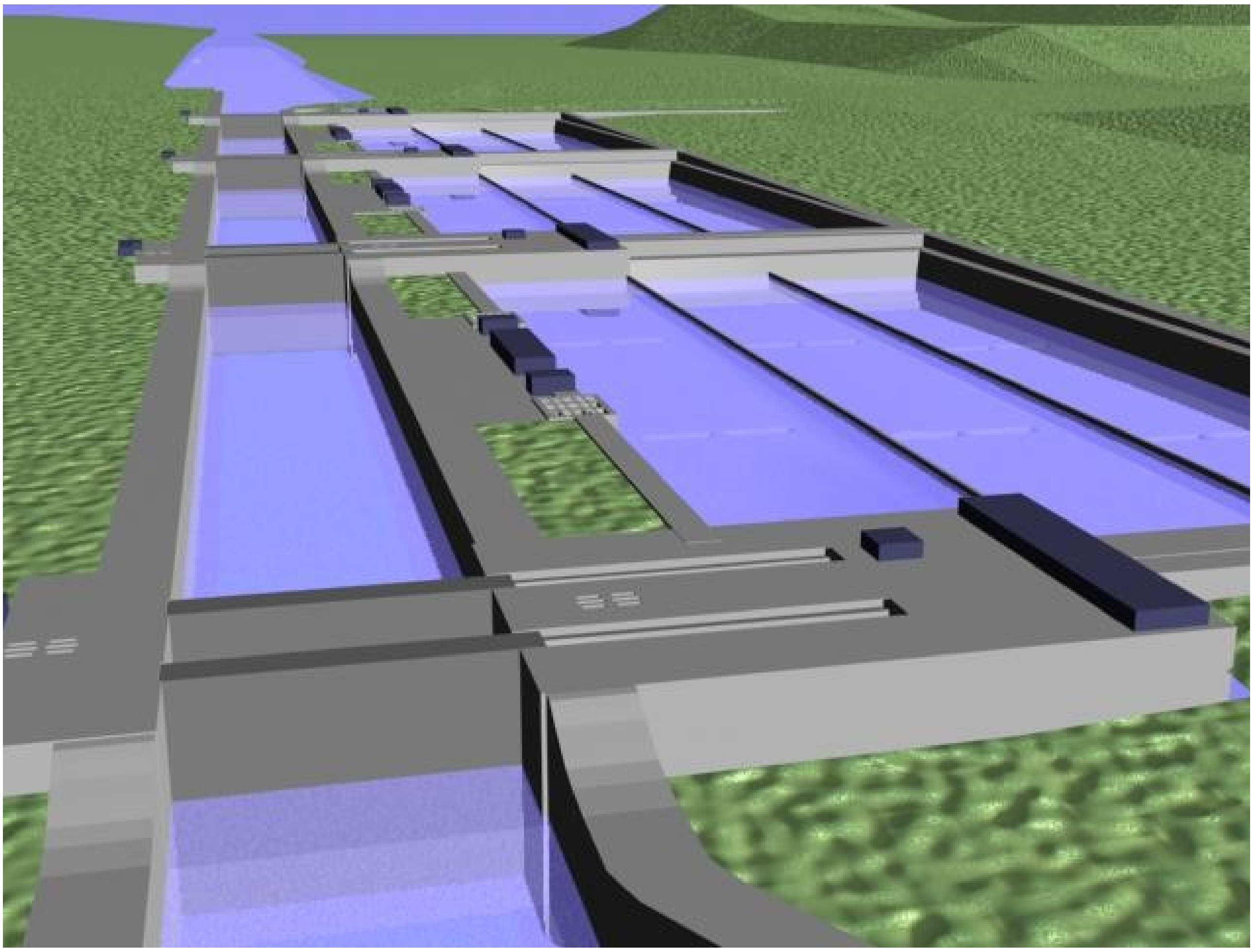


Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual design - Contract SAA-143351

EXECUTIVE SUMMARY

Tractebel Development Engineering
TECHNUM
COYNE ET BELLIER
Compagnie Nationale du Rhône

in association with

SBE
PORT OF ANTWERP
FITA AOSO

contents

Executive Summary

Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual Design: Triple lift lock system with 3x3 water saving basins

This report contains the conceptual design of a triple lift lock structure with 3x3 water saving basins for the new Post Panamax locks at the Pacific and Atlantic sides of the Panama Canal.

Reference is made to the 2002 report "Conceptual Design of a triple lift lock system at the Pacific side of the Panama Canal", part of contract SAA97462 awarded to CPP.

The actual study is the subject of a new contract SAA143351 awarded to CPP in November 2004.

The new design criteria for the lock structures are given in the report of task 2-Part A with reference P/A/2revA-v02 dated 29/04/05.

In general, following main modifications have been applied to the original Pacific design:

- reduction of lock width by 5m from 61m to 55m;
- reduction of vessel beam;
- tug boat assisted positioning system instead of locomotives;
- new channel alignment;
- new seismic conditions;
- reduction of minimum water depth (16.8 instead of 18.3m).

For the Atlantic design, the same lock system will be retained, but requires harmonization in order to cope with specific local conditions (geotechnical and seismic conditions, topography, Atlantic tidal levels, channel alignment).

The final reports contain following subtasks for both locks:

- a. Lock siting and Lay-out
- b. Lock Walls
- c. Emptying and Filling System
- d. Lock Operating Gates
- e. Culvert and Conduit Gates
- f. Operating Machinery
- g. Lighting
- h. Electrical Power and Power Requirements
- i. Entrance Walls
- j. Operating Structures
- l. Construction Plan and Schedule
- m. Quantities

For each of these tasks a separate report has been prepared, although for some specific tasks reference has been made to the original concept design (2002) especially when there are practically no changes made. Even so, for some tasks of the harmonization of the Atlantic locks, reference is made to the Pacific locks, as it is obvious that whenever possible the same concept has been retained for both locations.

Before resuming the results of these conceptual studies, it is important to remind that the design is especially based on following special criteria and requirements, as has been discussed on several occasions with ACP:

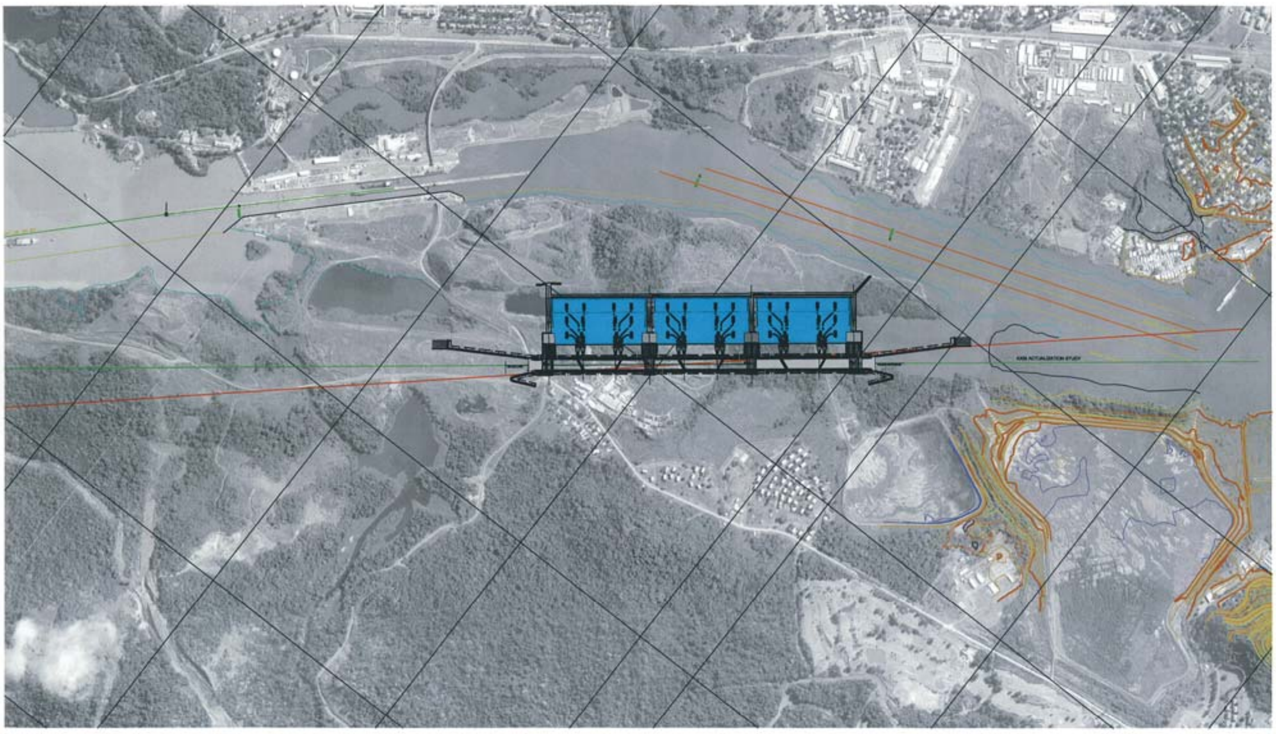
- The new locks are a demand driven system, and its operating times determine the capacity of the system.
- Reliability is another basic requirement, as any shutdown time means loss of income.
- Maintenance has to be kept to a minimum.
- Construction cost should be minimized.
- Operation facilities and systems should be kept simple and reliable.

Consequently, ACP has chosen a triple lift lock system, equipped with 3x3 water saving basins.

a. Lock siting and layout

PACIFIC ALIGNMENT AND LOCK SITING

The ACP has done further investigation and analysis on this item in order to minimize the excavation volumes. The final result is a sligfhtly curved alignment which allows to shift the channel towards the existing Miraflores locks. At the same time, the lock structure can be shifted towards the Pacific Ocean, again reducing the lock excavation volumes. Nautical access conditions and geological circumstances remain practically unchanged when compared to the original study.



Aerial view of the lock siting at the Pacific side of the Panama Canal

ATLANTIC ALIGNMENT AND LOCK SITING

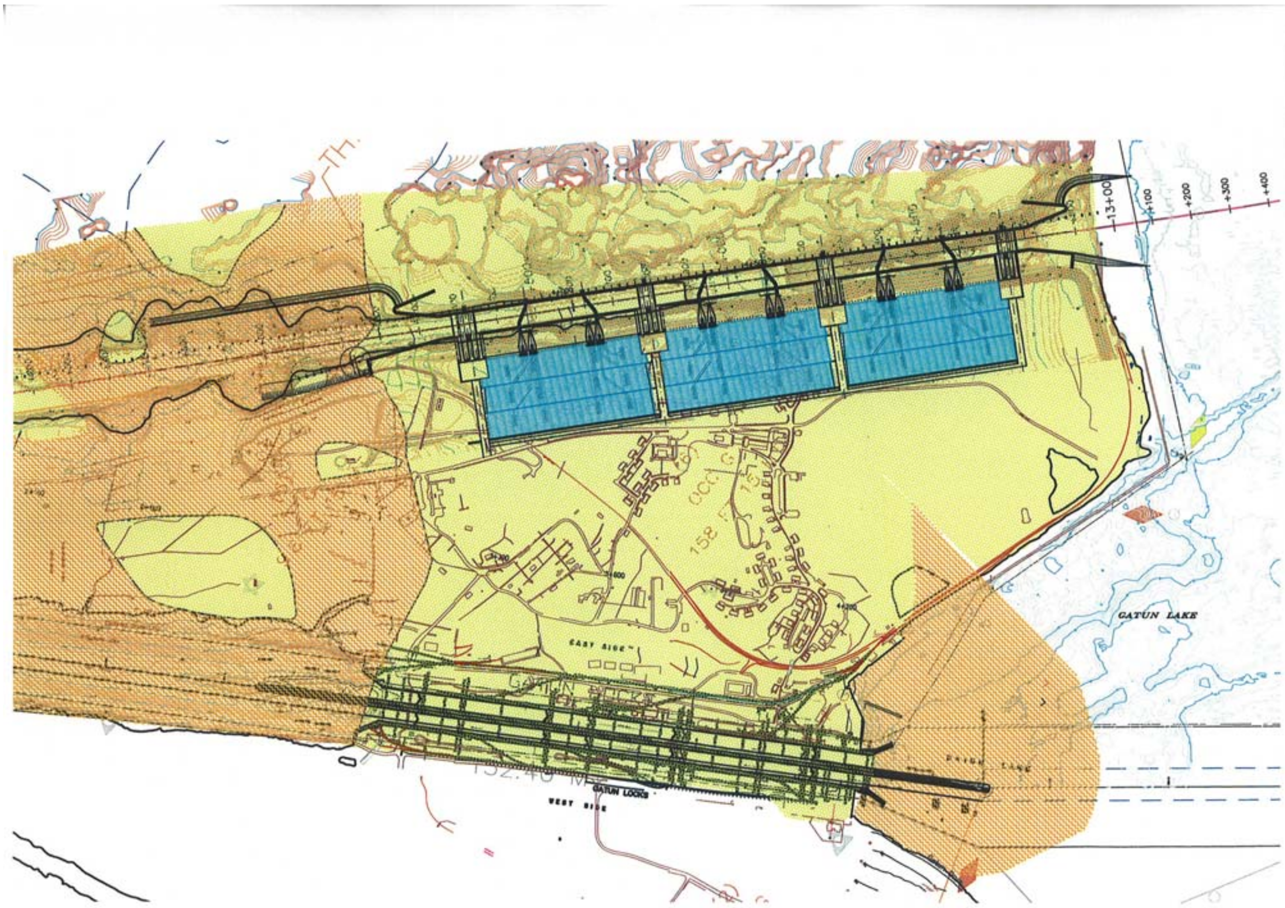
The ACP investigated the alignment at the Atlantic side prior to the actual study. An alignment has been retained that coincides with the 1942 third lock excavation, only a minor shifting of the axis to the Westside (direction of existing Gatun Locks) has been further evaluated. It was confirmed that this alignment is the best choice as far as excavation volumes are concerned.

This alignment is also very convenient from the nautical point of view: situated immediately near Gatun Lake at the southside, and connected to the Atlantic by a straight access channel.

Some special attention has been paid to the geotechnical conditions: the entire lock structure has been located in the Gatun rock formation; except for part of the Atlantic side entrance wall which is extending into the weak Atlantic Muck formation. In order to avoid working in bad soil conditions, it has been proposed to replace the gravity type entrance wall by flexible dolphins over the corresponding length.

It was found that excavation volumes for the Atlantic side are relatively high in comparison with the Pacific side. This is mainly due to the fact that the water saving basins require much more excavation.

Attention has to be drawn to the fact that the future 4th lane has not been taken into account when determining the lock alignment. The required excavations will be much larger than those actually determined for the third lane.



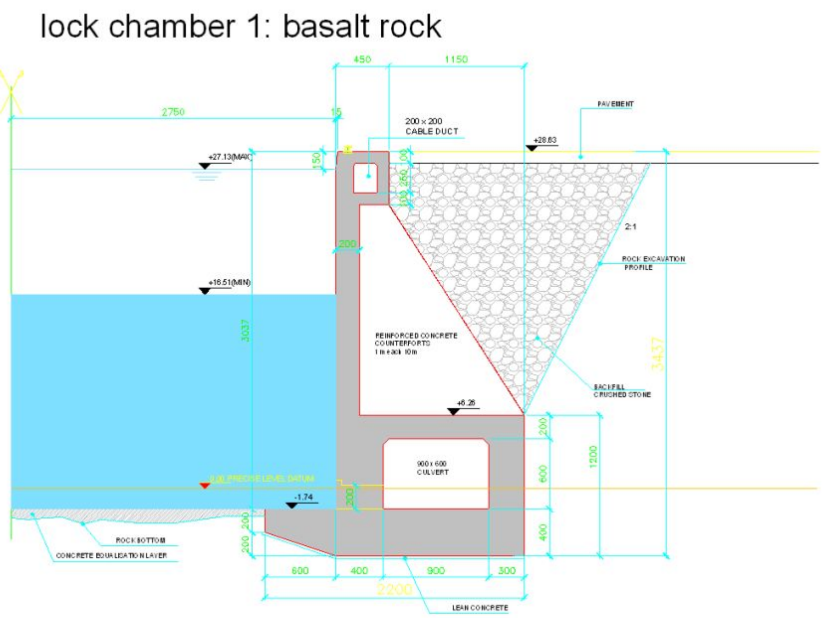
Aerial view of the lock siting at the Atlantic side of the Panama Canal

b. Lock walls

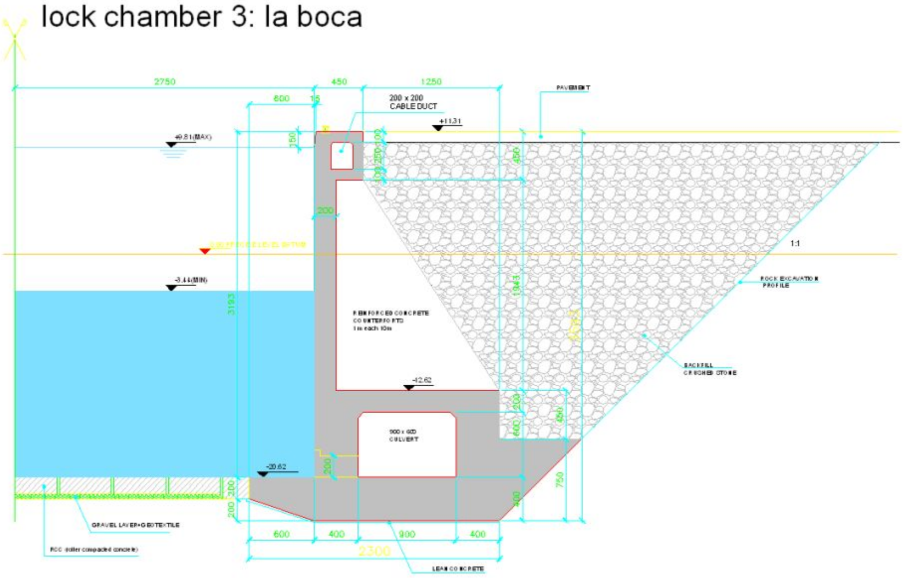
The choice of the lock wall type depends mainly on geo-technical and seismic conditions, loadings (water levels, sill levels) and filling and emptying system.
As the lock structure is situated mainly in rock bottoms (Basalt and La Boca at the Pacific side, Gatun rock at the Atlantic side), a number of possibilities of lock wall type are excluded.
As loading conditions are rather severe, and the requirement to have a very performing E/F system (with large culvert dimensions), the choice of a gravity type lock wall has not been difficult to make.
Of course it is clear that even the gravity type lock wall may lead to a lot of different alternatives which have to be investigated and optimized during further studies.

In this conceptual design, the lock wall dimensions have been chosen primarily in order to minimize excavation. A solution without using steel reinforcement (mass concrete gravity wall type), or a solution with RCC (roller compacted concrete) has not been considered anymore in this actual study as experience has shown that the application of reinforcement has become common practice in modern quay and lock wall construction methods, which also lead to more economical structures. However, during further and detailed design it is recommended to investigate if such a solution could be envisaged for the Panama locks. This is even more actual since the prices of steel reinforcement have been doubled over the last 18 months.

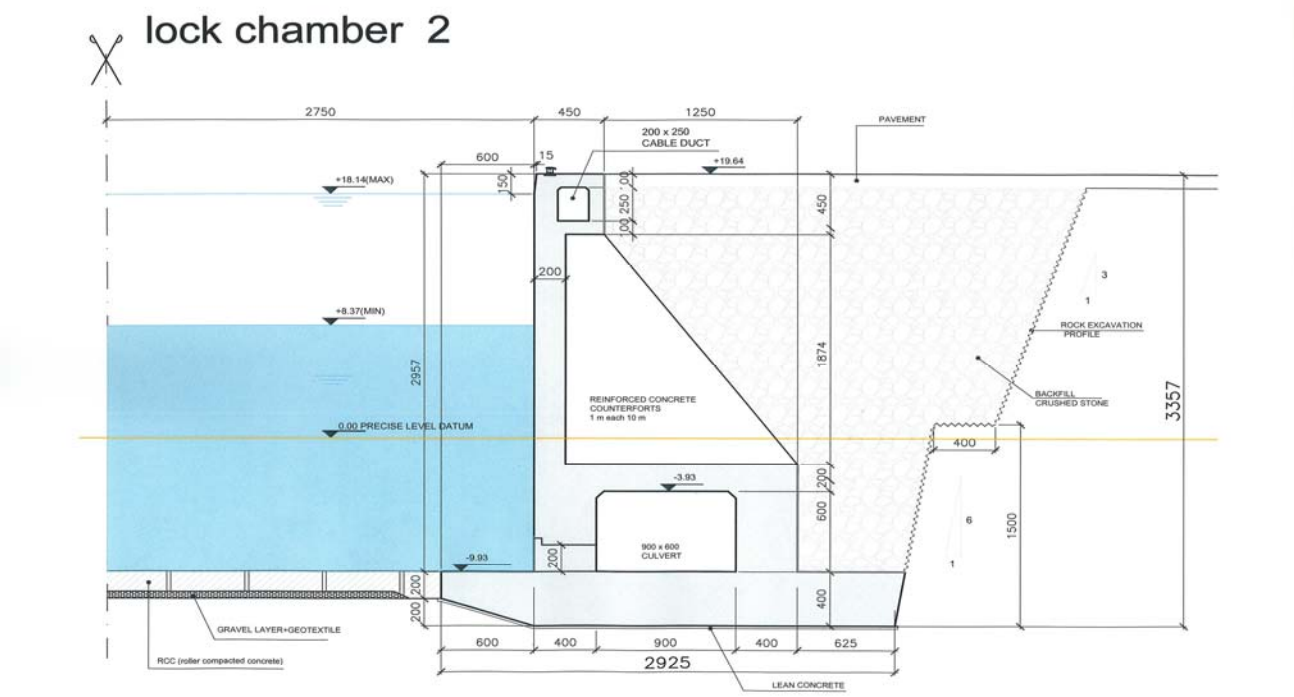
One major change in design criteria is most certainly the higher peak ground acceleration (PGA "a"-value), which was raised by ACP from 0.21g to 0.40g (Pacific) and 0.41g (Atlantic).
At the other hand, the removal of the locomotive tracks and the reduction in freeboard (1.5m instead of 3.0m) are favorable for the dimensioning of the lock wall structure. The new conditions together with the normal loading conditions and the local rock conditions have resulted in following typical cross sections for the lock walls.



Pacific side – typical cross section lock wall in Basalt



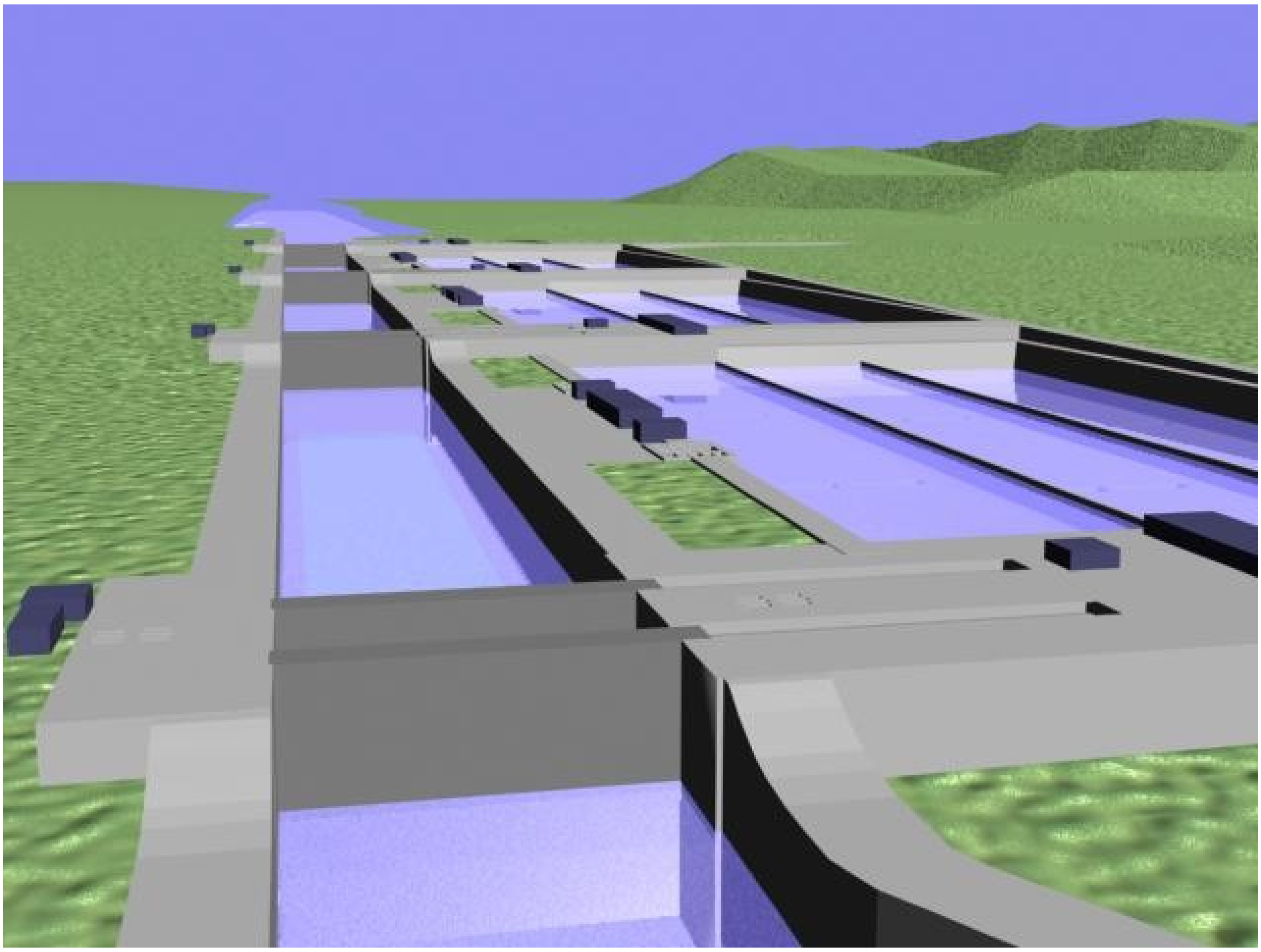
Pacific side – typical cross section lock wall in La Boca



Atlantic side – typical cross section lock wall in Gatun rock

c. Filling and emptying system – Water Saving Basins

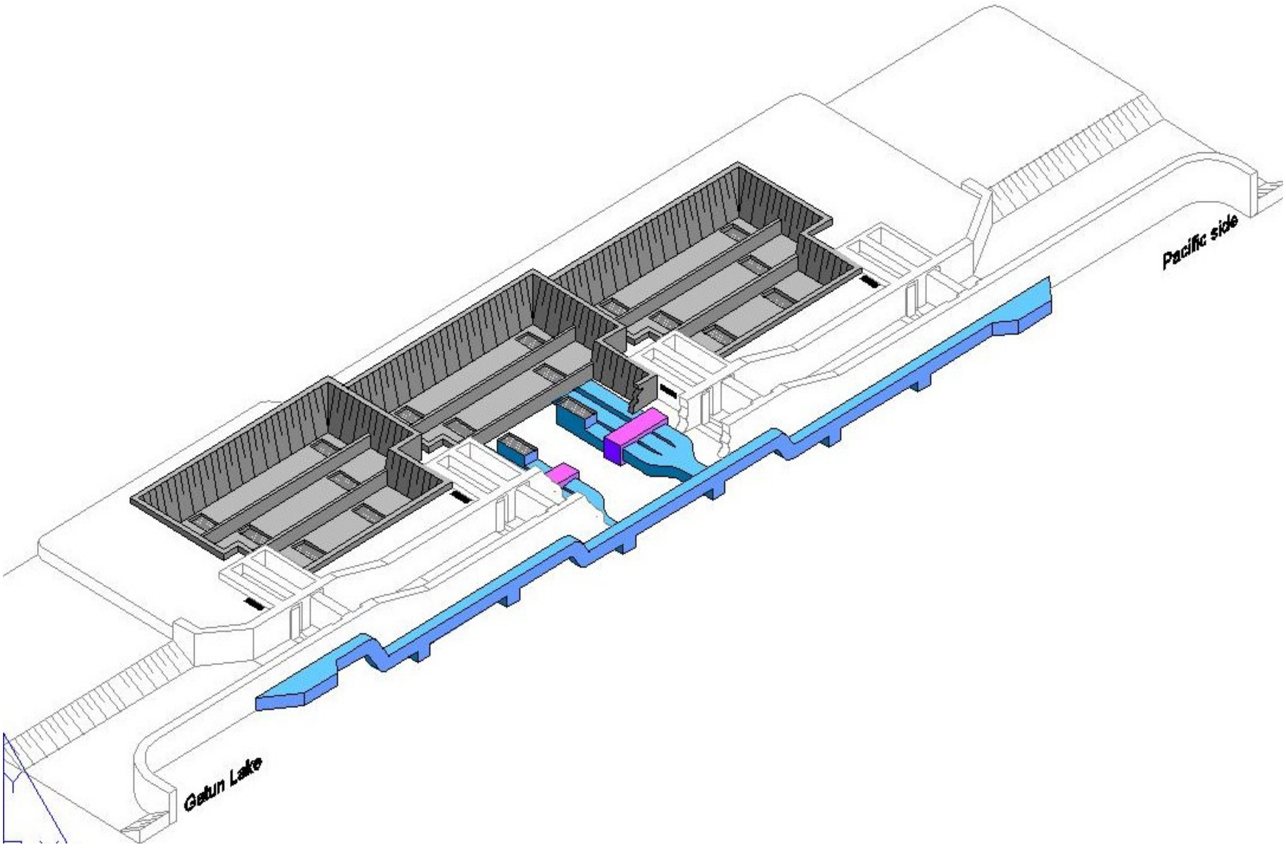
The ACP has retained a triple lift lock system with 3x3 side by side water saving basins for this actualization/harmonization study, as shown on following picture:



View on the triple lift lock system with 3x3 water saving basins

The hydraulic system originates from the 2002/2003 conceptual design and consists of longitudinal side wall culverts with ports, and water saving basins linked to these side wall culverts by means of conduits.
This system is composed of following main hydraulic elements:

- continuous culverts at both sides of the locks are integrated in the lock wall structure (dimensions Width x Height = 9m x 6m);
- both longitudinal culverts are linked with the lock chamber through the side walls by means of ports (2m x 2m), equally divided over the chamber length central part, in order to obtain as much as possible a well balanced filling and emptying;
- water saving basins (3 parallel basins for each lock chamber situated at one side of the lock) are linked to the culverts by two conduits per WSB. For each culvert (total of 12 conduits per chamber with a section of W x H = 4m x 5m at the Atlantic side and W x H = 4.5m x 6m at the Pacific side).



Layout of the E/F system – only one culvert shown

Attention has to be drawn to the fact that the system has been selected as being integrated in the lock walls, which is much more cost-efficient and maintenance free than a system with openings in the lock bottom floor (such as in the existing locks). The ports in the side walls can be closed by means of bulk head slots, and this way it is no longer necessary to retain the dry lock chamber condition. This allows a more economic design of several lock structure elements (lock gates and lock walls).

One of the most important conditions for the design of the hydraulic system are the hawser forces exerted on the ship during filling/emptying of the lock chambers.
Hawser forces are mainly induced by an unbalanced filling/emptying of the lock chamber. In this case of a side wall E/F system with 20 ports each side discharging not always at the same rate and time, water movement in the chambers occurs and consequently exerts forces on the ship. As the ship is tightened up by means of a positioning system, the hawser forces are limited to the capacity of this system, which could in the actual case be the tugboat system or simply a fixed mooring against one of the lock walls.
In this phase of conceptual design, without having the possibilities of measuring forces on a physical scale model, the hawser forces analysis has been performed by means of the combination of two hydraulic models. The first one being the Flowmaster software which calculates the discharge – time series through the ports, and a second one (2D/3D Delft) which calculates the movement in the lock chamber as a consequence of the uneven port discharges.
Several scenarios have been analyzed, and further optimization has been done in order to demonstrate that the hawser forces can be kept at a level which corresponds with the hawser forces criteria depending on the positioning system.

In general it has been concluded that the E/F system as proposed, with integrated water saving basins, is a technical-economical advantageous solution for the new Post-Panamax locks. It needs further optimization in a next study phase by means of more detailed numerical modeling, and finally it will have to be validated on a physical scale model.
Attention has been drawn to the fact that the same system, without using the water saving basins and if required to be working at the same capacity (number of ship transits per day), would need to be modified. However it is the consultant's recommendation that water saving basins need to be installed, operating the locks without water saving basins will then be considered as a special operating mode which allows increasing the valve opening times and rates.

d. Lock Gates.

Lock gate selection and analysis for the triple lift lock configuration has led to the application of the "Rolling gate" type. This choice has been justified by means of a multi criteria analysis, performed during the original 2002 conceptual design, to evaluate the miter gates and the rolling gates. The rolling gate type is the only existing lock gate type for this size of Post Panamax locks, and has been successfully used in Europe, especially in Belgium where the locks of Berendrecht, Zandvliet in Antwerp and Vandamme in Zeebruges are the largest in the world.

Picture of a rolling gate in its recess in the P. Vandamme lock in Zeebruges – Belgium

Furthermore, the rolling gate type has some particular advantages that are of utmost importance for the new locks that will be demand driven. One main advantage is certainly that the gate is moved horizontally in the transversal direction of the lock, into a lock gate recess chamber, which can be easily dewatered, and as such represents an ideal maintenance place and position. As there are two lock gates and lock gate chambers on each lock head, it is practically impossible that the traffic should be interrupted due to failure of a lock gate. Moreover, a lock gate can be floated and can be towed away as a vessel (for example if replacement is required, or when using the gate as a bulkhead to dewater the lock chambers).
The rolling gates have been designed for the normal operating conditions, as the dry lock chamber condition will not be retained as a basic requirement. Although, the outer gates have been designed to withstand the total water head that occurs during the dry lock chamber situation.
The lock gate structure has been analyzed using 2D/3D-structural engineering software and according to the expert's experience with rolling gates in Belgium. This analysis allowed to determine and verify the general dimensions of the different gates, the dimensions of the steel truss structure, and consequently it was possible to make a fairly accurate estimation of the weight of the steel structure.
Other auxiliaries, such as wheel barrow wagons, supports, etc, have been assessed according to the experience with the Berendrecht and Van Cauwelaert locks in Antwerp, which were also designed by the CPP-experts.
The main differences as compared to the original design (2002) are the reduced lock width (55m instead of 61m) and led to a further reduction of the lock gate size and weight. At the other hand, the new seismic design criteria have been taken into account.
It was also investigated if the lock gates, both on Pacific as on Atlantic side, could be standardized. This was indeed possible:

- the upper gates are identical for both Pacific and Atlantic (Gatun Lake side)
- the intermediate gates would only slightly differ in height and can thus be given the same overall dimensions (although it is the consultants opinion that this advantage should not be overestimated as there is a main consequence for the top levels of the lock heads, which will have to be adapted artificially)
- the downstream gates are different (due to the high tidal variation at the Pacific side)

Following table shows the main dimensions and weights of the different lock rolling gates:

GATE	**PA1 AT1**	**PA2-PA3 AT2-AT3**	**PA4**	**AT4**
Width (outside plating)	**7 m**	**10 m**	**11 m**	**10 m**
Spacing between vertical frames	**3.18 m**	**3.18 m**	**3.18 m**	**3.18 m**
Weight per lateral area (height x length)	**1340 kg/m²**	**1480 kg/m²**	**1500 kg/m²**	**1450 kg/m²**
Weight of gate structure	**1550 tons**	**2550 tons**	**2700 tons**	**2450 tons**

e. Culvert and conduit gates.

Similar to the selection procedure for the lock gates, a multi criteria analysis has been carried out to select the most convenient valve type for lock culverts and conduits. The most suitable valve was found to be the vertical fixed wheel type moved by means of a vertical hydraulic cylinder.

This is the same type of valve and operating system that is used at the Berendrecht lock, and has proved to be very reliable.

Nowadays vertical-lift valves of that type are preferred for big locks because they are cheaper to build and do not require the large space that is necessary for other valves as reverse tainter type for example.

In order to guarantee a maximum of reliability, the valves on the culverts are made redundant (two parallel valves per culvert, each operating on half of the culvert section. Each valve has a rectangular section of 4.5m wide x 6m high at the Pacific side as well as on the Atlantic side.

The valves on the conduits (in between the water saving basins and the lock chamber culverts) are not made redundant as such, but there are always two conduits for one WSB, which in fact gives the same redundancy as for the culverts. The valves on the culverts are 4.5m wide x 6m high at the Pacific locks and 4m wide x 5m high at the Atlantic side.

The valves have been designed for maximum operating and maintenance conditions, can easily be set in dry conditions using bulkheads at both sides of the valves, and can be reached through vertical shafts on both sides.

There are no noticeable changes required in this actualization of the former conceptual design, except for the changes in size of the gates.

f. Operating Machinery

Control system

The control system shall be efficient, safe and reliable and will require a minimum of staffing. The proposition for the control system of the 3-rd lane of locks of Panama is a distributed control system with several PLC's and a redundant optical fibre network connecting all the devices. Operator workstations shall be installed in the central control room and shall allow the control of all the installation. This is a very open system that allows future PLC's extensions by the simple connection of new devices on the network. But for reasons of redundancy and proximity during exceptional or maintenance operations, a local control near the concerned equipment shall be supplied.

Gate operating machinery

Each rolling gate is moved by steel cables connected at anchorage points by compensating beams on both sides of the gate and wound around the cable drums of a winch. The two cable drums are driven by variable speed motors through gear boxes.

That type of machinery has been successfully used on the biggest existing lock gates (Berendrecht, Zandvliet, Zeebruges,..).

The two main AC motors are duplicating the drive in the event of failure of one motor. Moreover, a small emergency motor can be used to move the gate at reduced speed if the two main motors are not available.

Valves operating machinery

The culvert and conduit valves are operated by hydraulic cylinders. The pressure oil to open a valve is provided by separate hydraulic power units. The valves can be closed by gravity.

Each valve can be locally operated during maintenance from a control board located next to the hydraulic power unit.

The hydraulic cylinder solution is widely used and the technique has improved a lot especially by increasing the size and operating pressure. ACP has in particular a good experience through the replacement of existing operating mechanism of miter gates by hydraulic cylinders.

In general, there are no noticeable changes required in this actualization of the former conceptual design, except for the slight reduction of required power, which is not very significant when compared to the total investment cost of the Electro-mechanical equipment.

g. Lighting

Based on the experience of ACP, a lighting system is proposed that solves the main problems of the existing system. The most important problems of existing locks lighting are first of all the lack of visibility at the extreme ends of the lock chamber and in the lock chamber between ship and walls. The lights on high mast produce a glare that interferes with the pilot’s visibility, in addition, they are subject to corrosion and maintenance problems.

The lock chambers and gates will be illuminated by use of small 150W floodlights turned down in lighted vertical recesses in order to solve the problem of lack of visibility of the water level in the locks chamber and the space between the ship and the chamber walls. It will also provide to the pilot a clear cut reference.

It is clear that the high masts solution giving actually satisfaction to operating people does not need to be replaced by anything else. It was tried to facilitate the maintenance of the lighting fixtures by use of a ladder and platform combined with safety harness. To reduce the interference of high mast lighting with the pilot’s visibility and to reduce light pollution, the use of asymmetric lamps and deflectors are recommended. Corrosion effects can be reduced by improving the quality of material and tightness level.

The number of masts has been based on a illumination level of 100 lux (instead of 86 lux at Gatun) remaining almost constant all over the working area.

Due to the reduction in length of the entrance walls, the number of light poles has been reduced accordingly.

h. Electrical Power and Power Requirements

The main difference with the 2002 original conceptual design is the removal of the power transformers dedicated to the locomotives.

i. Entrance walls.

Entrance walls are the transitional part between the narrow (55m) lock entrance and the wider access channel or lake (in the case of the Atlantic locks at Gatun Lake side). They have been reduced in length due to the fact that the locomotive operated positioning system has been replaced by tugboats. Nevertheless entrance walls are required and will be used according to general international design guidelines to facilitate the entrance maneuver of the vessels, and in case of emergency to safely moor the ship.

j. Operating Structures

In general, the layout of the operating structures has not changed in this actualization/harmonization study when compared to the original conceptual design of 2002.

l. Construction Plan and Schedule

The construction plan and schedule of the 2002 conceptual design of the triple lift lock structure with water saving basins has been revised and updated in function of the new quantities, both for the Pacific and the Atlantic sides.
With these new quantities the Pacific locks execution time is reduced to 5 years, while the Atlantic side locks would require 6 years for construction as the excavation volumes are quite larger.

m. Cost Estimation

The scope of work for the actualization/harmonization does not include the cost estimation because ACP uses her own estimation method which differs from the CPP methodology. The scope of work only requires to prepare the quantities for both projects.

Nevertheless, as it is no real effort to estimate the construction cost at the same unit prices as the original conceptual design, and as it is the only way to evaluate the relative cost reduction linked to the size reduction of the lock, the cost estimation has been added to the report.
It should be mentioned that no indexation has been added to the unit prices of 2002, neither has it be taken into account that the steel prices have practically been doubled since then.

The result of this comparison are as follows:

- - Original triple lift lock with 3x3 WSB (2002) : 978,419,427.00USD
- - Pacific Actualization: 794,125,115.00USD
- - Atlantic Harmonization: 827,000,000.00USD

Conclusions and Recommendations.

The actualization study has allowed to adapt the original conceptual design of the Pacific triple lift lock structure with 3x3 water saving basins to the new design criteria. At the same time it was possible to further optimize the hydraulic E/F system in function of the hawser forces criteria.
Mainly due to reduced excavation and smaller lock width and reduced entrance wall length, a cost saving of +/- 19% has been realized.
The same concept design has been used for the harmonization study of the Atlantic locks. The main difference with the Pacific being the small tidal variation at the Atlantic Ocean, the large excavation volumes especially for the water saving basins, and the different rock base characteristics.

As at this moment it is being investigated if a system with 3x2 water saving basins would be beneficial (technical-economical), a possible solution to reduce the excavation volumes would be to retain such solution for the Atlantic side.

A next design phase before proceeding with tendering will require following scope of work:

- further optimization of the E/F system and related hawser forces analysis, by means of numerical modeling;
- physical model testing, with validation of the system;
- preparation of a reference design documentation (technical specifications, performance criteria, drawings, quantities survey and detailed cost estimation, planning of construction works), allowing to proceed with a tender procedure.

However it is necessary that ACP decides on the type of lock system to be retained, as well as on the positioning system that is going to be implemented in the Post Panamax locks.
The mitigation measures to be taken against or to prevent salt water intrusion into the canal have not yet been taken into account in lock design, but should be during further studies.

Some other tests and measurements still need to be carried out:

- detailed topographic and bathymetric measurements;
- additional soil investigation (field and laboratory);

May 2005.

Diseño Conceptual de las Esclusas Post Panamax

TASK 2

PART A: GENERAL DESIGN CRITERIA

RevA-Pacific Actualization and Atlantic Harmonization

Tractebel Development Engineering | TECHNUM | COYNE ET BELLIER | Compagnie Nationale du Rhône

in association with

SBE | PORT OF ANTWERP | FITA AOSO

1 General

1.1 SCOPE

Task 2 of this conceptual design study for a triple lift lock configuration at the Pacific and Atlantic side of the Panama Canal 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

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, the design criteria will be implemented as far as this is required on this actual concept design level.

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.
Licitación N°. SAA-143351
Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual Design.

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 "Definic ión, 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

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
- Seismic Design Guides for Port Structures – PIANC 2001

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

The main dimensions of the ships to be taken into account are:

- length : 385.76 m (1265ft)
- width : 48.78 m (160ft)
- draft : 15.24 m (50ft)

1.4.2 LOCK DIMENSIONS

1.4.2.1 Length

The useful length of the lock chamber is fixed at **426.72** m.(1400ft). It is the distance between the downstream inner lock gate plating and the upstream vertical concrete sill.
The chambers are equipped with two gates for reasons of :

- security
- maintenance

1.4.2.2 Width

The width between the lock walls has been fixed at **55.00 m**.

1.4.2.3 Waterdepth

The **minimum** nautical waterdepth over the sills in the lock chamber is **16.76 m (55ft)**. A higher value of 18.30m has been recommended in order to obtain a minimum underkeel clearance of 3.0m inside the locks.
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	+27.13m PLD (89ft) +24.99 m PLD (82ft)
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
Atlantic Ocean	Maximum Level Minimum Level	+0.41 m PLD -0.15 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.
- 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 of the access channel and the lock system has been determined by ACP.
- The locks will be operated by means of tugboat assistance, and not by locomotives.
- The lock gates will be of the rolling gate type.
- The hydraulic emptying and filling system will be a side wall integrated culvert system with lateral ports.
- The locks will be operated using 3 water saving basins connected to each lock chamber.

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 has been evaluated. The dry lock chamber condition is considered as a fundamental load condition for the lock walls and the outer lock gates.

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.

With rolling gates it is be possible to perform maintenance in the gate recess.

The filling and emptying system is located in the lock walls, and needs no special maintenance.

CPP Consorcio Post-Panamax

Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual Design – Contract SAA-143351

ATLANTIC LOCKS HARMONIZATION

Task 2a – Design Criteria – Geology – Geotechnics

Tractebel Development Engineering · TECHNUM · COYNE ET BELLIER · Compagnie Nationale du Rhône

in association with

SBE · PORT OF ANTWERP · FITA AOSO

1 General considerations

The excavation along the initially designed alignment started in 1939. This alignment is shown on drawings in **Ref. 1 and 2** and on drawing D3-0-200.

ACP recently decided to shift the axis about 38 m to the West. It is this new alternative A-1 that makes the object of the present study.

2 Consulted Documents and relevant data

The following documents were consulted:

➢ *Drawings*

- **Ref. 1** *Mapa geológico final con series A1, A2 y G (1:5000)*
- **Ref. 2** *Geologic map - Atlantic side (1:5000) (con ubicación de perfiles geológicos)*
- **Ref. 3** *Geological profiles A-A, B-B, C-C, D-D, E-E, F-F for Alternative A-1 (Atlantic side)*

➢ *Reports*

- **Ref. 4** *The third locks project, Final report on modified third locks project, Part II – Design, Chapter 5 – Foundation and slopes, August 1943*

- **Ref. 5** *Third set of locks project, Summary results of test on rock formations (Atlantic side), prepared by Soil and Material Laboratory of ACP, August 2001*

- **Ref. 6** *Third set of locks project, Summary results of test on rock formations (Atlantic side), prepared by Soil and Material Laboratory of ACP, March 2002*

➢ *Drillhole logs*

- **Ref. 7** *TA1-1 to 5 and TA1C-1 to 11*

3 Site visits

A first attempt by ground to inspect the western upper part of the 1939 excavation was unsuccessful owing to the high vegetal and soil cover.

A second site visit was organised on 10/12/2004. The access by boat from the northern plug (Atlantic side) allowed inspecting the 65 years old excavation.

As expected and suggested by morphology, the contrast is very sharp between (i) Gatún Formation and (ii) Atlantic Muck Formation

(i) The Gatún Formation continues to stand mostly sub-vertical cuts. The original cuts, including traces of excavating tools, are very well preserved (**Photos 1 and 2**). They are up to about 15-m high towards the southern end (Gatún Lake side). Exposed rock mass is very massive, featuring mainly transitional changes between the different sedimentary units, with few open bedding joints **(Photo 3 and 4)**. Some cross bedding fabrics are visible in the S section. Bedding planes, near horizontal to the N, get slightly steeper towards the southern end, where the dip may reach up to 10°. Some small size overhangs caused by differential weathering and erosion have been observed.

(ii) The Atlantic Muck Formation underlies very flat topography. No typical black, organic-rich material is visible in the visited section, but only recent, red-brown residual soil.

4 Core inspection

Cores from relevant boreholes, representative of local lithology and weathering profile, have been inspected. The visual inspection has been completed with the examination of colour paper prints of most of the cores.

5 Geology

The regional geological setting can be schematically described as a complex, but roughly NE-SW trending antiform. It is ascertained by the presence of older rocks in the central part of the isthmus and of younger rocks towards the coasts. Swampy coastal deposits (muck) filled the topographic lows during the slow Pliocene emergence.

5.1 LITHOLOGY

The bedrock in the project area is relatively homogenous, consisting of shallow marine sediments including a significant amount of volcanic material. They are assigned to the Gatún Formation that lies toward the top of the Tertiary sequence composed of sedimentary and, subordinately, volcanic rocks. The overburden, comprising residual soil, highly weathered bedrock and the swampy muck deposits filling former irregularities, can exceed locally 30 m-thick in the Atlantic side. In the Gatún side, sound bedrock lies at shallower depths of only 3 to about 14 m.

The geological description of the drillholes is presented in Ref. 7. The weathering and lithologic profiles of the boreholes relevant for the A-1 alignment are summarised in Table 1 (this report). Accordingly, a complete geological profile in the concerned area includes:

- residual silt, clay (max. 6 m thick) and spoil from the 1939 excavation
- typical Atlantic Muck (soft, black to dark grey, organic-rich, highly plastic clay and silt), up to 30-m thick to the N, but is not present to the S

- weathered bedrock, residual product of the underlying rocks of the Gatún Formation, 0.5 to 5 m thick
- fresh bedrock, consisting of tuffaceous sandstone, siltstone, conglomerate, pumiceous sandstone and tuff.

5.2 SITE STRUCTURAL SETTING

The sedimentary sequence forms a homocline, with low dip towards the NNW, almost perpendicular to the alignment of the locks. Bedding dip varies from about 2° in the N (Atlantic side) to about 10 - 12° in the S (Gatún side). Local steeper bedding dip is related to syn-sedimentary fabrics (e. g. cross bedding), not to tectonic deformation.

Sedimentary units are mainly thickly bedded, generally with transitional contacts, with few bedding joints. Fine lamination, characteristic for some horizons, is also chiefly tight.

No significant folds or faults have been identified so far. One minor fault is reported in **Ref.4**: exposed during excavation in the upper part of the east wall of the cut near “Station 121+00”, it consists of two narrow shear zones steeply dipping to the E, separated by undisturbed rock.

Figure 1 highlights some of the main morpho-structural lineaments. NNE and NE trending lineaments are among the most frequent and persistent elements. Both these trends are parallel to regional faults or fault segments, namely the Gatún fault and the Limon fault. They clearly control tectonic depressions, further evidenced by the rims of the Gatún Lake. The shape of the main central depression suggests either a pull apart basin, indicative of strike slip movement, or simply tensional regime and normal faulting.

Another relevant detail is the lack of significant morphologic evidence of faulting in the area of the proposed lock site. This is consistent with the lack of faults in the 1939 excavation. It is finally noticed that the western and the eastern blocks feature different morphologic patterns.

More detailed analysis based on accurate aerial documents is needed to refine this analysis and to assess the significance for the project design.

It is recommended that ACP further investigates the possible fault to the west of Gatun Dam as present studies are considering this site for future dam construction.

6 Geotechnical aspects

Abundant subsurface investigations documented, since the early stages of the study, the presence of thick swampy deposits in the Atlantic side. For obvious reasons, the heavy structures of the locks will be conveniently located in the Gatún side, where the bedrock is shallower.

6.1 GEOTECHNICAL PARAMETERS OF SOIL AND WEATHERED BEDROCK

Taking into account the potential high spatial variability of the overburden characteristics, only data from relevant boreholes, i.e. those holes in the Gatún side driven through significant soil cover, were considered. These boreholes are: TA1-4, TA1C-6 and TA1C-9.

The main findings are summarised below.

- The overburden mainly consists of inorganic elastic sandy silt / silty sand (MH) and silty sand with non plastic fines (SM) types.
- No typical black muck was intercepted in this area.
- There seem to be constant distinction between the MH and the SM soils: the SM soils are generally characterised by lower plasticity index and, in particular, higher relative density as inferred from the SPT blow count.
- The colour seems to be an effective indicator of the material’s relative density: grey colour is correlated with high blow counts, in the range of medium to very dense sands, whereas reddish brown or dark brown soils are generally indicative of very loose to loose materials.

Screening the results from other areas confirmed the above-mentioned tendencies, which can be therefore used as guidelines for future phases of the project.

No shear testing of such materials is reported. Some results are available for samples of “Atlantic muck” at the Gatún dam site. For load conditions similar to, or slightly higher than those expected in the shallow slope cuts, effective shear strength parameters would range between c’ = 55 kPa, Φ’ = 28° and c’ = 14 kPa, Φ’ = 33°. Considering that the MH soil along the A-1 lineament has greater strength than the typical black muck, presumably tested at Gatún, the first couple of parameters can be adopted as the lower limit of the shear strength.

6.2 GEOTECHNICAL PARAMETERS OF THE BEDROCK

A summary of the RQD values as reported in the drill logs (**Ref.7**) is presented in **Table 2**. Available laboratory tests’ results are summarised in **Tables 3 to 6**. The different rock types were dully sampled and tested. Analysis has been carried out both globally and separately for the A-1 and A-2 series, considering each of the rock types (**Table 4**).

The main characteristics are reminded below.

- *Drill core quality*

Cores commonly hydrate and swell when exposed to air. RQD values are generally high, with more than 60% of the cored length in the very good to excellent range (RQD > 75%). Very poor RQD values (15% of the cored length with RQD < 25%) are concentrated at four locations: TA1-1, TA1C-2, TA1C10 and TA1C-11. Only the latter two boreholes are located within the area of interest for the locks, towards the Gatún Lake end.

- *Index properties*

The high porosity is the most striking feature of the fine-grained tuffaceous and pumiceous rocks, leading to:

- Low unit weight, averaging about 1880 kg/m^3, ranging between 1600 and 2396 kg/m^3. The lower values characterise the fine-grained sediments, largely prevailing over the conglomerates in the rock foundation.
- Moderate (coarse-grained rocks) to high (for fine-grained rocks) water content, in the range 10 to 69%, averaging 33%.

- *Unconfined compressive strength*

The strength values are highly variable but they are all in the low strength range, between 2 to 25 MPa, with an average around 6 MPa. After eliminating eccentric values (25 MPa and 18 MPa), the average value decreased to 5.7 MPa. Separate statistics were done for each lineament and each side, Atlantic and Gatún. They showed roughly similar average results and dispersion of values.

➢ *Elasticity modulus*

Low values, which range between 0.16 and 4.1 GPa, averaging between 1 and 1.3 GPa for the prevailing rock types, i.e. tuffaceous sandstone and sandstone. These values are representative for intact rock matrix of low stiffness.

➢ *Shear strength*

Various laboratory and field shear strength tests were carried out during the 1939 campaign, but test conditions are not reported in the consulted document (Ref. 4). High cohesion and low friction angle values suggest that the pore pressure has not been accounted for, yielding apparent shear strength parameters.

Only one direct shear test and one ring shear tests have been performed by ACP in 2001, on remoulded silty sand derived from tuffaceous sandstone (borehole TA2-4 @ 28.6 - 30.0 m). The tests have been carried out at low shear velocity (drained) and under normal loads consistent with the actual loading under operating conditions. The following characteristics were obtained:

- direct shear: C_{peak} = 21 kPa, Φ_{peak} = 31°
- ring shear: C_{peak} = 3 kPa, Φ_{peak} = 29° and C_{res} = 9 kPa, Φ_{res} = 20°

➢ *Permeability*

A number of about 80 tests were carried out in 9 boreholes (3 of which in or close to the actual locks site: TA1C-5, TA1C-7 and TA1C-9). These tests were performed in 3-m long stages, under effective maximum pressures of 3.7 to 4.6 bars, consistent with expected operating conditions. The results showed that bedrock is generally impervious (< 3 LU). Bypassing and high takes were sporadic and indicate that the rock foundation can be locally affected by fracturing.

➢ *Correlation graphs*

Several correlation graphs have been drafted, in particular the variation of the strength and of the modulus of elasticity with depth, to determine a possible vertical geotechnical zoning. No correlation is visible over the tested depth. In order to retrieve potential guidelines for the classification of the rock foundation, relationships between the geomechanical parameters and the physical properties were also looked for. Except for the straightforward relationships, they mainly showed the lack of correlation. Examples are given in **Figures 2 to 5.**

6.3 GEOTECHNICAL ZONING

In the upper part of the geological profile, two types of soil were differentiated by distinct geotechnical properties: MH type and SC type. The two types of materials can be interlayered, but the MH soils are dominant. In addition, the limit between highly weathered bedrock and overburden is rather transitional. It is consequently reasonable to adopt for the entire upper unit the characteristics of the less resistant, MH type material.

As far as the fresh bedrock is concerned, no significant correlation of geotechnical parameters with depth could be revealed (**Figures 2 and 5**). Furthermore, the most significant parameters, namely compressive strength and modulus of elasticity show high dispersion. Based on these considerations, the entire rock mass underneath soil and highly weathered bedrock will be defined as a single geotechnical unit

Consequently, two geotechnical units will be distinguished in the foundation of the locks: (i) soil cover and highly weathered rock and (ii) sound bedrock.

7 Engineering geology

The locks will be located in the Gatún side area, characterised by relatively thin soil cover and lack of black muck.

The geological setting and the geotechnical parameters are expected to vary globally in the N - S direction. Consequently, the geological conditions will not change significantly along the new alignment (38 m away from the original one) and geotechnical characteristics can be safely extrapolated from the existing studies along the initial axis. Likewise, available results from the investigation of the bedrock along the A-2 alignment are considered representative and taken into account in the geotechnical analysis.

The rockmass in the foundation of the locks can be characterised as mainly massive, affected by only one dominant, widely spaced joint set (the bedding) and local, subordinate diffuse jointing. Local low RQD values and sporadic high water takes during permeability testing indicate that the bedrock is locally affected by fracturing.

At the presently adopted levels for the three steps, the rock foundation is expected to be adequate and characterised by similar strength. In particular, no significant soft layer was identified to depth. The geological sections C-C and F-F show the different rock types that will form the rock foundation of the locks. According to the previous considerations, they will all be characterised as a single geotechnical unit. The recommended strength and deformability parameters of this rockmass are proposed below. The local poor condition rock masses ascertained by low RQD values and high permeability values are not accounted for by these parameters.

7.1 RECOMMENDED PARAMETERS FOR THE ROCK FOUNDATION

Average values for the relevant characteristics are:

- Unit weight: 1880 kg/m^3
- Unconfined compressive strength: 5.7 MPa
- Modulus of elasticity: 1.2 GPa

Considering the results of the plate load tests carried out in 1939, ranging between 10 and 37 MPa, an average compressive strength value of 6 MPa, before applying safety factors, seems reasonable. Finally, a value of 6 MPa / 3 = 2 MPa can be adopted for design purposes.

The average rockmass' modulus of elasticity appears less constrained. At this stage, the average value of 1.2 GPa can be adopted, but a consequent safety factor should be applied.

No reliable shear strength testing results are available. Specific tests on intact rock specimens and/or in situ direct shear tests are necessary for more accurate determination.

The rockmass characteristics were analysed during the site visit, in view of rating (RMR classification system) and estimating its strength and deformability parameters with empirical methods such as the Hoek and Brown failure criterion. However, this rockmass is too massive and classification systems based on the characteristics of discontinuities are not appropriate.

Finally, it is reminded that an allowable bearing capacity of about 2 MPa and a modulus of elasticity of 2200 MPa had been retained for the design in 1939.

7.2 CUT SLOPES

Specific slopes are to be considered in accordance with the characteristics of the two geotechnical units: (i) the sound bedrock and (ii) soil and highly weathered bedrock. In the absence of reliable criteria, due to the lack of specific shear strength testing and site conditions not appropriate for empirical estimations, the following recommended slopes are only indicative. Testing is recommended at the end of this report. The slopes will be in any case modified to fit local structural findings during excavation.

(i) Sound bedrock

Maximum height of cuts in bedrock is about 55 m. Past experience showed that the rocks of the Gatún Formation could stand unsupported, near vertical slopes. In addition, no adverse discontinuities have been identified so far.

The following slopes can be adopted in the locks area:

- the bottom 15 m : 6V / 1H, with horizontal, 4-m wide berm at the top
- above the berm: 3V / 1H

(ii) Weathered rock and residual soil

As seen in Table 1, maximum depth of excavation in these deposits will be in the order of 14 m.

It has been earlier mentioned that shear strength parameters determined on Atlantic muck at the Gatún dam site can be adopted as the lower limit of the shear strength at the new locks site: c' = 55 kPa, Φ' = 28°. Under these circumstances, slopes of 1V / 3H are estimated to be safe.

Specific calculations based on shear strength testing might result in the optimisation of these slopes.

8 Conclusions

The geological setting is relatively simple. It consists of a sedimentary sequence with sub-horizontal dip in very gentle slope towards the Atlantic. No major tectonic accident, nor adverse discontinuities were identified so far.

Two geotechnical units were distinguished: (i) the soil cover and weathered bedrock and (ii) the sound bedrock. Their respective geotechnical parameters were estimated based on the available data. The considerable porosity that characterises all the tested rocks is considered responsible for relatively low compressive strength and elastic modulus values, which in addition are highly variable. Average values, prior to applying safety factors, are 6 MPa for the unconfined compressive strength and 1.2 GPa for the modulus of elasticity.

No relevant shear testing was conducted on materials from the foundation at the actual locks site. Cut slopes are recommended but they need to be refined during the next design phase.

The bedrock appears as generally massive, affected by one main family of widely spaced discontinuities (the bedding). The massive character prevents empirical estimation of the rock mass strength parameters starting from rockmass rating that relies on characterisation of discontinuities. An additional, relatively light program of investigations is recommended herein. It is partly justified by shifting of the locks axis, and partly by the necessity to provide accurate input geotechnical parameters for the advanced design.

9 Recommendations

Future investigations are recommended to refine the site characterization for the benefit of the design.

The natural site conditions are not expected to vary considerably. However, shifting of the axis will need a minimum of investigations to confirm the interpreted conditions along the final lineament:

- Refraction seismic profiling for (i) defining the weathering sequence and (ii) refining the rock mass characteristics; it is reminded that the deformability of the rock foundation is poorly constrained and seismic data can bring additional elements to better define this parameter.

- Electric panel to check possible fault on thickening weathered deposits around TA1C-9

- Core drilling for calibration of the geophysical profiles and sampling

- Triaxial testing on selected rock and soil samples

Given the local site conditions (flat topography and abundant drilling data) geophysical surveys are expected to supply accurate terrain models and to contribute to the optimisation of the layout and design rapidly and at low cost.

