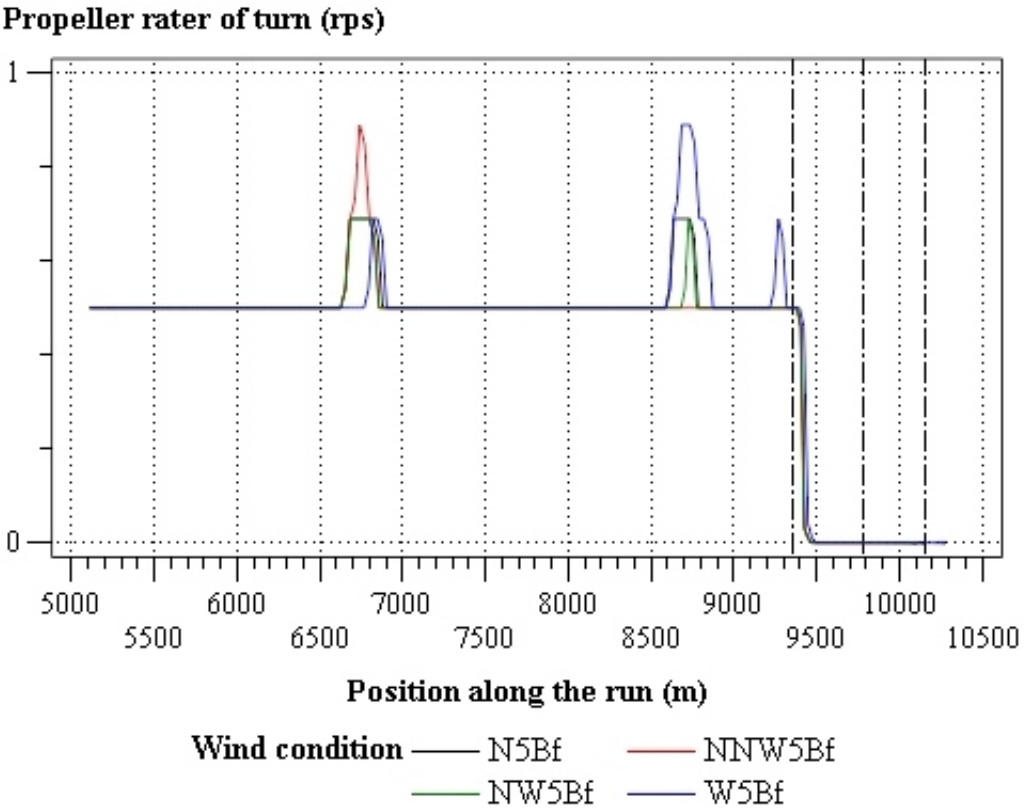
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 23b
	SHIP'S VELOCITY FOR CONDITION 4: LOW WATER, 5 BEAUFORT WIND	



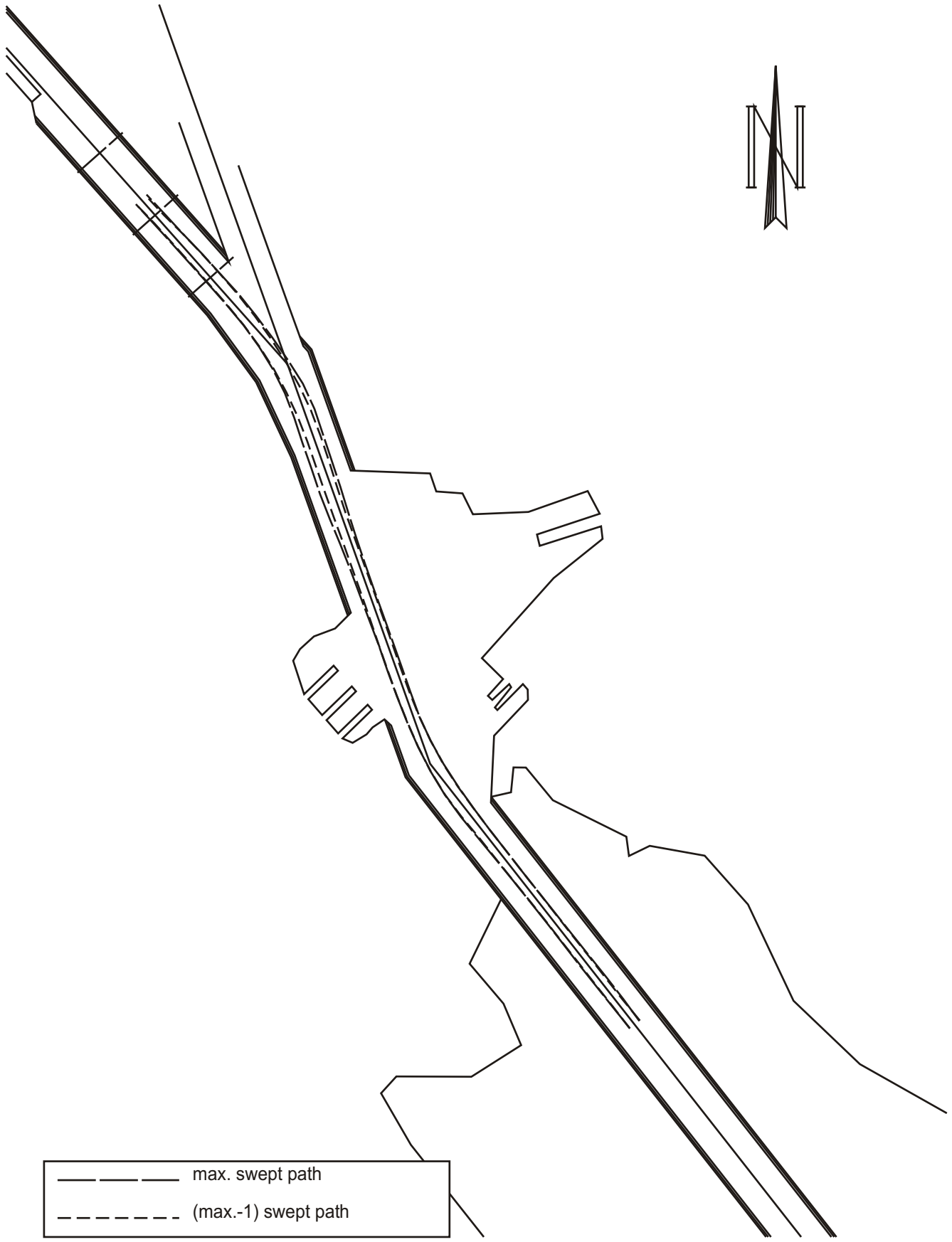
W.L. 02.0204

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 23c
	RUDDER AND ENGINE COMMAND FOR CONDITION 4: LOW WATER, 5 BEAUFORT WIND	



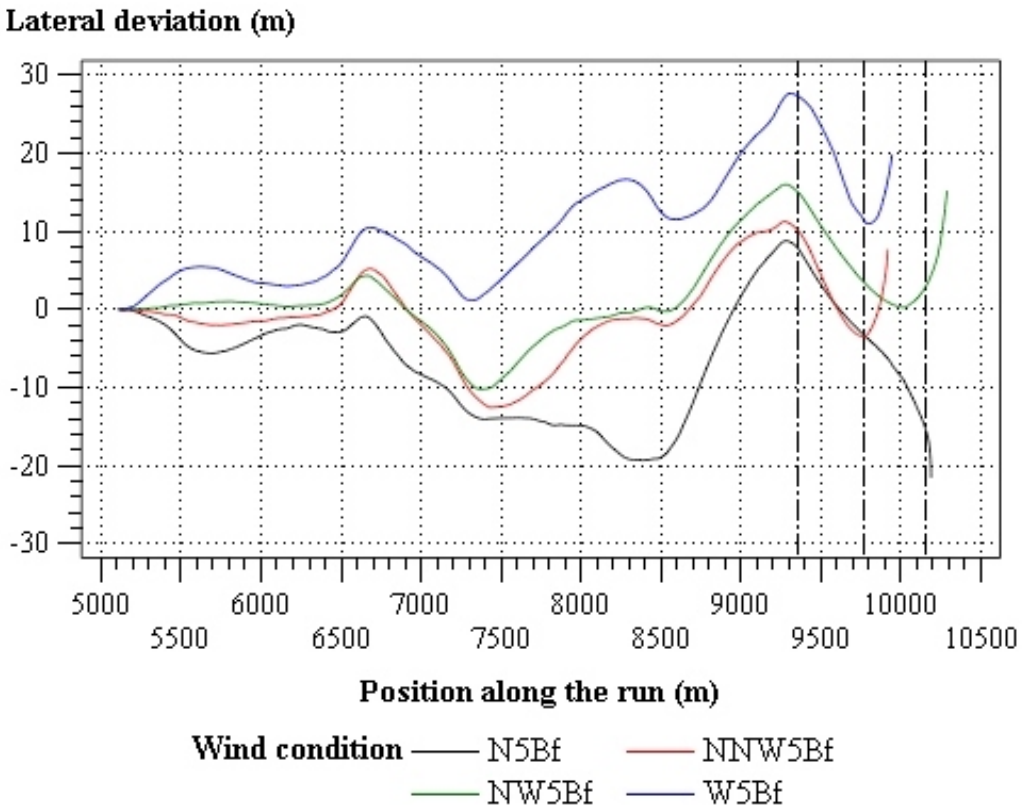


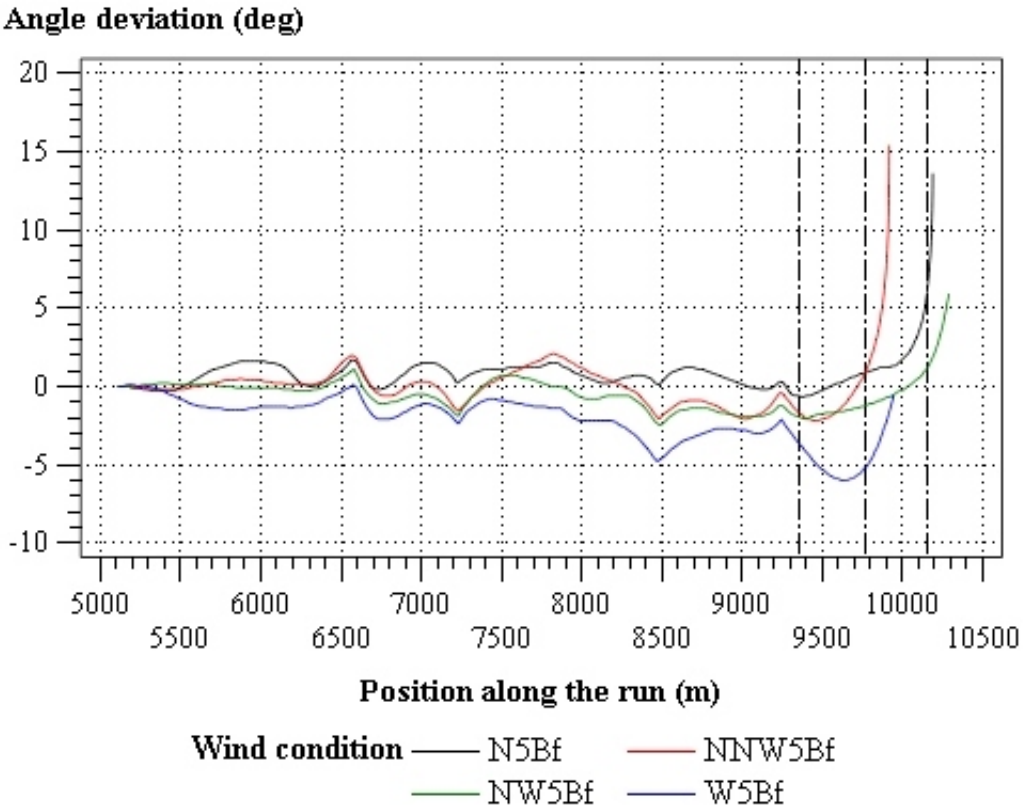
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 24
Scale: 1/25000	SWEPT PATH FOR CONDITION 4: LOW WATER, 5 BEAUFORT WIND	



W.L. 02.0206

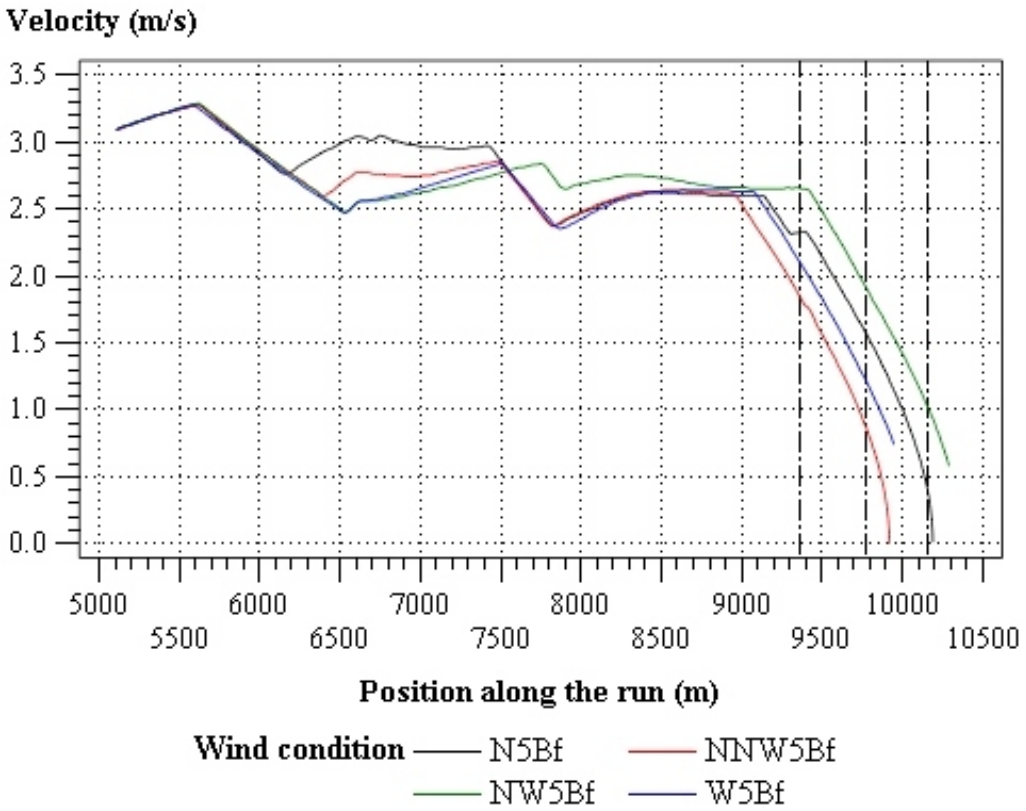
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 25a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 5: RISING TIDE, 5 BEAUFORT WIND	



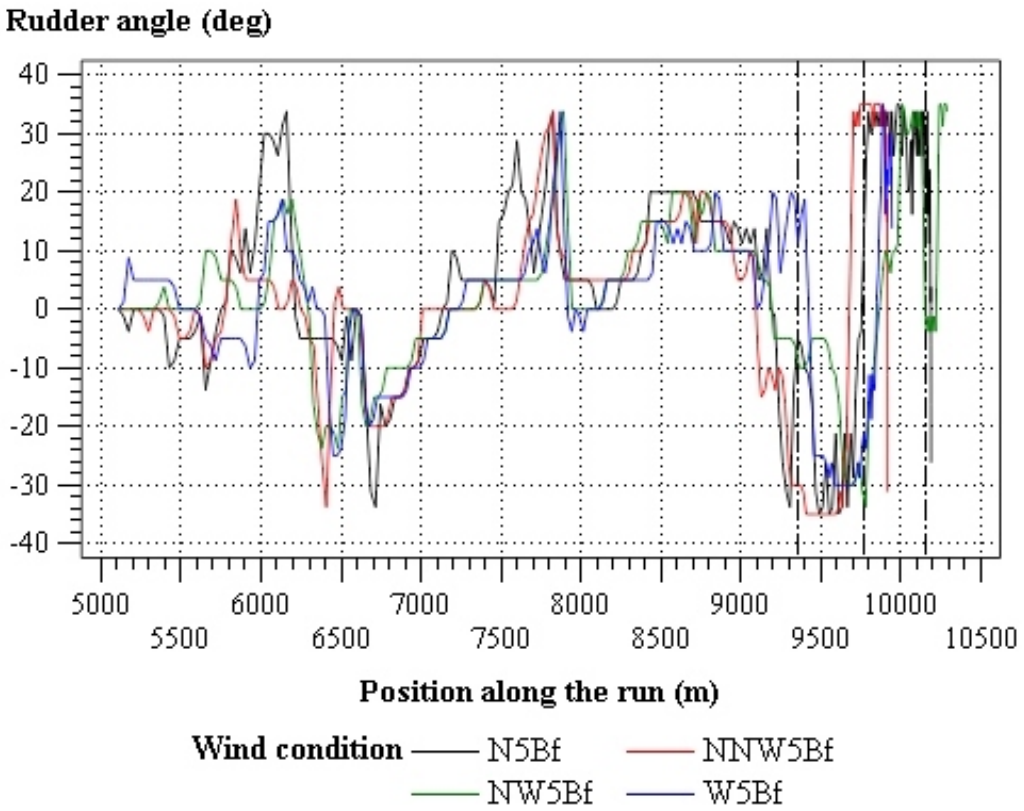


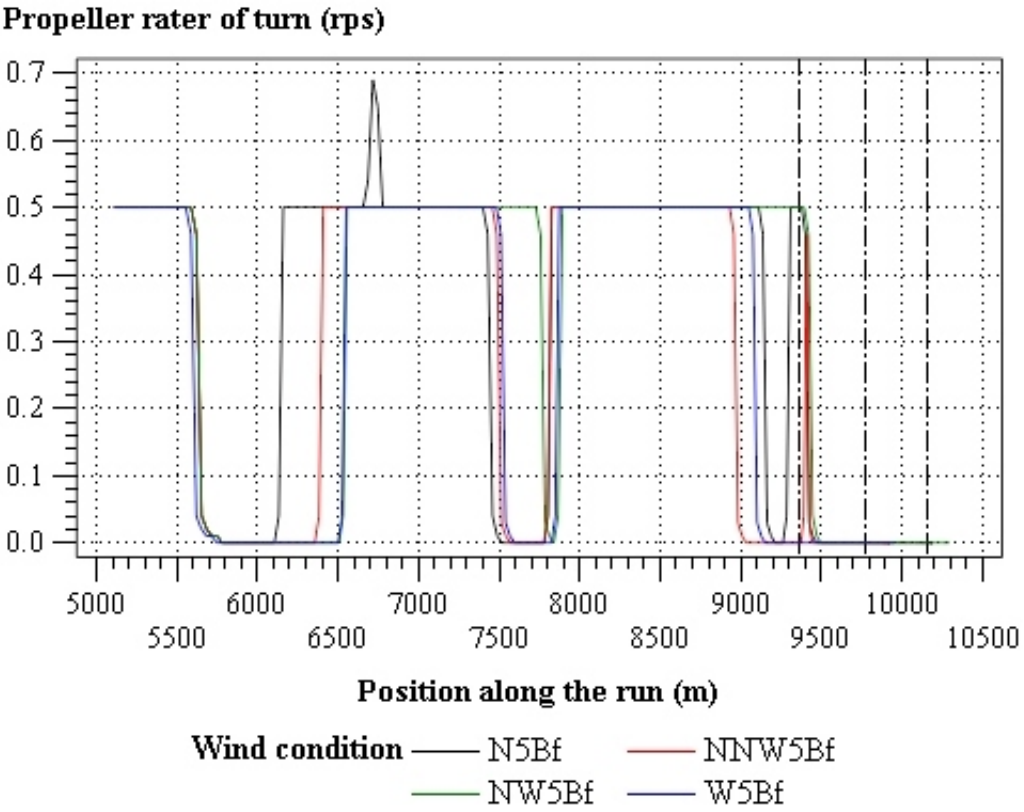
W.L. 02.0207

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 25b
	SHIP'S VELOCITY FOR CONDITION 5: RISING TIDE, 5 BEAUFORT WIND	

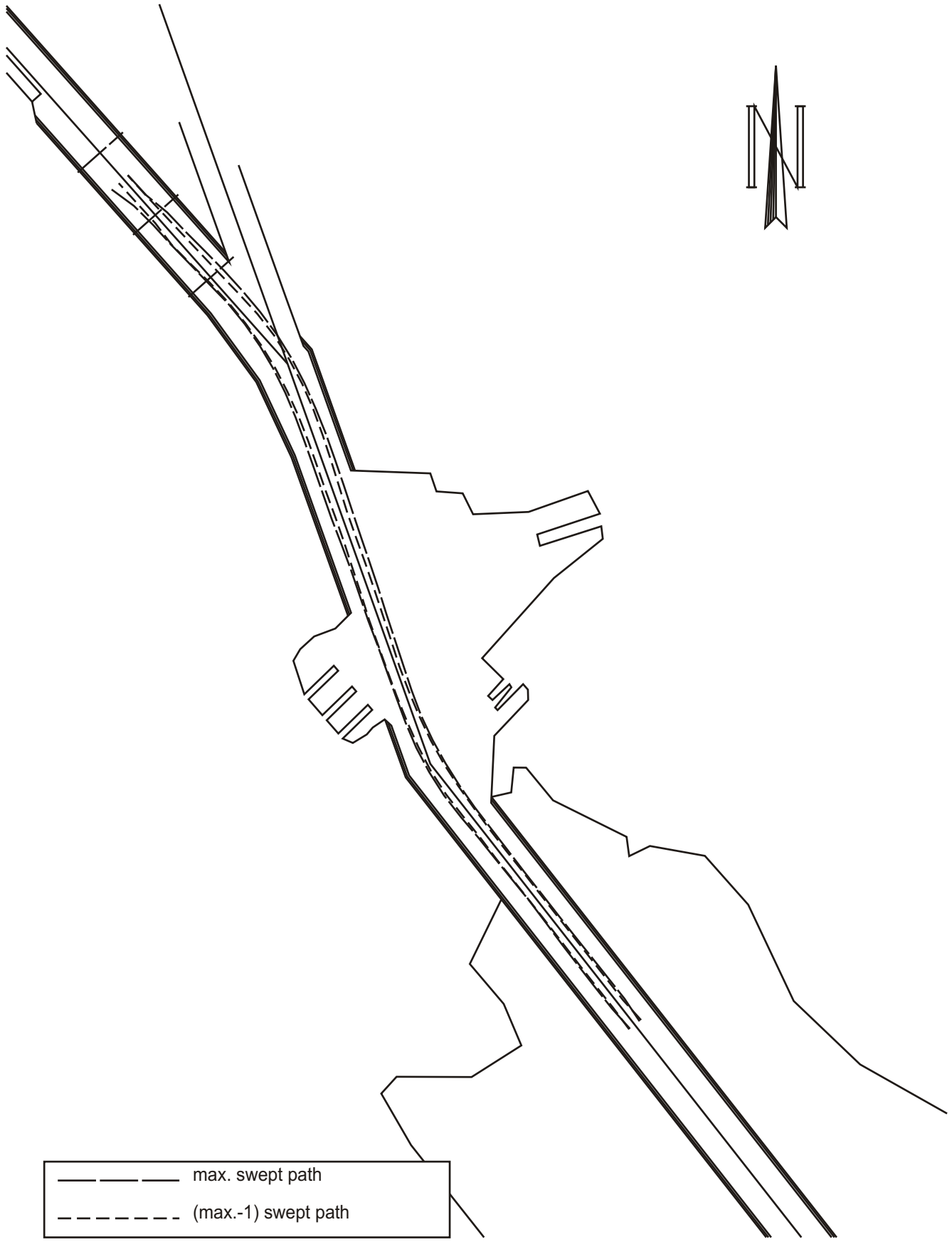


FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 25c
	RUDDER AND ENGINE COMMAND FOR CONDITION 5: RISING TIDE, 5 BEAUFORT WIND	



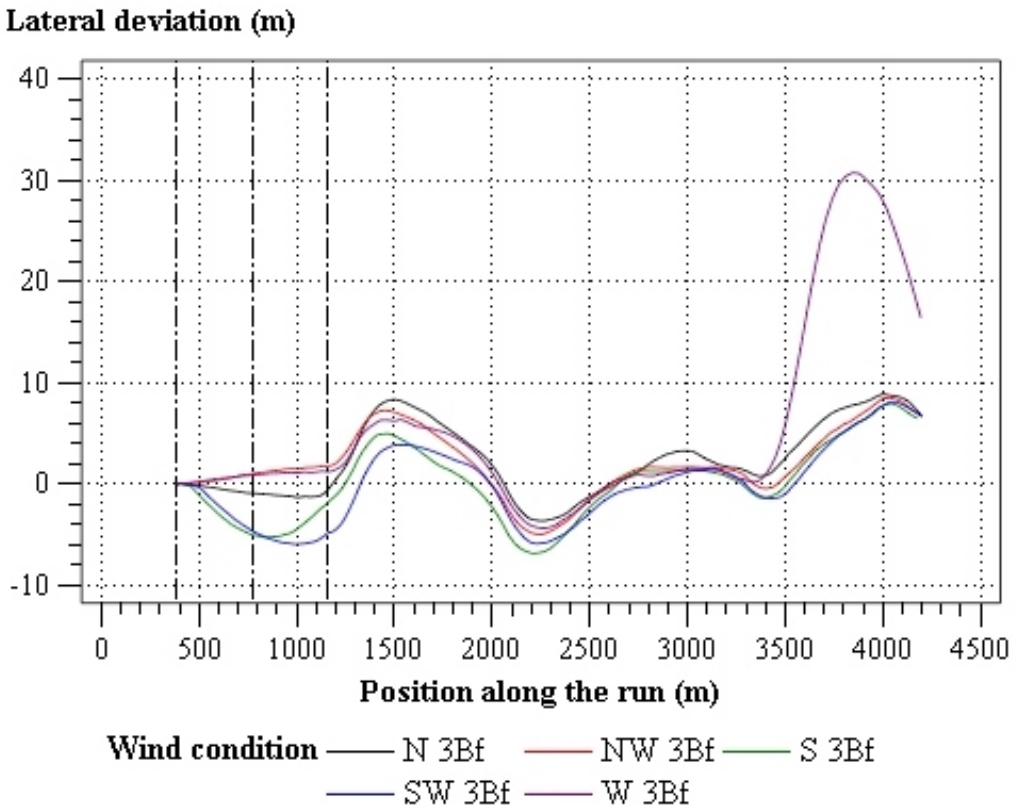


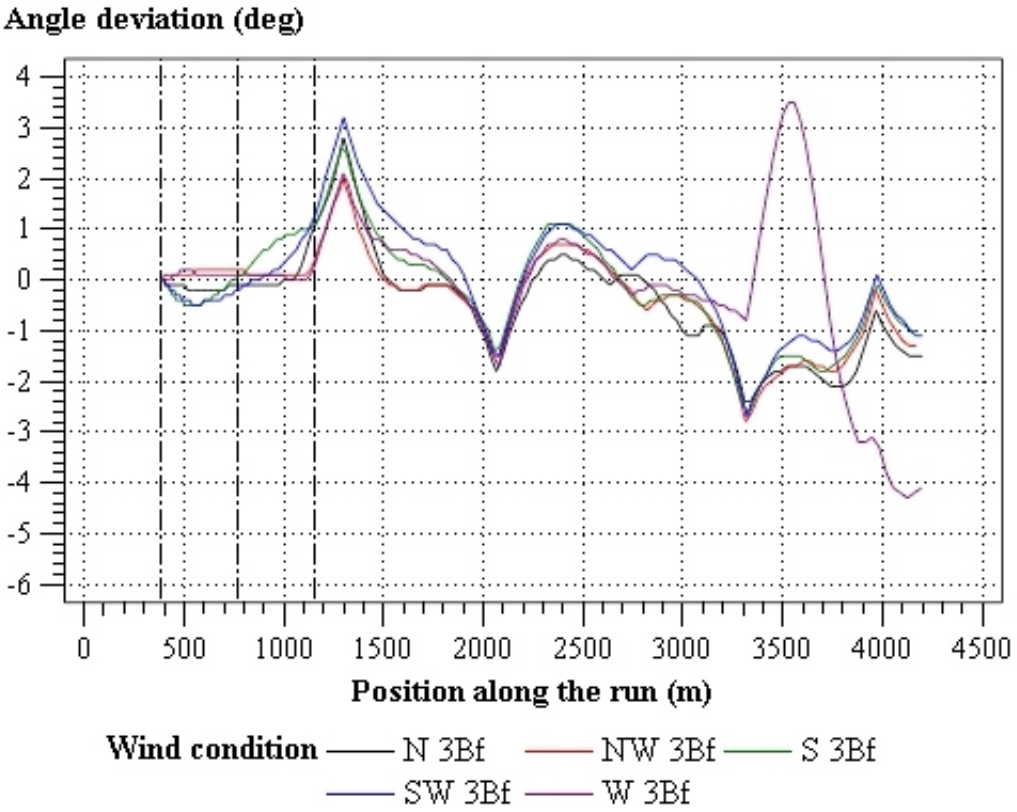
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 26
Scale: 1/25000	SWEPT PATH FOR CONDITION 5: RISING TIDE, 5 BEAUFORT WIND	



W.L. 02.0210

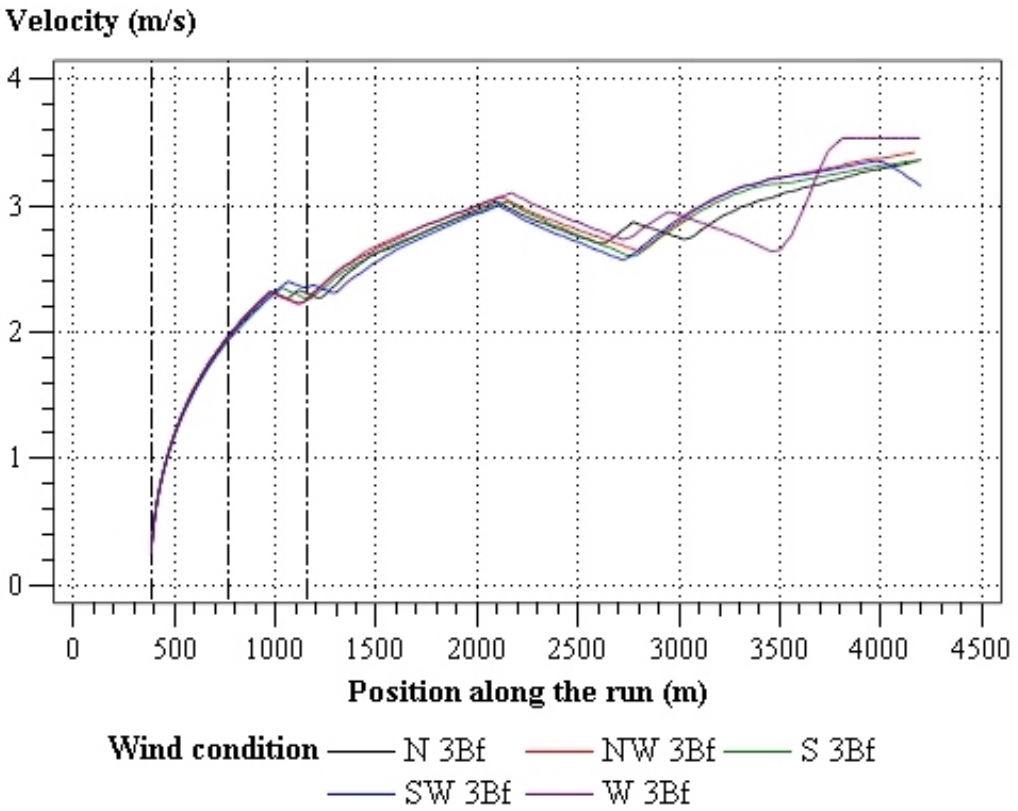
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 27a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 6: OUTGOING TIDE, 3 BEAUFORT WIND	





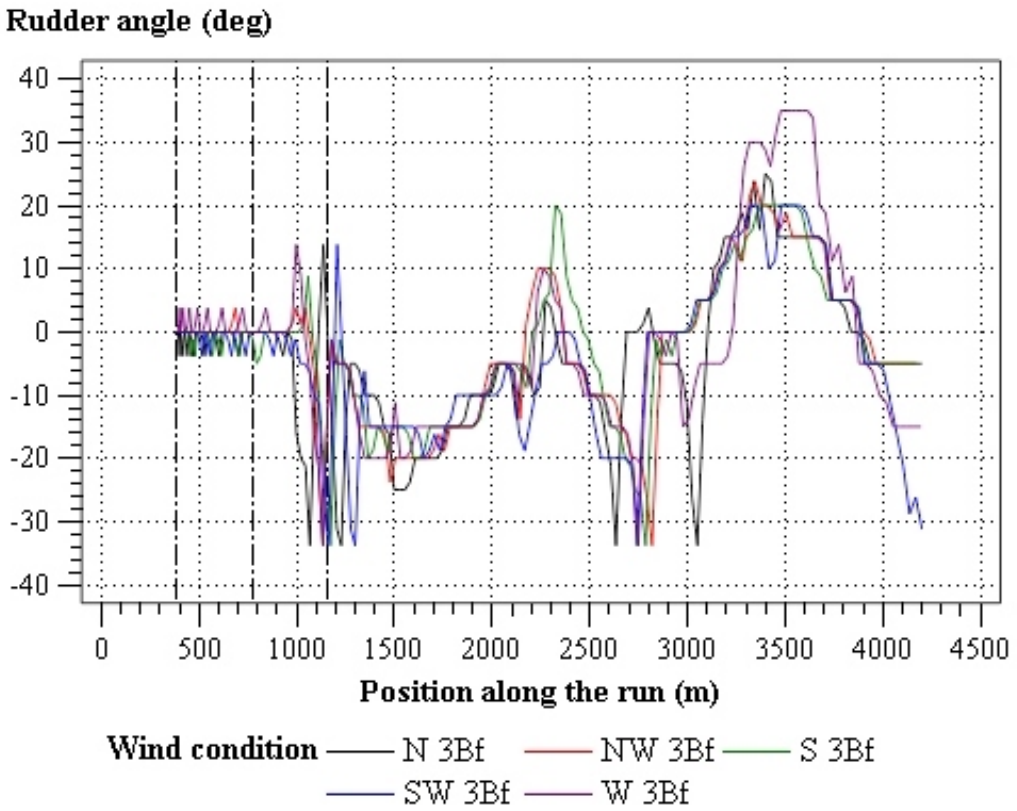
W.L. 02.0211

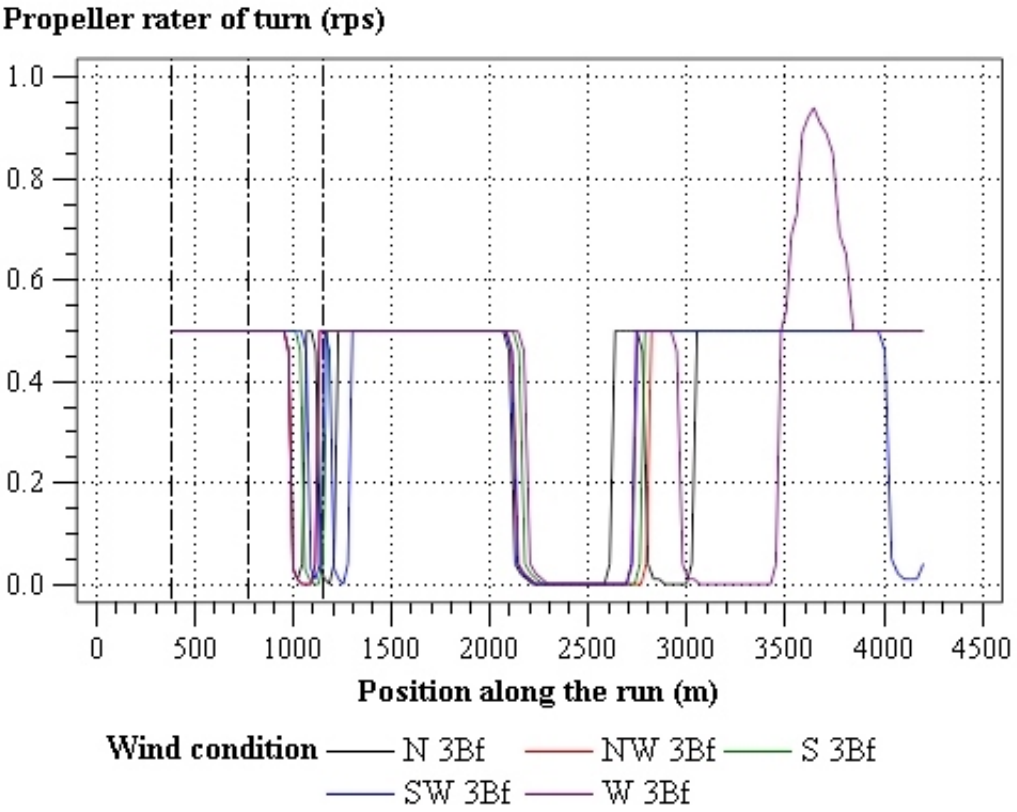
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 27b
	SHIP'S VELOCITY FOR CONDITION 6: OUTGOING TIDE, 3 BEAUFORT WIND	



W.L. 02.0212

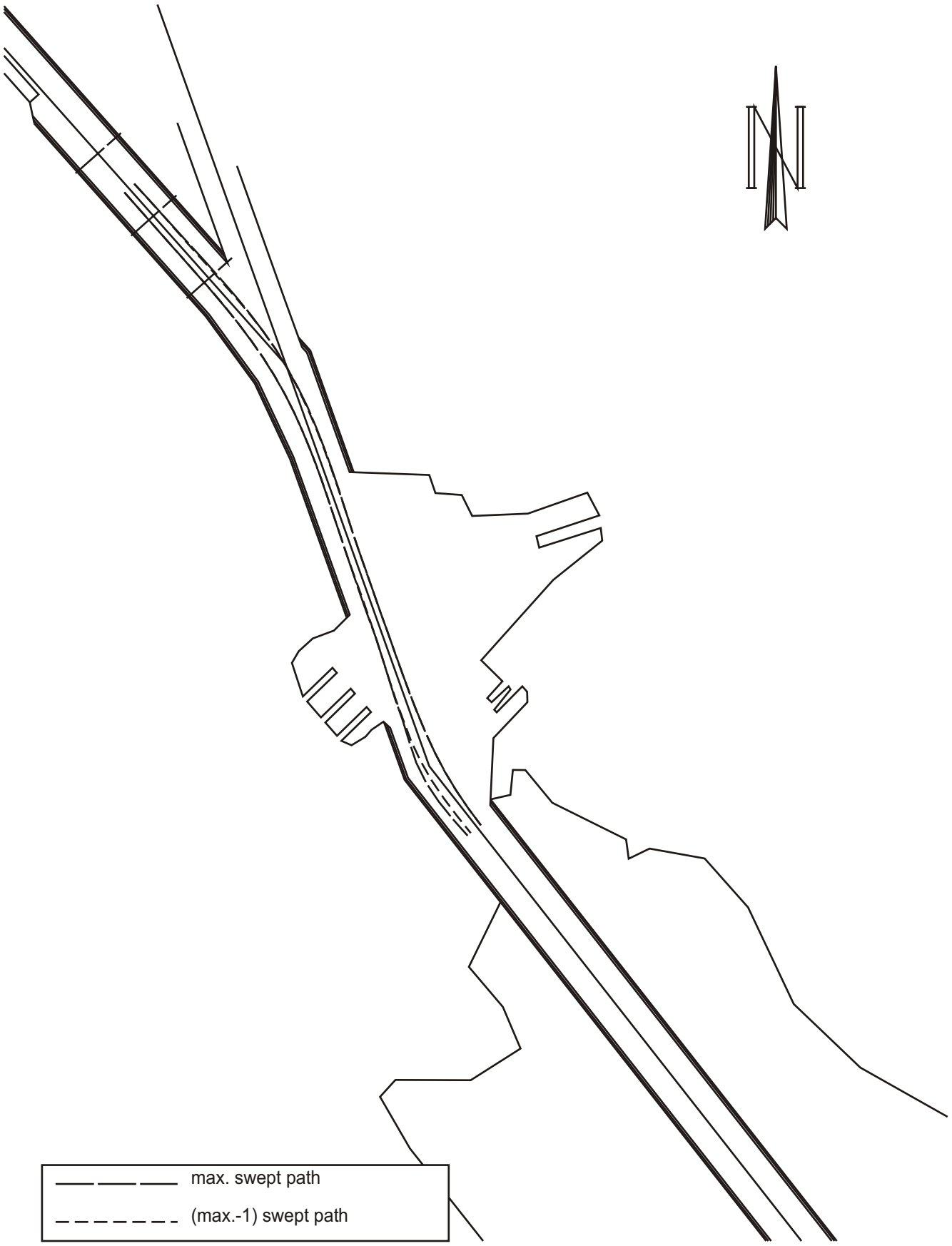
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 27c
	RUDDER AND ENGINE COMMAND FOR CONDITION 6: OUTGOING TIDE, 3 BEAUFORT WIND	