10 Tables

Table 1. Third set of locks, Atlantic side - summary of lithology and weathering profile in drillholes series "TA1" and TAC1" (selected A-1 alternative)

Drillhole	Overburden				Weathered bedrock	Sound bedrock
	Fill	Clay/Silt/ Saprosoil	Transition	Muck		
TA1-1		0 - 6.5			?	6.5 - 40
TA1-2			0 - 1.5	1.5 - 13.6	13.6 - 15.45	15.45 - 35
TA1-3		0 - 6.0			6.0 - 6.7	6.7 - 40
TA1-4	0 - 3.0	3.0 - 11.7			11.7 - 14.6	14.6 - 51
TA1-5		0 - 1.7			1.7 - 2.4	2.4 - 51
TA1C-1	0 - 0.3	0.3 - 2.0			2.0 - 3.3	3.3 - 50
TA1C-2	0 - 2.0		2.0 - 8.0	8.0 - 9.5	9.5 - 12.0	12.0 - 35
TA1C-3	0 - 4.2			4.2 - 35.0		
TA1C-4	0 - 2.0	2.0 - 8.0		8.0 - 14.5		
		14.5 - 18.0			18.0 - 19.0	19.0 - 40
TA1C-5	0 - 1.5			1.5 - 9.6	9.6 - 10.9	10.9 - 40
TA1C-6	0 - 7.0	7.0 - 9.1			9.1 - 10.7	10.7 - 50
TA1C-7	0 - 3.25				?	3.25 - 40
TA1C-8					?	0 (?) - 50
TA1C-9		0 - 11.2			11.2 - 13.4 (?)	13.4 (?) - 40
TA1C-10		0 - 1.0			1.0 - 6.1	6.1 - 56
TA1C-11		0 - 1.3			1.3 - 2.4	2.4 - 80

Table 2. Third set of locks, Atlantic side - RQD summary for drillholes series "TA1" and TAC1" (selected A-1 alternative)

Drillhole	Relevant length for RQD (m)	RQD									
		<25%		25 - 50%		50 - 75%		75 - 90%		> 90%	
TA1-1	34	6 - 13	21%	13 - 18 35 - 38	24%		0%	22 - 24 26 - 29	15%	18 - 22 24 - 26 29 - 35 38 - 40	40%
TA1-2	20		0%	15 - 19	20%	19 - 26	35%	26 - 28	10%	28 - 35	35%
TA1-3	34		0%		0%	17 - 20	9%	6 - 9	9%	9 - 17 20 - 40	82%
TA1-4	37		0%	21 - 23	5%	15 - 21 23 - 27	27%	27 - 51	65%	14 - 15	3%
TA1-5	49		0%	2 - 5 23 - 26	12%	26 - 32 35 - 38 44 - 47	24%	6 - 23	35%	5 - 6 32 - 35 38 - 44 47 - 51	29%
TA1C-1	47		0%	25 - 27 47 - 50	11%	3 - 9	12%	9 - 25 37 - 43	47%	27 - 37 43 - 47	30%
TA1C-2	23	12 - 35	100%		0%		0%		0%		0%
TA1C-3	*All in overburden*										
TA1C-4	21		0%		0%	19 - 24	24%			24 - 40	76%
TA1C-5	29		0%		0%	11 - 13 19 - 22	17%	17 - 19	7%	13 - 17 22 - 40	76%
TA1C-6	40	10 - 12	5%	25 - 26	3%	12 - 14	5%	16 - 25	22%	14 - 16 26 - 50	65%
TA1C-7	37		0%		0%	23 - 30	19%	9 - 23 32 - 38	54%	3 - 9 30 - 32 38 - 40	27%
TA1C-8	50		0%	15 - 24 43 - 45	22%		0%	0 - 15 24 - 26	34%	26 - 43 45 - 50	44%
TA1C-9	27		0%		0%	23 - 25	7%		0%	13 - 23 25 - 40	93%
TA1C-10	53	20 - 26 28 - 30 34 - 39	25%	15 - 18	6%	3 - 9 18 - 20 30 - 34 46 - 47 51 - 53 54 - 56	32%	44 - 46	4%	9 - 15 26 - 28 39 - 44 47 - 51 53 - 54	33%
TA1C-11	78	19 - 41 51 - 53 54 - 56 58 - 72 77 - 80	55%	2 - 3 8 - 10 16 - 18 41 - 51 72 - 77	26%	3 - 8 56 - 58	9%	10 - 16 18 - 19	9%	53 - 54	1%
TOTAL	***579***	*88*	***15%***	*60*	***10%***	*83*	***14%***	*130*	***23%***	*218*	***38%***

Pumiceous sandstone, tuffaceous sandstone and tuff generally yield greater RQD (e.g. TA1C-4, 5, 8)

Table 3. Third set of locks Atlantic side - summary of tests results on rocks from the Gatún Formation (ACP 2001 - 2002)

(1/3)

No.	Drillhole	Depth	Unit weight	Elastic modulus	UCS	Specific gravity	Water content	Rock type
			kg/m^3	E (MPa)	σ_c (MPa)		%	
1	TA1-1	12.10 / 12.40	1832	646	5.0	2.00	36	Tuffaceous sandstone
2	TA1-1	12.10 / 12.40	1845	733	5.8	2.00	37	Tuffaceous sandstone
3	TA1-1	24.26 / 24.66	1883	841	4.4	2.10	33	Tuffaceous sandstone
4	TA1-1	24.26 / 24.66	1885	590	5.2	2.10	34	Tuffaceous sandstone
5	TA1-2	18.85 / 19.03	1805	1385	5.4	2.10	40	Sandstone
6	TA1-2	32.89 / 33.17	2015	755	3.0	2.10	25	Sandstone
7	TA1-3	12.24 / 12.69	1778	1681	6.5	2.31	43	Tuffaceous sandstone
8	TA1-3	12.24 / 12.69	1785	1865	6.5	2.31	40	Tuffaceous sandstone
9	TA1-3	21.43 / 21.95	1859	1253	4.5	2.30	31	Tuffaceous sandstone
10	TA1-3	21.43 / 21.95	1881	814	3.3	2.30	34	Tuffaceous sandstone
11	TA1-5	2.81 / 3.04	1900	942	2.8	2.00	24	Conglomeratic sandstone
12	TA1-5	9.90 / 10.18	1822	927	6.4	2.00	35	Tuffaceous sandstone
13	TA1-5	23.85 / 24.35	1836	1008	6.1	2.10	35	Tuffaceous sandstone
14	TA1-5	23.85 / 24.35	1872	789	6.2	2.10	35	Tuffaceous sandstone
15	TA1-5	38.85 / 39.42	1969	754	3.5	2.10	31	Tuffaceous sandstone
16	TA1-5	38.85 / 39.42	1942	867	4.3	2.10	30	Tuffaceous sandstone
17	TA2-1	25.20 / 25.54	2111	1162	3.9	2.00	16	Tuffaceous sandstone
18	TA2-1	27.09 / 27.49	1726	1654	5.1	2.00	43	Tuffaceous sandstone
19	TA2-1	27.09 / 27.49	1811	1955	7.7	2.00	41	Tuffaceous sandstone
20	TA2-3	26.15 / 26.55	1851	960	5.6	2.00	37	Tuffaceous sandstone
21	TA2-3	26.15 / 26.55	1829	919	5.1	2.00	38	Tuffaceous sandstone
22	TA2-3	29.60 / 30.00	2027	1395	5.7	2.10	25	Tuffaceous sandstone
23	TA2-3	29.60 / 30.00	2159	1061	5.5	2.10	25	Tuffaceous sandstone
24	TA2-4	9.70 / 10.00	1803	1638	8.6	2.30	40	Tuffaceous sandstone
25	TA2-4	19.63 / 20.03	1871	2987	10.8	2.10	35	Tuffaceous sandstone
26	TA2-4	19.63 / 20.03	1889	2586	9.2	2.10	35	Tuffaceous sandstone
27	TA2-4	28.60 / 29.18	1982	1837	5.0	2.10	28	Tuffaceous sandstone
28	TA2-4	28.60 / 29.18	2094	1242	6.0	2.10	28	Tuffaceous sandstone
29	TA2-6	12.45 / 12.80	1959	802	4.6	2.00	28	Tuffaceous sandstone
30	TA2-6	12.45 / 12.80	1953	988	5.4	2.00	27	Tuffaceous sandstone
31	TA2-6	17.14 / 17.55	1970	1918	4.4	2.00	23	Conglomerate
32	TA2-6	17.14 / 17.55	1936	1385	5.1	2.00	23	Conglomerate
33	TA2-7	7.71 / 8.14	1875	1069	6.1	2.10	31	Sandstone
34	TA2-7	7.71 / 8.14	1856	1189	6.0	2.10	31	Sandstone
35	TA2-7	10.04 / 10.45	2096	1844	7.5	2.10	19	Sandstone
36	TA2-7	10.04 / 10.45	1939	1072	4.7	2.10	26	Sandstone
37	TA2-7	12.78 / 13.06	2088	1579	4.2	2.10	20	Conglomeratic sandstone
38	TA2-7	22.20 / 22.71	1688	1522	4.3	1.76	42	Pumiceous sandstone
39	TA2-7	22.20 / 22.71	1675	1941	4.6	1.76	47	Pumiceous sandstone
40	TA2-7	25.55 / 25.89	1683	1378	4.0	1.76	47	Pumiceous sandstone
41	TA2-7	28.54 / 29.15	1482	1148	4.4	1.76	43	Pumiceous sandstone
42	TA2-7	36.30 / 36.72	2151	3229	6.7	2.20	15	Conglomeratic sandstone
43	TA2-7	36.30 / 36.72	2133	2100	3.4	2.20	15	Conglomeratic sandstone
44	TA2-7	39.43 / 40.00	1911	1772	8.0	2.20	33	Sandstone
45	TA2-7	43.58 / 44.00	1875	782	3.4	2.40	35	Conglomeratic sandstone
46	TA2-7	43.58 / 44.00	1884	1008	3.6	2.40	37	Conglomeratic sandstone

Table 3. Third set of locks Atlantic side - summary of tests results on rocks from the Gatún Formation (ACP 2001 - 2002)

(2/3)

No.	Drillhole	Depth	Unit weight kg/m^3	Elastic modulus E (MPa)	UCS σ_c (MPa)	Specific gravity	Water content %	Rock type
47	TA1C-1	14.11 / 14.40	1890	537	5.0	2.00	31	Sandstone
48	TA1C-1	14.11 / 14.40	1903	828	10.0	2.00	29	Sandstone
49	TA1C-1	15.23 / 15.54	1899	409	4.0	2.00	33	Sandstone
50	TA1C-1	15.23 / 15.54	1874	552	5.0	2.00	33	Sandstone
51	TA1C-2	19.74 / 19.87	2396	3241	25.0	2.40	10	Sandstone
52	TA1C-2	34.64 / 34.85	1990	1583	11.0	2.40	33	Sandstone
53	TA1C-4	20.68 / 21.00	2049	774	5.0	2.00	24	Sandstone
54	TA1C-4	20.68 / 21.00	1978	784	5.0	2.00	26	Sandstone
55	TA1C-5	47.20 / 47.45	1677	1533	10.0	2.00	44	Sandstone
56	TA1C-5	47.20 / 47.45	1678	1724	11.0	2.00	36	Sandstone
57	TA1C-6	43.18 / 43.40	1787	2638	15.0	2.00	34	Sandstone
58	TA1C-7	28.47 / 28.70	1905	517	3.0	2.10	28	Sandstone
59	TA1C-7	34.10 / 34.36	1745	1011	5.0	2.10	34	Sandstone
60	TA1C-7	34.10 / 34.36	1746	876	5.0	2.10	41	Sandstone
61	TA1C-8	25.81 / 26.09	1811	580	4.0	2.00	31	Sandstone
62	TA1C-8	25.81 / 26.09	1784	724	5.0	2.00	40	Sandstone
63	TA1C-8	40.57 / 40.90	2043	1164	7.0	2.30	14	Conglomerate
64	TA1C-8	40.57 / 40.90	2184	4127	18.0	2.30	13	Conglomerate
65	TA1C-9	20.27 / 20.50	1621	907	6.0	2.00	54	Sandstone
66	TA1C-9	23.02 / 23.28	1641	2483	13.0	2.00	52	Tuff
67	TA1C-9	23.02 / 23.28	1713	3065	16.0	2.00	47	Tuff
68	TA1C-10	26.04 / 26.24	1668	595	5.0	2.00	54	Sandstone
69	TA1C-10	32.91 / 33.15	1926	711	2.0	2.00	30	Sandstone
70	TA1C-11	28.20 / 28.44	1600	852	6.0	2.00	56	Sandstone
71	TA1C-11	74.46 / 74.65	2118	567	3.0	2.00	17	Sandstone
72	TA2C-1	29.83 / 30.14	1932	1705	9.0	2.00	30	Sandstone
73	TA2C-1	29.83 / 30.14	1916	2355	9.0	2.00	35	Sandstone
74	TA2C-2	29.71 / 29.98	1894	401	3.0	2.00	28	Sandstone
75	TA2C-2	29.71 / 29.98	1966	1314	4.0	2.00	26	Sandstone
76	TA2C-3	34.24 / 34.53	1817	544	3.0	2.00	33	Sandstone
77	TA2C-3	34.24 / 34.53	1929	743	5.0	2.00	28	Sandstone
78	TA2C-3	38.97 / 39.35	1901	840	5.0	2.00	30	Sandstone
79	TA2C-3	38.97 / 39.35	1898	1008	5.0	2.00	29	Sandstone
80	TA2C-4	25.31 / 25.55	1956	2299	5.0	2.10	35	Sandstone
81	TA2C-4	29.26 / 29.45	2070	726	5.0	2.20	23	Microconglomerate
82	TA2C-4	32.86 / 33.11	1833	2483	7.0	2.30	36	Conglomerate
83	TA2C-4	32.86 / 33.11	1888	1910	6.0	2.30	32	Conglomerate
84	TA2C-5	31.24 / 31.52	1794	1552	3.0	2.00	35	Sandstone
85	TA2C-5	31.24 / 31.52	1832	242	2.0	2.00	33	Sandstone
86	TA2C-5	49.88 / 50.20	1679	1452	8.0	2.00	48	Sandstone
87	TA2C-5	49.88 / 50.20	1703	2420	9.0	2.00	46	Sandstone
88	TA2C-6	35.53 / 35.76	1842	872	4.0	2.30	34	Conglomerate
89	TA2C-7	22.54 / 22.79	2033	167	2.0	2.20	24	Microconglomerate
90	TA2C-8	13.25 / 13.51	1924	2793	13.0	2.00	31	Siltstone
91	TA2C-8	13.25 / 13.51	1897	1264	7.0	2.00	38	Siltstone
92	TA2C-8	15.88 / 16.18	1826	1586	8.0	2.00	41	Siltstone
93	TA2C-8	15.88 / 16.18	1807	1505	8.0	2.00	39	Siltstone
94	TA2C-8	26.80 / 27.07	2071	616	3.0	2.30	16	Conglomerate
95	TA2C-9	26.99 / 27.24	1864	956	5.0	2.30	29	Conglomerate
96	TA2C-10	22.90 / 23.14	1733	552	3.0	2.20	43	Microconglomerate
97	TA2C-11	16.66 / 17.10	1905	759	5.0	2.00	30	Sandstone
98	TA2C-11	16.66 / 17.10	1915	615	5.0	2.00	30	Sandstone
99	TA2C-11	16.66 / 17.10	1910	575	4.0	2.00	31	Sandstone
100	TA2C-12	49.09 / 49.41	2248	2000	9.0	2.20	11	Conglomerate
101	TA2C-12	49.09 / 49.41	2209	1793	10.0	2.20	20	Conglomerate
102	TA2C-13	7.91 / 8.25	1817	586	6.0	2.00	35	Sandstone
103	TA2C-13	7.91 / 8.25	1839	750	5.0	2.00	38	Sandstone

Table 3. Third set of locks Atlantic side - summary of tests results on rocks from the Gatún Formation (ACP 2001 - 2002)

(3/3)

No.	Drillhole	Depth	Unit weight	Elastic modulus	UCS	Specific gravity	Water content	Rock type
			kg/m^3	E (MPa)	σ_c (MPa)		%	
104	TA2C-15	31.21 / 31.58	1501	2000	4.0	2.00	52	Sandstone
105	TA2C-15	31.21 / 31.58	1552	1897	8.0	2.00	63	Sandstone
106	TA2C-16	20.69 / 20.95	1558	1138	5.0	1.90	56	Tuff
107	TA2C-16	29.90 / 30.17	1934	1667	8.0	2.00	26	Sandstone
108	TA2C-16	29.90 / 30.17	1928	1517	9.0	2.00	26	Sandstone
109	TA2C-17	16.65 / 16.97	1873	874	5.0	2.00	69	Sandstone
110	TA2C-17	16.65 / 16.97	1890	690	4.0	2.00	32	Sandstone

Table 4. Third set of locks Atlantic side -Summary of the geotechnical characteristics of rocks in the Gatún Formation (ACP, 2001 - 2002)

Rock type	Axis	No. of values	Unit weight kg/m^3				Specific gravity				Water content %				Elastic modulus E (MPa)				UCS Sc (MPa)			
			Average	Min	Max	St Dev	Average	Min	Max	St Dev	Average	Min	Max	St Dev	Average	Min	Max	St Dev	Average	Min	Max	St Dev
Tuff	A-1	2	1677	1641	1713	nr	2	2	2	nr	50	47	52	nr	2774	2483	3065	nr	14.5	13	16	nr
Tuff	A-2	1	1558	nr	nr	nr	1.9	nr	nr	nr	56	nr	nr	nr	1138	nr	nr	nr	5	nr	nr	nr
Tuff	A-1 & A-2	3	1637	1558	1713	nr	1.97	1.9	2	nr	52	47	56	nr	2229	1138	3065	nr	11.3	5	16	nr
Siltstone	A-2	4	1864	1807	1924	nr	2	2	2	nr	37	31	41	nr	1787	1264	2793	nr	9	7	13	nr
Sandstone	A-1	23	1864	1600	2396	182	2.06	2	2.4	0.12	34	10	56	11	1047	409	3241	703	6.9	2	25	5.1
Sandstone	A-2	29	1861	1501	2096	121	2.02	2	2.2	0.05	35	19	69	11	1233	242	2420	624	5.7	2	9	2.1
Sandstone	A-1 & A-2	52	1862	1501	2396	149	2.04	2	2.4	0.09	34	10	69	11	1151	242	3241	660	6.2	2	25	3.7
TfSandst	A-1	13	1861	1778	1969	55	2.14	2	2.31	0.12	35	30	43	4	982	590	1865	390	5.2	4.3	6.5	1.1
TfSandst	A-2	14	1933	1726	2159	130	2.06	2	2.3	0.08	32	16	43	8	1513	802	2987	651	6.3	3.9	10.8	2
TfSandst	A-1 & A-2	27	1898	1726	2159	106	2.1	2	2.31	0.11	33	16	43	6	1258	590	2987	596	5.8	3.3	10.8	1.7
PumSandst	A-2	4	1632	1482	1688	nr	1.76	1.76	1.76	nr	45	42	47	nr	1497	1148	1941	nr	4.3	4	4.6	nr
Microconglomerate	A-2	3	1945	1733	2070	nr	2.2	2.2	2.2	nr	30	23	43	nr	482	167	726	nr	3.3	2	5	nr
CglSandst	A-1	1	1900	nr	nr	nr	2	nr	nr	nr	24	nr	nr	nr	942	nr	nr	nr	2.8	nr	nr	nr
CglSandst	A-2	5	2026	1875	2151	136	2.26	2.1	2.4	0.13	24	15	37	11	1740	782	3229	978	4.3	3.4	6.7	1.4
CglSandst	A-1 & A-2	6	2005	1875	2151	132	2.22	2	2.4	0.16	24	15	37	10	1607	782	3229	933	4	2.8	6.7	1.4
Conglomerate	A-1	2	2114	2043	2184	nr	2.3	2.3	2.3	nr	14	13	14	nr	2646	1164	4127	nr	12.5	7	18	nr
Conglomerate	A-2	9	1985	1833	2248	157	2.21	2	2.3	0.13	25	11	36	8	1548	616	2483	623	5.9	3	10	2.3
Conglomerate	A-1 & A-2	11	2008	1833	2248	153	2.23	2	2.3	0.12	23	11	36	9	1748	616	4127	973	7.1	3	18	4.2
All	A-1	41	1867	1600	2396	156	2.09	2	2.4	0.13	34	10	56	10	1186	409	4127	831	6.9	2	25	4.7
All	A-2	69	1890	1462	2248	154	2.06	1.76	2.4	0.14	33	11	69	11	1381	167	3229	667	5.7	2	13	2.2
All	A-1 & A-2	110	1881	1482	2396	155	2.07	1.76	2.4	0.13	33	10	69	11	1308	167	4127	735	6.2	2	25	3.4

nr = not relevant, for low number of tests

Table 5. Third set of locks Atlantic side - Summary of bearing tests on rocks of the Gatún Formation

Source	Type of test	Number of tests	Bearing capacity (MPa)	Modulus of elasticity (MPa)	Comments
Report 1943	Field plate load	na, >5	16.1 - 37.7	2585	Average values of field tests, in text, p.19. 7-, 16- and 40-inches-diameter plates.
			10.2 - 12.9		Yellow-green sandstone, 7- and 16-inch-diameter plate (Figure 5.23)
			12.7 - 13.1		Gray sandstone, 7-inch-diameter plate (Figure 5.24)
			9.9		Tuff (Figure 5.25)
	Lab plate load	na	12.0 - 12.2		3-in-diameter plate, 10-inch-diameter sample. Confined by grouting in cylindrical steel container.

Table 6. Third set of locks Atlantic side - Summary of shear strength tests results on rocks of the Gatún Formation

Source	Type of test	Number of tests	Shear strength				Comments
			C peak (MPa)	Φ peak	C res (MPa)	Φ res	
Report 1943 (Ref.4)	Field direct shear	na	0.41	38°			Smooth surface (joint concrete block / rock), on dark gray sandstone
			0.76	30°			Grooved surface - more representative, recommended in 1943 for the wall-lock design
	Lab triaxial and unconfined compression tests *	na	1.18	23.4°			Yellow-green sandstone
			1.7	17°			Gray sandstone
			1.7	15,6°			Average lab tests, used for design
ACP 2001 - 2002	Direct shear	1	0.021	31°			Remolded tuffaceous sandstone Normal stress = 50 - 800 kPa
	Ring shear	1	0.003	29°	0.009	20°	

* Test conditions are not reported in the consulted document. High cohesion and low friction angle values suggest that the pore pressure has not been accounted for.

11 Pictures

Pictures 1 and 2

12 Figures

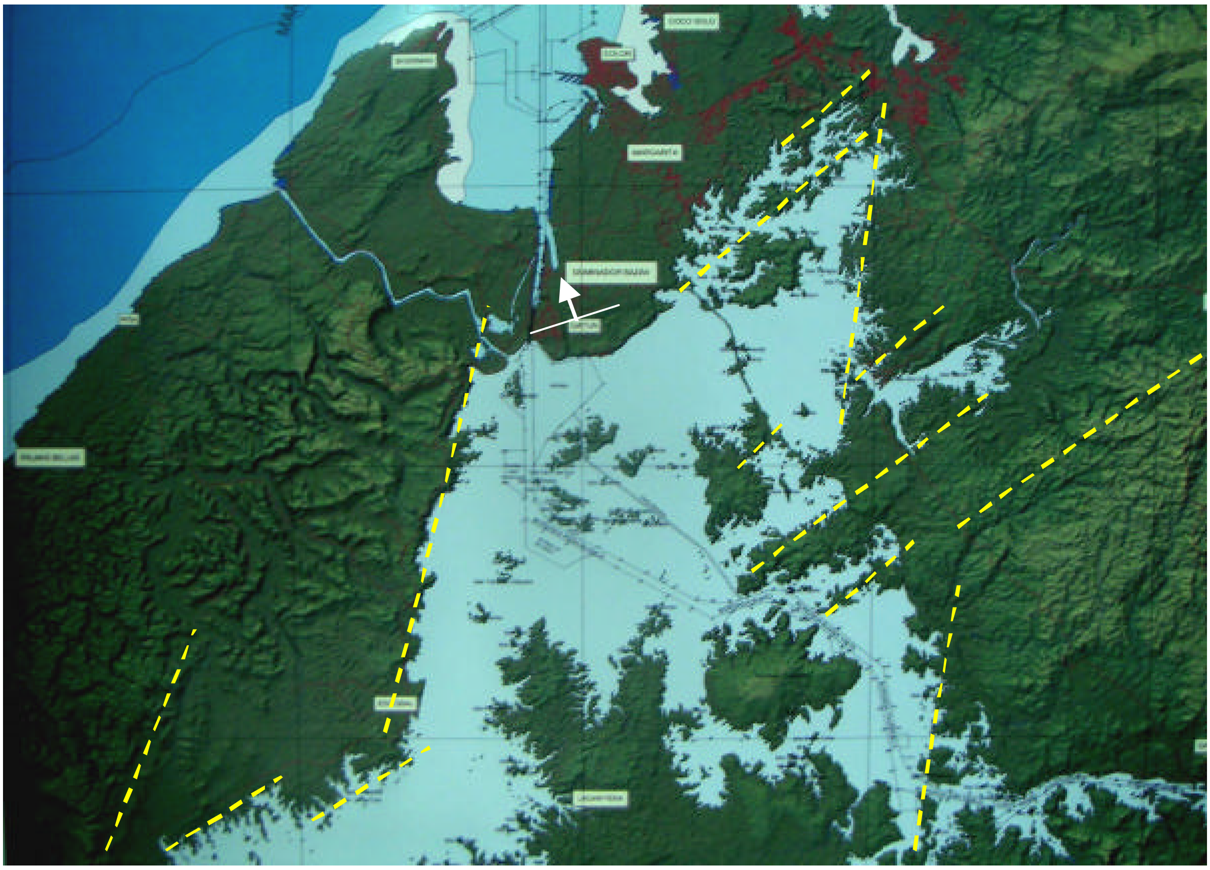


Figure 1

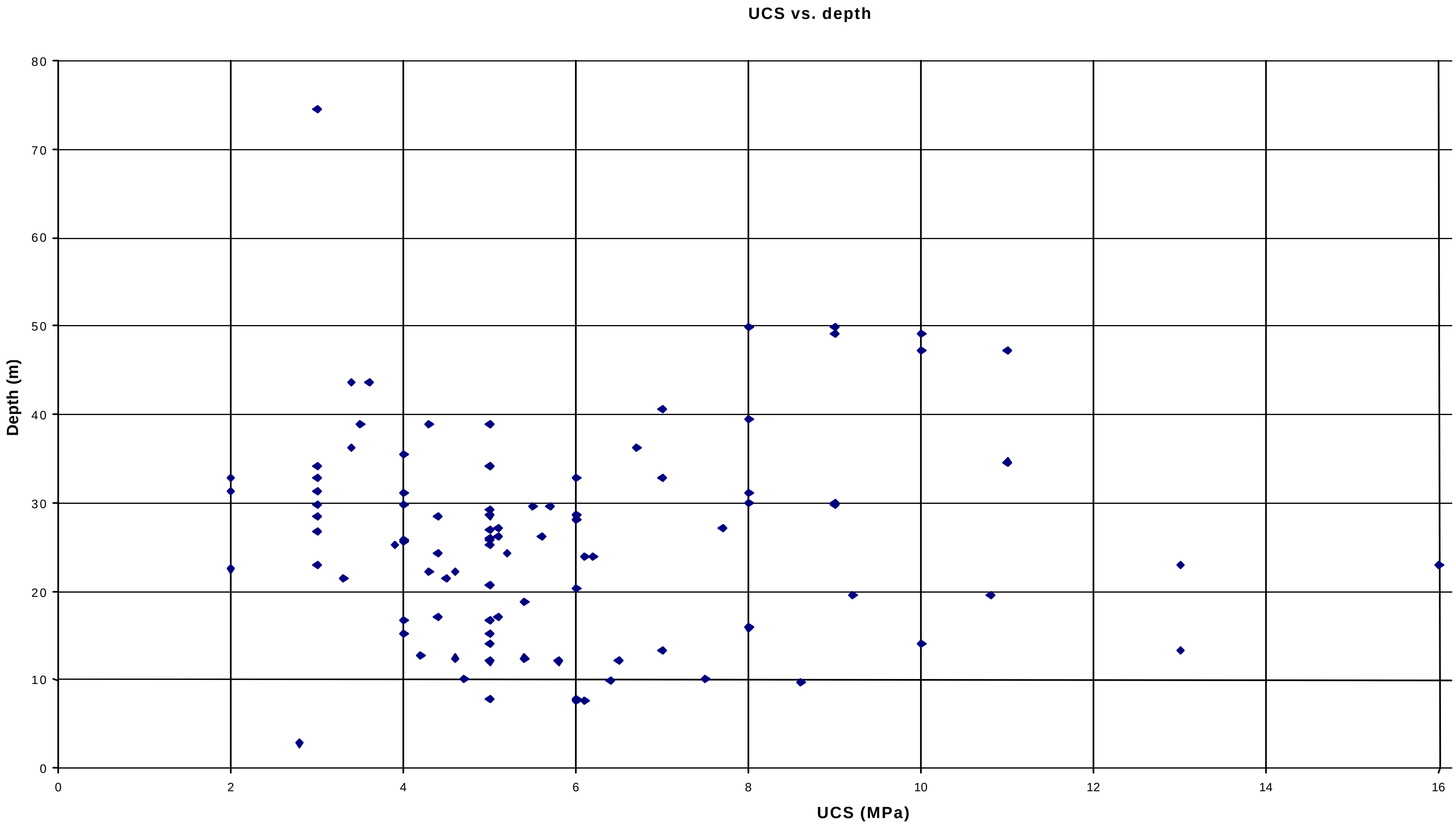

UCS vs. depth
Depth (m)
UCS (MPa)
0
10
20
30
40
50
60
70
80
0
2
4
6
8
10
12
14
16

Figure 2

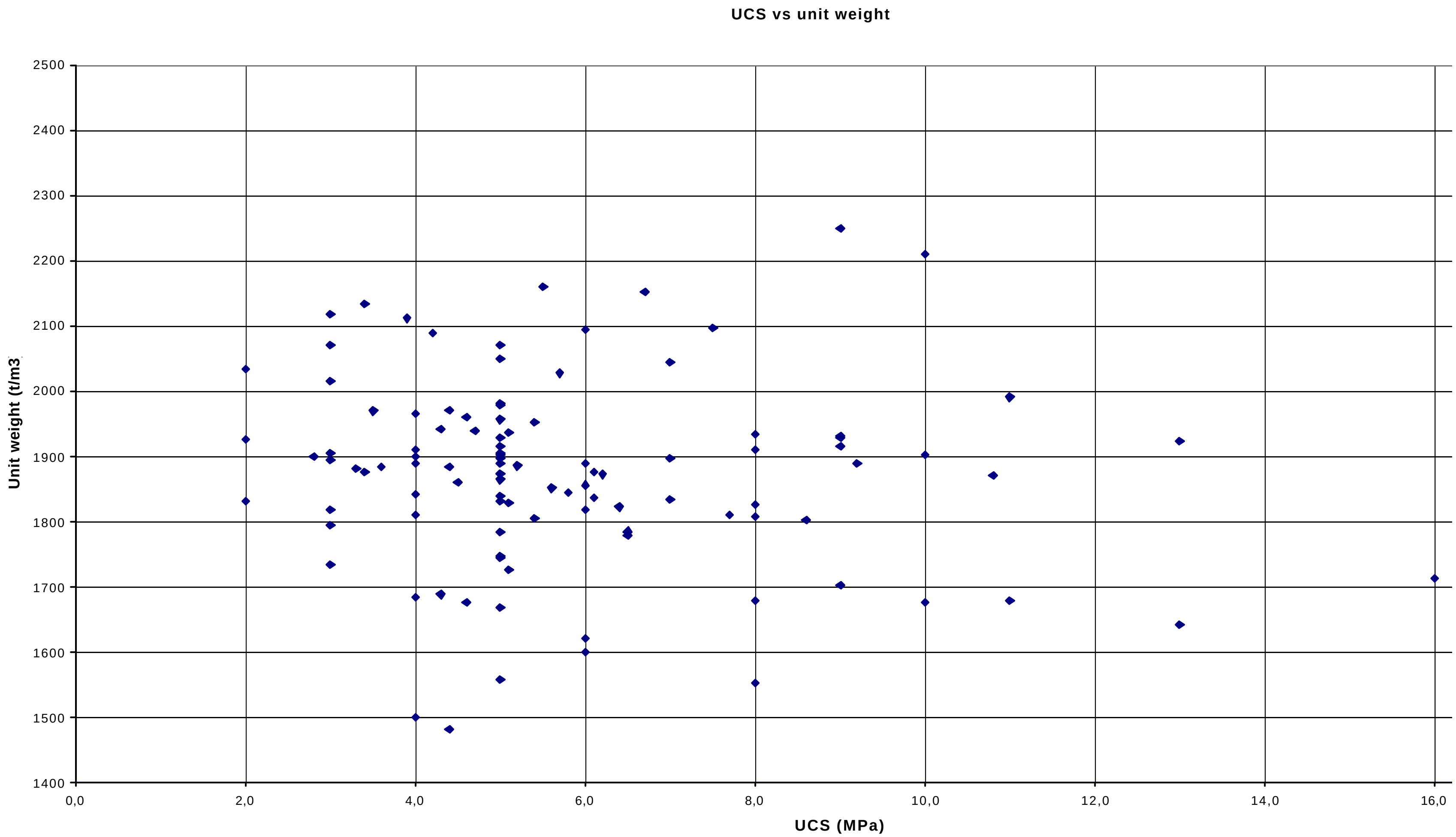

UCS vs unit weight
Unit weight (t/m3)
2500
2400
2300
2200
2100
2000
1900
1800
1700
1600
1500
1400
0,0
2,0
4,0
6,0
8,0
10,0
12,0
14,0
16,0
UCS (MPa)

Figure 3

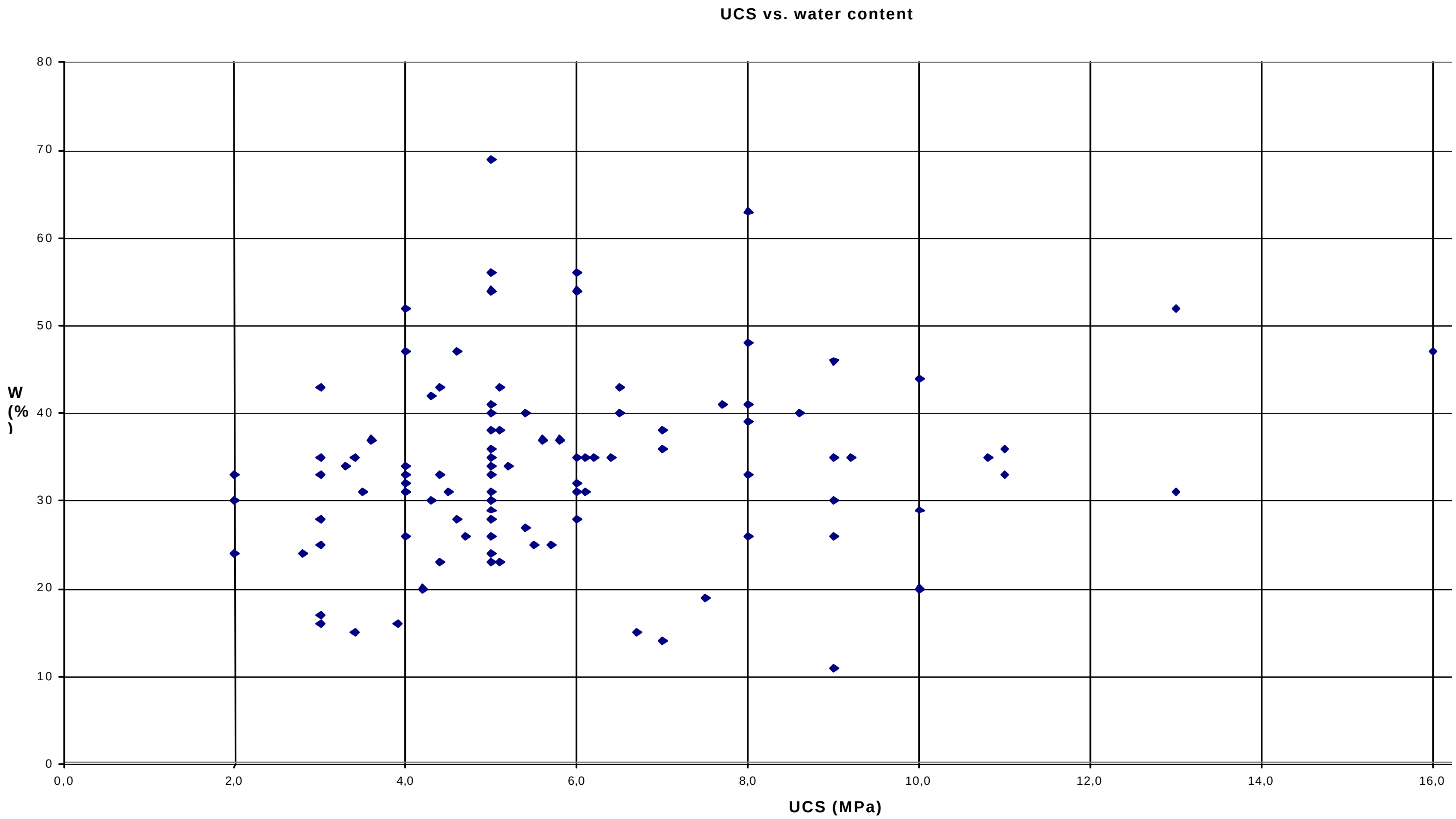

UCS vs. water content
W (%)
80
70
60
50
40
30
20
10
0
0,0
2,0
4,0
6,0
8,0
10,0
12,0
14,0
16,0
UCS (MPa)

Figure 4

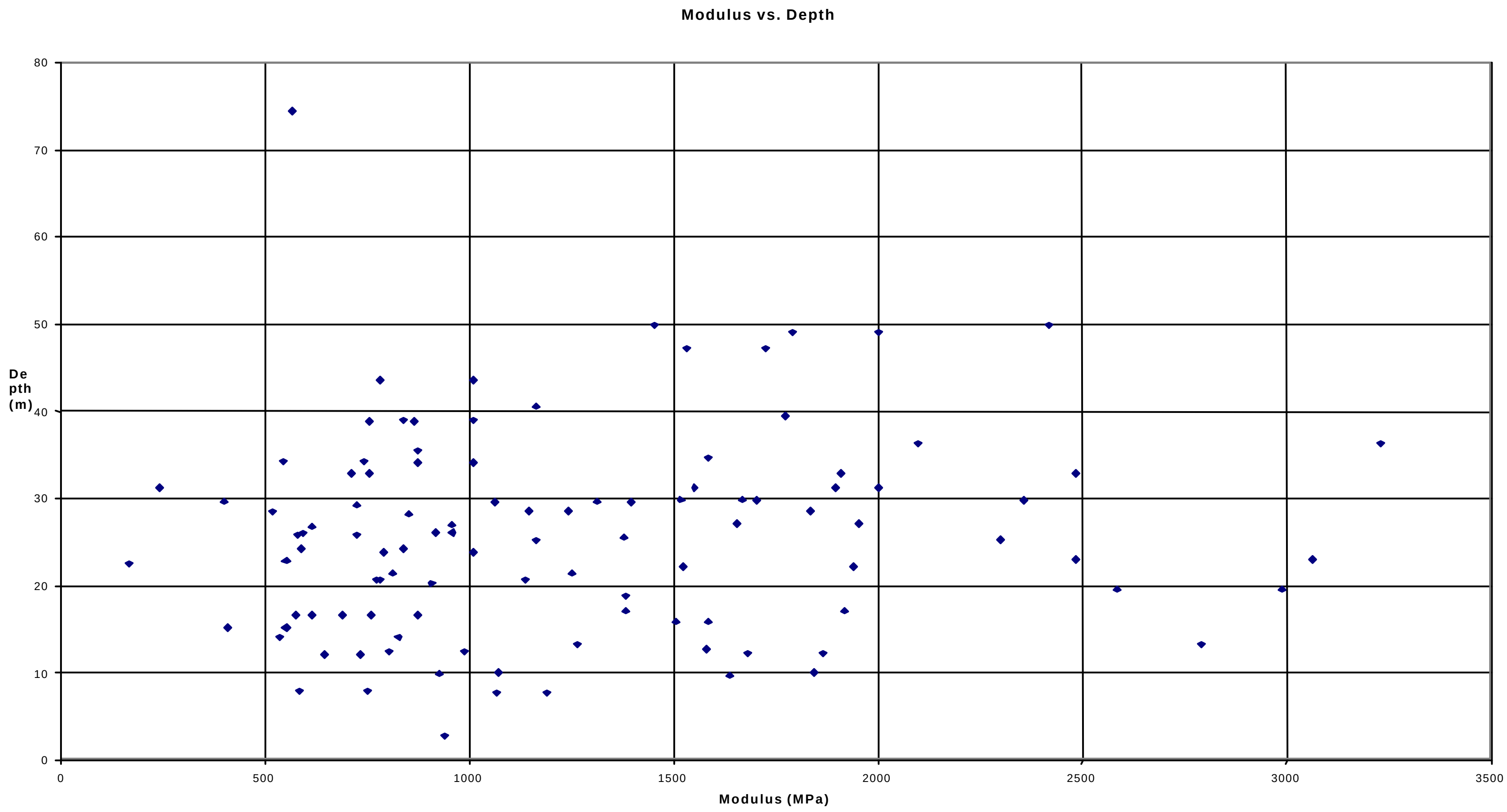

Modulus vs. Depth
Depth (m)
80
70
60
50
40
30
20
10
0
0
500
1000
1500
2000
2500
3000
3500
Modulus (MPa)

Figure 5

CPP Consorcio Post-Panamax

Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual Design - Contract SAA-143351

ATLANTIC LOCKS HARMONIZATION

Task A4a – LOCK SITING

Rev 0

in association with

1 Alignment

An optimized alignment has been worked out by ACP and is shown on drawing D3-0-200. This alignment takes advantage of the existing excavation, and as a consequence, excavation volumes are minimized. It can be concluded that the retained alignment is the one that requires the lowest excavation volume as far as the lock structure is concerned.

From the nautical point of view there are no special considerations to be made for this specific alignment:

- at the Atlantic entrance a straight access channel will allow to reach the lock structure without special maneuverings, the lock entrance/exit is also sufficiently far away from the intersection with the existing access to the Gatun locks (+/-3800m)
- at the Gatun Lake side, the lock entrance is situated close to the lake and should not cause any nautical access problem.

As explained in the geological report (Task 2 – Design Criteria – Geology / Geo-technical), two main types of foundation substratum are recognized in the lock area :

- Gatun rock formation;
- Atlantic Muck.

The latter being a weak soil type, not suitable to bear a gravity type structure, it was self-evident to locate the entire new lock structure in the Gatun rock formation. This lock siting is described in next chapter.

2 Lock Siting

As described above, the lock siting was determined according to the optimum position along the alignment considering nautical accessibility, minimum excavation and favorable soil conditions.
Especially the geo-technical conditions allowed to locate the lock structure entirely in the Gatun Rock formation.

As far as the entrance walls are concerned, following options have been retained:

- at the Gatun lake side, there is sufficient room in the Lake for safe mooring of waiting vessels without requiring a costly entrance wall (such as foreseen at the Pacific locks). The entrance structure has been minimized as indicated on the drawing D3-0-200. Whenever a fourth lane would be constructed, the two lines can be separated more efficiently by means of the installation of dolphins in the center line.
- at the Atlantic Ocean side, the lock entrance connects with the access channel. It requires a mooring facility for a vessel which cannot enter the locks for whatever reason. This entrance wall is situated partly in Gatun Rock formation and partly in the Atlantic Muck. It is suggested to construct a gravity type wall in the rock formation, while the remaining length in the weak subsoil has been equipped with flexible dolphins. These dolphins consist of steel tubular piles, which can be driven into the soil by means of heavy driving equipment. In this way one avoids to make deep excavations in difficult subsoil conditions. It should be noted that flexible dolphins are very suitable in these specific subsoil conditions, but not in hard rock as they would require expensive installation methods and would not work as efficient due to the very high E-modulus of the rock as compared to soil.

The water saving basins are located at the west side of the lock structure, as the future fourth lane is supposed to be constructed at the east side. The excavation volumes required for the construction of the nine water saving basins is rather high (especially when comparing with the Pacific side), this is due to the fact that the actual level of the site is rather elevated (above +30.00mPLD).
If it should be considered to locate the water saving basins at the east side of the new lock structure, the excavations would certainly be much higher, as the levels at this side vary from +30.0 to +90.00mPLD.
However, although the proposed siting seems the most economic as far as excavation in this phase is concerned, it should be pointed out that the construction of the fourth lane at the east side will require far more excavation.

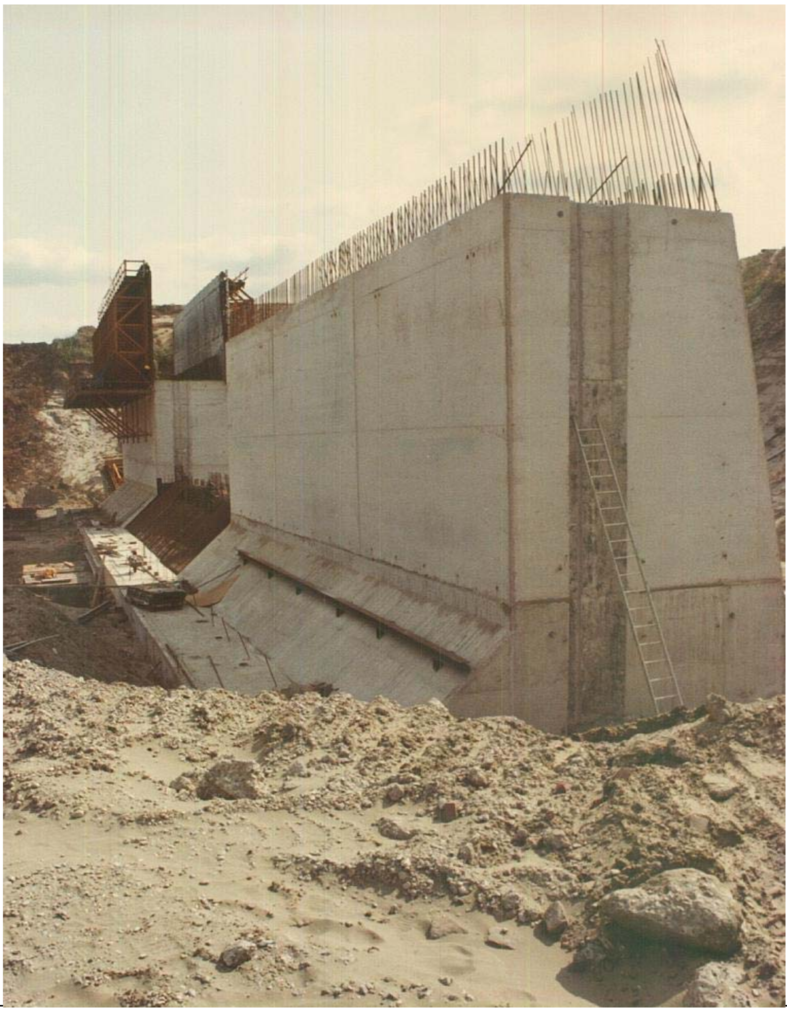
3 Constructibility

The existing excavation is actually isolated from either Gatun Lake and the Atlantic Ocean. Construction work will require at first that the water is pumped out, and consecutively excavation can be started;

Although slopes in Gatun Rock formation can be held very steep, care will have to be taken in the areas adjacent to the Atlantic Muck, as well near the Gatun Lake. Embankments, sloping angles will have to be studied in detail when more site-specific geo-technical data will be available.

The entire lock structure has to be realized in a dry construction pit, this will require most probably injection of rock cavities and/or fissures, as well as drainage of percolating water.

In the meantime, access channels can be dredged/excavated at both sides behind the earth retaining dams.



Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual design - Contract SAA-143351

ATLANTIC LOCKS HARMONIZATION

TASK A4b – LOCK WALLS

Rev A

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PORT OF ANTWERP
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1 Lock walls

1.1 DESIGN CRITERIA

Design criteria for the new lock structures have been given in the report of Task A2 - "Design Criteria" of the Atlantic Locks Harmonization study.

1.2 TECHNICAL DESCRIPTION OF LOCK WALL STRUCTURE

The lock walls will be situated in the "Gatun" rock formation. Although the characteristics of this formation can be considered as fairly good corresponding to a rock mass type IV, with UCS-values ranging from 2-20 MPa, they are considerably lower than those of the basalt formation (RMT I-II, UCS-values 40-100 MPa), which is found at the Pacific side of the Canal.

Furthermore, it is also clear that the deformation behavior of the "Gatun" formation will be different than the one of the basalt formation under vertical loading. (Deformation modules ranging from 1–1,3 GPa in Gatun rock, compared to 8-12,5 GPa in basalt).

Due to the different geotechnical situation, it will be necessary to excavate with flatter sloping angles in the "Gatun" formation, thus increasing the excavation volumes. Under these circumstances with less favorable rock characteristics, the wall structure will be larger at the footing, and comparable with the structure which was retained for the "La Boca" formation at the Pacific side.

Below, a detailed description is given of the wall type. Reference is made to the following drawings:

- D4-B-201 Longitudinal view on the left bank lock wall
- D4-B-202 Longitudinal view on the right bank lock wall
- D4-B-203 Plan view lock chamber 1
- D4-B-204 Plan view lock chamber 2
- D4-B-205 Plan view lock chamber 3
- D4-B-206 Cross section lock walls

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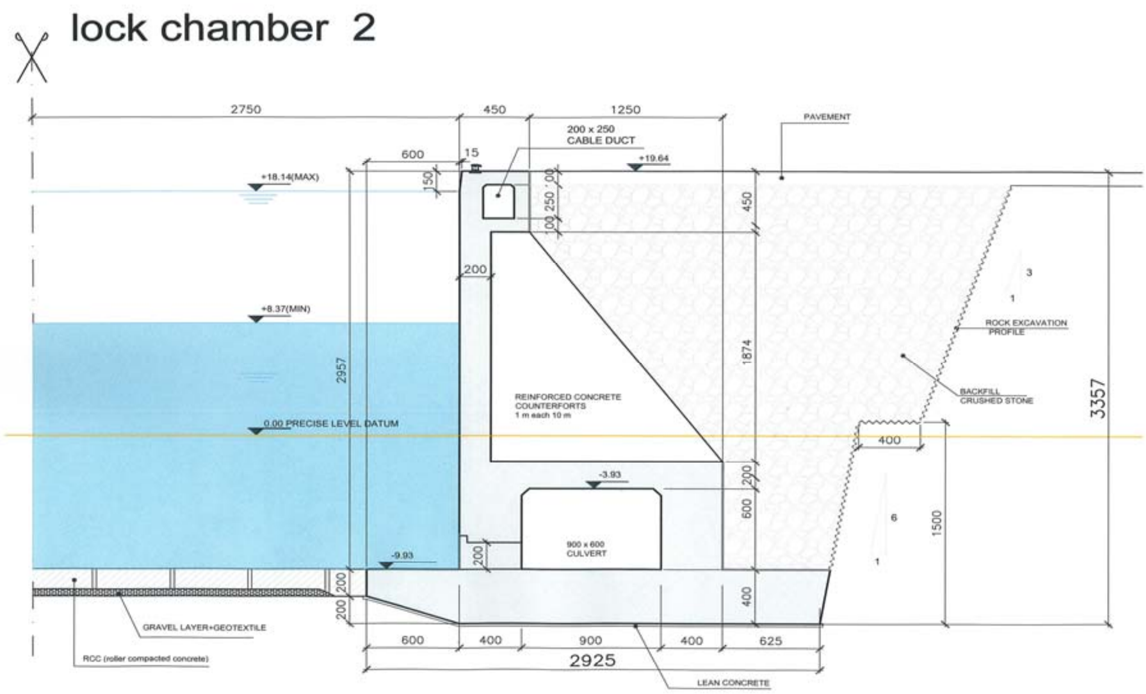


Typical Cross-section Lock Wall

The structure is a reinforced concrete structure with reinforced concrete counterforts at regular distances. At the top surface of the wall, additional structural concrete is required to install a cable duct and other quay side equipment, including bollards, ladders and lighting recesses.

The culvert with dimensions 6m x 9 m (H x W), which extends over the entire length of the lock wall, is integrated in the lower part of the lock wall.

Re-use of excavated material, fragmented to the proper size, for backfilling purposes, will lead to economical benefit. Besides, the high frictional properties of the broken rock allow for reduction of the horizontal pressure, exerted by the fill on the vertical retaining wall, leading to structural optimization.
The width at the foundation level has been determined in function of required safety against overturning; sliding effects being negligible due to the embedment in solid rock.

1.3 LOCK WALL ANALYSIS

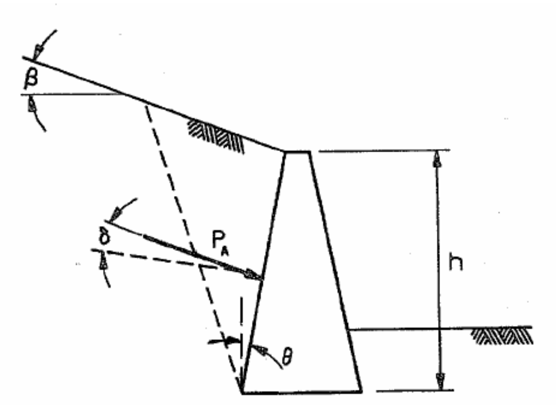
1.3.1 LOADS

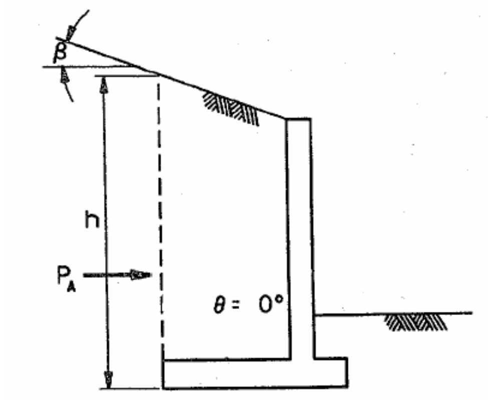
A. Self Weight (load case 1 = LC1)

Concrete	γ= 25.0 kN/m³
Wet backfill	γ= 20.0 kN/m³
Dry backfill	γ= 18.0 kN/m³

B. Earth Pressure (LC 2)

As the counterfort retaining wall nears geometrically to a cantilever wall type, the active lateral pressures will be calculated for a Rankine situation:





$$K_a = \frac{\cos^2(\varphi' - \theta)}{\cos^2\theta\cos(\theta+\delta)\left[1+\sqrt{\frac{\sin(\varphi'+\delta)\sin(\varphi'-\beta)}{\cos(\delta+\theta)\cos(\beta-\theta)}}\right]^2}$$

with δ = 0°; β = 0° and θ = 0°

The angle of friction in the filling of crushed stone is 45°

C. Water Pressure (LC 3)

Inside the lock, the minimum water level will be applied.
At the backside of the wall the maximum water level of the lock chamber will be applied.

D. Vertical live load

On the surface a vertical live load p=10 kN/m² is applied, this load case is included in normal earth pressure load case (LC2). In seismic conditions, only half of the live load is taken into account (5 kN/m²) and is included in the seismic earth pressures load case (LC6).

E. Bollards (LC 4)

The forces needed to move and hold a ship into the lock depend on:
- the water displacement of the ship;
- the velocity of entering the lock;
- the angle of the lines;
- the ratio cross section ship/lock;
- the movement of the ship during filling/emptying of the lock.

The calculation of these forces is very complicated and has been assessed during former studies (Hawser Forces Analysis by Flanders Hydraulics September 2003 / Hawser Forces Analysis by Flanders Hydraulics (CPP) March 2005). These calculations have been made for positioning systems both with locomotives and tugboats. In fact the maximum allowed hawser forces are entirely dependent on the breaking load of the hawsers and the positioning system. In the case of vessels tied up with mooring lines at fixed bollards, the breaking load could easily be higher than those determined in the above mentioned analyses. Maximum breaking loads of usual nylon mooring lines for large seagoing vessels are normally +/- 80tons (allowable working load 75%). If two lines would be used on only one bollard, this would give a maximum working load of 120tons.
For this concept design, the maximum bollard pull has consequently been set at 1500kN, a figure that corresponds very well with the recommendations of EAU1996 (page 143 – section 5.12).

As the wall is divided into segments of about 30m, the total horizontal load applied to one running meter of lock wall is 50kN/m'.

F. Seismic Loads (LC 5, 6, 7 and 8)

F.1 Earthquake level

ACP stated in its Memorandum of 20 Jan 2005 on 'Seismic Design Criteria' that an earthquake with a return period of 5000 years should be considered as Maximum Credible Earthquake (= MCE) and taken equal to the Maximum Design Earthquake (= MDE).

F.2 Performance Grade

The highest performance grade (Grade S) is applicable for
- critical structures with potential for extensive loss of human life and property
- key structures that are required to be serviceable for recovery from earthquake disaster
- critical structures that, if disrupted, devastate economic and social activities

A catastrophic failure of the locks may cause flooding in the terminal cities of the canal.

This means that the level of damages is:

- Structural: minor or no damage
- Little or no loss of serviceability

for a MCE-event.

F.3 Analysis type

As this report concerns conceptual design, a simplified analysis will be used considering equivalent static forces to apply the seismic loads on the structure.

F.4 Representative ground acceleration values

Based on the review of probabilistic seismic hazard analysis by Winter (2005), following PGA-values can be taken as representative for the site at Gatun:

Return period [years]	Representative PGA	Level	Load case
100	0.06		
500	0.15		
1,000	**0.22**		
2,500	0.32		
5,000	**0.41**	MDE =MCE	LC6
10,000	0.51		

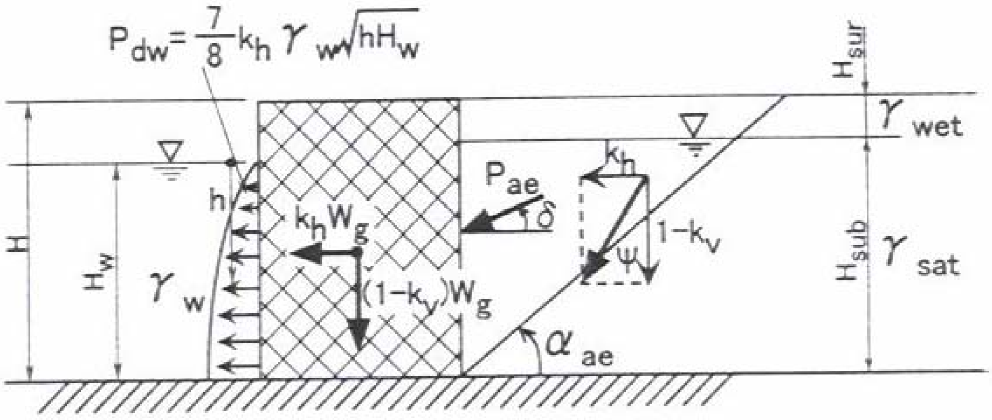
The seismic coefficient k_e for use in retaining structures is defined as follows for Special Class Structures:

$$k_e = PGA/g \quad \text{for } PGA < 0.2\ g$$

$$= \frac{1}{3}\left(\frac{PGA}{g}\right)^{\frac{1}{3}} \quad \text{for } PGA \geq 0.2\ g$$

With according to Eurocode design (CEN 1994)

$k_{h\ (horizontal)} = k_e$
$k_{v(vertical)} = ½ k_e$



(a)

The seismic loads are calculated separately for earth pressure (LC 6), equivalent horizontal (LC 7) and vertical (LC 8) reaction forces on structural masses and water bodies (LC 5).

G. Water Pressure in Dry Lock Conditions (LC 9)

Identical to load case 3 but without water pressure at the front side (lock empty).

1.3.2 LOAD COMBINATIONS

According to ROSA 2000

A. Quasi Permanent Load Combination

QP1 1*LC1 + 1*LC2 + 1*LC3 + 1*LC4

B. Rare Load Combination

R1 1*LC1 + 1*LC2 + 1*LC9

C. Fundamental Load Combinations

F1 1.20*LC1 + 1.20*LC2 + 1*LC3 + 1.20*LC4
F2 0.90*LC1 + 1.20*LC2 + 1*LC3 + 1.20*LC4
F3 1.20*LC1 + 1.20*LC2 + 1*LC9
F4 0.90*LC1 + 1.20*LC2 + 1*LC9

D. Accidental Load Combination - MCE

MCE 1*LC1 + 1*LC3 + 0.25*LC5 + 1*LC6b + 0.25*LC7 + 0.12*LC8

1.3.3 CRITERIA

According to ROSA 2000

A. Sliding along the base

For the construction in the Gatun Formation, a concrete floor will prevent sliding.

B. Turn over

Load Combination	Criterion
QP1	Compression zone A' > 90% A
R1	Compression zone A' > 75% A
F1 to F4	Compression zone A' > 10% A
MCE	Compression zone A' > 10% A

C. Load Bearing Capacity

$\gamma_d \cdot q_{ref} < q_u$
γ_d . = 1.4 for load combination F1 to F4
γ_d . = 1.0 for load combination MCE

q_u = ultimate load bearing capacity
q_{ref} = reference pressure

D. Deformations

The deformations will be checked for frequent, rare and accidental load combinations QP1, R1 and MCE.

E. Concrete Stresses

Fundamental load combinations: F1 tot F4: 1.125 x $\sigma_c < 0.85\ f_{ck}/1.5$
Accidental load combination: MCE: $\sigma_c < 0.85\ f_{ck}$

The factor 1.125 is a result of combination of the different factors in the load combinations between ROSA 2000 and Eurocode

F. Global Stability

The global stability of the lock wall is not considered critical as the foundation is seated in sound rock.

1.3.4 SOFTWARE

The calculations are made by means of “Esa-PrimaWin 3.60”, a finite element program. The lock wall is modeled as a 2D WALL structure, allowing forces acting in its plane.

A non-linear analysis is performed to allow for compression only supports.

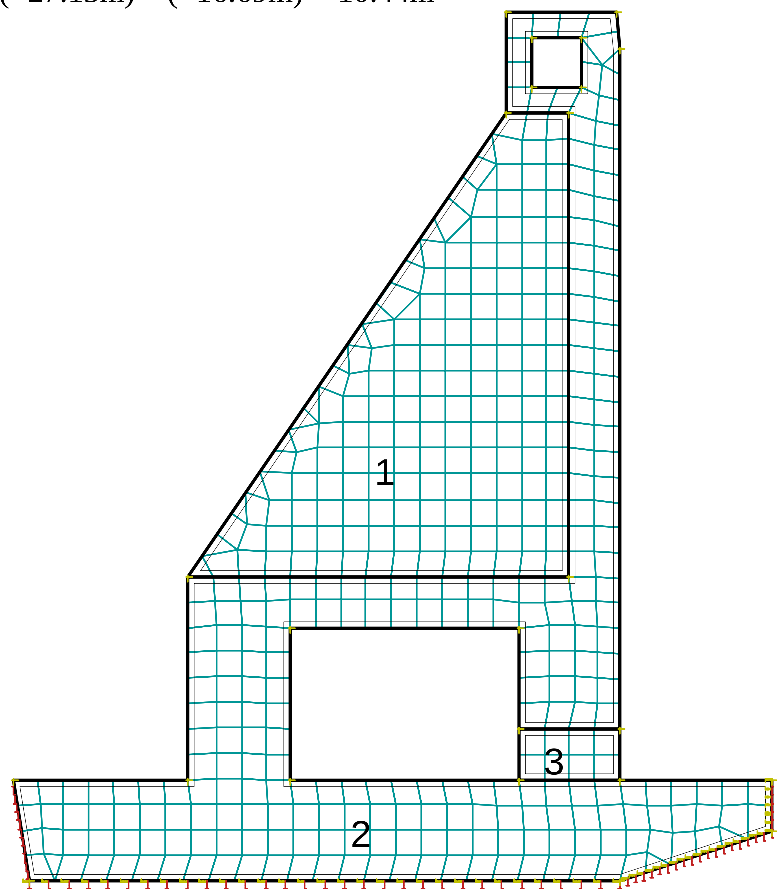
1.4 FOUNDATION IN GATUN ROCK

1.4.1 INPUT DATA

A. Geometry

The analysis below is performed for the lock wall of chamber 1, corresponding to the highest retaining height and the highest water level difference of the triple lift structure.

- $\Delta H = (+28.63m) - (-5.61m) = 34.24m$
- $\Delta WL = (+27.13m) - (+16.69m) = 10.44m$



The FE model consists of the following three 2D macro structural elements :

No.	Element	Thickness
1	Reinforced concrete counterforts	0.1 m (1m each 10 m)
2	Retaining wall	1 m
3	Culvert port (2 m x 2 m)	0.867 m (1 port each 15 m)

B. Materials

Name:		
C25/30-cracked		
	E modulus	20,000.00 MPa
	Poisson coefficient.	0.20
	Density	2,500.00 kg/m^3
	Expansion coefficient	0.01 mm/m.K

For the material corresponding to the reinforced concrete counterforts, the submerged weight of 15 kN/m^3 is taken into account, as the water pressure of LC3 are immediately applied to the main wall element.

C. Supports

The rock base is modeled by means of linear supports.

Gatun Rock Formation:

- The embedded part of the retaining wall is characterized by a linear support of, K = 100 MN/m/m', working in compression only.
- Parallel to the base, sliding is restricted through a frictional resistance of 30 MN/m/m'.

RCC:

- The side face of the front toe in contact with the floor of RCC is characterized by a linear support of, K = 1,000 MN/m/m', working in compression only.
- Parallel to this, sliding is restricted through a frictional resistance of 300 MN/m/m'.

D. Loads

D.1 Self Weight (LC1)

The total weight of the structure: 659 ton/m'.

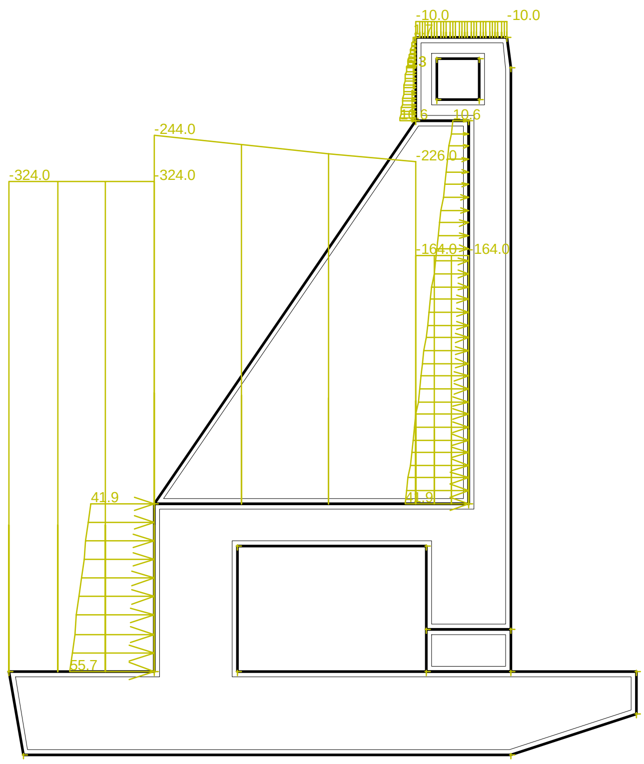
D.2 Rock fill (LC2)

from	28.63		27.13	
			λav =	λah =
λa =	0.1716		0.0000	0.1716
ϕ =	45	0.79	δ =	0
θ =	0	0.00		
γ =	18			
z	p		**pv**	**ph**
28.63	0		0.000	0.000
27.13	27		0.000	4.632

from	27.13	to	24.63	
			λav =	λah =
λa =	0.1716		0.0000	0.1716
ϕ =	45	0.79	δ =	0
θ =	0	0.00		
γ =	10			
z	p		**pv**	**ph**
27.13	27		0.000	4.632
24.63	52		0.000	8.922

from	24.63	to	6.39	
			λav =	λah =
λa =	0.1716		0.0000	0.1716
ϕ =	45	0.79	δ =	0
θ =	0	0.00		
γ =	10			
z	p		**pv**	**ph**
24.63	52		0.000	8.922
6.39	234.4		0.000	40.217

from	6.39	to	-1.61	
			λav =	λah =
λa =	0.1716		0.0000	0.1716
ϕ =	45	0.79	δ =	0
θ =	0	0.00		
γ =	10			
z	p		**pv**	**ph**
6.39	234.4		0.000	40.217
−1.61	314.4		0.000	53.943



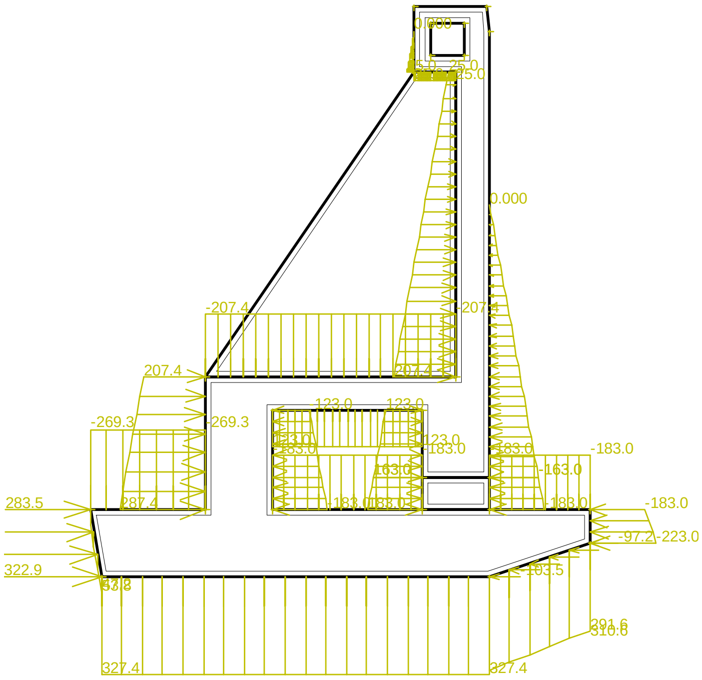
Soil weight on culvert block

X	pv
-6.916667	314
0	314
0	234
12.5	216
12.5	164
15	164

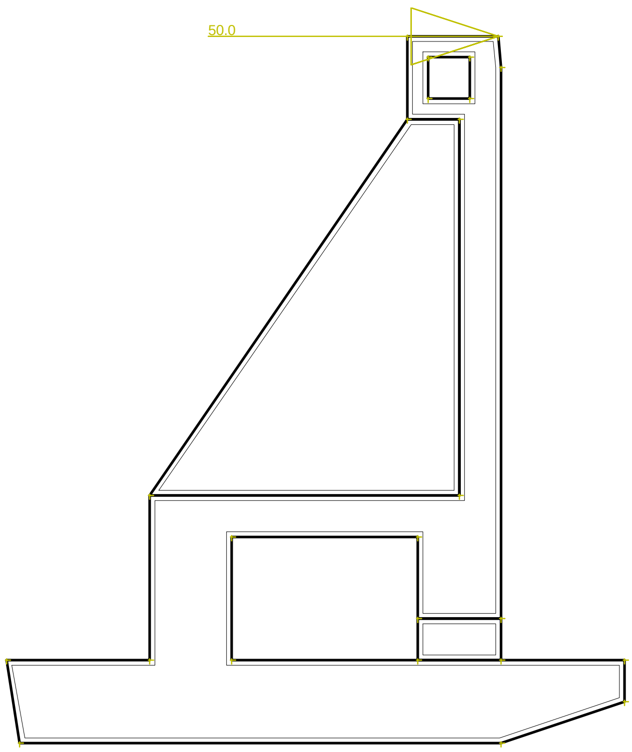
Remark: The active lateral pressures on the vertical through the rear edge of the retaining wall are transferred to the front wall elements for ease of implementation.

D.3 Water Pressure (LC 3)

Water level inside the lock: +16.69m
Water level outside the lock: +27.13m



D.4 Bollard pull (LC 4)



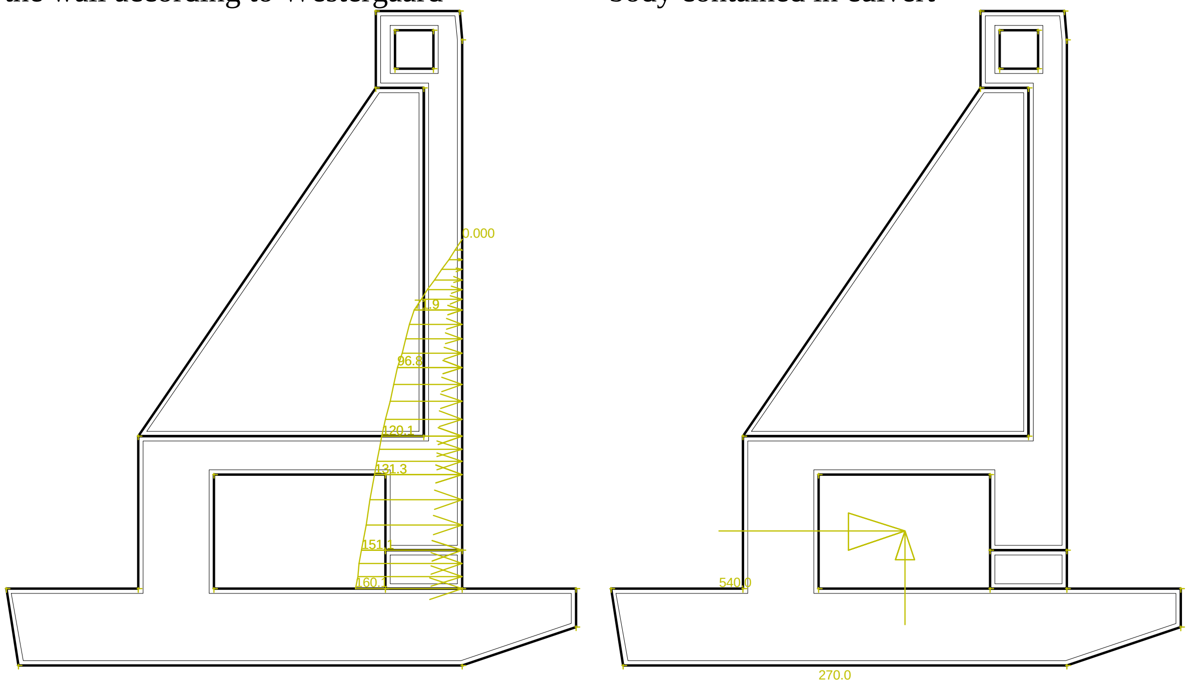
D.5 Water Pressure – Seismic loading (LC 5)

Water level inside the lock: +16.69m
Water level outside the lock: +27.13m

Additional water pressures generated by seismic action are:

Water suction forces at the outside face of the wall according to Westergaard

Equivalent seismic reaction forces on water body contained in culvert



The seismic water pressure load case is valid for a unit value of k_e, and is multiplied by the respective k_e value in the corresponding load combinations.

D.6 Seismic Loads (LC 6, 7 and 8)

LC 6

The active lateral earth thrust under seismic conditions is calculated based on the Mononobe-Okabe (M-O) methodology. The M-O method is an extension of Coulomb's theory, wherein the M-O method takes into account the inertial forces acting on the soil mass during earthquake loading. It was developed to assess the stability of massive gravity walls, assuming that the retaining wall and the failure wedge act as rigid bodies.

The inertia forces are then accounted for by considering a seismic inertia angle, Ψ = atan ($k_h/(1-k_v)$), in which k_h represents the horizontal seismic coefficient or the modified horizontal seismic coefficient for dry and submerged layers respectively.

As the counterfort retaining wall under consideration is geometrically near to cantilever wall type, a Rankine situation is assumed for the calculation of the seismic active earth pressures on the vertical through the rear edge of structure. Consequently, the weight and inertia forces of the soil masses above the structure's rear base have to be taken into consideration.

Remark:

- The active lateral pressures and the horizontal inertia components of the soil masses are transferred to the front wall elements for ease of implementation.
- Half of the vertical live load is taken is accounted for in the seismic earth pressures.

LC 7 and 8

The inertia forces on the structural weight are calculated as follows:

$G_v = (1-k_v)\ G$

$G_h = k_h\ G$

With G the weight of the filling

G_v the vertical component of the weight in seismic conditions

G_h the horizontal component of weight in seismic conditions

PGA	0.41	g	k_e	0.248
k_h	0.248		k_v	0.124

CFR	0.1
Width	23.917

from	28.63		27.13		1.50			
			λav =	λah =				
λa =	0.3218		0.0000	0.3218		k_h'	0.248	
φ =	45	0.79	δ =	0	0.00	ψ	15.8	0.28
θ =	0	0.00						
γ =	18							
z	p		pv	ph1				
28.63	4.381		0.000	1.410				
27.13	28.038		0.000	9.021				

Q1v	0.00
Q1h	7.82
Q1	7.82

Width =	19.41667
b =	4.5
ph2	86.547
	ph
	87.957
	95.569

from	27.13	to	24.63		2.50			
			λav =	λah =				
λa =	0.5698		0.0000	0.5698		k_h'	0.495	
φ =	45	0.79	δ =	0	0.00	ψ	29.5	0.51
θ =	0	0.00						
γ =	10							
z	p		pv	ph1				
27.13	28.038		0.000	15.976				
24.63	49.942		0.000	28.458				

Q2v	0.00
Q2h	55.54
Q2	55.54

Width =	19.41667
b =	4.5
ph2	96.164
	ph
	112.140
	124.622

from	24.63	to	6.39		18.24			
			λav =	λah =				
λa =	0.5698		0.0000	0.5698		k_h'	0.495	
φ =	45	0.79	δ =	0	0.00	ψ	29.5	0.51
θ =	0	0.00						
γ =	10							
z	p		pv	ph1				
24.63	49.942		0.000	28.458				
6.39	209.758		0.000	119.523				

Q2v	0.00
Q2h	1349.58
Q2	1349.58

Width =	21.91667
b =	2
ph2	
	ph
	125.860
	210.734

from	6.39	to	-1.61		8.00			
			λav =	λah =				
λa =	0.5698		0.0000	0.5698		k_h'	0.495	
φ =	45	0.79	δ =	0	0.00	ψ	29.5	0.51
θ =	0	0.00						
γ =	10							
z	p		pv	ph1				
6.39	209.758		0.000	119.523				
−1.61	279.853		0.000	159.464				

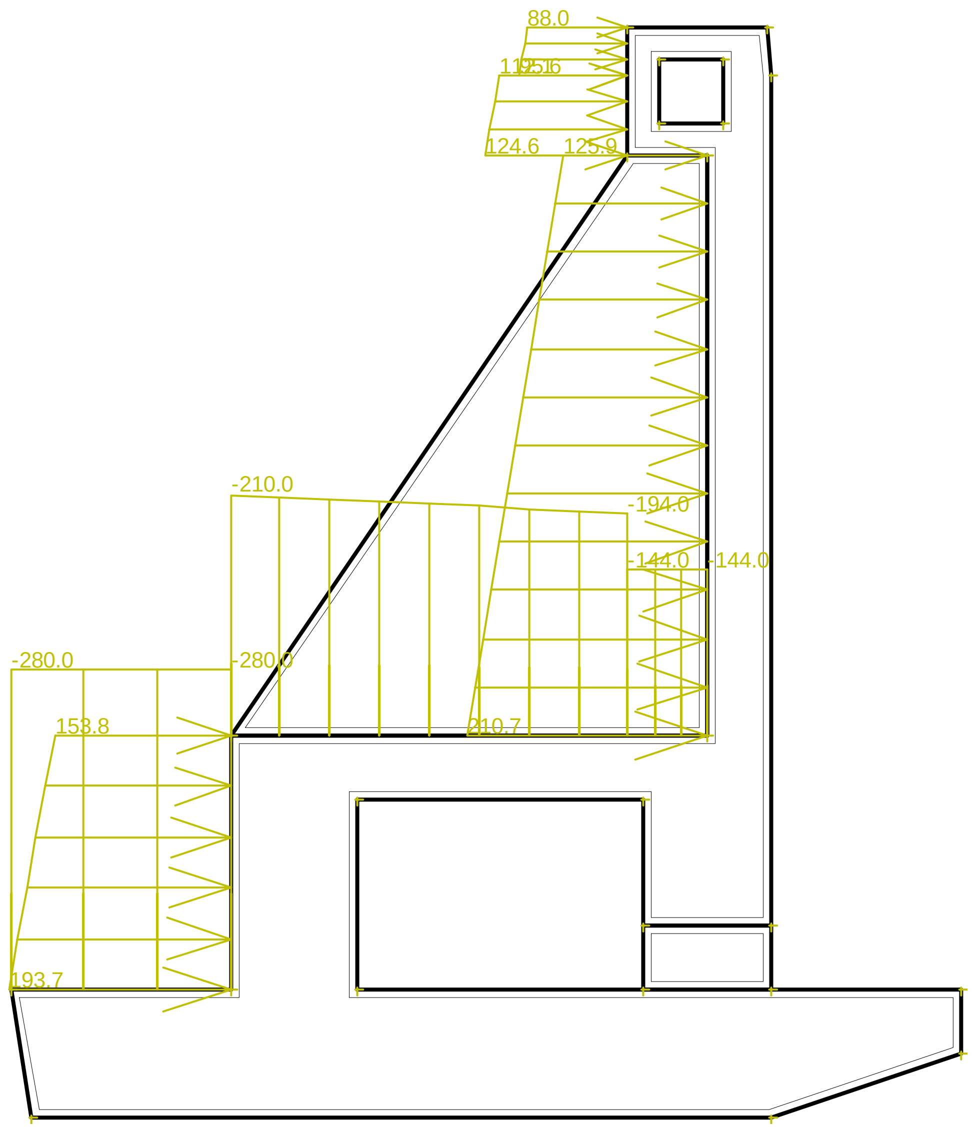
Q2v	0.00
Q2h	1115.95
Q2	1115.95

Width =	6.916667
b =	17
ph2	34.256
	ph
	153.779
	193.719

2528.89

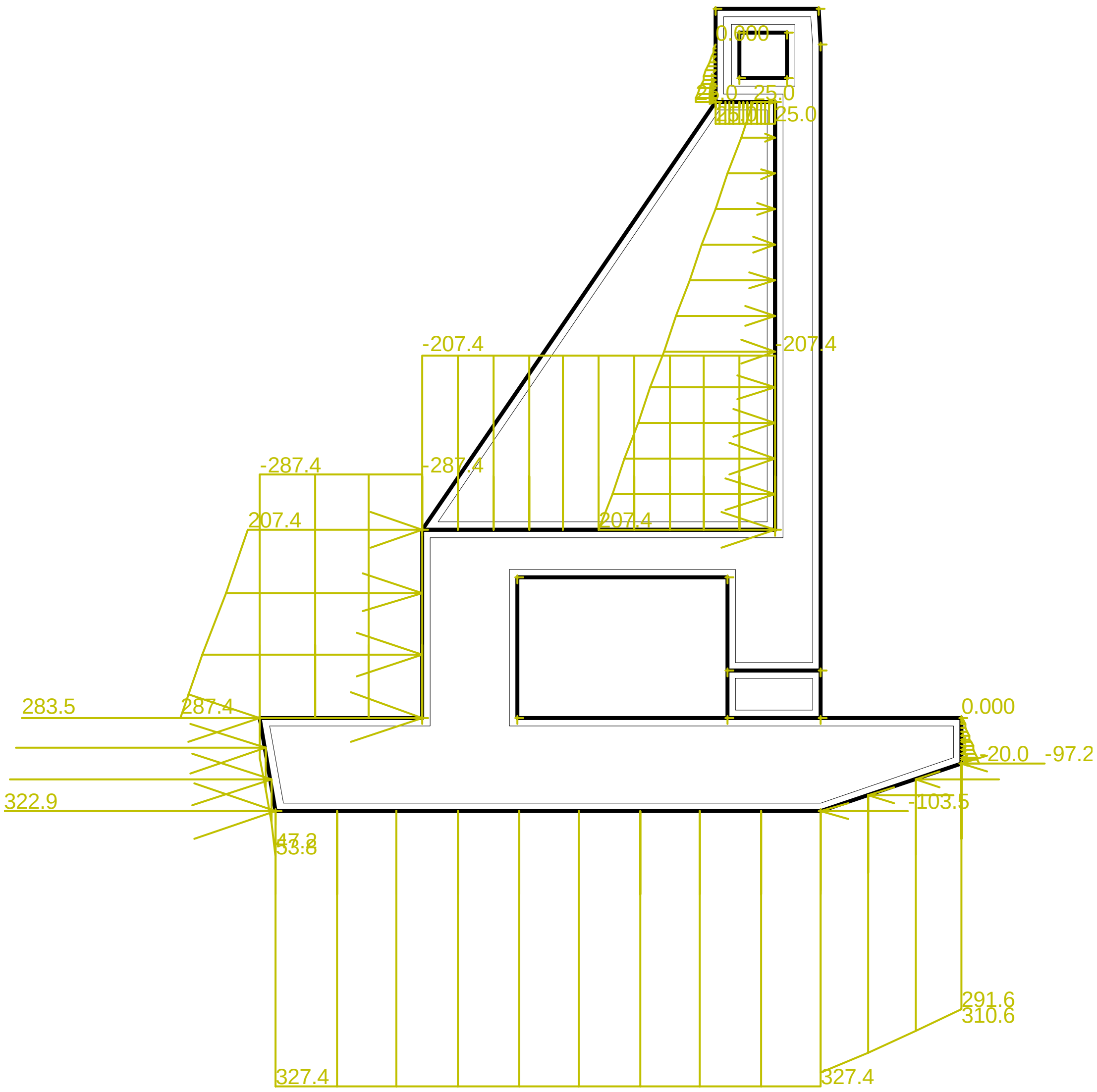
Soil weight on culvert block

X	pv
-6.916667	280
0	280
0	210
12.5	194
12.5	144
15	144



D.7 Water Pressure in Dry Lock Conditions (LC9)

Water level inside the lock: -1.61 m
Water level outside the lock: +27.13m

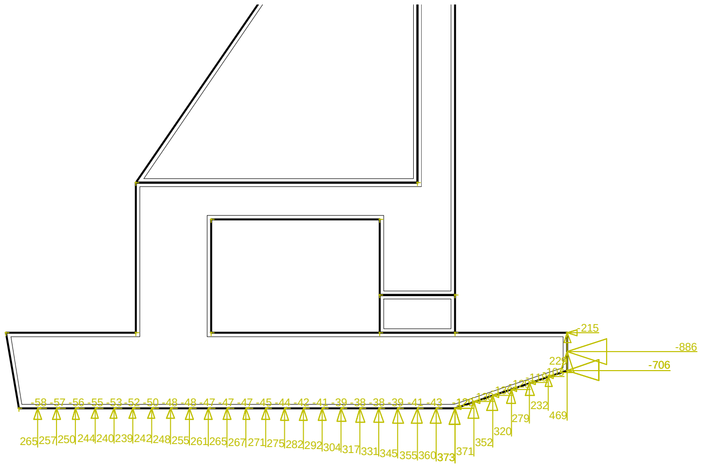


At the rear side of the wall and the bottom the same water pressure is applied as in case 3. This is a very conservative assumption as the pressure should normally be equal to zero at the front toe of the footing.

1.4.2 TURN OVER

A. Quasi Permanent Load Combination (QP1) – Lock in service

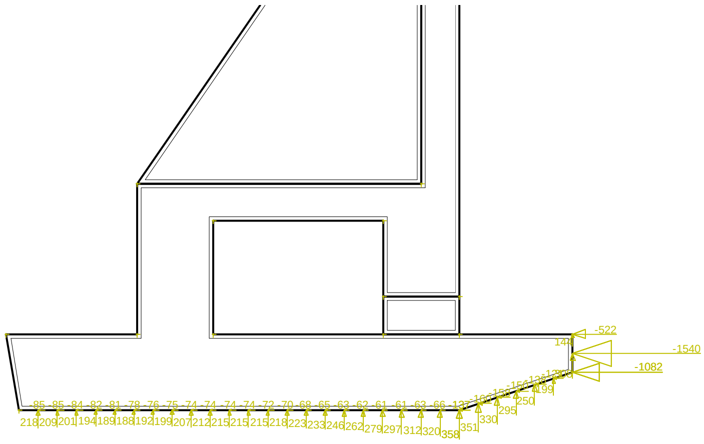
Criterion: Compression zone A' (= 97%)> 90% A



The magnitude of the vectors shown represent the support reactions in each node and not the soil pressure.

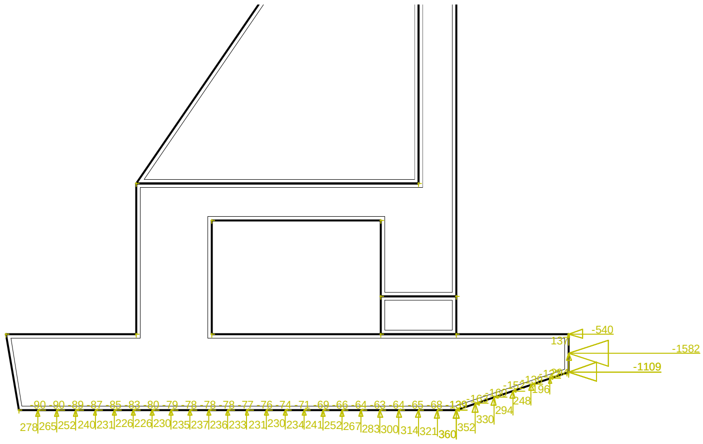
B. Rare Load Combination (R1) – Dry Lock Conditions

Criterion: Compression zone A'(= 97%) > 75% A



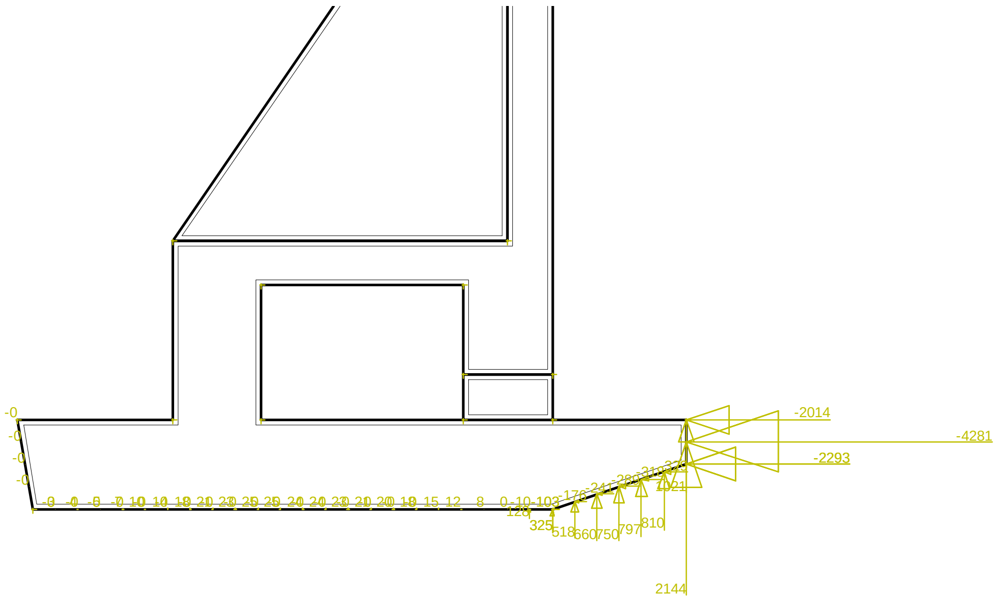
C. Fundamental Load Combinations F1 to F4

Criterion: Compression zone A' (= 97%)> 10% A



D. Accidental Load Combination – Earth Quake MCE

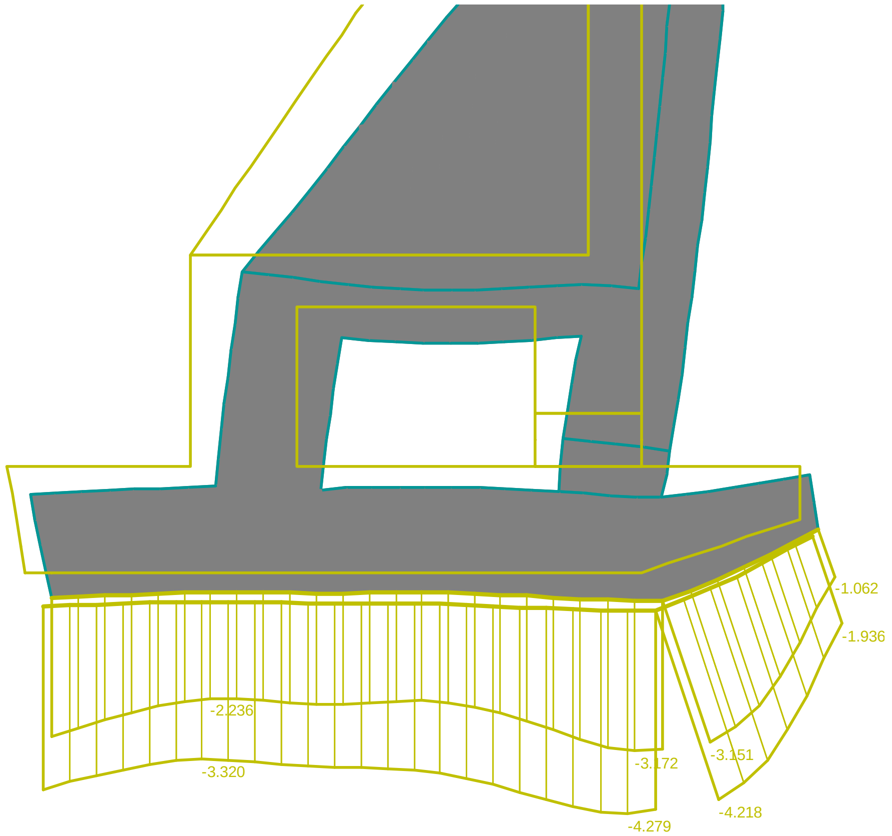
Criterion: Compression zone A' (= 25 %)> 10% A.



1.4.3 LOAD BEARING CAPACITY

A. Fundamental Load Combinations F1 to F4

Criterion: $\gamma_d \cdot q_{ref} < q_u$
with $\gamma_d \cdot = 1.4$



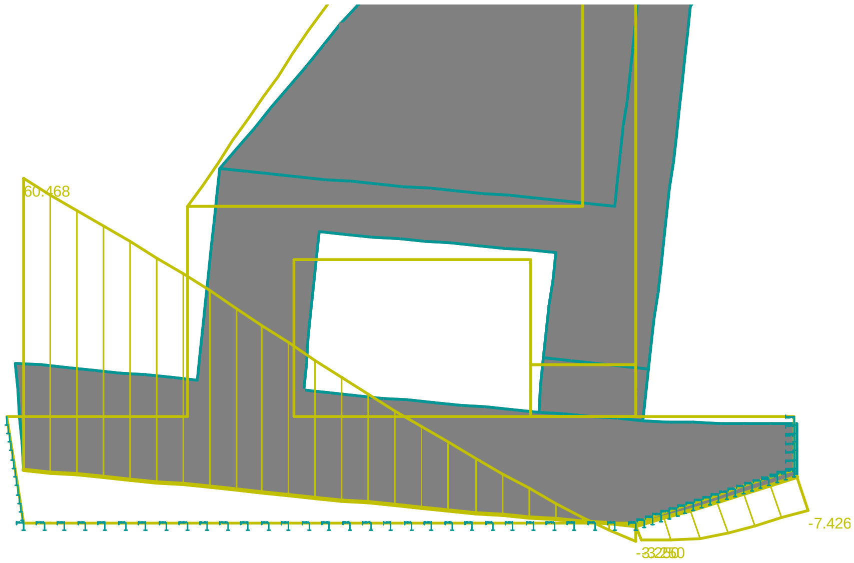
The maximum vertical displacement at the bottom of the wall is 4.279 mm, which is negligible.
This corresponds to a maximum vertical pressure of 0.0043 x 100,000 kN/m² = 428 kN/m² = 0.43 Mpa.
Consequently:

$$\gamma_d \cdot q_{ref} = 1.4*0.428 = 0.60 < q_u = 2 \text{ MPa}$$

(According to report R2-A the Uni-axial Compressive Strength of the Gatun rock is 2 MPa).

B. Accidental load combination MCE

Criterion: $\gamma_d \, . \, q_{ref} < q_u$
with γ_d . = 1.0 for accidental load combinations



The maximal vertical displacement at the bottom of the wall is 7.426 mm
This means a vertical pressure of 0.0074 x 100,000 kN/m² = 743 kN/m² = 0.74 MPa.
Consequently:

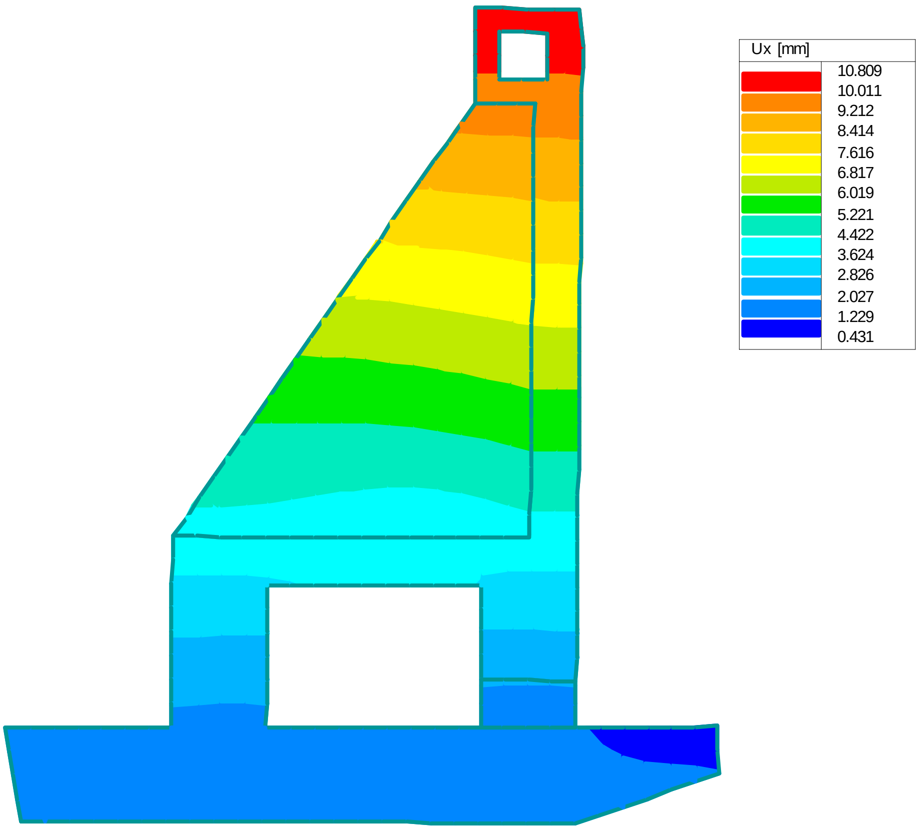
$$\gamma_d \, . \, q_{ref} = 1.0*0.743 = 0.74 < q_u = 2 \text{ MPa}$$

(According report R2-A the Uni-axial Compressive Strength of the Gatun rock is 2 MPa).

1.4.4 DEFORMATIONS

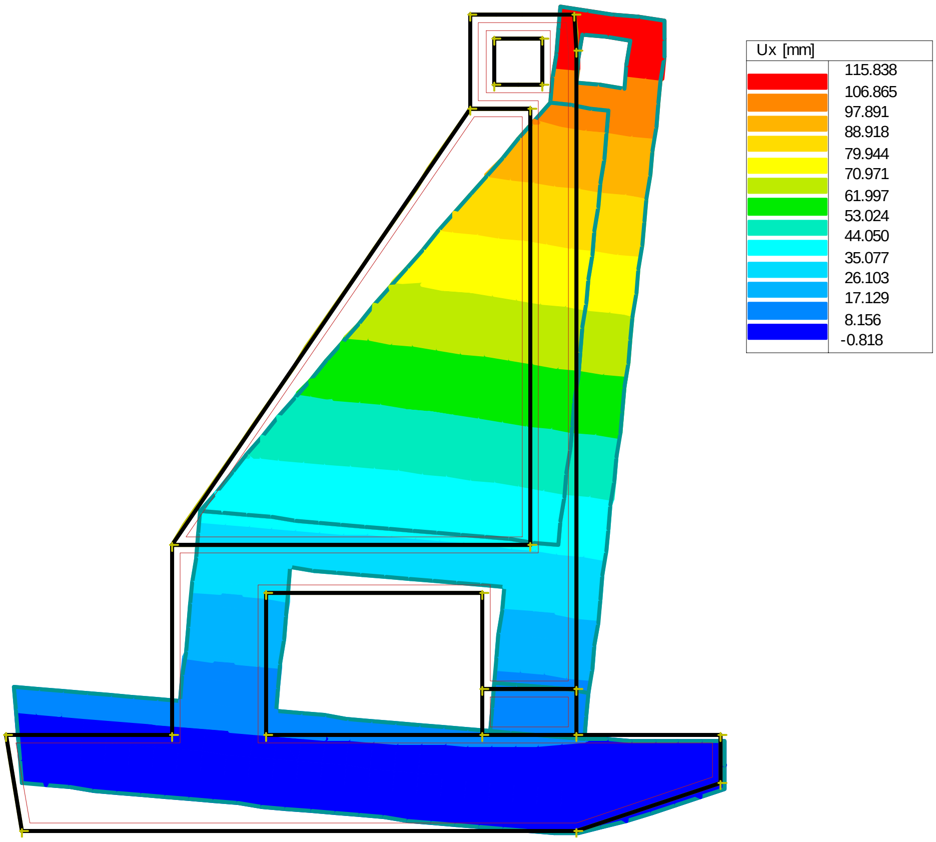
A. Load Combination QP1 (Lock in Service)

Maximal horizontal displacement, $u_{x,max}$ = 10.8 mm



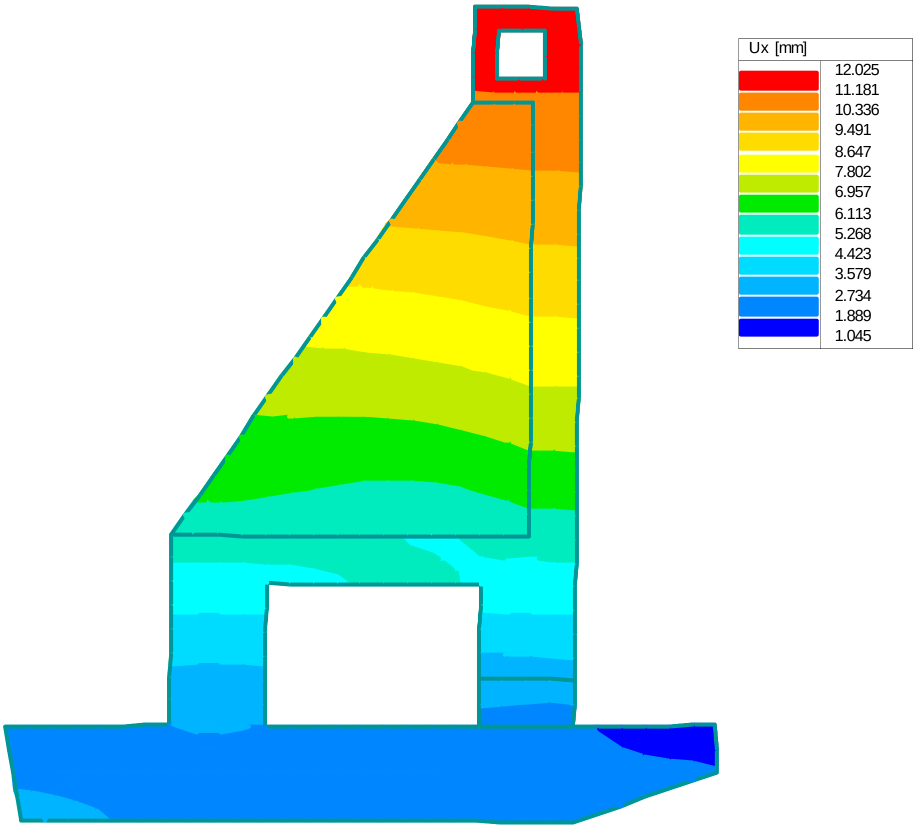
B. Load Combination MCE (Earth Quake)

Maximal horizontal displacement, $u_{x,max}$ = 115.8 mm (reference case)



C. Load Combination R1 (Dry Lock Conditions)

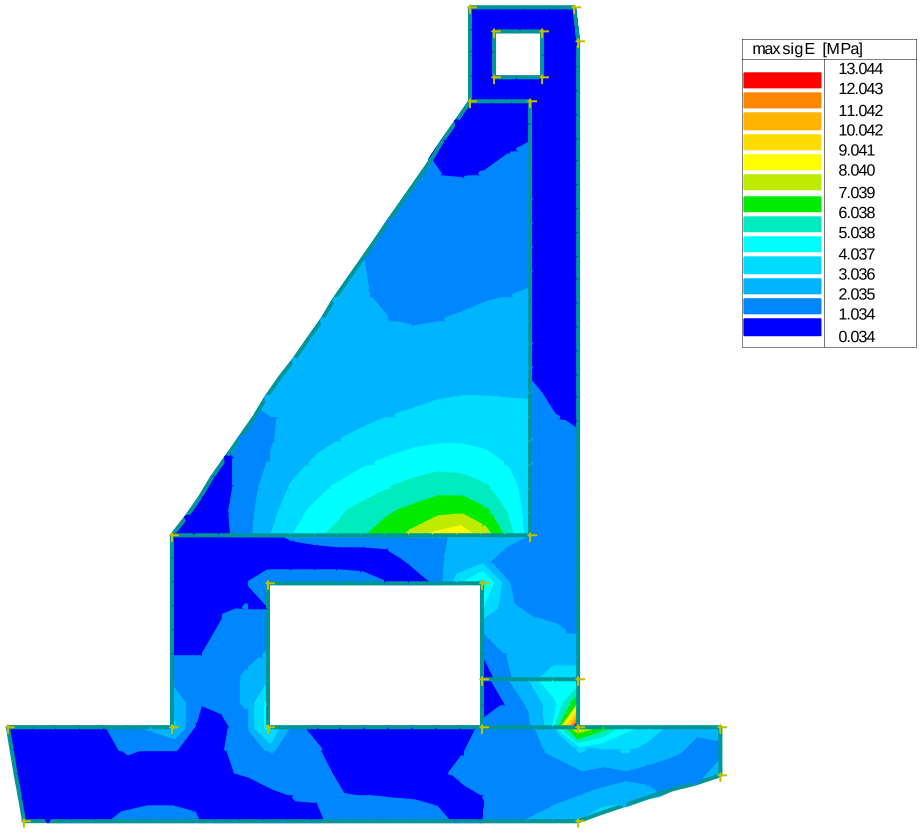
Maximal horizontal displacement, $u_{x,max}$ = 12.0 mm



1.4.5 CONCRETE STRESSES

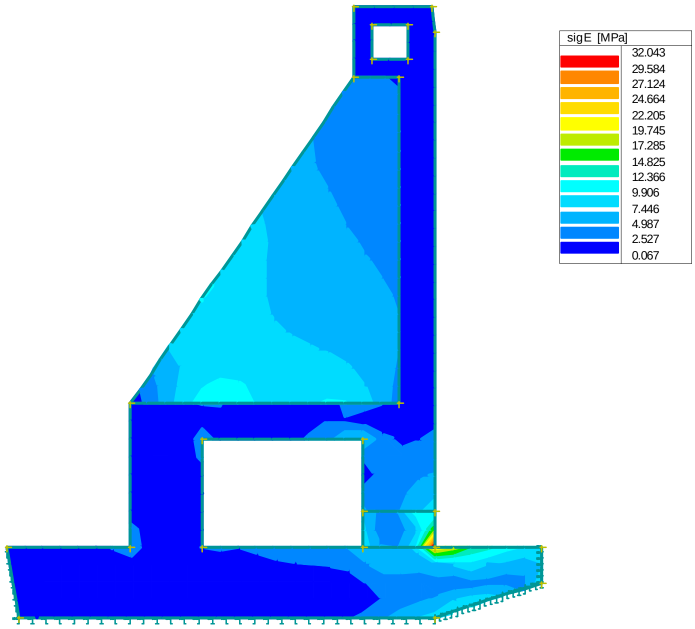
A. Fundamental load combinations F1 tot F4:

$1.125 \times \sigma_c < 0.85\ f_{ck}/1.5$
with $f_{ck} = 25 N/mm^2$:
$\sigma_c = 13.04\ N/mm^2 > 12.5\ N/mm^2$
=> Local reinforcement at culvert outlet will be needed

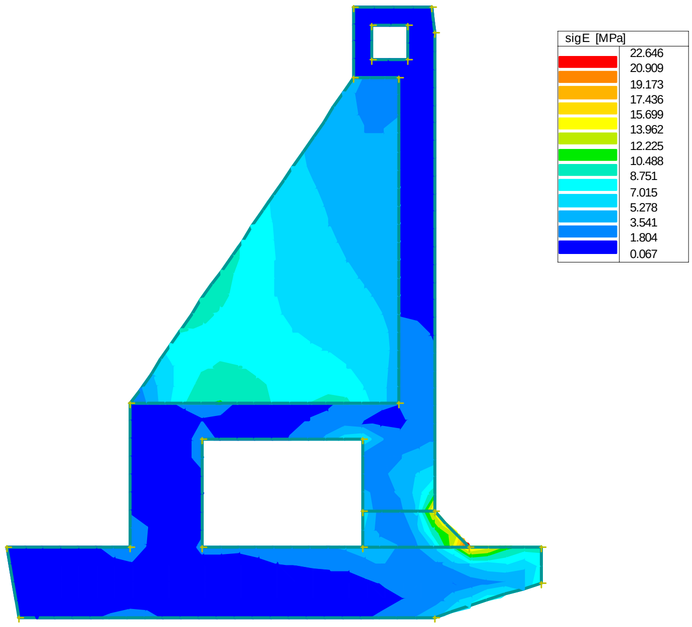


B. Accidental load combination MCE:

$\sigma_c < 0.85\ f_{ck}$
with $f_{ck} = 25N/mm^2$:
$\sigma_c = 32.04\ N/mm^2 > 21.25\ N/mm^2$
=> As concrete stresses are significantly higher, an adaptation of the geometry is recommended.



$\sigma_c = 22.65\ N/mm^2 > 21.25\ N/mm^2$
=> Local reinforcement will still be necessary



1.4.6 SUMMARY

Criterion	QP1 (Quasi Permanent Load Combination - Service)	R1 (Rare Load Combination - Dry Lock Condition)	F (Envelope of Fundamental Load Combinations)	MCE (Accidental Load Combination - MCE)
Compression zone A’ > 90% A	97%	-	-	-
Compression zone A’ > 75% A	-	97%	-	
Compression zone A’ > 10% A	-	-	97%	25%
Load Bearing Capacity				
1.4 $q_{ref} < q_u$ (MPa)	-	-	0.599	-
$q_{ref} < q_u$ (MPa)	-	-	-	0.743
Deformations				
maximal displacement (mm)	10.8	12.0	-	115.8
Concrete stresses				
1.125 x σc < 0.85 fck/1.5	**-**	**-**	**104%**	**-**
σc < 0.85 fck	**-**	**-**	**-**	**151%**
Deep Seeted Sliding				
	-	-	-	-

2 Lock heads

2.1 DESIGN CRITERIA

Design criteria for the new lock structures have been given in the report of Task A2 - "Design Criteria" of the Atlantic Locks Harmonization study.

2.2 TECHNICAL DESCRIPTION OF LOCK HEAD STRUCTURE

Reference is made to following drawings:

- D4-B-207/209 Lock head 2
- D4-B-210/212 Lock head 1
- D4-B-213/215 Lock head 3
- D4-B-216/218 Lock head 4

The lock head is a massive construction housing the lock gates and culvert valves.

One major condition is the geo-technical situation of the lock siting. The lock heads will be situated in the "Gatun" rock formation. Although the characteristics of this formation can be considered as fairly good corresponding to a rock mass type IV, with UCS-values ranging from 2-20 MPa, they are considerably lower than those of the basalt formation (RMT I-II, UCS-values 40-100 MPa) which is found at the Pacific side of the Canal..

Furthermore, it is also clear that the deformation behavior of the "Gatun" formation will be different than the one of the basalt formation under vertical loading.
(Deformation modules ranging from 1–1,3 GPa in Gatun rock, compared to 8-12,5 GPa in basalt).

In this report a section taken through the lock gate recesses is checked.

2.3 LOCK HEAD ANALYSIS

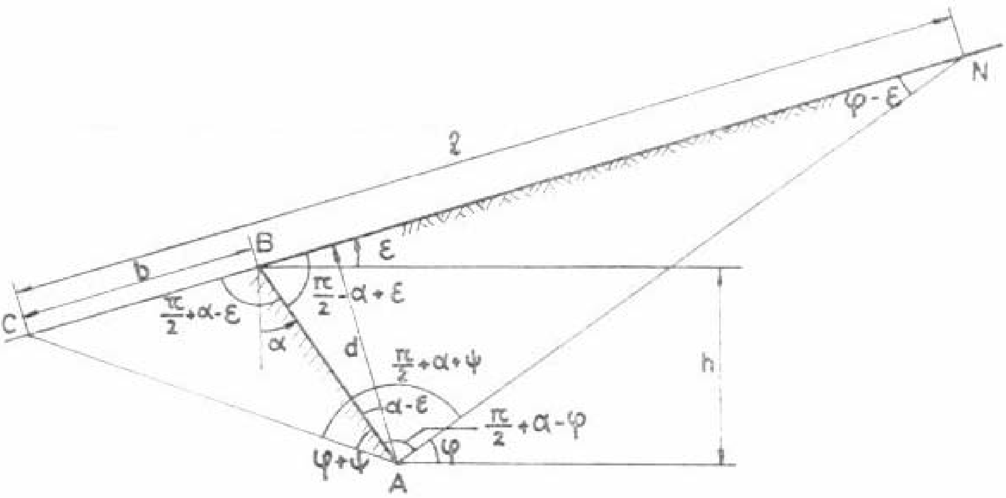
2.3.1 LOADS

A. Self Weight (case 1)

Concrete γ= 24.5 kN/m³
Wet Soil γ= 20.0 kN/m³
Dry Soil γ= 18.0 kN/m³

B. Earth Pressure (case 2)

The active earth pressure will be calculated using the formula of COULOMB-PONCELET



$$\lambda_a = \frac{\cos^2(\varphi - \alpha)}{\cos^2\alpha \cos(\alpha + \psi)\left(1 + \sqrt{\dfrac{\sin(\varphi + \psi)\sin(\varphi - \varepsilon)}{\cos(\alpha + \psi)\cos(\varepsilon - \alpha)}}\right)^2}$$

The angle of friction in the filling of crushed stone and sand is 45°
On the surface a load p=10 kN/m² is applied. (5 kN/m² in seismic conditions)

C. Water Pressure (case 3)

At the side of the upper chamber the maximum water level of that chamber will be applied. This level will also be applied in the recess at that side. At the side of the lower chamber the minimum water level of that chamber will be applied. This level will also be applied in the recess at that side.

D. Seismic Loads (case 4 and 8)

D.1 Earthquake level

ACP stated in its Memerandum of 20 Jan 2005 on ‘Seismic Design Criteria’ that an earthquake with a return period of 5,000 years should be considered as Maximum Credible Earthquake (= MCE) and taken equal to the Maximum Design Earthquake (= MDE).

D.2 Performance Grade

The highest performance grade (Grade S) is applicable for

- critical structures with potential for extensive loss of human life and property
- key structures that are required to be serviceable for recovery from earthquake disaster
- critical structures that, if disrupted, devastate economic and social activities

A catastrophic failure of the locks may cause flooding in the terminal cities of the canal. This means that the level of damages is:

- Structural: minor or no damage
- Little or no loss of serviceability

for a MCE-event.

D.3 Analysis type

As this report concerns a conceptual design we will use the simplified analysis.

D.4 Representative ground acceleration values

Based on the review of probabilistic seismic hazard analysis by Winter (2005), following PGA-values can be taken as representative for the site at Gatun:

Return period [years]	Representative PGA	Criterion
100	0.06	
500	0.15	
1000	**0.22**	
2500	0.32	
5000	**0.41**	MDE = MCE
10000	0.51	

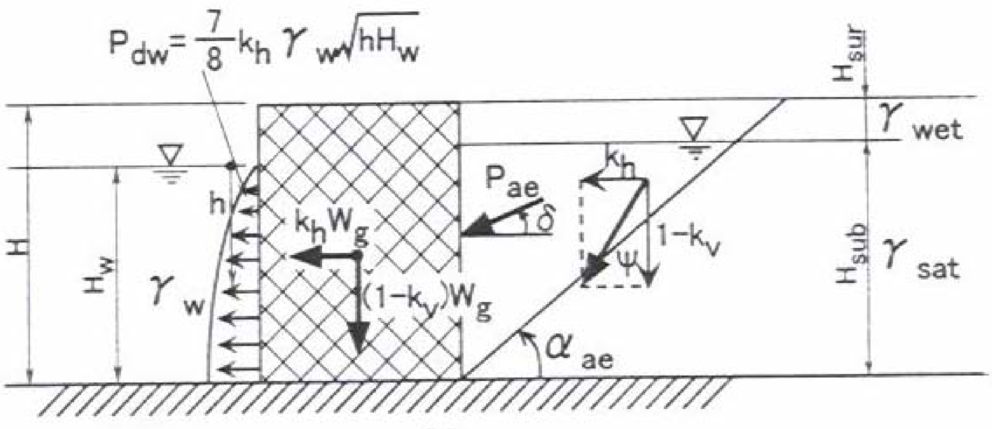
The seismic coefficient k_e for use in retaining structures is defined as follows for Special Class Structures:

$$k_e = PGA/g \quad \text{for } PGA < 0.2\ g$$

$$= \frac{1}{3}\left(\frac{PGA}{g}\right)^{\frac{1}{3}} \quad \text{for } PGA \geq 0.2\ g$$

With according to Eurocode design (CEN 1994)

$k_{h\ (horizontal)} = k_e$
$k_{v(vertical)} = ½k_e$



(a)

E. Water Pressure in Dry Recess Conditions (case 5 & 6)

In case 5 the one recess on the side of the upper chamber is put in dry conditions. In case 6 both recesses at put in dry conditions.

F. Upper Wagon (case 7)

The reaction forces of the upper wagon are taken form the report Lock Gates.

2.3.2 LOAD COMBINATIONS

According to ROSA 2000

A. Quasi Permanent Load Combination

CASE 101	= 1.00 x	[1]	+ 1.00 x	[2]	+ 1.00 x	[3]	+ 1.00 x	[7]

B. Accidental Load Combination - MCE

CASE 102	= 1.00 x	[1]	+ 1.00 x	[3]	+ 1.00 x	[4a]	+ 1.00 x	[8a]

C. Rare Load Combination

CASE 103	= 1.00 x	[1]	+ 1.00 x	[2]	+ 1.00 x	[5]
CASE 104	= 1.00 x	[1]	+ 1.00 x	[2]	+ 1.00 x	[6]

D. Fundamental Load Combinations

CASE 105	= 1.20 x	[1]	+ 1.20 x	[2]	+ 1.00 x	[3]	+ 1.20 x	[7]
CASE 106	= 0.90 x	[1]	+ 1.20 x	[2]	+ 1.00 x	[3]	+ 1.20 x	[7]
CASE 107	= 1.20 x	[1]	+ 1.20 x	[2]	+ 1.00 x	[5]		
CASE 108	= 0.90 x	[1]	+ 1.20 x	[2]	+ 1.00 x	[5]		
CASE 109	= 1.20 x	[1]	+ 1.20 x	[2]	+ 1.00 x	[6]		
CASE 110	= 0.90 x	[1]	+ 1.20 x	[2]	+ 1.00 x	[6]		

2.3.3 CRITERIA

According to ROSA 2000

A. Sliding along the base

Due to the ground pressure at both sides of the lock head, sliding will not occur.

B. Turn over

The lock head can not turn over, due to dimensions and the ground pressure at both sides of the head.

C. Load Bearing Capacity

$\gamma_d \cdot q_{ref} < q_u$
γ_d . = 1.4 for load combination 105 to 110
γ_d . = 1.0 for load combination 102 (MCE)

qu = ultimate load bearing capacity
qref = reference pressure

D. Deformations

The deformations will be checked for frequent and rare load combinations 101, 102 (MCE), 103 and 104.

E. Concrete Stresses

Fundamental load combinations: 105 tot 110: 1.125 x σc < 0.85 fck/1.5
Accidental load combination: 102 (MCE): σc < 0.85 fck

The factor 1.125 is caused by the different factors in the load combinations between ROSA 2000 and EuroCode.

F. Global Stability

Global stability will not be a problem, when the items in part C, D and E are checked.

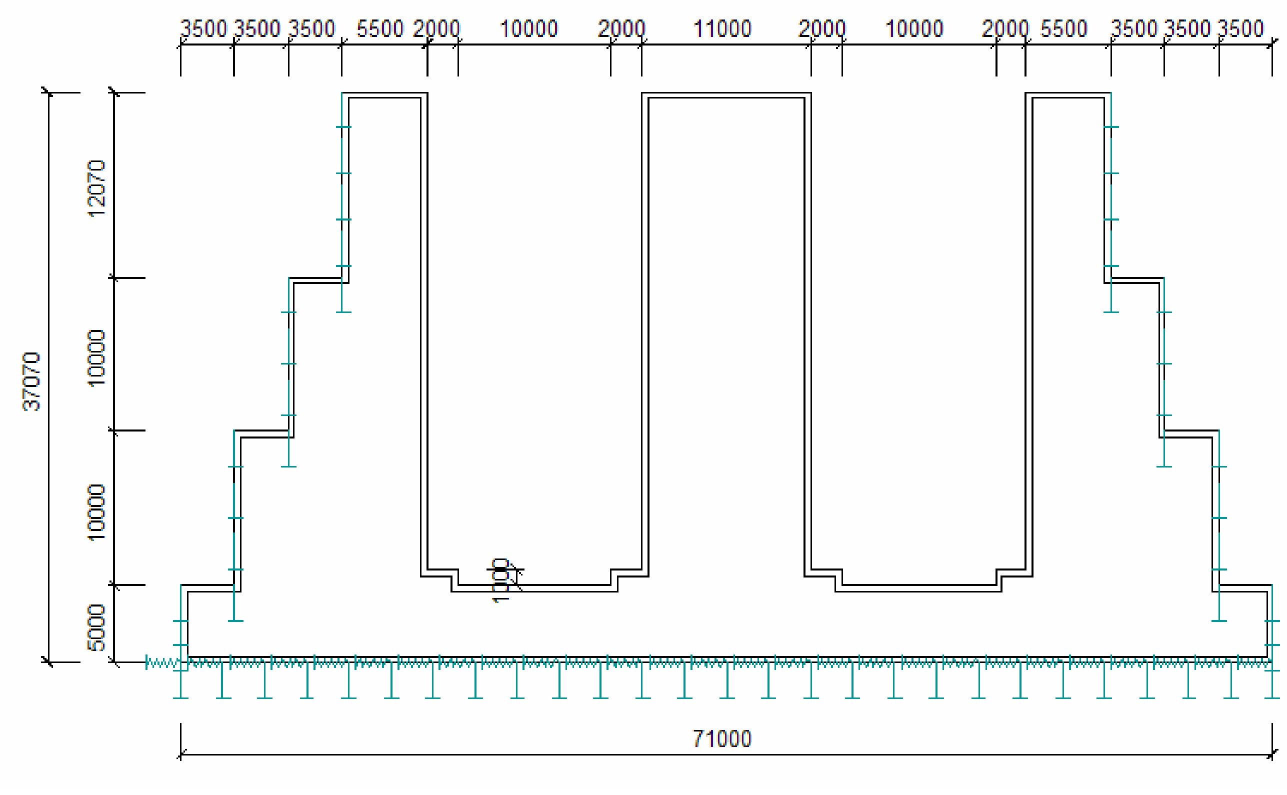
2.3.4 SOFTWARE

The calculations are made by means of “Esa Prima win”, a finite element program. The model we will use is a 2D-model.

2.4 FOUNDATION IN GATUN ROCK

2.4.1 INPUT DATA

A. Geometry



B. Materials

Material	E kN/m²	Nu	W/V kN/m3	Alpha
CONCRETE	19,620,000.000	0.100	24.525	0.0000100

C. Supports

The ‘Gatun Rock’ base is modeled by means of linear supports.

Vertical lock head elements:
Linear support with K= 100.000 kN/m² is introduced.
This support only works when compressed.

Horizontal rock base:
Linear support with K= 100.000 kN/m² is introduced.
This support only works when compressed. A small horizontal support is given to the rock base, to avoid trivial results of the calculation.

The K-values are estimated values.