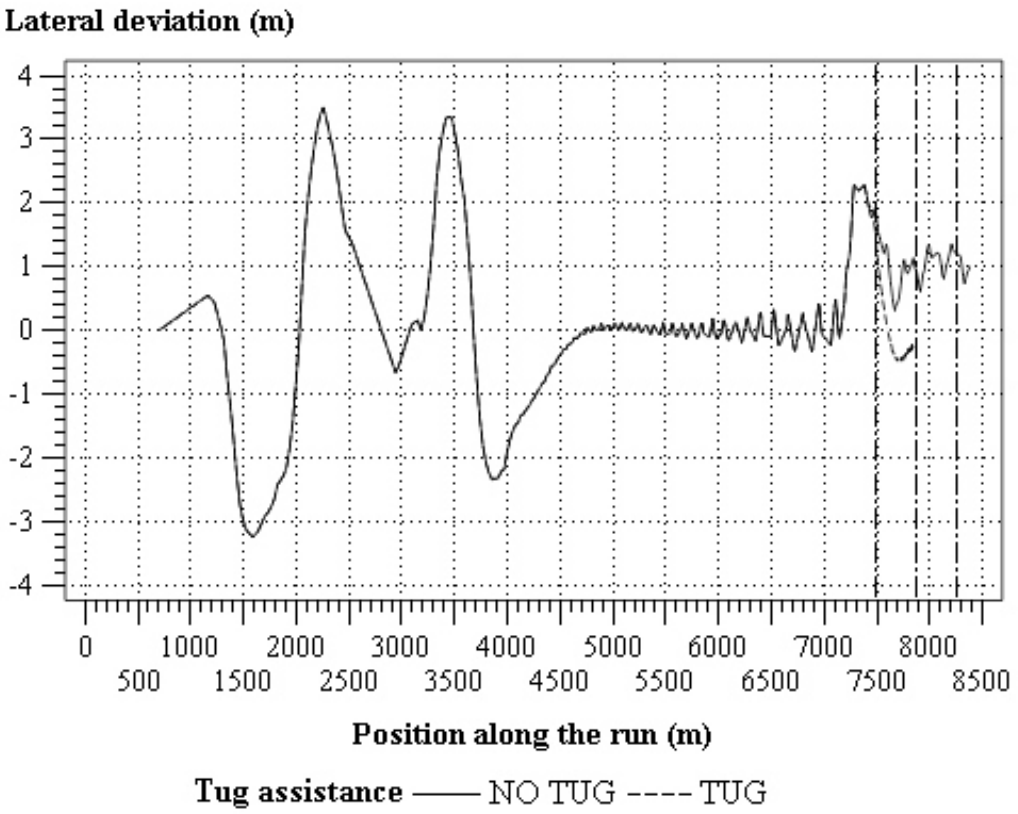
W.L. 02.0213

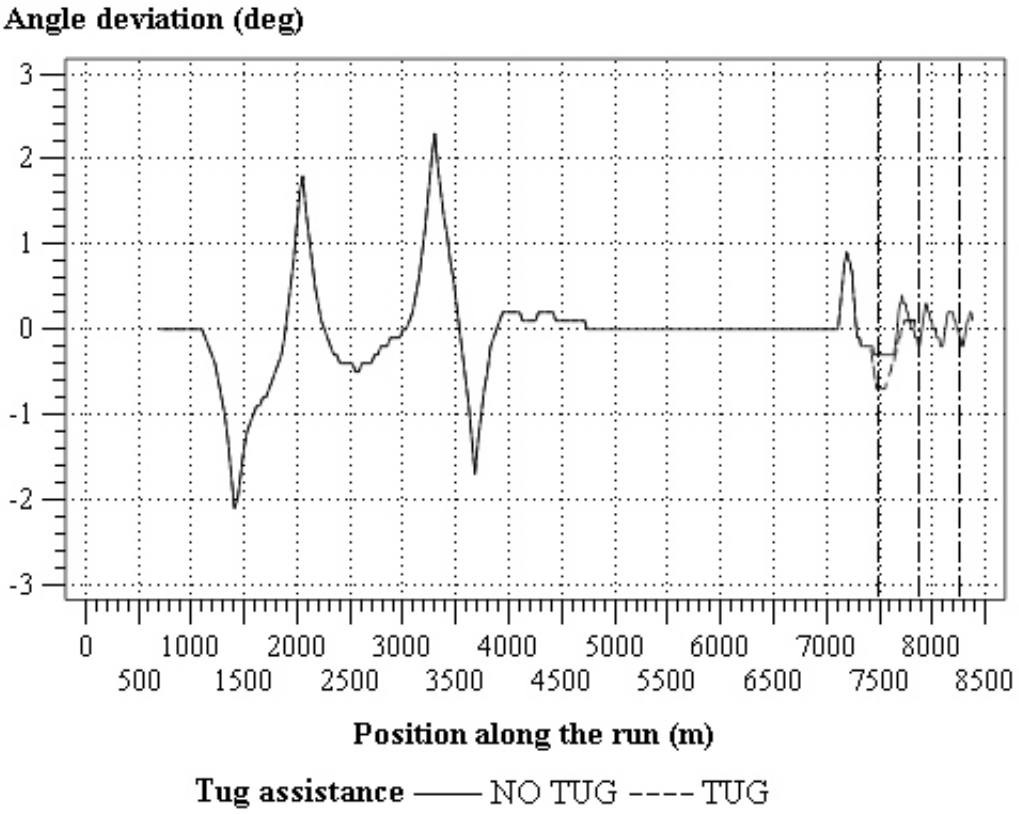
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 28
Scale: 1/25000	SWEPT PATH FOR CONDITION 6: OUTGOING TIDE, 3 BEAUFORT WIND	



W.L. 02.0214

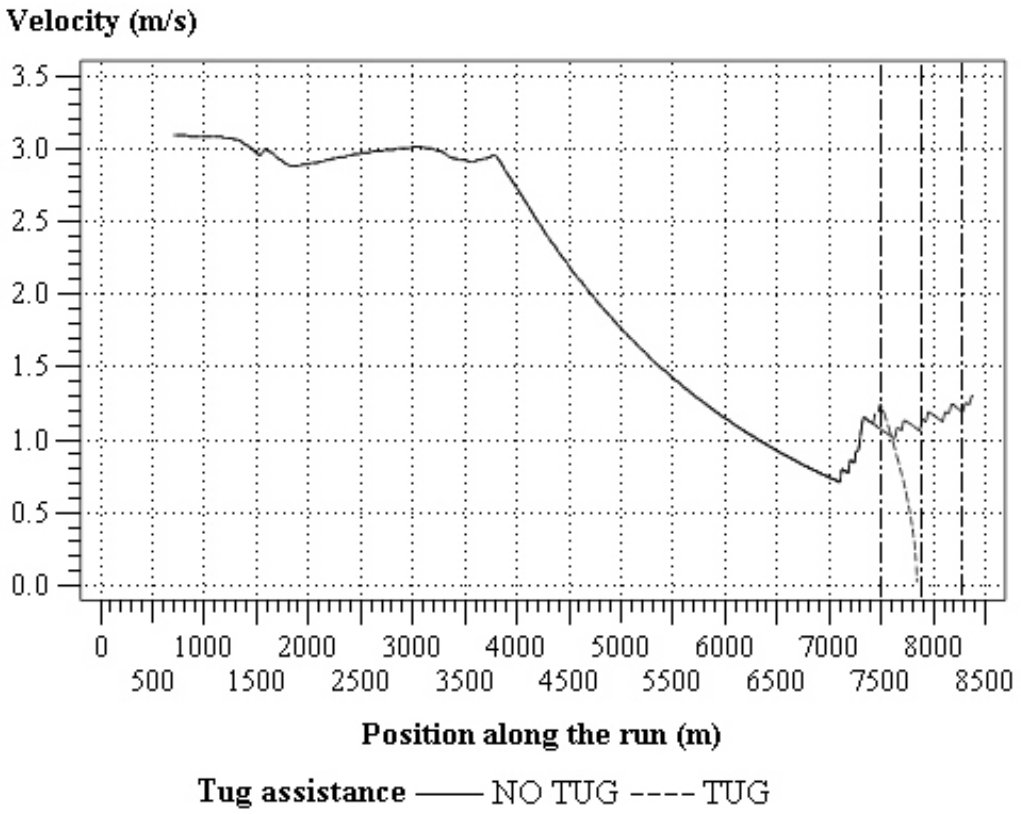
FALNDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 29a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 7: NO WIND	



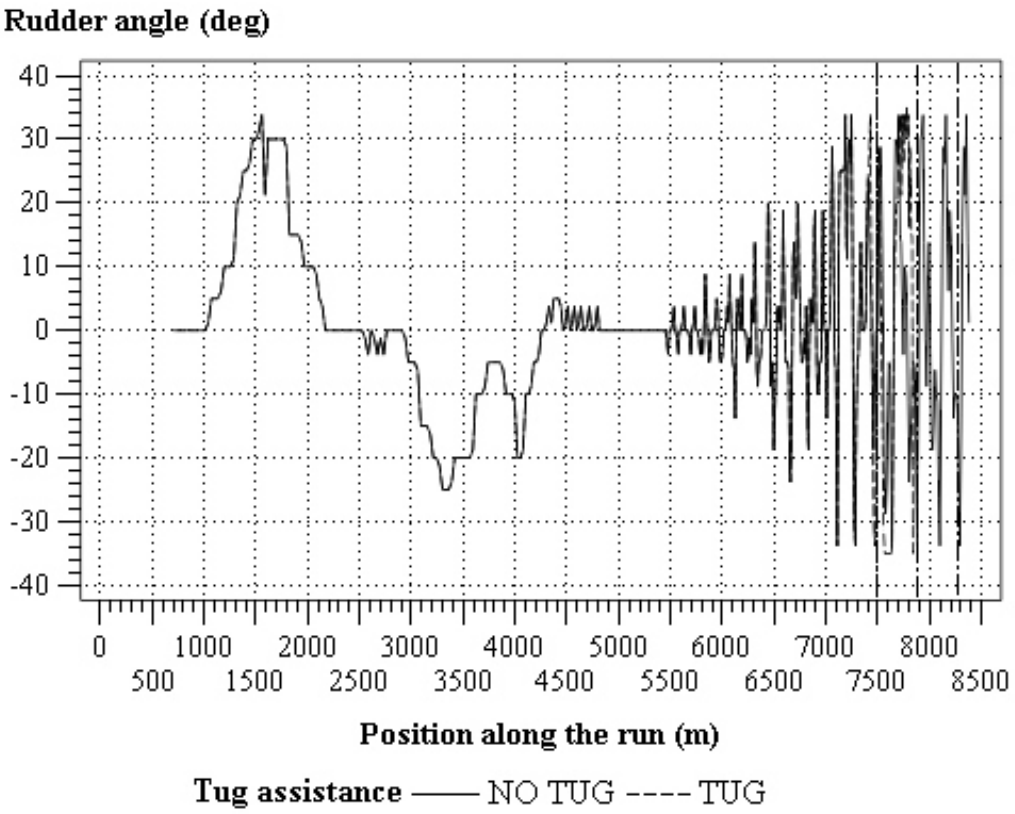


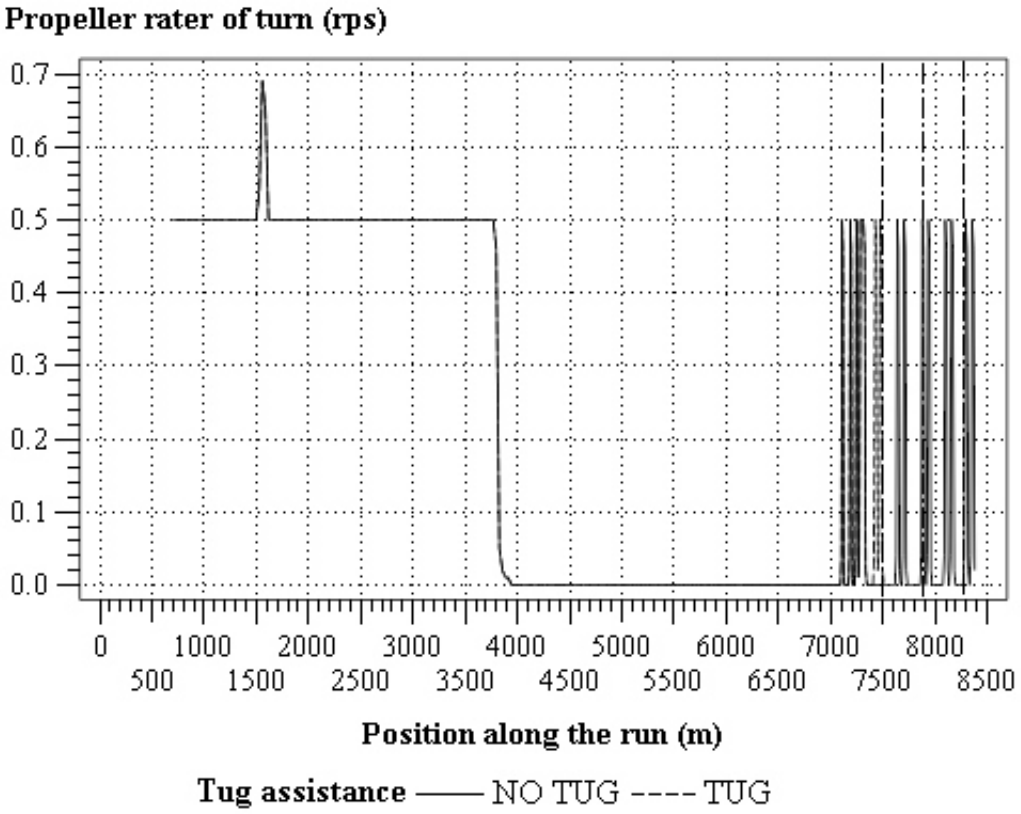
W.L. 02.0215

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 29b
	SHIP'S VELOCITY FOR CONDITION 7: NO WIND	

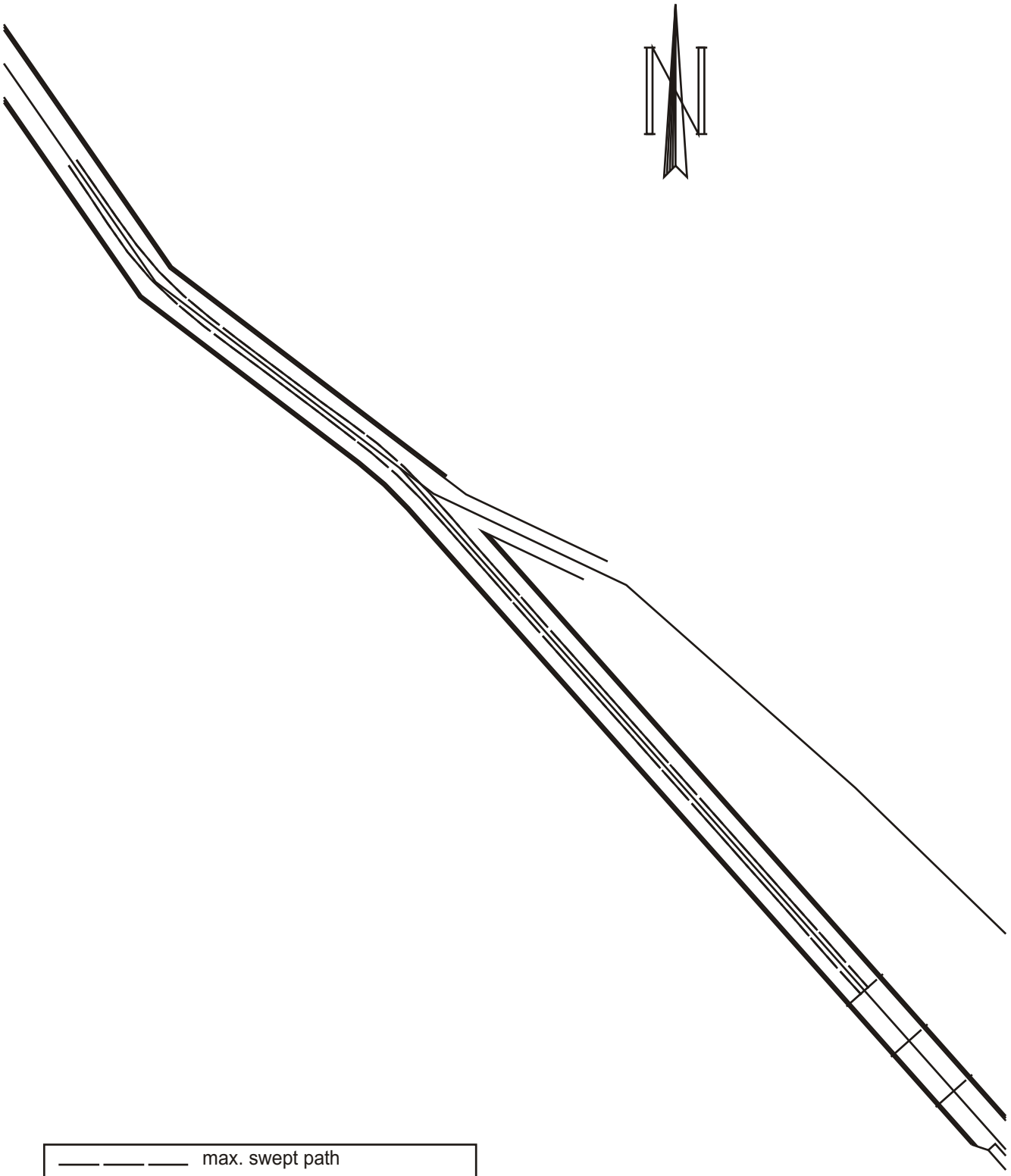


FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 29c
	RUDDER AND ENGINE COMMAND FOR CONDITION 7: NO WIND	



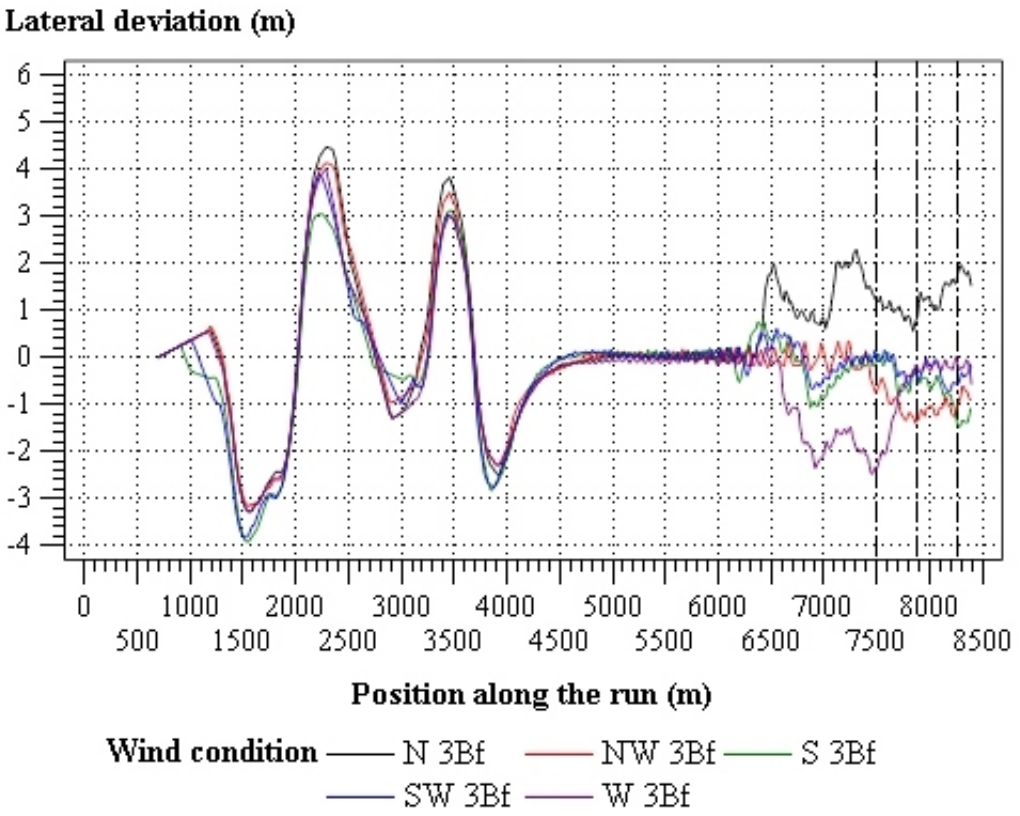


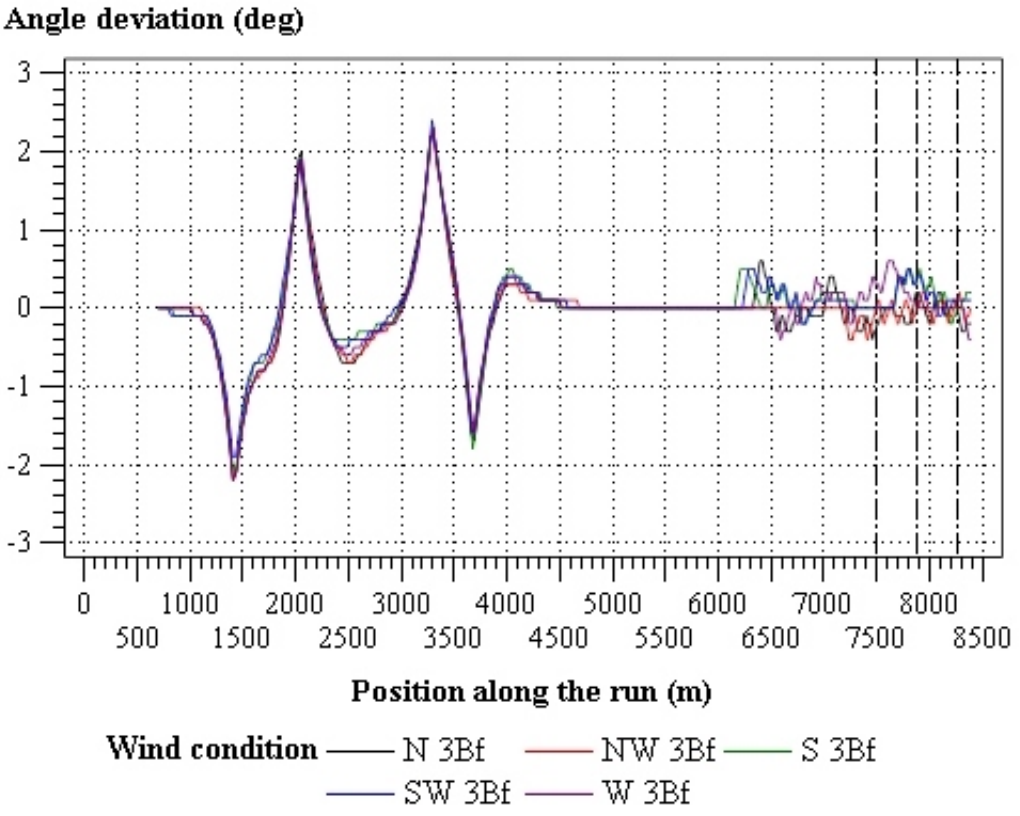
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 30
Scale: 1/33000	SWEPT PATH FOR CONDITION 7: NO WIND	



W.L. 02.0218

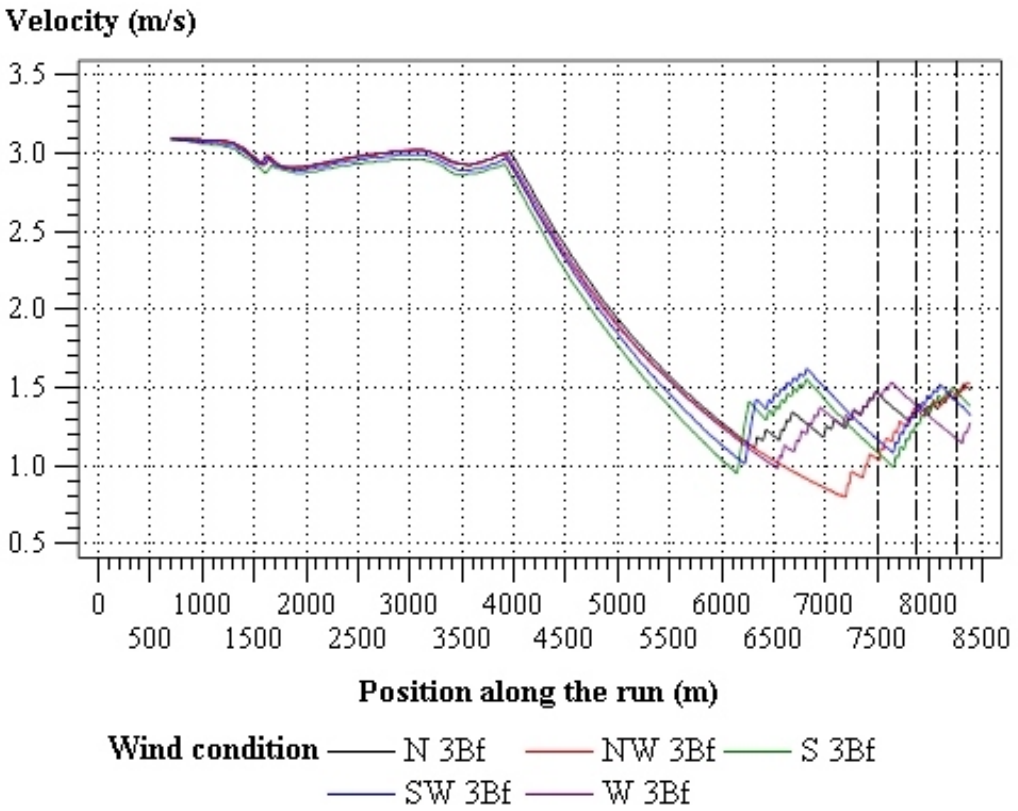
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 31a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 8: 3 BEAUFORT WIND	





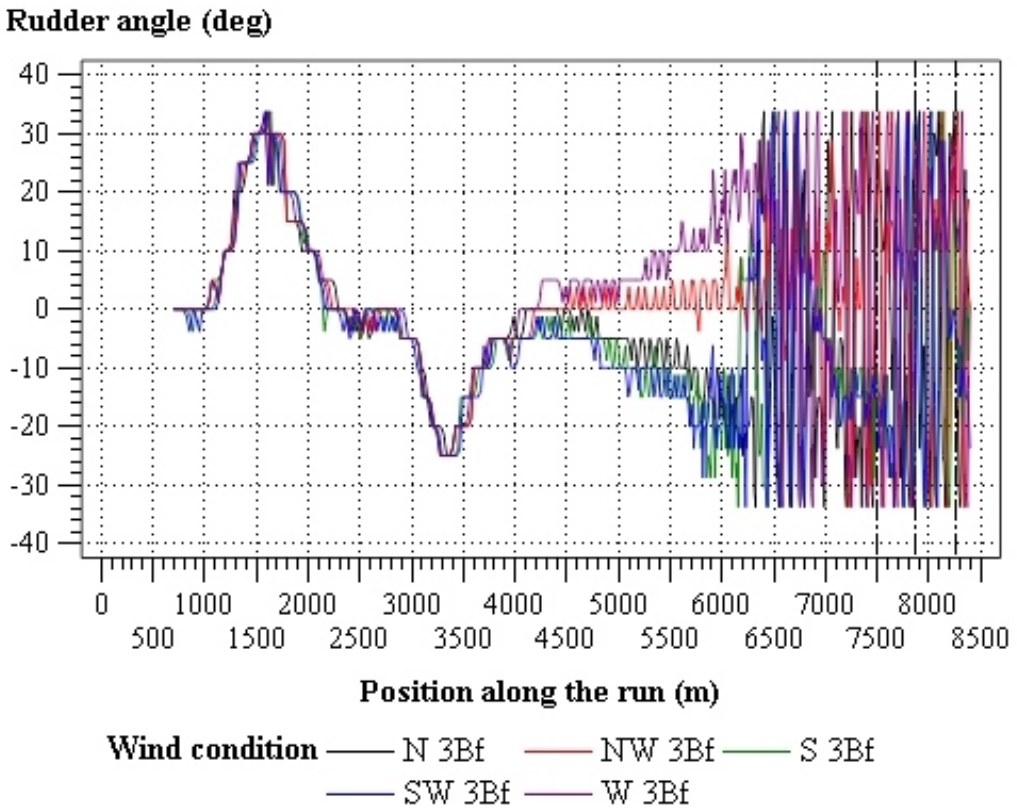
W.L. 02.0219

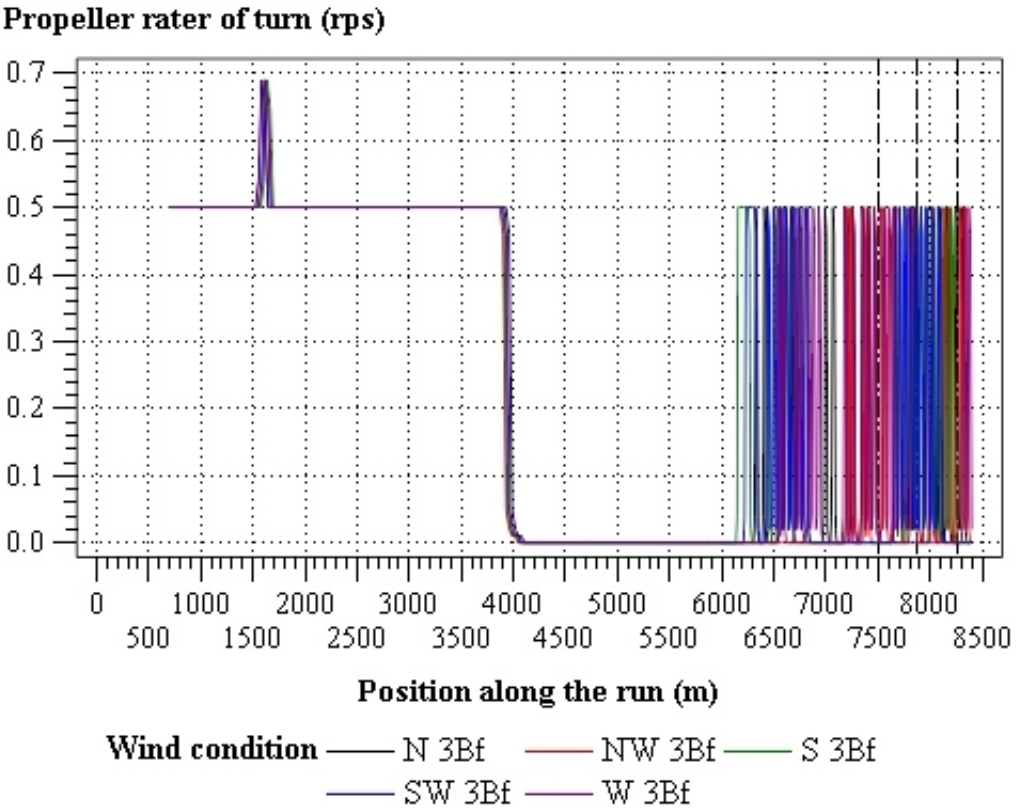
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 31b
	SHIP'S VELOCITY FOR CONDITION 8: 3 BEAUFORT WIND	



W.L. 02.0220

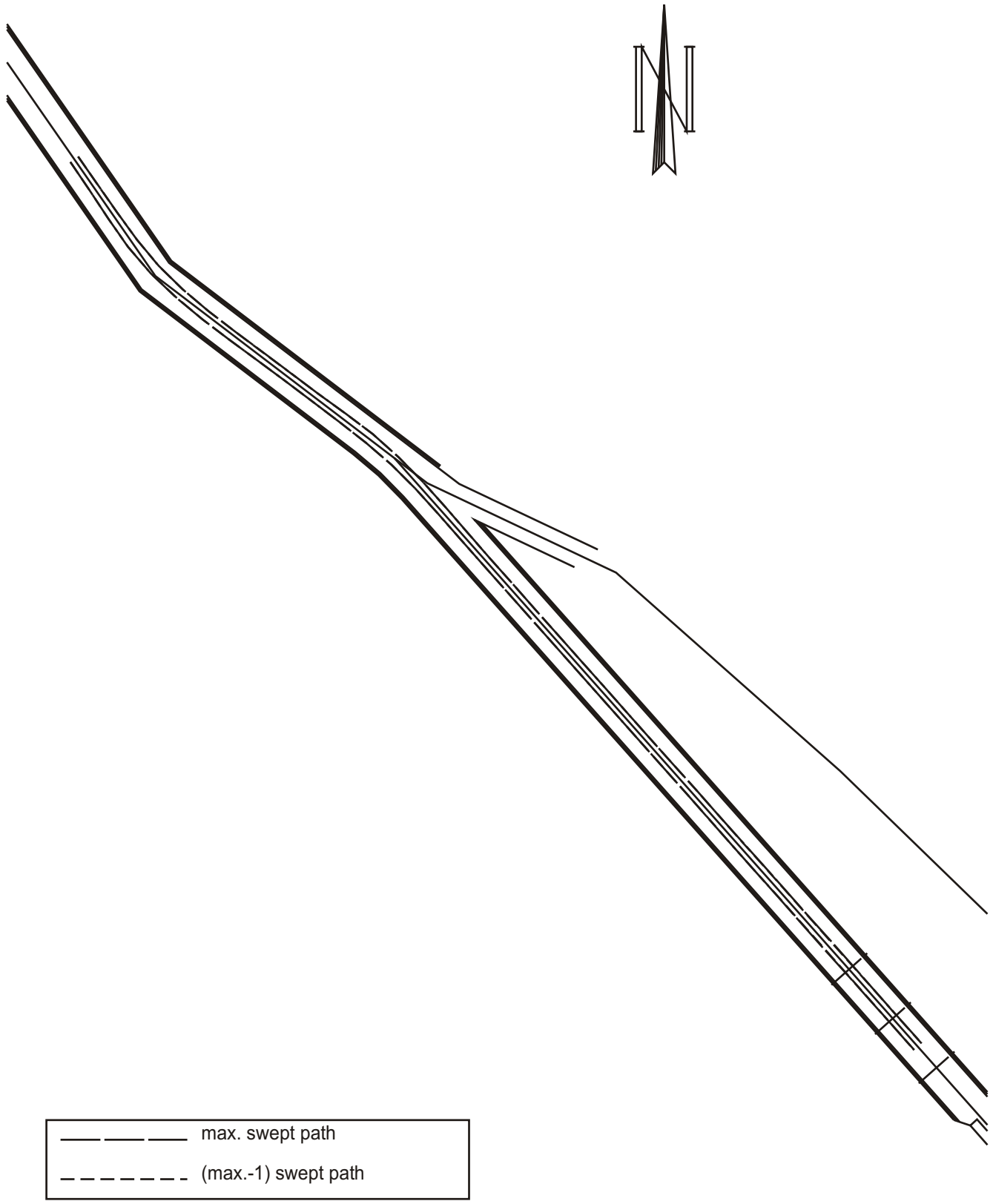
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 31c
	RUDDER AND ENGINE COMMAND FOR CONDITION 8: 3 BEAUFORT WIND	





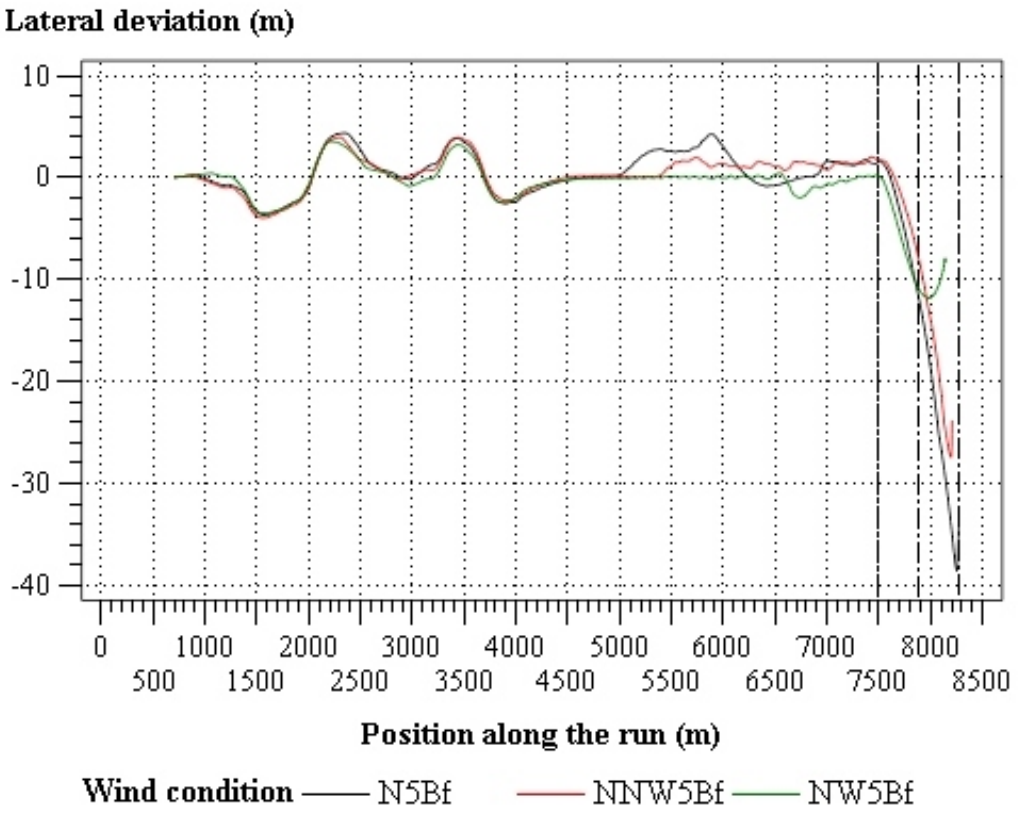
W.L. 02.0221

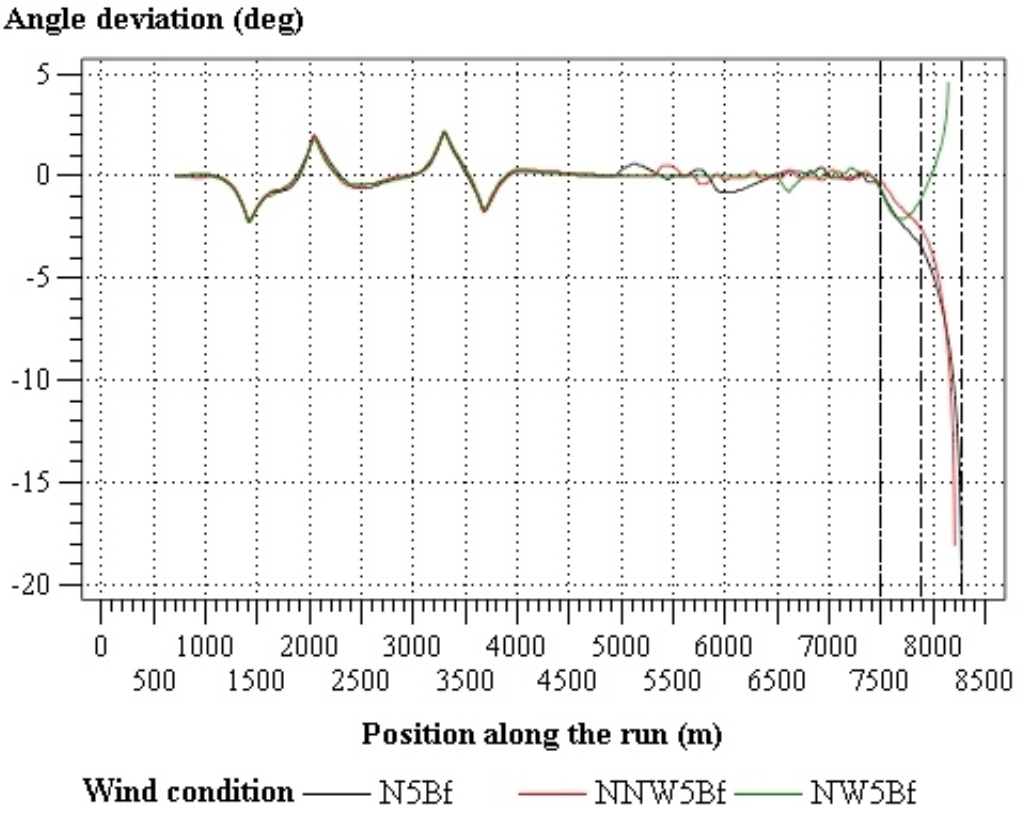
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 32
Scale: 1/33000	SWEPT PATH FOR CONDITION 8: 3 BEAUFORT WIND	



W.L. 02.0222

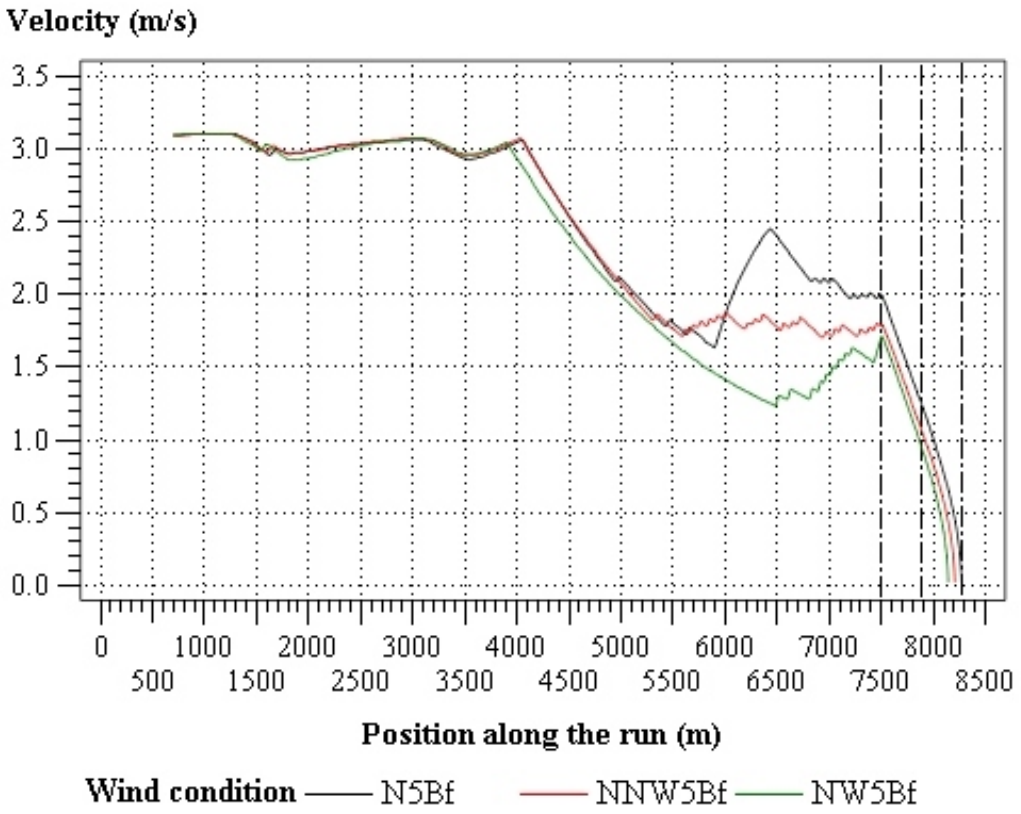
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 33a
	LATERAL AND ANGULAR DEVIATION FOR CONDITION 9: 3 BEAUFORT WIND	