D. Loads

D.1 Self Weight (case 1)

D.2 Earth Pressure (case 2)

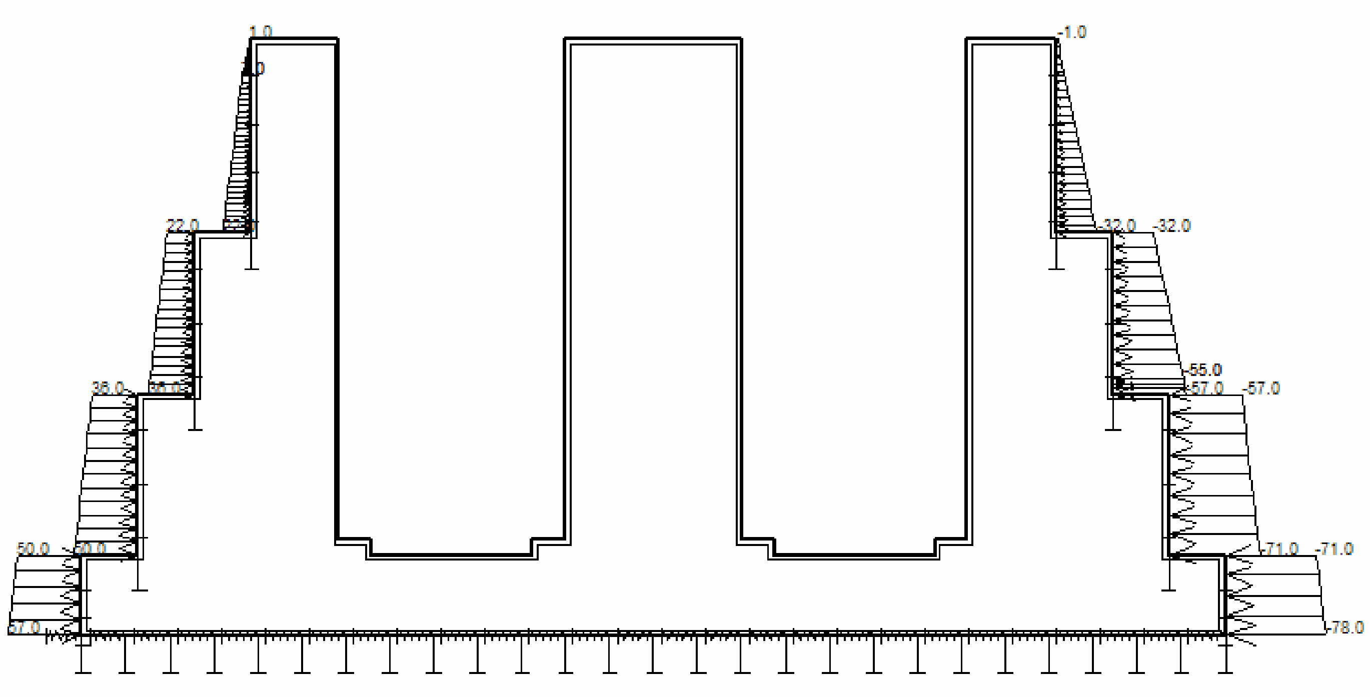
side upper chamber

		lambda ah	lambda av
lambda a	0,1620	0,1403	0,0810
depth (m)	pressure (kN/m²)	ph (kN/m²)	pv (kN/m²)
0	10	1	1
2.33	52	7	4
12.07	159	22	13
22.07	259	36	21
32.07	359	50	29
37.07	409	57	33

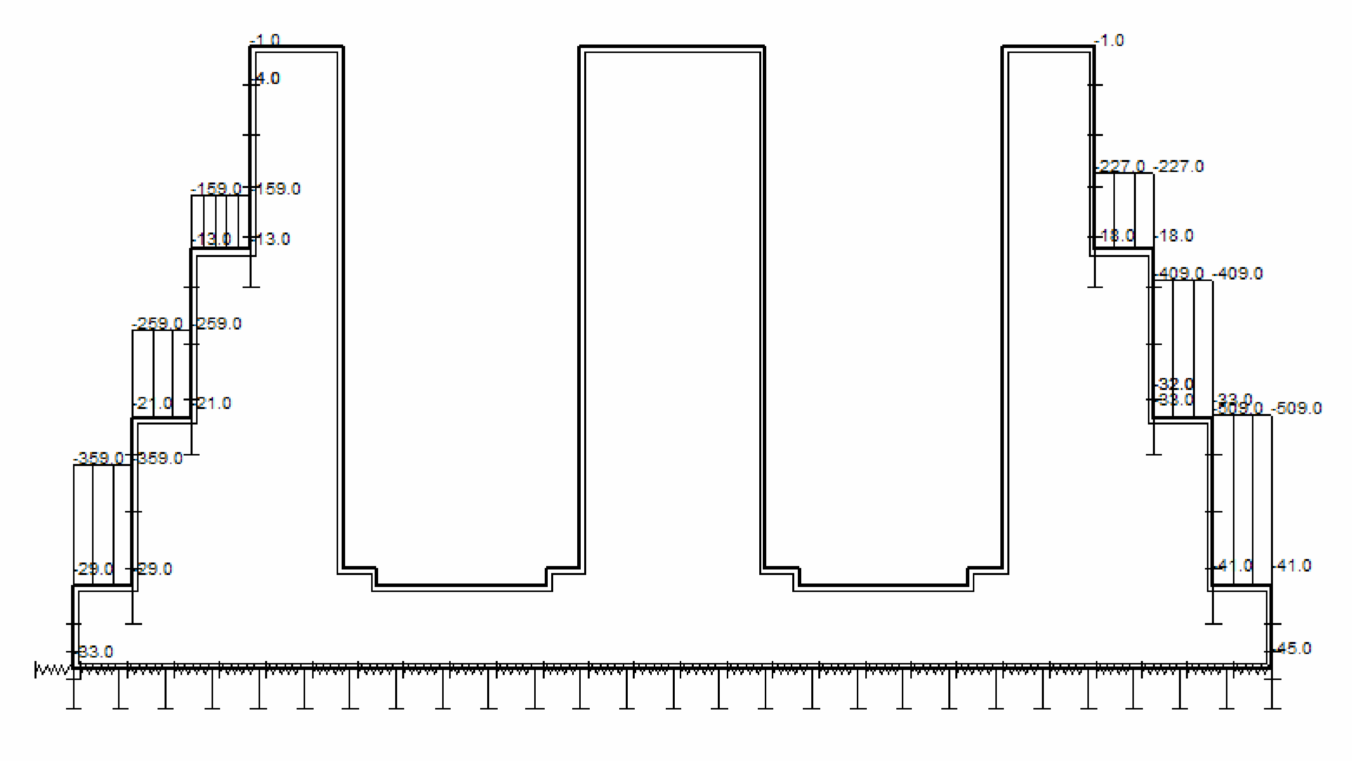
side lower chamber

		lambda ah	lambda av
lambda a	0,1620	0,1403	0,0810
depth (m)	pressure (kN/m²)	ph (kN/m²)	pv (kN/m²)
0	10	1	1
12.07	227	32	18
21.09	390	55	32
22.07	409	57	33
32.07	509	71	41
37.07	559	78	45

Horizontal component



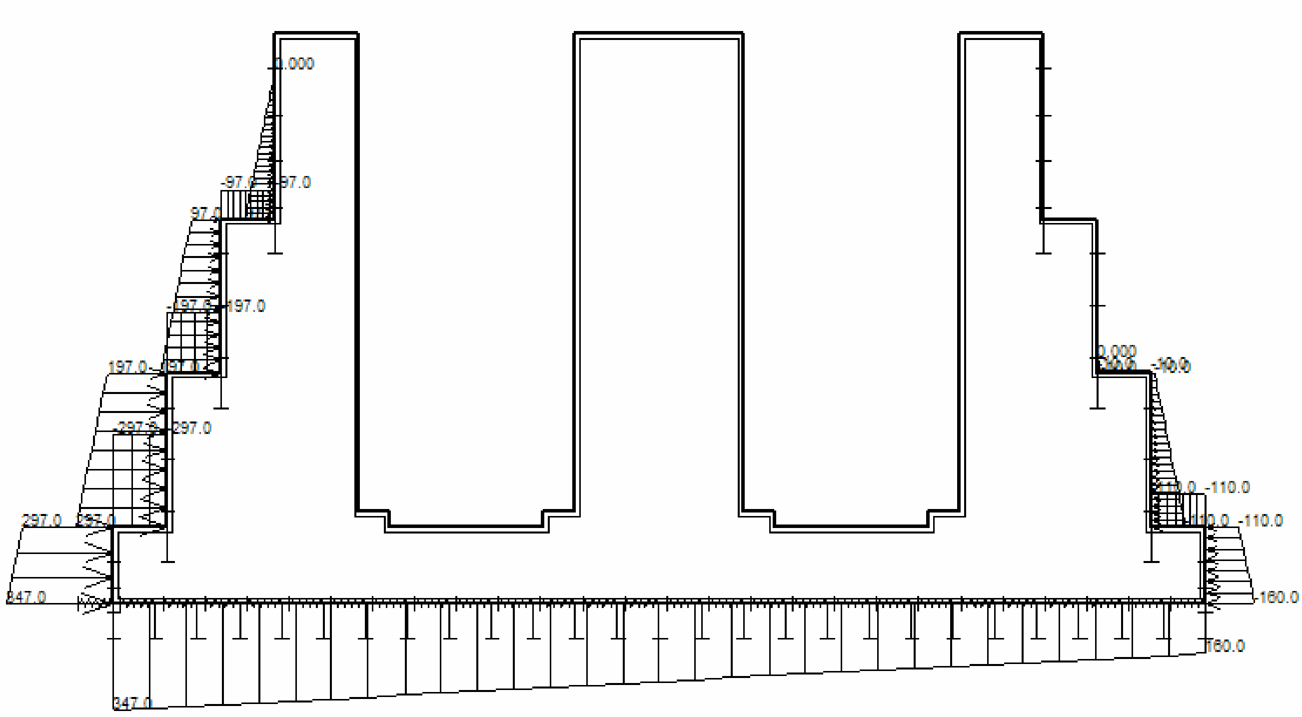
Vertical component



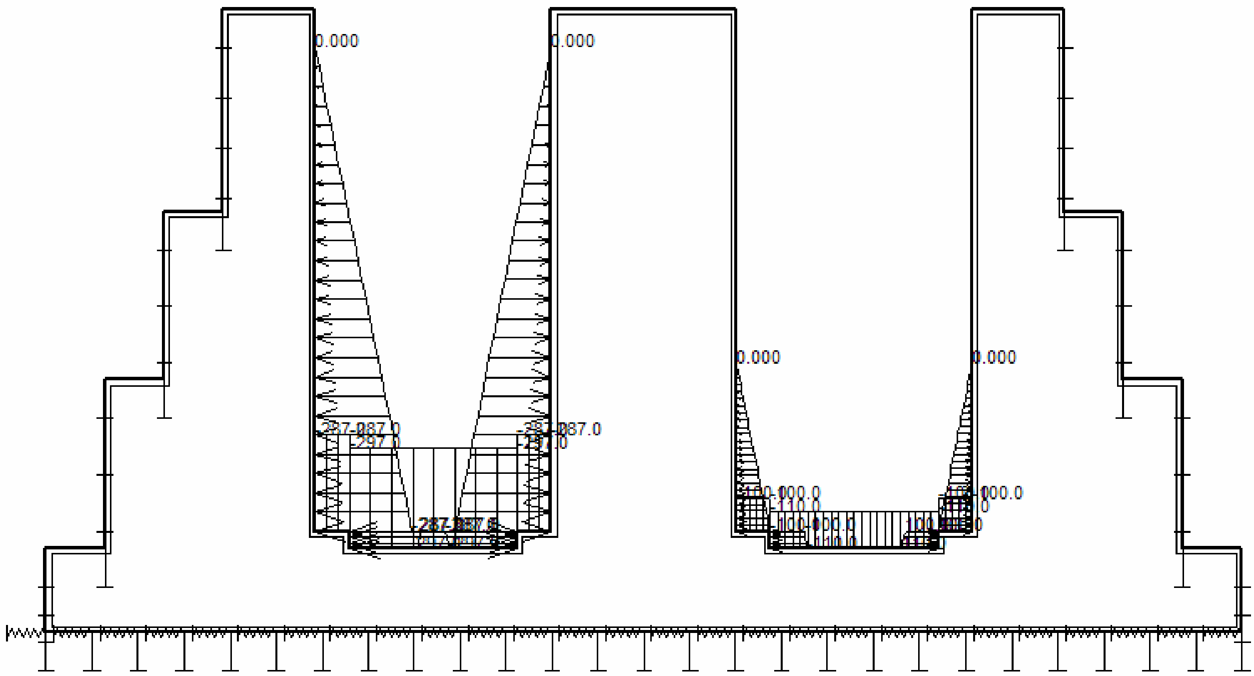
D.3 Water Pressure (case 3)

Water level side upper chamber (maximum): + 27.13 m
Water level side lower chamber (minimum): + 8.37 m

Water pressure onto the sides and bottom of the lock head



Water pressure inside the lock head



D.4 Seismic Loads

The inertia forces on the structural weight are calculated as follows:

$G_v = (1-k_v)\ G$

$G_h = k_h\ G$

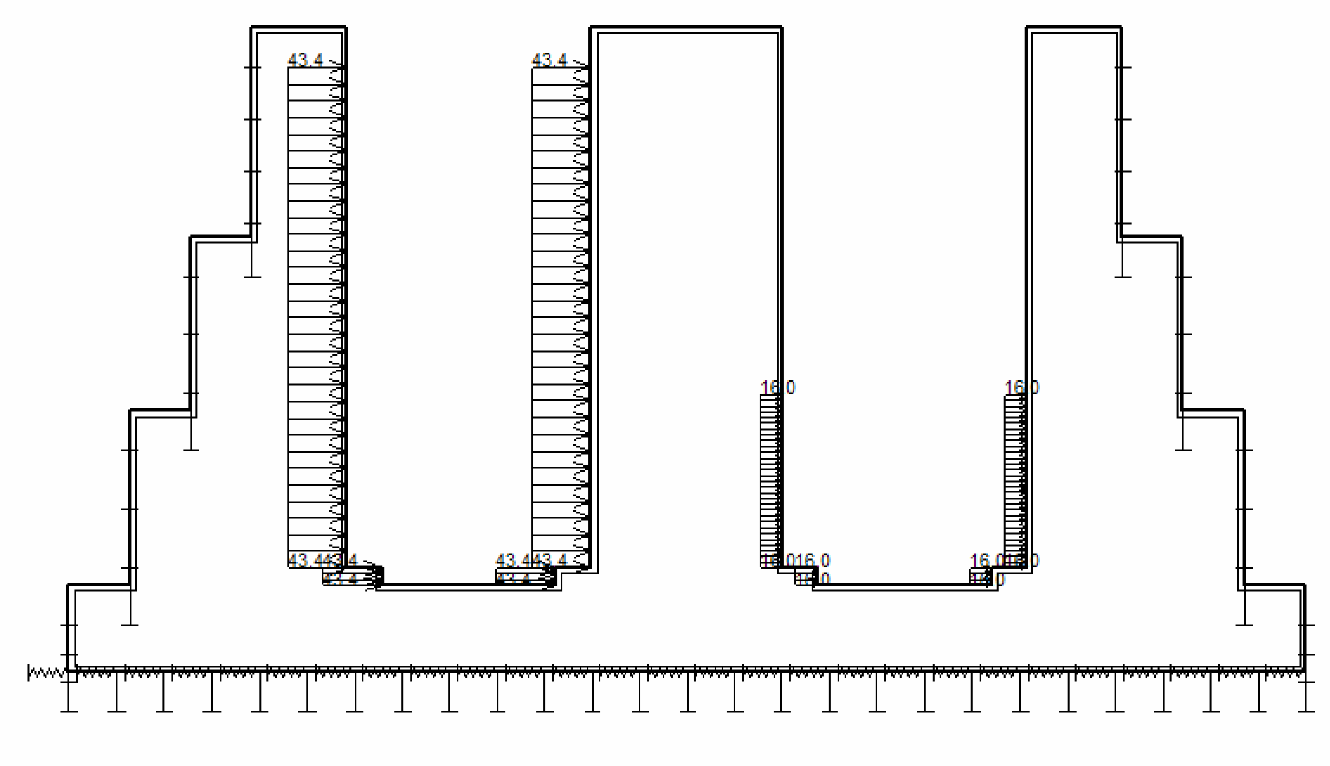
With G the weight of the structure including the backfill above the heel or toe of the structure and any water within the backfill

G_v the vertical component of the weight in seismic conditions

G_h the horizontal component of weight in seismic conditions

According to the PIANC regulations for Seismic design of Port Structures, the earth pressures due to seismic action are calculated using the Mononobe-Okabe equation. The inertia forces are then accounted for by considering a seismic inertia angle, Ψ = atan $(k_h/(1-k_v))$, in which k_h represents the horizontal seismic coefficient or the modified horizontal seismic coefficient for dry and submerged layers respectively.

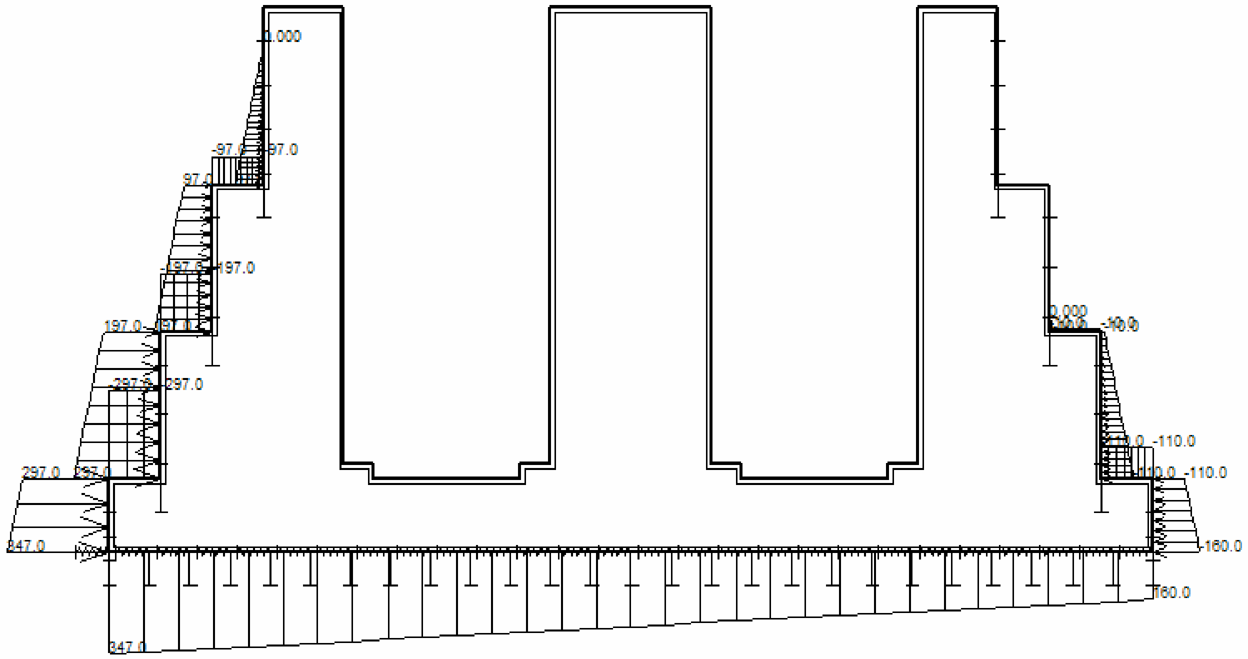
Additional water pressures generated by seismic action are taken into account according to Westergaard formula. The seismic water pressure load case is valid for a unit value of k_e, and is multiplied by the respective k_e value in the corresponding load combinations.



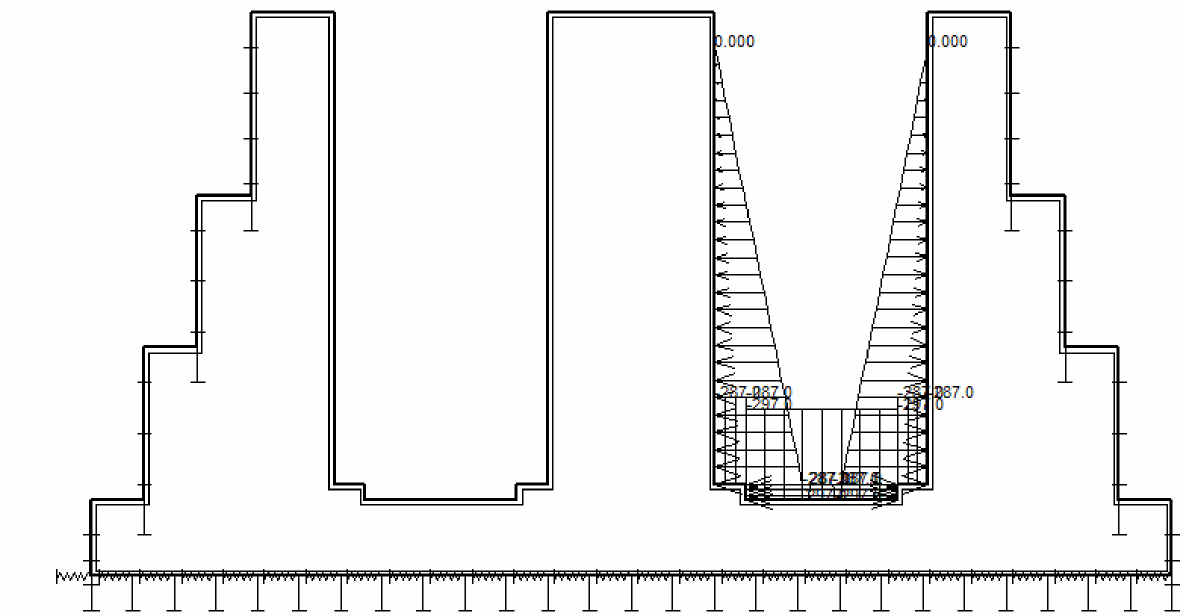
D.5 Water Pressure in Dry Lock Conditions (case 5)

In case 5 the one recess on the side of the upper chamber is put in dry conditions. In case 6 both recesses at put in dry conditions.

Case 6: normal water pressure without water pressure inside the lock head



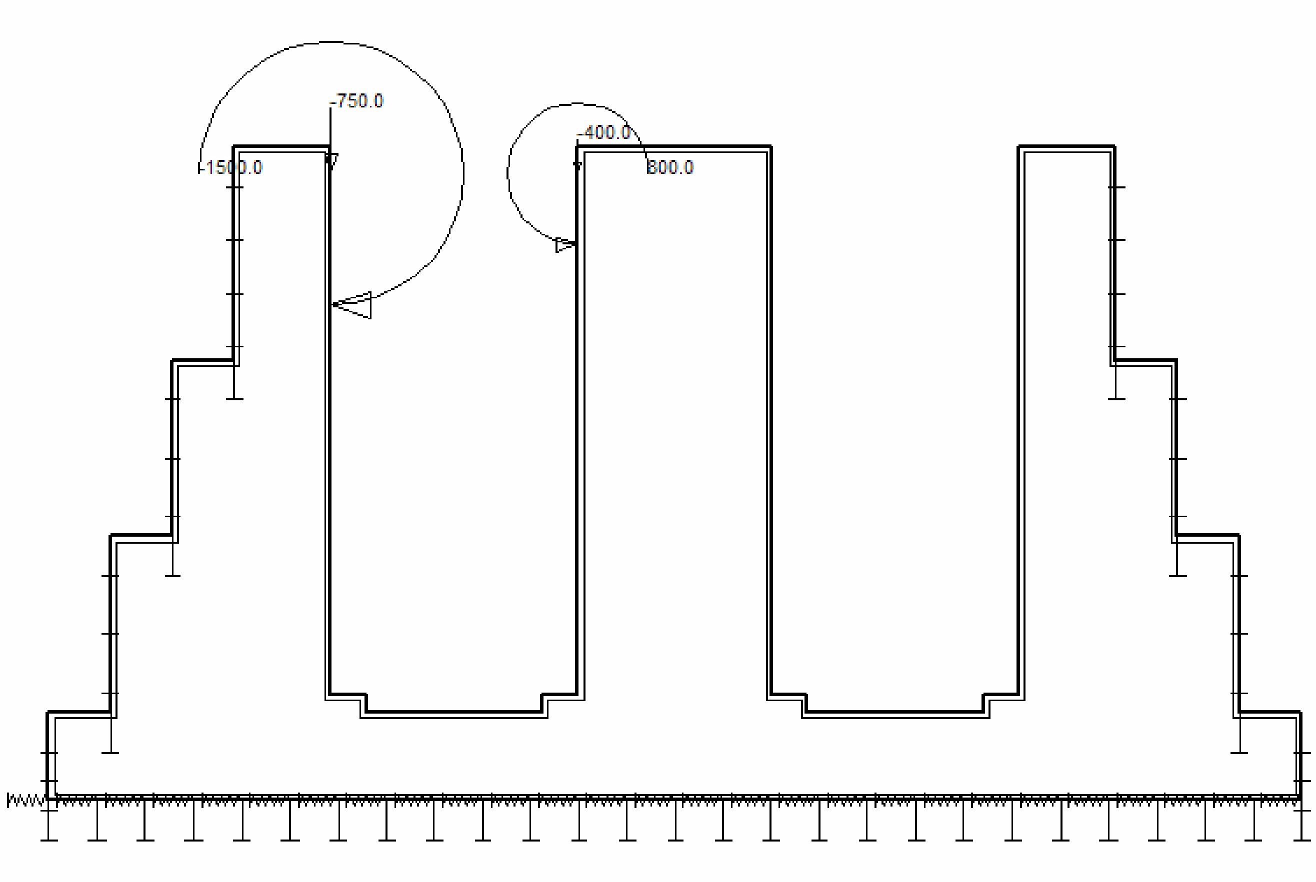
Case 5: normal water pressure combined with a maximum water level in the gate recess near the lowest chamber



D.6 Upper wagon (case 7)

Loads due to the upper wagon are taken from report R4-D-403.

Gates 2,3		Side of flooded zone	Other side	Total vert. reaction
Base case: H = 11 m	Upper wagon	1,500 kN	800 kN	2,300 kN
	Lower wagon	1,900 kN	1,200 kN	3,100 kN



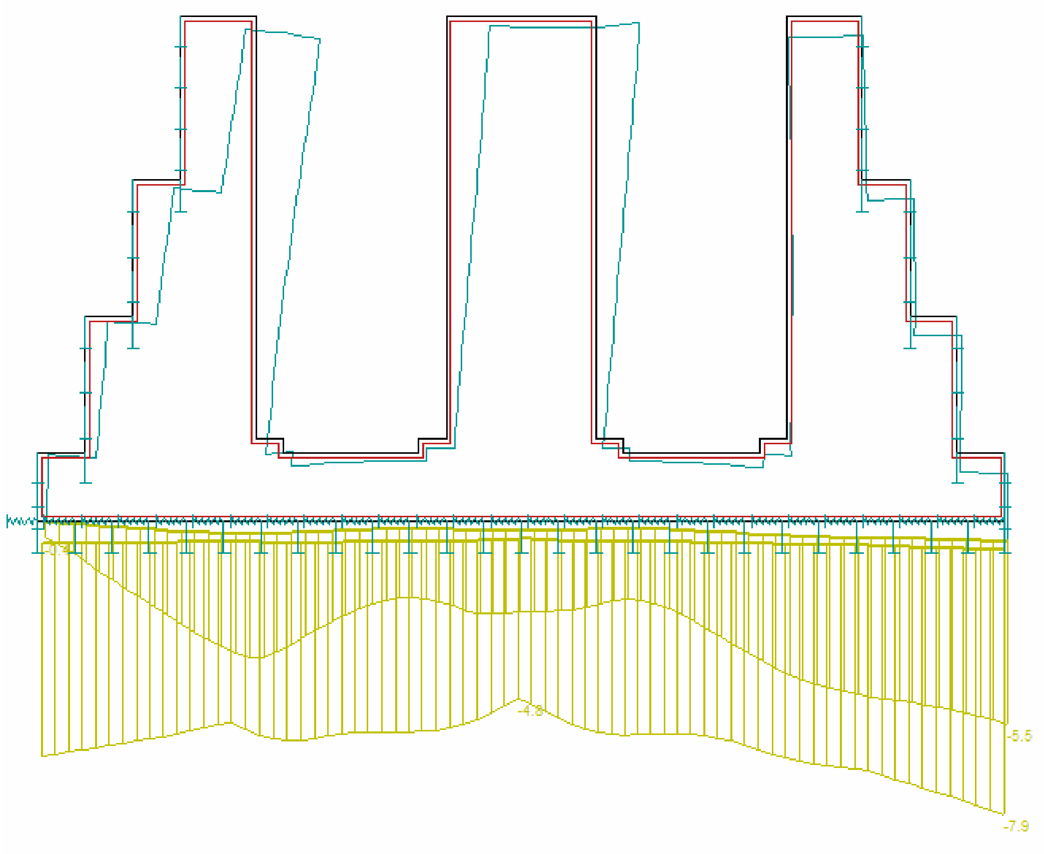
2.4.2 LOAD BEARING CAPACITY

A. Fundamental Load Combinations 105 to 110

Criterion: $\gamma_d \,.\, q_{ref} < q_u$
with $\gamma_d \,. = 1.4$

The maximal vertical displacement at the bottom of the wall is 7.9 mm
This means a vertical pressure of 0.0079 x 100,000 kN/m² = 790 kN/m² = 0.79 Mpa

Vertical displacement

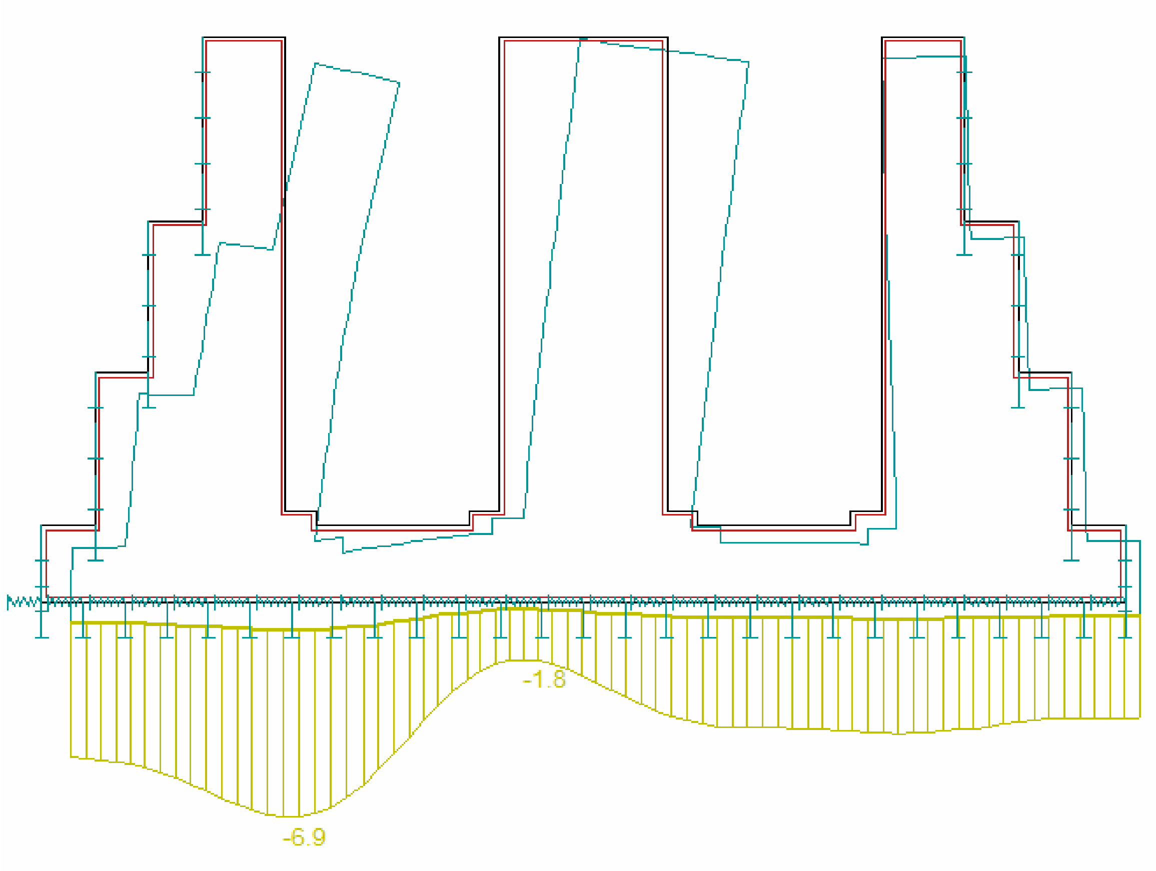


B. Accidental load combination 102 - MCE

Criterion: $\gamma_d \cdot q_{ref} < q_u$
with $\gamma_d . = 1.0$

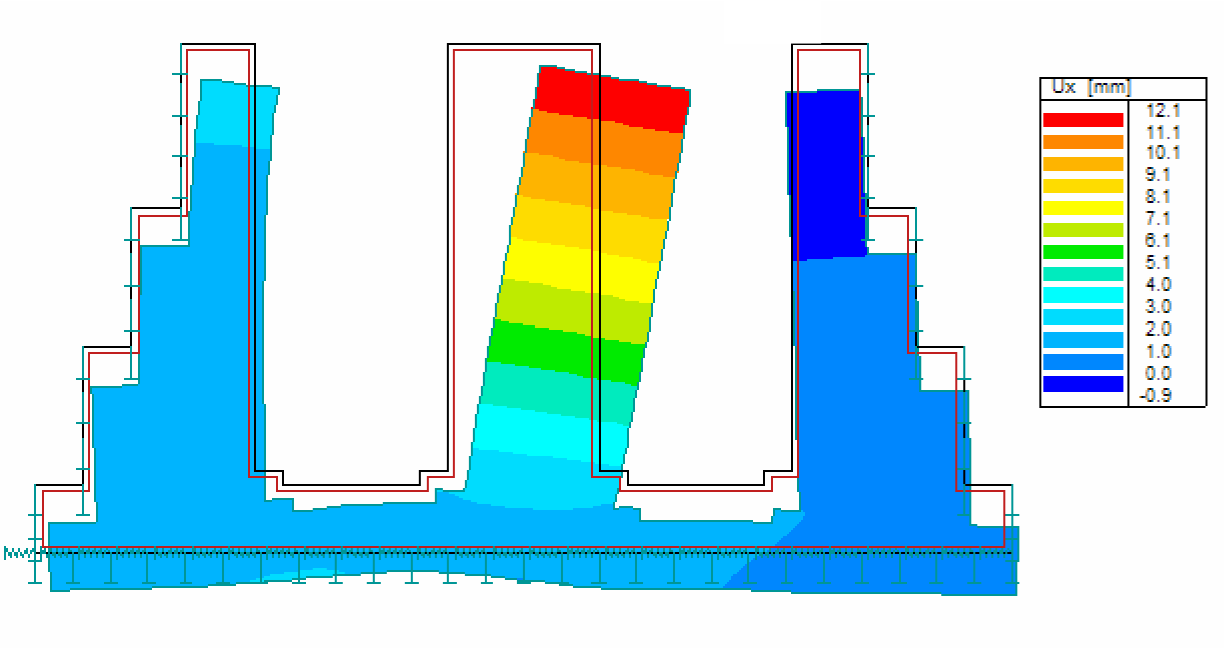
The maximal vertical displacement at the bottom of the wall is 6.9 mm
This means a vertical pressure of 0.0069 x 100,000 kN/m² = 690 kN/m² = 0.69 MPa

Vertical displacement

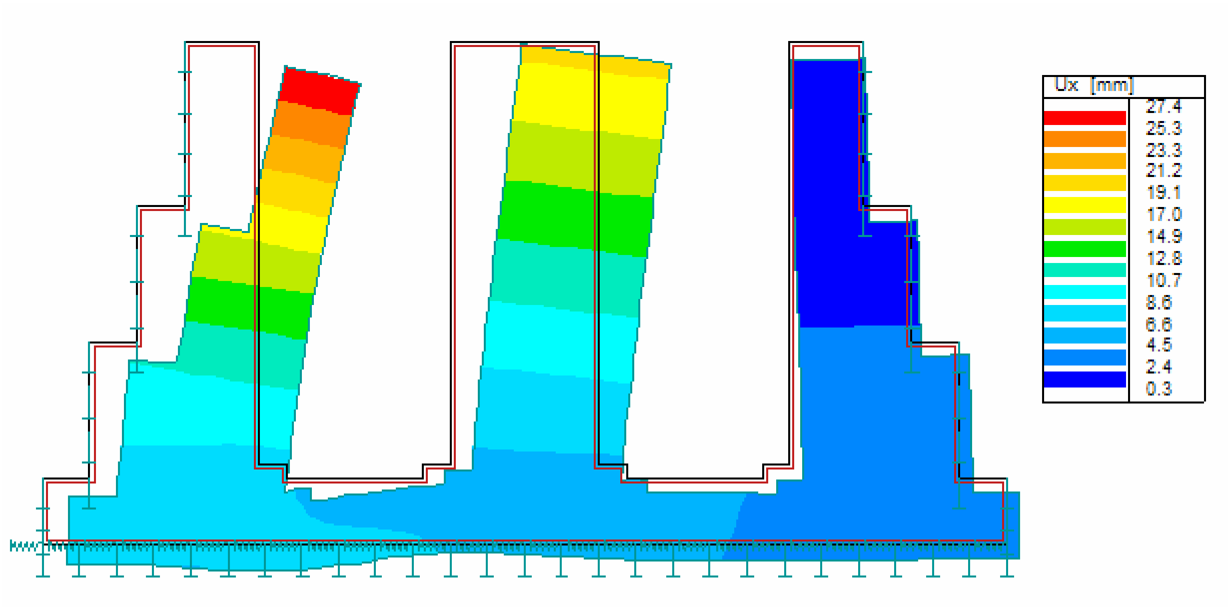


2.4.3 DEFORMATIONS

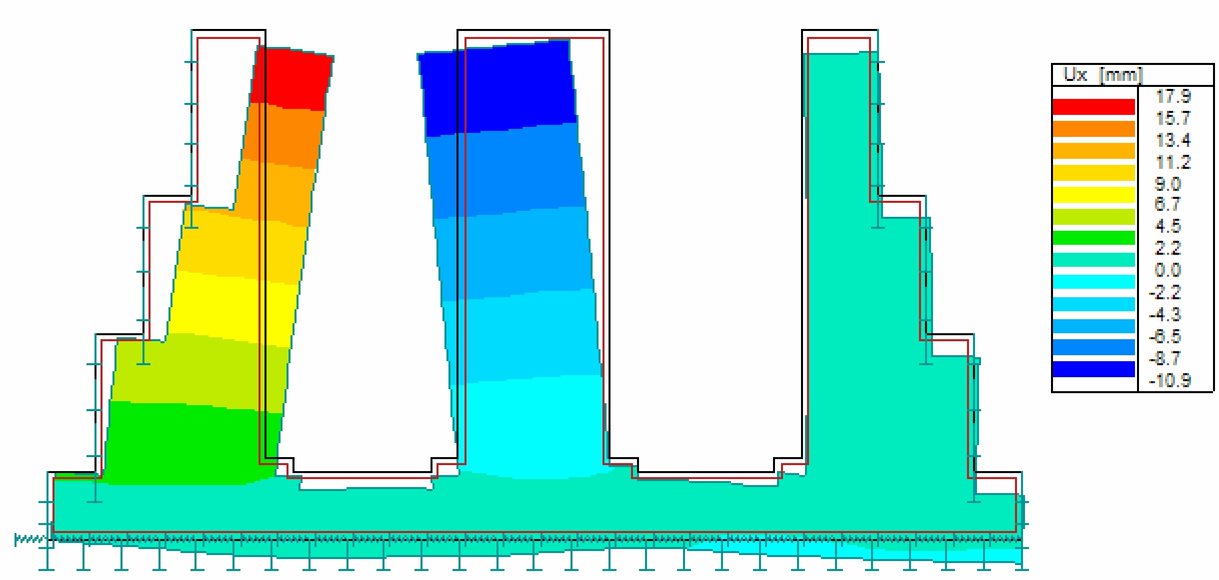
A. Load Combination 101 (Lock in Service)



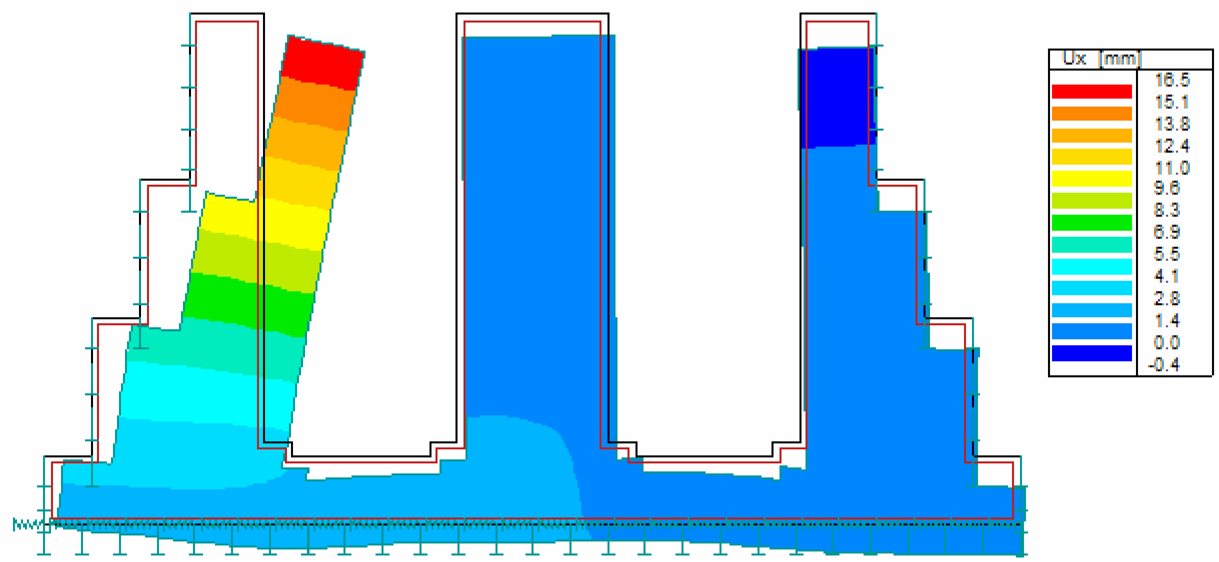
B. Load Combination 102 (MCE)



C. Load Combination 103 (1 gate recess in dry conditions)



D. Load Combination 104 (2 gate recesses in dry conditions)



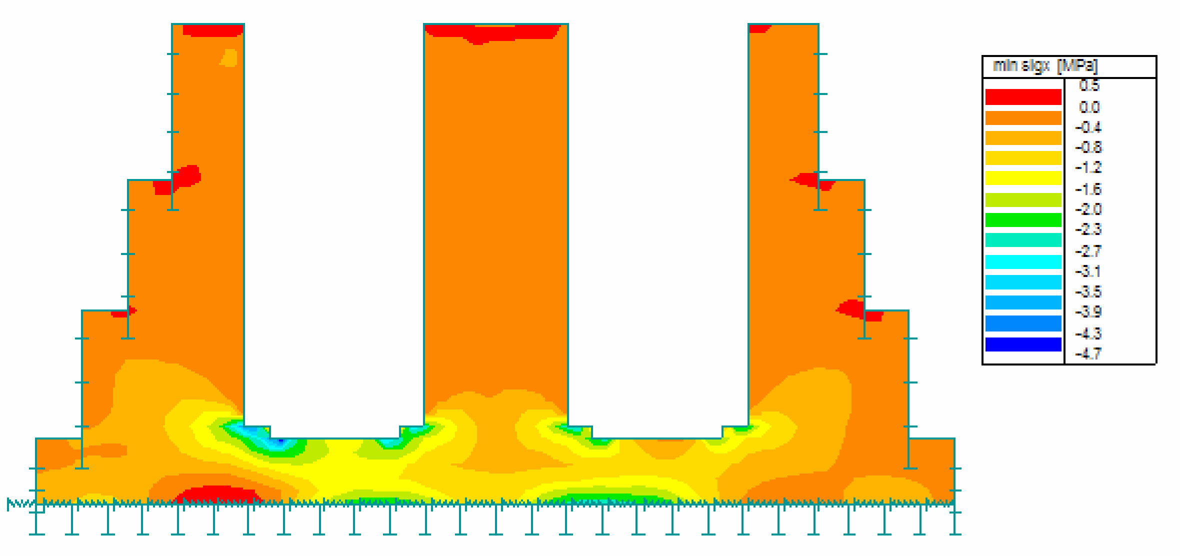
2.4.4 CONCRETE STRESSES

A. Fundamental load combinations 105 tot 110

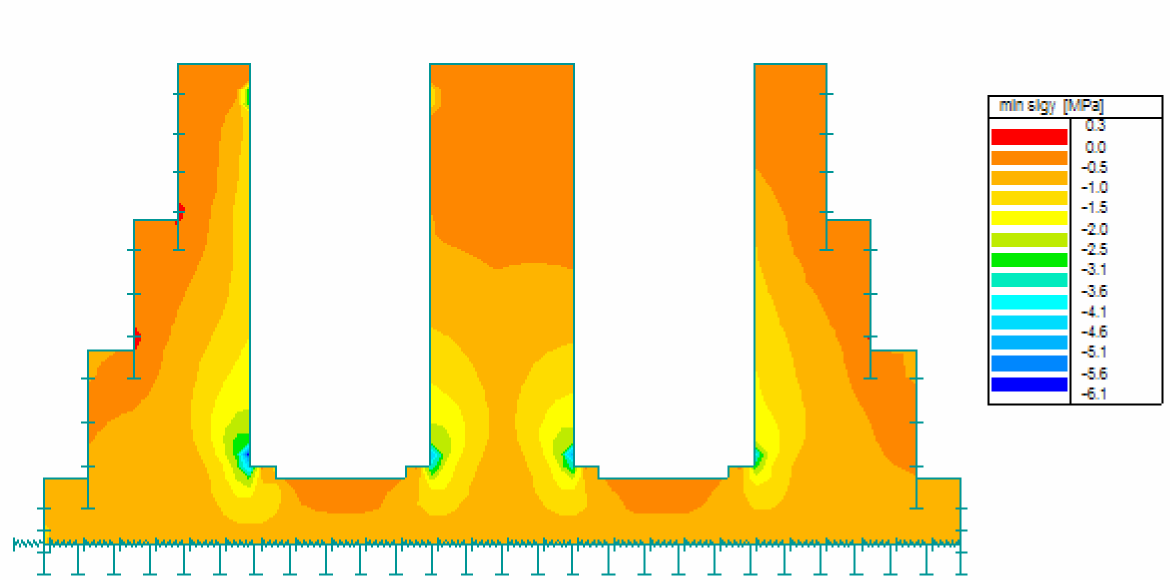
Criterion: $1.125 \times \sigma c < 0.85\ fck/1.5$

This means $\sigma c < 12.5$ N/mm²

Stresses in horizontal direction – maximum compression = 4.7 N/mm²



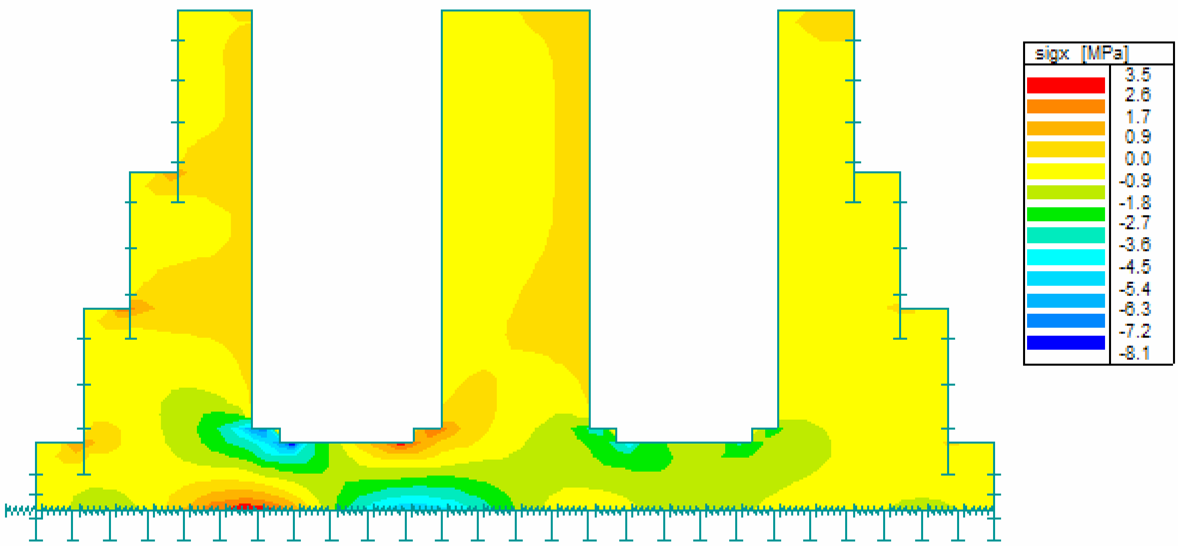
Stresses in vertical direction – maximum compression = 6.1 N/mm²



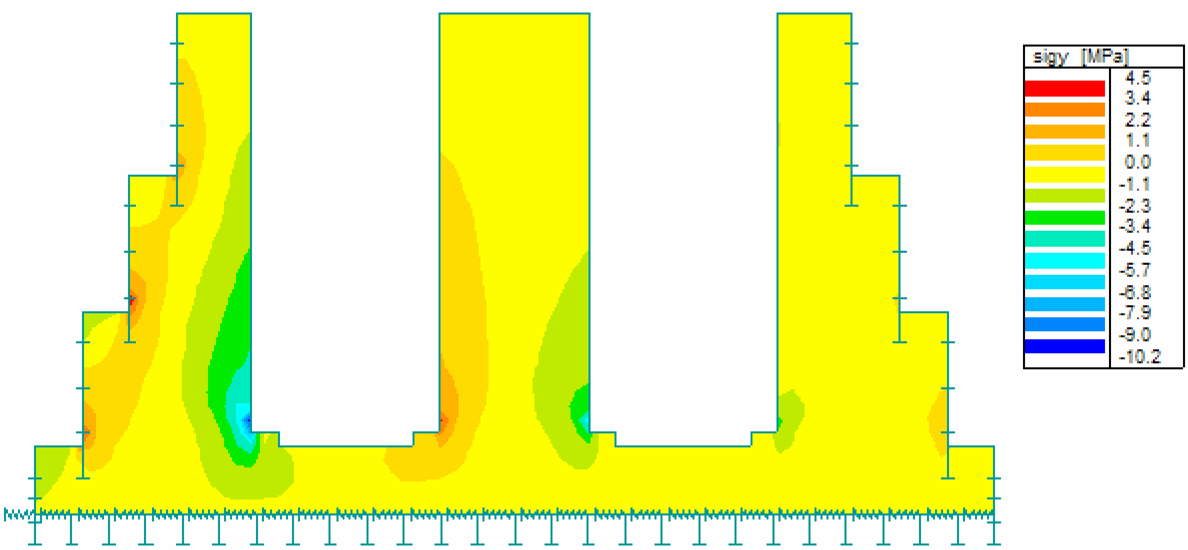
B. Accidental load combination 102 - MCE

Criterion: σc < 0.85 fck
This means σc < 21.25 N/mm²

Stresses in horizontal direction – maximum compression = 8.1 N/mm²



Stresses in vertical direction – maximum compression = 10.2 N/mm²



2.4.5 SUMMARY

	quasi permanent load combination - service	accidental load combination - MCE	rare load combination - dry gate recess	rare load combination - dry gate recess	Fundamental load combination	Fundamental load combination	Fundamental load combination	Fundamental load combination	Fundamental load combination	Fundamental load combination
	101	**102**	**103**	**104**	**105**	**106**	**107**	**108**	**109**	**110**
Load Bearing Capacity										
$1.4\ q_{ref} < q_u$	-	-	-	-	OK	OK	OK	OK	OK	OK
$q_{ref} < q_u$	-	OK	-	-	-	-	-	-	-	-
Deformations										
Max. hor. displacement (cm)	1.21	2.74	1.79	1.65	-	-	-	-	-	-
Concrete stresses										
$1.125 \times \sigma_c < 0.85$ fck/1.5	-	-	-	-	OK	OK	OK	OK	OK	OK
$\sigma_c < 0.85$ fck	-	OK	-	-	-	-	-	-	-	-

2.5 CONTACT STRESSES

2.5.1 BOTTOM SEAL

A. Forces onto the bottom seal

Maximum horizontal reaction forces at the sill bearing occur during retaining of the (maximum) water level difference. They result directly from the 2D-beam grid model computations presented in appendix A of the report ‘lay out of rolling gates’ after an optimization of the stiffness of the cantilevering bottom ends of the vertical frames. Doing so uplift from the sill bearing of the skin plating at the gate bottom edges is avoided.
To derive the maximum contact stress, the minimum spreading length (along the azobé wooden beams of 500 mm width) is indicated in the tables below.

The results mentioned in the report of the lay-out of the rolling gates are the following:

	Reaction force at frame (3.15 m length)	Max. contact stress (min. 540 mm)
Gate PA1	7,500 kN (ULS)	28 N/mm² (ULS)

	Reaction force at frame (3.18 m length)	Max. contact stress (min. 540 mm)
Gate PA2-3	11,400 kN (ULS)	42 N/mm² (ULS)
Gate PA4	10,400 kN (ULS)	39 N/mm² (ULS)

B. Calculation of the contact stresses

The bottom seal is made of two basalt (or a similar material) elements: one for the direct contact with the gate and one making the placement of a habitat possible.

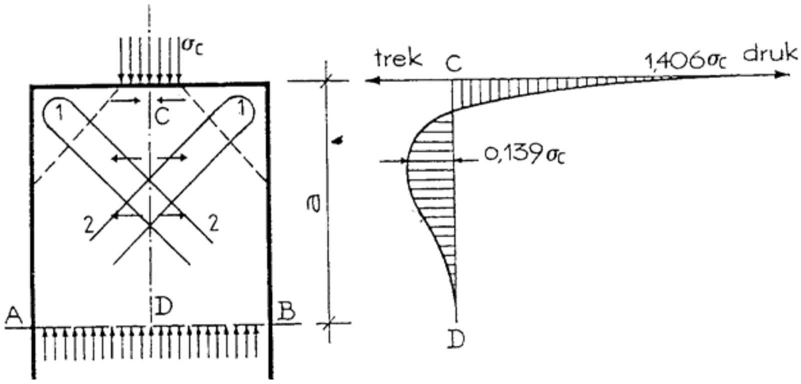
Because of the gap in the second stone we can not spread the force vertically. We use in our calculation a contact height of 35 cm.

Horizontally the force can be spread over an angle of 45 degrees. A contact width of (200 cm + 54 cm + 200 cm) = 454 cm is obtained.

Thus the contact stresses onto the concrete can be derived:

$$11{,}400 \text{ kN} / (35 \text{ cm} \times 454 \text{ cm}) = 7.17 \text{ N/mm}^2$$

The maximum tensile stresses within the seal caused by the contact stress can be estimated by formulas in literature.



$\sigma_t = 0.139 \times \sigma_c$
$\sigma_t = 0.139 \times 42 \text{ N/mm}^2 = 5.84 \text{ N/mm}^2 < 6.67 \text{ N/mm}^2 = 10 \text{ N/mm}^2/1.5$

The tensile strength is kept below the admissible tensile strength of the material, which is mentioned in the design criteria. No additional safety factor is used for the contact stresses because the dry lock condition is considered as a rare load combination (safety factor = 1).

2.5.2 LATERAL BEARINGS

A. Forces onto the lateral bearings

Maximum horizontal reaction forces at the lateral bearings (on the lock walls) occur during retaining of the (maximum) water level difference. From the 2D-beam grid model computations presented in appendix A of the report 'lay-out of rolling gates', the reaction forces at the horizontal frames are obtained (corresponding to the same optimized stiffness distribution). Subsequently the part of the vertical frame structure at the contact with the lock walls is modeled as a continuously elastically supported beam, which is loaded by these reaction forces. The maximum contact stresses are listed below.

The results mentioned in the report of the lay-out of the rolling gates are listed below.

Gate 1	Max. contact stress (at R1)
	35 N/mm² (*)

Gates 2,3,4	Max. contact stress at R1	Max. contact stress at R3
	45 N/mm²	52 N/mm² (*)

(*) Based on our experience with 3D-modelling of the recently designed new rolling gates for the Van Cauwelaert lock (port of Antwerp) we expect that the contact stresses at R4 are overestimated at least by 10 N/mm² due to the underestimation of the beam stiffness at the air chamber. Therefore the contact stress at R1 should be considered as determinative, like in the case of Gate PA1.

B. Calculation of the contact stresses

The vertical seal is made of a basalt (or a similar material) element. The sealing at the side with the gate recesses is different from the one on the other side, because it has to work as seal for the bulkhead.

The maximum contact stresses that are given remain constant along several meters in height. Therefore we cannot spread the force in the vertical direction.

Horizontally the force can be spread over the width of the sealing stone. A contact width of 160 cm has been obtained.

Thus the contact stresses onto the concrete can be derived:

$$52 \text{ N/mm}^2 \times 500\text{mm}/1600\text{mm} = 16.25 \text{ N/mm}^2$$

The maximum tensile stresses within the seal caused by the contact stress can be estimated by formulas in literature.

$$\sigma_t = 0.139 \times \sigma_c$$
$$\sigma_t = 0.139 \times 45 \text{ N/mm}^2 = 6.26 \text{ N/mm}^2 < 6.67 \text{ N/mm}^2 = 10 \text{ N/mm}^2/1.5$$

The tensile strength is kept below the admissible tensile strength of the material, which is mentioned in the design criteria. No additional safety factor is used for the contact stresses because the dry lock is considered as a rare load combination (safety factor = 1).

3 Literature

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Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual Design
Contract SAA-143351

ATLANTIC LOCKS HARMONIZATION

TASK A4c – Emptying and Filling System

Rev A

Tractebel Development Engineering
TECHNUM
COYNE ET BELLIER
Compagnie Nationale du Rhône

in association with

SBE
PORT OF ANTWERP
FITA AOSO

ATLANTIC SIDE HARMONIZATION - List of annexes

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0 Executive summary

This second report on hydraulics deals with the Atlantic side harmonization of the studies performed for the locks at the Pacific side of the Panama Canal. It follows a first one, concerning the Pacific side actualization study.

Following the recommendations, choices and Terms of Reference of ACP, a system with three lock chambers equipped with 3 water saving basins each, allowing **to save nearly 87 %** [1]**of the total water required to lock 1 ship** (semi convoy mode), has been retained.

The chambers and water saving basins levels have been set up using the same software as for the former studies. This software gives the minimum and maximum water levels reached in the chambers and the 9 basins and provides the water usage and the water saving rate for every lockage as well as the daily number of up- and down lockages.

Based on the results of the Pacific side configuration, the scope of work of this configuration identifies **a side-wall culvert and ports filling and emptying system to be studied.**

The system has been modeled and pre-designed with the hydraulic calculation software Flowmaster™.

A first stage consisted in running calculations with the same hydraulic system (i.e. culvert, conduit & port size, schedules of the valves …) as the one designed for the Pacific locks, which led to a feasible but not optimized configuration.

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1 *The three-step lock system saves 2/3 of the volume of a **single lock** chamber.*

Moreover, the water saving basins save: $e = \dfrac{n}{n+2}$ *= 60 % of the 1/3 remained (with the same area for WSB and lock chamber : m= wsb area / lock area = 1), where n= 3 is the number of water saving basins per lock*

So, the total water saving rate is e':

$$e' = \frac{2}{3} + \frac{1}{3}e = \frac{3n+4}{3n+6}$$

For n=3, e' = 0.87, i.e. 87 %

In a second stage, an adaptation of the WSB conduits and valves has been tested in order to optimize the construction costs. Because of the smaller tidal amplitude in the Atlantic Ocean than in the Pacific Ocean, the maximum heads between lock chambers and WSBs are also smaller and the size of the WSB conduits can thus be reduced.

On the other hand, the average head between Gatun Lake and the Ocean does not change compared to the Pacific side, neither the maximum head between two lock chambers. Consequently, the main longitudinal culverts are given the same dimensions as the Pacific ones

The calculated filling and emptying times fit in with the times required by the TOR and/or the design values. The velocities reached in culverts, conduits and ports are acceptable, taking into account that the maximum velocities could be reduced by providing adapted shapes to the circuits' components (especially ports) on one hand and opening and closing rates of the valves on the other hand.

The system provides quite a uniform flow distribution and an upstream-downstream and east-west balanced filling. The remaining dissymmetry, that could be responsible for strengths on the hawsers exceeding the acceptable level, has been examined in the Pacific side study. Due to the fact that hydraulic conditions are more favorable on this side of the canal, no new hawser forces analysis was made.

Nevertheless, solutions to further reduce the hawser forces would be identical as those proposed for the Pacific side:

- Modifying the valve opening diagram;
- Modifying the ports dimensions.
- Concentrating the ports towards the gravity center of the lock chamber

The next stage, which is defined as the preliminary design of the filling and emptying system should mainly allow to:

- Optimize the culvert, conduit and port dimensions, shape and number of ports;
- Optimize the valve opening/closing schedule;
- Define the port distribution along the lock chamber, their position and orientation;
- Evaluate accurately the expected strengths on the hawsers.

This stage will require a detailed study with Flowmaster™ in combination with the 2D/3D Delft numerical model and finally on a physical scale model.

This conceptual design is made for locks using water saving basins, according to the TOR. If the water saving basins are not used, the E/F times may have to be increased, or the E/F system may need to be adapted / modified, as the heads could be much larger in certain configurations..

1 Foreword

1.1 CONTRACT

This report is performed within the scope of the contract n° SAA-143351 awarded on November 24, 2004 to the Consortium named CPP (Consorcio Post Panamax) by the client ACP (Autoridad del Canal de Panama).

This report concerns the second part of the contract: new Panama Canal lock system conceptual design harmonization study, Atlantic side.

This new contract was awarded to update the previous studies by integrating some technical modifications (lock dimensions, ship handling system...), and to harmonize the study at the Atlantic side (see 1st part of the contract: Pacific locks Actualization)

1.2 SCOPE OF WORK

This report deals with the harmonization of the triple lift lock study on the Atlantic side of the Panama Canal.

The harmonization is performed on the basis of the former studies realized by CPP and takes into consideration the choices made by ACP:

- Using a 3 lift lock system, each lock being equipped with 3 water saving basins;
- Using tug boats instead of locomotives as positioning system;
- Operation of the locks with rolling gates;
- Using a filling and emptying hydraulic system with longitudinal side wall culverts and ports;
- Reduction of the lock width from 61.00 to 55.00 m.

The scope of work of the harmonization study mainly consists in:

- Setting the levels of the chamber and the saving basins, using the new data: Gatun lake levels, double sinusoid for the Atlantic levels (the ratio between the lock chamber and the WSB area remains equal to 1);
- Calculate the heads between the different pools (Gatun lake, chambers, water saving basins, Atlantic Ocean);
- Calculate the water usage and the water saving rate;
- Optimizing the F/E system using Flowmaster ™ ;
- Designing at a conceptual level the hydraulic system of the solution described above;
- Determining approximately the number of ships passing through the locks in the semi convoy mode.

2 Introduction

2.1 BACKGROUND

The actual locks of the Panama Canal are going to be saturated in less than ten years. In addition, the new Post Panamax vessels cannot pass through the locks.

The “Autoridad del Canal de Panama” is conducting a study to evaluate the feasibility of a third lane of locks for bigger vessels. However, there is a lack of water at the present time to operate the new locks, that’s why new solutions have to be found.

None of these kinds of locks have yet been designed. The engineering work starts with a conceptual design study. The purpose of this study is not to design the locks in detail, but to allow ACP to choose the best solution according to the following subjects: hydraulic filling and emptying systems, water saving basins, type of gates, civil works, electro-mechanical equipment,…

2.2 BRIEF DESCRIPTION

The Panama Canal is mainly composed of the Gatun Lake, and two sets of locks, one on the Atlantic side and one on the Pacific side of the Canal.

The existing Atlantic side locks are composed of two lines each of them equipped with three lock chambers.

Each lock chamber has the following dimensions:

- Useful length : 305 m
- Useful width : 33.50 m
- Maximum vessel draft : 12 m (39.5 ft) in tropical fresh water

The maximum tidal range on the Atlantic coast is approximately 0.5m; the level of the Gatun Lake can be lowered by a little less than 2 m during the dry season (from 26.67 m PLD to 24.84 m PLD). ACP plans to raise the Gatun lake maximum level, in order to improve the channel transit capacity (see new levels further)

All the locks are able to handle 65,000 dwt ships, called Panamax ships, scheduled in semi-convoy mode, i.e. the ships sail 12 hours in one way and the next 12 hours in the other way.

The present locks will be saturated in a few years. In addition they are not able to pass the larger ships: 105,000 dwt container vessel, 140,000 dwt bulk carrier and further up to 200,000 dwt ships, called Post Panamax. The ACP (Autoridad del Canal de Panama) has then decided to investigate possibilities to construct new and larger locks.

The main issue from a hydraulic point of view is the lack of water to operate these new locks. Existing locks are already consuming nearly all the available water of the watershed. During the dry season, the resource is lower than the consumption and the level of the lake decreases. In addition, some other needs (municipal and industrial), increase continuously, competing with the water demand required to operate the locks.

The present hydraulic project has two main purposes:

- Design efficient hydraulic circuits for filling and emptying operations, and for limited hawser forces
- Propose water saving systems (in addition to new resources)

The present project on the Atlantic side consists in the stepping of the head at one single place, by means of a triple lift lock system to the East of the present alignment.

3 Terms of reference

3.1 LEVELS

Note: PLD (Precise Level Datum) is the reference system for Panama Canal. All levels in this report refer to that reference.

3.1.1 GATUN LAKE

Maximum level: +27.13 m PLD (89 ‘)
Minimum level: +24.99 m PLD (82 ‘)

3.1.2 ATLANTIC OCEAN

Ranging from – 0.15 m to +0.41 m PLD

3.2 SIZES - DIMENSIONS

- Useful length of the locks : 426.72 m
- Useful width of the locks : 55.00 m
- Draft (**minimum** water over the sills) : 16.76 m
- Freeboard : 2.13 m (to be verified)

3.3 WATER SAVING RATE

The three-step lock with 3 water saving basins per lock system shall save 60 % per lock and 87 % * of the total water required to lock a ship.

** see footnote page C/0-1*

3.4 OPERATING TIME

It is desired to keep the filling or emptying time in an 8-10 min lap time without using the water saving basins for a single lift of the triple-lift lock system. This time is given rather as a guideline than as a TOR, as the TOR of the study are set up for locks operating with water saving basins.

The time needed for a lockage using the water saving basins is not specified, nevertheless it must not exceed too much the filling /emptying times without using water saving basins. The guidelines of the times are given in Chapter 5.1.

3.5 NUMBER OF SOLUTIONS TO BE STUDIED

Only one solution for the filling and emptying system has to be studied in this harmonization study. It is the system with side wall culverts and ports.