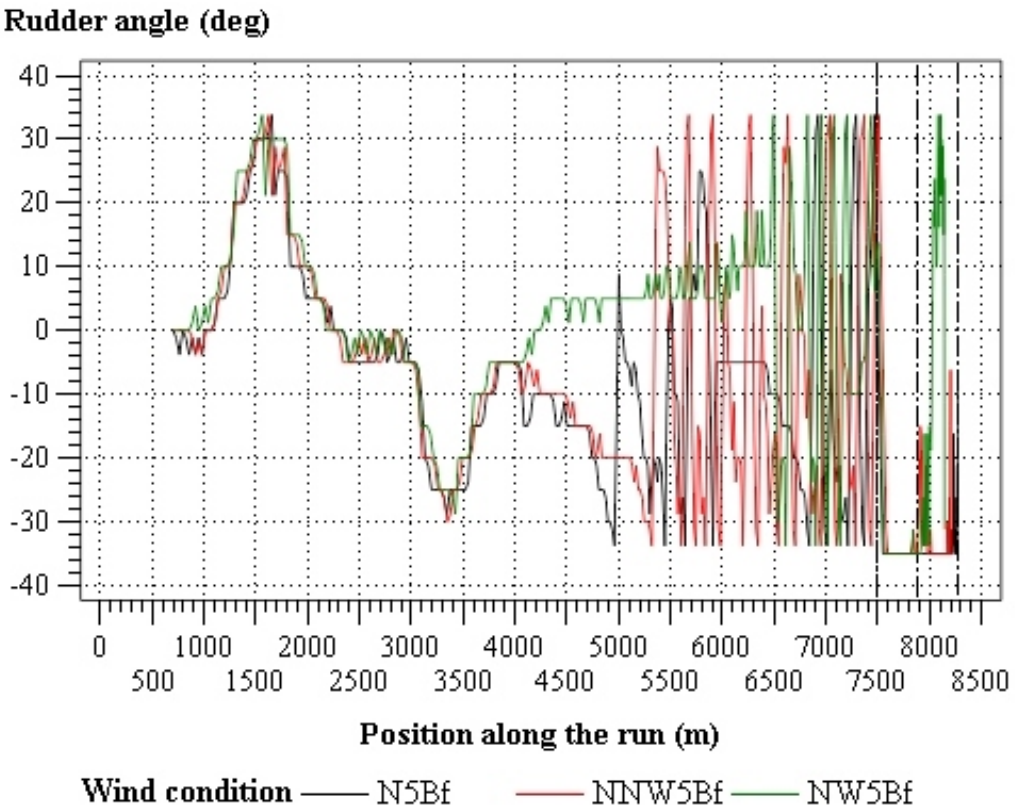
W.L. 02.0223

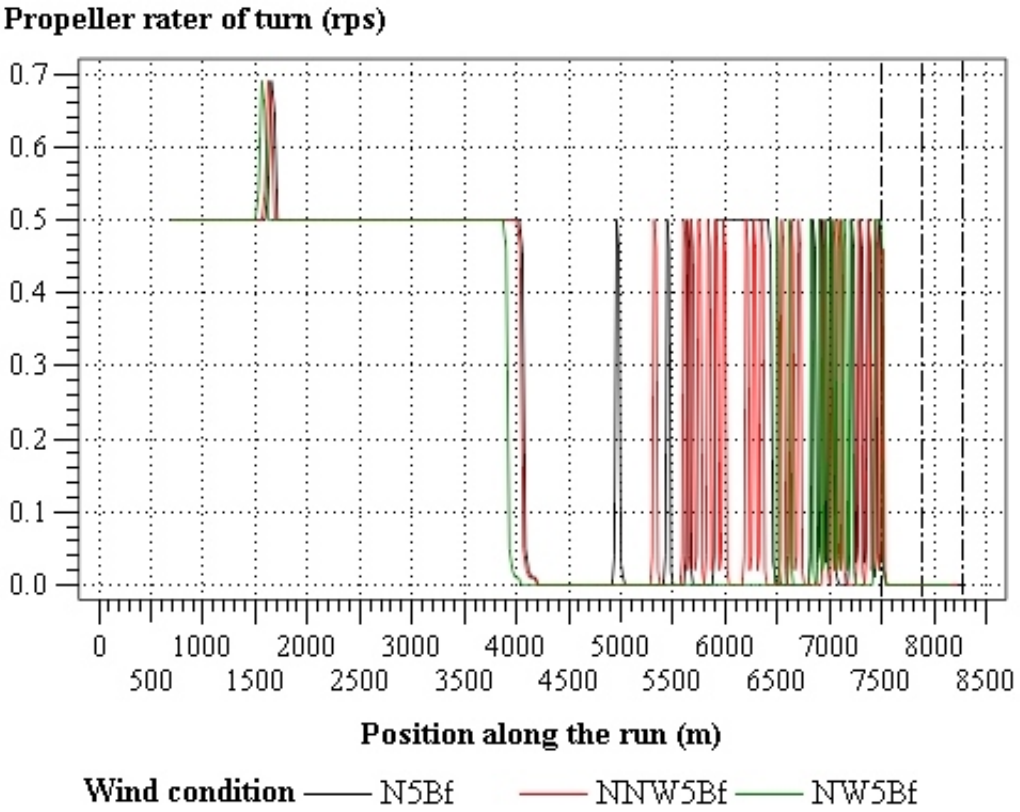
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 33b
	SHIP'S VELOCITY FOR CONDITION 9: 3 BEAUFORT WIND	



W.L. 02.0224

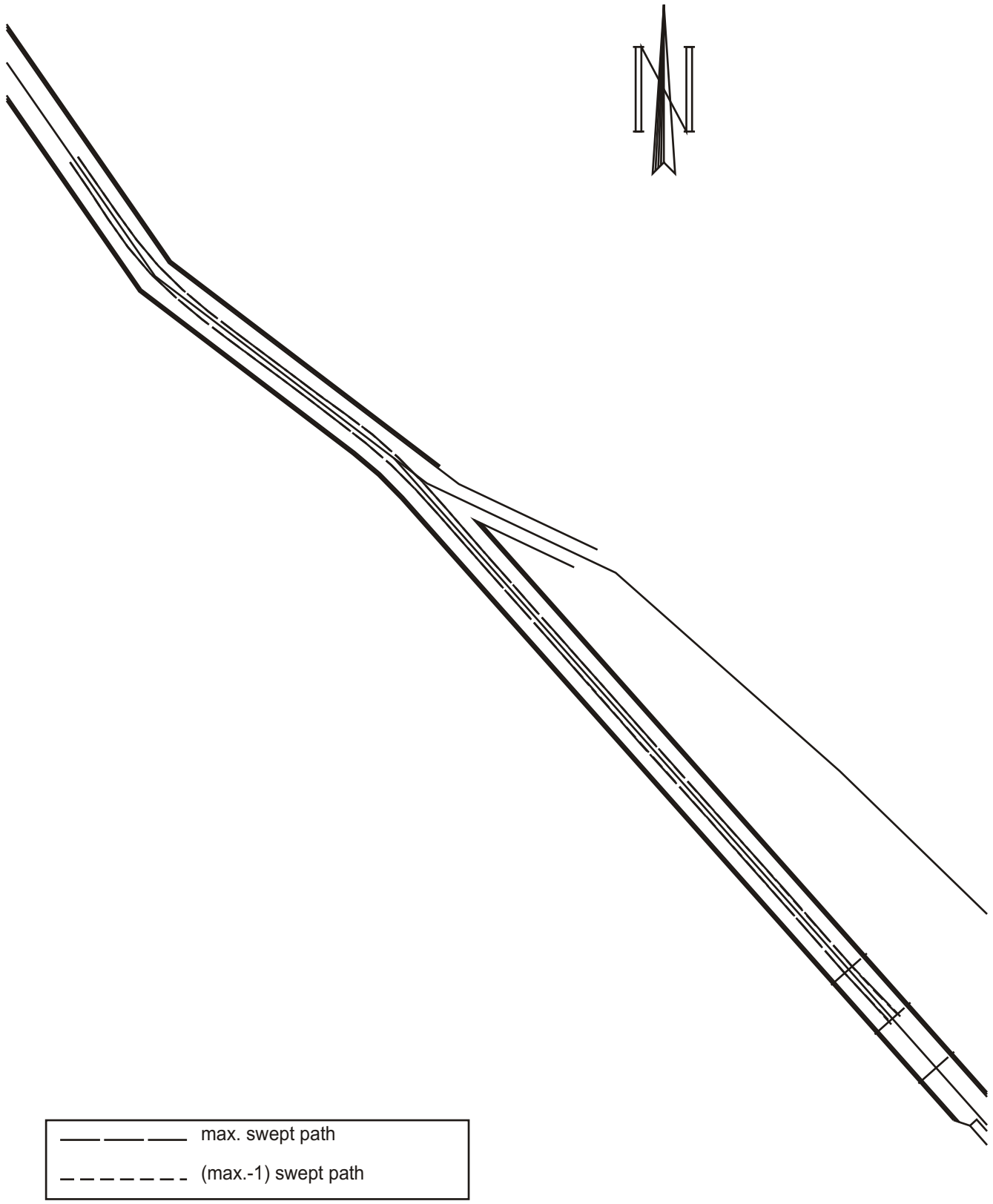
FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 33c
	RUDDER AND ENGINE COMMAND FOR CONDITION 9: 3 BEAUFORT WIND	





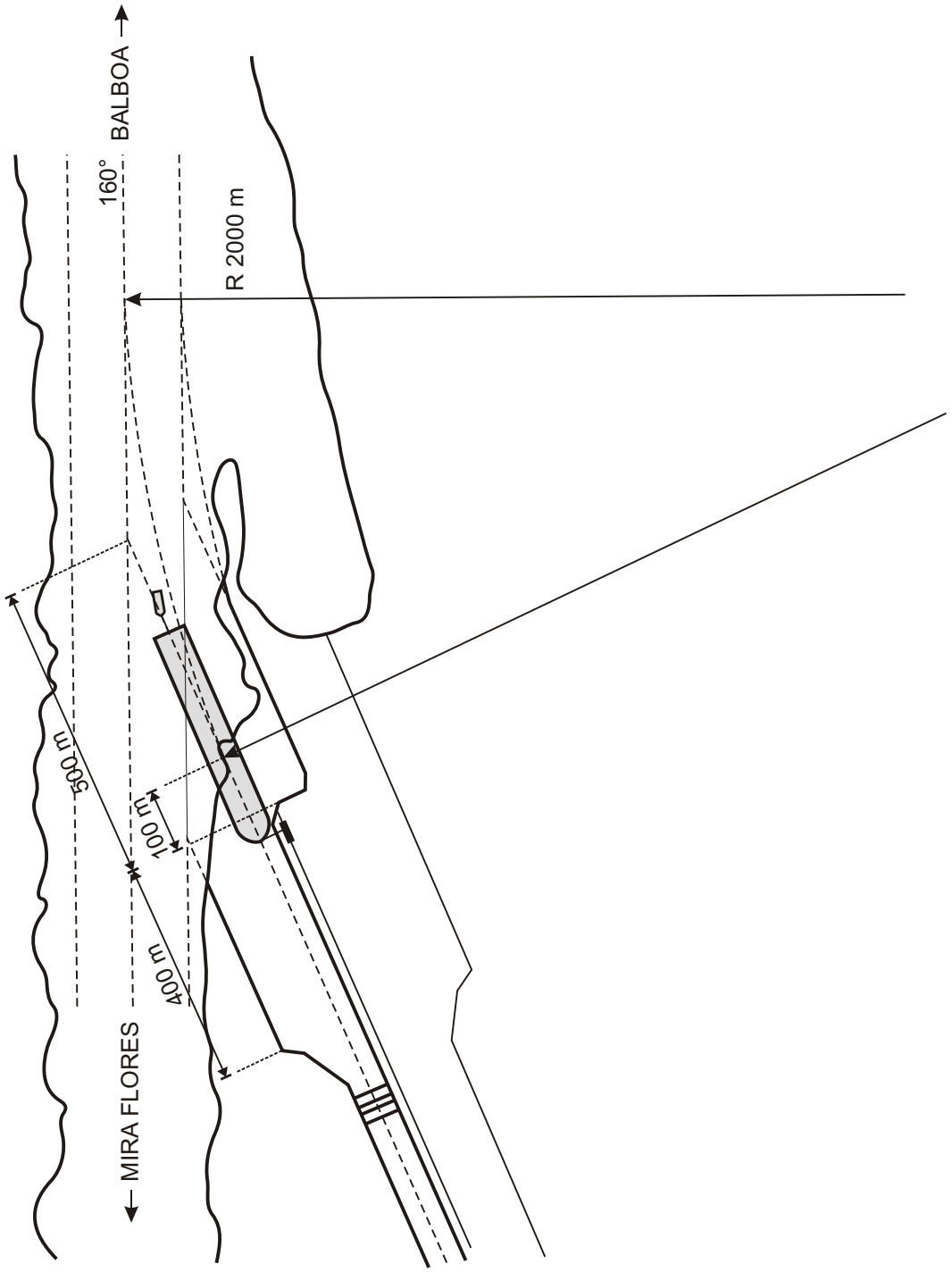
W.L. 02.0225

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 34
Scale: 1/33000	SWEPT PATH FOR CONDITION 9: 3 BEAUFORT WIND	



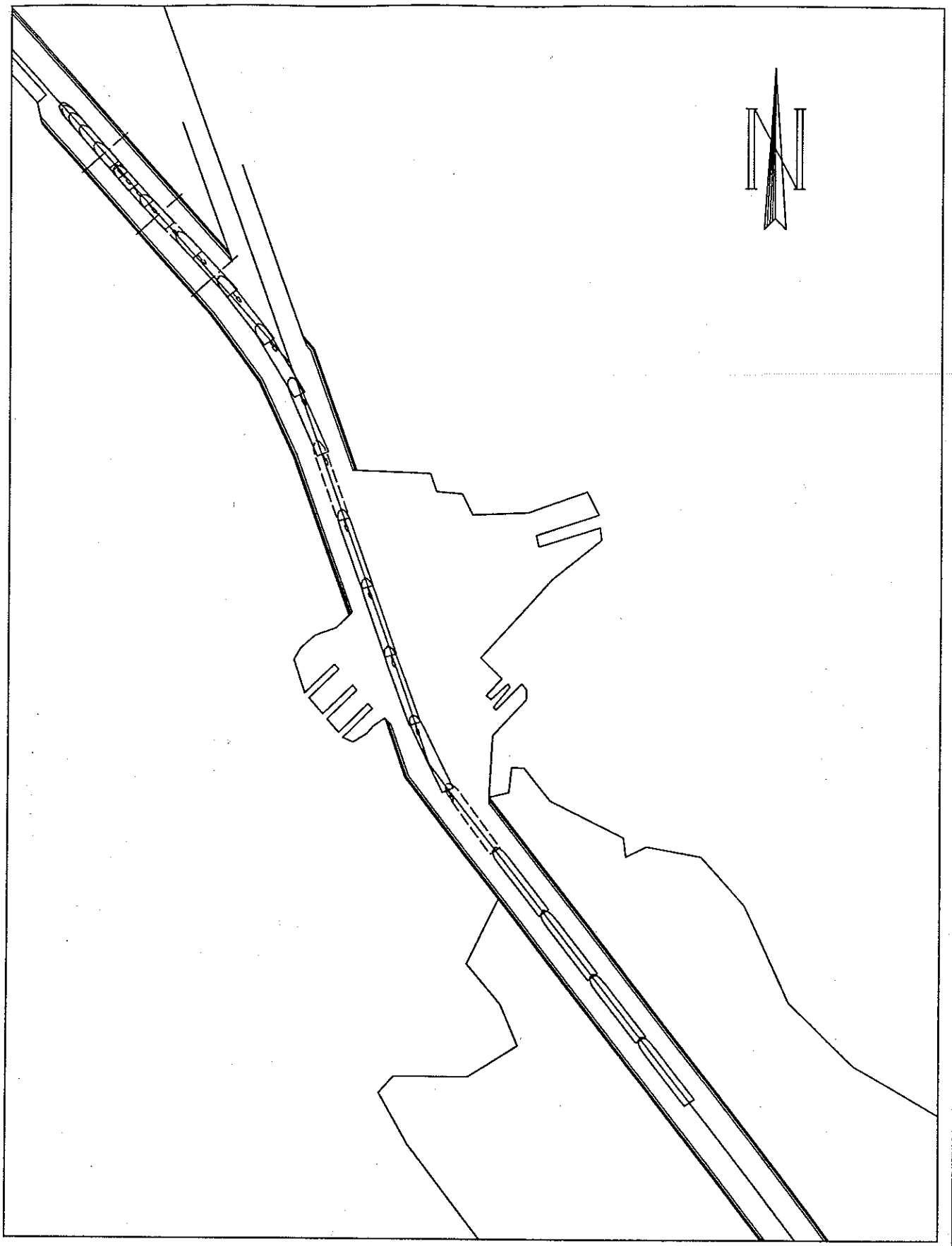
W.L. 02.0226

FLANDERS HYDRAULICS	MOD. 673 SIMULATION STUDY PANAMA CANAL	Figure 35
	ALIGNMENT P2	



W.L. 02.0227

Track Plots



Plotinterval 120. s

Scale 1:/25000.

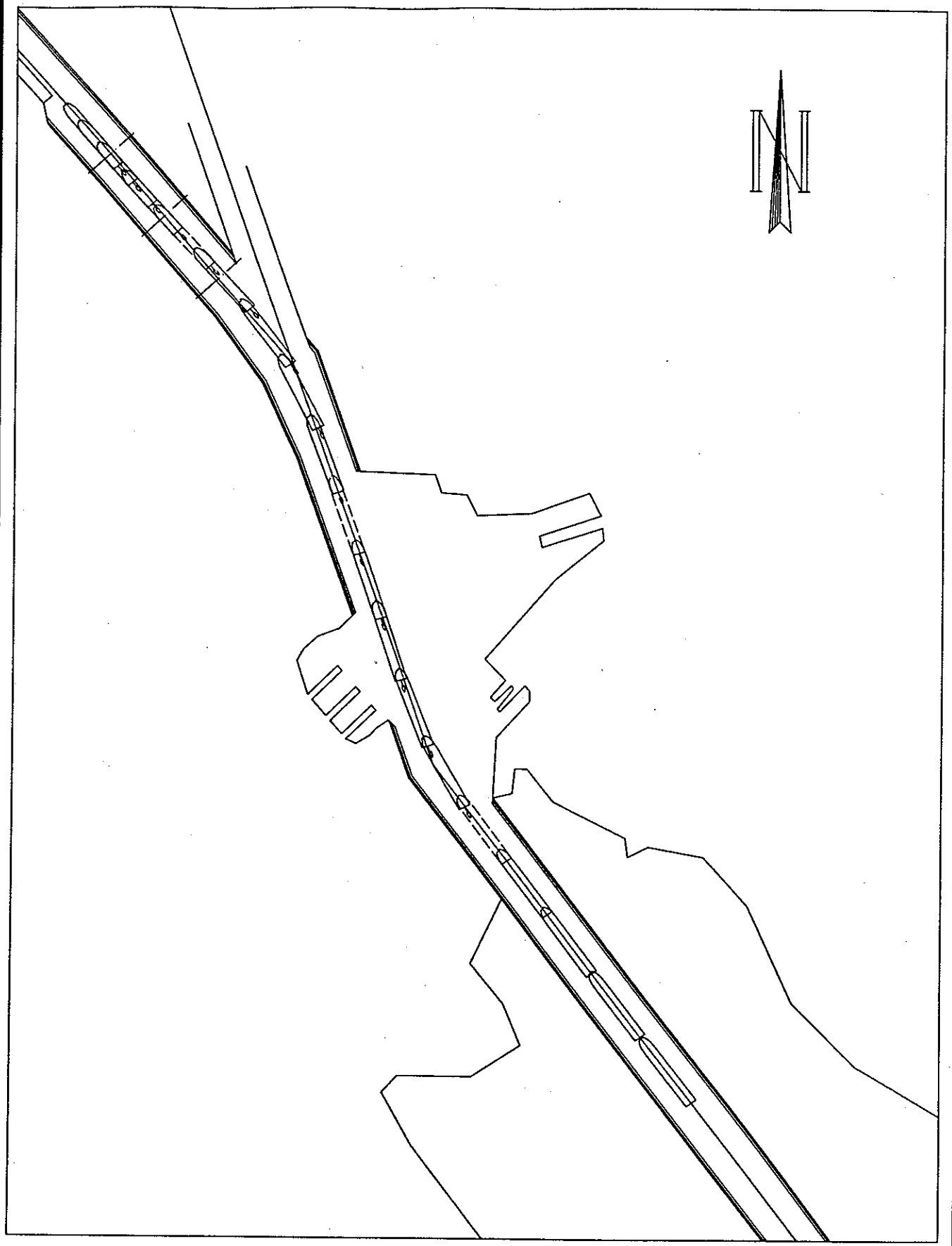
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition: 1
Run : 000
Wind : no wind
Tide : Low Water

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-1



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal Sailing to lock from Pacific Ocean.	Condition : 1 Run : 001 Wind : no wind Tide : +3.42m, flood

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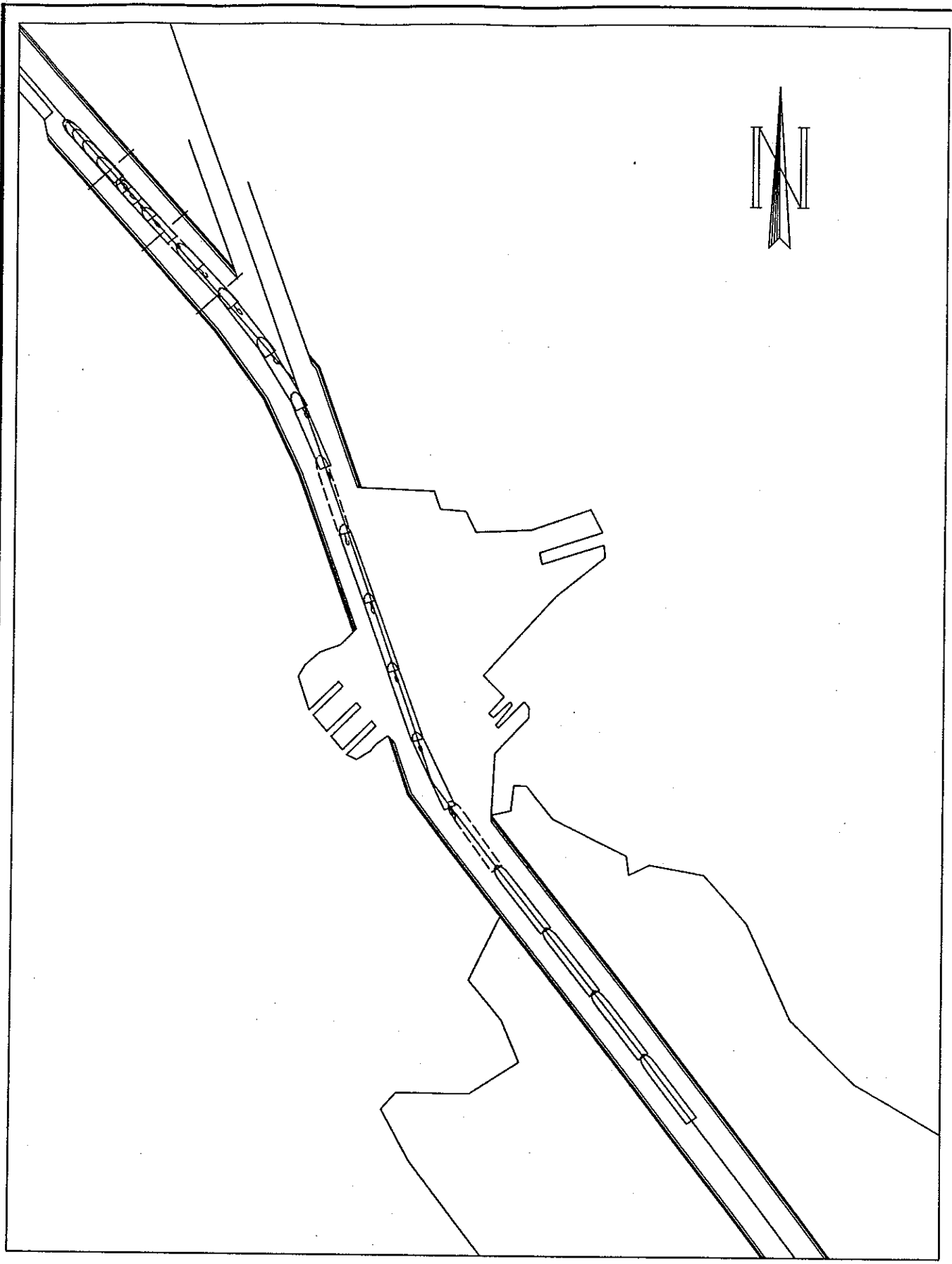

Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1APanama Canal
Sailing to lock from Pacific Ocean.

Condition : 2
Run : 002
Wind : S 3 Bf
Tide : Low Water

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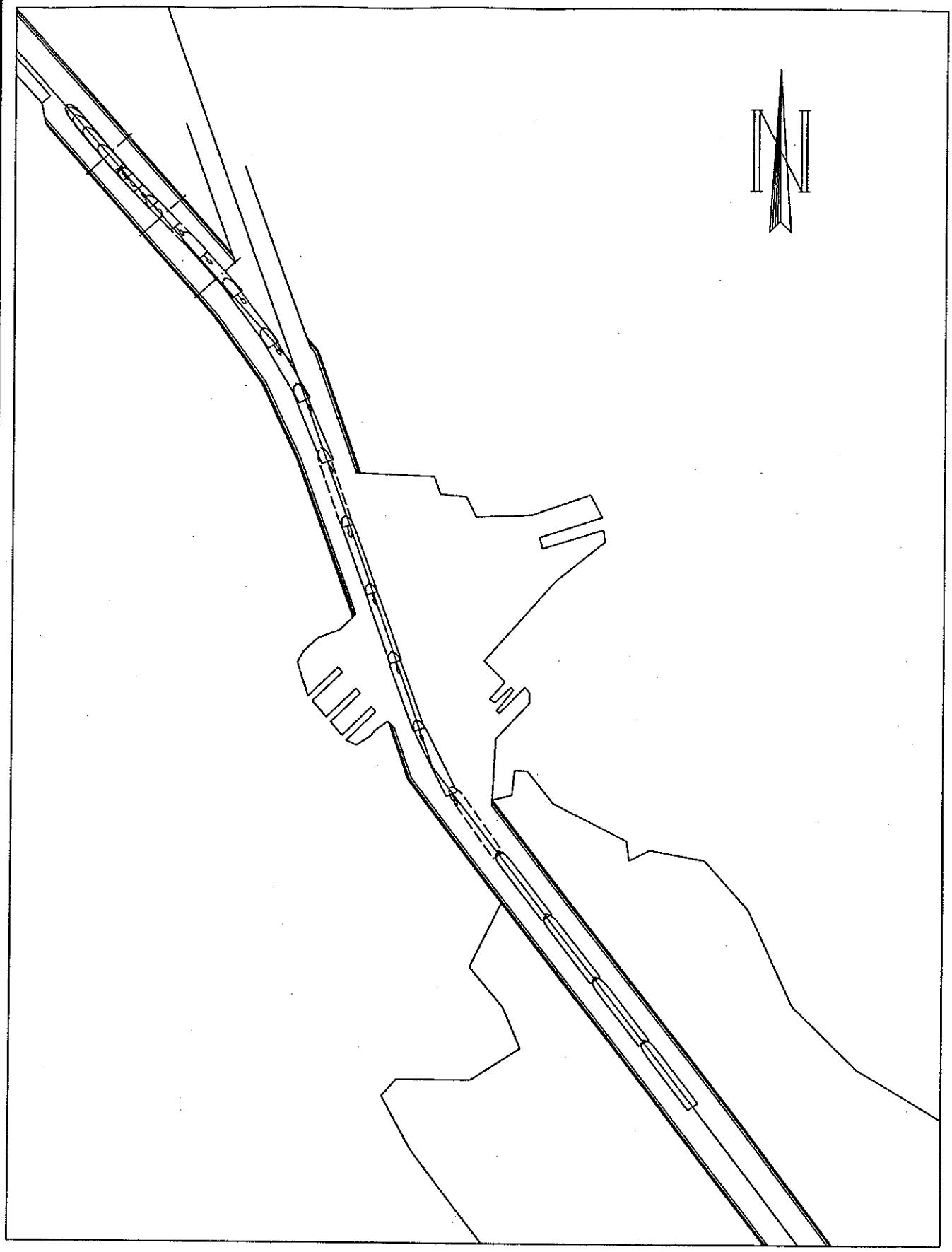
M673

A-3



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 2
Run : 004
Wind : SW 3 Bf
Tide : Low Water

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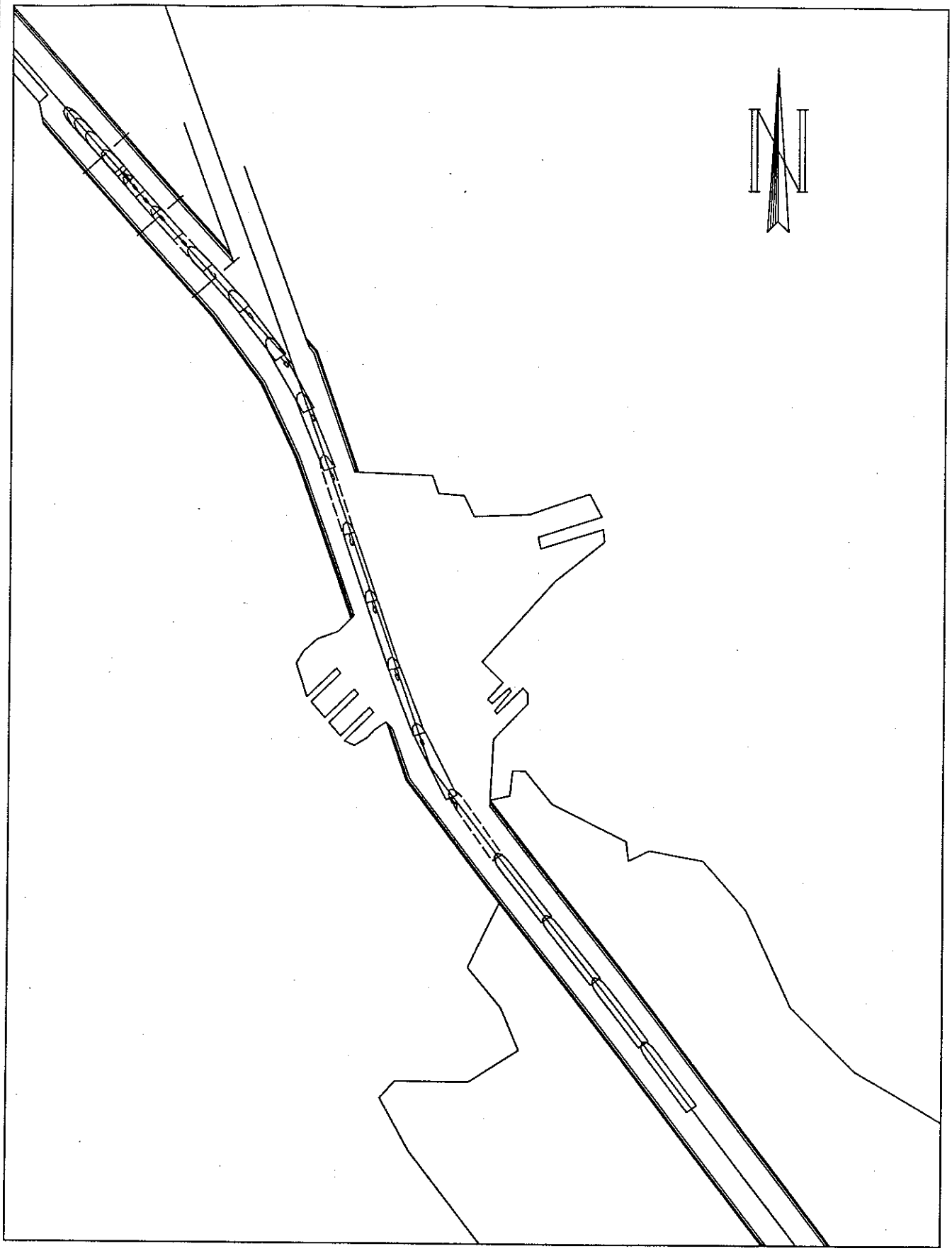
M673

A-4



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 2
Run : 027
Wind : N 3 Bf
Tide : Low Water

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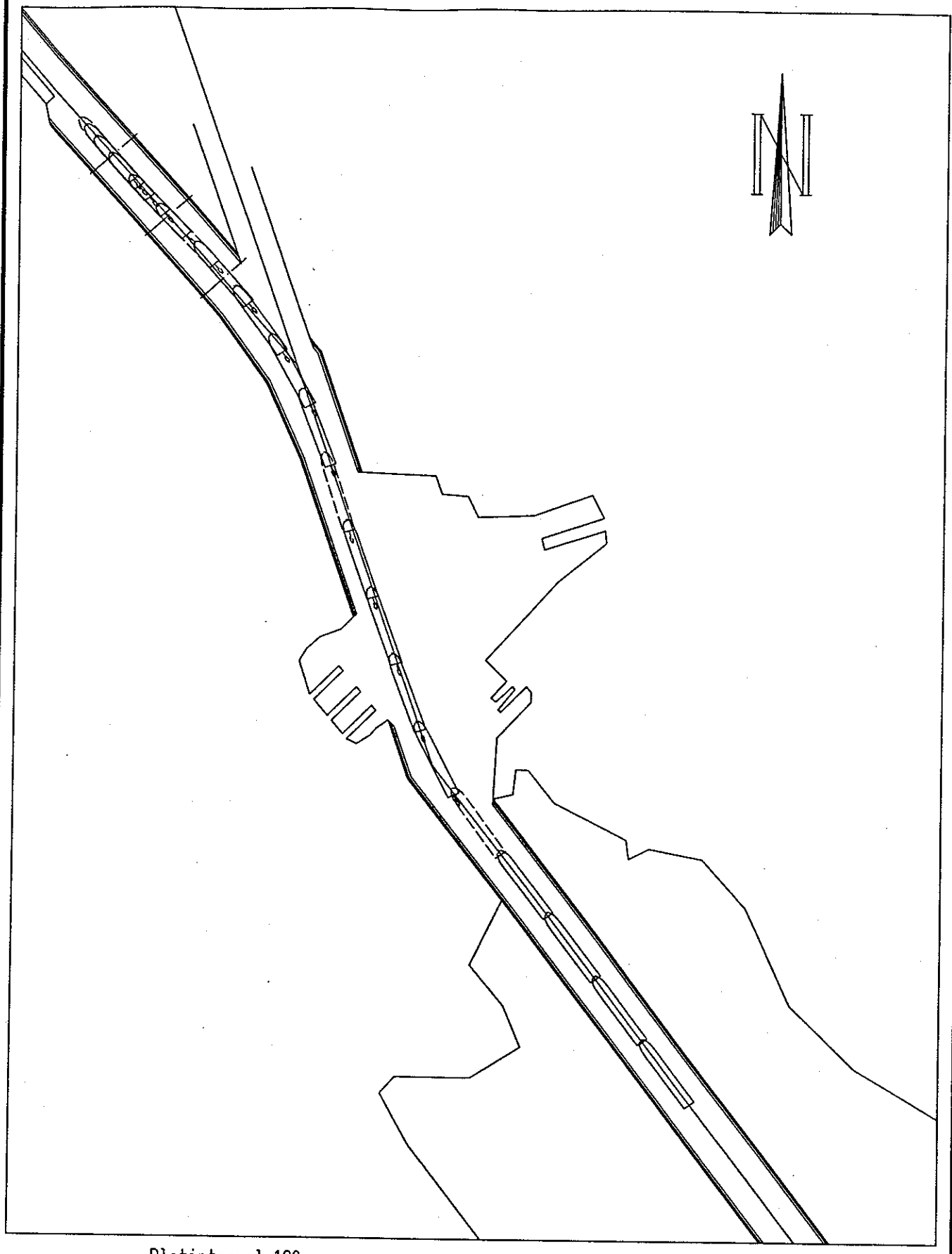
M673

A-5



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 2
Run : 028
Wind : NW 3 Bf
Tide : Low Water

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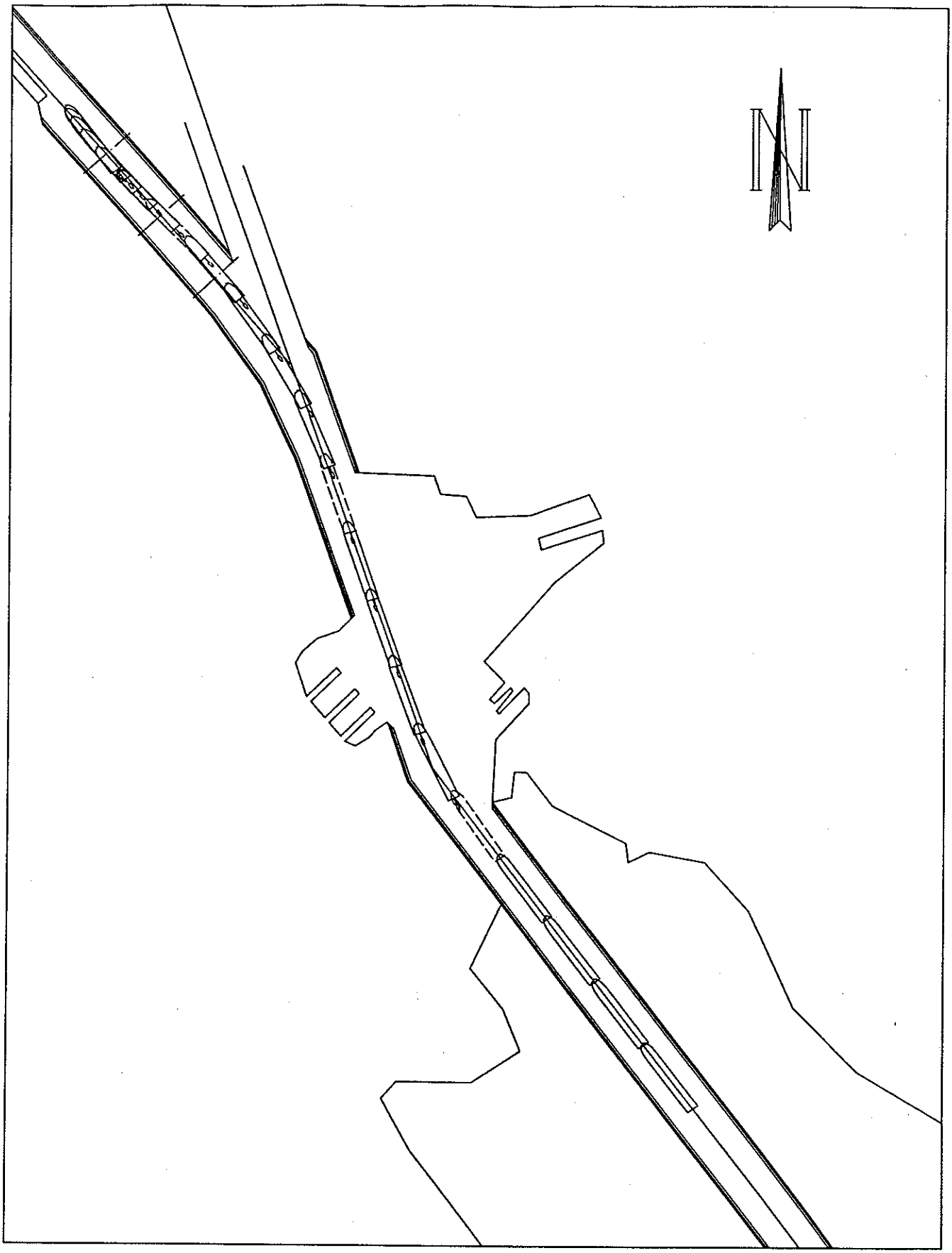
Borgerhout-Antwerpen

M673

A-6



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P14-Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 2
Run : 029
Wind : W 3 Bf
Tide : Low Water

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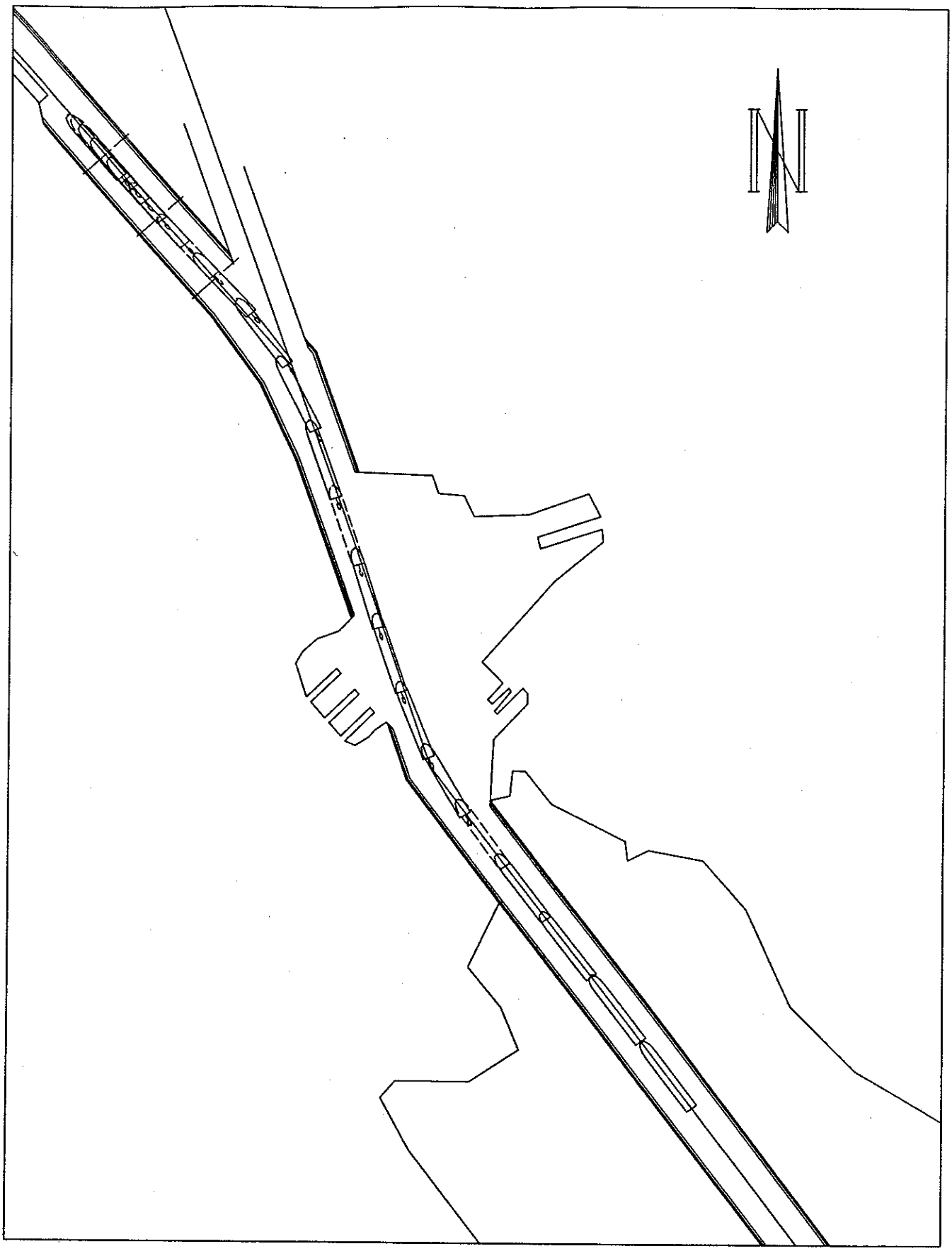
Borgerhout-Antwerpen

M673

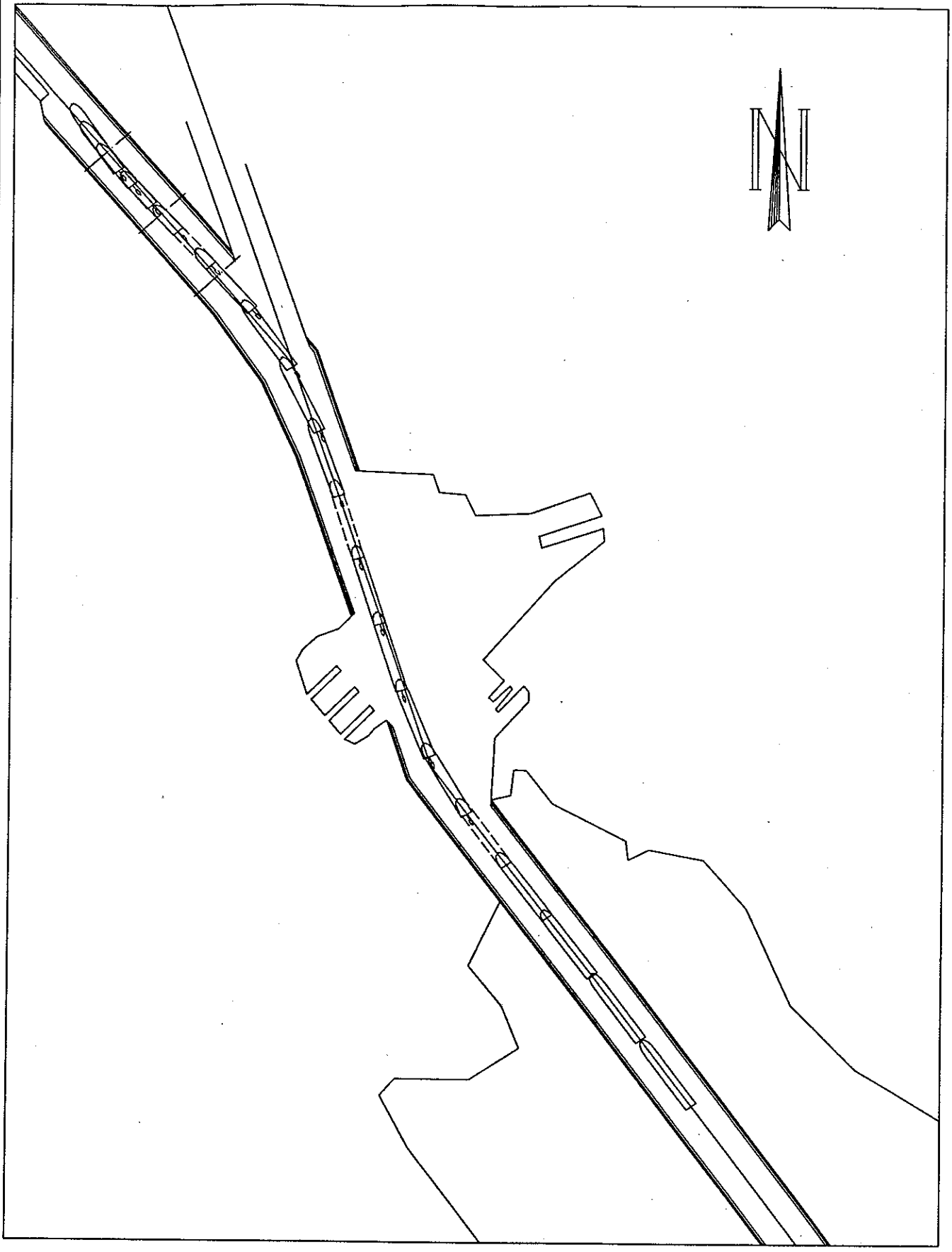
A-7



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment PIA Panama Canal Sailing to lock from Pacific Ocean.	Condition : 3 Run : 034 Wind : N 3 Bf Tide : +3.42m, flood
WATERBOUWKUNDIG LABORATORIUM Borgerhout-Antwerpen	M673 / A-8



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 3
Run : 035
Wind : NW 3 Bf
Tide : +3.42m, flood

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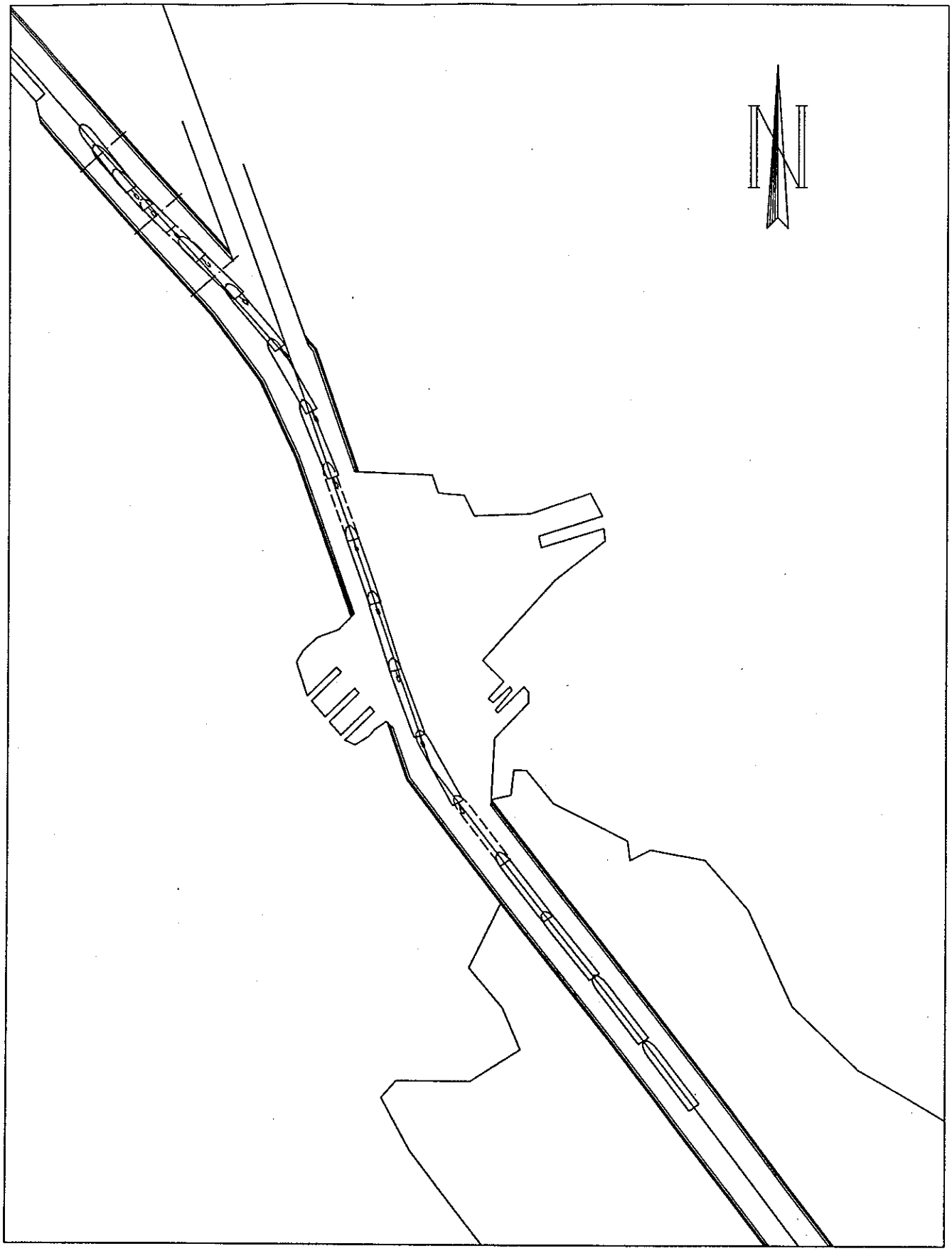
M673

A-9



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1APanama Canal
Sailing to lock from Pacific Ocean.

Condition : 3
Run : 036
Wind : W 3 Bf
Tide : +3.42m, flood

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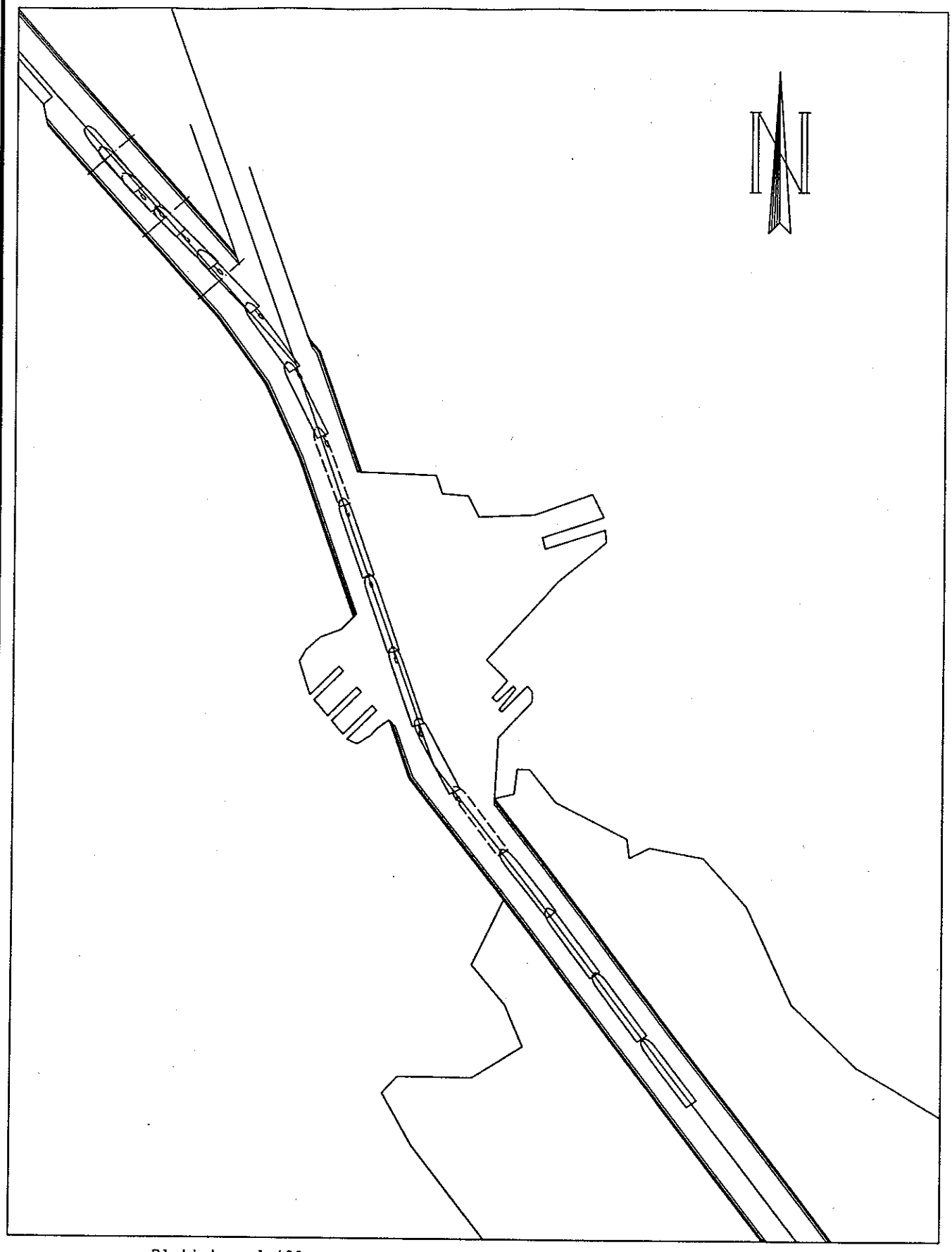
M673

A-10



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1ÅPanama Canal
Sailing to lock from Pacific Ocean.

Condition : 3
Run : 037
Wind : SW 3 Bf
Tide : +3.42m, flood

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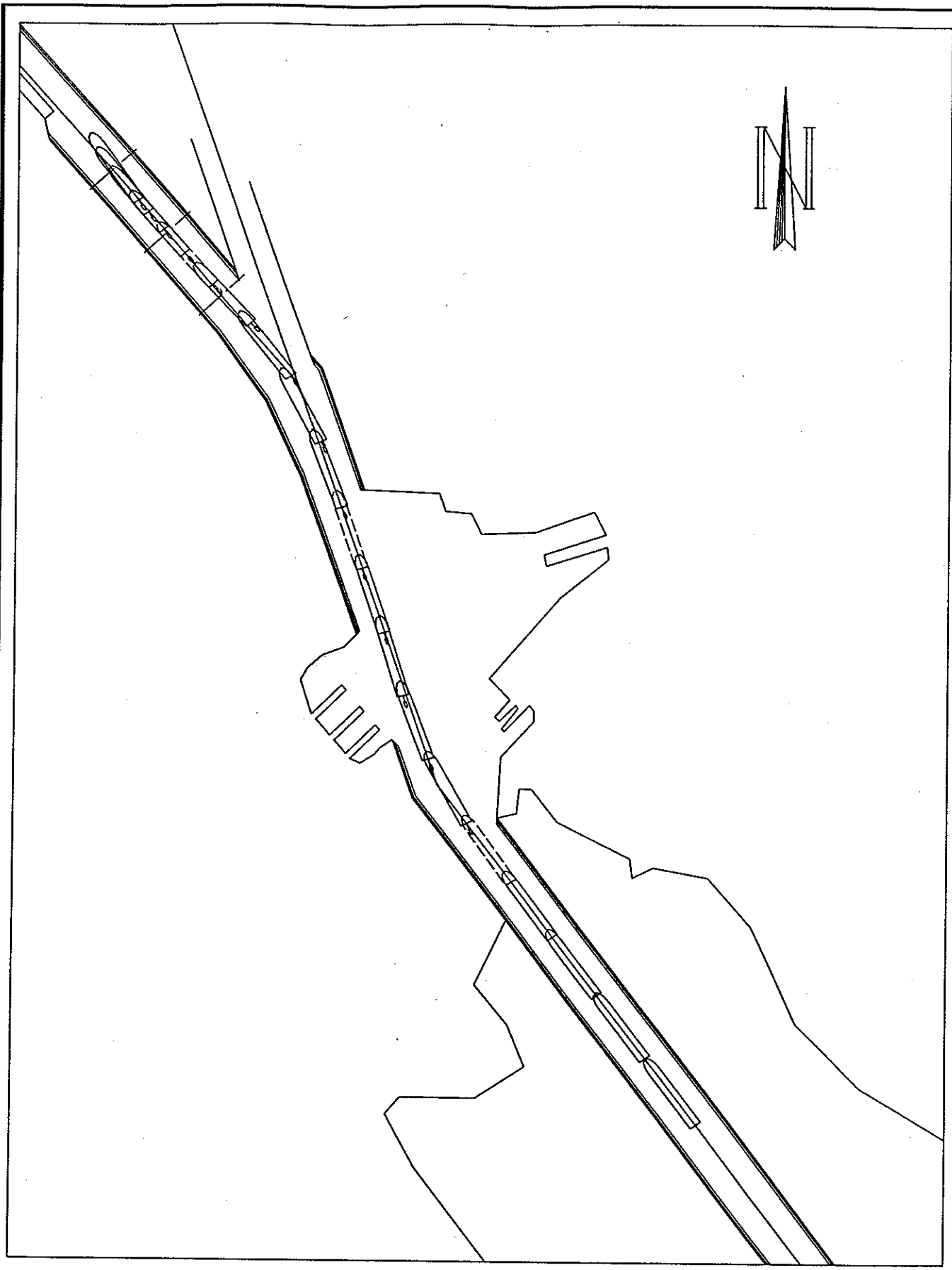
M673

A-11



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1APanama Canal
Sailing to lock from Pacific Ocean.

Condition : 3
Run : 038
Wind : S 3 Bf
Tide : +3.42m, flood

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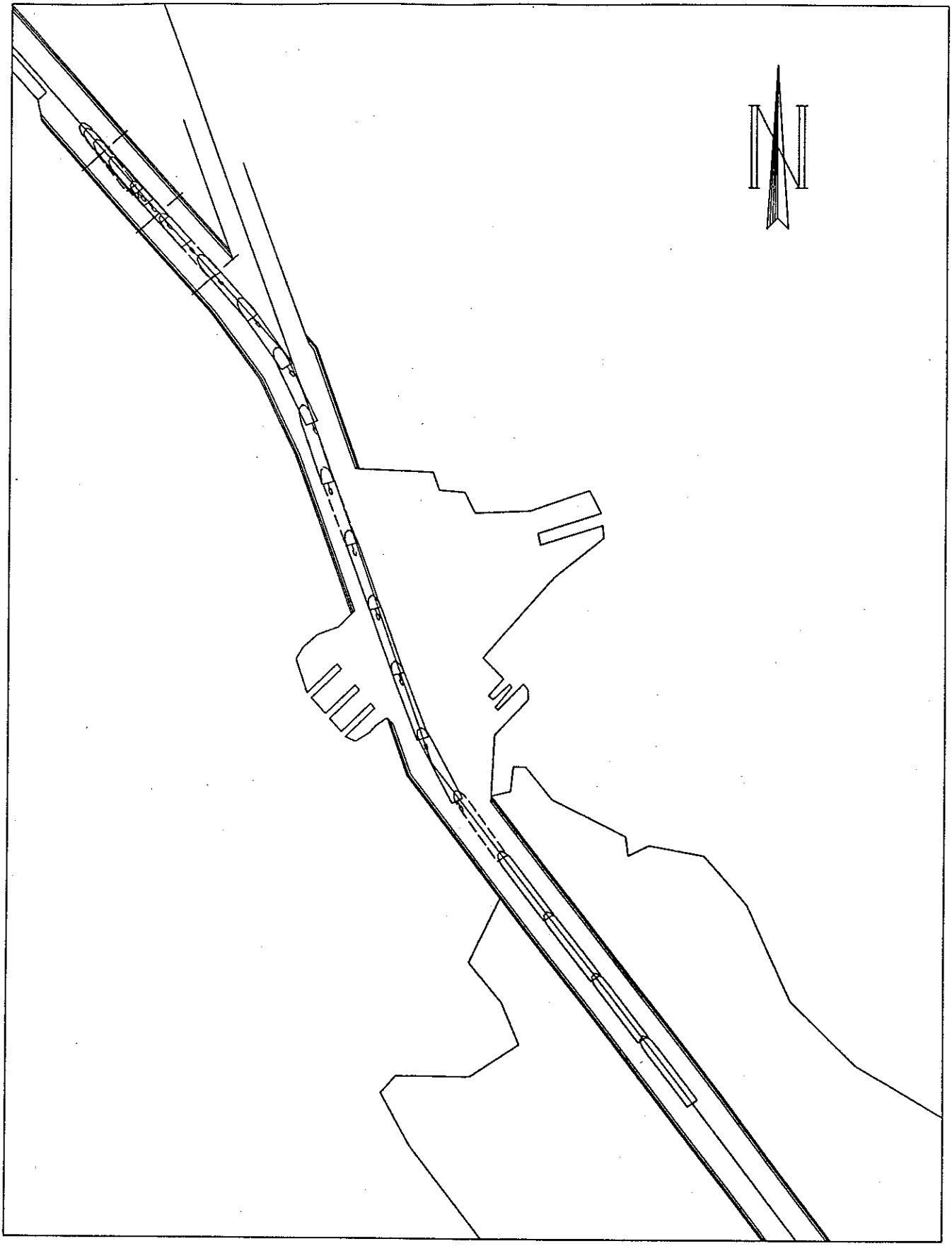
M673

A-12



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1APanama Canal
Sailing to lock from Pacific Ocean.

Condition : 4
Run : 007
Wind : N 5 Bf
Tide : Low Water

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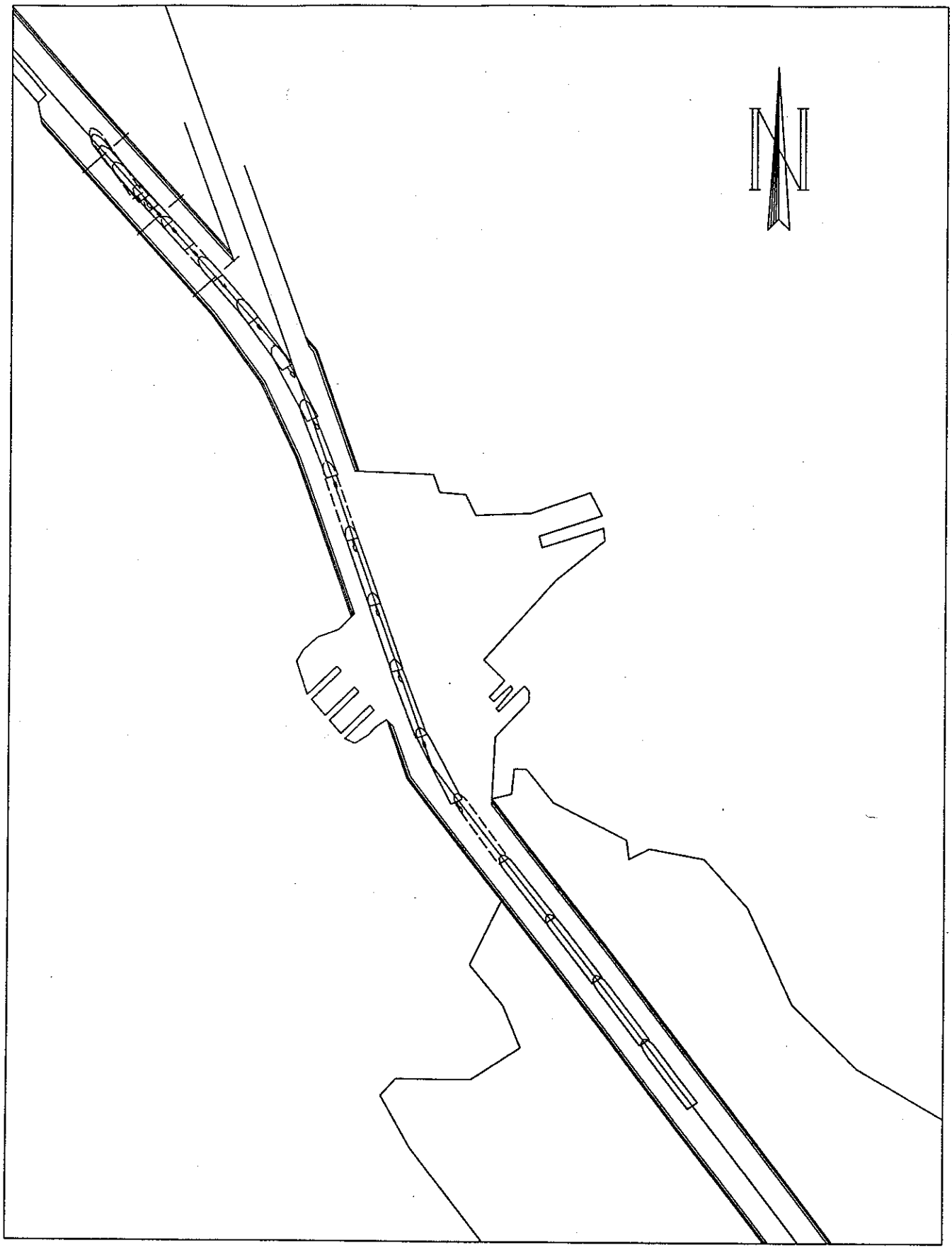
Borgerhout-Antwerpen

M673

A-13



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 4
Run : 009
Wind : NNW 5 Bf
Tide : Low Water

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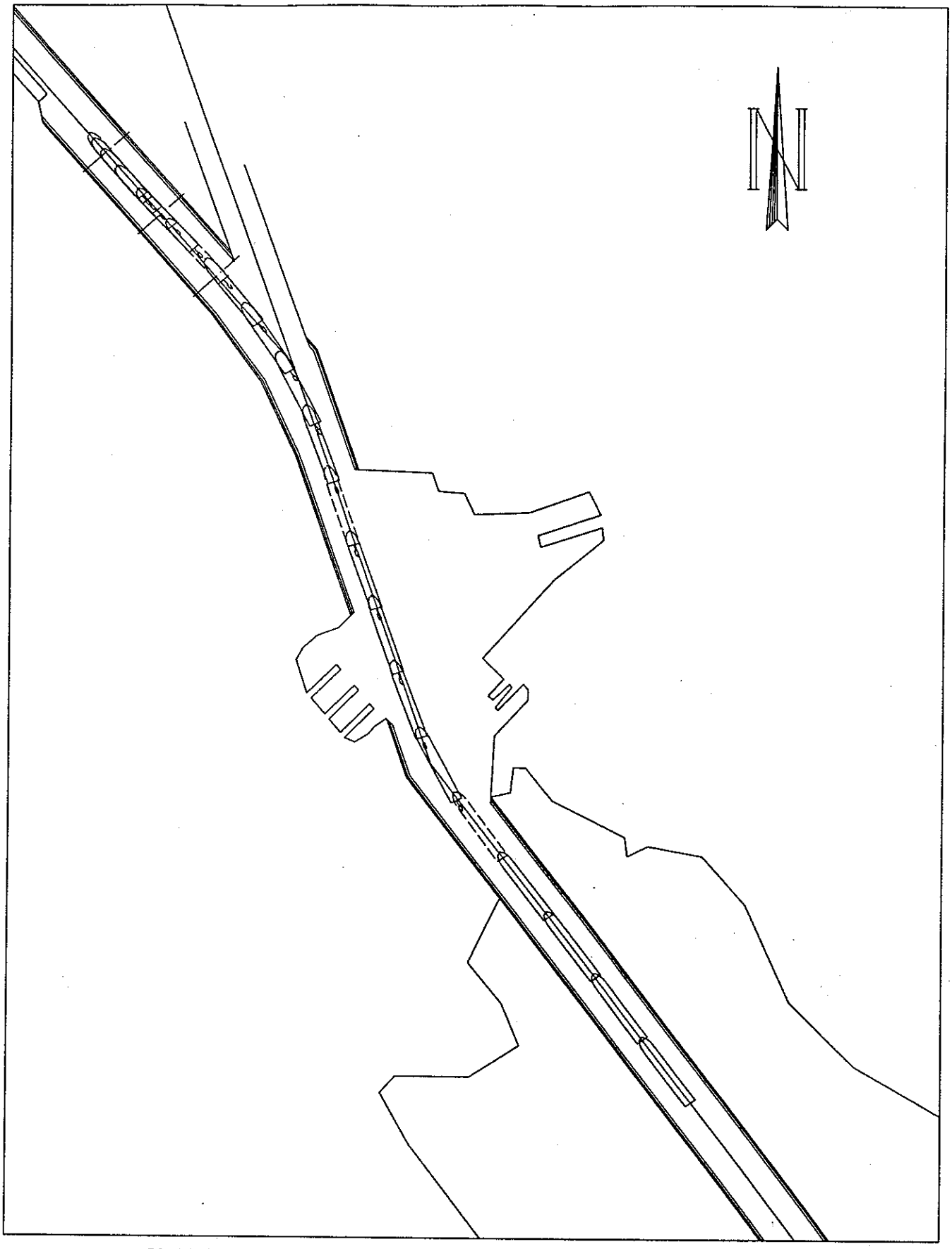
M673

A-14



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A-Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 4
Run : 011
Wind : NW 5 Bf
Tide : Low Water

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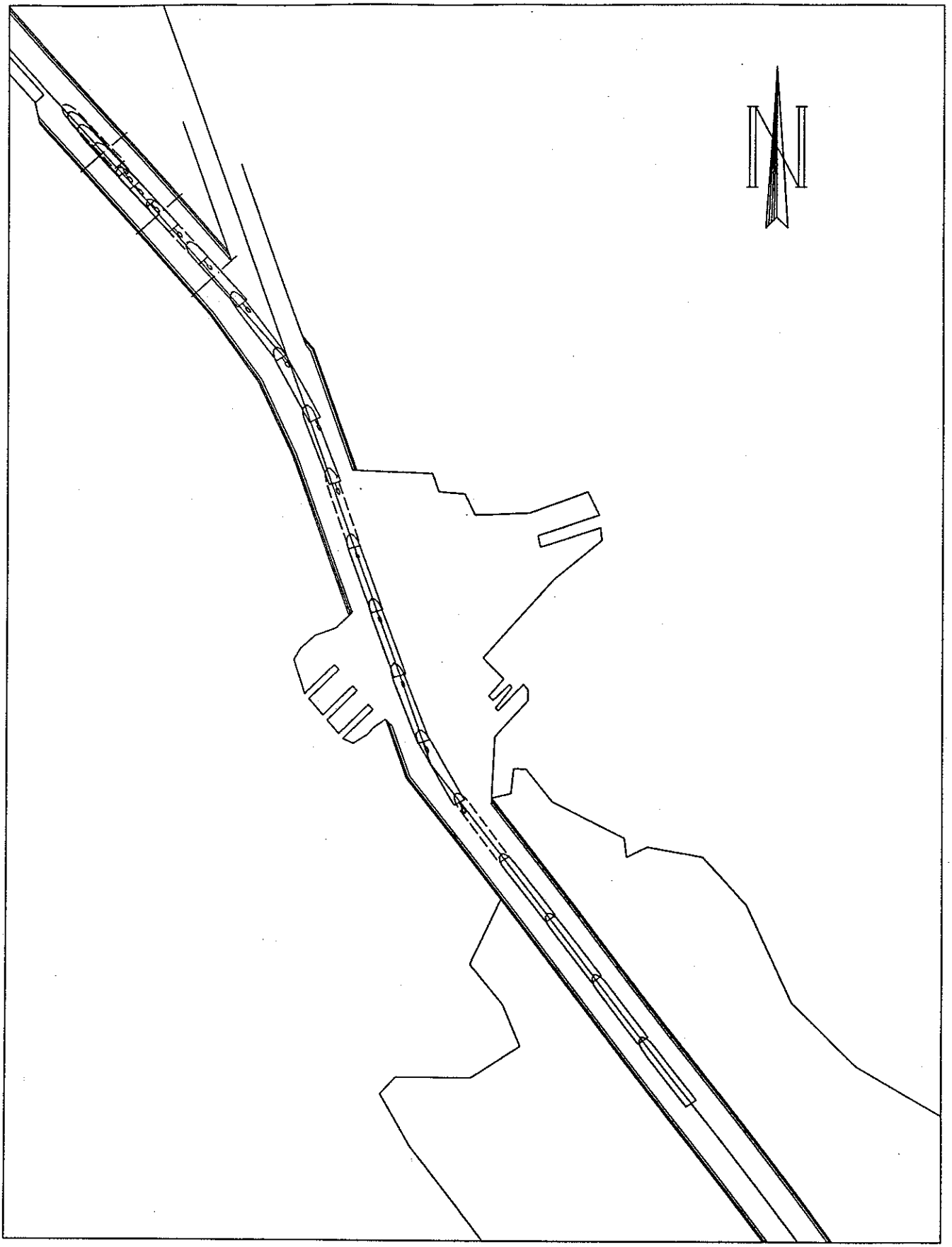
Borgerhout-Antwerpen

M673

A-15



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1APanama Canal
Sailing to lock from Pacific Ocean.

Condition : 4
Run : 024
Wind : W 5 Bf
Tide : Low Water

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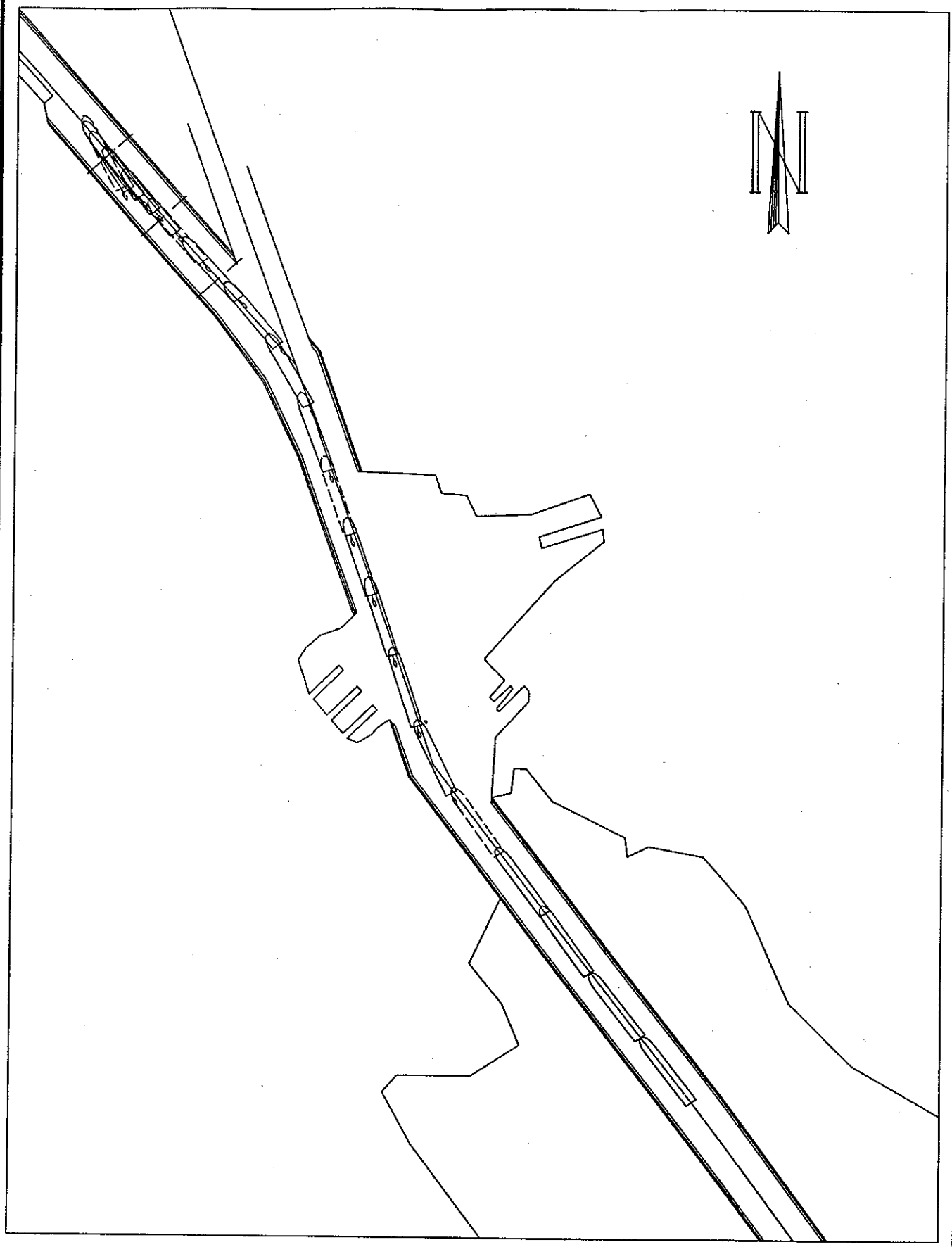
M673

A-16



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 5
Run : 008
Wind : N 5 Bf
Tide : +3.42m, flood

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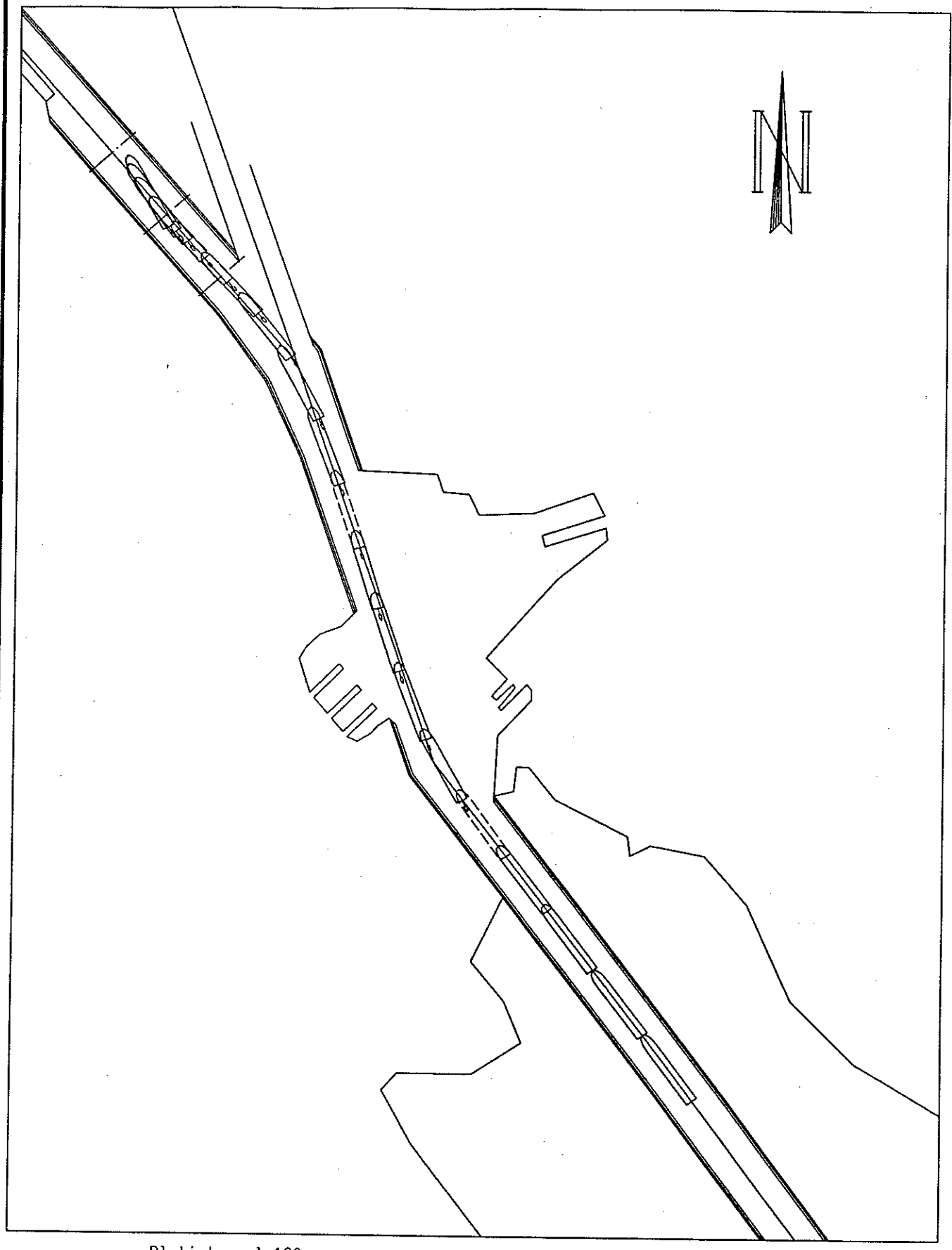
M673

A-17



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 5
Run : 010
Wind : NNW 5 Bf
Tide : +3.42m, flood

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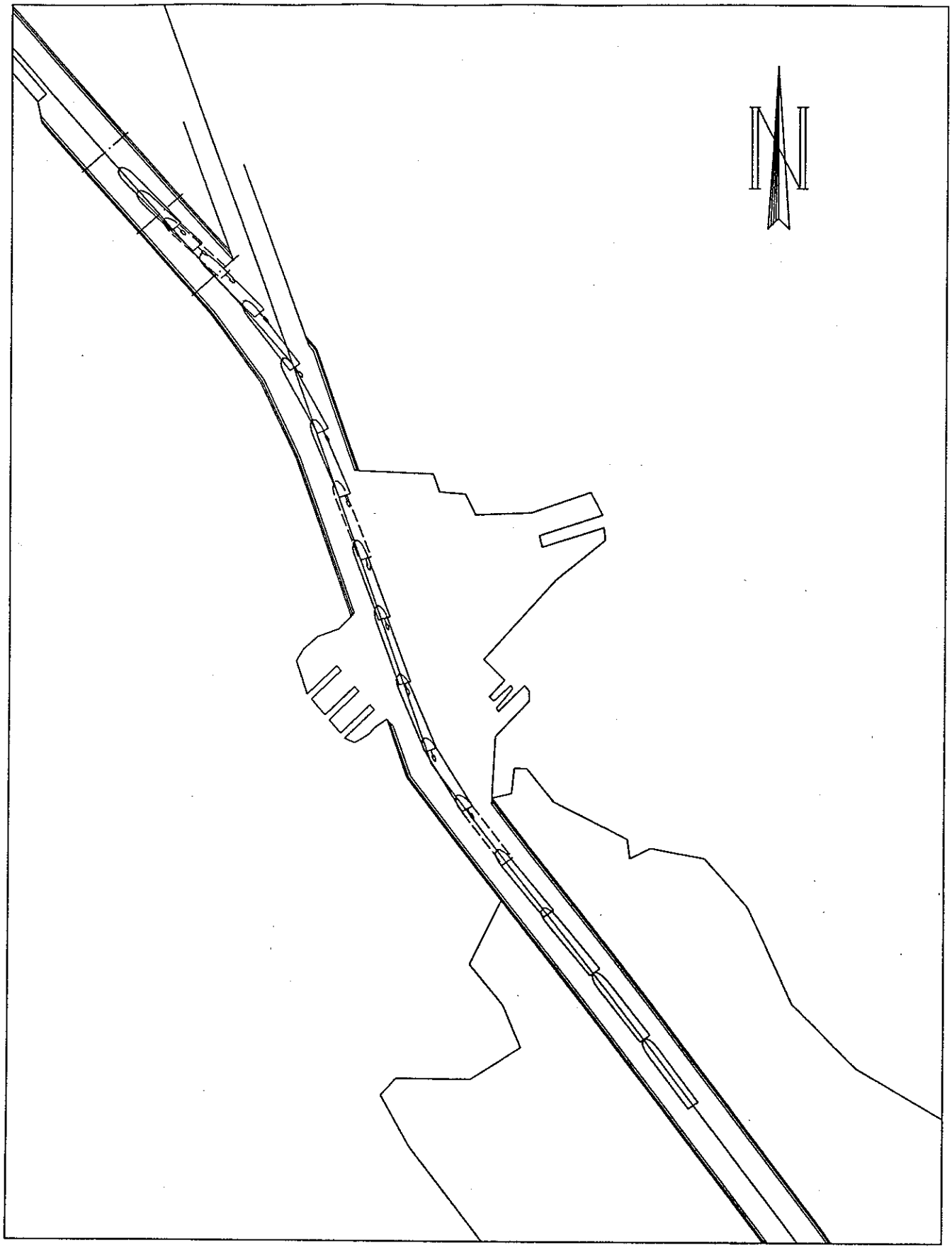
M673

A-18



Plotinterval 120. s

Scale 1:/25000.