3.6 COST AND MAINTENANCE OF THE SYSTEM

Special attention will be paid to these points:

- A compromise has to be reached between the level of efficiency of the system and the corresponding costs.
- The redundancy of the system is very important to obtain a high level of reliability, minimizing traffic interruptions.

4 Design criteria and assumptions

4.1 DESIGN CRITERIA

Reference is made to the reports:

- R2 - A : Part A General Design Criteria
- R2 - B : Part B Specific Design Criteria
- R4 - C : Filling and emptying system
- P4 – C : Pacific Locks Actualization – E/F system

4.2 ASSUMPTIONS

- Saving 87 % of the lockage water implies the use of 3 water saving basins per step or 9 water saving basins for the global 3 step system (see 3.3), no additional recycling system has been retained by ACP

- The water saving basins will be built only on one side of the locks

- Only three side-by-side water saving basins will be studied

- Considering the operating time, and to be in accordance with the results of the first study of the triple lift configuration, Pacific side, the target time for filling and emptying the chamber using the water saving basins is approximately 51 minutes. This has led to a reduction of the dimensions of the culverts and valves at the Pacific side. The more favorable hydraulic conditions on the Atlantic side will also lead to a further reduction of the WSB conduit size, .

- The surface of the lock which is taken into consideration for the filling / emptying simulations is 27 500 m². It includes the surface between the adjacent gates, the surface of the gate recesses and part of the gate surface (95 %), as shown on the sketch below:

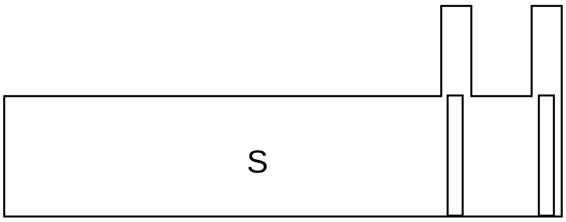


Figure 4.2-a

- The valves will be of the vertical plane valve type, rectangular, with rollers.

- Each valve will be surrounded by slots in order to insert stoplogs for maintenance or repair works. According to ACP's demand, these slots will have the same size as the main valve slots so that they can be equipped with auxiliary valves if required. Access shafts for material and personnel will be provided on both sides of the main valves.

- The scheduling in semi convoy mode, which is the actual way to lock ships, will be retained for the 3rd lane of locks in that configuration.

- The design of the filling system will take into consideration the possibility for smaller ships to pass through the new lock. This will affect in particular the admissible rate of turbulence in the chamber.

5 Levels in lock chambers and water saving basins

All elevations are referred to precise level datum (P.L.D)

5.1 PRESENTATION OF THE METHOD AND INPUT DATA

This study consists in the calculation of the water surface elevation, both in chambers and water saving basins (WSB), anytime during a lockage and taking into account the tidal variation of the ocean. It allows calculating chamber and WSB main dimensions (bottom and top elevation, gate height, etc. …).

All the calculations are carried out with the software developed by the Consultant as already presented in the previous studies.

All the data entered in the program are detailed hereafter:

- **The Atlantic Ocean levels**

Data concerning the tides on the Atlantic side are observed by the consultant from the web site of the SHOM (Service Hydrographique et Océanographique de la Marine) since September 20th at Cristobal.

The observation period runs over 105 days (20/09/04 to 03/01/05). These data are given in annex 1.1.

From the data collected, it appears:

- The maximum value is : + 0.44 m PLD
- The minimum value is : - 0.11 m PLD
- Tides have a period of 12.47 hours (high tide to high tide or low tide to low tide)
- There is a second variation of the tides in amplitude, with a period of 14 days

ACP also provided data about the tides at Coco Colo station to the consultant (see annex 1.2).

The extreme values (year extremes 1909-1991) were neglected (they were possibly due to atmospheric phenomena). Only the 19 year mean values over the period 1973-1991 have been retained.

The table below resumes the values in feet PLD and m PLD

19 year means 1973 - 1991	feet PLD	m PLD
mean monthly highest high water	1.341	**0.409**
mean high water	0.973	0.297
mean sea level	0.355	0.108
mean low water	-0.171	-0.052
mean monthly lowest low water	-0.480	**-0.146**

Table 5.1-a

It appears that the values collected from the web are 3 to 4 cm higher than the data furnished by ACP. The difference could be due to the difference between the PLD reference and the reference used on the SHOM site (The zero value is roughly equivalent to the Lowest Astronomical Tide in the area). It was finally decided to retain the values highlighted in yellow in the table.

Consequently, the tide of the Atlantic Ocean has been taken into account considering both its daily and monthly variations:

- Daily variations are represented by a 12.47 hours period.
- Monthly variations are represented by a 14 days period.

The resulting equation entered in the program is shown below:

$$Z(t) = a + \frac{1}{2} * b * \sin\left(\frac{2\pi t}{per\,1 * 60}\right) * \left(c + d * \sin\left(\frac{2\pi t}{per\,2 * 24 * 60}\right)\right)$$

Where:

- t is the time in minutes
- Z(t) is the level of the Atlantic Ocean in m PLD
- a = 0.1315 is the mean sea level value in m PLD
- b = 0.555 m is the maximum tide amplitude
- per 1 = 12.47 is the period of daily variations in hours
- per 2 = 14 is the period of monthly variations in days
- c = 0.68 and d = 0.32 are coefficients of the second sinusoidal curve.

The curve resulting from this equation does not match exactly with the real tidal variation of the Atlantic Ocean (see annex 1.3).

Nevertheless, this equation allows to reach the extreme levels, and the shift observed between real and calculated tides has no consequence on the minimum and maximum levels reached in lock chambers and water saving basins.

➢ The Gatun lake levels

According to the TOR, the maximum and minimum values are:

Maximum level: .... +27.13 m PLD (89 ‘)
Minimum level: .... +24.99 m PLD (82 ‘)

The maximum level is resulting from another study (‘’Raising Gatun Lake”), The minimum level results from an examination of the variations of the lake levels over 18 years (period 1980 – 1997), see annex 1.4. Annex 1.5 shows the Gatun lake levels variations from 1966 to 2000.

➢ Freeboard and minimum water depth

Freeboards and minimum water depths taken into account for the design of the structures are according to the design criteria:

	Water depth (m)	Freeboard (m)
Chamber	18.30	1.50
Water saving basins	1.00	0.80

Table 5.1-b

The minimum water depth of 18.30m in the lock chambers is higher than those specified in the terms of reference. This is due to the fact that it was shown that the corresponding UKC of 1.50m affects too much the hawser forces.

The freeboard of 1.5 m is confirmed both in the PIANC report on locks and in Dutch literature on lock design (Ontwerp van Schutsluizen – RWS 2000). In Berendrecht the maximum water level is +7.50 m above the mean low sea level and the lock heads are at +9.00 m above this level.

Freeboard depends on the water oscillation amplitudes in the chamber during lockage. A scale model is required to get precise values of these oscillations; same remarks for the water saving basin freeboard.

As far as the water saving basins are concerned, the minimum water depth of 1 meter will be confirmed in this actual study.

NB: Adopted values do not affect the results of calculation.

➢ Scheduling

The simulations are undertaken with the ‘’semi convoy mode’’ schedule.

➢ **Characteristic operating times**

Operating times in the simulations are the following:

	duration time (min)
Gate opening or closing	5
Chamber filling or emptying time (using WSB)	5
Water saving basin filling or emptying	4
Inner cycle ship displacement	12
Mean value for entry of 1st ship and exit of last ship	20
Re-initialization of water levels at the turn around	30

Table 5.1-c

Note: the purpose of the software is to set the bottom levels of the chambers and the water saving basins; the indicated times do not affect these levels when modified.

Nevertheless, it gives information about the mean daily water consumption and the number of ship transits through the locks in both directions.

➢ **Generic names for locks and water saving basins**

The 3 lock chambers are identified as: upper, middle and lower
Water saving basins are identified as: top, intermediate and bottom
So, when the upper-bottom wsb is addressed, it would mean the bottom water saving basin of the upper chamber.

5.2 WATER LEVELS CALCULATION

A specific software has been developed to calculate the water levels in the chambers and the water saving basins during the different stages of a lockage in the case of **a triple lift lock system with three water saving basins per lock chamber**.

The software allows simulating cycles scheduled in semi-convoy mode:

- From Gatun lake to Atlantic Ocean during 12 hours
- From Atlantic Ocean to Gatun lake during 12 hours

The software also gives the water saving rate and the water usage during lockage and calculates the heads of each filling or emptying operation.

The equations used in the software are based on the equalization of water levels between the chamber and the water saving basins (see the explanation below).

5.2.1 DESCRIPTION OF THE CALCULATION METHOD

➢ **Equalization of the levels between a chamber and a WSB**

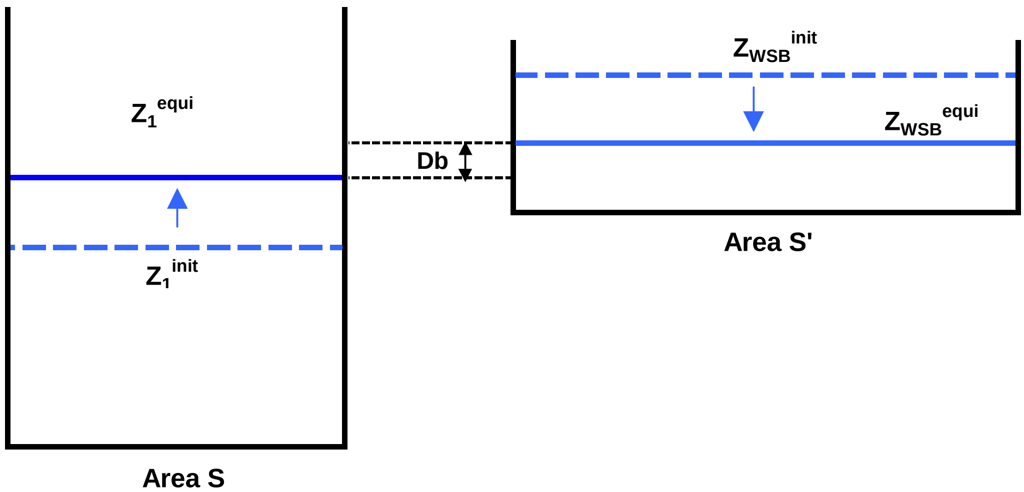


Figure 5.2.1-a

Equation system

$$\begin{cases} \left[Z_{WSB}^{init} - Z_{WSB}^{equi}\right]S' = \left[Z_1^{equi} - Z_1^{init}\right]S \\ Z_{WSB}^{equi} = Z_1^{equi} + Db \end{cases}$$

System solution

$$Z_1^{equi} = \frac{Z_1^{init} * S + Z_{WSB}^{init} * S' - Db * S'}{S + S'} = \frac{Z_1^{init} + m * \left(Z_{WSB}^{init} - Db\right)}{1 + m} \quad \text{with} \quad m = \frac{S'}{S}$$

$$Z_{WSB}^{equi} = \frac{Z_1^{init} + m * Z_{WSB}^{init} + Db}{1 + m}$$

Remark: Db (residual filling depth) is usually about 10 to 20 cm.
This residual filling depth was adopted in coordination with the electro-mechanical experts, in order to save time (the last 10 to 20 cm would need too much time to be filled or emptied). The valves are closed before equalization. In this case the water saved is not exactly 60%, as it was shown during the first presentations in 2002 (e.g. for Db = 0.1 and m = 1, water saving rate = 58.7 %). The right value of 60% may be reached by increasing the area of the wsb, which is reflected in the civil drawings. Anticipated valve closure is taken into account by Flowmaster™ software

➢ **Equalization of the levels between two chambers**

1- First stage : filling of lower chamber from middle chamber, or middle chamber from upper chamber

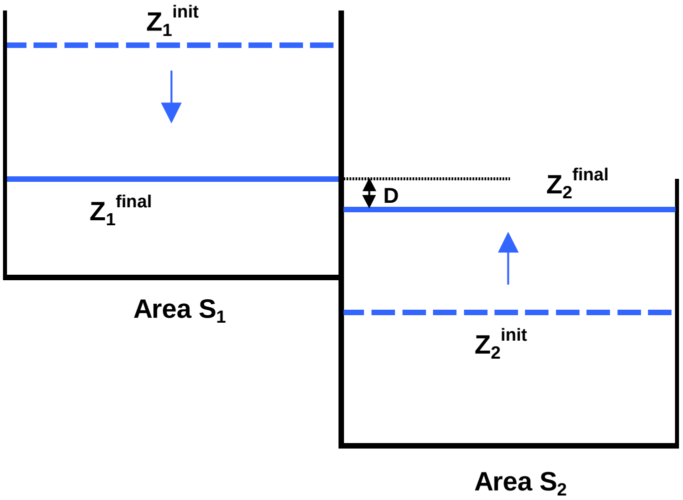


Figure 5.2.1-b

<u>Equation system</u>

$$\begin{cases} \left[Z_1^{init} - Z_1^{final}\right]S_1 = \left[Z_2^{final} - Z_2^{init}\right]S_2 \\ Z_1^{final} = Z_2^{final} + D \end{cases}$$

<u>System solution</u>

Before the opening of the gate, the levels are:

$$Z_2^{final} = \frac{Z_1^{init} * S_1 + Z_2^{init} * S_2 - D * S_1}{S_1 + S_2}$$

$$Z_1^{final} = \frac{Z_1^{init} * S_1 + Z_2^{init} * S_2 + D * S_2}{S_1 + S_2}$$

Remark : in the case of rolling gate, D is equal to zero, i.e. .$Z_2^{final} = Z_1^{final}$
In the PIANC report on locks, part 6: gates and valve § 2.4 : rolling gates
it is indicated that the gates are usually operated with equalization of levels
but are so heavy constructed that it is possible to move them before equalization
A device to detect the breaking of the gate seals at equalization will be installed, allowing to start opening of gates

2- Second stage : opening of the gate

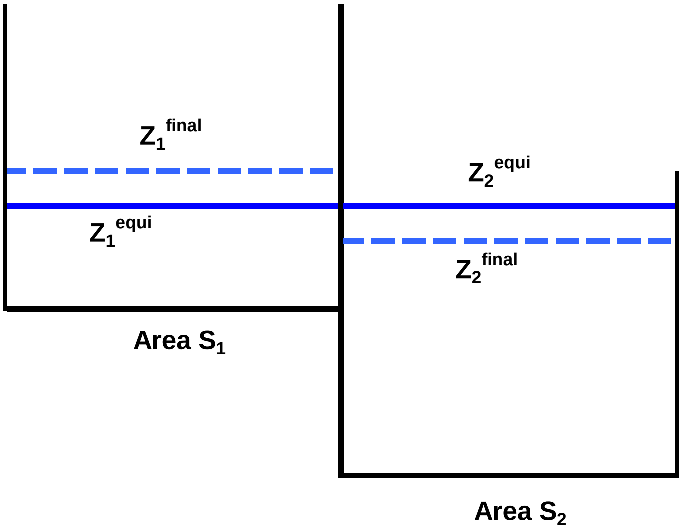


Figure 5.2.1-c

$$Z_1^{equi} = Z_2^{equi} = \frac{Z_1^{final} * S_1 + Z_2^{final} * S_2}{S_1 + S_2}$$

5.2.2 INITIALIZATION OF WATER SURFACE ELEVATION IN THE CHAMBER AND WATER SAVING BASINS

At the beginning of a simulation, the initialization of the water surface elevation in the chambers and water saving basins depends on the head between Gatun Lake and Atlantic Ocean level and on the direction of the lockage.

The drawings below illustrate the way of initializing the water surface elevations (drawings made for *m* = 1)

- Lockage from Lake to Ocean

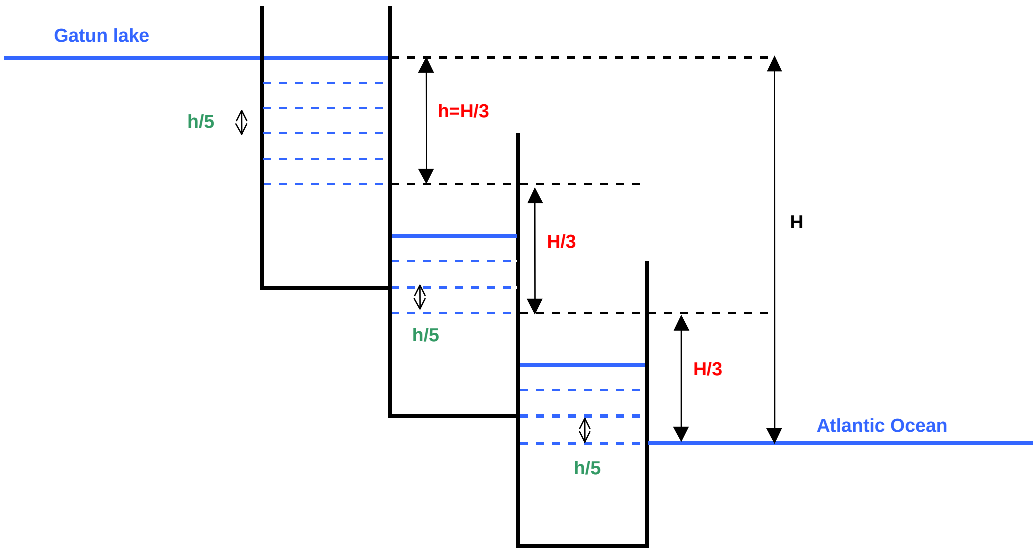


Figure 5.2.2-a

When the simulation starts, the water surface elevations are initialized as below:

- Upper chamber : .... Z_{lake}
  Upper-top WSB : .... $3h/5 + 2H/3 + Z_{mean\ ocean}$
  Upper-intermediate WSB : $2h/5 + 2H/3 + Z_{mean\ ocean}$
  Upper-bottom WSB : ... $h/5 + 2H/3 + Z_{mean\ ocean}$

- Middle chamber : .... $3h/5 + H/3 + Z_{mean\ ocean}$
  Middle-top WSB : ..... $3h/5 + H/3 + Z_{mean\ ocean}$
  Middle-intermediate WSB : $2h/5 + H/3 + Z_{mean\ ocean}$
  Middle-bottom WSB :.. $h/5 + H/3 + Z_{mean\ ocean}$

- Lower chamber :........... $3h/5 + Z_{mean\ ocean}$
  Lower-top WSB : ...... $3h/5 + Z_{mean\ ocean}$
  Lower-intermediate WSB : $2h/5 + Z_{mean\ ocean}$
  Lower-bottom WSB :... $h/5 + Z_{mean\ ocean}$

With $Z_{mean\ ocean}$ = 0.13 m PLD

➢ Lockage from Ocean to Lake

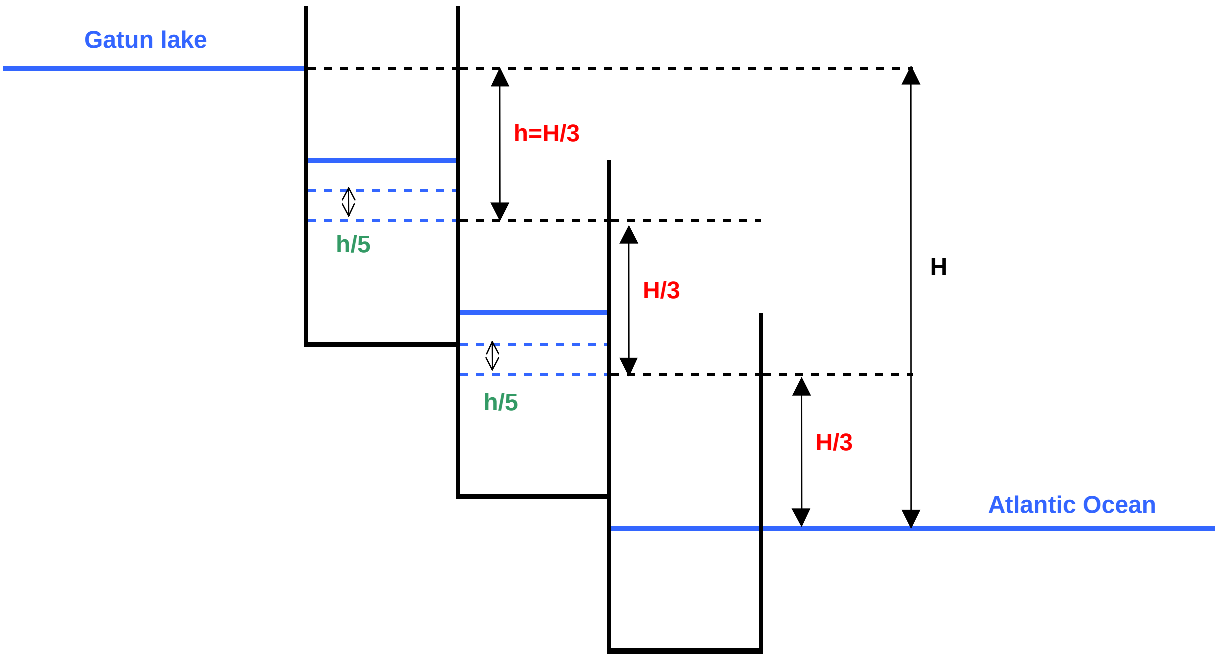


Figure 5.2.2-b

When the simulation starts, the water surface elevations are initialized as below:

- Upper chamber : .... $2h/5 + 2H/3 + Z_{mean\ ocean}$
  Upper-top WSB : .... $4h/5 + 2H/3 + Z_{mean\ ocean}$
  Upper-intermediate WSB : $3h/5 + 2H/3 + Z_{mean\ ocean}$
  Upper-bottom WSB : . $2h/5 + 2H/3 + Z_{mean\ ocean}$

- Middle chamber : .... $2h/5 + H/3 + Z_{mean\ ocean}$
  Middle-top WSB : ..... $4h/5 + H/3 + Z_{mean\ ocean}$
  Middle-intermediate WSB : $3h/5 + H/3 + Z_{mean\ ocean}$
  Middle-bottom WSB : $2h/5 + H/3 + Z_{mean\ ocean}$

- Lower chamber : .... $Z_{ocean}(t_0)$
  Lower-top WSB : ...... $4h/5 + Z_{mean\ ocean}$
  Lower-intermediate WSB : $3h/5 + Z_{mean\ ocean}$
  Lower-bottom WSB : $2h/5 + Z_{mean\ ocean}$

With $Z_{mean\ ocean} = 0.13$ m PLD
And $Z_{ocean}(t_0)$ level Atlantic Ocean at the beginning of the cycle

5.3 DEFINITION OF THE BOTTOMS' SETTING SCENARIOS

The levels of the chambers and the water saving basins given below have been set with the combination of the following values:

- Gatun Lake : + 27.13 m PLD ; + 24.99 m PLD
- Atlantic Ocean : ranging from +0.41 m PLD to – 0.15 m PLD

➢ **For this harmonization study**, the water saving basins and lock chamber have the same area (*m = 1*),

➢ **For this harmonization study**, the residual filling depth was set to Db = 0.0 m

For the simulation, the initial levels have been calculated with the mean value of the Atlantic Ocean level. The scenarios have been tested over a 160 cycles period (1 cycle corresponds to 12 hours of down-lockage and 12 hours of up-lockage), i.e. the total duration time of the simulation represents nearly 80 days.

5.4 RESULTS

5.4.1 LEVELS

The results obtained according to the scenarios defined above for the levels of the upper, middle and lower chamber, as well as the levels in the water saving basins are presented in the following table:

Limit conditions		Level in m PLD												
Gatun Lake level in m PLD	Atlantic tide range in m PLD		Upper chamber	Top WSB	Inter. WSB	Bottom WSB	Middle chamber	Top. WSB	Inter. WSB	Bottom WSB	Lower chamber	Top WSB	Inter. WSB	Bottom WSB
27.13	[-0.15 ; +0.41]	maxi	27.13	25.33	23.53	21.74	18.14	16.35	14.56	12.76	9.19	7.40	5.63	3.87
		mini	18.11	23.51	21.71	19.91	9.08	14.51	12.70	10.89	-0.15	5.45	3.61	1.76
24.99	[-0.15 ; +0.41]	maxi	24.99	23.34	21.68	20.03	16.72	15.07	13.41	11.76	8.47	6.83	5.20	3.58
		mini	16.69	21.67	20.01	18.35	8.37	13.37	11.70	10.04	-0.15	5.02	3.33	1.61

Table 5.4.1-a

5.4.2 HEADS

5.4.2.1 Lock to lock operations

The table below represents the heads obtained after 160 cycles of 12 hours in semi convoy mode (i.e. 80 days simulations)

Gatun lake	Ocean amplitude	Gatun / upper lock	upper lock / mid lock	mid lock / lower lock	lower lock / ocean
maxi 27.73	max	3.61	7.22	7.27	3.97
	min	3.60	7.18	7.13	3.37
mini 24.99	max	3.32	6.64	6.70	3.68
	min	3.31	6.61	6.56	3.09
synthesis	max	**3.61**	**7.22**	**7.27**	**3.97**
	min	**3.31**	**6.61**	**6.56**	**3.09**

Table 5.4.2.1-a

It can be clearly seen that on Atlantic side, the range of heads is much more reduced as compared to the Pacific side

5.4.2.2 Wsb to lock & lock to wsb operations

Lake Gatun	24.99 m PLD	UPPER LOCK			MIDDLE LOCK			LOWER LOCK			extreme values
		top	intermediate	bottom	top	intermediate	bottom	top	intermediate	bottom	
filling lock from wsb	max	-3.31	-3.31	-3.31	-3.30	-3.29	-3.27	-3.22	-3.16	-3.06	-3.69
	min	-3.32	-3.32	-3.32	-3.33	-3.34	-3.36	-3.41	-3.48	-3.69	-3.06
emptying lock into wsb	max	3.32	3.32	3.32	3.33	3.33	3.33	3.36	3.38	3.42	3.42
	min	3.31	3.31	3.31	3.30	3.30	3.30	3.24	3.23	3.20	3.20

Table 5.4.2.2-a

Lake Gatun	27.13 m PLD	UPPER LOCK			MIDDLE LOCK			LOWER LOCK			extreme values
		top	intermediate	bottom	top	intermediate	bottom	top	intermediate	bottom	
filling lock from wsb	max	-3.59	-3.59	-3.59	-3.58	-3.58	-3.55	-3.50	-3.44	-3.35	-3.98
	min	-3.61	-3.61	-3.61	-3.61	-3.62	-3.64	-3.70	-3.77	-3.98	-3.35
emptying lock into wsb	max	3.62	3.61	3.61	3.61	3.61	3.62	3.65	3.66	3.71	3.71
	min	3.60	3.59	3.59	3.59	3.59	3.58	3.52	3.52	3.49	3.49

Table 5.4.2.2-b

The same remark can be made for the water saving basins: the range of heads is once again much lower than at the Pacific side

5.4.2.3 Comparison with Pacific heads

Annex 1.6 shows the distribution of heads in the different operations, lock to lock, wsb to lock and lock to wsb. This annex confirms the observation made in §§ 5.4.2.1 & 5.4.2.2: first, the variations of heads are relatively small at Atlantic side, while the heads themselves are smaller than at the Pacific side.

Another remark: at the Atlantic side, heads are always positive :

- Lock to lock operations : 3.09 m to 7.27 m
- Water saving basin to lock operations : 3.06 m to 3.98 m
- Lock to water saving basin operations : 3.20 m to 3.71 m

while the heads can be equal to 0 at the Pacific side, either in lock to lock operation as in wsb to lock operation.

These remarks will allow to operate the Atlantic locks in a more simple way than at the Pacific side, and the hawser forces are expected to be lower than at the Pacific side.

6 Water saving rate

The triple lift lock system equipped with 3x3 water saving basins allows a very good water saving rate (real value of 85.83 % instead of theoretical one of 86.67 %). This difference is due to the reset of the levels at noon when changing the sailing direction for the ships (semi convoy mode).

6.1 CALCULATION BY THE SOFTWARE

The total head between the Gatun Lake and the Atlantic varies in relation to the Atlantic Ocean tides and Gatun Lake level. Specific calculations are made by the software to assess the water consumption (volume of water taken from the Gatun Lake) and the water saving rate for each lockage.

- Water usage

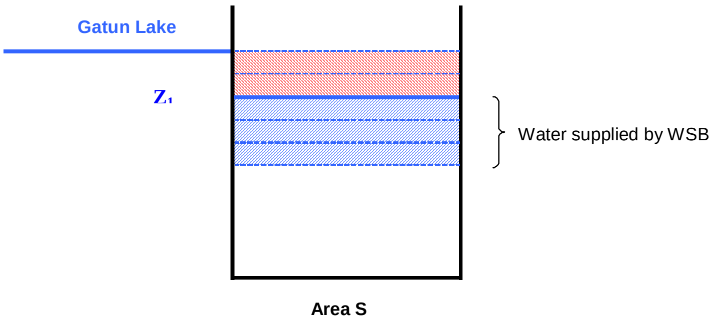


Figure 6.1-a

Let Z_1 be the water level in the upper lock at the end of the WSB-to-lock chamber operations. The software calculates the volume taken from the lake by:

$$V_{lake} = (Z_{lake} - Z_1) * S$$

This volume is calculated for each filling operation of the lock.

- Water saving rate

Figure 6.1-b

Let Z_{mini} be the water level in the upper lock at the beginning of the filling operation. Without any water saving basin, the volume of water required to fill the lock chamber would be equal to:

$$V_{\max i} = (Z_{lake} - Z_{\min i}) * S$$

The water saving basins allow to save a volume equal to:

$$V_{saved} = (Z_1 - Z_{\min i}) * S$$

The water saving rate is then calculated by the relation:

$$e = \frac{V_{saved}}{V_{\max i}}$$

The water saving rate is calculated for every filling operation of the lock and at the end of the simulation. The final water saving rate is calculated by:

$$E = \frac{\sum V_{saved}}{\sum V_{\max i}}$$

The results achieved with the software are coherent with the theoretical ones.

6.2 RECOMMENDATIONS

The initial chamber and water saving basin bottom levels and wall elevations, the number of possible transits, and the water usage calculations are established with the present hypothesis and data introduced in the software developed by the Consortium CPP. These results are still applicable and are not affected by the final filling and emptying times resulting from Flowmaster™ hydraulic analysis and the final design operating times.

7 Filling and emptying systems

General criteria

Based on the results of the Pacific side study of the triple lift lock system, as well as the single lift lock and the double lift lock, the terms of reference for the triple lift lock study assumed that the best solution to study is the following:

- A filling / emptying system with lateral culverts and ports. The culverts extend over the whole length of the lock.
- A system of 3 side-by-side water saving basins per lock chamber, connected to the lateral culverts by means of conduits.

This system has been retained for the present harmonization study.

8 Hydraulic design of the filling / emptying system

In accordance to the previous chapter, the system to be studied includes a lateral alimentation with culverts and ports incorporated in the lock walls.

The modifications of the F/E system proposed hereafter take into account the design of the 3 lift lock system of the Pacific side and the Atlantic specific site conditions, especially the tidal variation.

8.1 SOFTWARE AND METHODS USED IN THE STUDY

The emptying/filling system has been studied with the software FLOWMASTER 2. Software Flowmaster 2 is a Community Trade Mark of Flowmaster Holding BV. A description of this tool has been given in the report of the first configuration.

8.2 CALIBRATION OF THE SOFTWARE

The calibration of the software was based on the physical model study of the Panama Canal locks performed in 1942. (see annex 3.4 of the first configuration report).

The Flowmaster model gave acceptable results, since the flow rate difference was found to be less than 10 %. All documents concerning this test and two other ones are given in annex 3.4 of the first configuration report (2002).

8.3 HYDRAULIC DESIGN

The filling/emptying system is identical to the actualized Pacific system. The longitudinal culverts are used for the filling and emptying of the locks. It has been demonstrated for the first configuration that this system was to be preferred because:

- Of its ability to provide a good distribution of the flow;
- Of its reliability (redundancy in case of a valve default);
- Construction of an expensive concrete bottom floor can be avoided.

8.3.1 SIZE OF CULVERTS, CONDUITS AND PORTS

The main difference between the Pacific side and the Atlantic side conditions concerns the small range of the Atlantic Ocean tidal levels. As a result, the former chapters have shown that this little range was responsible for relatively small level variations in the chambers and WSB. While at the Pacific side the range was considerably growing from the upstream lock to the downstream one, including the WSB, at the Atlantic side the heads between the different pools are quite identical, whatever pool is considered.

It was then possible to reduce the dimensions of conduits between the WSB and the main culverts, the maximum head ranging from 3 m to 4 m approximately, while reaching up to 8.40 m at the Pacific side.

On the contrary, the remaining head between two lock chambers is always of the same magnitude, approximately 2 x 2/5 of the total head of 10 m, or 8 m approximately. This figure is close to the maximum head between the lock chambers at the Pacific side, which is 8.10 m. As a consequence the same size of the main culverts and valves has been maintained.

The hydraulic design leads to the following dimensions:

	Shape	Size (WxH)	Section (m²)	Quantity	Total section (m²)
Side-wall culverts	rectangular	9.00 m x 6 m	54	2	108
Valves	rectangular	4.5 m x 6 m	27		27
WSB-to-lock culvert	rectangular	4.0 x 5.0 m	20	4 per basin	80
WSB valves	rectangular	4.0 x 5.0 m	20	4 per basin	80
Ports	rectangular	2 m x 2 m	4	40, 20 per lock side	160

Table 8.3.1-a

As far as the water saving basins are concerned, the number of conduits, the section and the total section of the conduits and ports is to be considered for each of the three saving basins.

8.3.2 FLOWMASTER MODEL PARAMETERS

The same model parameters as for the Pacific side study have been retained. The main information concerning the most important components is the following:

- **Culvert and conduit**: the culvert and conduit sizes are given above; the absolute roughness of the inner surface is 0.025 mm, which corresponds to the absolute roughness of a smooth concrete pipe (cf. Internal Flow System from D.S. Miller).

- **Valve**: the valves used in the model are valve gates with a section equal to the pipe section.

- **Discrete loss**: in order to take into account the kinetic energy dissipation in each chamber (or component assimilated to a chamber), discrete losses have been introduced downstream the ports with a loss coefficient of 1. The reverse flow coefficient is taken equal to 0.5.

- **T-junctions**: T-junctions are used to model the ports. Loss coefficients in T-junctions are automatically set depending on the two branch flow and area ratios. The calibration of those components demonstrated that they are suitable to model the ports.

8.4 FILLING AND EMPTYING TIMES

8.4.1 ELEMENTARY OPERATIONS

About 30 simulations (compared to 70 at the Pacific side) have been run with Flowmaster™ in order to estimate the filling and emptying times, the flow rate and maximum velocities reached in the culverts. The simulations take into account the variations of Gatun Lake and Atlantic Ocean levels.

According to the terms of reference, the simulations have been performed for lock operation using water saving basins.

The table below gives an overview of the filling and emptying times for a lockage with water saving basins. This table takes into account the levels calculated with CPP's software: minimum and maximum heads that can be reached between the Lake or Ocean and the lock chambers or between a water saving basin and a lock chamber either during filling or emptying phases.

The filling and emptying times have been determined for a valve opening time of 2 min (either for Lake or Ocean-to-lock chamber or water saving basin-to-lock chamber operations). At any time it was possible to fulfill the design criteria.

The maximum global time has to be calculated by considering the worst case: highest head between the downstream lock and the Ocean. The heads between the WSB and the lock chambers are then given by CPP's software.

Operation	Head in m	Opening time of the valves (sec)	F/E time (sec)	Highest mean velocity (m/s)	
				Culvert	Ports
Upper lock filling	3.60 max head	120 (2 mn)	280 (4 mn 40 s)	4.7	4.7
	3.30 min head	120 (2 mn)	270 (4 mn 30 s)	4.5	4.5
Filling / Emptying locks	7.30 max head	120 (2 mn)	255 (4 mn 15 s)	5.1	6.2
	6.60 min head	120 (2 mn)	245 (4 mn 5 s)	4.8	5.8
Emptying lower lock	4.00 max head	120 (2 mn)	280 (4 mn 40 s)	5.1	6.2
	3.10 min head	120 (2 mn)	250 (4 mn 10 s)	4.3	5.3
Filling the lower lock's WSB	3.70 max head	120 (2 mn)	200 (3 mn 20 s)	4.7	2.6
	3.20 min head	120 (2 mn)	190 (3 mn 10 s)	4.3	2.3
Emptying the lower lock's WSB	4.00 max head	120 (2 mn)	220 (3 mn 40 s)	4.6	2.6
	3.10 min head	120 (2 mn)	200 (3 mn 20 s)	3.9	2.2

Table 8.4.1-a

For all those simulations, the velocities in culverts and ports do not exceed 7 m/s, there is consequently no erosion risk on of the culverts and ports. As Flowmaster only gives average velocities in each section, it will be necessary to verify on the scale model that this velocity isn't exceeded too much in some critical flowing sections,

When compared to the Pacific side, all valves can be opened in 2' without generating unacceptable velocities in the ports.

Consequently, whatever elementary operation is considered, the operating times will be inferior to the target time.

8.4.2 GLOBAL HYDRAULIC TIME OF A WHOLE LOCKAGE OPERATION BETWEEN THE GATUN LAKE AND THE OCEAN

According to the preceding and when comparing to the Pacific side actualization study, the total hydraulic duration of a whole lockage will be far less than the design value of 51'.

8.4.3 DETAILED RESULTS

The graphs representing the flow rate and the water surface elevation in the lock chamber are given in annexes 2 to 7.

Similarly as for the Pacific side, the results are mainly of 3 types:

Type 1: flows in the ports when filling or emptying the WSB. The graphs show the same magnitude of gap between the first and last ports, which is low and will not generate important hawsers forces.

See annexes 2 and 7.

Type 2: flows in the ports when emptying a lock. The head being close to the Pacific one, the graphs show the same unbalanced distribution in the ports, due to the connection of the longitudinal culverts at their upstream extremity. See annex 4.

Type 3: flows in the ports when filling a lock. The results are very similar to those of the Pacific side. See annexes 3 and 5.

9 Cavitation and air demand

Cavitation spreading is examined for the regular operation (i.e. with the water saving basins), according to the TOR.

- *Lock to lock*

As the head between two chambers or between the Gatun Lake and the upper lock is always smaller than 7.30 m and the minimum water height in the chamber is 18.3 m (Head < h), cavitation cannot appear.

Due to the large water depth of the valves relatively to the water levels in the pools, the pressure on the valve remains higher than the vapor pressure, which guaranties that cavitation will not appear, except maybe in the very first seconds (the Flowmaster™ calculation is not sufficiently accurate to confirm the total absence of cavitation in such a time span, but the physical scale model will allow to conclude).

- *Water saving basins to lock*

As the head between the water saving basins and a chamber is always smaller than 4.0 m and the minimum water height in the chamber is 18.3 m (Head < h), cavitation cannot appear.

- *Lock to Ocean*

As the head between the lower lock chamber and the Ocean is always smaller than 4.0 m and the minimum water height in the chamber is 18.3 m (Head < h), cavitation cannot appear.

10 Strengths on the hawsers

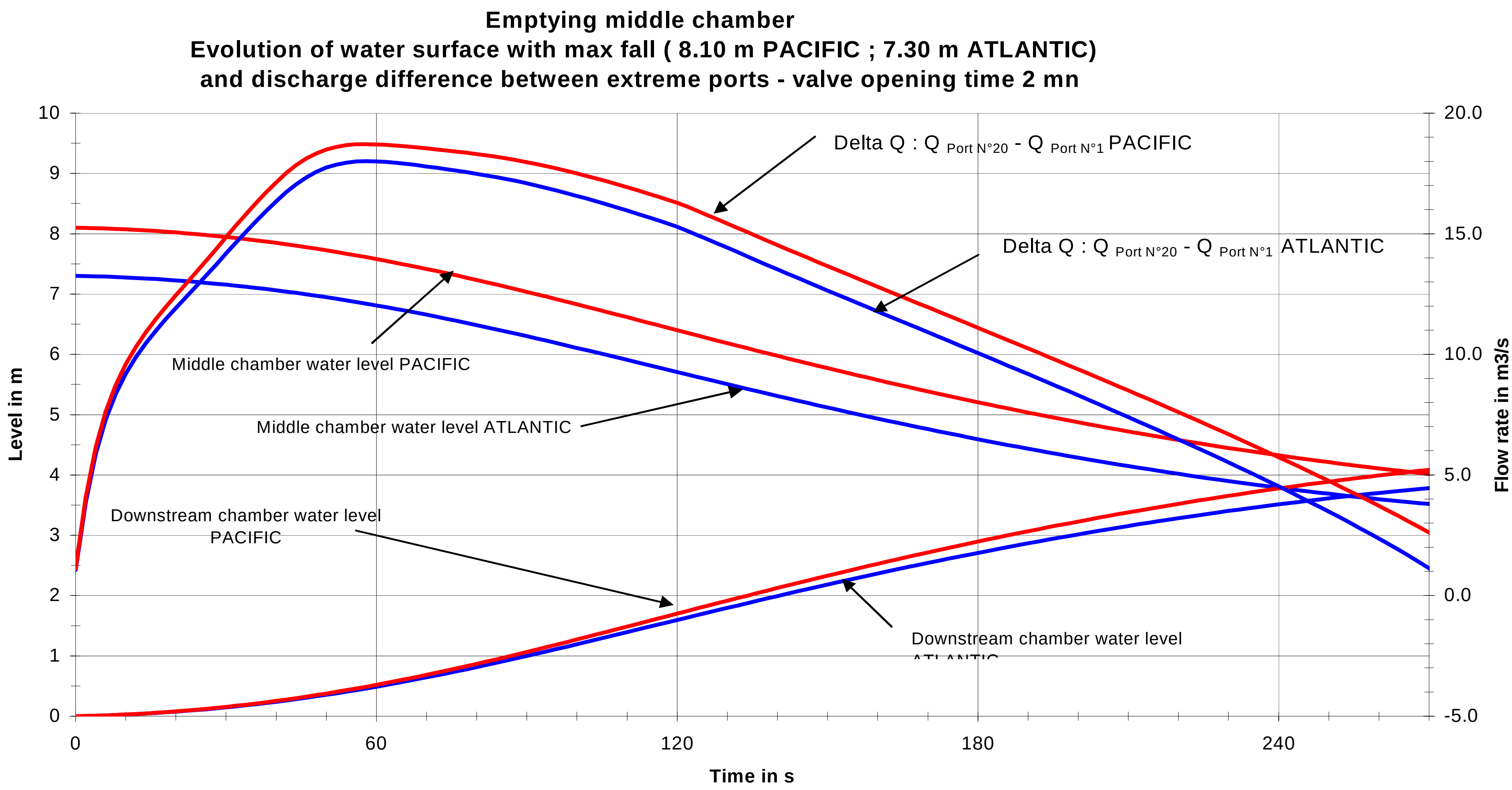
As demonstrated in the report of the Actualization study at the Pacific side, the three basic ways to reduce the hawser forces, mainly by reducing the differences of discharges between the ports, are:

- Unsteady stage: by modifying the opening diagram of the valve. Opening the valve more slowly makes it possible to obtain a better balanced discharge distribution in the first stage;
- Quasi-steady stage: by reducing the efficient area of the ports that have the higher discharge, the distribution can be better balanced;
- Concentrating the ports closer to the "gravity center" of the lock chamber: it has already been shown by means of physical model testing that this leads to a reduction of the hawser forces. This is particularly valid for the 3rd lane of Post Panamax locks, where single big vessels are expected to pass through the locks.

The comparison between the Atlantic side and the Pacific side show that **the discharge differences between the extreme ports** (see annexes 3.1 to 3.3 in both reports), being a criterion for the hawser forces, as it was demonstrated in the Pacific side actualization report, **are lower on the Atlantic side** for the most constraining case (lock to lock operation, head of 7.30 m) than on the Pacific side for the most constraining case (lock to lock operation, head of 8.10 m), i.e. the distribution of flows is more homogeneous. The graph 10.1 on next page showing the superposition of these Pacific and Atlantic worst cases shows it more clearly.

Consequently, the hawser forces are expected to be lower on the Atlantic side

This will allow to apply the same methods of reducing the hawser forces as those that will be developed for the Pacific side.


Emptying middle chamber
Evolution of water surface with max fall (8.10 m PACIFIC ; 7.30 m ATLANTIC)
and discharge difference between extreme ports - valve opening time 2 mn
Delta Q : Q Port N°20 - Q Port N°1 PACIFIC
Delta Q : Q Port N°20 - Q Port N°1 ATLANTIC
Middle chamber water level PACIFIC
Middle chamber water level ATLANTIC
Downstream chamber water level
PACIFIC
Downstream chamber water level
Level in m
Flow rate in m3/s
Time in s
0
60
120
180
240
10
9
8
7
6
5
4
3
2
1
0
20.0
15.0
10.0
5.0
0.0
-5.0

Graph 10.1

11 Flows between the gates

The Panama triple lift lock system on the Atlantic side will, just like the Pacific side, use double gates between two steps, in order to improve the security. Consequently, the volume between two adjacent gates has to be filled or emptied simultaneously with the chamber.

It concerns the water volume in the gates and the volume of the gate recesses.

The calculations would lead to very similar conclusions as those obtained for the Pacific side, in terms of level differences and additional time required to equalize the levels.

12 Conclusion and recommendations

The harmonization study has confirmed that the filling and emptying system that was designed for the Pacific side, provided that some geometrical modifications are made, is perfectly adapted to the Atlantic configuration. It allows to comply with the design criteria and guidelines.

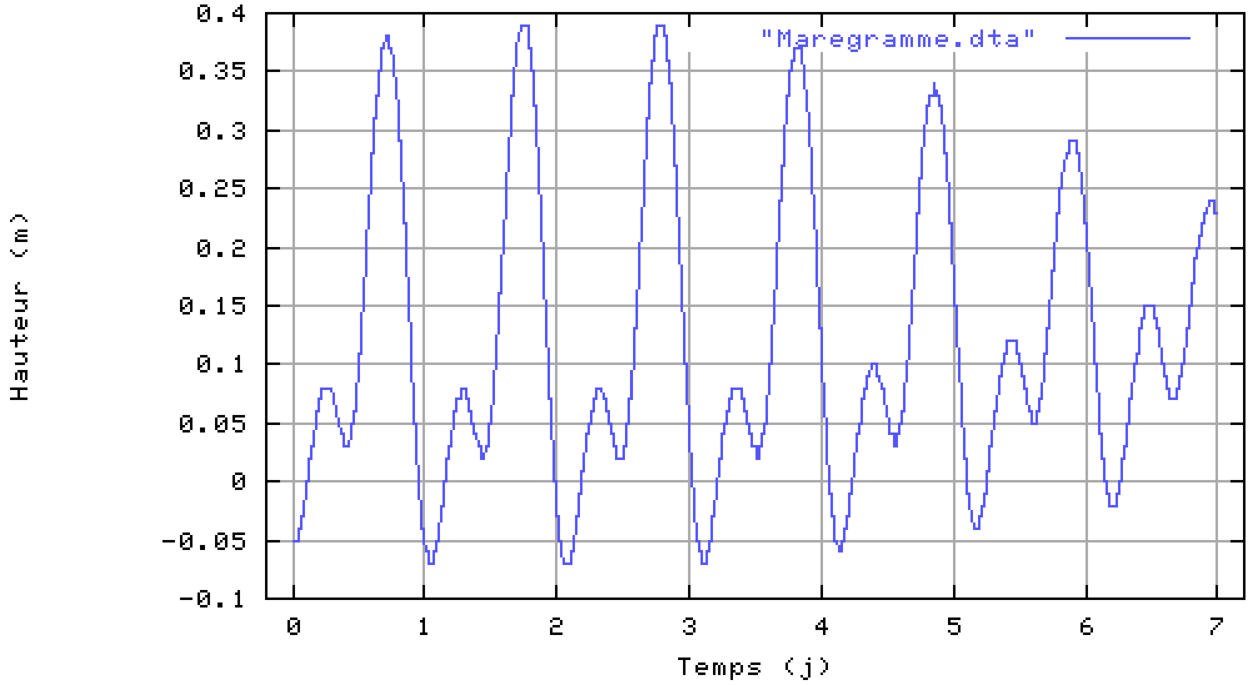
This study confirmed that the Atlantic side operating conditions are less complicated than at the Pacific side, mainly due to the lower tidal amplitude. Consequently, similar optimizations will be considered for both Pacific and Atlantic locks.

It is therefore suggested to optimize the Pacific side system in a first phase, by means of more detailed numerical modeling (based upon the Flowmaster™ software in combination with the 2D simulation of the water flow in the lock chamber and more refined numerical modelling of forces and dynamics of ship and vessel positioning system).

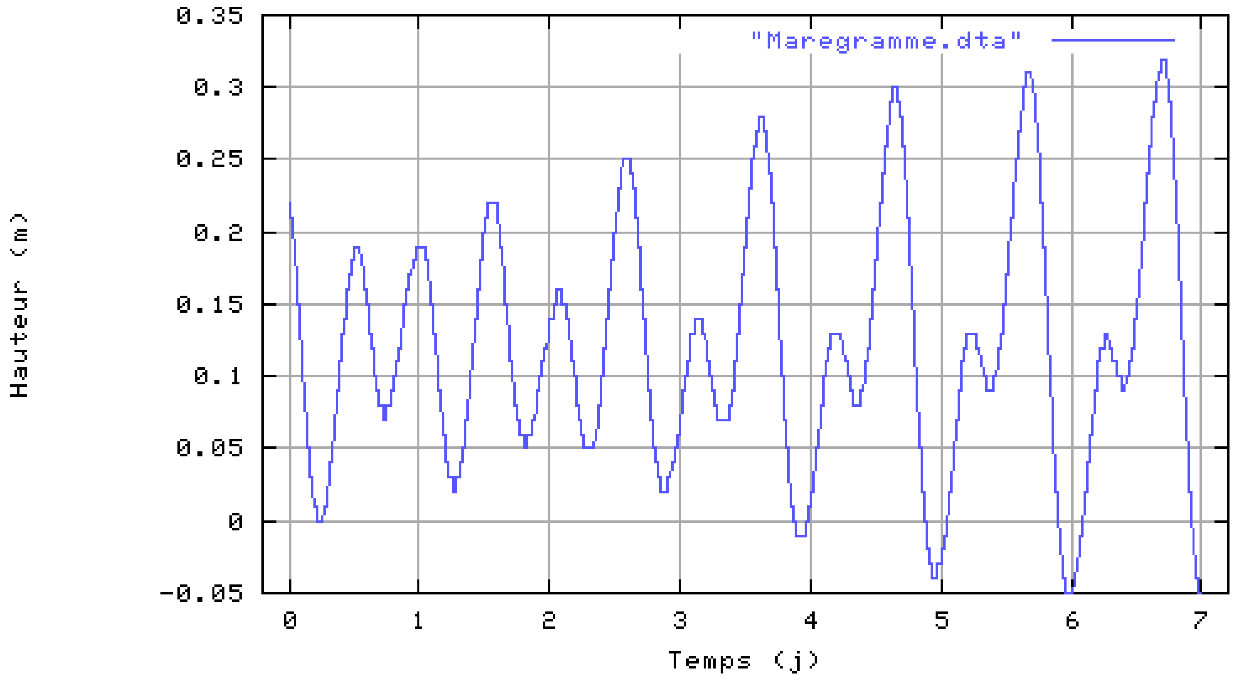
Then, the Atlantic side could be optimized and validated by some ultimate Flowmaster™ simulations in combination with a final validation on the physical scale model

13 Annexes

ANNEXES



20/09/04



27/09/04

PANAMA CANAL COMMISSION
ENGINEERING AND CONSTRUCTION BUREAU
Meteorological and Hydrographic Branch

atención
Cheryl George

PANAMA CANAL TIDAL DATA

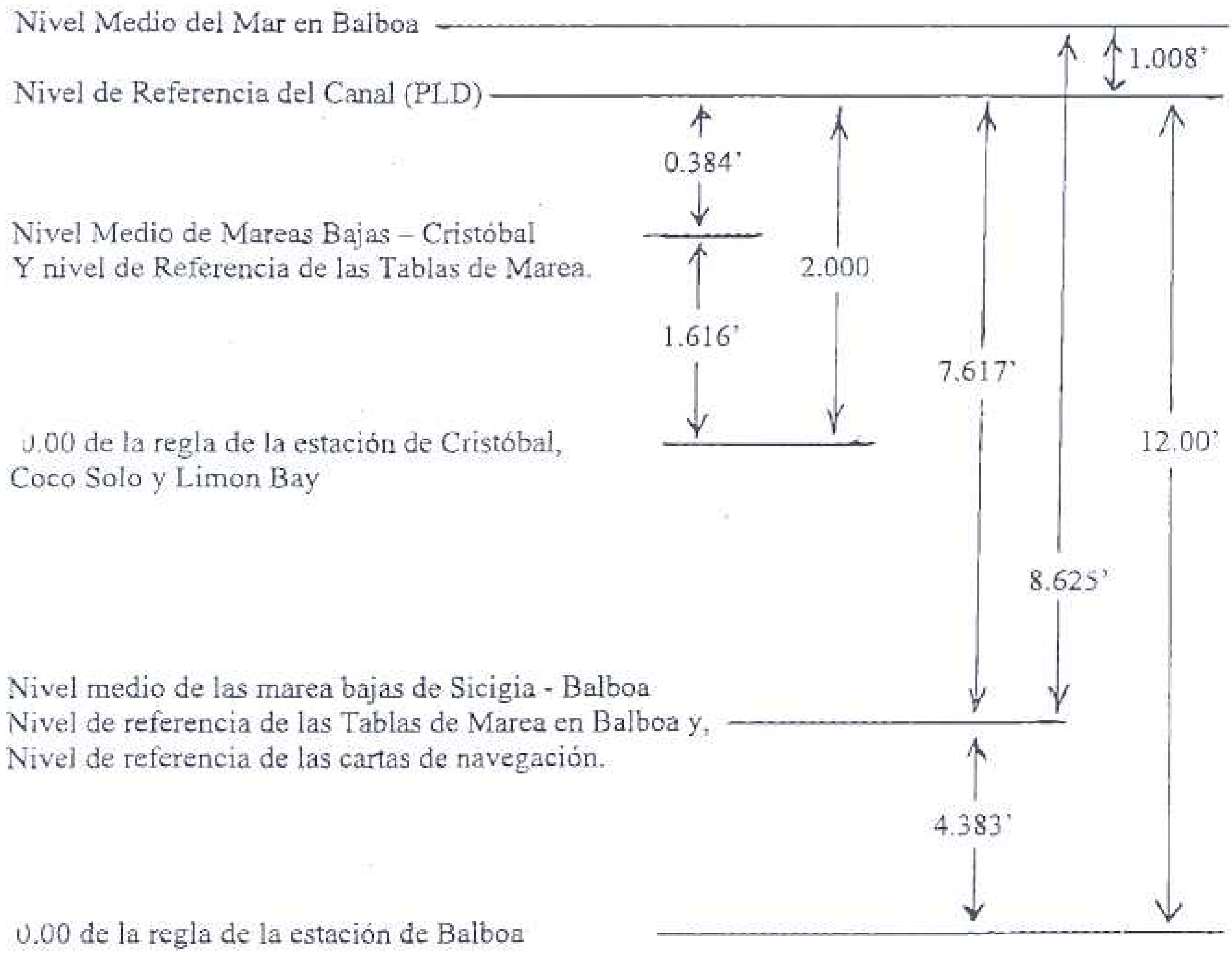
DIABLO (Pacific Coast)	Datum for Pamama Canal Tide Records Feet (Zero of Diablo Tide Gage)	Datum for Navigational Charts and Tide Tables Feet (Mean Low Water Springs)	Panama Canal Precise Level Datum Feet
Data for Diablo tidal values *			
(1) Zero of Balboa Tide Gage	0.000	- 4.383	-12.000
(2) Mean Low Water Springs	+ 4.383	0.000	- 7.617
(3) Precise Level Datum	+12.000	+ 7.617	0.000
19 year means, 1973-1991			
Mean monthly highest high water	+21.920	+17.537	+ 9.920
Mean high water	+19.022	+14.639	+ 7.022
Mean sea level	+13.008	+ 8.625	+ 1.008
Mean low water	+ 6.438	+ 2.005	- 5.562
Mean monthly lowest low water	+ 2.856	- 1.527	- 9.144
83 year extremes, 1909 - 1991			
Higest high water	+23.800	+19.417	+11.800
Lowest high water	+14.220	+ 9.837	+ 2.220
Higest low water	+10.930	+ 6.547	- 1.070
Lowest low water	+ 0.700	- 3.683	-11.300

COCO SOLO (Atlantic Coast)	(Zero of Coco Solo Tide Gage)	(Mean Low Water)	
Data for Coco Solo tidal values *			
(1) Zero for Coco Solo tide gage	0.000	- 1.616	- 2.000
(2) Mean low water	+ 1.616	0.000	- 0.384
(3) Precise Level Datum	+ 2.000	+ 0.384	0.000
19 year means, 1973 - 1991			
Mean monthly highest high water	+ 3.341	+ 1.725	+ 1.341
Mean high water	+ 2.973	+ 1.357	+ 0.973
Mean sea level	+ 2.355	+ 0.739	+ 0.355
Mean low water	+ 1.829	+ 0.213	- 0.171
Mean monthly lowest low water	+ 1.520	- 0.096	- 0.480
83 year extremes, 1909-1991			
Highest high water	+ 3.860	+ 2.244	+ 1.860
Lowest low water	+ 0.710	- 0.906	- 1.290

* Note: Data for Dialo tidal values refers to the data at Balboa and Diablo. Data for Coco Solo tidal values refers to the data at Cristobal and Coco Solo.