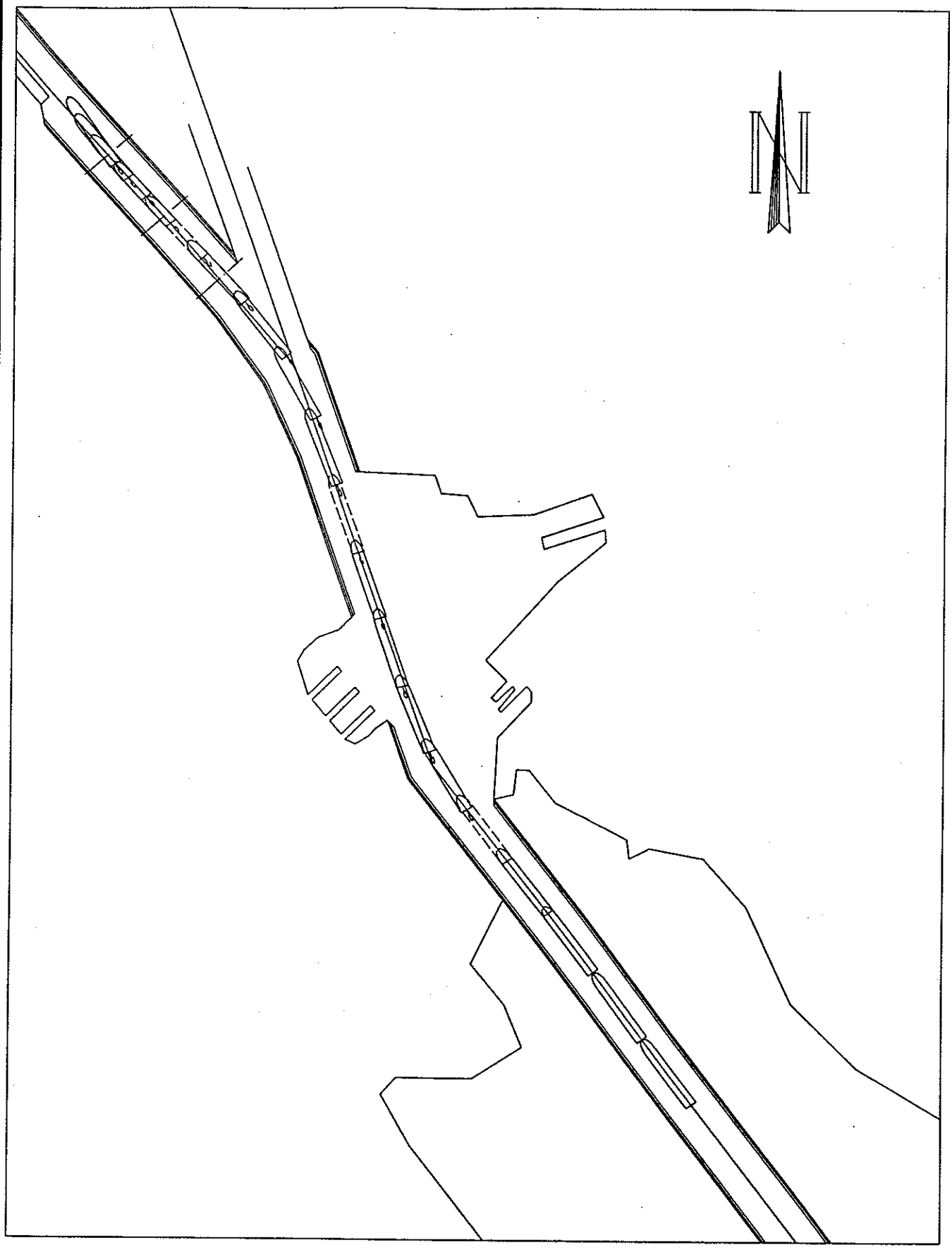
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 5
Run : 046
Wind : W 5 Bf
Tide : +3.42m, flood

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-19



Plotinterval 120. s

Scale 1:/25000.

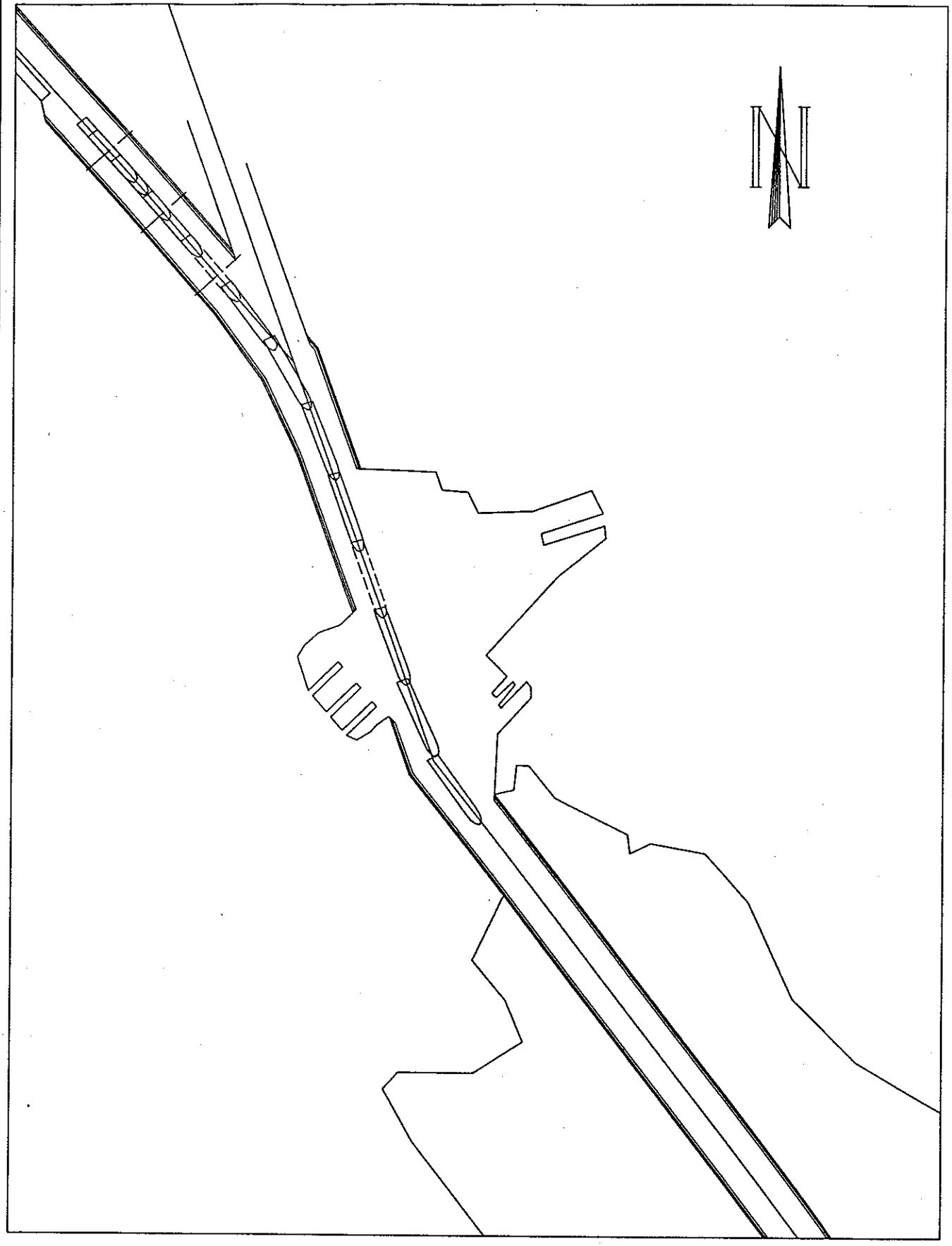
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Pacific Ocean.

Condition : 5
Run : 047
Wind : NW 5 Bf
Tide : +3.42m, flood

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-20



Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1APanama Canal
Sailing from lock to Pacific Ocean.

Condition : 6
Run : 040 (no tugs)
Wind : N 3 Bf
Tide : +2.82m, ebb

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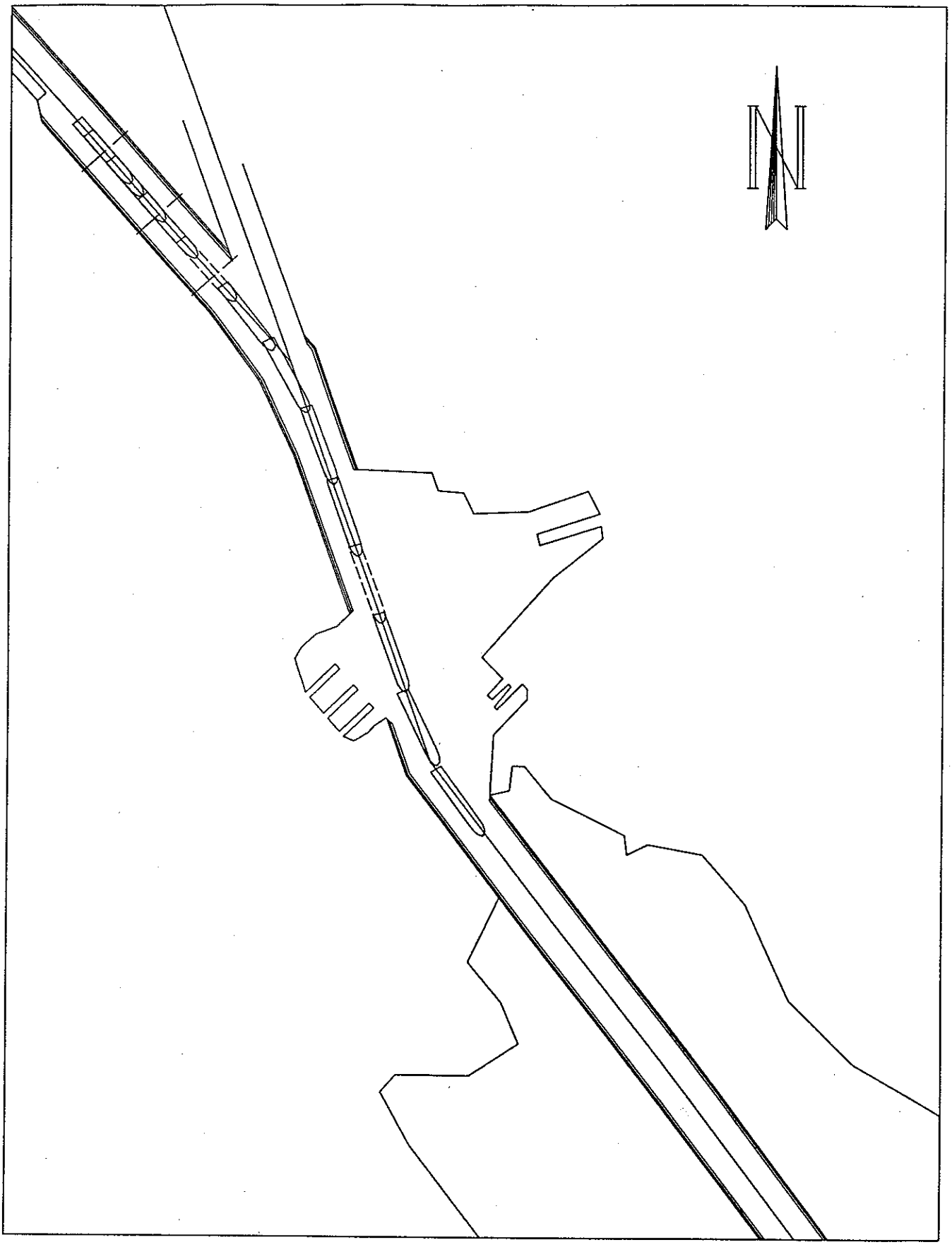
M673

A-21

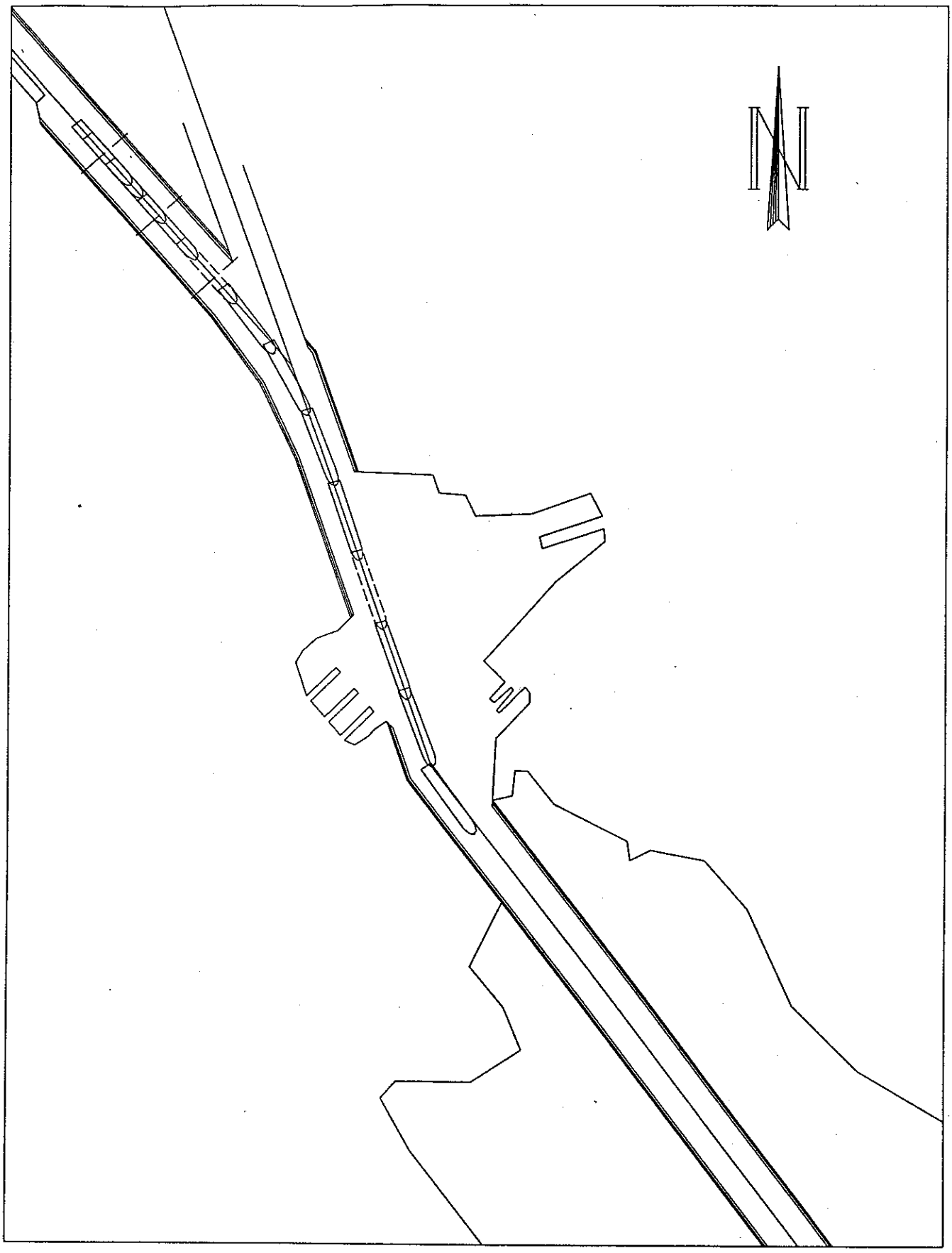


Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal Sailing from lock to Pacific Ocean.	Condition : 6 Run : 041 (no tugs) Wind : NW 3 Bf Tide : +2.82m, ebb

WATERBOUWKUNDIG LABORATORIUM Borgerhout-Antwerpen	M673	A-22

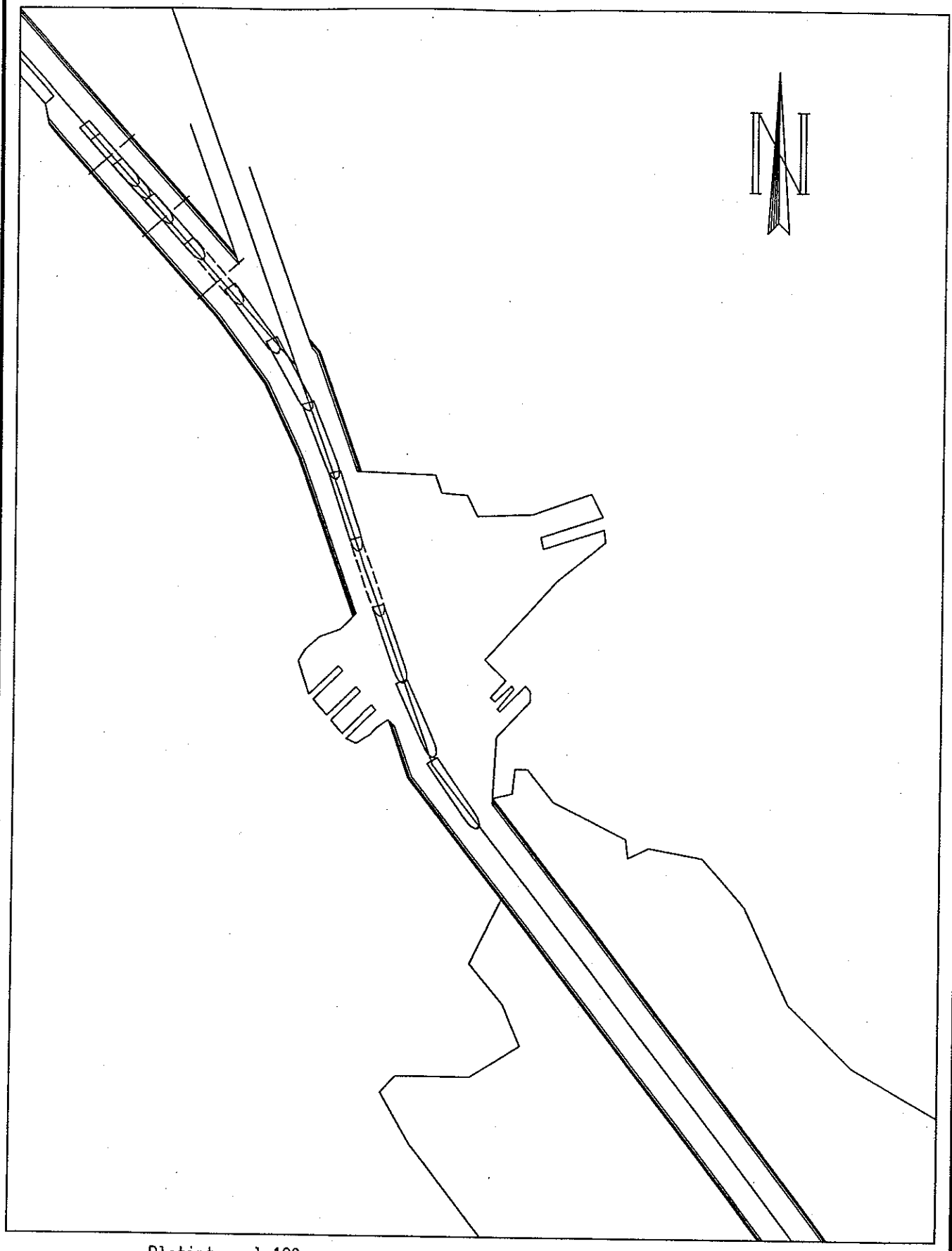


Plotinterval 120. s

Scale 1:/25000.

Fast-time: Channel alignment P1A Panama Canal Sailing from lock to Pacific Ocean.	Condition : 6 Run : 042 (no tugs) Wind : W 3 Bf Tide : +2.82m, ebb

WATERBOUWKUNDIG LABORATORIUM Borgerhout-Antwerpen	M673	A-23



Plotinterval 120. s

Scale 1:/25000.

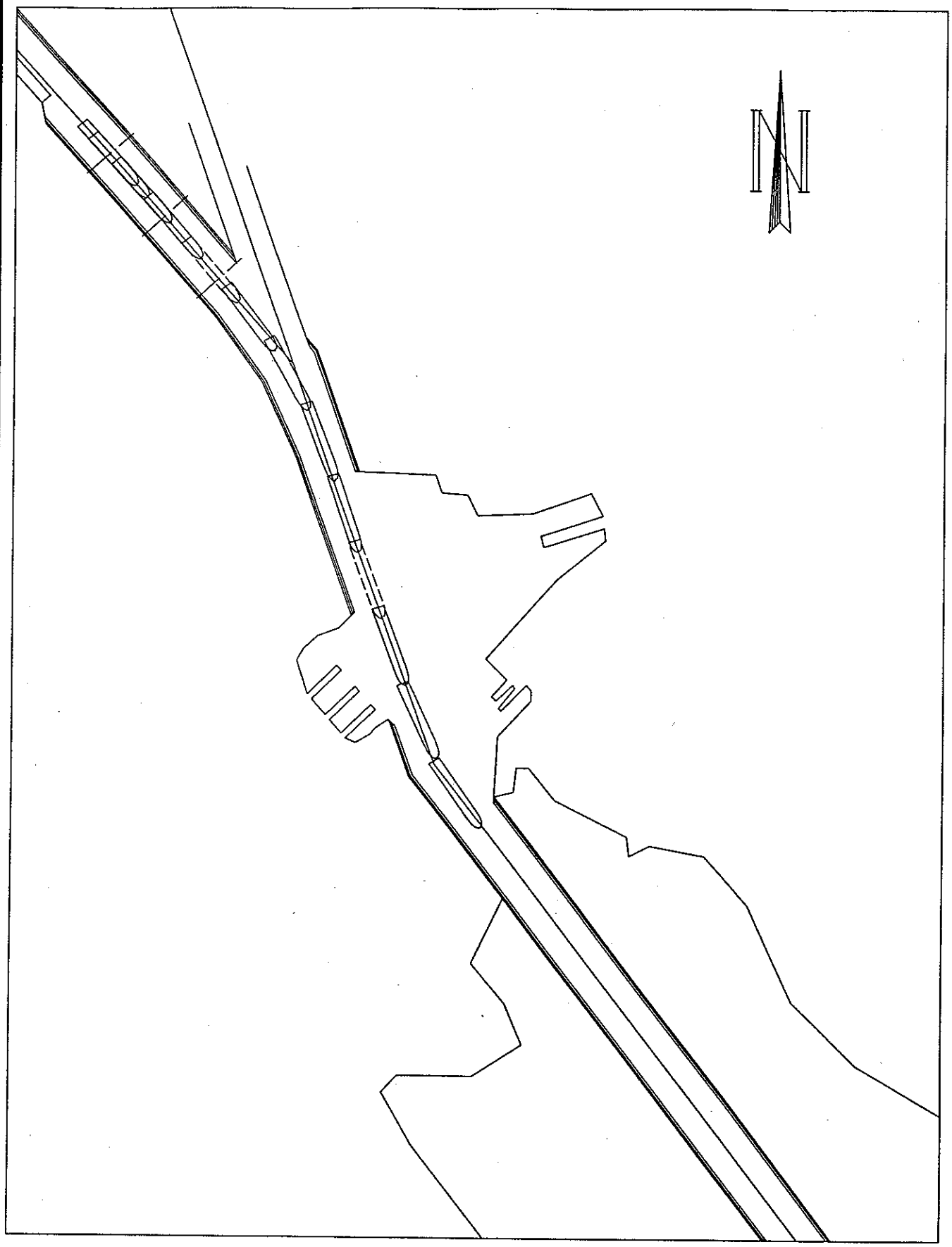
Fast-time: Channel alignment P1A Panama Canal
Sailing from lock to Pacific Ocean.

Condition : 6
Run : 043 (no tugs)
Wind : SW 3 Bf
Tide : +2.82m, ebb

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-24



Plotinterval 120. s

Scale 1:/25000.

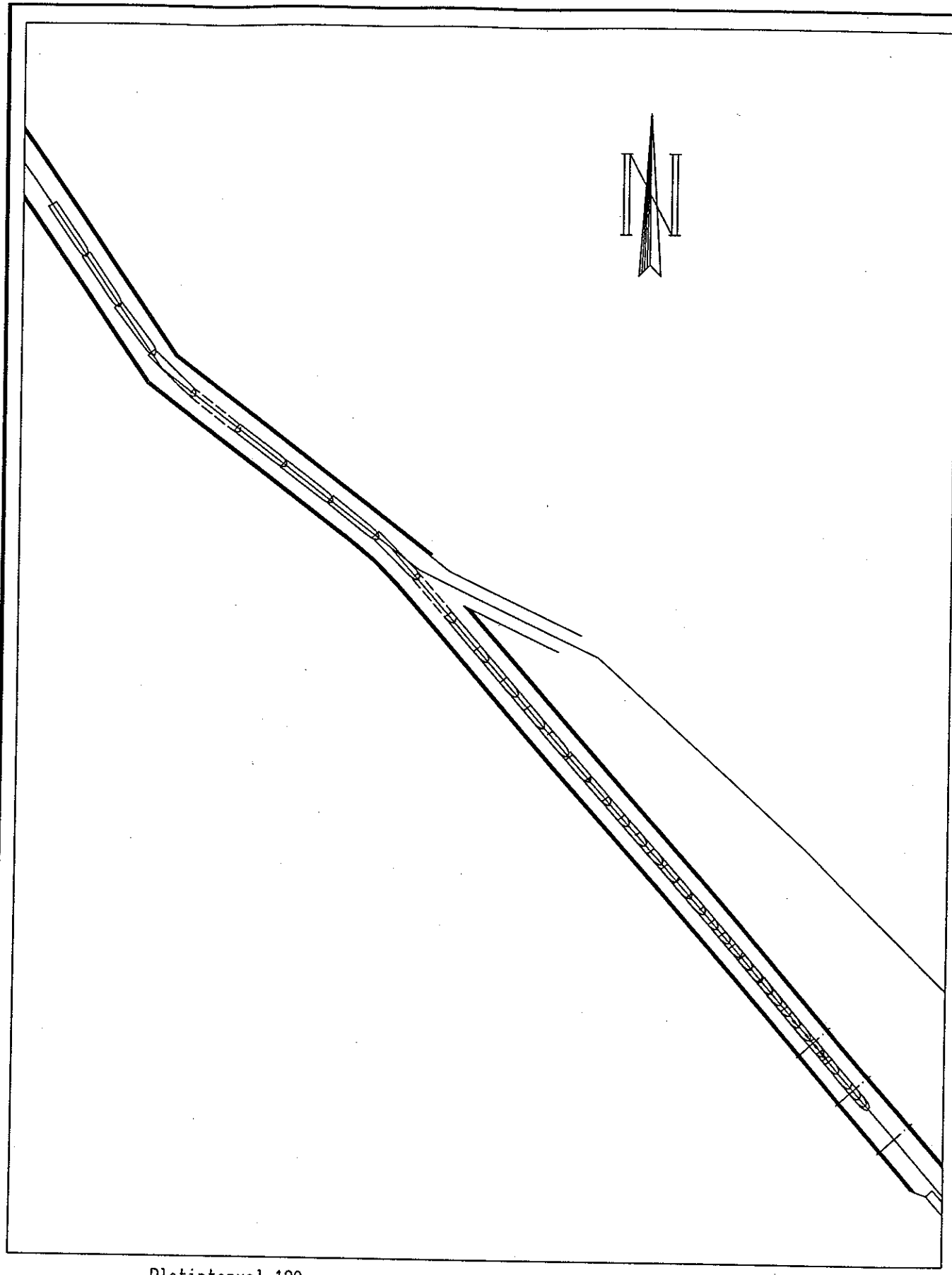
Fast-time: Channel alignment P1A Panama Canal
Sailing from lock to Pacific Ocean.

Condition : 6
Run : 044 (no tugs)
Wind : S 3 Bf
Tide : +2.82m, ebb

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-25



Plotinterval 120. s

Scale 1:/33000.

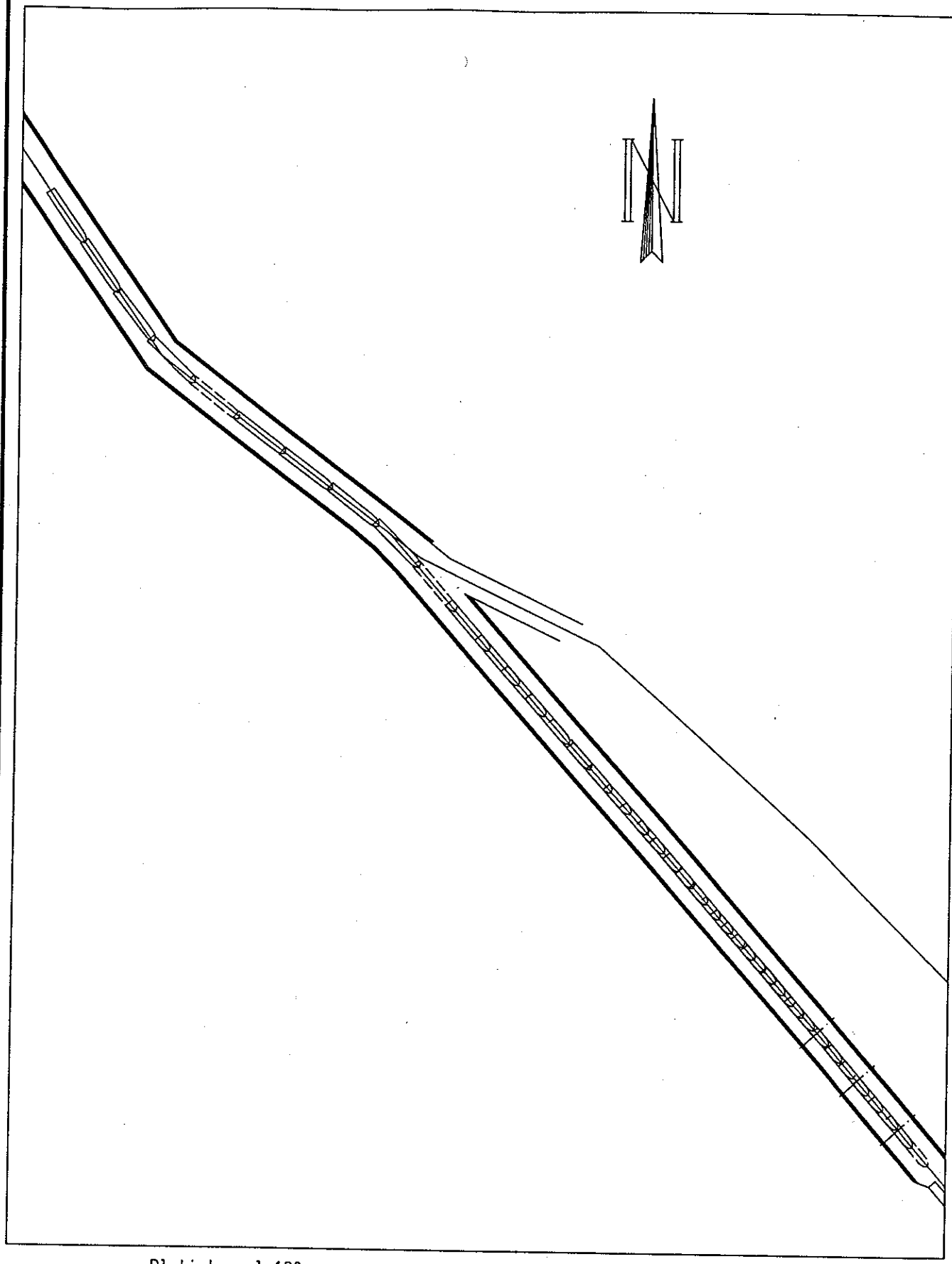
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Lake Gatn .

Condition : 7
Run : 013
Wind : no wind
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-26



Plotinterval 120. s

Scale 1:/33000.

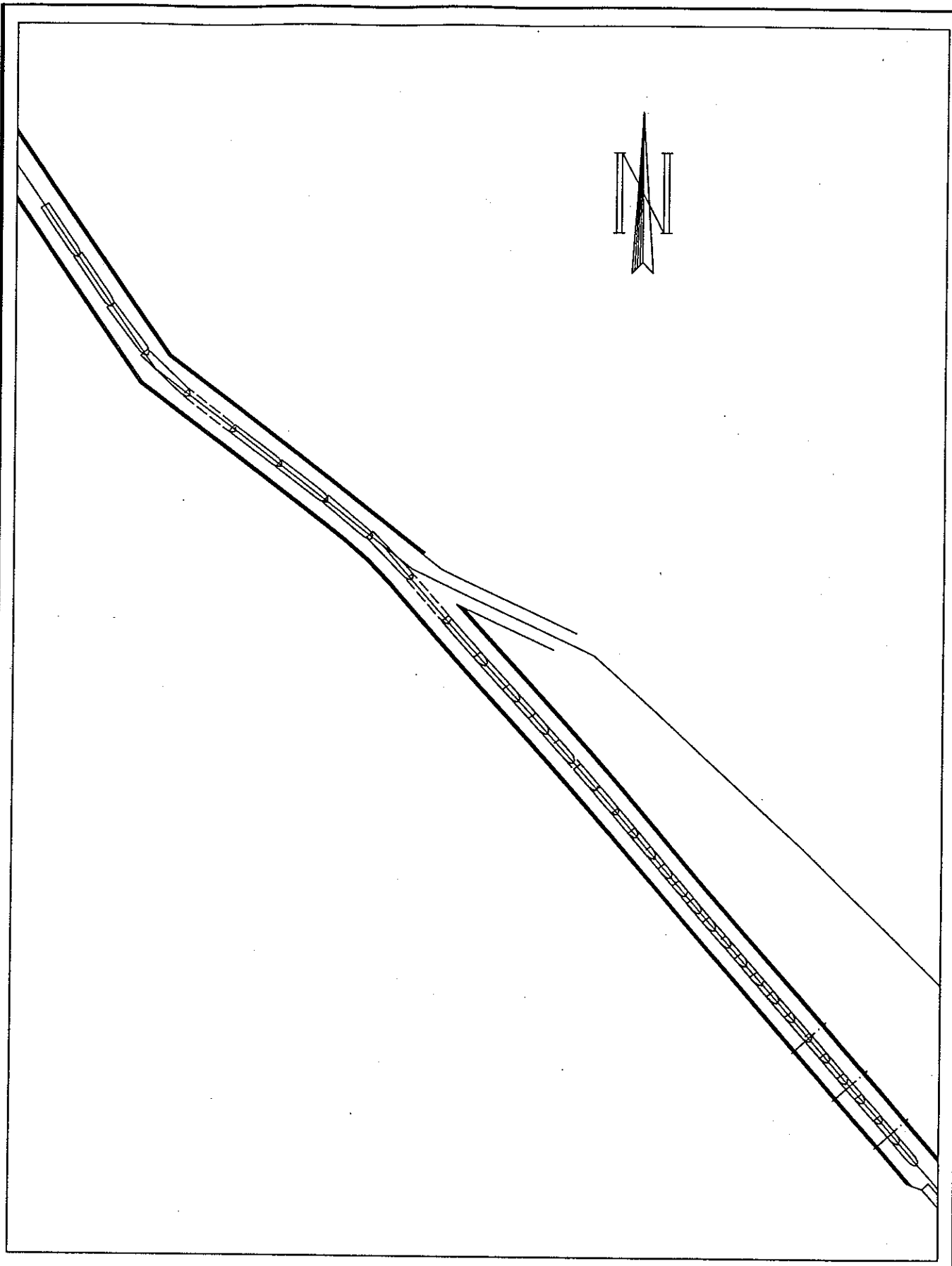
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Lake Gatn.

Condition : 7
Run : 018 (no tugs)
Wind : no wind
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-27



Plotinterval 120. s

Scale 1:/33000.

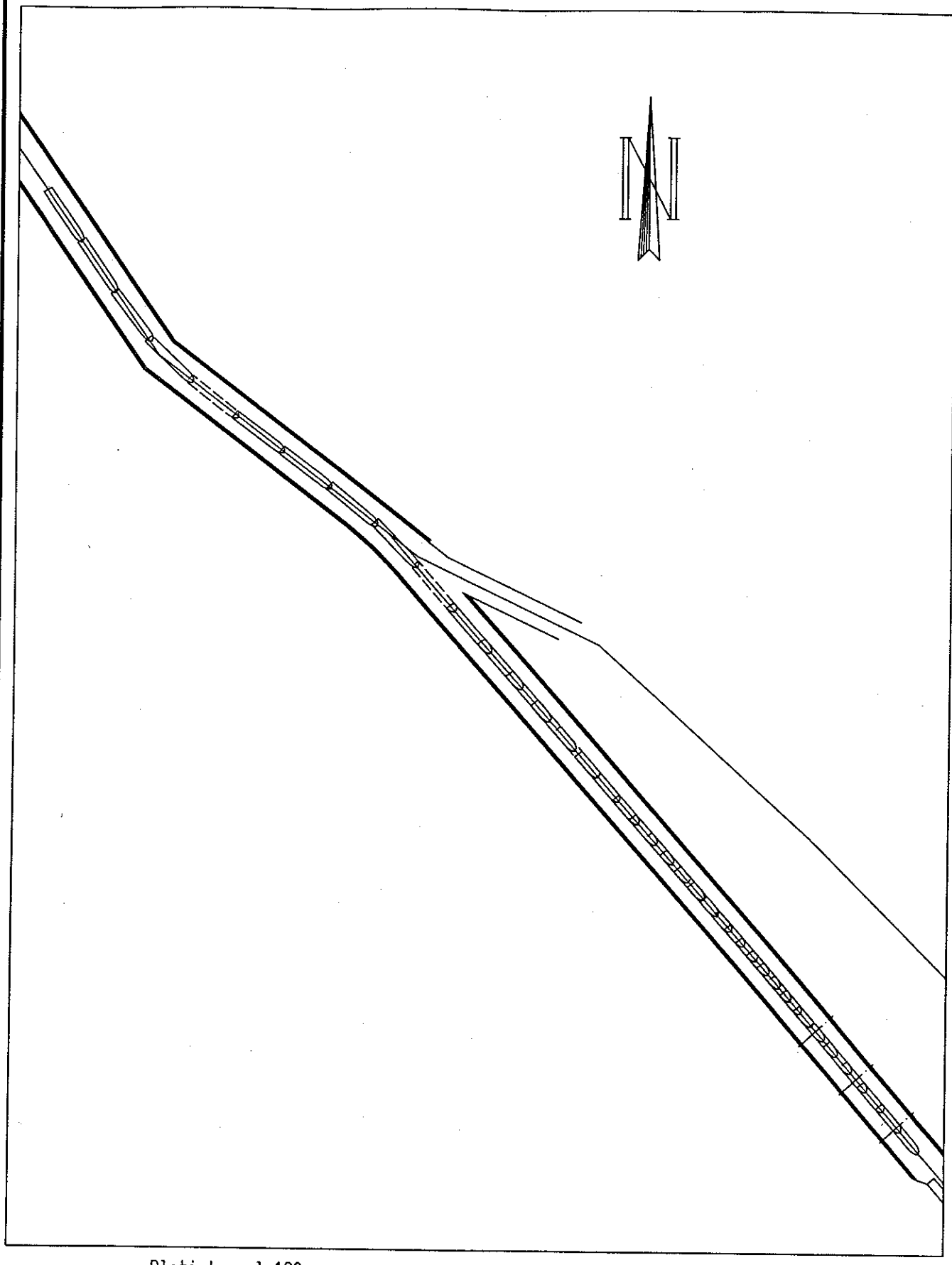
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Lake Gatn .

Condition : 8
Run : 030 (no tugs)
Wind : N 3 Bf
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-28



Plotinterval 120. s

Scale 1:/33000.