AUTORIDAD DEL CANAL DE PANAMA

NIVELES DE REFERENCIA PARA ALTURAS DE MAREA



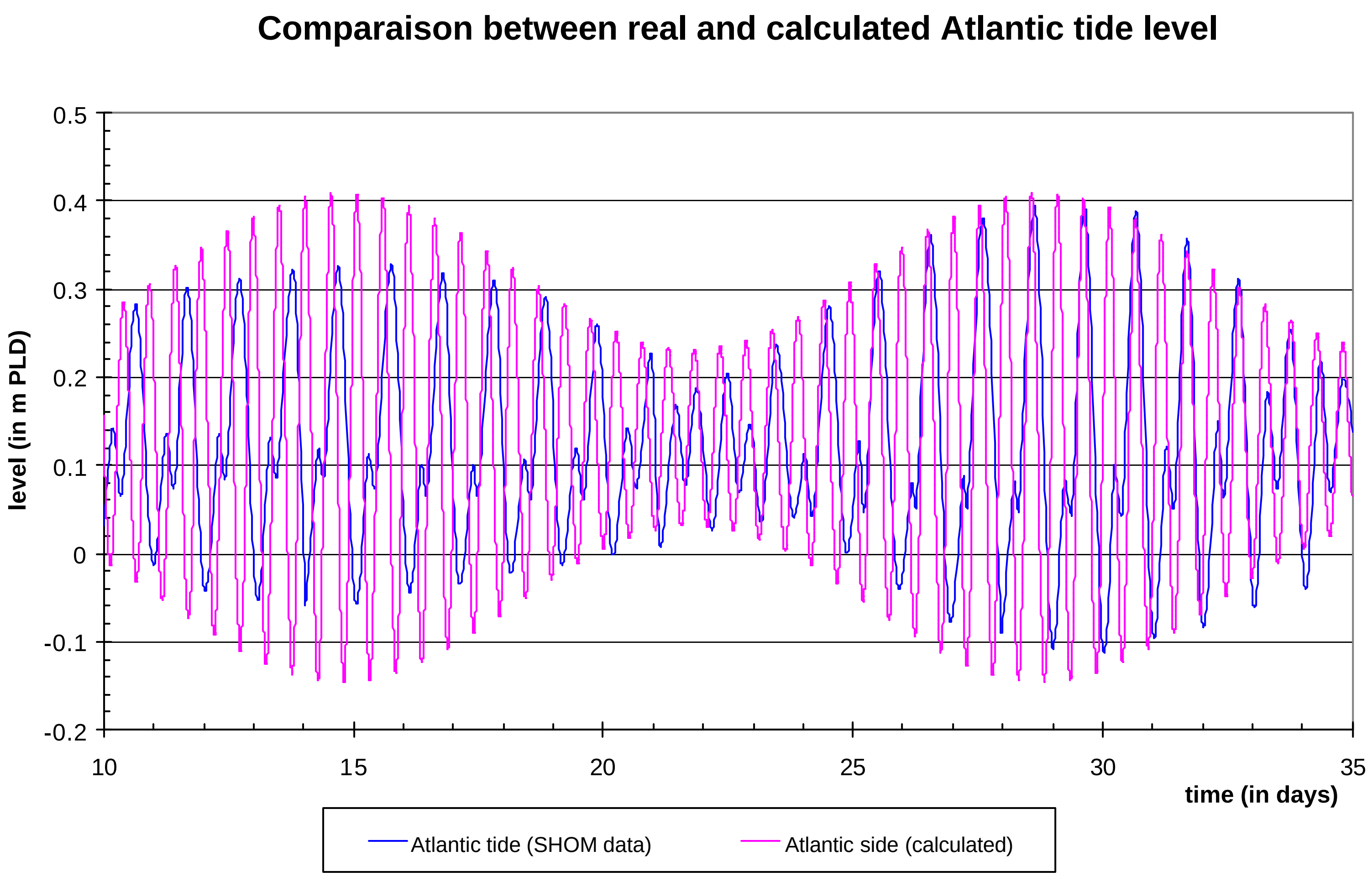
En el Pacífico:

Elevación en PLD = Tabla de Mareas – 7.617'
Elevación en PLD = Estación Diablo Heights – 12.00'
Tabla de Mareas = Estación Diablo Heights – 4.383'

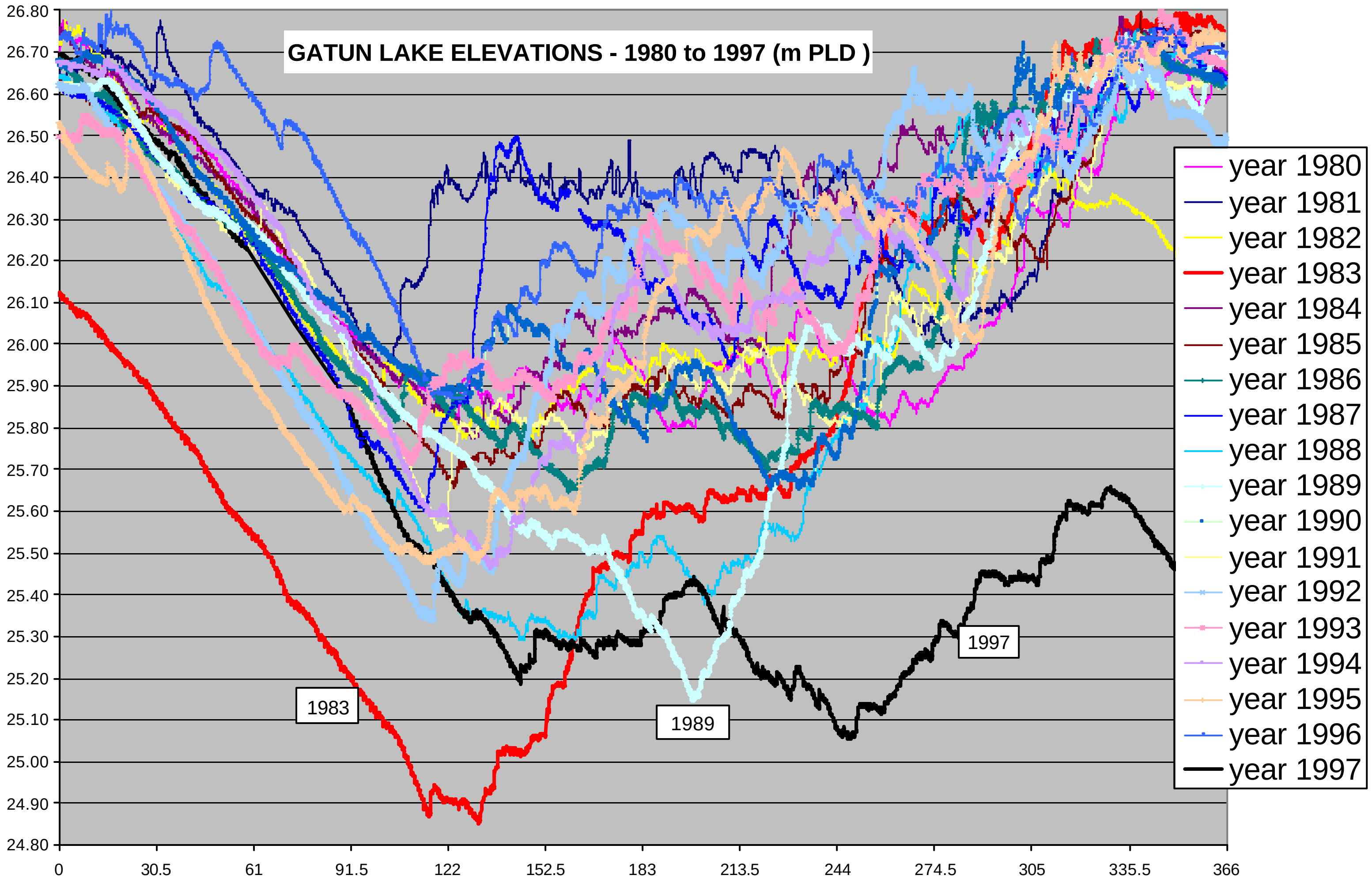
En el Atlántico:

Elevación en PLD = Tabla de Mareas – 0.384'
Elevación el PLD = Estación Limon Bay – 2.00'
Tabla de Mareas = Estación Limon Bay – 1.616'

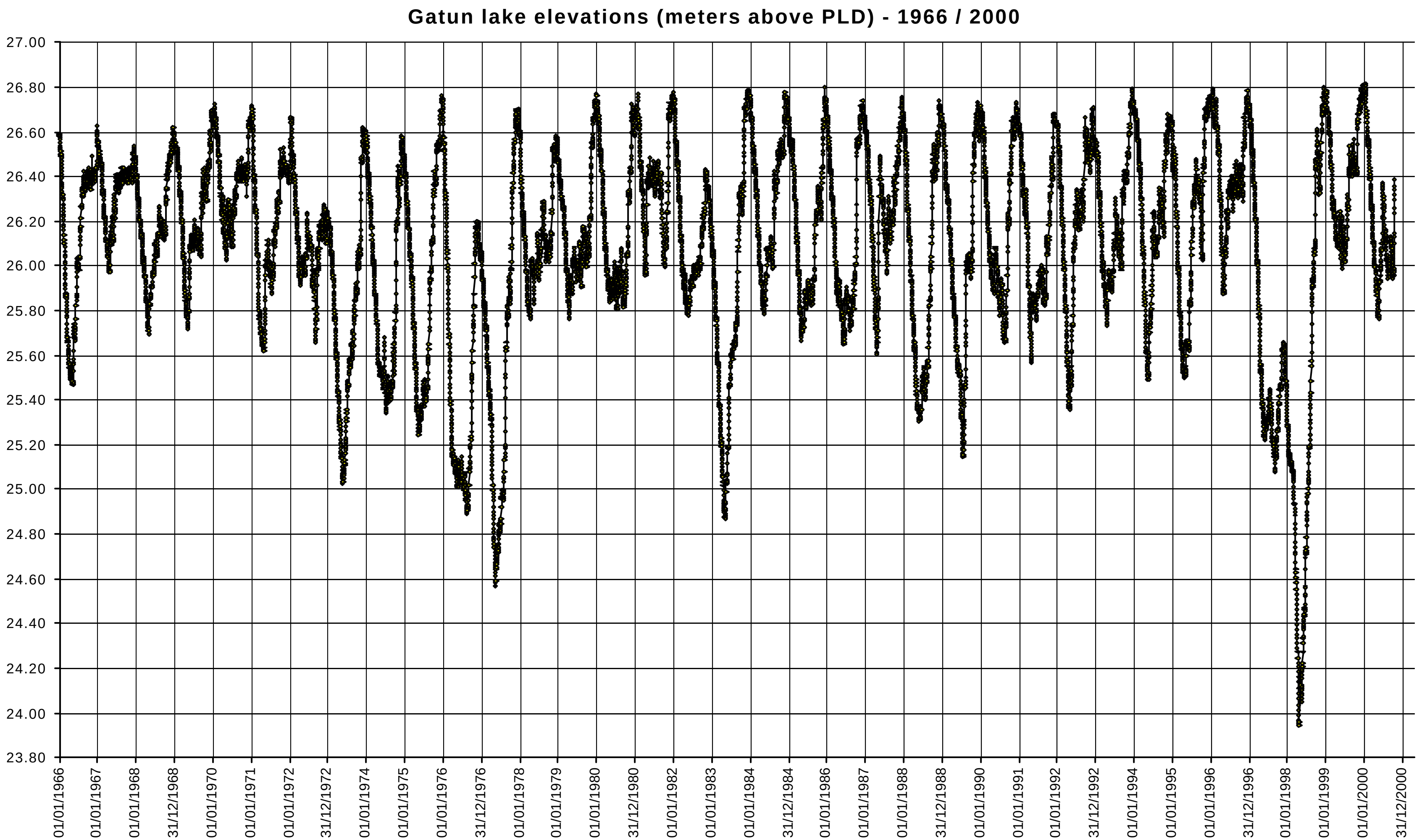
Modesto Echevers
Sección de Meteorología e Hidrología
4 de Enero de 2001


Comparaison between real and calculated Atlantic tide level
level (in m PLD)
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
10
15
20
25
30
35
time (in days)
Atlantic tide (SHOM data)
Atlantic side (calculated)

Annex 1-4.doc


GATUN LAKE ELEVATIONS - 1980 to 1997 (m PLD)
26.80
26.70
26.60
26.50
26.40
26.30
26.20
26.10
26.00
25.90
25.80
25.70
25.60
25.50
25.40
25.30
25.20
25.10
25.00
24.90
24.80
0
30.5
61
91.5
122
152.5
183
213.5
244
274.5
305
335.5
366
1983
1989
1997
year 1980
year 1981
year 1982
year 1983
year 1984
year 1985
year 1986
year 1987
year 1988
year 1989
year 1990
year 1991
year 1992
year 1993
year 1994
year 1995
year 1996
year 1997

Annex 1-5.doc


Gatun lake elevations (meters above PLD) - 1966 / 2000
27.00
26.80
26.60
26.40
26.20
26.00
25.80
25.60
25.40
25.20
25.00
24.80
24.60
24.40
24.20
24.00
23.80
01/01/1966
01/01/1967
01/01/1968
31/12/1968
01/01/1970
01/01/1971
01/01/1972
31/12/1972
01/01/1974
01/01/1975
01/01/1976
31/12/1976
01/01/1978
01/01/1979
01/01/1980
31/12/1980
01/01/1982
01/01/1983
01/01/1984
31/12/1984
01/01/1986
01/01/1987
01/01/1988
31/12/1988
01/01/1990
01/01/1991
01/01/1992
31/12/1992
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31/12/1996
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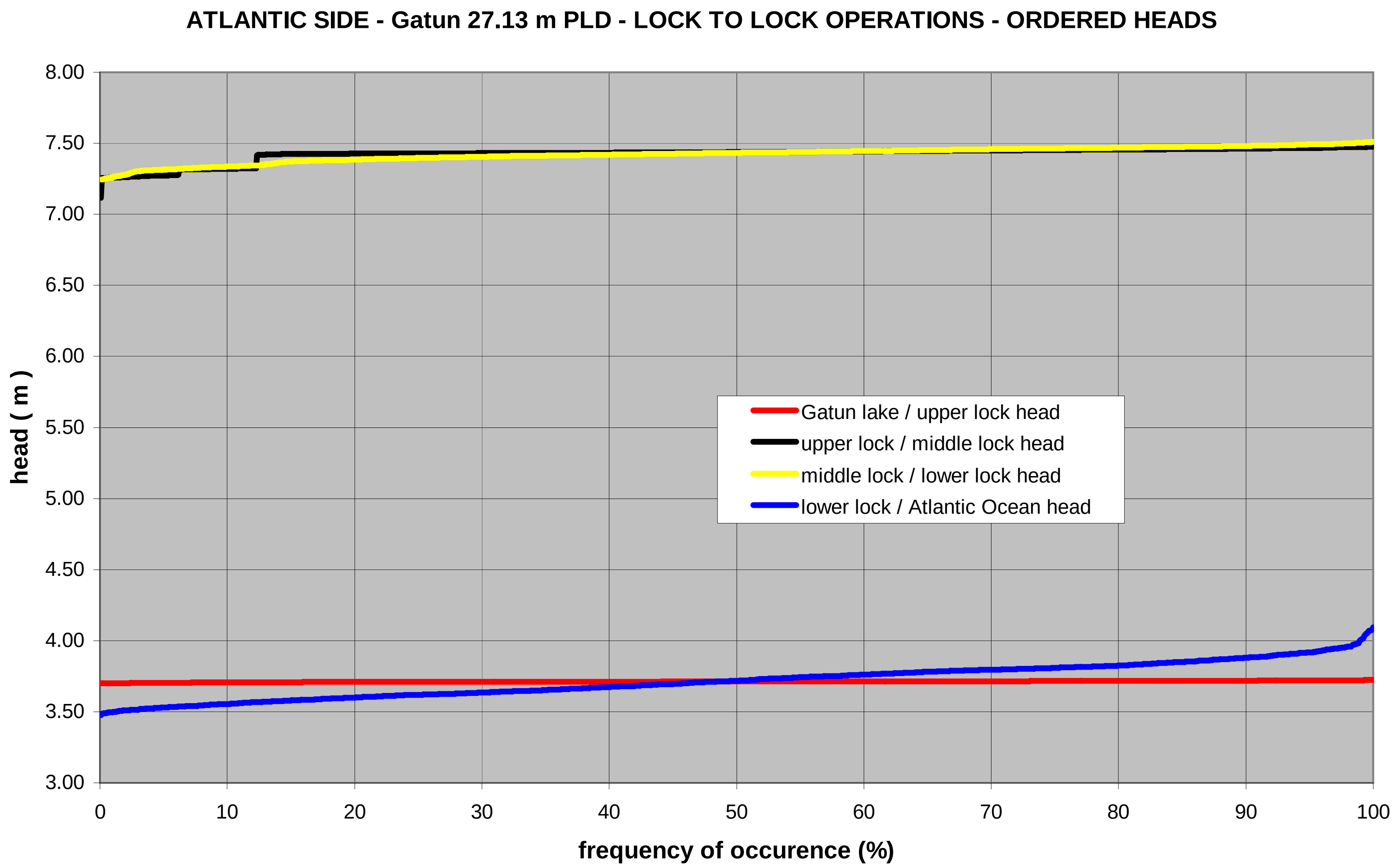
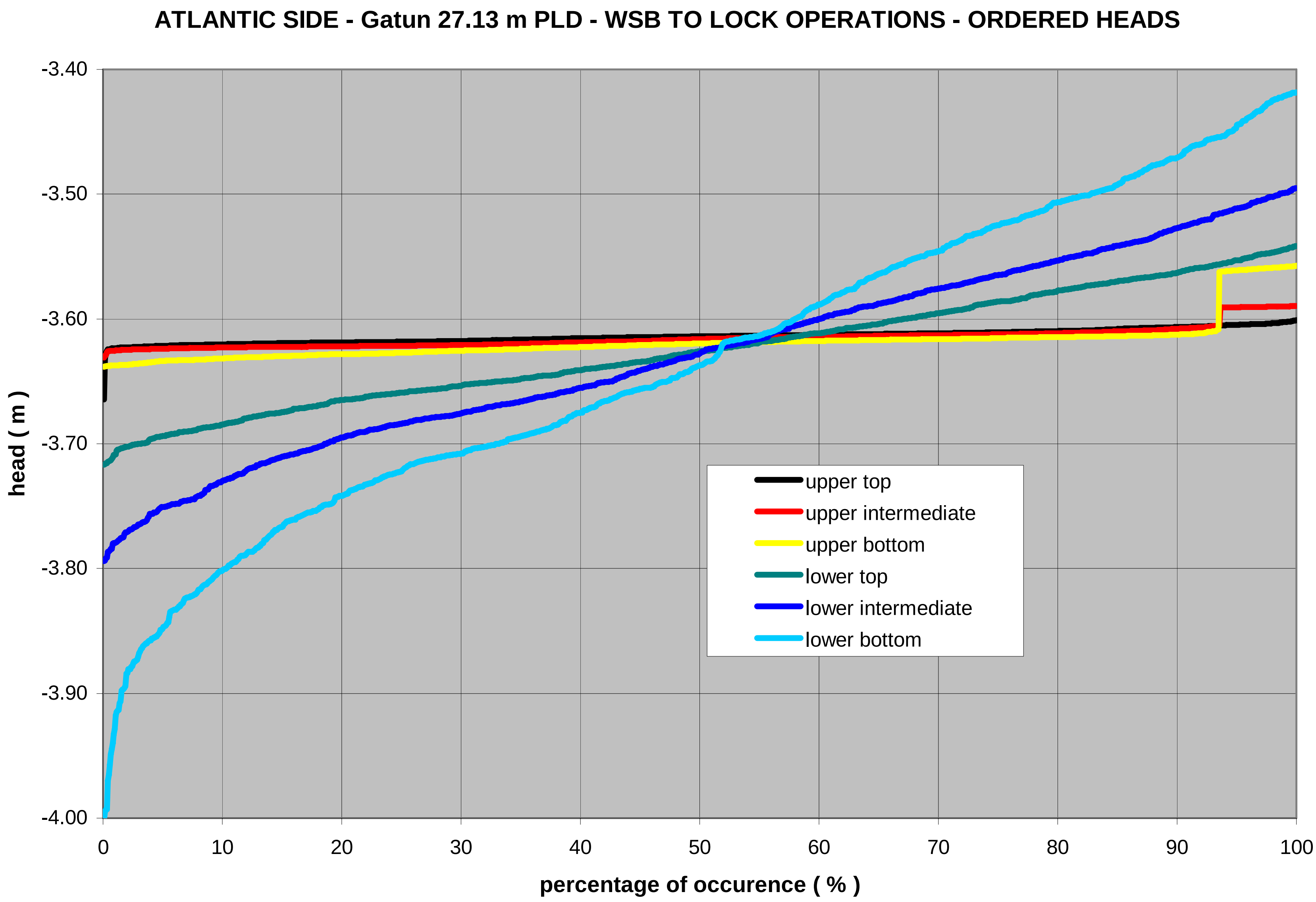
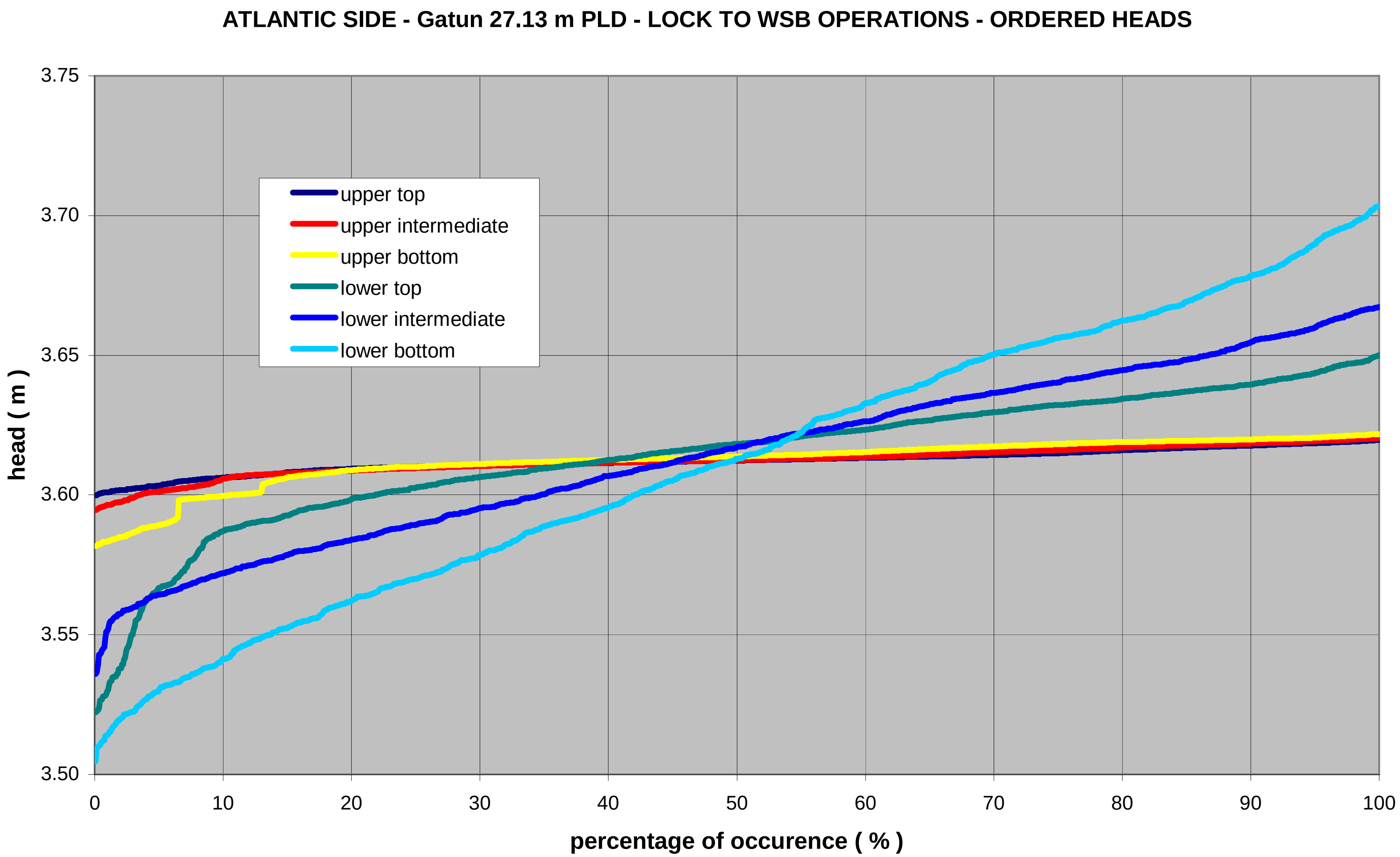
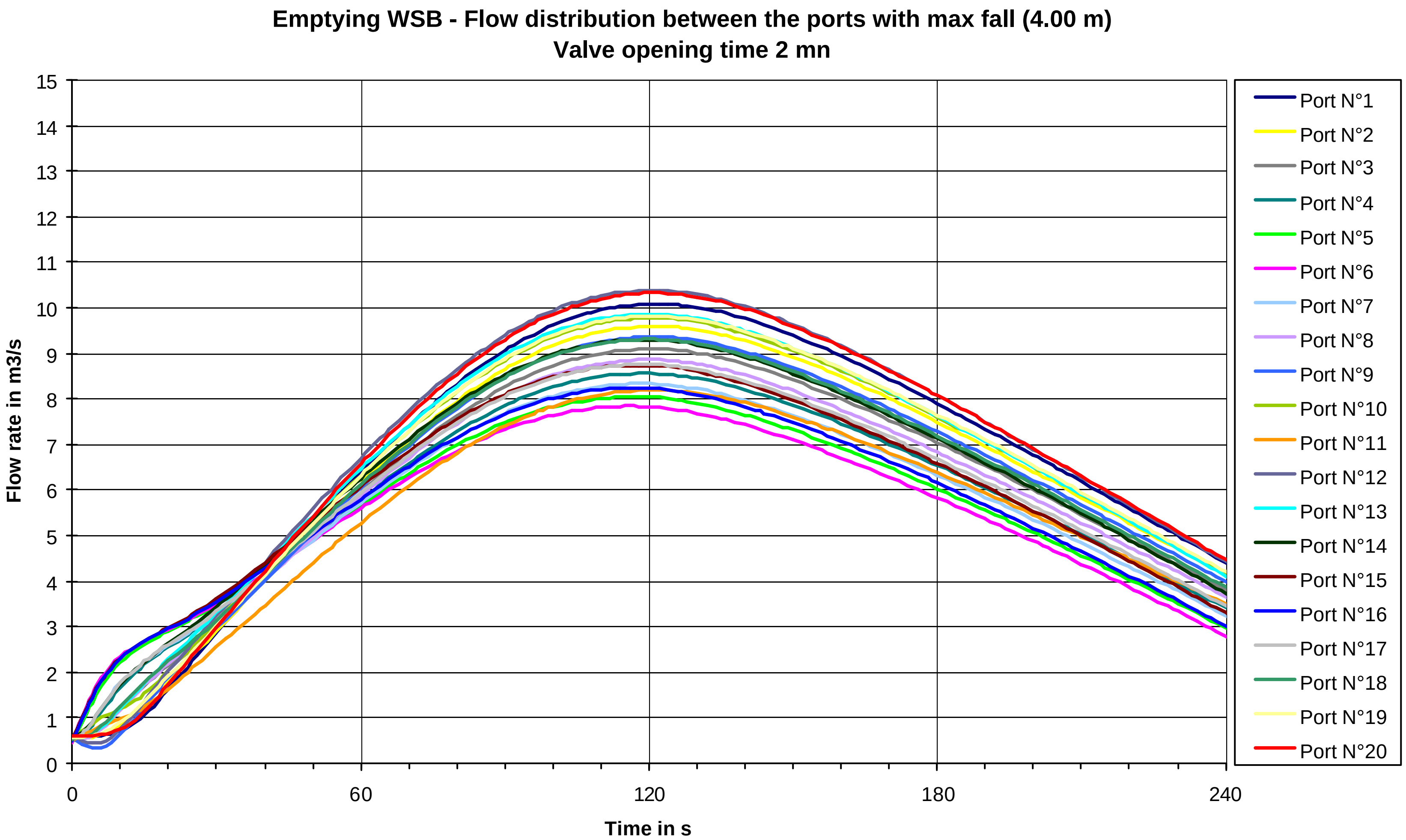
01/01/2000
31/12/2000

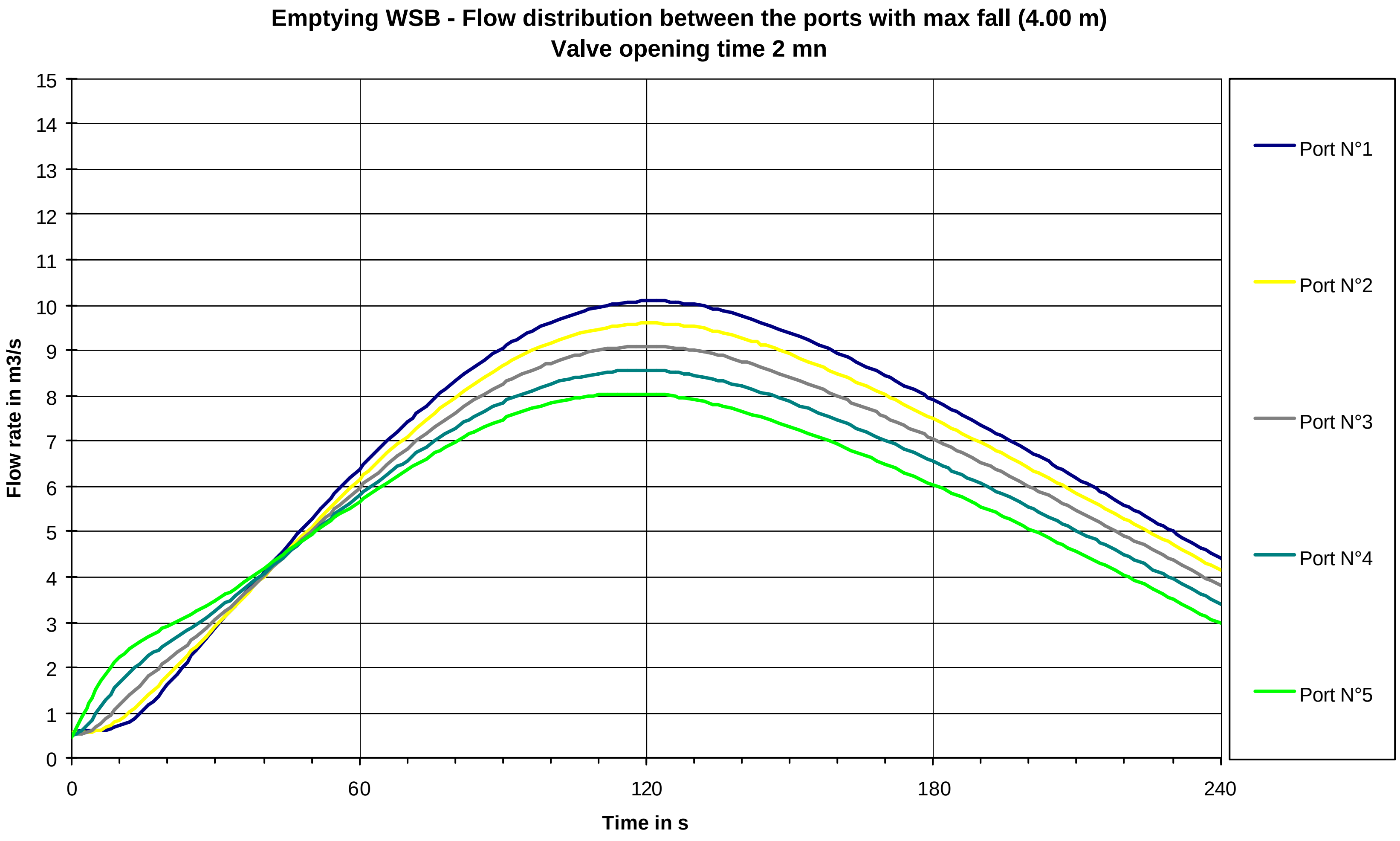

ATLANTIC SIDE - Gatun 27.13 m PLD - LOCK TO LOCK OPERATIONS - ORDERED HEADS
head (m)
8.00
7.50
7.00
6.50
6.00
5.50
5.00
4.50
4.00
3.50
3.00
0
10
20
30
40
50
60
70
80
90
100
frequency of occurence (%)
Gatun lake / upper lock head
upper lock / middle lock head
middle lock / lower lock head
lower lock / Atlantic Ocean head


ATLANTIC SIDE - Gatun 27.13 m PLD - WSB TO LOCK OPERATIONS - ORDERED HEADS
-3.40
-3.50
-3.60
-3.70
-3.80
-3.90
-4.00
head (m)
0
10
20
30
40
50
60
70
80
90
100
percentage of occurence (%)
upper top
upper intermediate
upper bottom
lower top
lower intermediate
lower bottom


ATLANTIC SIDE - Gatun 27.13 m PLD - LOCK TO WSB OPERATIONS - ORDERED HEADS
head (m)
3.75
3.70
3.65
3.60
3.55
3.50
0
10
20
30
40
50
60
70
80
90
100
percentage of occurence (%)
upper top
upper intermediate
upper bottom
lower top
lower intermediate
lower bottom

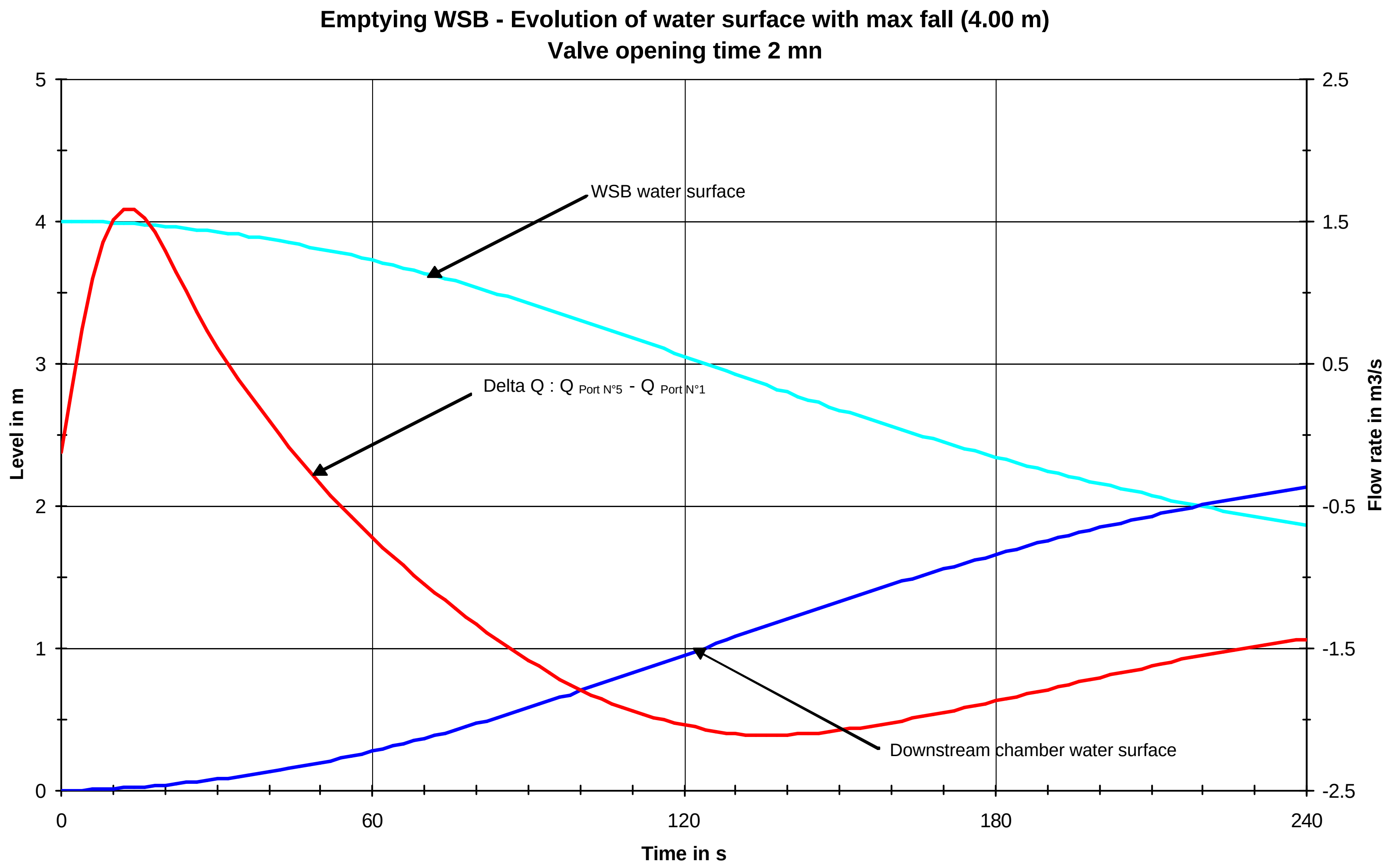

Emptying WSB - Flow distribution between the ports with max fall (4.00 m)
Valve opening time 2 mn
Flow rate in m3/s
Time in s
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
60
120
180
240
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5
Port N°6
Port N°7
Port N°8
Port N°9
Port N°10
Port N°11
Port N°12
Port N°13
Port N°14
Port N°15
Port N°16
Port N°17
Port N°18
Port N°19
Port N°20

Annex 2-2.doc



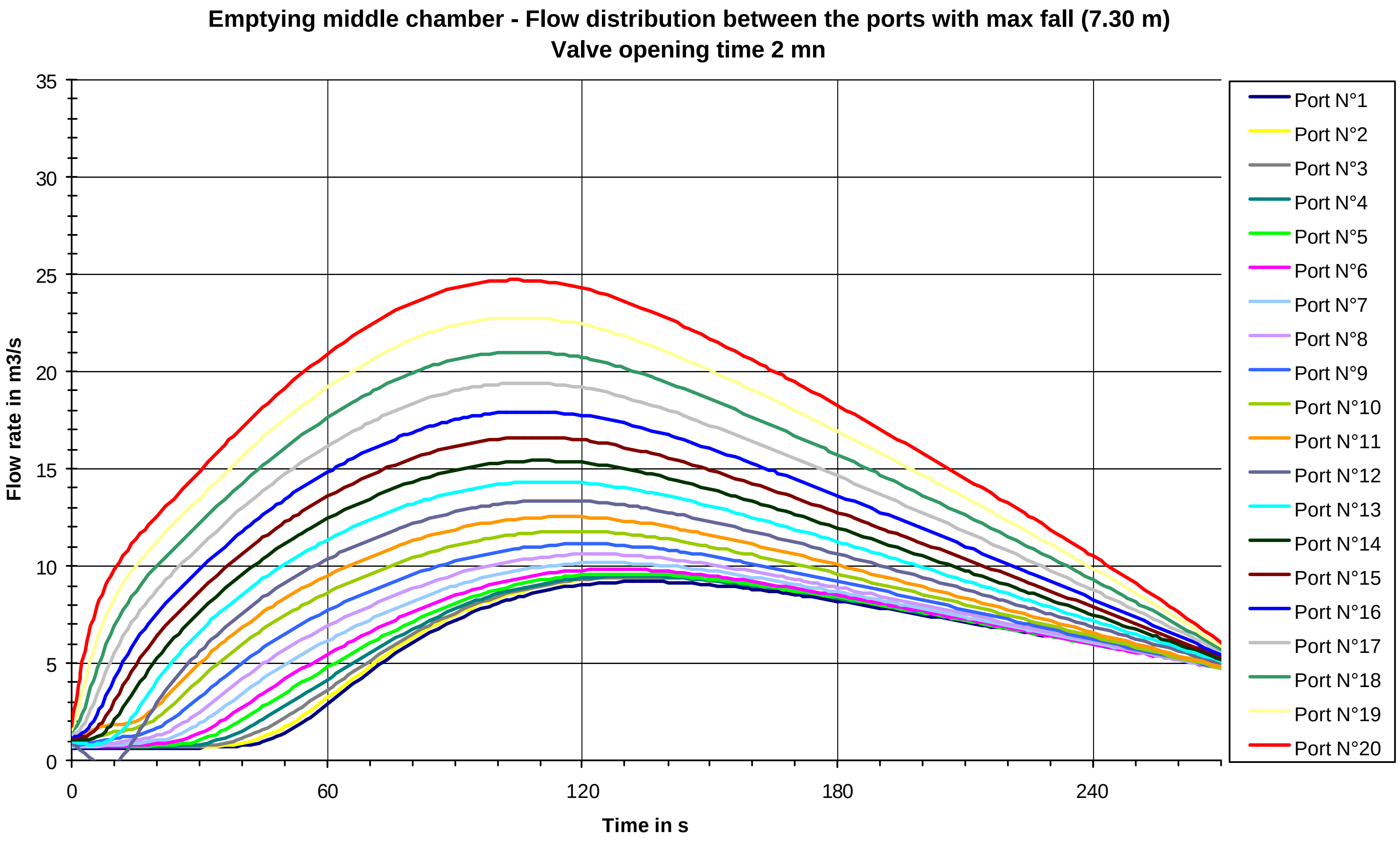
Emptying WSB - Flow distribution between the ports with max fall (4.00 m)
Valve opening time 2 mn
Flow rate in m3/s
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Time in s
0
60
120
180
240
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5

Annex 2-3.doc



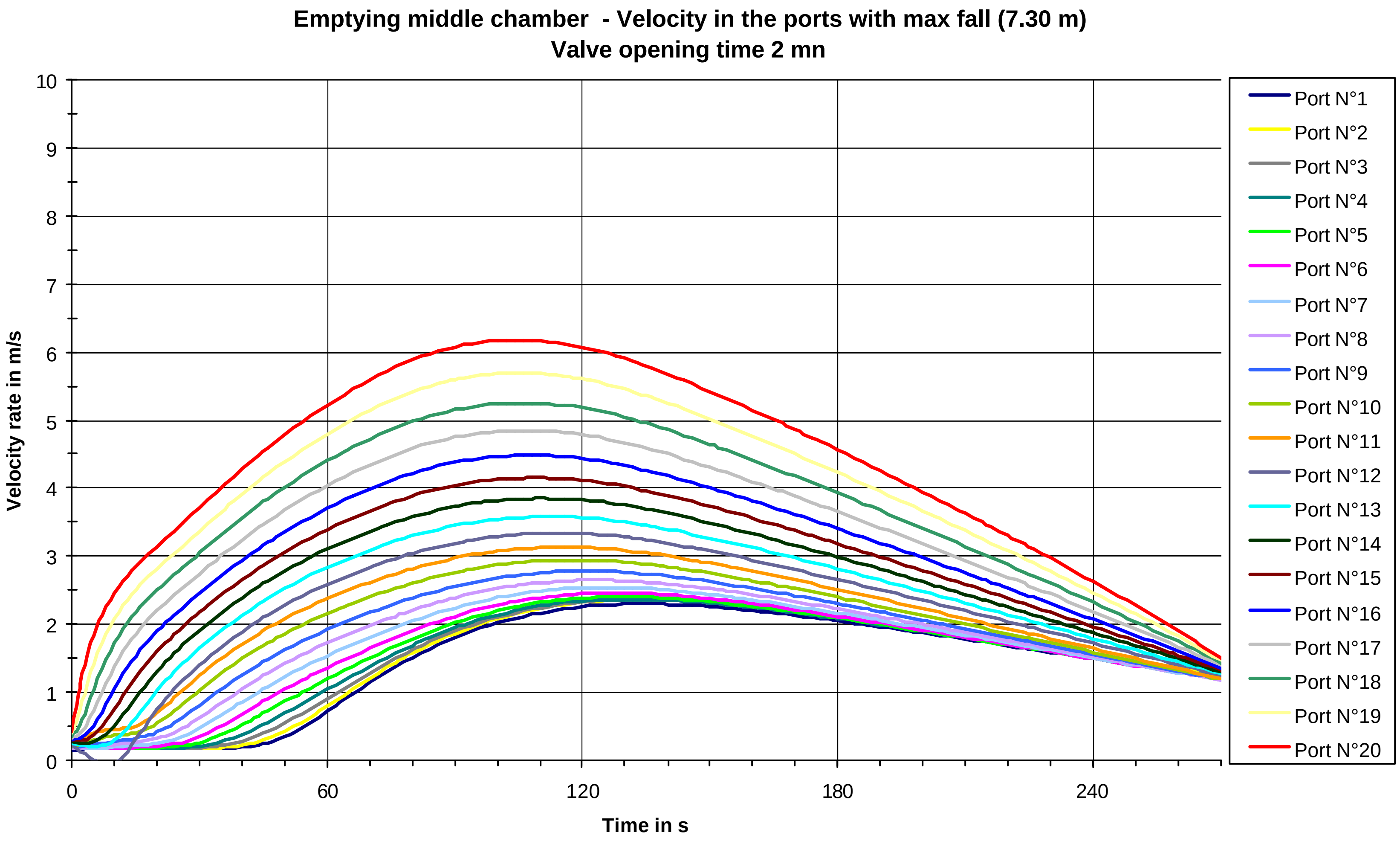
Emptying WSB - Evolution of water surface with max fall (4.00 m)
Valve opening time 2 mn
WSB water surface
Delta Q : Q Port N°5 - Q Port N°1
Downstream chamber water surface
Level in m
Flow rate in m3/s
Time in s
0
1
2
3
4
5
2.5
1.5
0.5
-0.5
-1.5
-2.5
0
60
120
180
240

Annex 3-1.doc



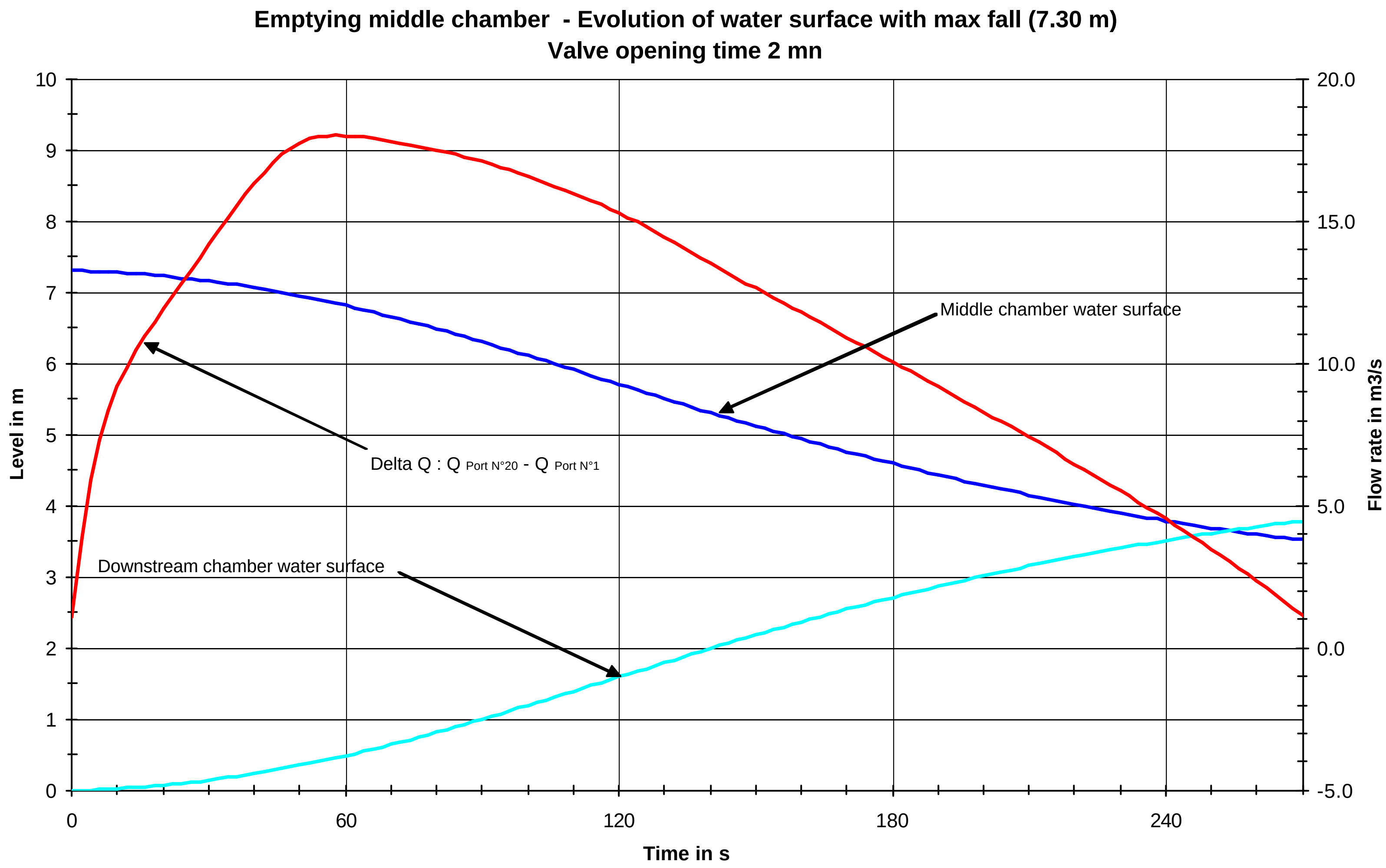
Emptying middle chamber - Flow distribution between the ports with max fall (7.30 m)
Valve opening time 2 mn
Flow rate in m3/s
35
30
25
20
15
10
5
0
0
60
120
180
240
Time in s
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5
Port N°6
Port N°7
Port N°8
Port N°9
Port N°10
Port N°11
Port N°12
Port N°13
Port N°14
Port N°15
Port N°16
Port N°17
Port N°18
Port N°19
Port N°20

Annex 3-2.doc

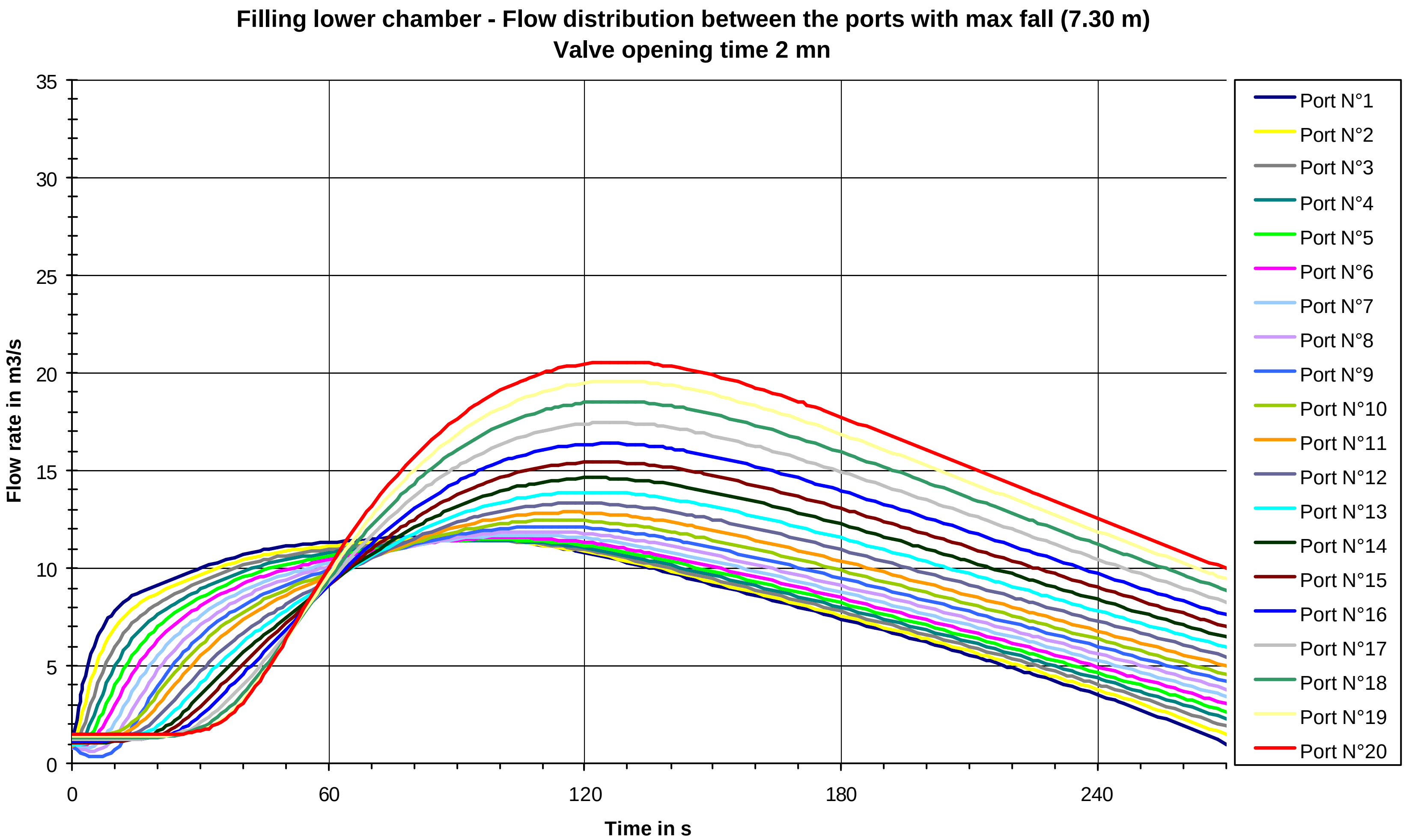


Emptying middle chamber - Velocity in the ports with max fall (7.30 m)
Valve opening time 2 mn
Velocity rate in m/s
10
9
8
7
6
5
4
3
2
1
0
0
60
120
180
240
Time in s
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5
Port N°6
Port N°7
Port N°8
Port N°9
Port N°10
Port N°11
Port N°12
Port N°13
Port N°14
Port N°15
Port N°16
Port N°17
Port N°18
Port N°19
Port N°20

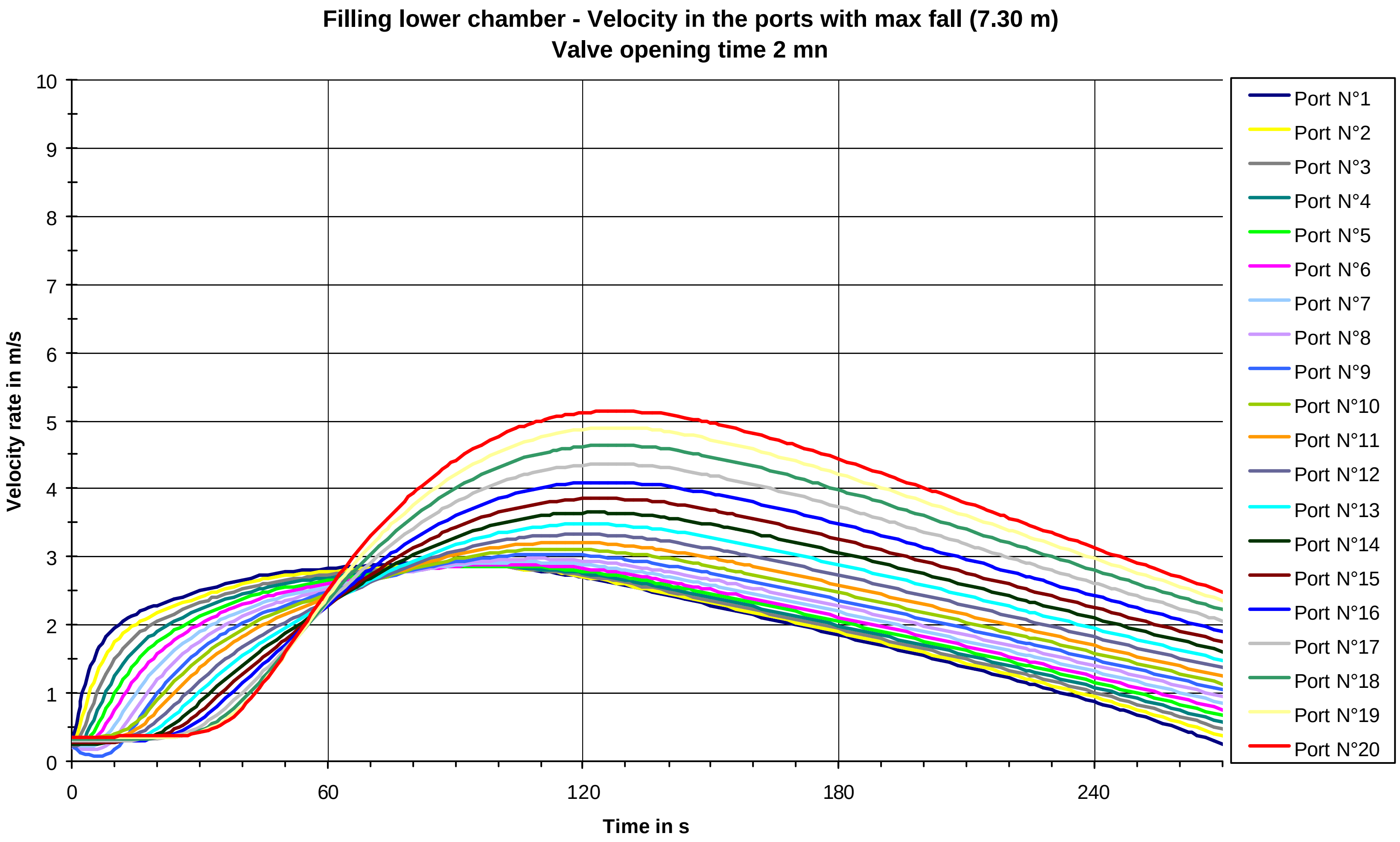
Annex 3-3.doc


Emptying middle chamber - Evolution of water surface with max fall (7.30 m)
Valve opening time 2 mn
Level in m
Flow rate in m3/s
Time in s
0
60
120
180
240
10
9
8
7
6
5
4
3
2
1
0
20.0
15.0
10.0
5.0
0.0
-5.0
Middle chamber water surface
Delta Q : Q Port N°20 - Q Port N°1
Downstream chamber water surface

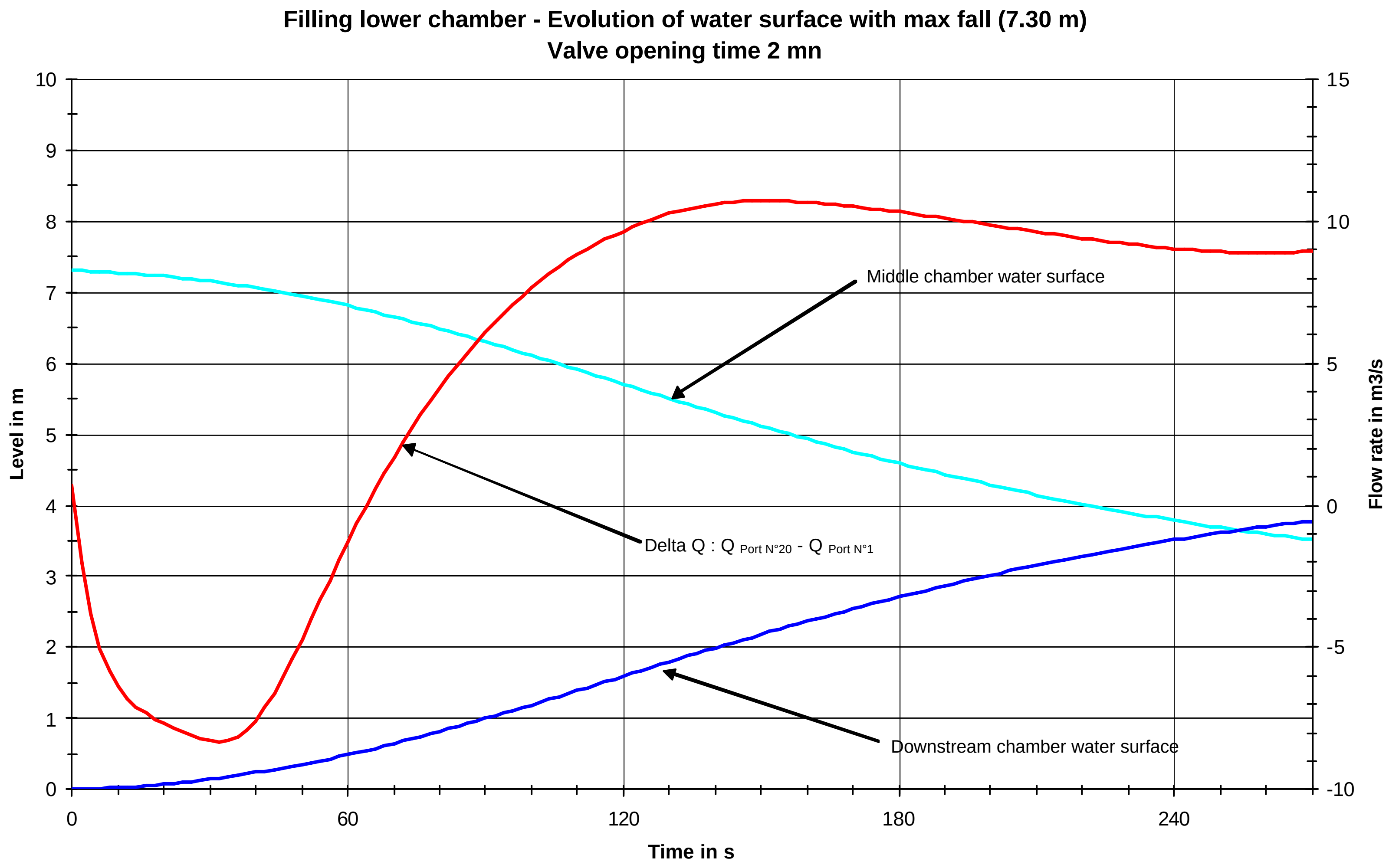
Annex 4-1.doc


Filling lower chamber - Flow distribution between the ports with max fall (7.30 m)
Valve opening time 2 mn
Flow rate in m3/s
Time in s
0
5
10
15
20
25
30
35
0
60
120
180
240
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5
Port N°6
Port N°7
Port N°8
Port N°9
Port N°10
Port N°11
Port N°12
Port N°13
Port N°14
Port N°15
Port N°16
Port N°17
Port N°18
Port N°19
Port N°20

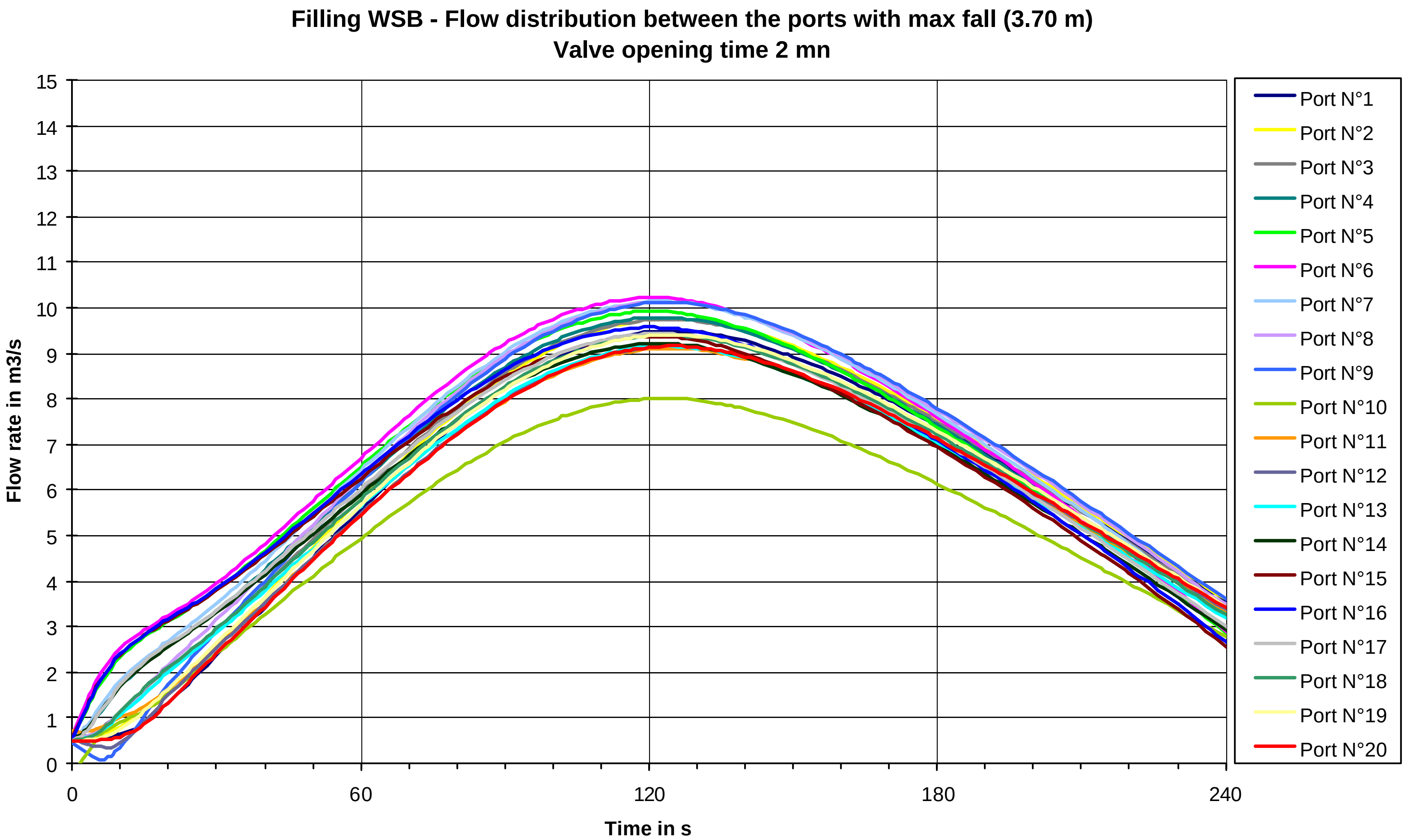
Annex 4-2.doc



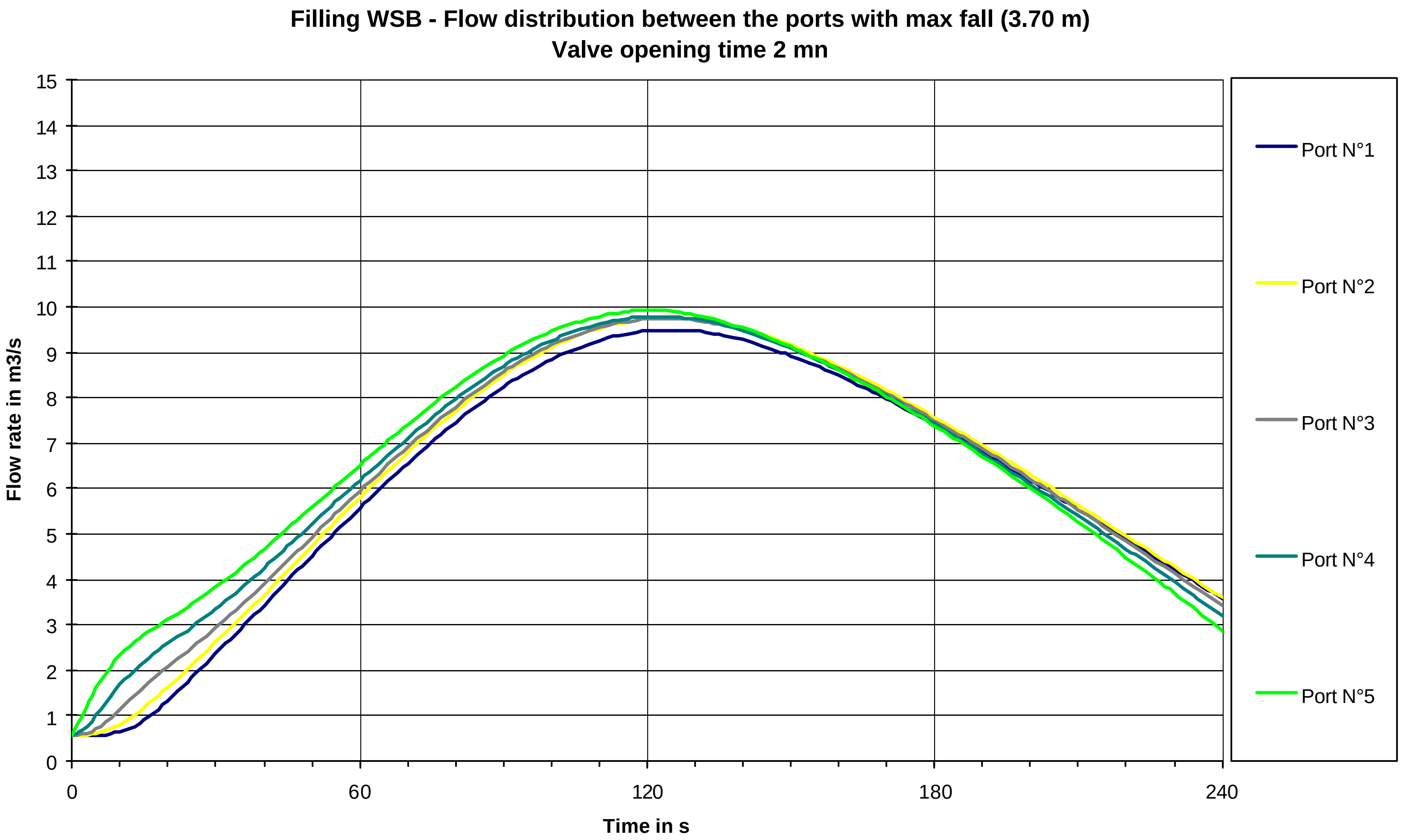
Filling lower chamber - Velocity in the ports with max fall (7.30 m)
Valve opening time 2 mn
Velocity rate in m/s
Time in s
0
1
2
3
4
5
6
7
8
9
10
0
60
120
180
240
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5
Port N°6
Port N°7
Port N°8
Port N°9
Port N°10
Port N°11
Port N°12
Port N°13
Port N°14
Port N°15
Port N°16
Port N°17
Port N°18
Port N°19
Port N°20



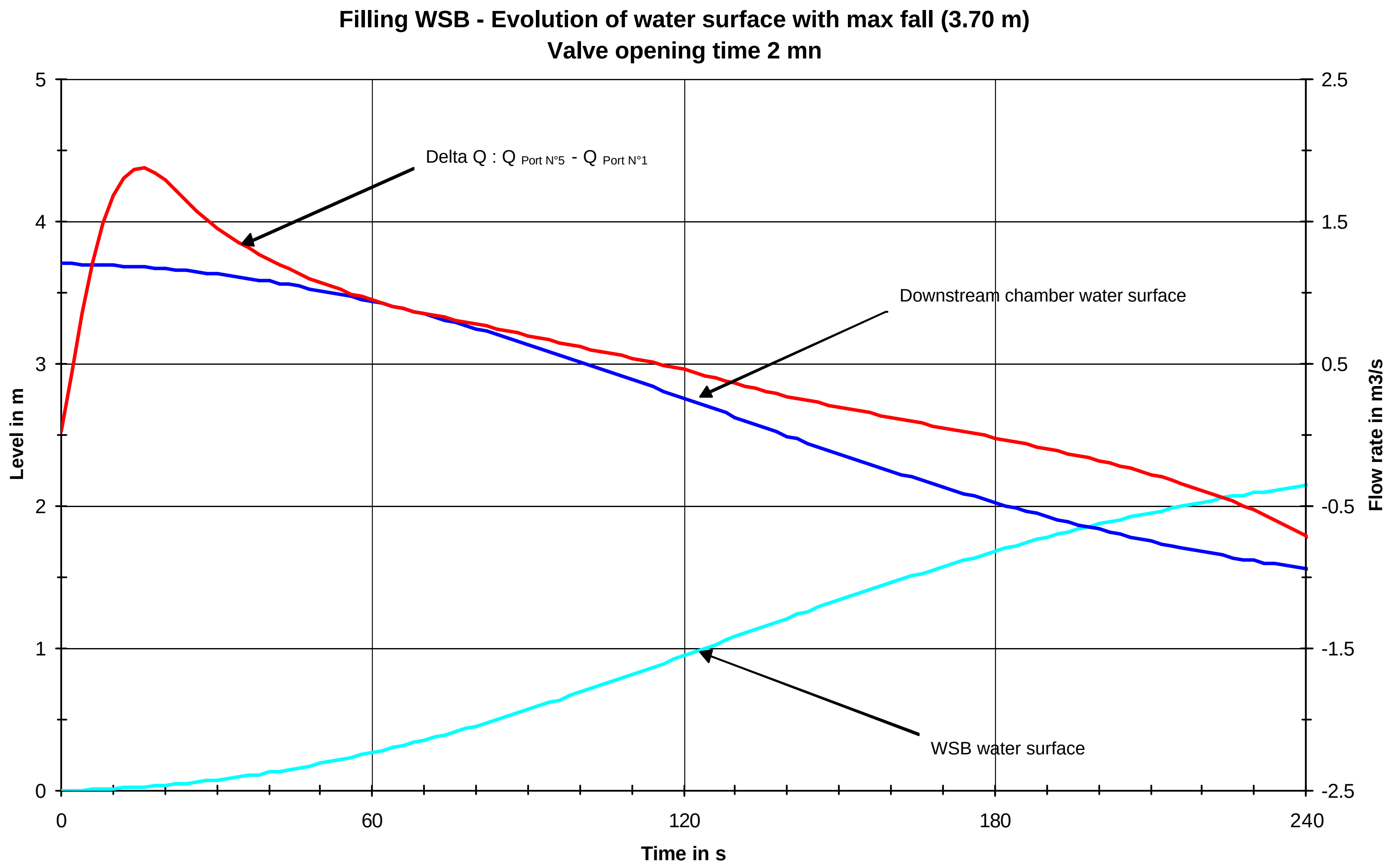
Filling lower chamber - Evolution of water surface with max fall (7.30 m)
Valve opening time 2 mn
Level in m
Flow rate in m3/s
Time in s
0
1
2
3
4
5
6
7
8
9
10
15
10
5
0
-5
-10
0
60
120
180
240
Middle chamber water surface
Delta Q : Q Port N°20 - Q Port N°1
Downstream chamber water surface


Filling WSB - Flow distribution between the ports with max fall (3.70 m)
Valve opening time 2 mn
Flow rate in m3/s
Time in s
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
60
120
180
240
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5
Port N°6
Port N°7
Port N°8
Port N°9
Port N°10
Port N°11
Port N°12
Port N°13
Port N°14
Port N°15
Port N°16
Port N°17
Port N°18
Port N°19
Port N°20

Annex 5-2.doc

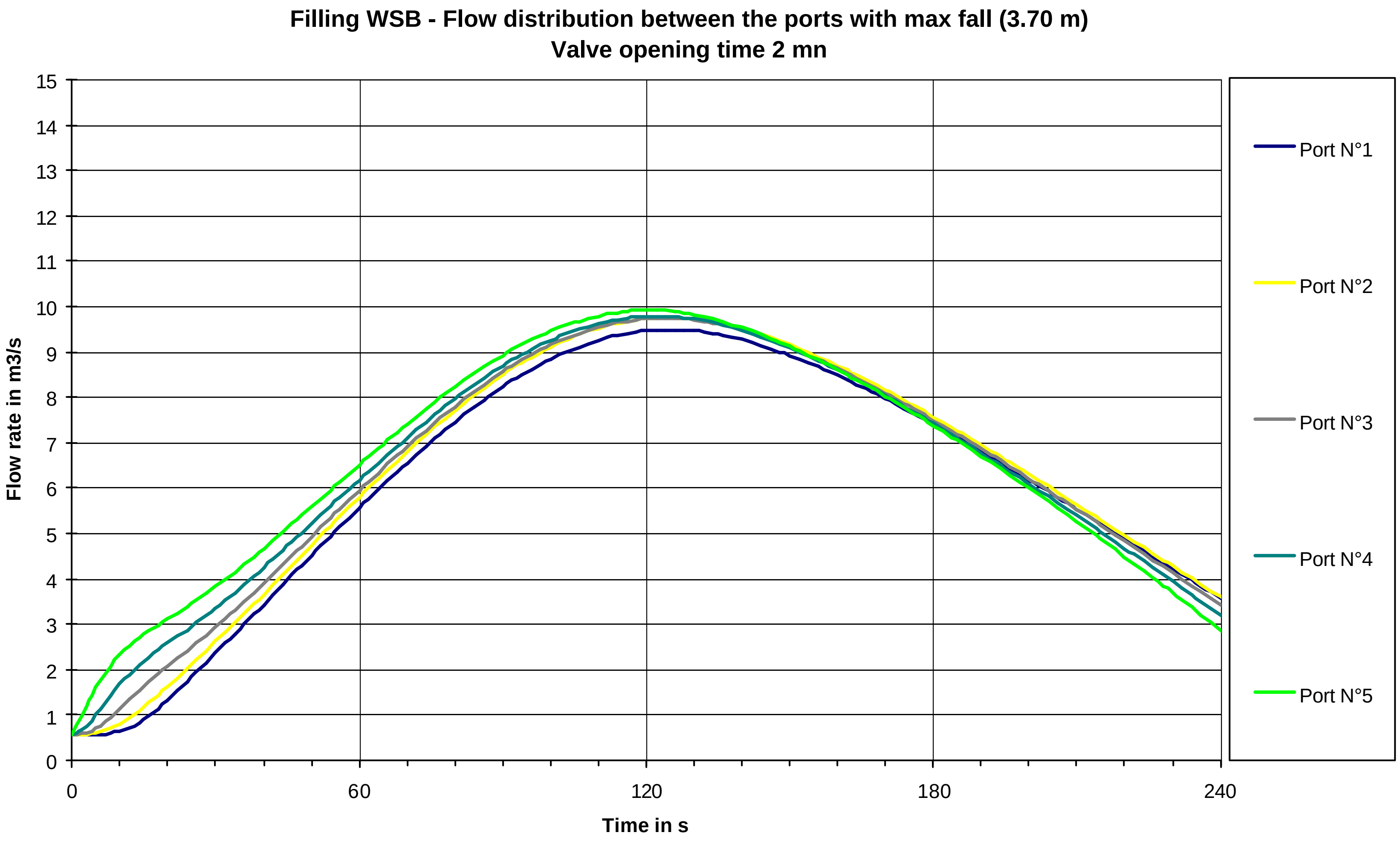


Filling WSB - Flow distribution between the ports with max fall (3.70 m)
Valve opening time 2 mn
Flow rate in m3/s
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
0
60
120
180
240
Time in s
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5

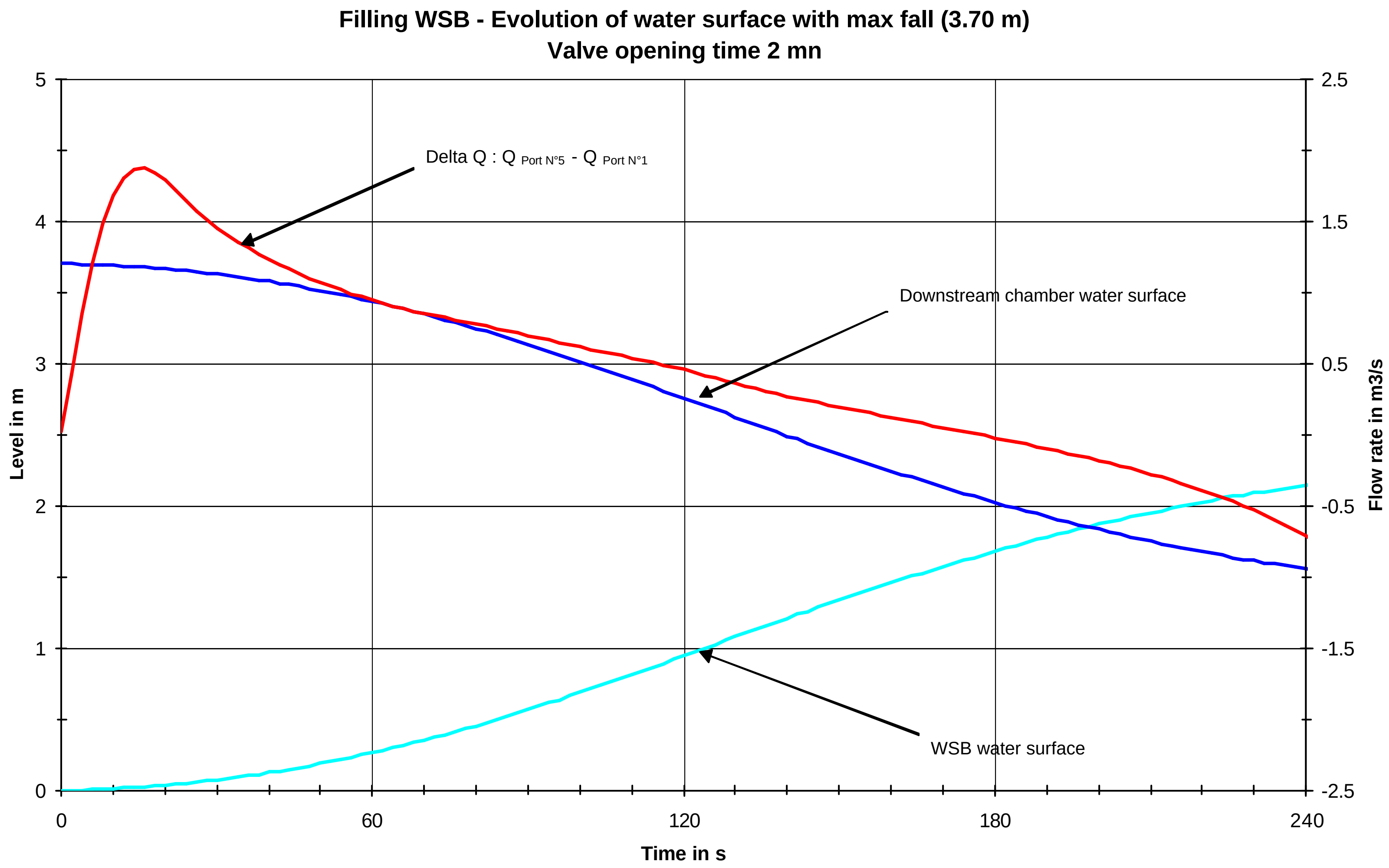


Filling WSB - Evolution of water surface with max fall (3.70 m)
Valve opening time 2 mn
Delta Q : Q Port N°5 - Q Port N°1
Downstream chamber water surface
WSB water surface
Level in m
Flow rate in m3/s
Time in s
0
60
120
180
240
0
1
2
3
4
5
-2.5
-1.5
-0.5
0.5
1.5
2.5

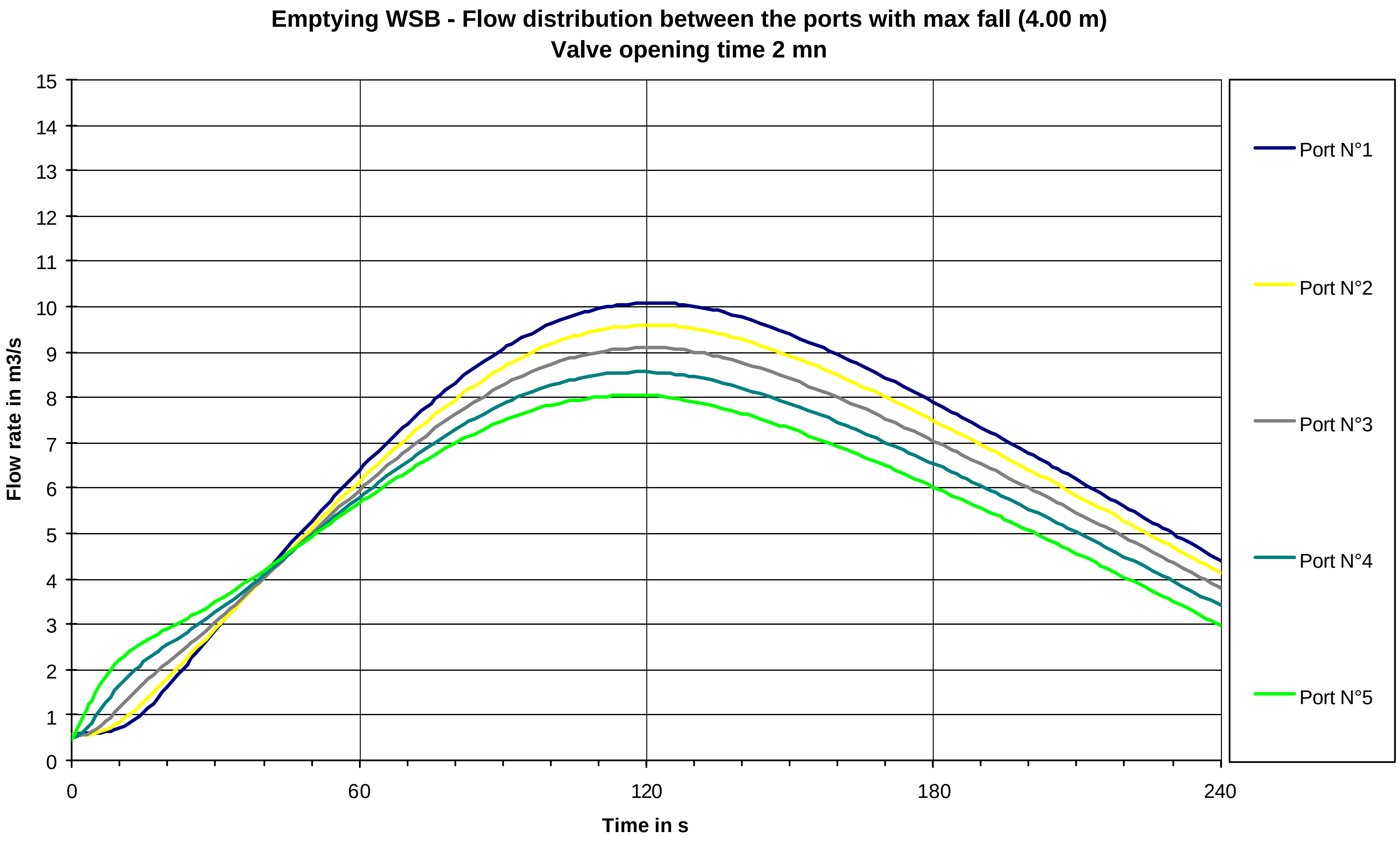
Annex 6-1.doc



Filling WSB - Flow distribution between the ports with max fall (3.70 m)
Valve opening time 2 mn
Flow rate in m3/s
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
0
60
120
180
240
Time in s
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5

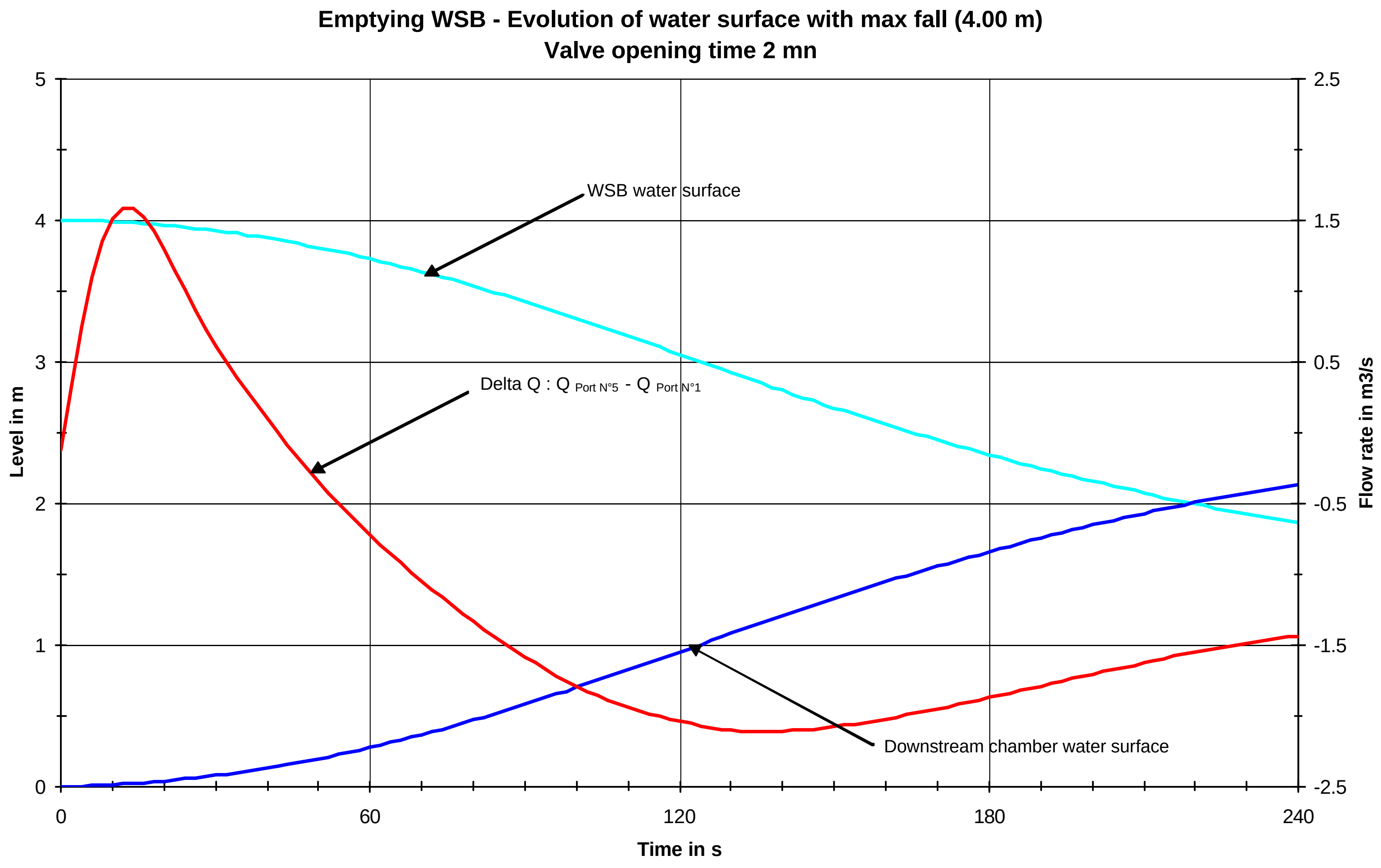

Filling WSB - Evolution of water surface with max fall (3.70 m)
Valve opening time 2 mn
Delta Q : Q Port N°5 - Q Port N°1
Downstream chamber water surface
WSB water surface
Level in m
Flow rate in m3/s
Time in s
0
1
2
3
4
5
2.5
1.5
0.5
-0.5
-1.5
-2.5
0
60
120
180
240

Annex 7-1.doc



Emptying WSB - Flow distribution between the ports with max fall (4.00 m)
Valve opening time 2 mn
Flow rate in m3/s
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
0
60
120
180
240
Time in s
Port N°1
Port N°2
Port N°3
Port N°4
Port N°5

Annex 7-2.doc



Emptying WSB - Evolution of water surface with max fall (4.00 m)
Valve opening time 2 mn
5
4
3
2
1
0
2.5
1.5
0.5
-0.5
-1.5
-2.5
Level in m
Flow rate in m3/s
WSB water surface
Delta Q : Q Port N°5 - Q Port N°1
Downstream chamber water surface
0
60
120
180
240
Time in s

Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Lock Conceptual design - Contract SAA-143351

ATLANTIC LOCKS HARMONIZATION

TASK A4d – LOCK GATES – Layout of rolling gates

Tractebel Development Engineering   TECHNUM   COYNE ET BELLIER   Compagnie Nationale du Rhône

in association with

SBE   PORT OF ANTWERP   FIRA AOSO

Layout of rolling gates

1 BASIC DATA

1.1 WATER AND BOTTOM LEVELS

Following water levels and corresponding bottom levels of the different lock chambers are taken in account for the concept design of the gates structures and layout. All level data are in m and refer to PLD. Note that (only) for lock head AT1 a minimum sill depth of 16.76 m (55 ft) is proposed.

	Gate AT1	Gate AT2	Gate AT3	Gate AT4
Max. water level upstream	+ 27.13	+ 27.13	+ 18.14	+ 9.19
Min. water level open/close gates	+ 24.99 (*)	+ 16.69	+ 8.37	- 0.15
Min. water level downstream	+ 16.69	+ 8.37	- 0.15 (**)	- 0.15 (**)
Sill level (downstream)	+ 8.23	- 1.61	- 9.93	- 18.45

(*) For maintenance operations in the gate recesses a safety allowance is considered: the minimum water level for floating operations of the gates is taken +23.00 m PLD (see 2.3)
(**) No minimum downstream water level below this value is considered for an ultimate loading case

1.2 GEOMETRY

The axis to axis distance between the vertical bearings of the gates at the lock walls is taken 57.24 m (i.e.: the width of the lock chamber plus an excess length of 2.24 m).
The center of the horizontal bearing at the bottom of the gates is positioned 0.40 m below the level of the sill. The axis level of the horizontal bottom frame will be situated 1.27 m above this level in order to facilitate access under the gates during maintenance and repair operations, in particular to replace the lower support wagon.

No specific optimum shaping is considered: at the relatively low operating speeds of gates the hydrodynamic forces are relatively small compared with the inertia forces acting simultaneously on the heavy lock gate structures and the large moving water masses which are exchanged between the gate recesses and the lock chamber during operations.

To achieve a smooth filling and emptying (F/E) of the lock chamber, with a minimum delay of the water level evolution in the space between a pair of lock gates in each lock head, openings may be introduced in the lower upstream skin plating panels. Consequently the layout of the stiffening of these panels (in compression!) has to be adapted accordingly.
Possibly even valves are to be built in the gates for a fast water level equalization throughout.

Upstream gates AT1
The skin plating extends over an additional 0.45 m (horizontal) and 0.25 m (vertical) from the above defined bearing axes. Thus the total horizontal length of the gate skin plating is 58.14 m.

Intermediate and downstream gates AT2 - AT3 resp. AT4
For gates AT2, AT3 and AT4 the axis to axis distance between the vertical bearings of the gates at the lock walls is taken 57.24 m like for gates AT1. Added to the width of the lock chamber (55 m), gates AT2, AT3 and AT4 also have an excess length of 2.24 m. At this conceptual design stage there is no evidence that additional excess length at the rear end would be required considering the accommodation in the lock head for the slot bulkhead (see 3.8) at the gate recesses.
The skin plating extends over an additional 0.45 m (horizontal) and 0.25 m (vertical) from the above defined bearing axes. Thus the total horizontal length of the gate skin plating is 58.14 m.

1.3 STANDARDIZATION

Based on the basic data listed above, the following assumptions on standardization are made for the concept design of the gates structures and layout.

1. The upstream gates AT1 are identical to the corresponding gates PA1 at the Pacific locks.
2. The intermediate gates AT2 and AT3 have the same structural layout, being identical to the intermediate gates PA2 and PA3 of the Pacific locks complex. As a consequence, gates AT2 have an increased height of 0.13 m and gates AT3 have an increased height of 0.80 m compared to the "tailor made" situations of 1 m freeboard above the local maximum water level. Conversely gates AT2 and AT3 have a level of air chamber which is 0.22 m lower than the level which normally would have been chosen.
   Because the proposed gate structures are practically "tailor made" to the Pacific locks, the said deviations from "tailor made" structures for the considered Atlantic locks case are somewhat larger; the additional cost of these are commented in section 2.5 of this report.
   An important advantage of the proposed standardization among the Atlantic and Pacific locks is a certain flexibility during the construction phase of the two lock structures: according to the progress of both works, a call can be made on the steel constructor to deliver these "standardized" gates on either site. Obviously the latter advantage doesn't apply to justify an extension of the lock gates standardization AT2 = AT3 = PA2 = PA3 to the most downstream gates AT4, as the corresponding gates PA4 have been tailor made. Note that such a "standardized" gate AT4 would

be 1.23 m (!) higher compared to the "tailor made" situation of 1 m freeboard. Therefore a "tailor made" structure is proposed and designed for gates AT4. Still for this gates also a "standardized" structure may be considered, the cost of which is commented in section 2.5 of this report.
3. The center of the horizontal bearing at the bottom of all gates remains positioned 0.40 m below the level of the sill at all lock heads; no deepening of the lock heads below the gates is proposed to minimize the risk of objects getting stuck between the seals of the gates. As a consequence, gates AT2 and AT3 have an increased freeboard (1.13 - 1.80 m) above the local maximum water level.
4. All relative measures of support rolling wagons and cantilevering branch remain identical among standardized gates, in order to keep all mechanical connection parts identical. As a consequence, support rolling wagons and their rails for gates AT2 and AT3 have an increased level above the local maximum water level, causing a build up with increased height above the lock platform at the lock heads AT2 and AT3.
5. Lower support rolling wagons will be standardized as much as possible (as a - preferably common - spare one is proposed); upper support rolling wagons may be "tailor made" (as no spares are proposed, considering its relative ease of maintenance thanks to its position above the water level).
6. The thickness of the skin plating and the type of stiffener profile may be differentiated for each panel in the cross section only. No differentiation is considered in the longitudinal direction of the gates, yielding a maximum economy of scale (e.g.: automatic welding) during construction and a minimum risk of human error during assembly.
7. The number of different plate thicknesses and types of stiffener profiles are kept to a minimum.

1.4 LOAD CASES ASSUMPTIONS

The following assumptions on load cases are made for the concept design of the gates structures and layout.

1. Traffic load is considered to be of a moderate level for which normal bridge deck designs apply.
2. The gates resist the water pressures due to the possible differences between the water levels listed in the table sub 1.1. An ultimate limit loading case with an increased maximum upstream water level by 1 m is considered (in combination with the same minimum downstream water level at gates AT3 and AT4). Gates AT1 and AT4 (reversed - see 3.9) are verified to be able to retain the water (outside the lock) in case of a lock chamber dewatering.

   As an accidental load case the gates are able to additionally resist the water pressures induced by “maximum credible” earthquakes (MCE) with a peak ground acceleration (PGA) up to 0.41 g. In this concept design the quasi-static “Westergaard” procedure is adopted to estimate these excess pressures. The horizontal seismic coefficient (kh) is taken according to the PIANC seismic design guidelines for port structures (see design criteria).

   A reduction factor according to Housner is applied for the case of water pressures generated in bodies of water with finite length (relative to the total waterdepth), i.c.: between the two skin platings of the lock gates and between two adjacent lock gates.

   More sophisticated (dynamic) analysis procedures may reveal more precise (less conservative) conclusions. These may be considered in a more detailed phase of the design.
3. During operations the gates are not subject to back pressures from the downstream side: locking procedures will be designed such that the water level at the downstream side of each gate never exceeds the water level at the upstream side.

4. The stability against rolling over of a floating gate is not considered as a determinative design criterion; however the metacentric height will be checked afterwards to assess the possible need for auxiliary equipment and provisions during floating operations.
5. Floating of the gates stabilized in upright position is still possible after complete flooding of three watertight zones (of each two compartments of the air chamber - length = 2 * 3.18 m = 6.36 m), e.g. due to local damage after a ship collision. It is assumed that a bulb of a huge ship can damage at the same time two adjacent transversal walls between watertight zones.

2 GATE STRUCTURE DESIGN

2.1 GENERAL

In what follows the concept design of the structure and layout is presented for rolling gates of the “wheelbarrow” type, i.e. with a submerged lower support wagon running on rails at the bottom of the lock chamber and an upper support wagon on rails above the water.

For all gates a symmetrical layout with double skin plating is proposed in order to achieve a maximum of safety against global collapse in the event of accidental ship collisions. Weight reduction through the use of compact compression members instead of compressed plating is marginal. Through an appropriate choice of the width of the gates (enabling to float them in a stable manner), material stress levels are kept sufficiently limited to guarantee safety against buckling risk, which is determinative in this case.

Note on buckling design standards: compressed plating is designed against buckling according to the Belgian standard NBN B 51-002 (August 1988), based on the linear elastic buckling theory (similar to for example DIN 4114) in combination with the safety concept adopted in the present Eurocode 3. The appropriate effective stiffeners are designed according to the technical notes of the CECM.

Observing these options, the same design procedure is followed as described in the methodology of the earlier CPP concept design study.
Note on fatigue: the gate structures (and most of the load bearing parts fixed to it) are subjected to cyclic loading and unloading, going along with the locking operations. The number of cycles over a 100 years life time design period is set at N = 25 cycles/day x 36525 days = 9.125×10^5 (see section 3.10 of the specific design criteria).

Based on our experience with common material stress levels (below the limit value for fatigue) for the proposed steel grade S355J2G3 and with common overall layouts of the structural framework of rolling gates, an a priori estimate is made of the total width and weight of the gates and the spacing between the vertical frames.

Thus the necessary height of the air chamber is estimated and a local analysis yields the necessary "equivalent" thickness of the plating (i.e.: taking in account the material consumption for the necessary longitudinal stiffeners – see 2.2). The position of the top and bottom of the air chamber is fixed observing the minimum water level at closing and opening operations (see 2.3) and an appropriate upper deck is designed for the traffic loads to be considered (2.4). Based on the assumption of an equivalent plate thickness of the intermediate horizontal frames, an overall cross section analysis yields an estimate of the torsion stiffness distribution over the horizontal frames, taking in account their contingence.
Then a 2D-beam grid model of the gate structure (integrated over the width) is drawn up. Subsequently the assumed material tension levels are verified, in particular for the design of the compressed parts against buckling and the (local) design of the longitudinal stiffeners. Both elements may have a considerable impact on the final weight distribution of the gate structure, allowing to verify the height of the air chamber, the reaction forces at the support wagons and the floating stability.

Hereafter the concept design of some particular parts of the lock gates is discussed. It will be indicated how a more detailed level of design may lead to optimization of the obtained results.

Finally the main characteristics are listed (2.5) for which convergence occurs with the design procedure mentioned above.
Gates AT2 and gates AT3 have an identical structure (occasionally denoted as AT2-3). Moreover it appears that gates AT2-3 and AT4 require the same width. Therefore an identical width is proposed for the three lower support wagons. Thus it is possible to limit the number of different spare parts, which are interchangeable for these three types of gates.
From the attached cross section drawings of the proposed concepts it can be seen that the proposed intermediate water levels do not allow an identical gate structure for gates AT2-3 and AT4.

2.2 SKIN PLATING AND STIFFENERS

The skin plating is divided in different panels between the horizontal structural frames considered. Each panel is designed separately to retain the water pressure (p) that acts upon it. The choice of the thickness (e) of the plating is based on experience, such to limit the material tension for a chosen width of the gates. It is to be confirmed and possibly optimized in a more detailed level of design.

If B denotes the height of a panel, the number n of longitudinal stiffeners is determined such that their spacing $l = B/(n+1)$ satisfies the following:

$$\frac{pl^2}{12} \leq \sigma_{max} \cdot \frac{e^2}{6} \qquad (2.1)$$

If L denotes the horizontal length of a panel between two vertical structural frames, the required stiffener profile (minimum moment of inertia I and bending module W, in combination with an effective strip of plating) results from:

$$\frac{(pl)L^2}{24} \leq \sigma_{max} \cdot W \qquad (2.2a)$$

$$\frac{(pl)L^3}{384\,E\,I} \leq \left(f \big/ L \right)_{\max} \qquad (2.2b)$$

where σ_{max} and $(f/L)_{max}$ are respectively the maximum allowed material stress level (for local load transfer) and (local) displacement of the stiffener profile in combination with an effective strip of plating (taking in account the shear lag effect), which are chosen based on experience with similar structures and taking in account the (global) material stresses derived from the 2D-beam grid model.

Note that the adopted design value of the (local) bending moment (2.2a) holds halfway between two vertical frames. The doubled value locally occurring at the connection with the vertical frames is resisted by additional triangular stiffener plates, which also transfer the corresponding (local) shear forces to the frames. In order to arrive at moderate material stress levels in plating and connecting welds (fatigue!) the σ_{max}-value is chosen appropriately for each panel and each stiffener according to the occurrence frequency (probability) of the loading case considered (accidental, ultimate or service limit state).

From a catalogue of commercially available standardized steel profiles, the necessary cross sectional area A is derived such that the “equivalent” plate thickness (e_{eq}) for the 2D-beam grid characteristics and the weight estimation of the considered panel becomes:

$$e_{eq} = e + \frac{n \cdot A}{B} \qquad (2.3)$$

2.3 POSITION OF TOP AIR CHAMBER

In general the top level of the air chamber is to be chosen such that (if possible) the air chamber remains entirely submerged during opening and closing of the gates, thus avoiding excessive loading on the support wagons.

In this case the top level of the air chamber is taken 0.50 m below the lowest water level at which opening and closing of the gates will take place (see basic data sub 1.1), allowing for possible water level fluctuations in the lock chambers.

For gate AT1 an additional safety margin below the estimated lowest lake Gatun level is considered, with a head allowance to keep the slot bulkhead pressed against the bearings of the gate recess. Thus it is possible in all circumstances to create a sufficient free working space under the floating gates in the gate recess. This consideration does not apply to the other gates, as the water level in their gate recess may be adjusted (elevated) if necessary. The thus lowered position of the air chamber of gate AT1 does not significantly affect its floating stability, as it has a relatively low level of center of gravity (compared to the other gates) and the air chamber remains in a relatively elevated position.

2.4 ROAD BRIDGE DECK

For this concept design, the road bridge deck layout is taken similar to the one of the Berendrecht lock gates, which has more or less similar horizontal dimensions of deck panels between the structural frames and the edges. Furthermore two movable deck panels at the extremities are proposed to bridge the height difference from the top level of the gates to the ground level of the surrounding concrete structure by a ramp of 10% inclination angle. This would be applied for gates AT1 only.
These layouts are subject to further investigation in a more detailed level of design.

2.5 CONCLUSIONS: MAIN DIMENSIONS AND CHARACTERISTICS

The general cross sectional layout of the different gates for the Atlantic locks complex is shown on the attached drawings. A lateral and aerial view and a section over half of the gate length is shown as well.

The proposed inner diameter of the vertical access tubes is 1.0 m and the outer diameter of the horizontal access tube in the air chamber is 2.3 m. The air chamber has one longitudinal separation wall, constituting two adjacent watertight compartments in each cross section.

Although differences occur in the height and in the freeboard above the minimum operating water level, it is found that an identical structure for gates AT2 and AT3 (identical to those of the corresponding gates PA2 and PA3 of the Pacific locks complex) is appropriate (see 1.3). Gates AT2 and AT3 will have an increased freeboard allowance above the maximum water level in the corresponding upstream lock chambers of 1.13 m and 1.80 m respectively, compared with the 1 m assumed for the other ("tailor made") gates. Gates AT1 are identical to the corresponding gates PA1 for the Pacific locks.

Below some dimensions and characteristics are listed for the proposed concepts of the gates. Note that these figures are to be confirmed and subject to revision in a more detailed level of design.
E.g.: additional longitudinal separation walls in the air chamber will increase the stability against rolling over of a floating gate, but will complicate the separate accesses of the watertight compartments from the central horizontal access tube. Moreover the weight of the gates will increase substantially and the entire layout of the gates (below the top level of the air chamber) is subject to revision.

Axis levels (m PLD) of horizontal frames	Gate AT1	Gate AT2	Gate AT3	Gate AT4
R1: top of upper frame = bridge deck	+ 28.13	+ 28.26	+ 19.94	+ 10.19
R2: intermediate frame above air chamber	not present	not present (optimized gate structure)		
R3: intermediate frame above air chamber	+ 24.94 (*)	+ 22.11[5] (*)	+ 13.79[5] (*)	+ 5.42 (*)
R4: top of air chamber	+ 21.75	+ 15.97	+ 7.65	- 0.65
R5: bottom of air chamber	+ 15.95	+ 9.47	+ 1.15	- 6.85
R6: intermediate frame below air chamber	+ 12.52[5] (*)	+ 4.36[5] (*)	- 3.95[5] (*)	- 12.21[5] (*)
R7: bottom frame	+ 9.10	- 0.74	- 9.06	- 17.58
R8: bottom bearing at the sill	+ 7.83	- 2.01	- 10.33	- 18.85
R9: bottom edge of skin plating	+ 7.58	- 2.26	- 11.58	- 19.10

(*) no horizontal connecting framework nor plating present

	Gate AT1	Gate AT2	Gate AT3	Gate AT4
Width (outside plating)	7 m	10 m	10 m	10 m
Spacing between vertical frames	3.18 m	3.18 m	3.18 m	3.18 m
Weight per lateral area (height x length)	1340 kg/m²	1480 kg/m²	1480 kg/m²	1450 kg/m²
Weight of gate structure	1550 tons	2550 tons	2550 tons	2450 tons
Weight excess "standardized" vs. "tailor made" gate structures (approx.)	none	~ 0.5%	~ 3%	~ 5%

In general the figures above for gates AT1 are considered to be reasonable and very well in line with existing rolling gates. Gates AT2-3 and AT4 are of an extraordinary size, excluding any possible comparison. In general it is concluded that the proposed solution with rolling gates is well feasible as even the large gates AT2-3 and AT4 may be operated and handled in a safe and durable way, meeting all the practical criteria set out in the terms of reference.

Gates AT1 and AT4 are suitable for dewatering the lock chamber. For gates AT1 this doesn't cause additional costs since the gates almost completely run dry at their downstream side already during normal locking operations when the water level in the upstream lock chamber equalizes with the water level in the downstream lock chamber (normal loading case).
In principle gates AT4 (double plated but like gates AT1 with seals only on the downstream side) might be reversed for this purpose and maneuvered to an appropriately shaped separate bearing. Subsequently the dewatering of the lock constitutes a less severe loading case than the service loading case during normal locking operations (see section 3.9). Considering the relatively moderate maximum downstream water level at the Atlantic side (+0.41 m PLD) to retain at the downstream lock head in case of dewatering, an additional smaller type of gate, similar to AT1, would be an optimum solution. Doing so the dimensions of the additional bearing details may be kept to a minimum as well. In a more detailed phase of design, the necessity of bolted connections (possibility of dismounting) for the cantilevering branch (see section 3.5) can be investigated versus the possibility of (preferably) fixed (pre-fabricated) welded connections to the gate structure. The latter option possibly requires an adaptation of the lock wall structure at the top of the additional bearings.

3 LAY OUT OF DETAILS

3.1 GENERAL

In this section the conceptual lay outs of some operational features and details of the proposed rolling lock gates are presented. As stated in the methodology of the earlier CPP concept design study, similar lay outs of corresponding details of the rolling gates in operation at the large locks in the port of Antwerp (Belgium) are proposed.
Relevant excerpts from archive drawings of the Berendrecht lock gates are included in this report.
The appropriateness of the use of these details or those of the recently designed new rolling gates for the Van Cauwelaert lock (port of Antwerp) in the actual gates in the Panama case is evaluated where necessary, also considering the computations presented in section 5.

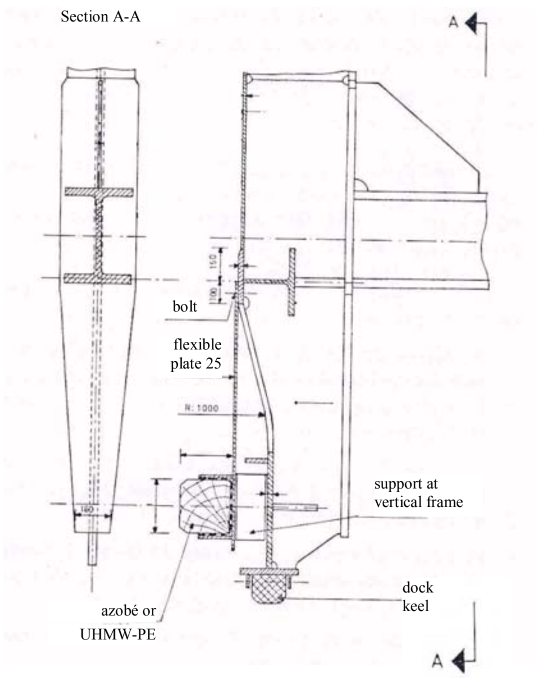
3.2 SEALS AND FENDERS

Both vertical (at the lock walls) as horizontal (at the bottom near the sill) seals are proposed to be massive beams. They have a rectangular cross section of 500 mm width and 250 mm thickness. The beams may be of azobé wood (considered in the computations) or a high performance composite material. The beams will be fixed on the skin plating on the downstream side of the gates between plate stiffeners of 150 mm width and 15 mm thickness.
These beams also constitute the supports of the gates against the bearings of the surrounding lock chamber structure (lock walls and sill). In section 5 it will be shown that the pressure levels in the beams require sufficient high strength materials, excluding currently available elastomeric materials.

3.3 FIXED SUPPORTS

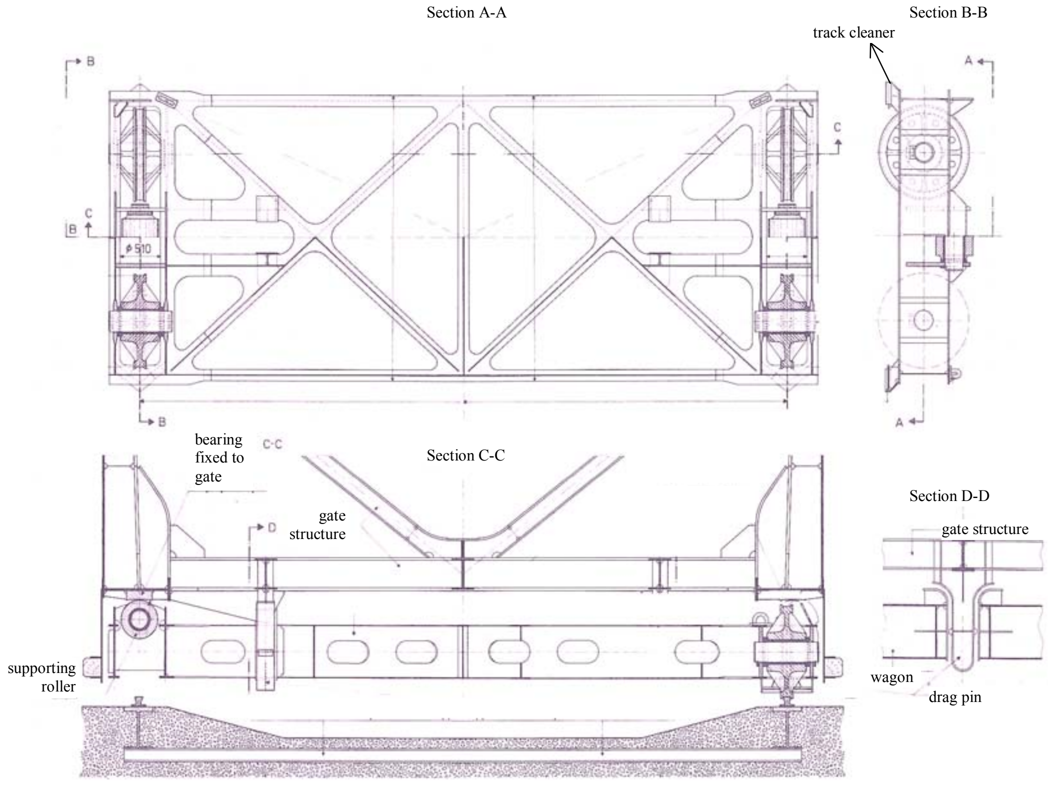
At the sill level the gates have cantilevering extensions of their vertical frames. At the bottom end of these extensions fixed supports are designed such that the gate can rest on the bottom of the lock chamber or a gate recess when the lower support wagon (see 3.4) is absent (e.g.: due to maintenance)

On the extensions at the downstream side of the gate a steel reinforced elastomeric block is fixed to support a flexible cantilevering extension of the skin plating (see illustration). This system allows to adjust to a certain degree the reaction forces which will be exerted on the gates at the sill. This may be necessary to avoid uplift from the sill bearing of the skin plating at the gate bottom edges due to an inappropriate distribution of stiffness over the gate structure.



Seals and bottom support

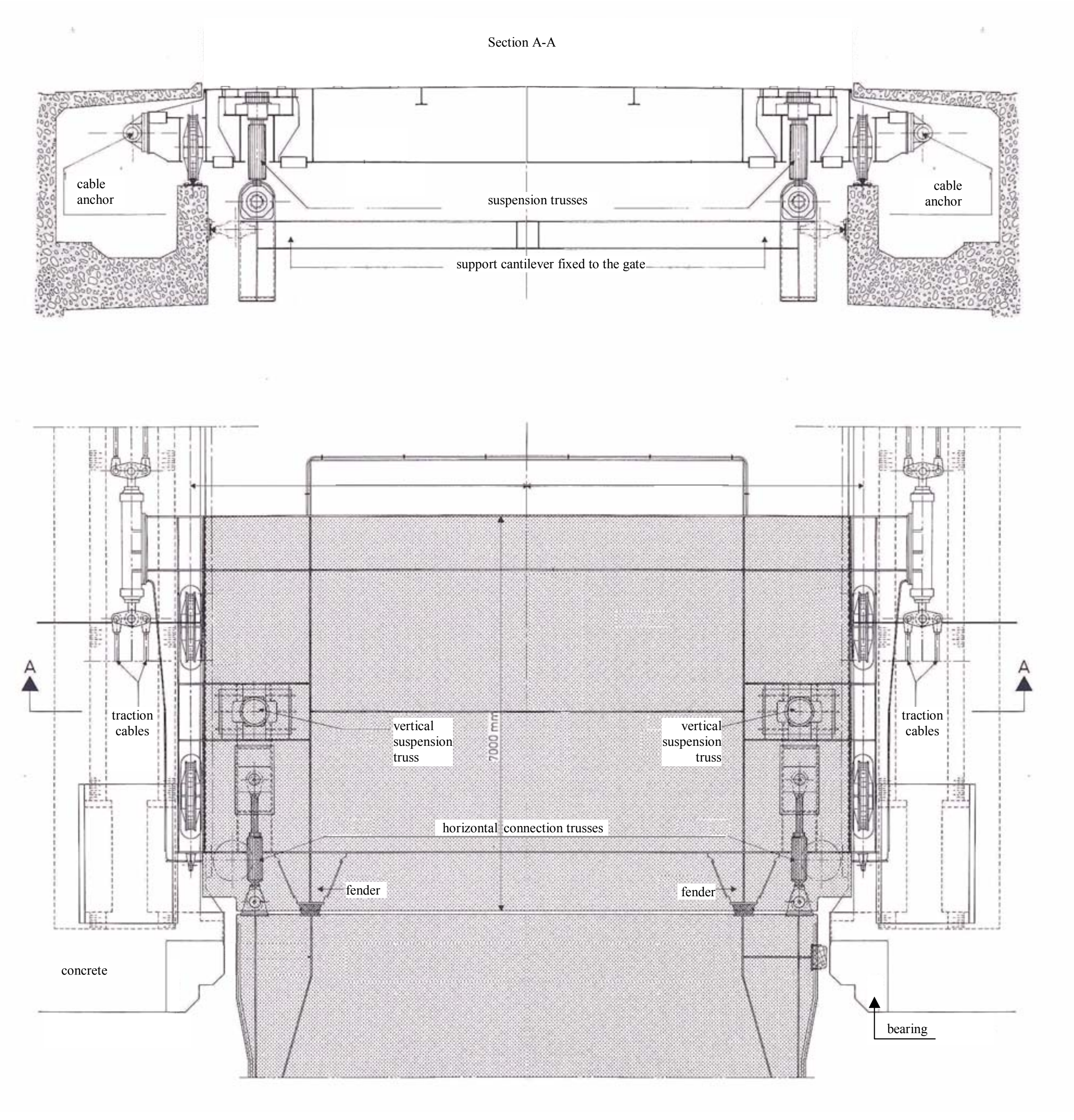
3.4 LOWER SUPPORT WAGON DOWNSTREAM GATES AT1



Lower support wagon

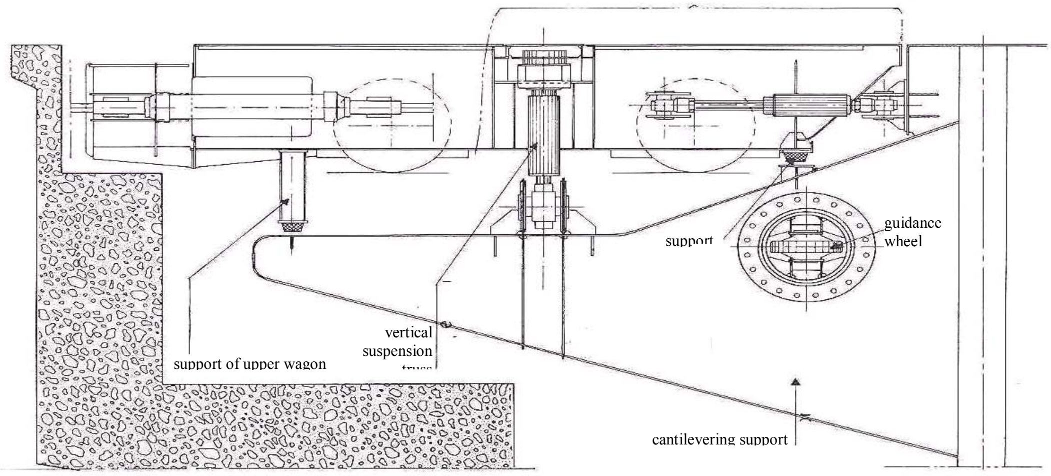
3.5 UPPER SUPPORT WAGON

From the assessment of the maximum vertical reaction forces over the range of operating conditions (see 5.2) it is concluded that a design similar to the upper support wagon of the Berendrecht lock is appropriate. A more extended framework design of I-shaped beams is proposed, similar to the recently designed new rolling gates for the Van Cauwelaert lock (port of Antwerp), as shown on the attached drawings. The flanges of the beams directly connect to a pair of longitudinal stiffeners of the skin plating of the gates. A lateral guidance wheel is built in at each side of the cantilevering supports.



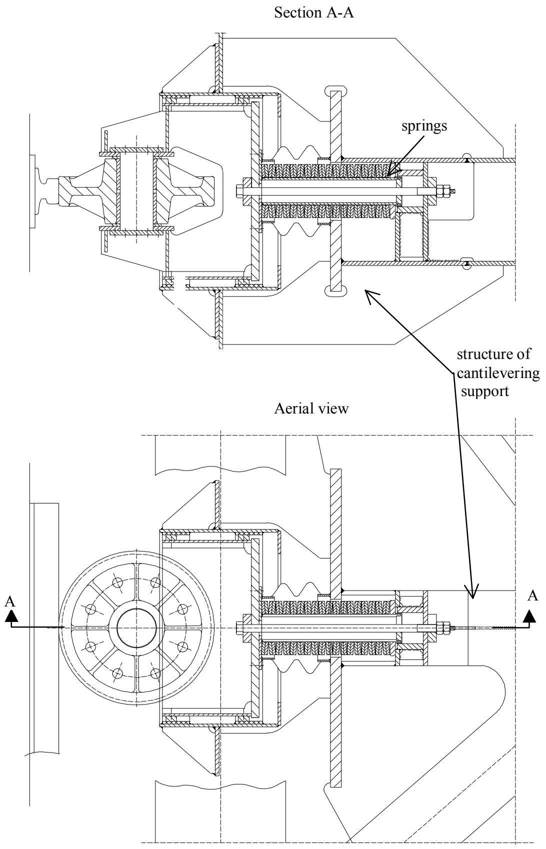
Upper support wagon

3.5.1 General view

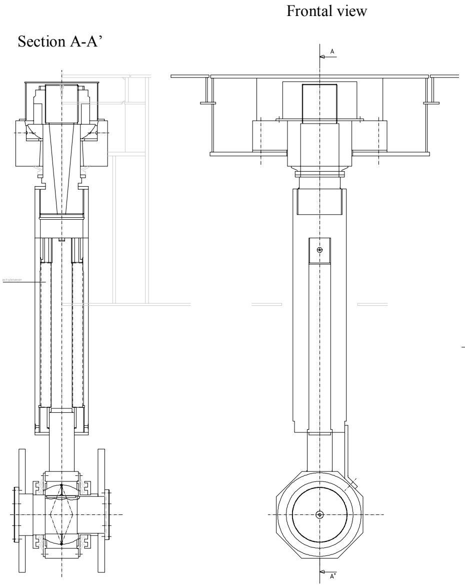


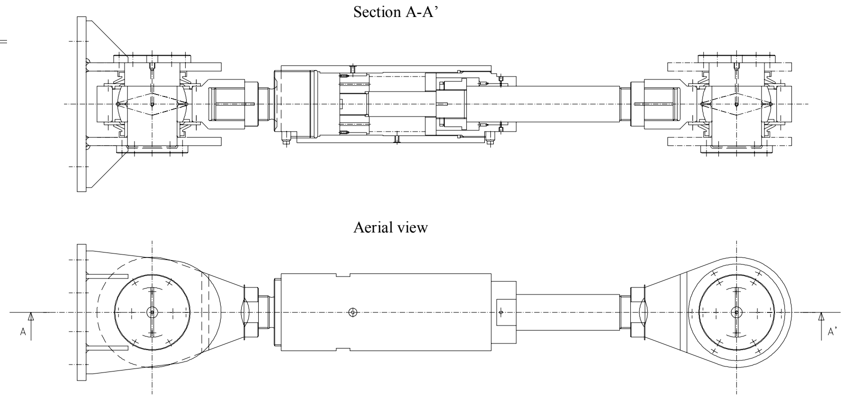
Lateral view of upper support wagon with cantilevering support

3.5.2 Details



Guidance wheel



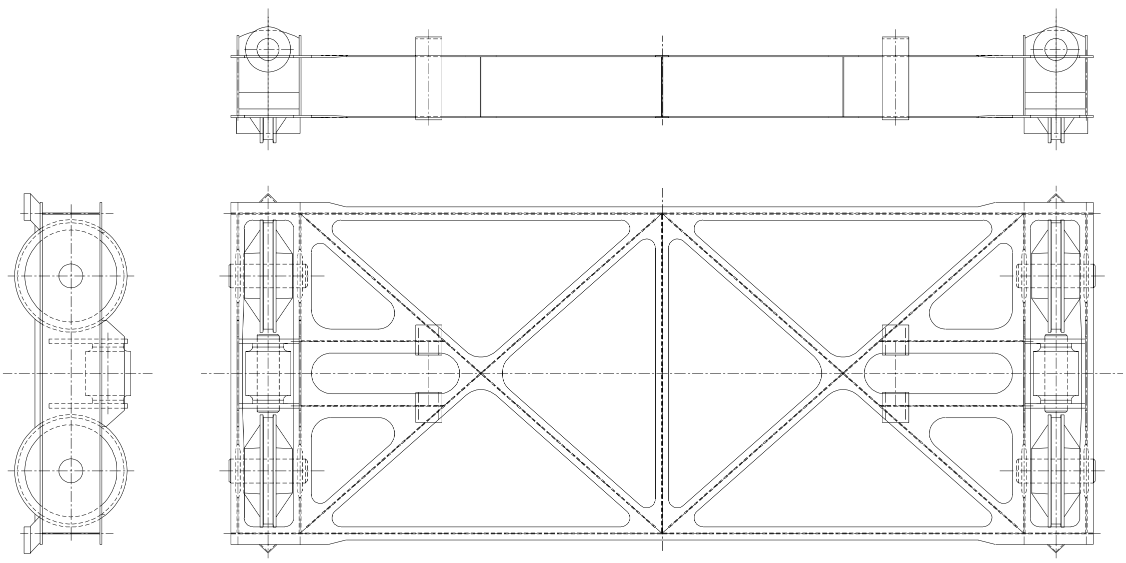


Horizontal connection truss

Vertical suspension truss

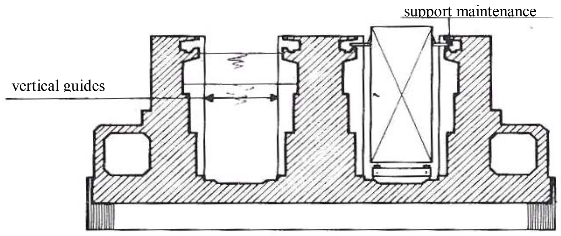
3.6 LOWER SUPPORT WAGON INTERMEDIATE AND DOWNSTREAM GATES

From the assessment of the maximum vertical reaction forces over the range of operating conditions (see 5.2) it is concluded that for the lower support wagons a design similar to the rolling gates of the Berendrecht lock (port of Antwerp) is appropriate. The necessary wheel bearing capacity does not exceed 1200 kN (see section 5.2), which also holds for the recently designed new rolling gates for the Van Cauwelaert lock (port of Antwerp).

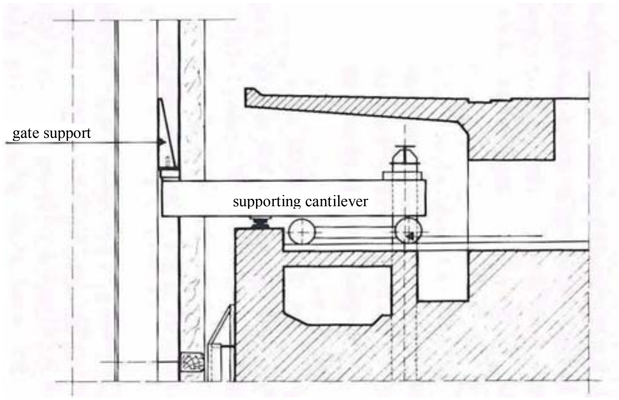


Lower support wagon AT2, AT3, AT4

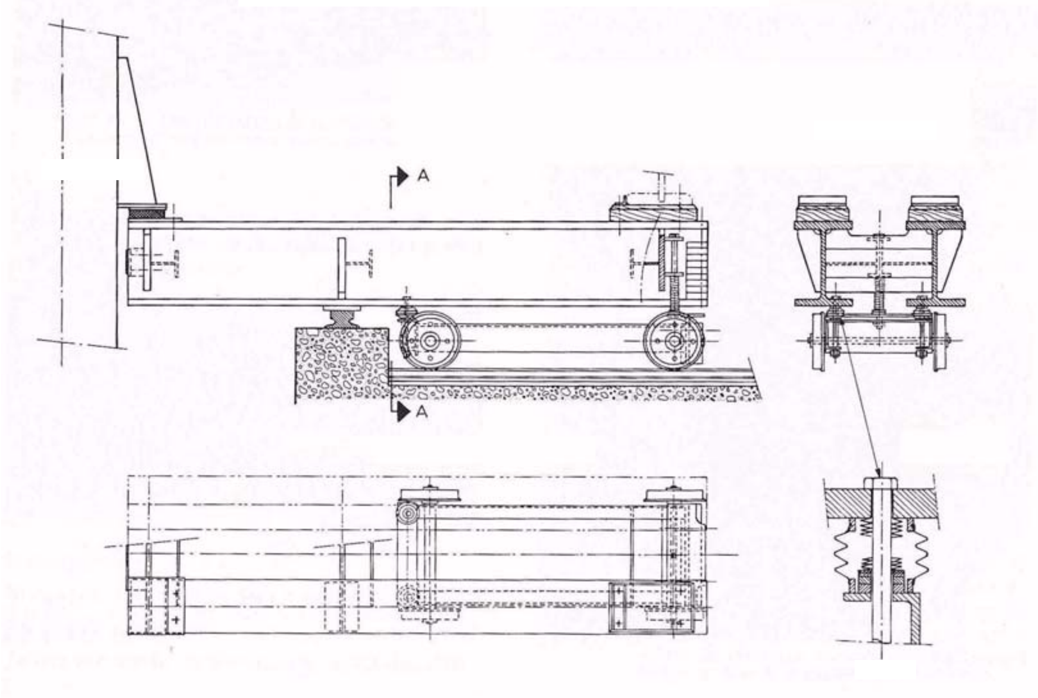
3.7 SUPPORT MAINTENANCE WAGON



Gate in suspended position

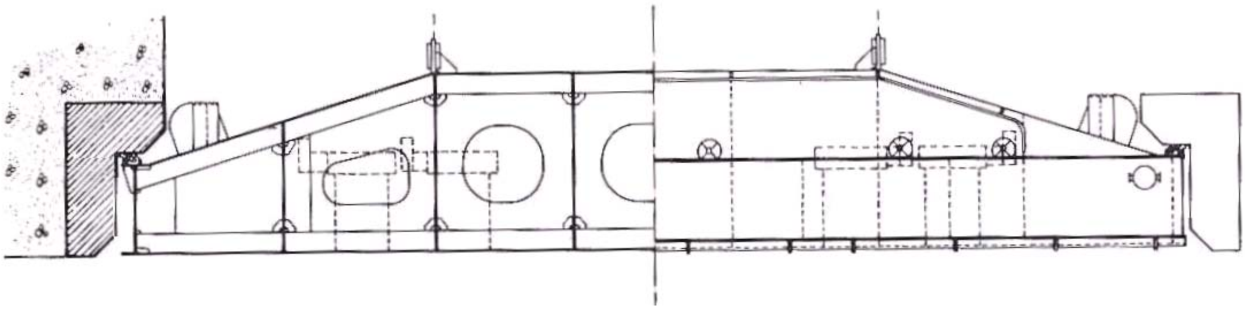


Support maintenance wagon : position in lock structure



3.8 SLOT BULKHEAD

The width of the proposed rolling gate structures is comparable to the width of the Berendrecht lock gates. Consequently a design similar to the slot bulkhead of the Berendrecht lock is appropriate (see illustration).



Slot bulkhead

3.9 NOTE ON LOCK DEWATERING PROCEDURE

At the *upstream* lock head the upstream side will be equipped with an additional shaping (among others including an additional sill) to provide an additional bearing. A gate AT1 is maneuvered from its gate recess and relocated to constitute a water retaining closure plate against this additional bearing.