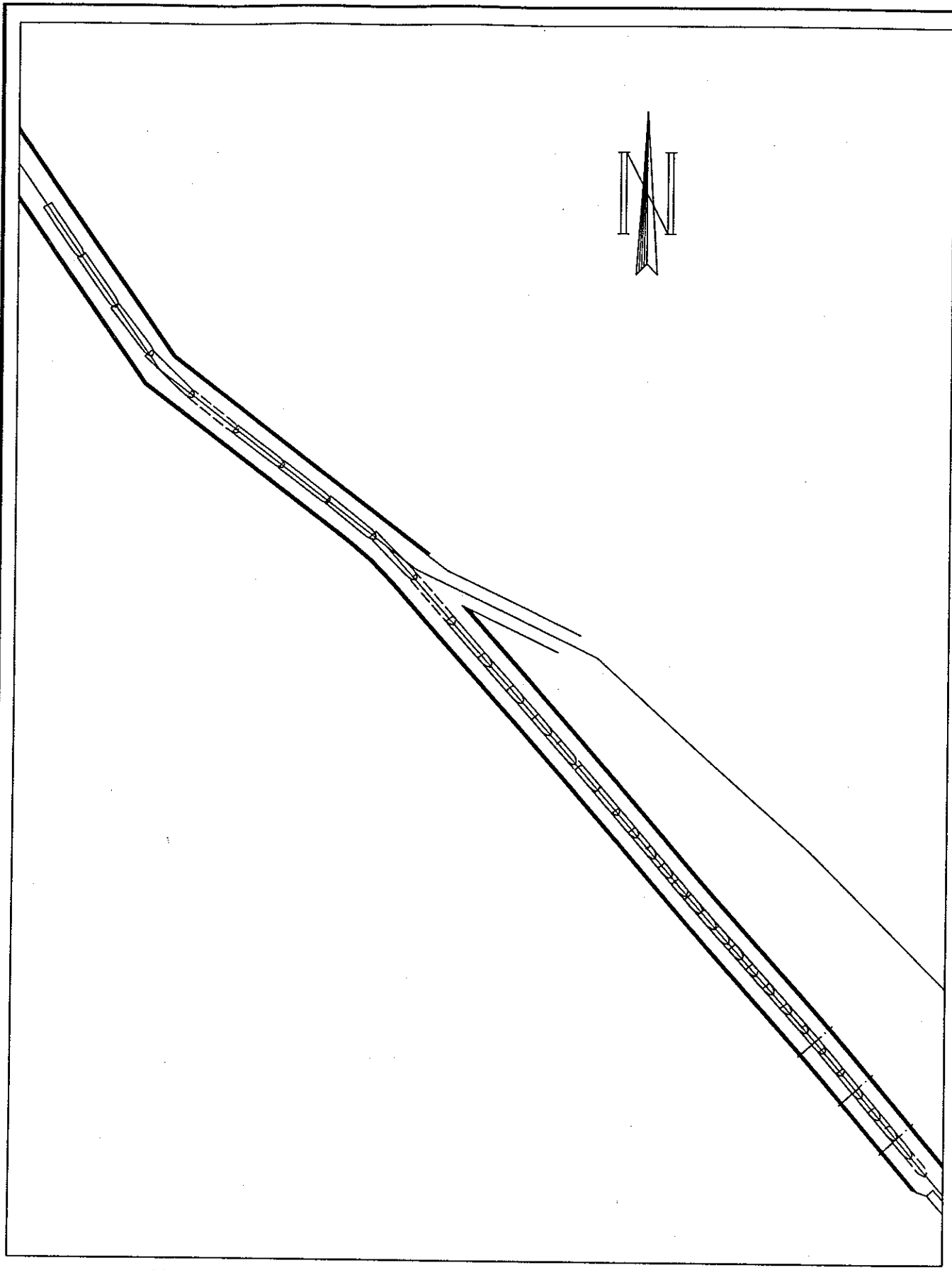
Fast-time: Channel alignment P1APanama Canal
Sailing to lock from Lake Gatn .

Condition : 8
Run : 031 (no tugs)
Wind : NW 3 Bf
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-29



Plotinterval 120. s

Scale 1:/33000.

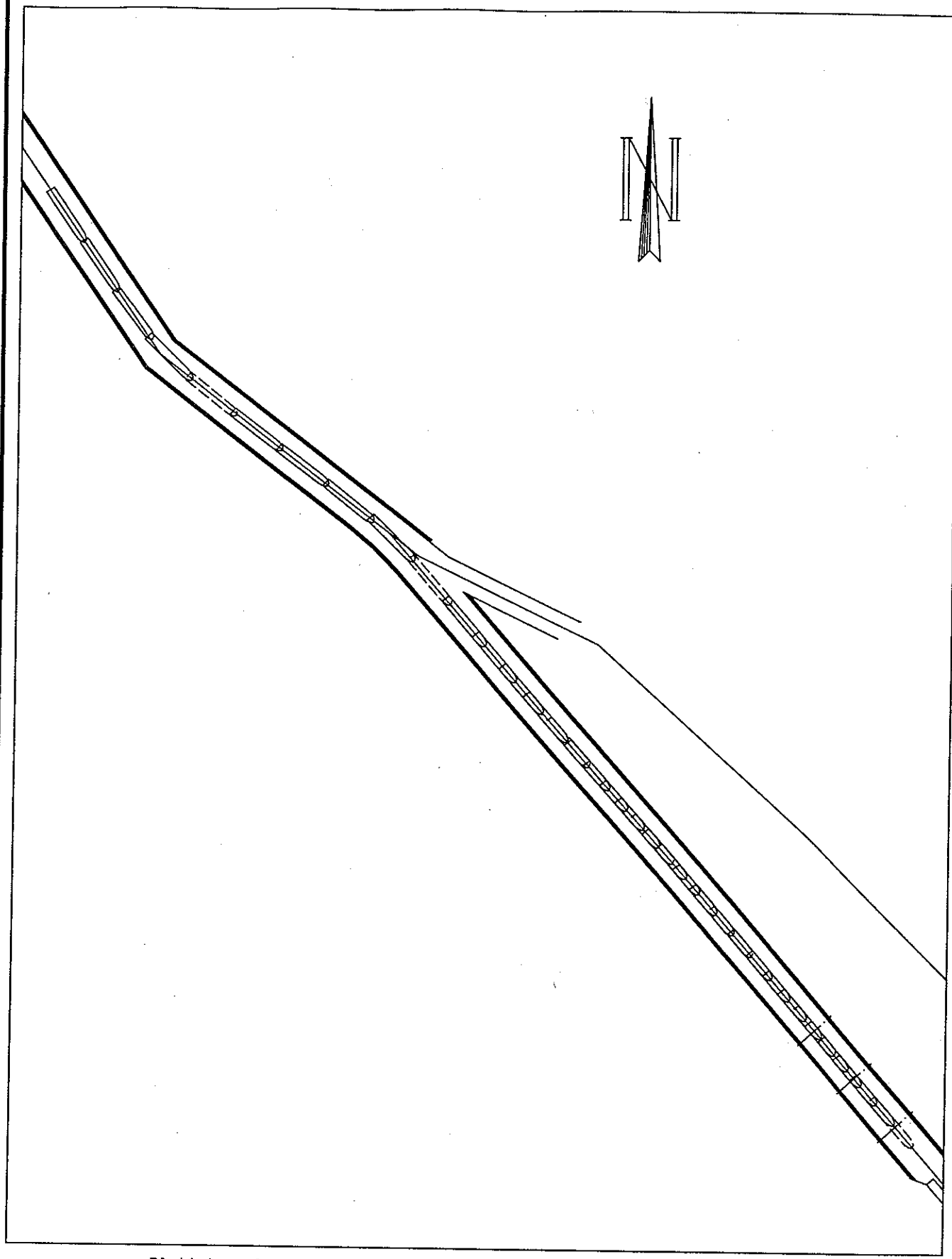
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Lake Gatn .

Condition : 8
Run : 032 (no tugs)
Wind : W 3 Bf
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-30



Plotinterval 120. s

Scale 1:/33000.

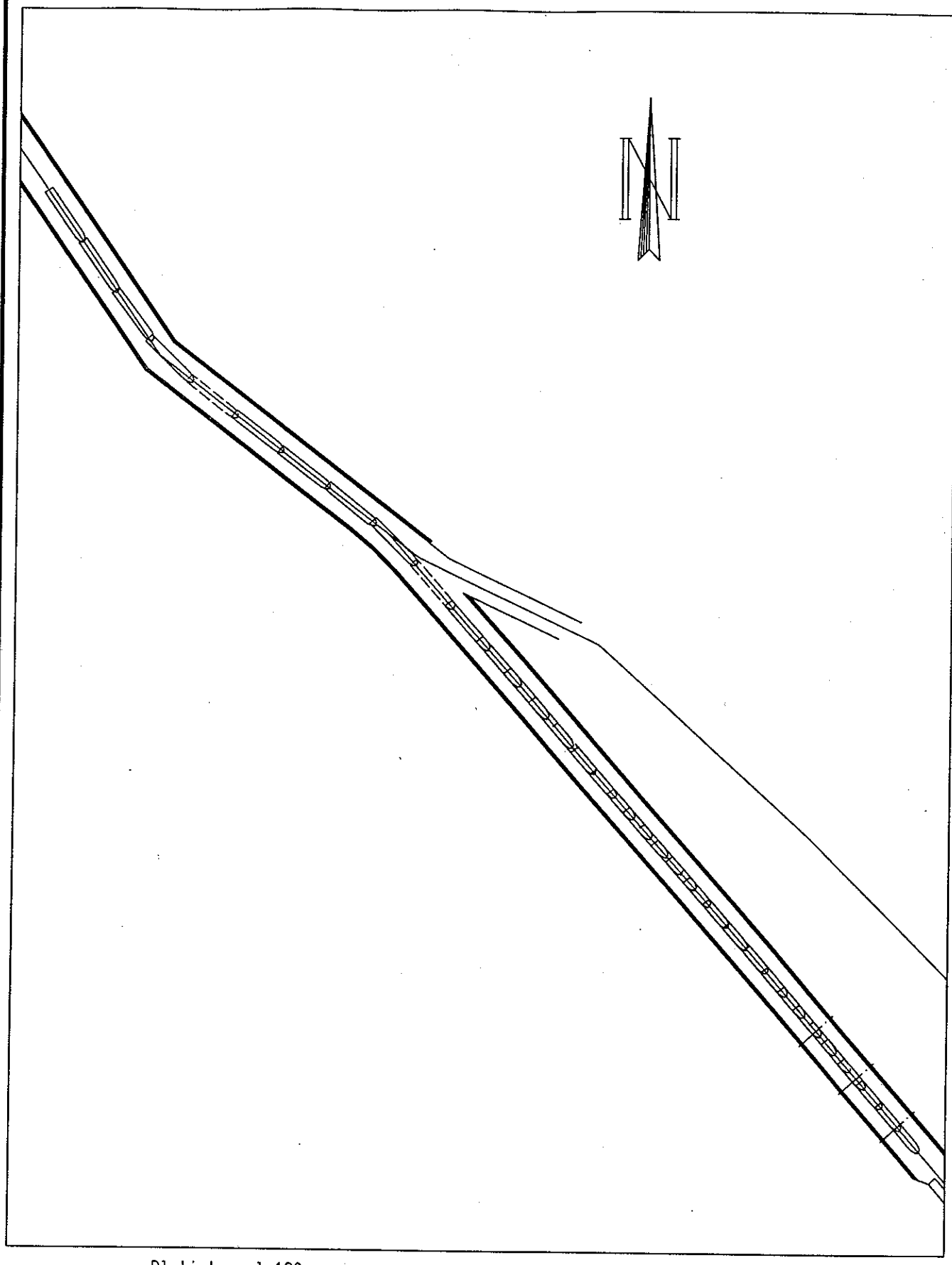
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Lake Gatn .

Condition : 8
Run : 033 (no tugs)
Wind : S 3 Bf
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-31



Plotinterval 120. s

Scale 1:/33000.

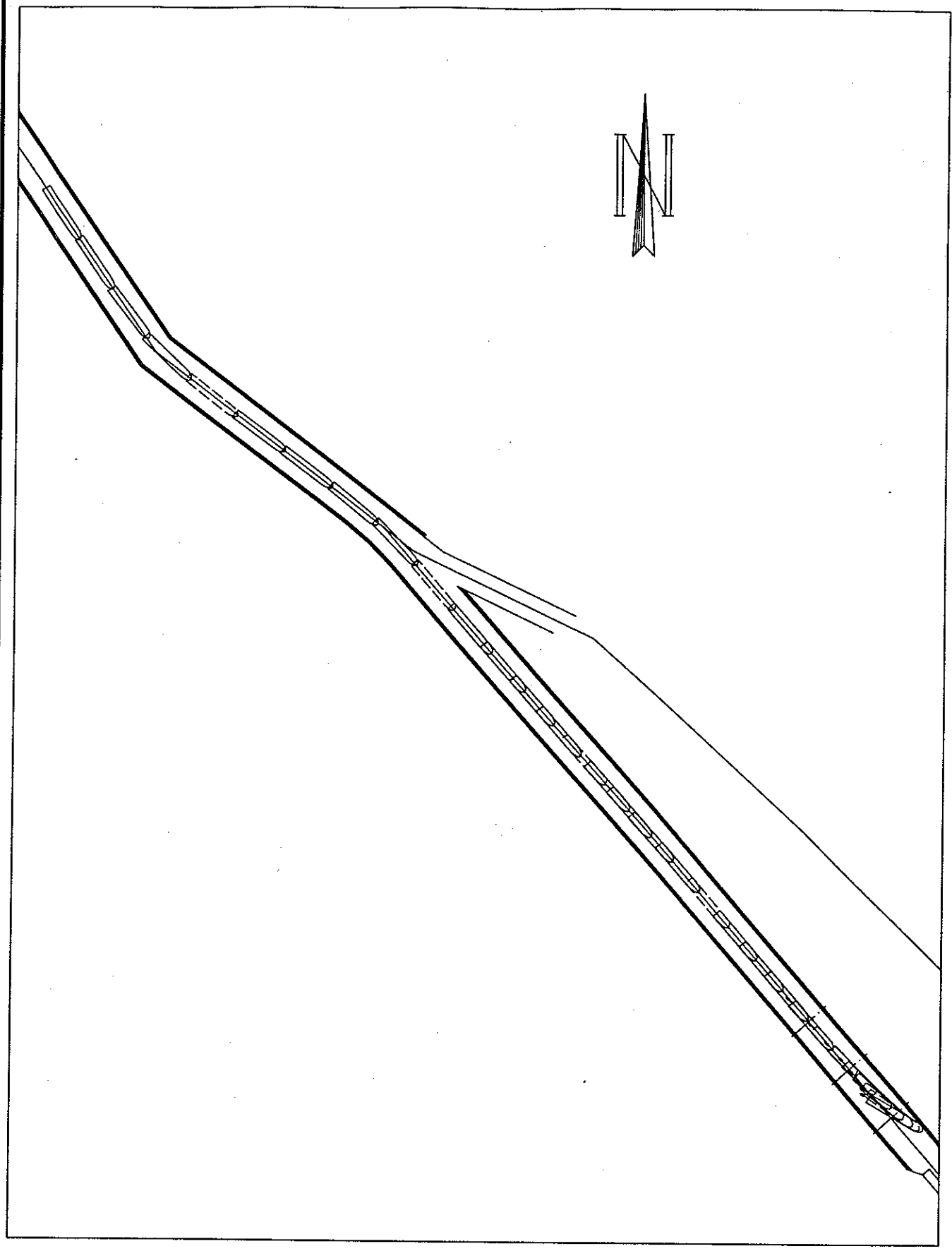
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Lake Gatn .

Condition : 8
Run : 019 (no tugs)
Wind : SW 3 Bf
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-32



Plotinterval 120. s

Scale 1:/33000.

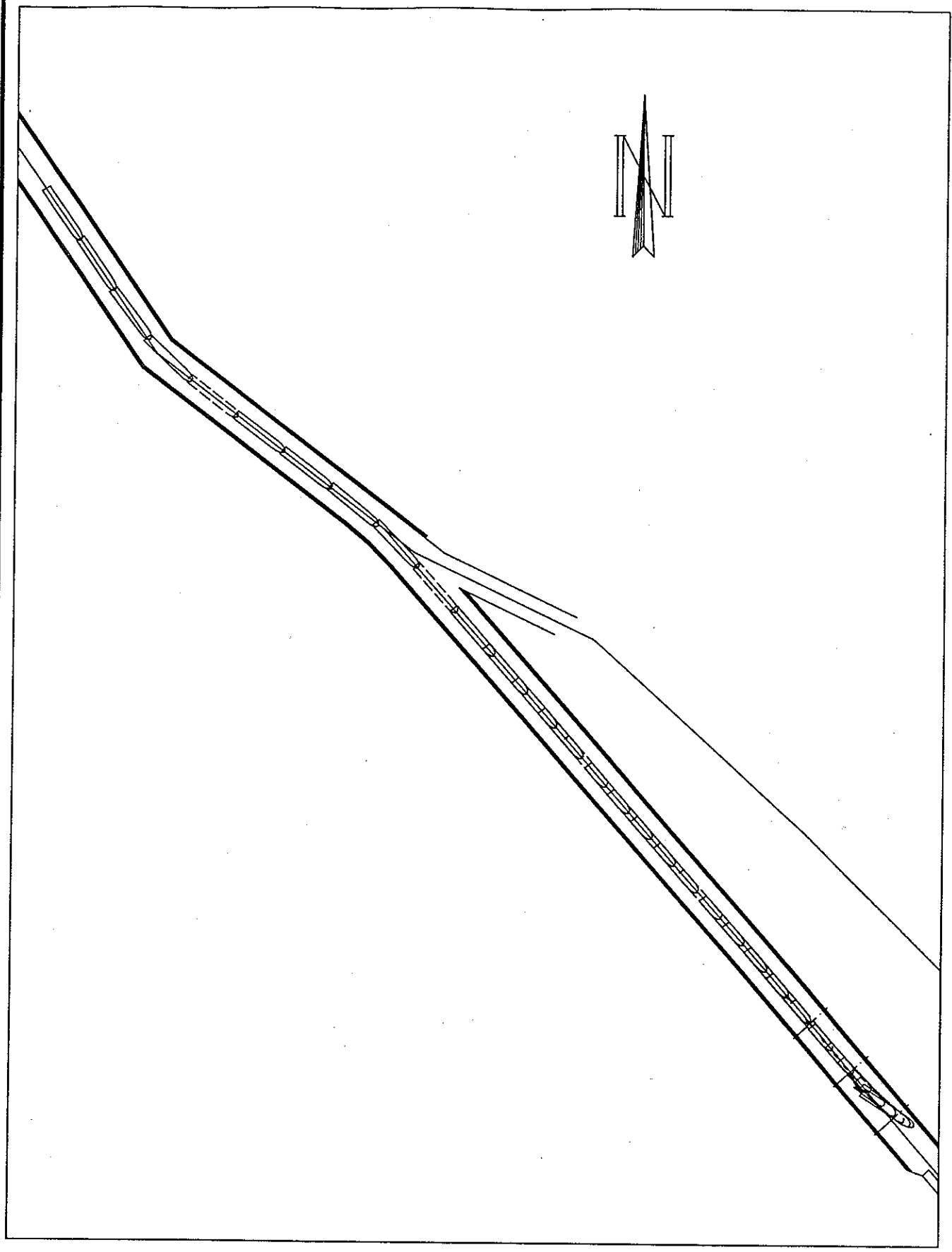
Fast-time: Channel alignment P1APanama Canal
Sailing to lock from Lake Gatn .

Condition : 9
Run : 015
Wind : N 5 Bf
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-33



Plotinterval 120. s

Scale 1:/33000.

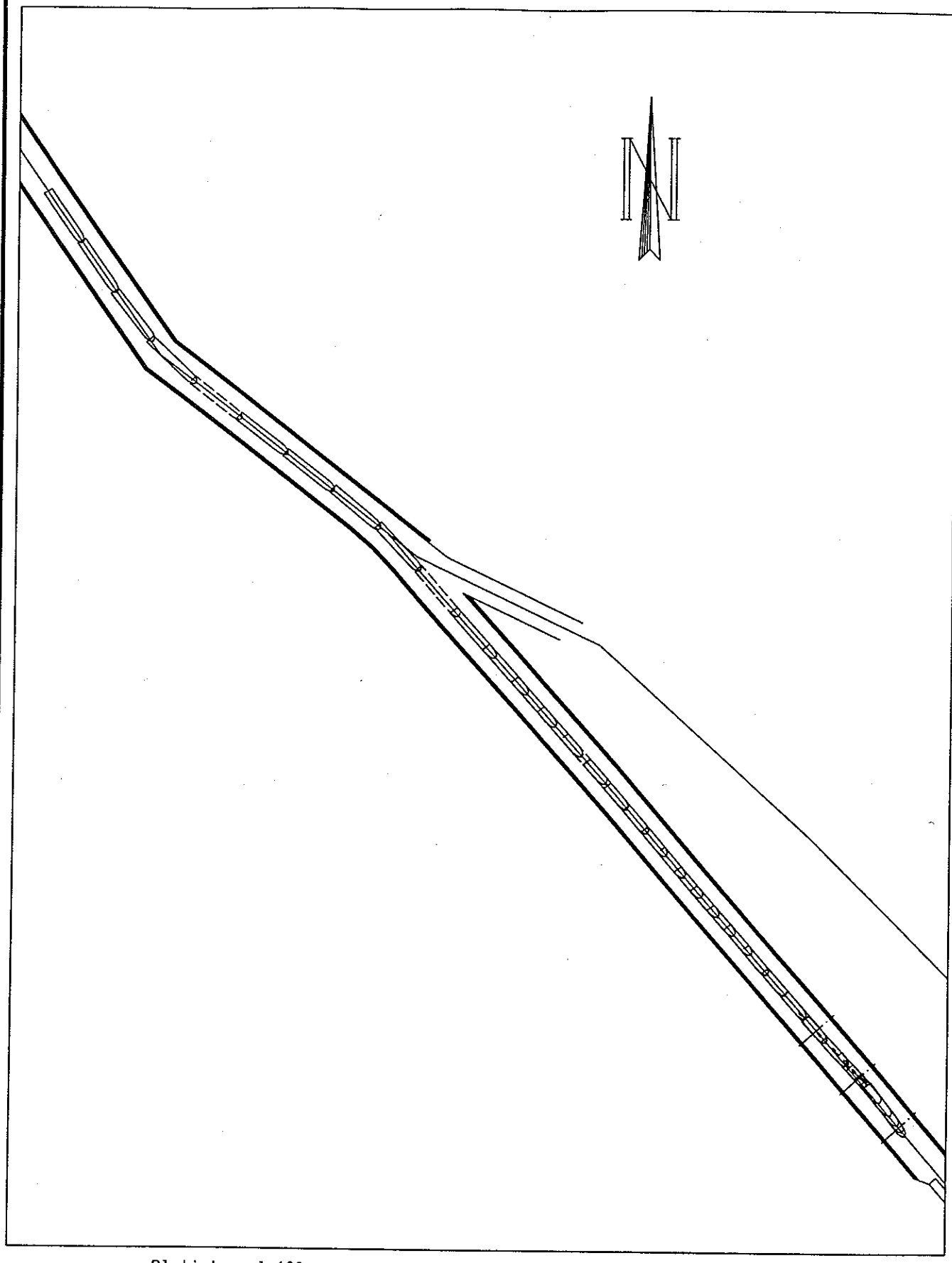
Fast-time: Channel alignment P1A Panama Canal
Sailing to lock from Lake Gatn .

Condition : 9
Run : 016
Wind : NNW 5 Bf
Tide : none

WATERBOUWKUNDIG LABORATORIUM
Borgerhout-Antwerpen

M673

A-34



Plotinterval 120. s

Scale 1:/33000.

Fast-time: Channel alignment P1A Panama Canal Sailing to lock from Lake Gatn .	Condition : 9 Run : 017 Wind : NW 5 Bf Tide : none

WATERBOUWKUNDIG LABORATORIUM Borgerhout-Antwerpen	M673	A-35

2 Geo-technical analysis

2.1 INTRODUCTION.

The geo-technical analysis of the site specific conditions has been reported under Task 2 "Design Criteria" – Chapter 4 of Part A "Geo-technical and seismic parameters". This report also contains the geo-technical sections along the P1A alignment, as well as the plan view showing the top of the different substrata which are of major importance to the lock construction.
The alignment P1A has been retained by ACP in the Terms of Reference of this Conceptual Design, and the purpose of task 3 is the optimization of this alignment in function of geo-technical circumstances and considerations, together with nautical conditions and excavation minimization requirements.

2.2 GEO-TECHNICAL ANALYSIS.

Reference is made to drawings D3-0-001 and 002, showing both alignments P1A and P1B on the geo-technical back ground of the lock site. It can easily be seen that the lock structure in alignment P1A is situated partly in the "La Boca" formation, and partly in the Basalt rock formation. Part A of the Design Criteria contains values of physical characteristics of both formations. It can be deducted that the strength and bearing capacity as well as deformation parameters of the Basalt rock are on the average 5 times larger than those of the "La Boca" formation. In order to avoid different behaviour of single elements in one lock structure, it seems reasonable to verify if the alignment P1A could be optimized in a way to have the lock structure entirely situated in the Basalt formation. This has been indicated clearly in the conclusions of the geo-technical analysis as follows :

"Construction of the new Post Panamax Locks requires sound and strong rock foundation that shall be mainly massive basalt within the area that was purposely selected between St.6+750 and St.8+000 of the proposed P1A alignment. Other components of the channel alignment shall be founded upon basalt, pyroclastics and sedimentary formations at a less extent. From the present assessment, it is suggested to optimise the site of the locks by a slight shifting of the currently proposed alignment towards southeast. Such a shifting should allow the locks to be almost entirely

founded in the local basalt mass (laccolithic rock mass). The heavy concrete and massive machinery installations necessary for operation of the locks should be founded on stable rock mass type, almost uniformly classified as RMTI, which undoubtedly corresponds to a very good rock mass foundation."

The suggested shifting of the alignment to the south-east is shown on drawings D3-0-002 and D3-0-004. From the geo-technical point of view, this alignment P1B is to be preferred above alignment P1A.

Finally it is recommended, whatever the chosen alignment may be, to undertake additional geo-technical and seismic investigation to mark out more exactly the extend of the different substrata, as well as the location of the faults which could possibly interfere with the lock site, as has been emphasized in the seismic report as follows:

- *"It is emphasised that the Pedro Miguel Fault seems to crosscut the Post-Panamax route near the planned lock site. According to the MCE associated to that fault, a maximum co-seismic displacement of 15-30cm cannot be disregarded although the degree of activity seems very low.*

<u>Recommendations</u> : in further stages of the design, it is highly recommended to ascertain (i) the actual presence of the Pedro-Miguel Fault on the site, and (ii) the degree of activity of this potential seismogenic source, throughout :

- *a "marine" seismic investigations along the Panama Canal between existing Miraflores and Pedro Miguel Locks (as it was performed north of Pedro Miguel locks)*
- *an on-shore seismic investigations along the Post Panamax Canal route on the expected southern extension of the Pedro Miguel fault.*
- *if the Post Panamax Locks have to be designed in the close surroundings of that fault, a detailed seismotectonic analysis of this fault will be required including superficial geophysics and paleosismological analysis throughout trenching if necessary."*
-

These additional tests and analyses have to be undertaken as soon as possible and immediately following on the conceptual design phase in order not to cause delay to the project planning.

3 Excavation Volumes

3.1 INTRODUCTION

Because of capacity reasons the need of a bypass canal on the Pacific side of the Panama Canal, bypassing the existing *Pedro Miguel* and *Miraflores* locks, has emerged.

Harza Engineering Company (2000) performed an evaluation of lock channel alignments investigating alignments both on the Pacific and the Atlantic entrances of the Panama Canal.

In this draft final report on the first lock configuration, the Pacific end bypass canal with a triple lift lock configuration on the west side of the currently existing *Pedro Miguel* and *Miraflores* locks is investigated.
Alignment *P1A* is currently the pursued option. This alignment is studied in detail using the available geotechnical information. A different alignment, called *P1B* is positioned on the east side of *P1A*, but still on the west side of the existing *Pedro Miguel* and *Miraflores* locks. This alternative alignment *P1B* is a more suitable alignment when the amount of canal excavation and the foundation conditions for the lock structures are considered. However from a nautical point of view alignment *P1B* needs to be rejected, due to the difficulty in accessing the bypass canal. Alignment *P1A* is shown on drawing D3-0-006, alignment *P1B* is shown on drawing D3-0-007.

The purpose of this report is to present the methodology and the results of the volume calculations for the excavation of the bypass canal (alignment *P1A*) on the Pacific side of the Panama Canal (third lane option only). An estimate of the cost of the excavation is also presented. Alignment *P1B* is examined in less detail.

3.2 DATA

3.2.1 SURVEY INFORMATION

All levels are referenced to *Precise Level Datum (PLD)*. All positions are indicated in *meter North American Datum 1927 (Universe Tranverse Mercator* projection).

The topography at the proposed location of the bypass canal is taken from the digital file *curvas-nivel.dwg*. The file *pachyd.txt* provides bathymetrical data at the location of the old excavation (of around 1940) and at the connection of the Pacific edge of the bypass canal with the existing canal (however not complete). The survey information of the file *cartagen.dwg*, containing the survey of the relatively recent excavation close to the *Pedro Miguel* locks, was incorporated. The (bathymetrical) survey information in certain low areas remains missing, as can be seen on drawing D3-0-008. The MX-software, used for the volume calculations, assumes an interpolated natural ground level in the areas where no survey information is available. The bathymetrical information at the location of the connection between the bypass canal and the Gatun side of the existing canal is missing and therefore the volume calculations exclude that volume.

3.2.2 CANAL/LOCK POSITION

The position of the alignments P1A and P1B is taken from drawing *D3-0-001 Lock Alignment P1A rev. 0* and *D3-0-002 Lock Alignment P1B rev. 0* respectively (third lane option). These drawings define the position of the canal and the lock with triple lift configuration (rolling gates, locomotives). The width of the canal is 220 m, 4 times the width of the design Post-Panamax vessel.

The canal bottom levels used in the calculations are listed in *Table 1*. The canal levels on both sides of the lock and close to them will be slightly lower (deeper) than the levels in *Table 1*, but this effect has been neglected in the calculations.

	Canal Level [*m PLD*]
Gatun-side of lock	+ 7.16
Pacific-side of lock	- 19.08

Table 1: bypass canal bottom elevations assumed for volume calculations

For *P1A* the following assumptions were made:

- Dimensions of the lock structures are based on the latest lock design as shown on drawing D4-A-003 (see drawing D3-0-006).
- The distance between the third lane axis and the eastern edge of the canal bottom is 99.5 m.

- The distance between the third lane axis and the western edge of the canal bottom is 120.5 m.

The excavation levels of the *P1A* lock chambers are indicated in *Table 2* (lock chamber 1 is the one on the Gatun side of the locks):

Location	Excavation Level [*m PLD*]
Chamber 1	- 4.4
Chamber 2	- 12.7
Chamber 3	- 22.1

Table 2: excavation levels of lock chambers for alignment P1A

For *P1B* the following assumptions were made:

- Dimensions of the lock structures are based on drawing D3-0-002rev0 (see drawing D3-0-007).
- The distance between the third lane axis and the eastern edge of the canal bottom is 107 m.
- The distance between the third lane axis and the western edge of the canal bottom is 113 m.

The excavation levels of the *P1B* lock chambers are indicated in *Table 3*:

Location	Excavation Level [*m PLD*]
Chamber 1	- 3.9
Chamber 2	- 12.2
Chamber 3	- 22.1

Table 3: excavation levels of lock chambers for alignment P1B

Both the lock structure dimensions and the canal positioning with regard to the third lane axis differ slightly (most recent information applied to the pursued *P1A* option).

3.2.3 GEOLOGICAL INFORMATION

The canal side slopes are determined based on the drawings:

- TERCER JUEGO DE ESCLUSAS, PACIFICO, ALINEAMIENTO P1, SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT, PERFIL 1 (ESTE), FEBRERO 2002 (filename: *SEC1 Y SEC1CR.dwg*).
- TERCER JUEGO DE ESCLUSAS, PACIFICO, ALINEAMIENTO P1, SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT, PERFIL 3 (OESTE), FEBRERO 2002 (filename: *SEC3 Y SEC3CR.dwg*).
- MAPA GEOLOGICO CON SERIE TP1, TP1C Y TP2 Y TYP2C + MIRAFLORES, JUNIO 5 DE 2002 (filename: *PERFILES 4 Y 5 SERIE P1, PM, Y M*.dwg).
- *CUT SLOPE TYPES.dwg*
- MAPA GEOLOGICO CON SERIE TP1, TP1C Y TP2, TP2C + MIRAFLORES, REVISADO POR CPP MARZO 2002, *D2-0-001.dwg*

On drawings D3-0-014 and D3-0-015 the topography as obtained on the eastern and western edge of the *P1A* canal bottom is superimposed on the geological profiles of drawing MAPA GEOLOGICO CON SERIE TP1, TP1C Y TP2 Y TYP2C + MIRAFLORES, JUNIO 5 DE 2002 (filename: *PERFILES 4 Y 5 SERIE P1, PM, Y M*.dwg). It can be seen that the natural ground level of the geological profile does not correspond with the topography in certain locations along the sections.

It appears there is a similar problem with the geological profiles at the proposed location of the *P1A* triple lift lock. The geological profiles referred to, are:

- PACIFICO, ALINEAMIENTO P1, SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT: PERFIL 1 (ESTE), ACP, FEBRERO 2002 (*sec1ysec1cr.dwg*).
- PACIFICO, ALINEAMIENTO P1, SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT: PERFIL 2 (CENTRO), ACP, FEBRERO 2002 (*sec2ysec2cr.dwg*).
- PACIFICO, ALINEAMIENTO P1, SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT: PERFIL 3 (OESTE), ACP, FEBRERO 2002 (*sec3ysec3cr.dwg*).

To determine the canal slopes the geological information of the above drawings is used. For the volume calculations the topography as described in section 3.2.1 is used.

Apart from the geological profiles mentioned above, the following digital geological data were also provided:

- xls-file *base de datos pozos M* containing the logs of M1-M439.
- xls-file *PM general data base* containing the logs of PM1-PM504.

On drawing D3-0-009 the location of the loggings M1-M439, PM1-PM504 is shown. It can be seen that for large parts of the *P1A* there is no digital geologic information available.

The folders *TP1C-01, …, TP1C-34, TP2C-01, …, TP2C-15* containing geological information, were also provided. The files have different extensions: *NDX, PDT, 000, 001, P* and they are unreadable without the appropriate *Logdraft* software.

3.3 SOFTWARE

The software used to calculate the excavation volume is MX (MOSS).

3.4 METHODOLOGY

3.4.1 GENERAL

MX is used to calculate the total volume to be excavated per alignment.

The results of the volume calculations are subdivided in sub-results for different parts along the canal (see drawings D3-0-006, D3-0-007):

- ch. 1000 – 6000.
- ch. 6000 – gate 1 (area 001).
- gate 1 – gate 2 (area 002).
- gate 2 – gate 3 (area 003).
- gate 3 – gate 4 (area 004).
- gate 4 - ch. 9000 (area 005).
- ch. 9000 – ch. 10000.

The volume of the saving basins for the triple lift lock are not incorporated in the calculation.

In case sufficient digital geological data would be available, the 3-D modelling of the different sub-volumes (for each of the strata) would be possible. Due to the lack of available digital loggings within the canal alignment it is not possible to make digital models of the surfaces separating the different layers.

The different geological sections available can be used to estimate the different sub-volumes. The advantage of using the longitudinal sections is that they comprise the full synthesis of the geological information of the area (not limited to merely borehole logs). Certainly when the density of the boreholes is relatively low, it is not without risk to use the interpolated surfaces between the boreholes. Using a geological longitudinal section over a certain width of the canal is an approximation, although it can be said that the information on the longitudinal section itself is already a kind of average as the different borings have not been done on single straight lines (scattered around the alignment of the longitudinal section).

3.4.2 SUB-VOLUMES

The available geological long-sections that are used for estimating the sub-volumes are:

- PACIFICO, ALINEAMIENTO P1, SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT: PERFIL 1 (ESTE), ACP, FEBRERO 2002 (*sec1ysec1cr.dwg*), providing a geological long-section between chainage (ch.) 6000 and 9000.

- PACIFICO, ALINEAMIENTO P1, SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT: PERFIL 2 (CENTRO), ACP, FEBRERO 2002 (*sec2ysec2cr.dwg*), providing a geological long-section between ch. 6000 and 9000.
- PACIFICO, ALINEAMIENTO P1, SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT: PERFIL 3 (OESTE), ACP, FEBRERO 2002 (*sec3ysec3cr.dwg*), providing a geological long-section between ch. 6000 and 9000.
- MAPA GEOLOGICO CON SERIE TP1, TP1C Y TP2 Y TYP2C + MIRAFLORES, JUNIO 5 DE 2002 (filename: *PERFILES 4 Y 5 SERIE P1, PM, Y M*.dwg), providing geological longitudinal sections between ch. 1000 and 6000 on the eastern and western edge of the *P1A* canal bottom.

The location of the 3 geological long-sections between ch. 6000 and 9000 is indicated on drawing D3-0-006.

For each of the geological long-sections the contribution of the different strata between the topography (as indicated on the geological long-sections) and the excavation profile is determined. On drawings D3-0-010, D3-0-011, D3-0-012 and D3-0-013 the excavation profile is indicated for the different geological long-sections. The uncertainty about the geological longitudinal sections (see chapter 3.4.2) will therefore result in an uncertainty about the divide of the total volume in the sub-volumes for the different strata.

The contribution of the different strata in the total volume (over the width of the canal) is determined according to the different geological longitudinal sections and their lateral positions.

3.4.3 CANAL SLOPES

From the outside geological longitudinal-sections the typical slope types can be deducted for both sides of the canal. Due to the variation in the lithography when moving in a direction perpendicular to the canal axis, this approach is an approximation.

In drawing *cutslopetypes.dwg* the stepped slope patterns for five different types of material are indicated (type A, B, C, D and E). On the longsections *sec1ysec1cr.dwg*, *sec3ysec3cr.dwg*, *perfiles4y5seriep1,pm,ym.dwg* the material types *A*, *B*, *C*, *D* and *E* are indicated. The stepped slope as indicated on *cutslopetypes.dwg* is replaced by a smooth slope yielding approximately the same overall slope as indicated on drawing *cutslopetypes.dwg*. In *Table 4* the slopes used in the volume calculations for the five types of material are indicated.

Material Type	Slope
very hard rock (type A)	45°
hard rock (type B)	40°
medium hard to medium soft rock (type C)	22°
soft rock to medium hard soil (type D)	15°
overburden (type E)	15°

Table 4: canal slopes per material type

For alignment *P1A* the western sound rock slopes of the canal are based on the longitudinal sections *perfiles4y5seriep1,pm,ym.dwg* (west side) for the lower chainages and on *sec3ysec3cr.dwg* for the higher chainages. In this way the following slopes are obtained for the western side of alignment *P1A*, underneath the overburden and weathered rock layers (see *Table 5*):

Chainage	Type of Sound Rock	Sound Rock Slope
1000 – 1755	D	15°
1755 – 2220	B	40°
2220 – 2970	D	15°
2970 – 3325	B	40°
3325 – 3650	D	15°
3650 – 6815	B	40°
6815 – 7585	C	22°
7585 – 8245	A	45°
8245 – 8610	C	22°
8610 – 10300	A	45°

Table 5: sound rock slopes for west side of P1A

For alignment *P1A* the eastern sound rock slopes of the canal are based on the longitudinal sections *perfiles4y5seriep1,pm,ym.dwg* (east side) for the lower chainages and on *sec1ysec1cr.dwg* for the higher chainages. In this way the following slopes are obtained for the eastern side of alignment *P1A*, underneath the overburden and weathered rock layers (see *Table 6*):

Chainage	Type of Sound Rock	Sound Rock Slope
1000 – 2060	D	15°
2060 – 3310	B	40°
3310 – 3775	C	22°
3775 – 7180	B	40°
7180 – 8300	A	45°
8300 – 10300	C	22°

Table 6: sound rock slopes for east side of P1A and for west side of P1B

For both the eastern and the western side slope of alignment *P1B* the sound rock slopes at the different chainages are based on the longitudinal sections *perfiles4y5seriep1,pm,ym.dwg* (east side) for the lower chainages and on *sec1ysec1cr.dwg* for the higher chainages. This is a rough approximation. For the eastern side the larger extent of the magmatic diabase/basalt formation (as can be seen on drawing *D2-0-001*) is taken into account. For the west side of *P1B* the same slopes are obtained as for the east side of *P1A* (see *Table 6*). For the east side of *P1B* the canal slopes are shown in *Table 7*.

Chainage	Type of Sound Rock	Sound Rock Slope
1000 – 2060	D	15°
2060 – 3310	B	40°
3310 – 3775	C	22°
3375 – 7000	B	40°
7000 – 9000	A	45°

Table 7: sound rock slopes for east side of P1B

On drawing *sec1ysec1cr.dwg* (east bank) it can be seen that from ch. 8500 till 8700 approximately a basalt intrusion is underlying the *La Boca* formation. For the determination of the canal slopes this basalt intrusion is neglected (type C slope is assumed). On drawing *perfiles4y5seriep1,pm,ym.dwg* it can be seen that for the east side between ch. 2200 and 2800 the *Cucaracha* formation is underlying the thick above layers. For the determination of the canal slopes the *Cucaracha* formation is neglected and a type B slope, corresponding with the above *Pedro Miguel* and magmatic layers, is assumed.

Because of the variation in lithology and discontinuities, the typical cut slopes that were used should be refined by detailed local rock mass stability assessments at the early stage of the excavation works. In this respect, detailed wedge stability calculations should be performed. The detailed design of the final slopes/rock support will then have to be done.

According to the different geological longitudinal sections the average thickness of the combined overburden and weathered rock layers is approximately 12 m (average overburden thickness 8 m, average weathered rock thickness 4 m). It is considerably smaller at certain locations and considerably larger at other locations. In view of the uncertainty about the geological longitudinal sections (see section 3.2.3), a uniform vertical thickness of 12 m is assigned to the combined overburden and weathered rock layer for the calculation of the total excavation volume. A slope angle of 15° can be assumed for this combined layer. On drawing D3-0-006 the sound rock slopes along the canal are drawn for *P1A*. An additional volume has to be added to the calculation results with the sound rock slopes to incorporate the milder slope at the top of the embankment.

In the volume calculation for *P1A* a slope of 1H:2V is assumed next to the lock wall.

In the volume calculation for *P1B* a vertical excavation is assumed next to the lock walls as high as the estimated lock wall (see drawing D3-0-007). Above that level the slopes of *Tables 6* and *7* are assumed.

3.5 RESULTS

3.5.1 P1A

For alignment *P1A* the following total excavation (sub-)volumes are obtained (see *Table 8)*:

Sub-area	Total excavation volume [m^3]
1000 – 3000	25 763 000
3000 – 6000	35 898 000
6000 – G1 (001)	2 018 000
G1 – G2 (002)	1 210 000
G2 – G3 (003)	1 921 000
G3 – G4 (004)	2 266 000
G4 – 9000 (005)	3 649 000
9000 – 10000	4 558 000
TOTAL	**77 283 000**

Table 8: P1A total excavation volumes

For sub-area *1000-6000* the following divide into the different materials is obtained (assigning an equal weight to both geological longitudinal sections of D3-0-010), see *Table 9*:

Sub-area *1000-6000*	Excavation volume [m^3]
overburden	9 810 000
weathered rock	7 742 000
basalt/diabase	12 939 000
form. Pedro Miguel	25 289 000
form. Cucaracha	5 881 000
TOTAL	**61 661 000**

Table 9: P1A excavation volumes for sub-area 1000-6000

For sub-area *6000-G1 (001)* the following divide into the different materials is obtained (assigning approximately twice the weight to D3-0-011("este") compared to D3-0-012("centro")), see *Table 10*:

Sub-area *6000-G1 (001)*	Excavation volume [m^3]
overburden	1 257 000
weathered rock	228 000
basalt/diabase	533 000
TOTAL	**2 018 000**

Table 10: P1A excavation volumes for sub-area 6000-G1 (001)

For sub-area *G1-G2 (002)* the following divide into the different materials is obtained (assigning equal weight to D3-0-011("este") and D3-0-012("centro")), see *Table 11*:

Sub-area *G1-G2 (002)*	**Excavation volume [m^3]**
overburden	811 000
weathered rock	157 000
basalt/diabase	206 000
form. La Boca	36 000
TOTAL	**1 210 000**

Table 11: P1A excavation volumes for sub-area G1-G2 (002)

For sub-area *G2-G3 (003)* the following divide into the different materials is obtained (assigning equal weight to D3-0-011("este") and D3-0-012("centro")), see *Table 12*:

Sub-area *G2-G3 (003)*	**Excavation volume [m^3]**
overburden	346 000
weathered rock	154 000
basalt/diabase	1 114 000
form. La Boca	307 000
TOTAL	**1 921 000**

Table 12: P1A excavation volumes for sub-area G2-G3 (003)

For sub-area *G3-G4 (004)* the following division into the different materials is obtained (assigning equal weight to D3-0-011("este") and D3-0-012("centro")), see *Table 13*:

Sub-area *G3-G4 (004)*	**Excavation volume [m^3]**
overburden	340 000
weathered rock	113 000
basalt/diabase	1 813 000
TOTAL	**2 266 000**

Table 13: P1A excavation volumes for sub-area G3-G4 (004)

For sub-area *G4-9000 (005)* the following divide into the different materials is obtained (assigning 1.5 times the weight of D3-0-012("centro") to D3-0-011("este")), see *Table 14*:

Sub-area *G4-9000 (005)*	**Excavation volume [m^3]**
overburden	1 496 000
weathered rock	438 000
basalt/diabase	511 000
form. La Boca	1 204 000
TOTAL	**3 649 000**

Table 14: P1A excavation volumes for sub-area G4-9000 (005)

For sub-area *9000-10000* the following divide into the different materials is obtained (assigning a weight of 0.48 to D3-0-011("este"), a weight of 0.42 to D3-0-012("centro"), a weight of 0.1 to D3-0-013("oeste"), see *Table 15*:

Sub-area *9000-10000*	**Excavation volume [m^3]**
overburden	2 051 000
weathered rock	729 000
basalt/diabase	410 000
form. La Boca	1 368 000
TOTAL	**4 558 000**

Table 15: P1A excavation volumes for sub-area 9000-10000

The breakdown of the total excavation volume for *P1A* obtained, is as follows (*Table 16*):

Material	**Total excavation volume [m^3]**
overburden	16 111 000
weathered rock	9 561 000
basalt/diabase	17 526 000
form. Pedro Miguel	25 289 000
form. La Boca	2 915 000
form. Cucaracha	5 881 000
TOTAL	**77 283 000**

Table 16: material breakdown for P1A excavation

3.5.2 P1B

For alignment *P1B* the following total excavation (sub-)volumes are obtained (see *Table 17*):

Sub-area	**Total excavation volume [m^3]**
1000 – 3000	23 053 000
3000 - 6000	30 041 000
6000 – G1 (001)	3 435 000
G1 – G2 (002)	997 000
G2 – G3 (003)	495 000
G3 – G4 (004)	1 122 000
G4 – 9000 (005)	1 161 000
9000 – 10000	1 968 000
TOTAL	**62 272 000**

Table 17: P1B total excavation volumes

Using the geological longitudinal sections on the eastern side of P1A, the proportion of the different materials to the total excavation volume can be estimated at the numbers indicated in *Table 18*:

Material	Total excavation volume [m^3]
overburden	13 519 000
weathered rock	8 332 000
basalt/diabase	13 506 000
form. Pedro Miguel	22 822 000
form. La Boca	1 011 000
form. Cucaracha	3 082 000
TOTAL	**62 272 000**

Table 18: material breakdown for P1B excavation

3.6 COST ESTIMATE

The cost of the excavation is closely linked with the method of excavation. The following rates can be used (ACP-origin):

- dry excavation of overburden material: 4 $/m^3.
- dry excavation of weathered rock: 5 $/m^3.
- dry excavation (incl. blasting) of rock: 8 $/m^3.
- dredging of overburden material: 6 $/m^3.
- dredging of weathered rock: 24 $/m^3.
- dredging (incl. blasting) of rock: 28 $/m^3.

The rates of the different excavation methods can vary substantially depending on market conditions, degrees of efficiency, economies of scale, etc.

The minimum amount of material to be excavated in the wet (*P1A)* can be identified from chainage 8800 up to the connection with the existing canal (Pacific end of the bypass canal) and a small edge of material (to form a cofferdam) at the Gatun side connection of the bypass canal with the existing canal. As mentioned above, the bathymetrical part of the connection between the bypass canal and the Gatun side of the existing canal is not included in the calculations due to the missing bathymetric data. The following minimal quantities to be excavated in the wet can be roughly estimated:

- overburden: 2 534 000 m^3.
- weathered rock: 958 000 m^3.
- sound rock: 2 918 000 m^3.

Provision needs to be made for pumping capacity to be able to work in the dry in the designated areas.

Therefore the following total cost estimate can be made:

- overburden, wet: 2 534 000 m^3 * 6 $/$m^3$ = 15 204 000 $
- weathered rock, wet: 958 000 m^3 * 24 $/$m^3$ = 22 992 000 $
- sound rock, wet: 2 918 000 m^3 * 28 $/$m^3$ = 81 704 000 $
- overburden, dry: 13 577 000 m^3 * 4 $/$m^3$ = 54 308 000 $
- weathered rock, dry: 8 603 000 m^3 * 5 $/$m^3$ = 43 015 000 $
- sound rock, dry: 48 693 000 m^3 * 8 $/$m^3$ = 389 544 000 $

Total: 606 767 000 $

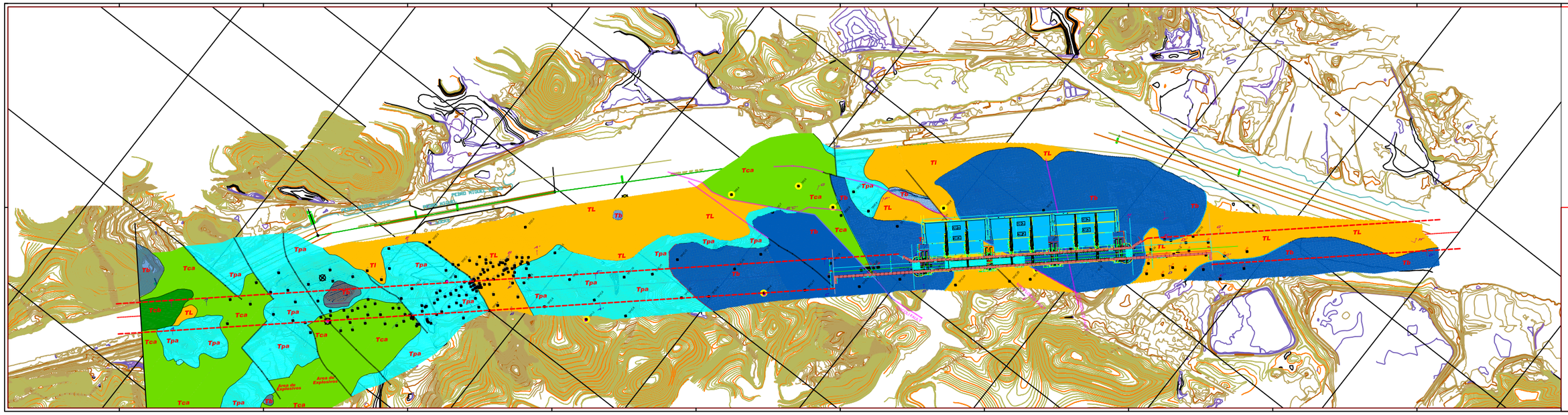
In order to avoid loosing large amounts of water from the Gatun side of the bypass canal to the existing canal, certain relative low areas need to be filled. On drawings D3-0-018 (*P1*) and D3-0-019 (*P2*) the areas where dikes need to be constructed are indicated. A volume estimate of the required material for filling is made for alignments P1A and P1B:

- *P1A*: 440 000 m^3.
- *P1B*: 1 000 000 m^3.

DRAWING LIST
TASK 3

2/22/2006

x	Rev.	Title
D3- 0 -001	C	Lock Alignment P1A : geotechnical situation
D3- 0 -002	B	Lock Alignment P1B : geotechnical situation
D3- 0 -003	C	Lock Alignment P1A : nautical design
D3- 0 -004	B	Lock Alignment P1B : nautical design
D3- 0 -005	A	Diversion rio cocoli
D3- 0 -006	A	Layout, Panama canal, alignment P1A
D3- 0 -007	B	Layout, Panama canal, alignment P1B
D3- 0 -008	A	Layout, Panama canal P1A, survey data
D3- 0 -009	A	Layout, Panama canal P1A, location of digital geological information
D3- 0 -010	A	geological longitudinal section + excavation profile, east side P1A, ch. 1000-6000 (sheet 1/2)
	A	geological longitudinal section + excavation profile, west side P1A, ch. 1000-6000 (sheet 2/2)
D3- 0 -011	A	geological longitudinal section + excavation profile, east side P1A, ch. 6000-9000
D3- 0 -012	A	geological longitudinal section + excavation profile, center P1A, ch. 6000-9000
D3- 0 -013	A	geological longitudinal section + excavation profile, west side P1A, ch. 6000-9000
D3- 0 -014	A	longitudinal section, Panama canal - alignment P1A, east side, comparison topography - geological section
D3- 0 -015	A	longitudinal section, Panama canal - alignment P1A, west side, comparison topography - geological section
D3- 0 -016	A	cross sections, Panama canal P1A, chainages : 1000-6000 & 9000-12000
D3- 0 -017	A	cross sections, Panama canal P1A, chainages 6000-9000
D3- 0 -018	A	layout Panama Canal P1A indication of 27-30m topographic levels
D3- 0 -019	A	layout Panama Canal P1B indication of 27-30m topographic levels



Símbologia Formación

Tb	Basaltos o Diabasa
Tpa	Formación Panamá
Tl	Lutitas
Tca	Formación Caimito
Tcb	Conglomerado

Pozos A.C.P.

Pozos Cuatro(T.Lotes

Secciones Geológicas

Contactos Geológicos

Fallas Geológicas

Extra(T.Acceso)

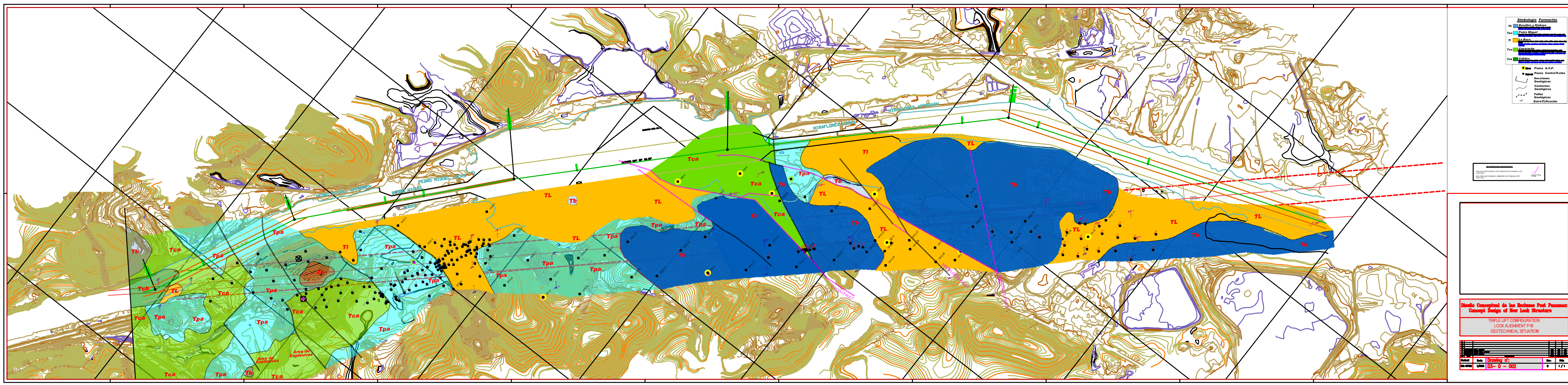
Acceso/Ubicación/Mina

Coordenadas UTM (Zona 17 N) Datum WGS84 (m)
Coordenadas Geográficas Datum WGS84 (DMS)

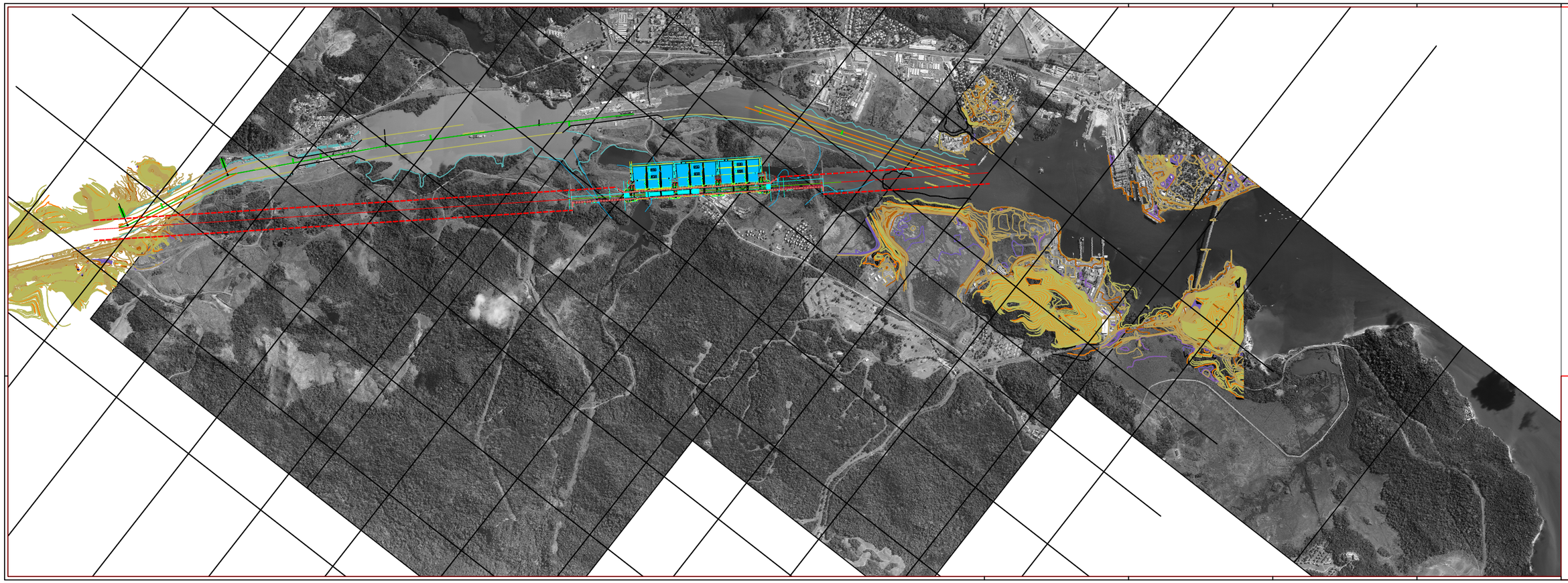
Nuevo Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
LOCK ALIGNMENT P1A/ GEOTECHNICAL SITUATION

Drawn			
Checked			
Approved			
Scale	Drawing N°:	Rev	Sheet
NO-SCALE	USBR	DS- 0 – 001	0	1 / 1



Simbología Formación
Tb Basaltos y Diabasa
Tpa Pedro Miguel
Tl La Boca
Tca Cucaracha
Tcb Culebra
Pozos A.C.P.
Pozos ContraTLstas
Secciones Geológicas
Contactos Geológicos
Fallas Geológicas
EstraTLficación
PEDRO MIGUEL APPROACH
PEDRO MIGUEL LOCKS
MIRAFLORES LOCKS
MIRAFLORES APPROACH
PACIFIC ENTRANCE
COCOLI FAULT
MIRAFLORES FAULT
Area de Explosivos
Tb
Tpa
TL
Tl
Tca
Tcb
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure
TRIPLE LIFT CONFIGURATION
LOCK ALIGNMENT P1B
GEOTECHNICAL SITUATION
Drawing nº:
D3- 0 - 002



Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
ALIGNMENT PS4 : NAUTICAL ACCESS

Revised	Date	Drawing N°:	Rev.	Rds.
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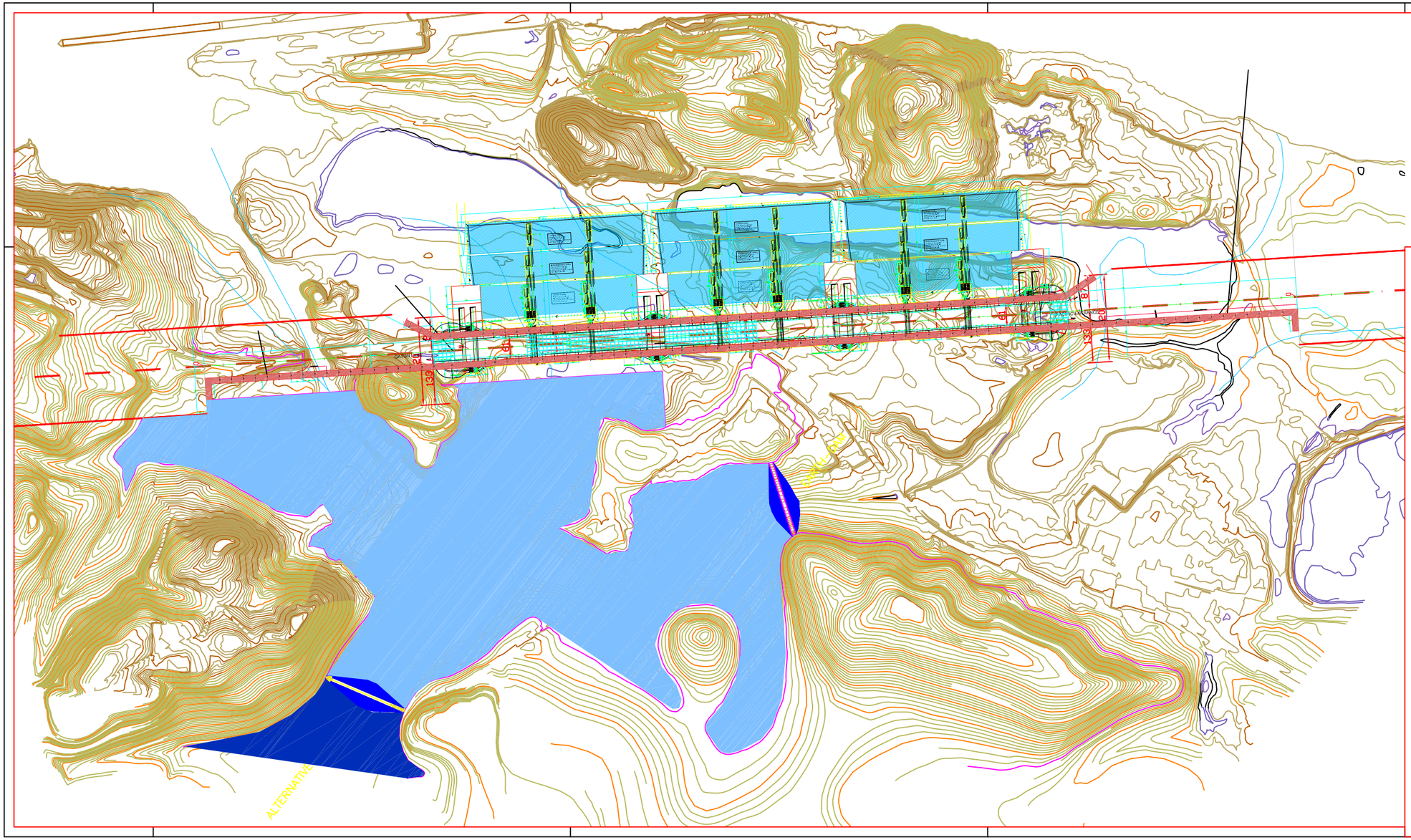
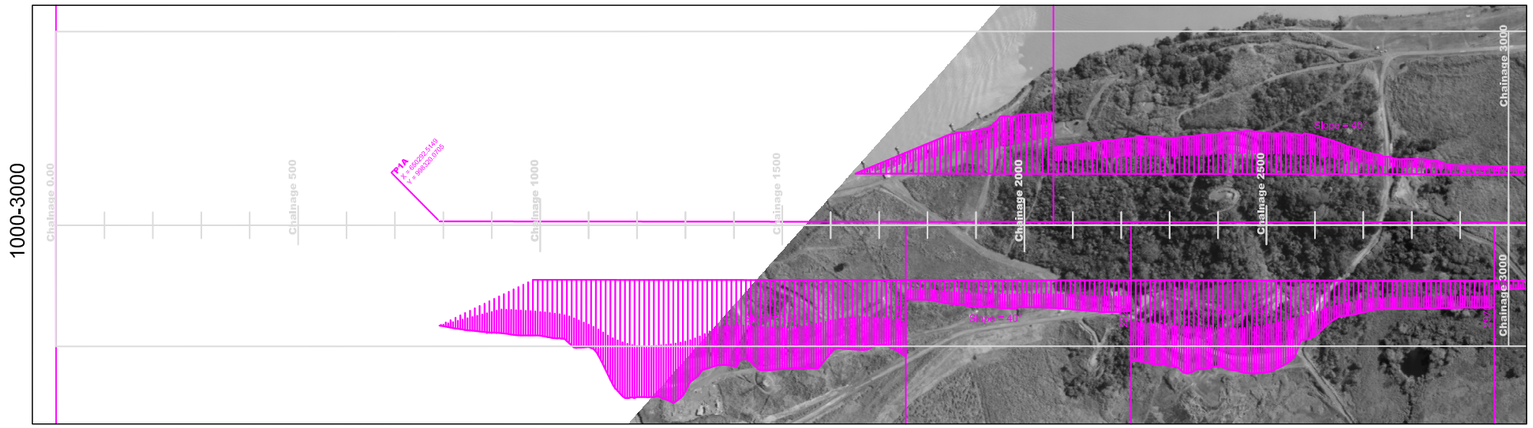
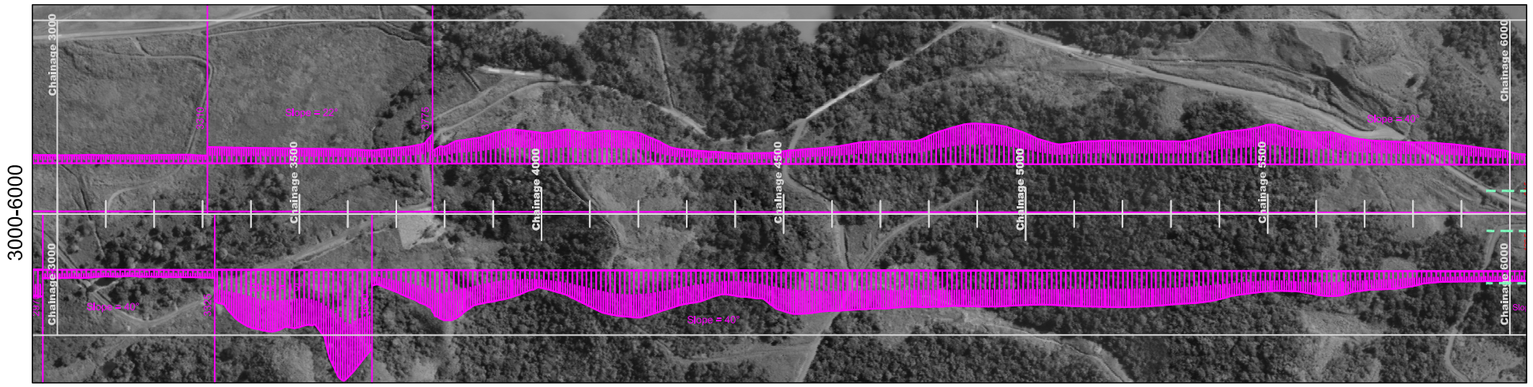


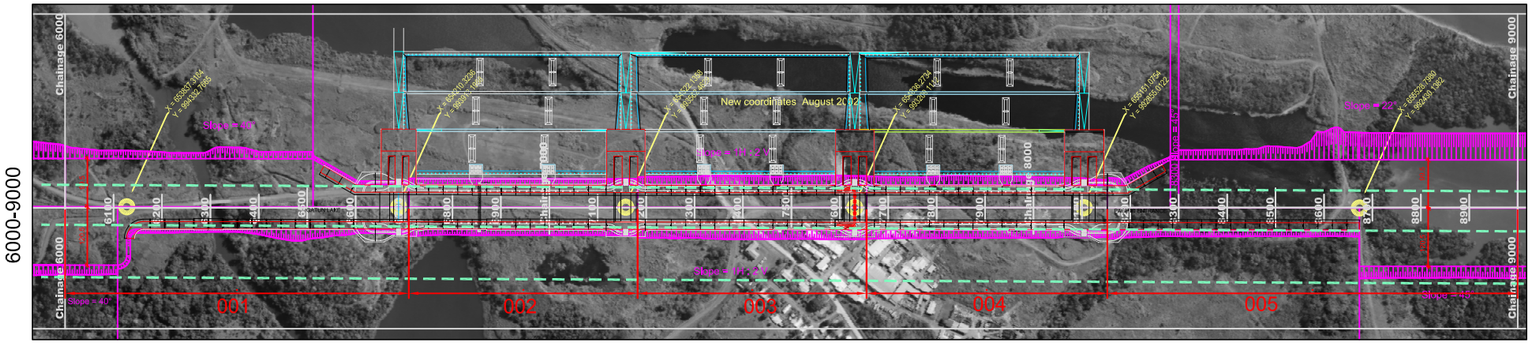
DRAWING NOT PLOTTED ON SCALE

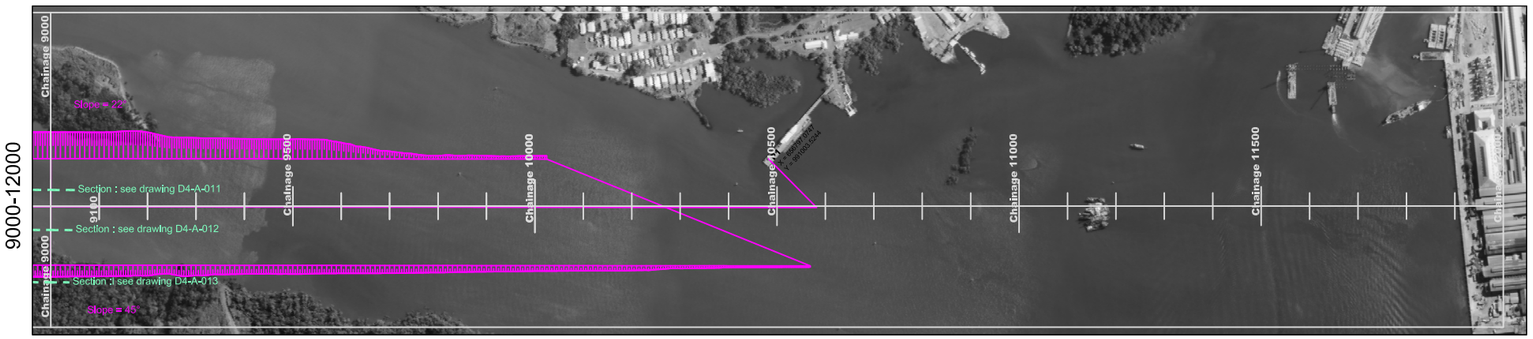
REV	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
F					
E					
D					
C					
B					
A	12/11/02	FINAL REPORT	BME	JDR	JDR
–	15/08/02	FIRST EDITION	BME	JDR	JDR

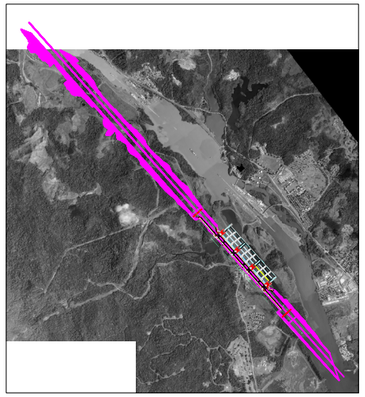
Contract	Scale:		Rev.:	Folio
SAA-97462	1/5000		A	1 / 1











CPP
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Avenue Ariane 7 B-1200 Brussels
In association with:
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IMDC

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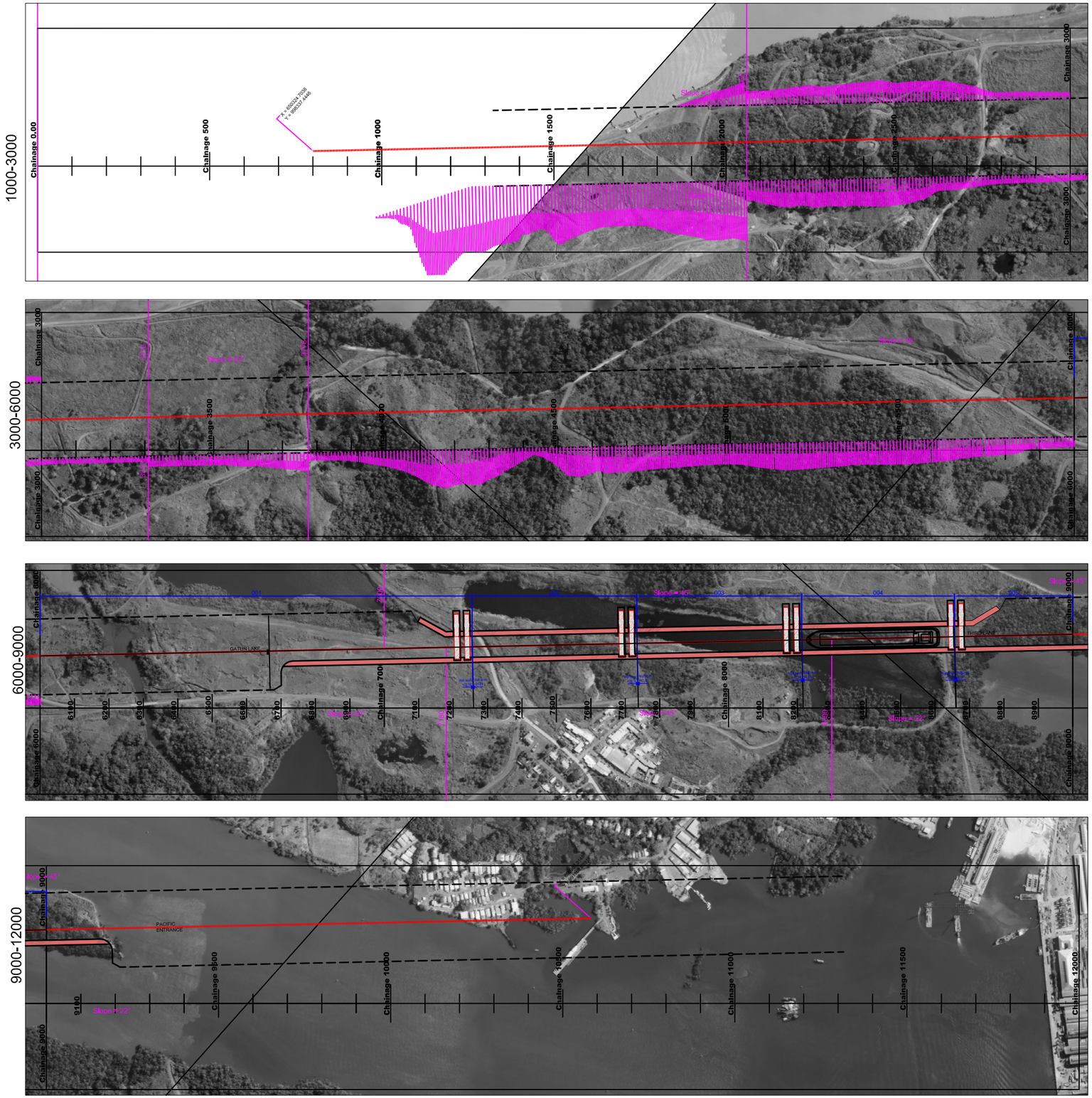
Autoridad del Canal de Panamá
Balboa
REPUBLICA DE PANAMA

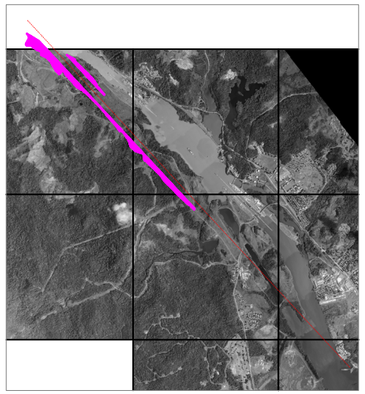
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
LAYOUT PANAMA CANAL
ALIGNMENT P1A

REV.	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
F					
E					
D					
C					
B					
A	12/11/02	FINAL REPORT	ERE	JGR	JGR
–	28/06/02	FIRST EDITION	ACD	FMA	JGR

Contract	Scale:	Drawing n°:	Rev.:	Folio
SAA-97462	1/4000	D3 – 0 – 006	A	1 / 1





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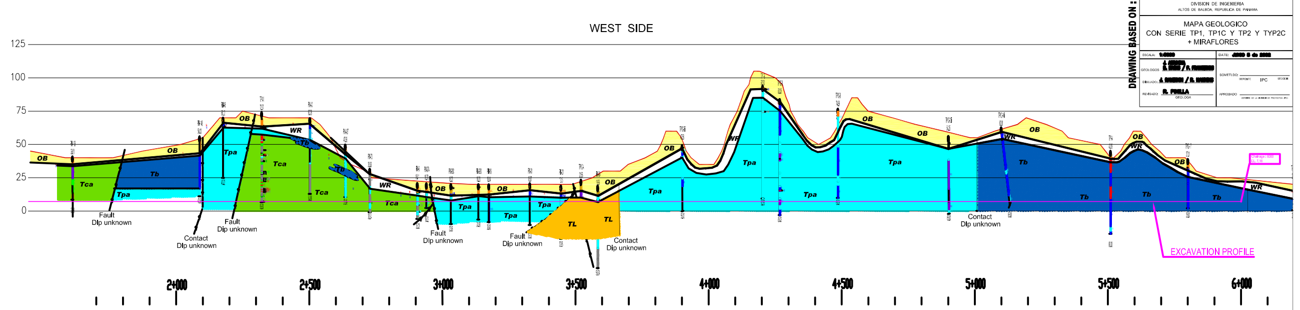
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
LAYOUT PANAMA CANAL
ALIGNMENT P.1B

Contract	Scale	Drawing n°:	Rev.:	Folio
SAA-97462	1/4000	D3 – 0 – 007	B	1 / 1

WEST SIDE



Overburden (OB)	Type E		Type E		Type E	Type E	
Weathered Rock (WR)	Type D		Type D		Type D	Type D	
Sound Rock (SR)	Type D	Type B	Type D	Type B	Type D	Type B	Type B

CUT SLOPE TYPE

DRAWING BASED ON :

AUTORIDAD DEL CANAL DE PANAMA
DEPARTAMENTO DE INGENIERIA Y SERVICIOS INDUSTRIALES
DIVISION DE INGENIERIA
ALTOS DE BALBOA, REPUBLICA DE PANAMA

MAPA GEOLOGICO
CON SERIE TP1, TP1C Y TP2 Y TYP2C
+ MIRAFLORES

ESCALA: 1:5000 | DATE: JUNIO 5 de 2002
GEOLOGOS:
DIBUJADO:
REVISADO: R. PINILLA — GEOLOGA
SOMETIDO: IPC
APROBADO:

Simbología *Formación*
- Tb — Basaltos y Diabasa
- Tpa — Pedro Miguel
- TL — La Boca
- Tca — Cucaracha
- Tcb — Culebra
- OB — OVERBURDEN
- WR — WEATHERED ROCK

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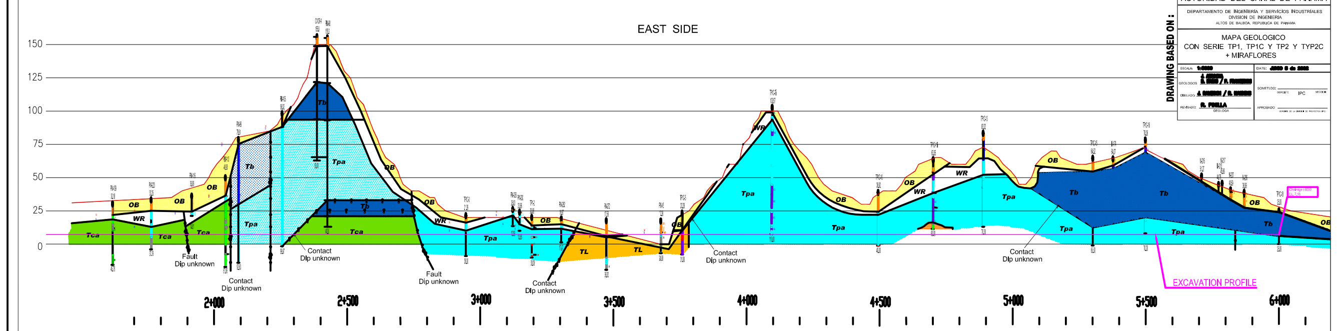
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
GEOLOGICAL LONGITUDINAL SECTION
+ EXCAVATION PROFILE
WEST SIDE P1A, CHAINAGE 1000-6000 Sht. 2 of 2

Contract	Scale	Drawing n°:	Rev.	Sht.
	1/-	D3 – 0 – 010	A	2/2

EAST SIDE



Overburden (OB)	Type E				Type E		Type E	Type E
Weathered Rock (WR)	Type D				Type D		Type D	Type D
Sound Rock (SR)	Type D	Type B	Type B / Type D	Type B	Type B	Type C	Type B	Type B

CUT SLOPE TYPE

DRAWING BASED ON :

AUTORIDAD DEL CANAL DE PANAMA
DEPARTAMENTO DE INGENIERIA Y SERVICIOS INDUSTRIALES
DIVISION DE INGENIERIA
ALTOS DE BALBOA, REPUBLICA DE PANAMA

MAPA GEOLOGICO
CON SERIE TP1, TP1C Y TP2 Y TYP2C
+ MIRAFLORES

ESCALA: 1:5000 | DATE: JUNIO 5 de 2002
GEOLOGOS:
DIBUJADO:
REVISADO: R. PINILLA — GEOLOGA
SOMETIDO: IPC
APROBADO:

Simbología *Formación*
- Tb — Basaltos y Diabasa
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- TL — La Boca
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- Tcb — Culebra
- OB — OVERBURDEN
- WR — WEATHERED ROCK

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TECHNUM
COYNE ET BELLIER
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Balboa
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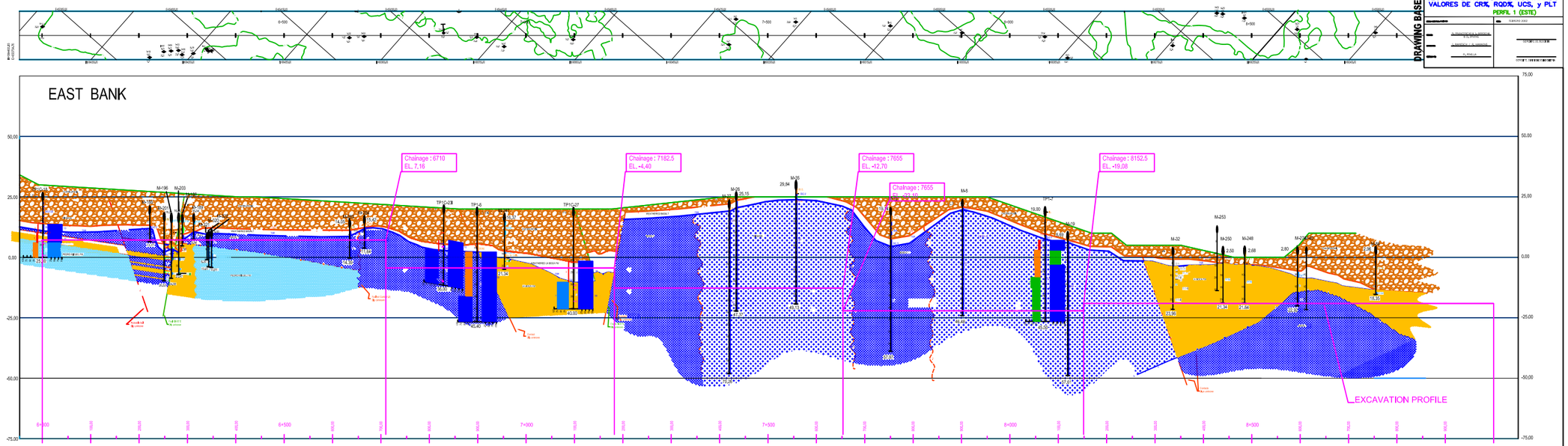
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
GEOLOGICAL LONGITUDINAL SECTION
+ EXCAVATION PROFILE
EAST SIDE P1A, CHAINAGE 1000-6000 Sht. 1 of 2

Contract	Scale	Drawing n°:	Rev.	Sht.
	1/-	D3 – 0 – 010	A	1/2



SOIL	TYPE E	TYPE E	TYPE E	TYPE E	TYPE E	TYPE E
WR	TYPE D	TYPE D	TYPE D	TYPE D	TYPE D	TYPE D
SR	TYPE B	TYPE B	TYPE B	TYPE A	TYPE A	TYPE C / TYPE A
RMT						

CUT SLOPES TYPES

DRAWING BASED ON :

TERCER JUEGO DE ESCLUSAS

PACIFICO — ALINEAMIENTO P1

SECCION LONGITUDINAL MOSTRANDO VALORES DE CRR, RQD%, UCS, y PLT

PERFIL 1 (ESTE)

FEBRUARY 2002

ALIGNEMENT DE LA ESCLUSA P.L. (PROP.)

ALIGNEMENT DE LA ESCLUSA P.L. (REAL)

ALIGNEMENT DE LA ESCLUSA P.L. (REAL) (2)

COORD. UTM NAD 27

COORD. UTM NAD 27

BASALTO/BASALT

DIABASA/DIABASE

SUELO RESIDUAL/OVERBURDEN

FORMACION LA ROCA/ LA ROCA FORMATION

FORMACION PEDRO MIGUEL/ PEDRO MIGUEL FORMATION

TWR INICIO DE ROCA METEORIZADA/TOP OF WEATHERED ROCK

TSR INICIO DE ROCA SANA/TOP OF SOUND ROCK

PISO DE ESCLUSAS PROPUESTO/PROPOSED LOCKS FLOOR

NAB NOMBRE DEL POZO/NAME OF BORING

ELEV. ELEVACION/ELEVATION

NLB PROF. DEL POZO/LENGTH OF BORING

CONTACTO/CONTACT

FALLA/FAULT

FALLA POSIBLE/POSSIBLE FAULT

PERCENTAGE SCALE

0 - 25%

25 - 50%

50 - 75%

75 - 90%

90 - 100%

RQD%

CRR

UCS (MPa)

PLT - UCS (MPa)

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División de Proyectos de Capacidad del Canal
Autoridad del Canal de Panamá
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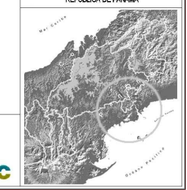
REPUBLICA DE PANAMA

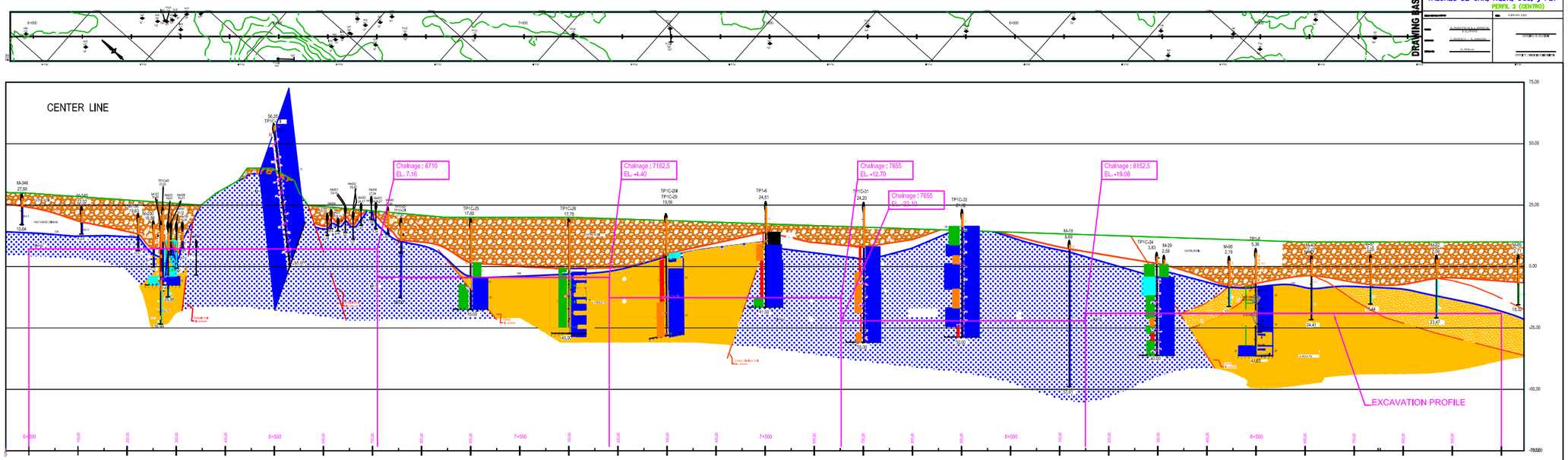


Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
GEOLOGICAL LONGITUDINAL SECTION
+ EXCAVATION PROFILE
EAST SIDE P1A, CHAINAGE 6000-9000

REV.	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
A	15/11/2002	FINAL REPORT	JMR	JRB	JMR
-	15/10/2002	FINAL DRAFT	JMR	JRB	JMR

Contract	Scale:	Drawing n°:	Rev.:	Folio
SAA-97462	1/3	D3 - 0 - 011	A	1 / 1



SOIL	TYPE E	TYPE E	TYPE E	TYPE E	TYPE E	TYPE E
WR	TYPE D		TYPE D	TYPE D	TYPE D	TYPE D
SR	TYPE B	TYPE B	TYPE C	TYPE A	TYPE A / TYPE C — TYPE C	TYPE C
RMT						

CUT SLOPES TYPES

DRAWING BASED ON :

TERCER JUEGO DE ESCLUSAS

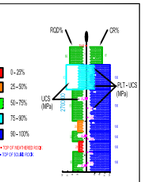
PACIFICO — ALINEAMIENTO P1

SECCION LONGITUDINAL MOSTRANDO VALORES DE CRR, RQD%, UCS, y PLT

PERFIL 2 (CENTRO)

FEBRUARY 2002

BASALTO/BASALT
DIABASA/DIABASE
SUELO RESIDUAL/OVERBURDEN
FORMACION LA BOCA/ LA BOCA FORMATION
FORMACION PEDRO MIGUEL/ PEDRO MIGUEL FORMATION
TWR PISO DE ROCA METEORIZADA/TOP OF WEATHERED ROCK
TSR PISO DE ROCA SANA/TOP OF SOUND ROCK
PISO DE ESCLUSAS PROPUESTO/PROPOSED LOCKS FLOOR
MAB NOMBRE DEL POZO/NAME OF BORING
2.00 ELEVACION/ELEVATION
10.0 PROF. DEL POZO/LENGTH OF BORING
CONTACTO/CONTACT
FALLA/FAULT
FALLA PROBABLE/PROBABLE FAULT
0 – 20%
25 – 50%
50 – 75%
75 – 100%
90 – 100%
TWR = TOP OF WEATHERED ROCK
TSR = TOP OF SOUND ROCK



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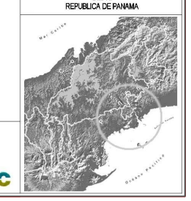
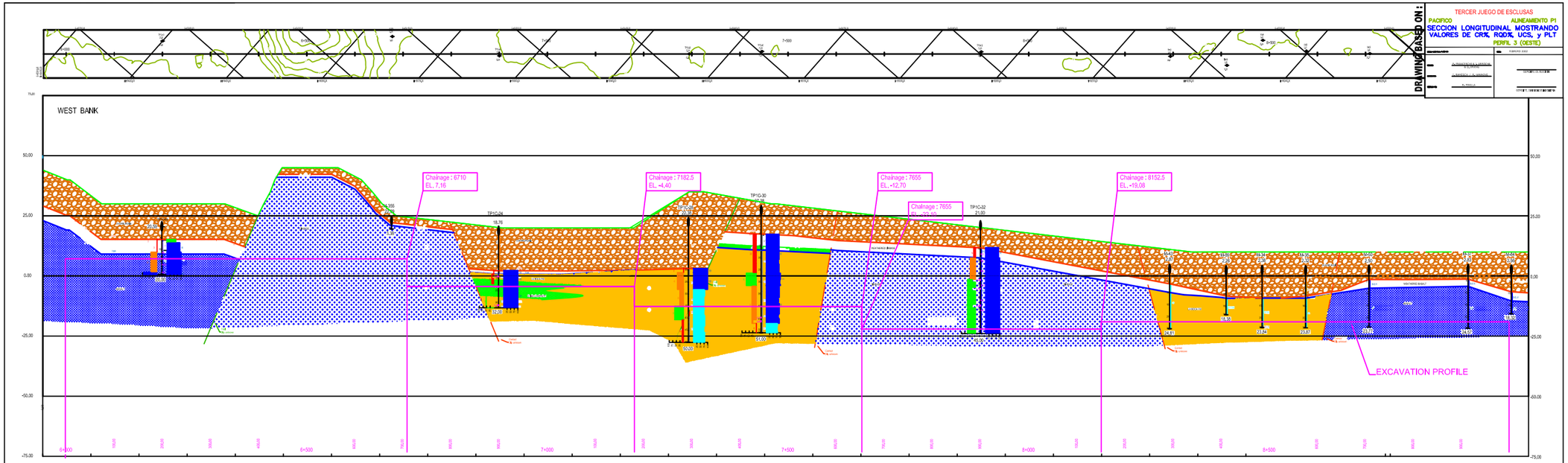


Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
GEOLOGICAL LONGITUDINAL SECTION
+ EXCAVATION PROFILE
CENTER P1A, CHAINAGE 6000-9000

REV	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
A	12/11/2001	FINAL REPORT	JGE	JGE	JGE
–	08/06/2001	FINAL REPORT	JGE	FCL	JGE

Contract	Scale:	Drawing n°:	Rev.:	Folio
SAA–97462	1/3	D3 – 0 – 012	A	1 / 1



OB	TYPE E	TYPE E	TYPE E	TYPE E	TYPE E	TYPE E	TYPE E
WR	TYPE D	TYPE D	TYPE D	TYPE D	TYPE D	TYPE D	TYPE D
SR	TYPE B	TYPE B	TYPE C	TYPE C	TYPE A	TYPE C	TYPE A

CUT SLOPES TYPES

TERCER JUEGO DE ESCLUSAS

PACIFICO — ALINEAMIENTO P1

SECCION LONGITUDINAL MOSTRANDO VALORES DE CR%, RQD%, UCS, y PLT

PERFIL 3 (OESTE)

DRAWING BASED ON :

BASALTO/BASALT
DIABASA/DIABASE
SUELO RESIDUAL/OVERBURDEN
FORMACION LA BOCA/ LA BOCA FORMATION
ZONA DE FALLA/FAULT ZONE
TWR INICIO DE ROCA METEORIZADA/TOP OF WEATHERED ROCK
TSR INICIO DE ROCA SANA/TOP OF SOUND ROCK
PESO DE ESCLUSAS PROPUESTO/PROPOSED LOCKS FLOOR
NOMBRE DEL POZO/NAME OF BORING
ELEVACION/ELEVATION
PROF. DEL POZO/LENGTH OF BORING
CONTACTO/CONTACT
FALLA/FAULT
FALLA PROBABLE/PROBABLE FAULT
PORCENTAJE/SCALE
0–25%
25–50%
50–75%
75–85%
85–100%
RQD%
CR%
UCS (MPa)
PLT-UCS (MPa)

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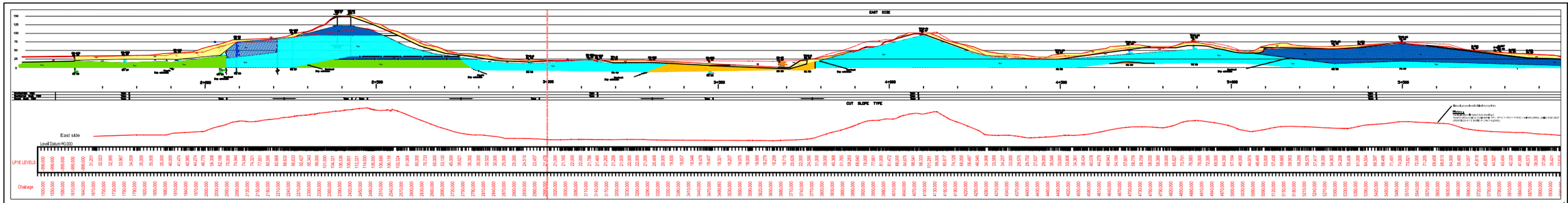
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION
GEOLOGICAL LONGITUDINAL SECTION
+ EXCAVATION PROFILE
WEST SIDE P1A, CHAINAGE 6000-9000

Rev.	Date	Description	Drawn	Checked	Approved
A	15/11/2011	FINAL REPORT	JBR	JCB	JBR
–	15/04/2011	FINAL DRAFT	JBR	JCB	JBR

Contract	Scale:	Drawing n°:	Rev.:	Folio
SAA–97462	1/3	D3 – 0 – 013	A	1 / 1



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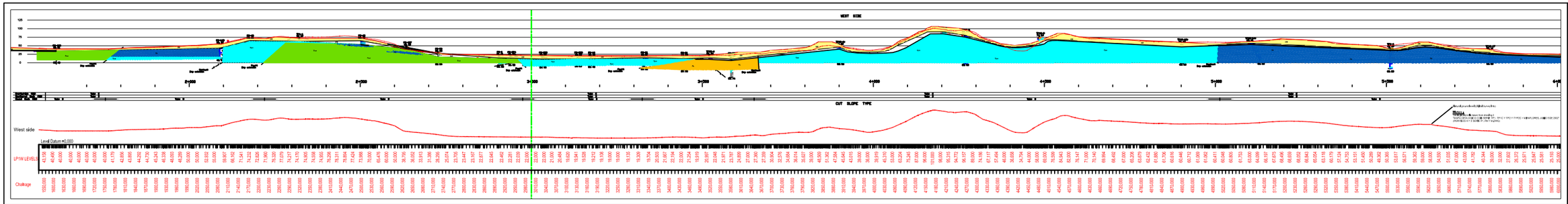
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION – LONGITUDINAL SECTION
PANAMA CANAL – ALIGNMENT P1A, EAST SIDE
COMPARISON TOPOGRAPHY GEOLOGICAL SECTION

Author	Date	Drawing nº:	Rev	Sht.
03-0146	Ufe	03 – 0 – 014	A	1/1



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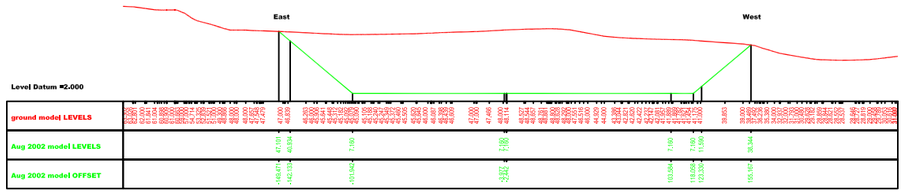
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION - LONGITUDINAL SECTION
PANAMA CANAL - ALIGNMENT P1A, WEST SIDE
COMPARISON TOPOGRAPHY GEOLOGICAL SECTION

Revision	Date	Drawing n°:	Rev.	Sheet
03-2010	1/Ju	D3 – 0 – 015	A	1 / 1



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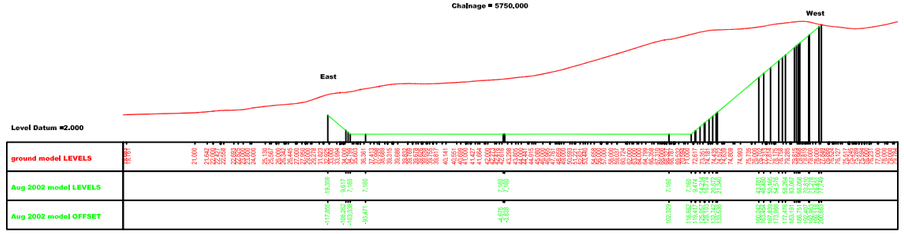

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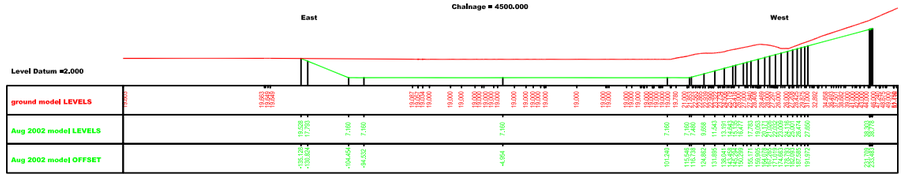

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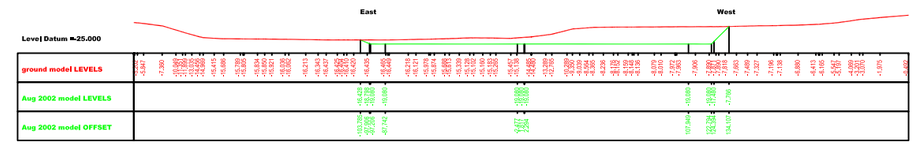
panama canal chainage 3000 - 6000
Horizontal Scale = 1:1250,Vertical Scale = 1:1250



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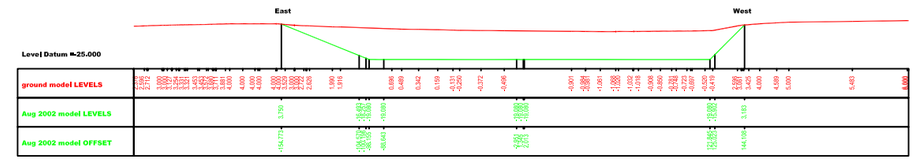

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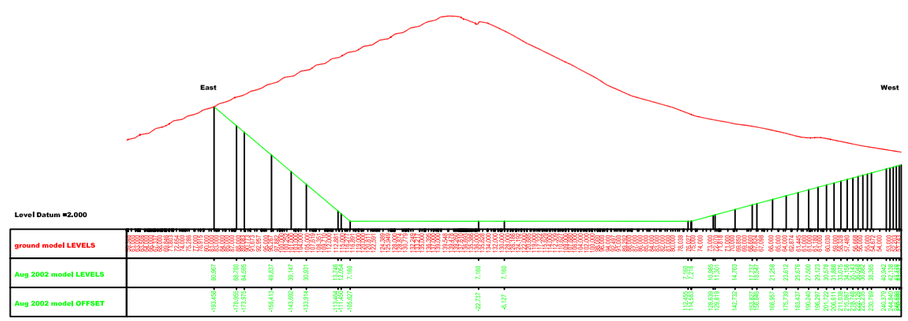
panama canal chainage 9000 - 12000
Horizontal Scale = 1:1250,Vertical Scale = 1:1250



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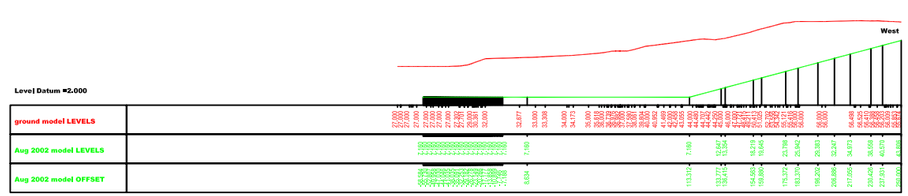

Chainage = 1500.000

pamama canal chainage 1000 - 3000
Horizontal Scale = 1:1250,Vertical Scale = 1:1250

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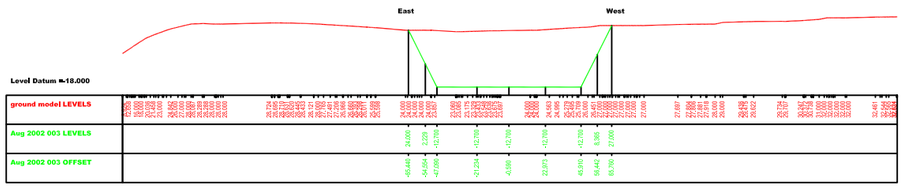
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION - CROSS SECTIONS
PANAMA CANAL P1A
CHAINAGES : 1000 - 6000 & 9000 - 12000

No.	Date	Description			
0	15/11/02	Final report	JDE	JDE	JDE
A	05/06/03	Draft Edition	JDE	JDE	JDE

Contract	Scale:	Drawing n°:	Rev.:	Folio
SM-97462	1/1250	D3 – 0 – 016	A	1 / 1



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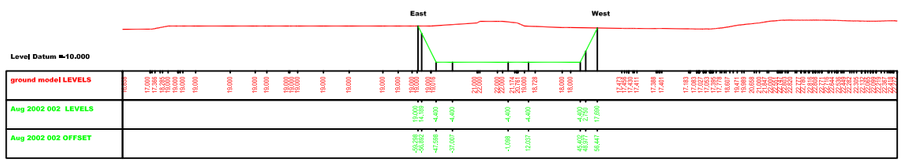

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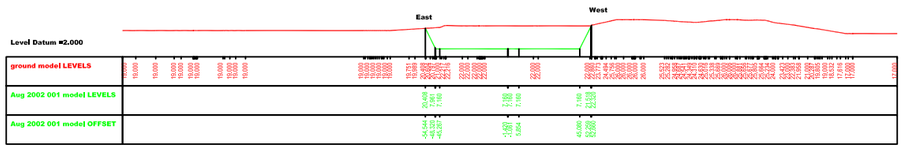

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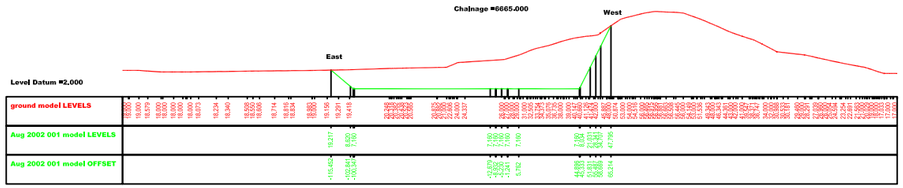

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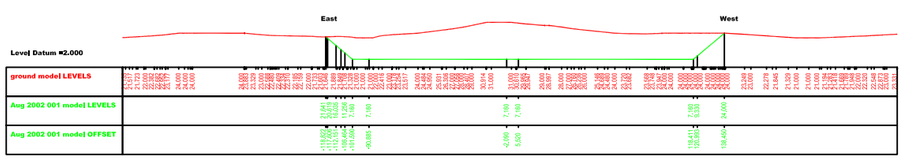

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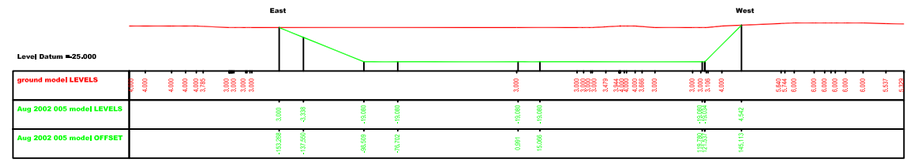

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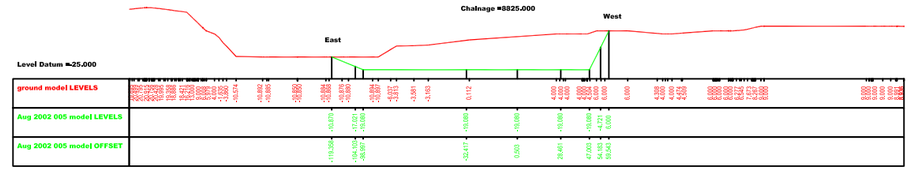

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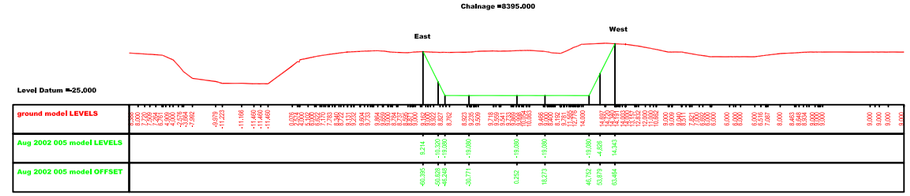

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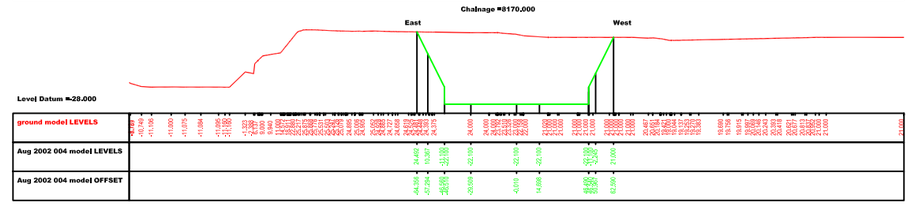

Chainage =7935.000

panama canal chainages 6000 - 9000
Horizontal Scale = 1:1250,Vertical Scale = 1:1250

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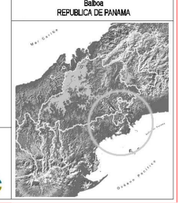
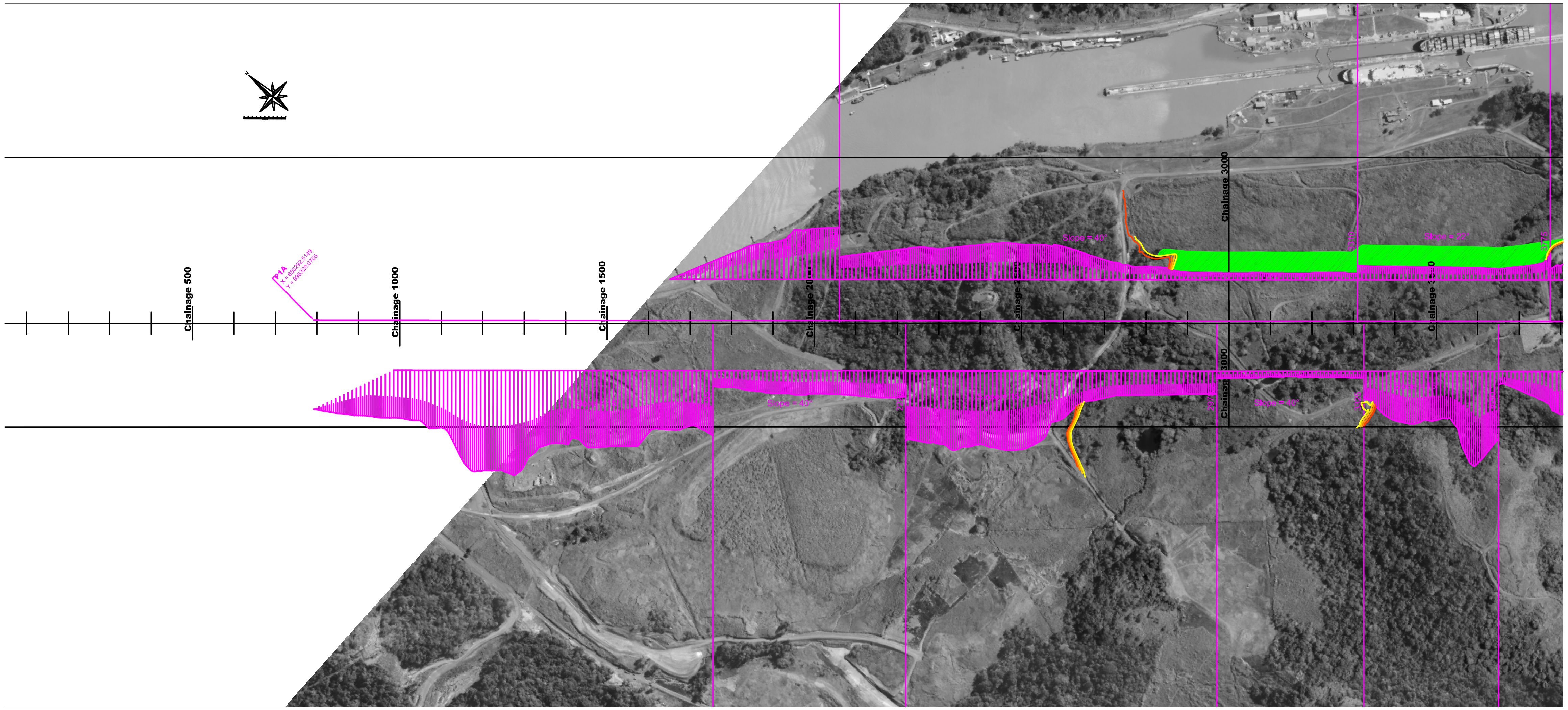


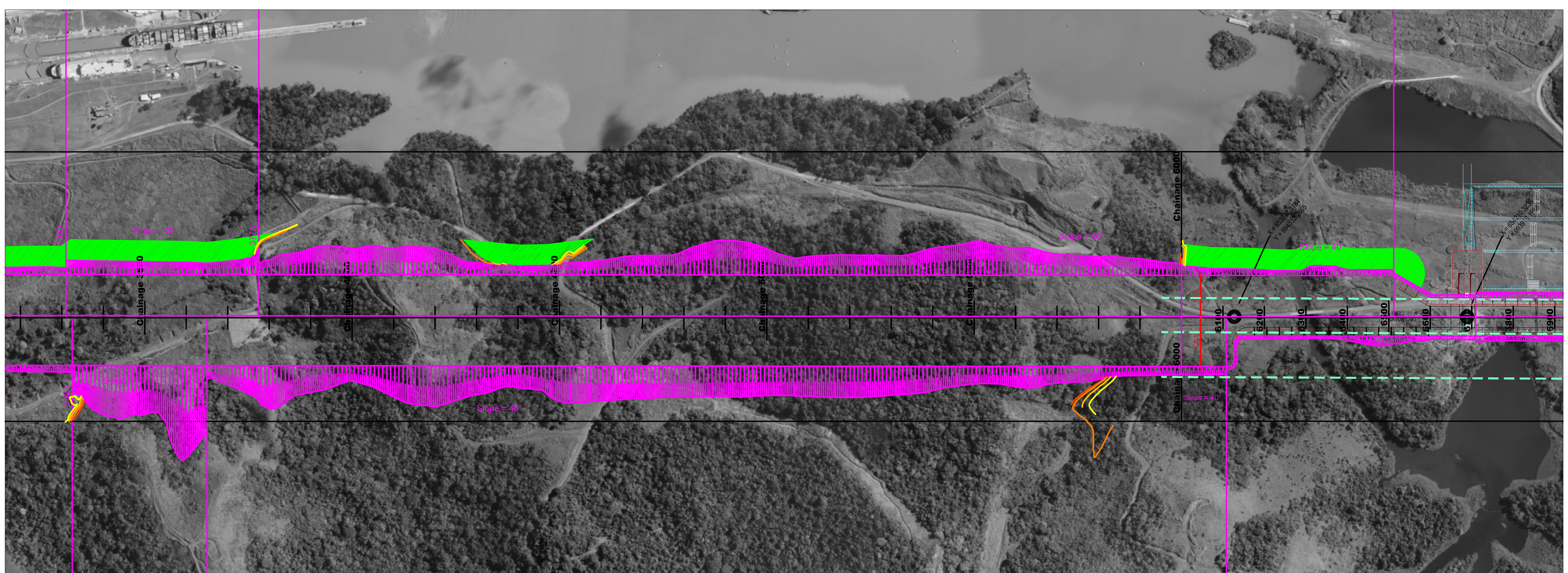
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

TRIPLE LIFT CONFIGURATION - CROSS SECTIONS
PANAMA CANAL P1A
CHAINAGES 6000 - 9000

REV.	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
A	12.11.08	Prel. issue	JM	JB	JB
B	26.01.09	Final issue	JM	JB	JB

Contract	Scale	Drawing n°:	Rev.	Folio
SM-97462	1/1250	D3 – 0 – 017	A	1 / 1





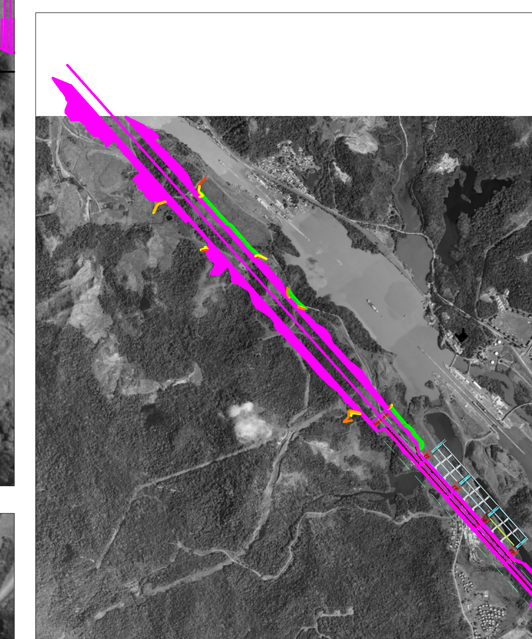
= FILL

= 27 METER LEVEL LINE

= 28 METER LEVEL LINE

= 29 METER LEVEL LINE

= 30 METER LEVEL LINE



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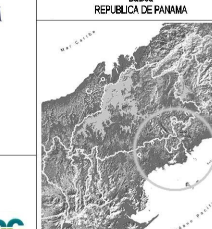
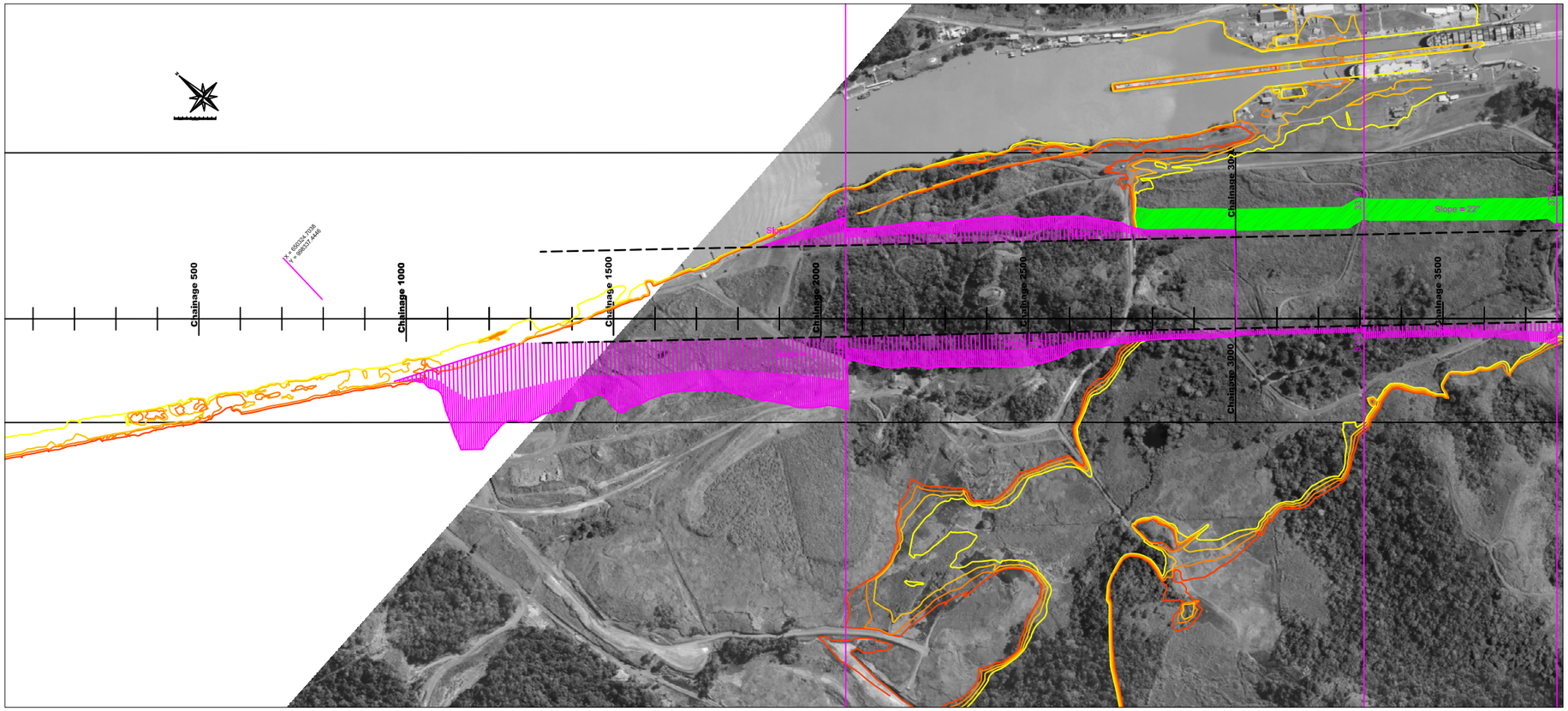


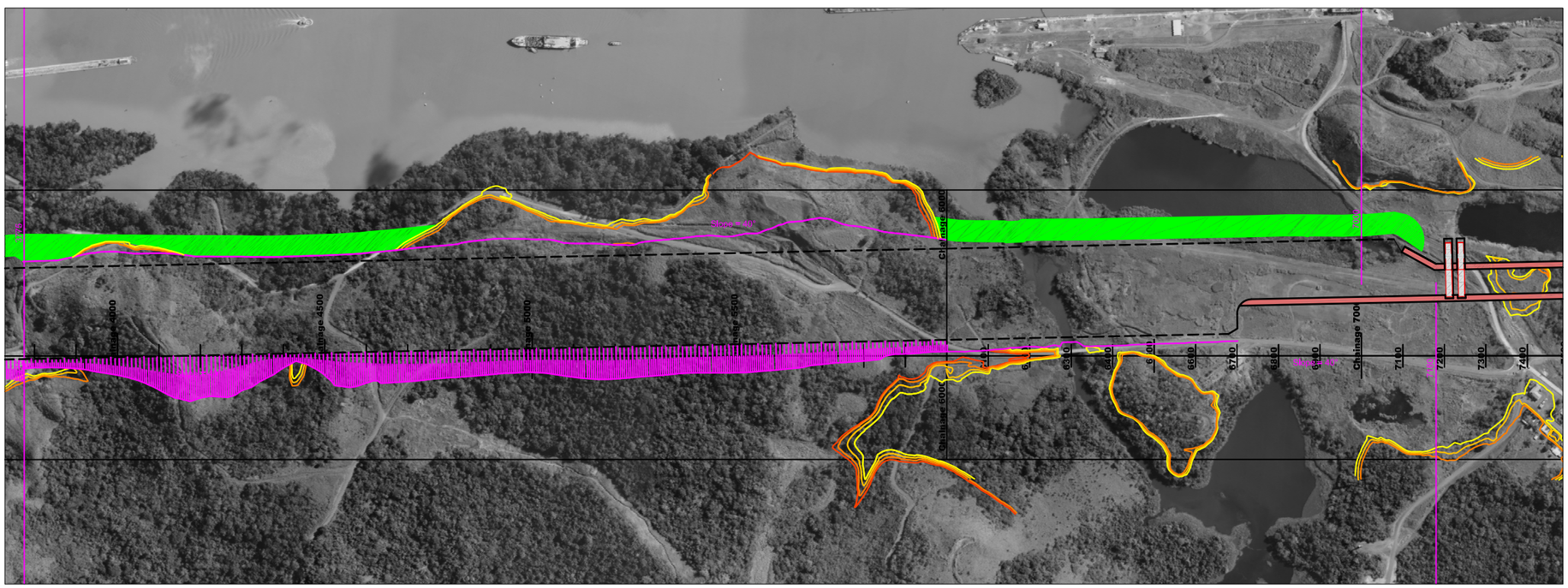
Diseño Conceptual de las Esclusas Post Panamax
Concept Design of New Lock Structure

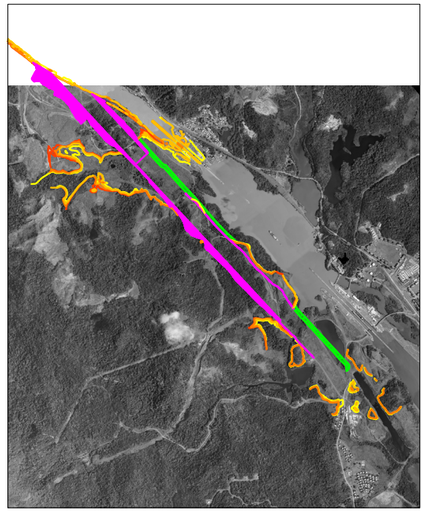
TRIPLE LIFT CONFIGURATION - LAYOUT
PANAMA CANAL P1A
INDICATION OF 27 - 30 M TOPOGRAPHIC LEVELS

REV.	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
F					
E					
D					
C					
B					
A	12/11/02	FINAL REPORT	BME	JDR	JDR
–	06/09/02	FIRST EDITION	ASS	PWA	JSS

Contract	Scale:	Drawing n°:	Rev.:	Folio
SAA-97462	1/4000	D3 – 0 – 018	A	1 / 1







= FILL

= 27 METER LEVEL LINE

= 28 METER LEVEL LINE

= 29 METER LEVEL LINE

= 30 METER LEVEL LINE

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Diseño Conceptual de las Esclusas Post Panamax
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TRIPLE LIFT CONFIGURATION - LAYOUT
PANAMA CANAL P1B
INDICATION OF 27 - 30 M TOPOGRAPHIC LEVELS

REV.	DATE	DESCRIPTION	DRAWN	CHECKED	APPROVED
F					
E					
D					
C					
B	12/11/02	FINAL REPORT	JBE	JCR	JCR
A	06/07/02	FIRST EDITION	JBE	JNA	JCR

Contract	Scale:	Drawing n°:	Rev.:	Folio
SAA-97462	1/4000	D3 – 0 – 019	A	1 / 1