At the *downstream* lock head the downstream side will be equipped with an additional shaping (among others including an additional sill) to provide an "inverse" bearing. Against this inverse bearing preferably an appropriate AT1-like gate is maneuvered (although in principle a gate AT4 might be used for this purpose as well) to constitute a water retaining closure plate. Gates AT4 have on only one side (downstream) seals, which have to be positioned relatively close to the bearings. On the other (upstream) side sufficient spacing between the gate and the wall of its gate recess will be provided for maneuvering such that the cumbersome dismounting and remounting of fender beams (at the bottom) is avoided. Lateral guidance of the operating gates AT2-3 and AT4 at the upstream side will be performed by appropriate wheel fenders, which are fixed to the (upstream) wall of the gate recess. These fenders may be dismounted and remounted relatively easy to provide the necessary lateral space for the gate maneuvers. Gates AT1 have lateral guidance wheels built in their cantilevering supports at both the upstream and the downstream side (see 3.5).

Working at the rails of the lower support wagon of all gates AT1, AT2, AT3 and AT4 will be performed inside a habitat tunnel, which is accessed through the slot bulkhead (section 3.8) from the dewatered corresponding gate recess. Thus it is not necessary to achieve (dry) lock chamber conditions for these maintenance operations and maneuvering of the gates from and back to their gate recesses is avoided.

4 SAMPLE COMPUTATIONS FOR GATE AT3

4.1 GENERAL

In this section sample computations are presented for the case of gate AT3 which is proposed to be identical to gates AT2 and to the corresponding gates PA2 and PA3 of the Pacific locks as well (see 1.3). The applied formulae and the corresponding results will be demonstrated as adopted for the concept design of the proposed rolling lock gates. In appendix A to this report the main results are listed for the (loading) case of gate PA3, which is determinative for gate AT3 (and AT2).
It should be noted that more thorough procedures can be followed in an advanced level of design and optimization. Some results of preliminary 3D-FEM computations are discussed as well. They are listed in more detail in appendix B.

4.2 LIST OF SYMBOLS

The following symbols will be used (in order of appearance):

H: "width" of the gates, i.e.: distance between outsides of plating (m)
L: spacing between two adjacent vertical structural frames (m)
B: height of a plating panel between two adjacent horizontal structural frames (m)
$e_{(eq)}$: (equivalent) thickness of a plating panel (m)
$p_{w.c}$: characteristic value of the water pressure on a plating panel (N/m²)
n: number of longitudinal stiffeners over the height B (-)
n_e: number of effective stiffeners against buckling (-)
l: spacing between longitudinal stiffeners (m)
b: spacing between effective stiffeners against buckling (m)
b_e: effective width, allowing for shear lag effect (-)
A: cross section area (m²)
I: (geometric) moment of inertia (m^4)
W: bending module (I/v) (m³)
R: subscript referring to (longitudinal) stiffener, in combination with effective plating strip (-)
α: shape factor of plating panel = L/B or L/b or B/L (-)
β: relative width of a plating panel = b/e (-)
δ: relative cross section = A/(B.e) (-)
γ^*: critical relative stiffening factor against buckling (-)
m = 4 for open sections of stiffeners; = 2.5 for closed (e.g.: omega) stiffener profiles
γ^{**} = m . γ^*
υ: Poisson's coefficient = 0.3 for structural steel; $12.(1-\nu^2) = 10.92$
σ: (subscript referring to) normal stress (both tension, compression) (N/m²)
τ: (subscript referring to) shear stress (N/m²)

x: (subscript referring to) longitudinal spatial co-ordinate (m)
y: (subscript referring to) transversal spatial co-ordinate (m)
ψ: ratio between minimum and maximum value of longitudinal normal stress at plate edges (-)
ρ: ratio between corrected values of shear stress and longitudinal normal stress (-)
c: correction coefficient (inverse of safety factor) (-)
c*: maximum critical value of correction coefficient c (-)
w: subscript referring to the web of an I-shaped beam (corresponding to a frame) (-)
ac: subscript referring to parts belonging to the air chamber (-)

4.3 2D-BEAM GRID MODEL

Both vertical and horizontal structural frames are modeled as I-shaped beams. The skin plating is assigned as flanges to the beams to which it is nearest. The longitudinal compression members of the upstream skin framework of the gates (instead of the plating) also constitute a flange for their corresponding beam. For the webs of the beams corresponding to the intermediate horizontal frames, an assumption is made on the equivalent plate thickness of the horizontal transversal connections.

An overall cross section analysis yields an estimate of the torsion stiffness distribution over the horizontal frames, taking in account their contingence.
Finally the stiffness of the flexible plates at the bottom is tuned to avoid uplift of the bottom corner edges of the skin plating of the gates.

With these geometric characteristics of each beam a 2D-beam grid of the gate structure (integrated over the width) is drawn up.
Doing so the distribution of the section forces and displacements is derived with the (in Europe widely used) ESA-primaWin structural engineering software. Subsequently the material tension levels at the extremities (flanges) of the cross section are computed, in particular for the design of the compressed parts against buckling.

An output document generated by the applied structural engineering software is attached as Appendix A, showing input and output (reaction forces (kN), section forces (kN, kNm), material stresses, deformations (mm)) for the 2D-beam grid model.

Note that the listed reaction forces shown at page 8 can be found as follows:

$$R_{normal} = 1000\ \frac{kg}{m^3}\cdot 10\,\frac{m}{s^2}\cdot\frac{(20.62\ m)^2}{2}\cdot 57.24\ m + 1000\ \frac{kg}{m^3}\cdot 10\,\frac{m}{s^2}\cdot 20.62\ m\cdot 57.24\ m\cdot 8.50\,m = 222000\ kN$$

$$R_{ultimate} = 1000\ \frac{kg}{m^3}\cdot 10\,\frac{m}{s^2}\cdot\frac{(22.89\ m)^2}{2}\cdot 57.24\ m + 1000\ \frac{kg}{m^3}\cdot 10\,\frac{m}{s^2}\cdot 22.89\ m\cdot 57.24\ m\cdot 7.38\,m = 246600\ kN$$

The resulting material stresses are shown for each frame in appendix A page 10-12 (load case “ULS”: self weight + dewatered chamber + MCE). The corresponding results for the normal service load case “SLS” are not shown.

4.4 DOWNSTREAM AIR CHAMBER PLATING

The plate thickness is chosen corresponding to the tension level derived from the 2D-beam grid model. The longitudinal stiffeners are chosen according to the procedure described in section 2.2. For the lowest zone of the downstream air chamber plating (panel R4-R5) of gate AT3 holds:

L = 3.18 m
B = 6.50 m
e = 16 mm
$p_{w.c}$ = 200 kN/m² (max. blow-out pressure) + 35.8 kN/m² (MCE, Westergaard) = 236 kN/m²

yielding:

n = 8 longitudinal stiffeners type 1/2IPE400
l = 0.722 m (= B/9)
b_e = 42.8 cm
I = 8072 cm^4
W = 544.8 cm³ (at flange edge of stiffener)
W = 1190 cm³ (at outside plating edge)
e_{eq} = 1.6 cm + 8 x 42.23 cm² / 650 cm = 2.12 cm

For these characteristics the relations (2.1), (2.2a) and (2.2b) of section 2.2 hold with σ_{max} = ca. 0.75 * 355/1.1 = 242 N/mm², leaving sufficient resistance for the characteristic global stress of 194 N/mm² (see appendix A p. 10). In the accidental load case (incl. MCE) considered, there is no need to arrive at a sufficiently moderate total stress to account for fatigue.

A similar procedure is followed for the other plating panels, yielding e_{eq}-values that are used to derive characteristics for (the flanges of) the I-shaped members of the 2D-beam grid model (see below).

Gate AT-PA 2-3 : Plating panel characteristics					
Double skin plating	downstream side (tension in base case)				
Panel	B	e	profile	n	e_{eq}
R1_top-R3_intermed	614.5	1.8	1/2 IPE400	7	2,28
R3_intermed-R4_top ac	614.5	1.8	1/2 IPE400	7	2,28
R4_top ac-R5_bottom_ac	650	1.6	1/2 IPE400	8	2,12
R5_bottom_ac-R6_intermed	510.5	1.6	1/2 IPE400	8	2,26
R6_intermed-R7_bottom	510.5	1.6	1/2 IPE400	8	2,26
Flexible plate	127	3.5	none	0	3,50
	upstream side (compression in base case)				
R4_top ac-R5_bottom_ac	650	2	1/2 IPE750-147	8	3,15

4.5 UPSTREAM AIR CHAMBER PLATING (COMPRESSED)

In order to obtain an appropriately stiffened plating panel to safely resist the longitudinal compression loading, the longitudinal stiffeners are chosen such that the buckling risk of stiffeners doesn't exceed the buckling risk of the separate plating strips between the stiffeners. This is obtained by choosing I_R above a critical value as follows:

$$I_R \geq \gamma^{**} \frac{Be^3}{12\,(1-\upsilon^2)} \tag{4.1}$$

where $$\gamma^{**} = m \cdot \mathrm{MAX}\left[\gamma^*_{\sigma_x} ; \gamma^*_{\sigma_y} ; \gamma^*_{\tau}\right] \tag{4.2}$$

The critical relative stiffening factors γ* for each simple loading case mainly depend on the shape parameter α of each panel. When the number of stiffeners increases, the shear force case is determinative. For three cases a closed formula is published in the technical notes of CECM (n_e = 1, 2 and infinite respectively):

$$\gamma^*_{\tau(1)} = \frac{5.4}{\alpha}\left(\frac{2}{\alpha} + \frac{2.5}{\alpha^2} - \frac{1}{\alpha^3} - 1\right) \tag{4.3.1}$$

$$\gamma^*_{\tau(2)} = \frac{12.1}{\alpha}\left(\frac{4.4}{\alpha} - 1\right) \tag{4.3.2}$$

$$\gamma^*_{\tau(\infty)} = \frac{28}{\alpha} - 20\,\alpha \tag{4.3.3}$$

For other n_e-values (> 2) an interpolation formula is used.

For the upper half upstream air chamber plating (panel R4-R5) of gate AT3, considering e = 20 mm (unlike the downstream panel, see 4.4) and n_e = 8 effective stiffeners against buckling, holds:

b = B/9 = 0.722 m
$\gamma^*_{\tau(1)}$ = 26.42, $\gamma^*_{\tau(2)}$ = 20.13, $\gamma^*_{\tau(\infty)}$ = 58.09
$\gamma^*_{\tau(7)}$ = 29.23
γ^{**} = 116.94
$I_{R\text{-}min}$ = 55684 cm^4

Choosing profiles 1/2IPE750-147 yields (with b_e = b and n = 8 stiffeners):

I_R = 58022 cm^4
e_{eq} = 2.0 cm + 8 x 93.75 cm² / 650 cm = 3.15 cm

Consequently the compressed plating panel safely resists buckling if the separate panels between the (effective) stiffeners have a sufficient safety against buckling. This is evaluated as follows.

The critical resistances against buckling for simple loading cases are respectively:

$$\sigma^o_{x,cr} = k_{\sigma_x}\,\sigma_E \tag{4.4.1}$$

$$\sigma^o_{y,cr} = k_{\sigma_y}\,\sigma_E \tag{4.4.2}$$

$$\tau^o_{cr} = k_\tau\,\sigma_E \tag{4.4.3}$$

wherein the Euler stress is defined as: $\sigma_E = \dfrac{\pi^2\, E}{12\,\beta^2\,(1-\nu^2)} = \dfrac{189800 \cdot {}^{N}\!/\!{}_{mm^2}}{\beta^2}$ (4.5.0)

with β = b/e the relative width of the plating panel

The k-factors follow from linear elastic buckling theory.
For $\psi = 1$ we have the following formulas:

$$k_{\sigma_x} = \left(\mathrm{MIN}[\alpha;1] + \frac{1}{\mathrm{MIN}[\alpha;1]} \right)^2 \quad (4.5.1)$$

$$k_{\sigma_y} = \frac{\left(\mathrm{MIN}[\alpha;1] + \frac{1}{\mathrm{MIN}[\alpha;1]} \right)^2}{\alpha^2} \quad (4.5.2)$$

$$k_\tau = \frac{5.34}{(\mathrm{MIN}[\alpha;1])^2} + \frac{4}{(\mathrm{MAX}[\alpha;1])^2} \quad (4.5.3)$$

Assuming proportionality between the critical resistances for the combined loading case and the actual stresses σ_x, σ_y and τ, the non-reduced ("ideal") combined critical comparison stress level is given by:

$$\sigma_{c,cr}^{id} = \frac{\sigma_c}{\frac{4-\rho}{4} \cdot \frac{\sigma_y}{\sigma_{y,cr}^o} + \sqrt{\left[\frac{\rho}{4}\,\frac{\sigma_y}{\sigma_{y,cr}^o}\right]^2 + \left[\frac{\sigma_x}{\sigma_{x,cr}^o}\right]^2}} \quad \text{for } \rho \le 1 \quad (4.6.1)$$

$$\sigma_{c,cr}^{id} = \frac{\sigma_c}{\frac{1+2\rho}{4\rho} \cdot \frac{\sigma_y}{\sigma_{y,cr}^o} + \sqrt{\left[\frac{2\rho-1}{4\rho}\,\frac{\sigma_y}{\sigma_{y,cr}^o}\right]^2 + \left[\frac{\tau}{\tau_{cr}^o}\right]^2}} \quad \text{for } \rho \ge 1 \quad (4.6.2)$$

wherein $\rho = \dfrac{\tau \cdot k_{\sigma x}}{\sigma_x \cdot k_\tau}$ (4.7)

which is reduced if $\sigma_{c,cr}^{id} > 0.8 \cdot f_y$ as follows:

$$\sigma_{c,cr}^{red} = y \cdot f_y \quad \text{with } y = \frac{0.8 + 0.2 \cdot \sqrt{1 - \frac{0.6}{x^2}}}{1 + \frac{0.04}{x^2}} \quad \text{and} \quad x = \frac{\sigma_{c,cr}^{id}}{f_y} > 0.8 \quad (4.8.1)$$

otherwise:

$$\sigma_{c,cr}^{red} = \sigma_{c,cr}^{id} \tag{4.8.2}$$

Finally the actual Von Mises comparison stress σ_c in the compressed plating panel

$$\sigma_c = \sqrt{\sigma_x{}^2 + \sigma_y{}^2 - \sigma_x\sigma_y + 3\,\tau^2} \tag{4.9}$$

is compared with the reduced combined critical comparison stress through a correction factor c defined as:

$$c = \frac{\sigma_c}{\sigma_{c,cr}^{red}} \tag{4.10}$$

Safety against buckling of the plating is guaranteed if this value does not exceed the critical limit value c* = 0.83 which holds for the determinative zone in the middle of the span (case with relatively small shear stress and almost pure compression ψ = 1). In general c* equals a weighted average of the squares of the simple load case critical values:

$$c^* = \sqrt{\frac{\left[\dfrac{\sigma_x \cdot c_x^*}{\sigma_{x,cr}^o}\right]^2 + \left[\dfrac{\sigma_y \cdot c_y^*}{\sigma_{y,cr}^o}\right]^2 + \left[\dfrac{\tau \cdot c_\tau^*}{\tau_{cr}^o}\right]^2}{\left[\dfrac{\sigma_x}{\sigma_{x,cr}^o}\right]^2 + \left[\dfrac{\sigma_y}{\sigma_{y,cr}^o}\right]^2 + \left[\dfrac{\tau}{\tau_{cr}^o}\right]^2}} \tag{4.11}$$

wherein c^*_x ranges from 0.83 (pure compression) to 1.05 (pure bending), c^*_y = 0.83 and c^*_τ = 1.05

The 2D-beam grid model yields the actual design values σ_x = 194 N/mm², σ_y = 52 N/mm² and τ = 8 N/mm² (see appendix A p. 10-12) in the middle of the span of the air chamber. The critical resistance values for the simple load cases are respectively: 582.2 N/mm², 161.0 N/mm² and 807.3 N/mm² based on the respective k-factors: 4.00, 1.11 and 5.55.

Using the equations above yields c = 0.67 < 0.83 = c* showing sufficient safety against plate buckling.

4.6 CONCLUSION: CHARACTERISTICS OF I-SHAPED BEAMS FOR 2D-BEAM GRID MODEL

Following the procedure described above, the characteristics of the I-shaped beams corresponding to the horizontal frames are obtained as listed in the table below (all units in cm).

Gate AT-PA 2-3 : Horizontal frame characteristics									
Hor. Frame	A_{comp}	$A_{tension}$	$A_{flanges}$	A_{web}	I_t	I_y	I_z	Wel_y	Wel_z
R1 top	881	701	1582	1489	1,71E+09	5,13E+08	5,51E+06	969.553	35.890
R3	1762	1402	3164	1000	1,05E+09	8,61E+08	4,41E+07	1.585.884	143.558
R4-top ac	1906	1390	3296	1987	9,40E+08	9,70E+08	4,63E+07	1.768.586	146.448
R5-bottom ac	1796	1266	3063	2387	9,00E+08	9,45E+08	3,55E+07	1.722.824	122.456
R6	1543	1155	2698	1000	9,00E+08	7,43E+08	2,51E+07	1.345.015	98.241
R7 bottom	975	1022	1996	1000	1,48E+09	5,78E+08	1,24E+07	1.138.911	65.098

The 2-D beam grid model with these characteristics was used to derive the material stresses used in the computations illustrated above. This shows that convergence has been reached. Thus it may be concluded that the proposed structure fulfils the stability rules.

4.7 NOTE ON ACCURACY AND MORE DETAILED LEVEL OF DESIGN

The above mentioned results based on the design procedure described in the methodology of the earlier CPP concept design study eventually should be subject to a verification analysis by means of a full 3D Finite Element modeling including plates and frameworks. Thus the spatial contingence of the frames can be assessed more accurately and the design of the beams of the vertical and horizontal frameworks can be optimized.
Based on our experience with such 3D-modeling verification exercises, e.g. with the recently designed new rolling gates for the Van Cauwelaert lock (port of Antwerp), we can state that the accuracy of the above described procedure certainly allows a cost estimate within the limit imposed by the terms of reference.
For the gate AT3 structure (proposed to be identical to the one of gates AT2, PA2 and PA3, see 1.3) a preliminary 3D-FEM analysis was carried out, indicating that the model assumptions adopted for this concept design are sufficiently accurate as stated. More details on the model set up and some computational results are listed in appendix B. Note that only a HALF gate is represented, taking in account the symmetry in structural geometry, loads and boundary conditions.

5 REACTION FORCES

5.1 GENERAL

Proceeding according to the methodology of the earlier CPP concept design study, as illustrated in section 4 of this report, the distribution of horizontal reaction forces on the water retaining gates is determined. Horizontal reaction forces occur both at the extremities of the gates (vertical lateral bearings on the lock walls) and at the bearing at the sill. Vertical forces occur at the upper and lower support wagons during opening and closing of the gates.

5.2 VERTICAL SUPPORTS - ROLLING WAGONS

Maximum vertical forces at the support wagons will occur during operations (closing, opening) of the gates. It is assumed that the wheels of both the support wagons should remain pressed with a minimum vertical reaction force (ca. 600 kN) on the rails to guarantee a proper functioning of the connections in operational circumstances. The vertical reaction forces increase when the water level decreases, causing the structure to be less submerged and thus its weight to be less relieved.

The rolling wagons are not designed for the vertical reaction forces occurring at water levels lower than the minimum values at opening/closing of the gates listed in section 1.1 (e.g. in the event of dewatering the gate recess or even the entire lock chamber). In such cases the lower support wagons will be removed (by pulling them from underneath the floating gates) and subsequently the lock gates will be placed on their fixed bottom supports (see 3.3).
The table below shows the total vertical reaction forces on the support wagons at the lowest operating water level of the gates in normal service circumstances (symmetrical distribution of the total vertical reaction force over the wheels of the wagons). In the water retaining cases the listed vertical reactions are to be modified according to the friction forces acting at the lateral bearings.

Total vertical reaction forces in normal service circumstances

Gate AT1	Highest water level (water retaining = open/close)	Lowest water level (open/close)	Lowest water level (operations)
Upper wagon	600 kN	750 kN	800 kN
Lower wagon	600 kN	900 kN	1000 kN

Gate AT2-3	Highest water level (water retaining)	Lowest water level (open/close)
Upper wagon	600 kN	1000 kN
Lower wagon	600 kN	1400 kN

Gate AT4	Highest water level (water retaining)	Lowest water level (open/close)
Upper wagon	600 kN	1000 kN
Lower wagon	600 kN	1400 kN

The support wagons are designed taking in account that three watertight zones (of each two compartments of the air chamber) may be flooded accidentally at the same time, causing an additional possible vertical loading of the support wagons. This additional loading depends on the position of the flooded zone (to be chosen appropriately for each wagon) and the size of the compartment.

Characteristic values of the maximum vertical reactions (at lowest operating water levels) are listed in the table below. The total vertical reaction on the lower support wagon always exceeds the one on the upper support wagon. For all gates AT1, AT2-3 and AT4 the total vertical reaction on the support wagons amounts ca. 10 to 15% of the total weight of the corresponding gate structure.

Gate AT1	Side of flooded zone	Other side	Total vert. reaction
Upper wagon	1200 kN	500 kN	1700 kN
Lower wagon	1500 kN	900 kN	2400 kN

Gate AT2-3	Side of flooded zone	Other side	Total vert. reaction
Upper wagon	1800 kN	700 kN	2500 kN
Lower wagon	2400 kN	1400 kN	3800 kN

Gate AT4	Side of flooded zone	Other side	Total vert. reaction
Upper wagon	1700 kN	700 kN	2400 kN
Lower wagon	2100 kN	1100 kN	3200 kN

Maximum vertical reaction forces with flooded compartments

The maximum values found for gates AT1 are similar to the corresponding ones for gates of the Berendrecht lock. The values for gates AT2-3 and AT4 are similar to the values for the corresponding rolling wagons of the recently designed new rolling gates of the Van Cauwelaert lock (port of Antwerp). Note that for the proposed four-wheel rolling wagons (two wheels at each side of a rolling wagon), each wheel needs a bearing capacity not exceeding 1200 kN. Identical corresponding rolling wagons are proposed for gates AT2-3 and AT4. The listed total vertical reactions at the upper wagon are determinative for the design of the cantilevering support of each gate (see section 3.5).

5.3 HORIZONTAL FORCES AT THE SILL BEARING

Maximum horizontal reaction forces at the sill bearing occur during retaining of the (maximum) water level difference. They result directly from the 2D-beam grid model computations presented in appendix A after an optimization of the stiffness of the cantilevering bottom ends of the vertical frames. Doing so uplift from the sill bearing of the skin plating at the gate bottom edges is avoided.
To derive the maximum contact stress, the minimum spreading length (along the azobé beams of 500 mm width) is mentioned in the tables below.
.

	Reaction force at frame (3.15 m length)	Max. contact stress (min. 540 mm)
Gate AT1	7500 kN (ULS)	28 N/mm² (ULS)

	Reaction force at frame (3.18 m length)	Max. contact stress (min. 540 mm)
Gate AT2-3	11400 kN (ULS)	42 N/mm² (ULS)

	Reaction force at frame (3.18 m length)	Max. contact stress (min. 540 mm)
Gate AT4	8040 kN (ULS)	30 N/mm² (ULS)

5.4 HORIZONTAL FORCES AT THE LATERAL BEARINGS

Maximum horizontal reaction forces at the lateral bearings (on the lock walls) occur during retaining of the (maximum) water level difference. From the 2D-beam grid model computations presented in appendix A, the reaction forces at the horizontal frames are obtained (corresponding to the same optimized stiffness distribution as mentioned sub 5.3). Subsequently the part of the vertical frame structure at the contact with the lock walls is modeled as a continuously elastically supported beam, which is loaded by these reaction forces. The maximum contact stresses (along azobé beams of 500 mm width) are listed below.

Gate AT1	Max. contact stress (at R1)
	35 N/mm² (*)

Gate AT2-3 and AT4	Max. contact stress at R1	Max. contact stress at R3
	45 N/mm²	52 N/mm² (*)

(*) Based on our experience with 3D-modelling of the recently designed new rolling gates for the Van Cauwelaert lock (port of Antwerp) we expect that the contact stresses at R4 are overestimated at least by 10 N/mm² due to the underestimation of the beam stiffness at the air chamber. Therefore the contact stress at R1 should be considered as determinative, like in the case of Gate AT1.

5.5 CONCLUSION

The above mentioned computational results show that similar layouts of the rolling support wagons and bearings of the rolling gates in operation at the large locks in the port of Antwerp (Belgium) are appropriate for use in the actual gates in the Panama case.
Moreover the pressure levels in the supporting and sealing beams require sufficient high strength materials, like azobé wood or high performance composite materials.

ANNEXES

Appendix A : 2D-beam grid model: Gate AT3 - 10 m 19 frames

Table of contents

Basic data
Type of structure : General XYZ

Number of nodes:	133
Number of members:	222
Number of 1D macros:	25
Number of bound. lines:	270
Number of 2D macros:	1
Number of profiles :	15
Number of cases:	5
Number of materials:	2

Material

Name:		
S 355		
	Ultimate strength	510.000 MPa
	Yield design	355.000 MPa
	E modulus	210000.00 MPa
	Poisson coeff.	0.30
	Density	0.000 kg/mm^3
	Extensibility	1.2e-005 mm/mm.K

Name:		
S 355LG		
	Ultimate strength	510.000 MPa
	Yield design	355.000 MPa
	E modulus	210000.00 MPa
	Poisson coeff.	0.30
	Density	0.000 kg/mm^3
	Extensibility	1.2e-005 mm/mm.K

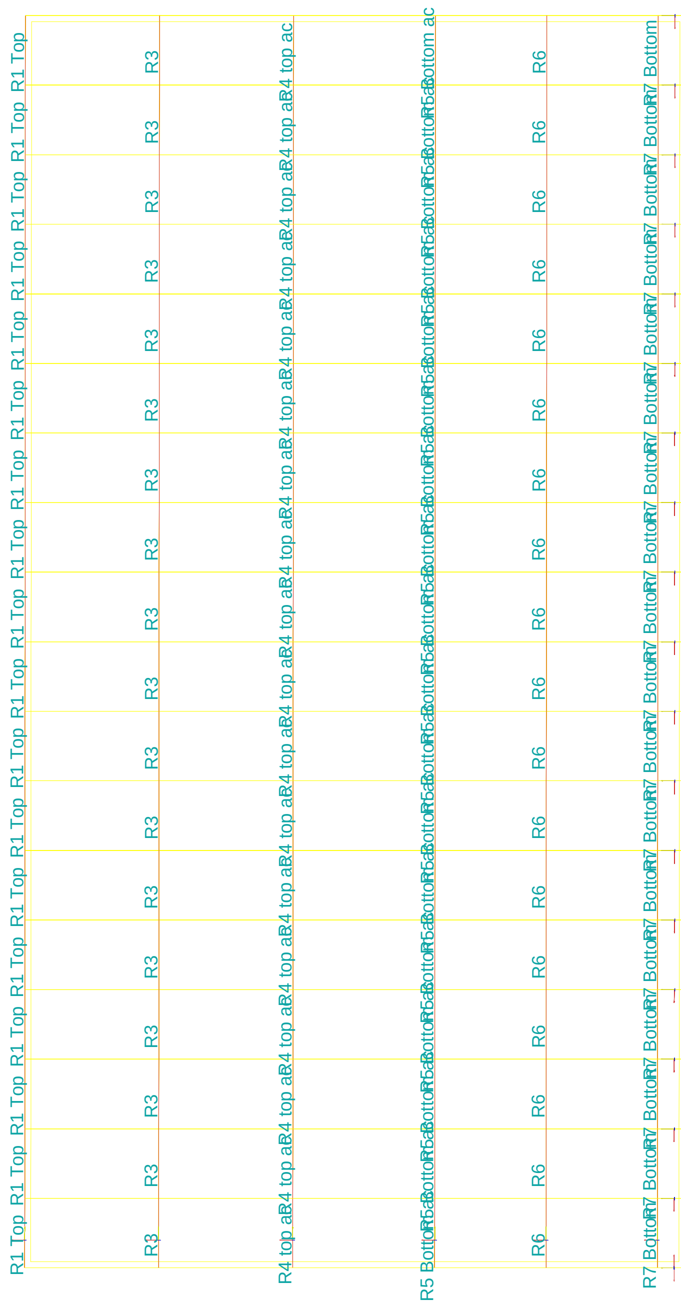
List of material
Group of members :
1/222

no.	Name:	quality	unit weight kg/m	length m	weight kg
1	R1 Top (Numerical)	S 355	2410.73	57.24	137990.47
2	R3 (Numerical)	S 355	3268.74	57.24	187102.67
3	R4 top ac (Numerical)	S 355	4147.15	57.24	237383.15
4	R5 Bottom ac (Numerical)	S 355	4278.25	57.24	244887.04
5	R6 (Numerical)	S 355	2902.93	57.24	166163.71
6	R7 Bottom (Numerical)	S 355	2351.86	57.24	134620.47
8	VF R1-R3 (1000.0,0.3,318.0,1.8)	S 355	1133.32	104.46	118392.29
9	VF R3-R4 (1000.0,0.3,318.0,1.8)	S 355	1133.32	104.46	118392.29
10	VF ac (1000.0,1.6,318.0,1.6)	S 355	2050.80	110.50	226613.06
11	VF R5 R6 (1000.0,0.3,318.0,1.6)	S 355	1033.56	86.78	89697.72
12	VF R6 R7 (1000.0,0.3,318.0,1.6)	S 355	1033.56	86.78	89697.72
13	Bottom cantilever (150.0,5.0,65.0,4.0)	S 355	965.55	24.13	23298.72
14	Urand lijf11 (1000.0,1.1,184.0,1.6,159.0)	S 355	1322.94	45.00	59532.52
15	Urand lijf 25 (1000.0,2.5,184.0,1.6,159.0)	S 355	2418.43	13.00	31439.57

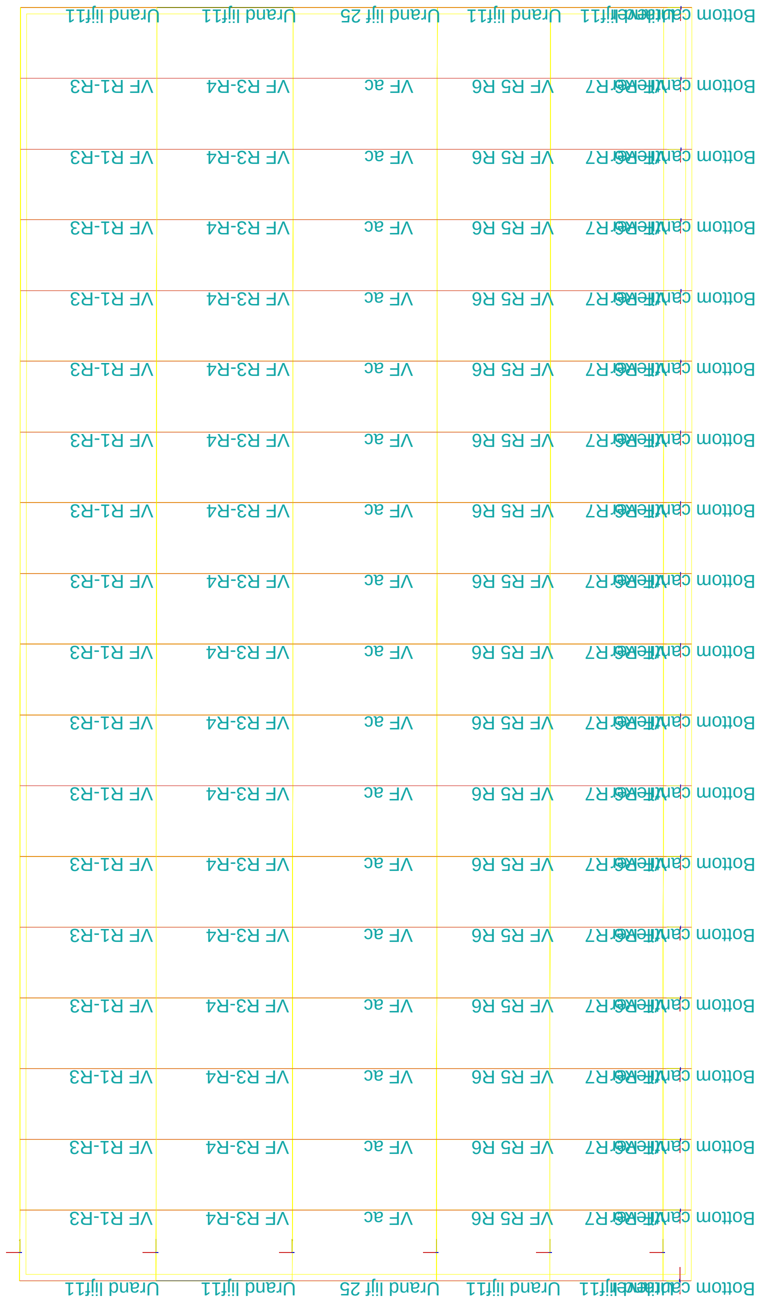
List of material - Macro2D
Group of members :
1/223

no.	Name:	quality	unit volume weight kgm^3	volume m^3	weight kg
300	S 355LG	S 355LG	7.85	86.63	680.07

The total weight of the structure: 1865891.46 kg
Surface for painting: 17862.85 m^2



Horizontal frames



Vertical frames

Supports

support	node	type	flexibility kN/mm-kNmm/rad	Size m
1	1	Y		0.20
2	11	X		0.20
3	19	Y		0.20
4	20	Y		0.20
5	30	X		0.20
6	38	Y		0.20
7	39	Y		0.20
8	49	X		0.20
9	57	Y		0.20
10	58	Y		0.20
11	68	X		0.20
12	76	Y		0.20
13	77	Y		0.20
14	87	X		0.20
15	95	Y		0.20
16	96	Z		0.20
17	97	YZ	ky =510.00	0.20
18	98	YZ	ky =510.00	0.20
19	99	YZ	ky =510.00	0.20
20	100	YZ	ky =510.00	0.20
21	101	YZ	ky =510.00	0.20
22	102	YZ	ky =510.00	0.20
23	103	YZ	ky =510.00	0.20
24	104	YZ	ky =510.00	0.20
25	105	YZ	ky =510.00	0.20
26	106	XYZ	ky =510.00	0.20
27	107	YZ	ky =510.00	0.20
28	108	YZ	ky =510.00	0.20
29	109	YZ	ky =510.00	0.20
30	110	YZ	ky =510.00	0.20
31	111	YZ	ky =510.00	0.20
32	112	YZ	ky =510.00	0.20
33	113	YZ	ky =510.00	0.20
34	114	Z		0.20
35	122	X		0.20
36	132	Y		0.20
37	133	Y		0.20

Loadcases

Case	Name:	Description
1	WR normal +18.30 -2.32 PLD	Variable - LC Water Excl.
2	WR ultimate +19.45 -3.44 PLD	Variable - LC Water Excl.
3	DWC (NOT)	Variable - LC Water Excl.
4	Self Weight	Self weight. Direction -Z
5	EQ : Westergaard	Variable - Earth quake

Loadcase no. 1 - Free loads

Rectangles

Index	Distribution	x m	y m	qx kN/m^2	qy kN/m^2	qz kN/m^2	System	Validity	Location
1	Dir Y	0.00	-2.32	0.00	-206.20	0.00	Global	All	Length
1		57.24	18.30	0.00	0.00	0.00			
2	Uniform	0.00	-2.32	0.00	-206.20	0.00	Global	All	Length
2		57.24	-10.82						

Loadcase no. 2 - Free loads

Rectangles

Index	Distribution	x m	y m	qx kN/m^2	qy kN/m^2	qz kN/m^2	System	Validity	Location
1	Dir Y	0.00	-3.44	0.00	-228.90	0.00	Global	All	Length
1		57.24	19.45	0.00	0.00	0.00			
2	Uniform	0.00	-10.82	0.00	-228.90	0.00	Global	All	Length
2		57.24	-3.44						

Loadcase no. 5 - Free loads

Rectangles

Index	Distribution	x m	y m	qx kN/m^2	qy kN/m^2	qz kN/m^2	System	Validity	Location
1	Dir Y	0.00	19.00	0.00	-10.10	0.00	Global	All	Length
1		57.24	19.30	0.00	0.00	0.00			
2	Dir Y	0.00	18.55	0.00	-16.00	0.00	Global	All	Length
2		57.24	19.00	0.00	-10.10	0.00			
3	Dir Y	0.00	18.10	0.00	-20.20	0.00	Global	All	Length
3		57.24	18.55	0.00	-16.00	0.00			
4	Dir Y	0.00	17.19	0.00	-26.70	0.00	Global	All	Length
4		57.24	18.10	0.00	-20.20	0.00			
5	Dir Y	0.00	15.99	0.00	-33.50	0.00	Global	All	Length
5		57.24	17.19	0.00	-26.70	0.00			
6	Dir Y	0.00	13.23	0.00	-45.30	0.00	Global	All	Length
6		57.24	15.99	0.00	-33.50	0.00			
7	Dir Y	0.00	7.16	0.00	-64.10	0.00	Global	All	Length
7		57.24	13.23	0.00	-45.30	0.00			
8	Dir Y	0.00	0.66	0.00	-79.40	0.00	Global	All	Length
8		57.24	7.16	0.00	-64.10	0.00			
9	Dir Y	0.00	-10.82	0.00	-100.60	0.00	Global	All	Length
9		57.24	0.66	0.00	-79.40	0.00			

Combinations

Combi	Norm	Case	coeff
1.Water	User-ultimate	1 WR normal +18.30 -2.32 PLD	1.35
1.Water	User-ultimate	2 WR ultimate +19.45 -3.44 PLD	1.20
1.Water	User-ultimate	3 DWC (NOT)	0.00
1.Water	User-ultimate	4 Self Weight	1.35
2.Water +EQ	User-ultimate	1 WR normal +18.30 -2.32 PLD	1.00
2.Water +EQ	User-ultimate	2 WR ultimate +19.45 -3.44 PLD	1.00
2.Water +EQ	User-ultimate	3 DWC (NOT)	0.00
2.Water +EQ	User-ultimate	4 Self Weight	1.00
2.Water +EQ	User-ultimate	5 EQ : Westergaard	1.00
3.	User-serviceability	1 WR normal +18.30 -2.32 PLD	1.00
3.	User-serviceability	2 WR ultimate +19.45 -3.44 PLD	1.00
3.	User-serviceability	3 DWC (NOT)	0.00
3.	User-serviceability	4 Self Weight	1.00
3.	User-serviceability	5 EQ : Westergaard	0.00

Basic rules for generation of ultimate load combinations:
1 : 1.35*LC1 / 1.20*LC2 / 0.00*LC3 / 1.35*LC4
2 : 1.00*LC1 / 1.00*LC2 / 0.00*LC3 / 1.00*LC4 / 1.00*LC5

Basic rules for generation of serviceability load combinations:
1 : 1.00*LC1 / 1.00*LC2 / 0.00*LC3 / 1.00*LC4 / 0.00*LC5

List of extreme ultimate load combinations
1/   2 : +1.00*LC4
2/   1 : +1.35*LC4
3/   2 : +1.00*LC4+1.00*LC5
4/   1 : +1.20*LC2+1.35*LC4
5/   1 : +1.35*LC1+1.35*LC4
6/   2 : +1.00*LC1+1.00*LC4+1.00*LC5
7/   2 : +1.00*LC2+1.00*LC4+1.00*LC5

List of extreme serviceability load combinations
1/   1 : +1.00*LC4
2/   1 : +1.00*LC1+1.00*LC4
3/   1 : +1.00*LC2+1.00*LC4

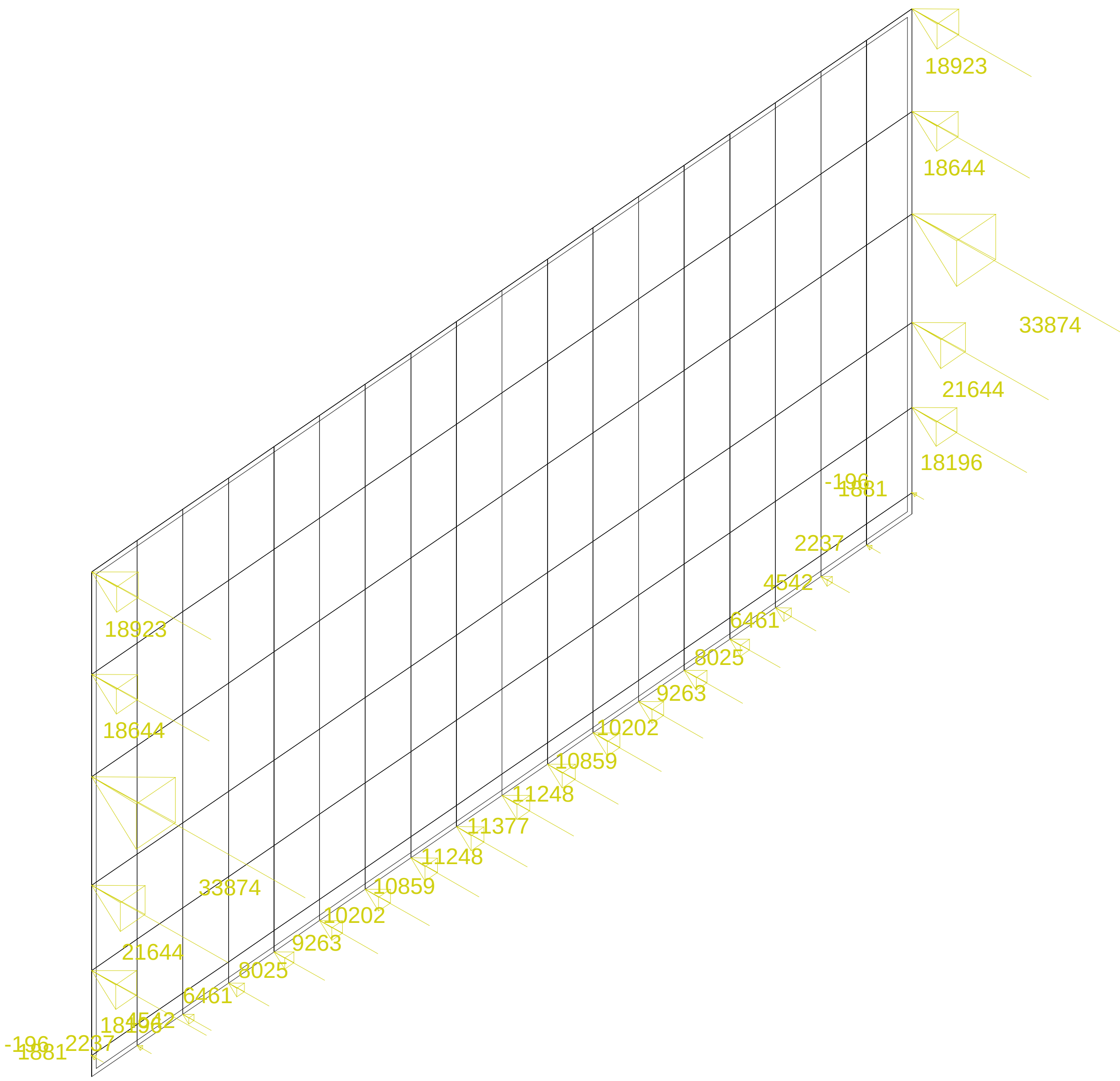
Calculation protocol.

Linear calculation

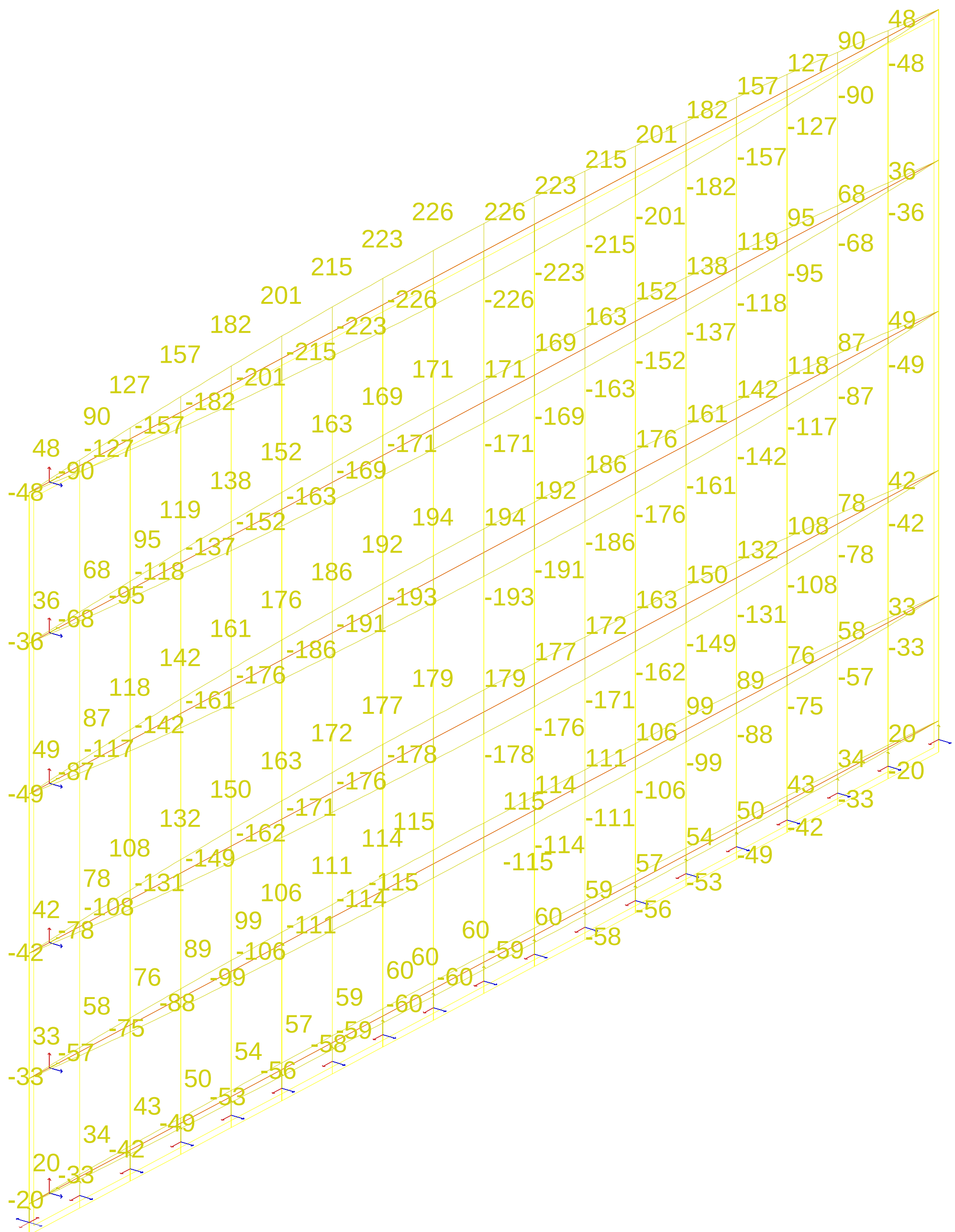
Number of 2D elements Number of 1D elements Number of mesh nodes Number of equations	10800 2289 11020 66120
Loadcases	LC 1 WR normal +18.30 -2.32 PLD
	LC 2 WR ultimate +19.45 -3.44 PLD
	LC 3 DWC (NOT)
	LC 4 Self Weight
	LC 5 EQ : Westergaard
Bending theory	Mindlin
Start of calculation	08.03.2005 17:37
End of calculation	08.03.2005 17:38

Sum of loads and reactions.

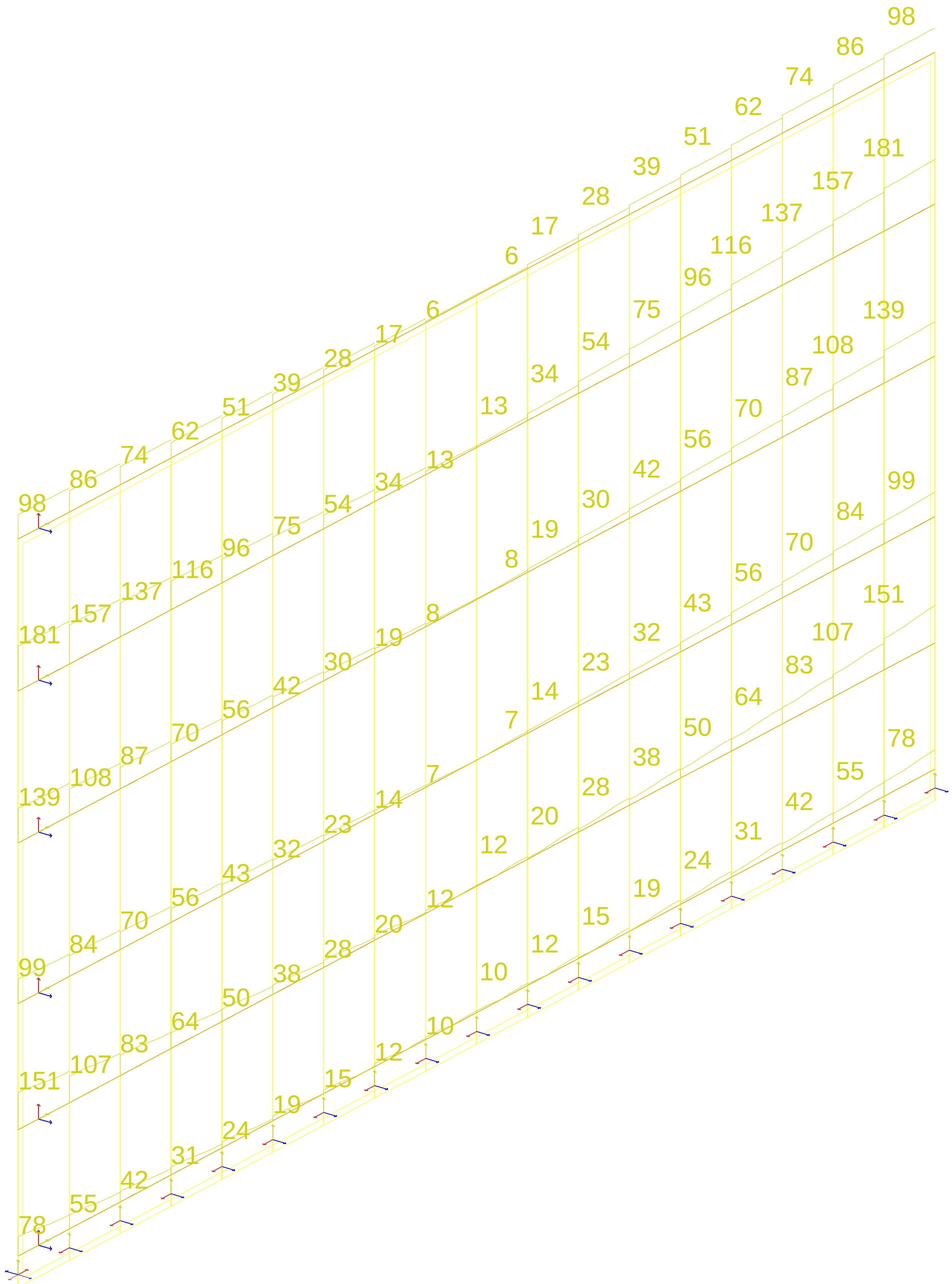
		X	Y	Z
loadcase 1	loads	0.0	-222015.5	0.0
	reactions	-0.0	222015.5	0.0
	contact	0.0	0.0	0.0
loadcase 2	loads	0.0	-246660.5	0.0
	reactions	-0.0	246660.5	0.0
	contact	0.0	0.0	0.0
loadcase 3	loads	0.0	-862.0	0.0
	reactions	-0.0	862.0	0.0
	contact	0.0	0.0	0.0
loadcase 4	loads	-0.0	-0.0	-18658.9
	reactions	0.0	0.0	18658.9
	contact	0.0	0.0	0.0
loadcase 5	loads	0.0	-115224.1	0.0
	reactions	-0.0	115224.1	0.0
	contact	0.0	0.0	0.0



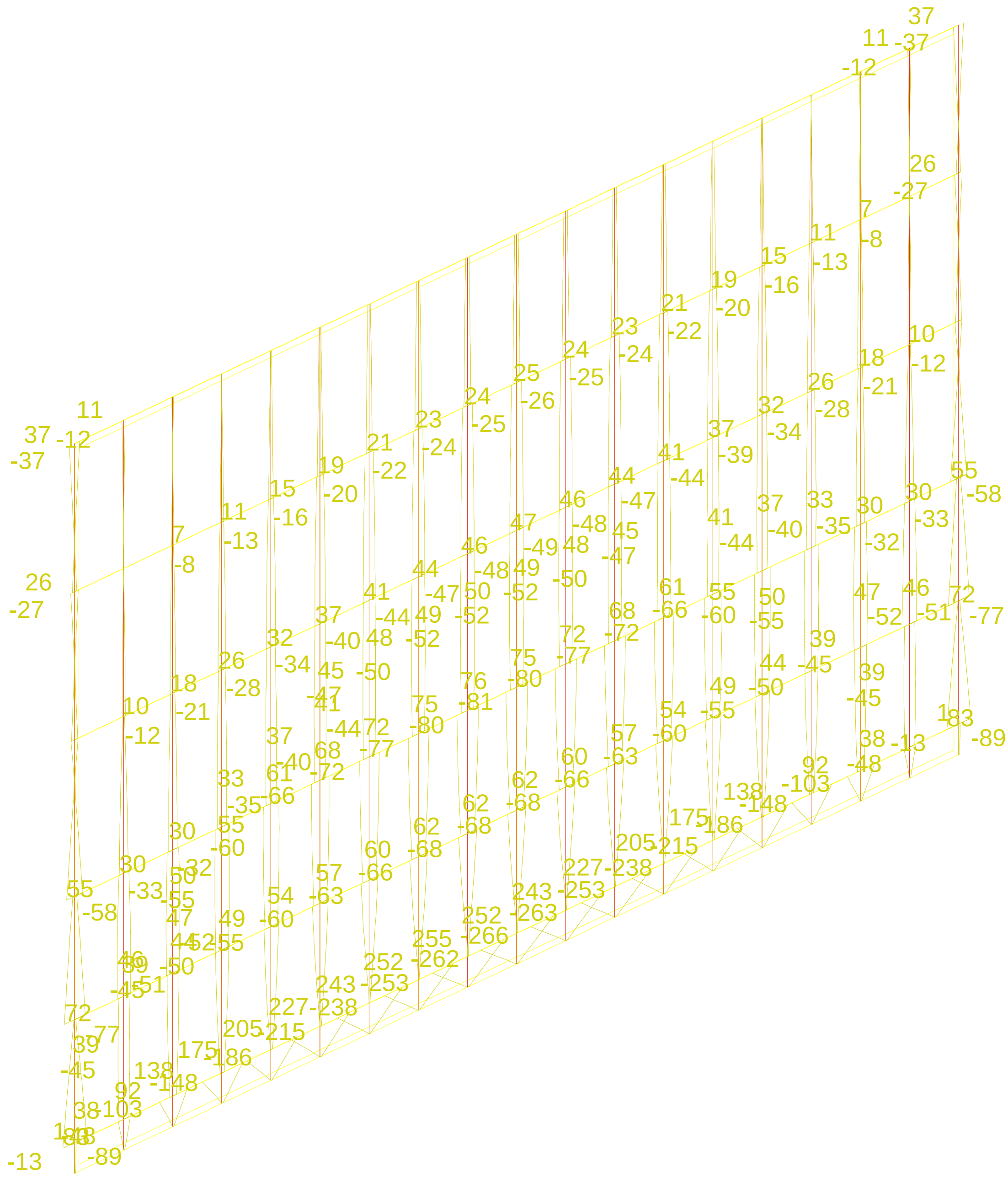
Reaction forces - ULS (SW+DWC+EQ)



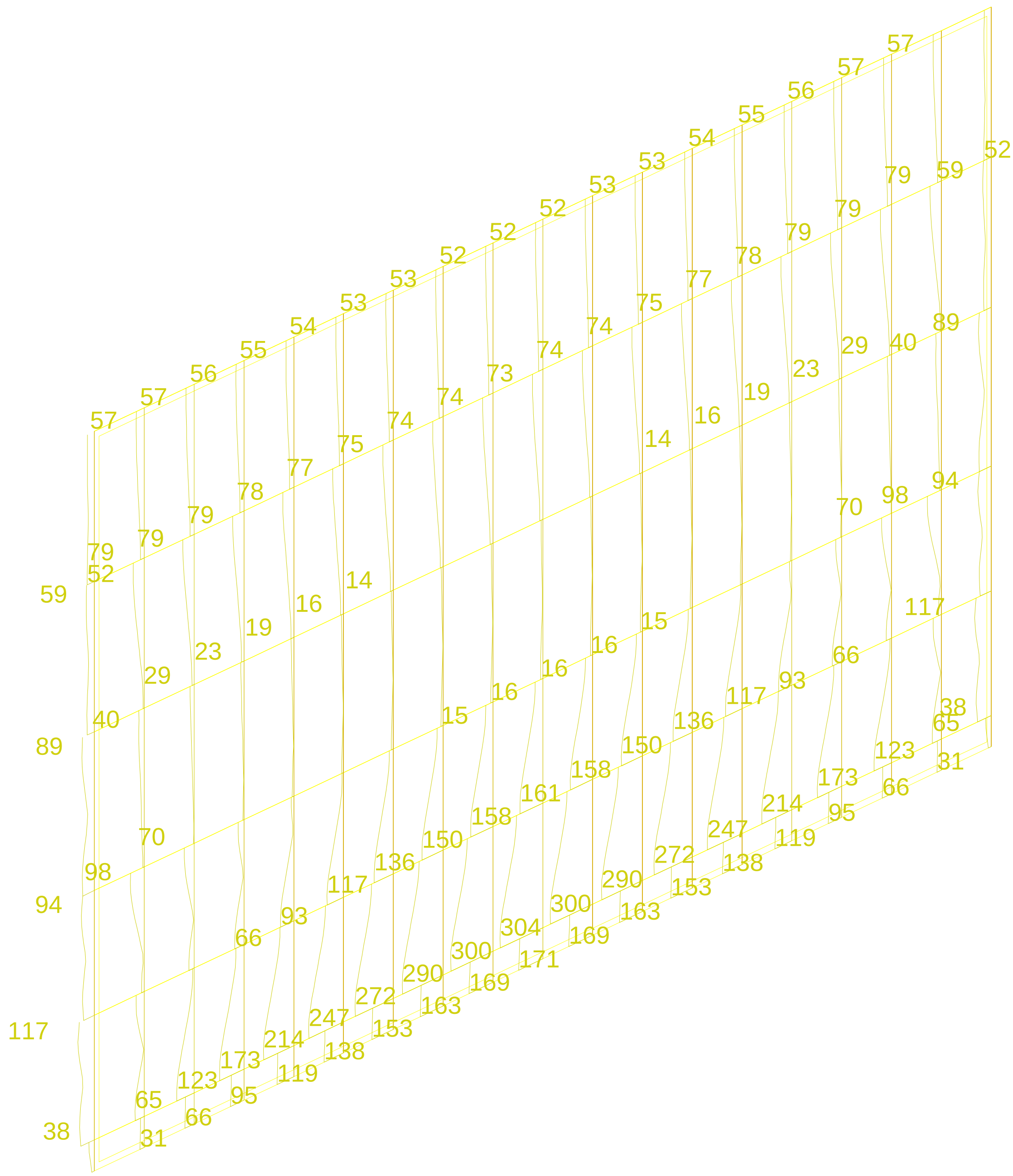
Normal stresses in horizontal frames : ULS



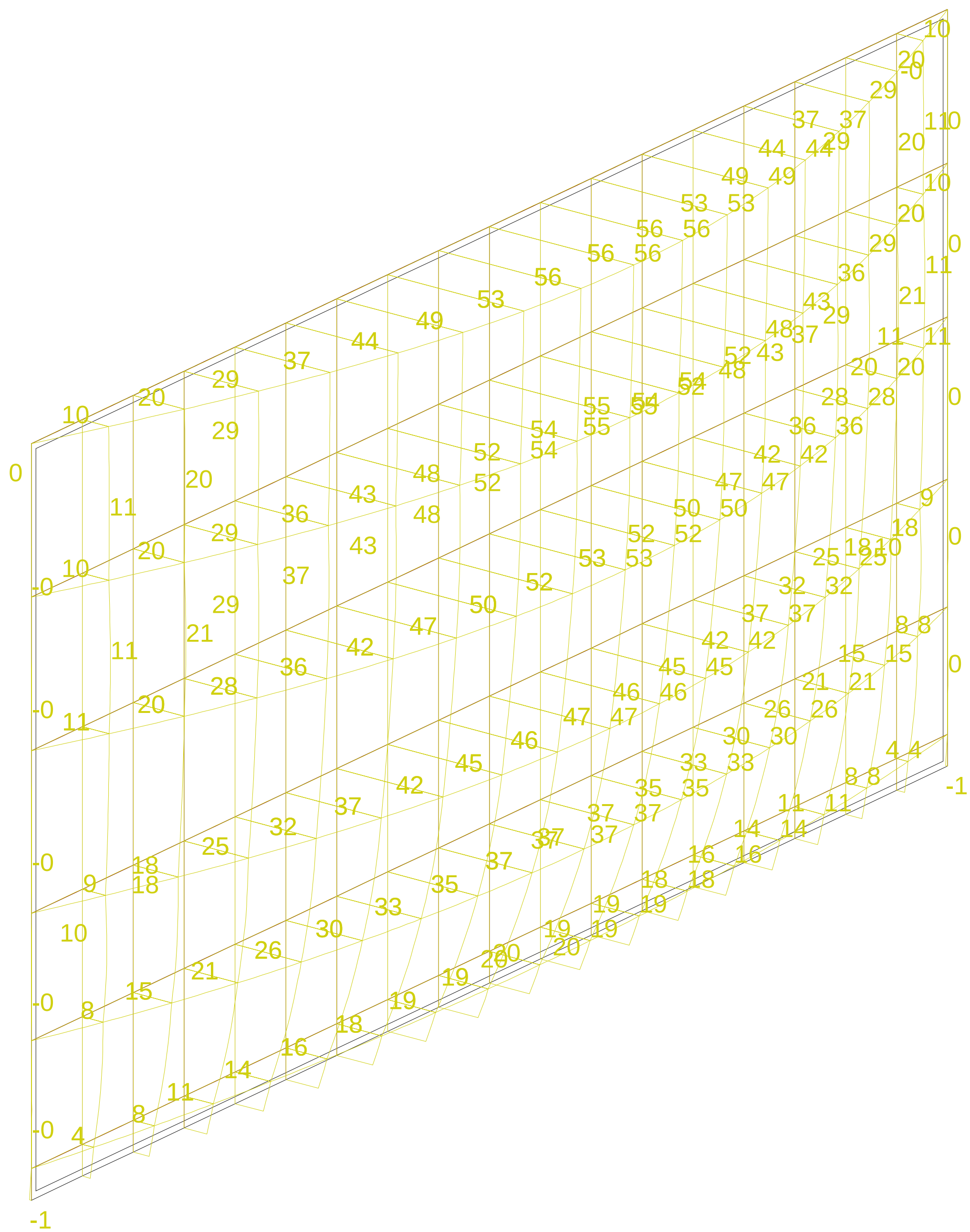
Shear stresses in horizontal frames : ULS



Normal stresses in vertical frames : ULS



Shear stresses in vertical frames : ULS



Deformations - uz in beams : SLS

Appendix B : 3D-FEM calculation half gate : Gate AT3 - 10 m 19 frames

Table of contents

Basic data
Type of structure : General XYZ

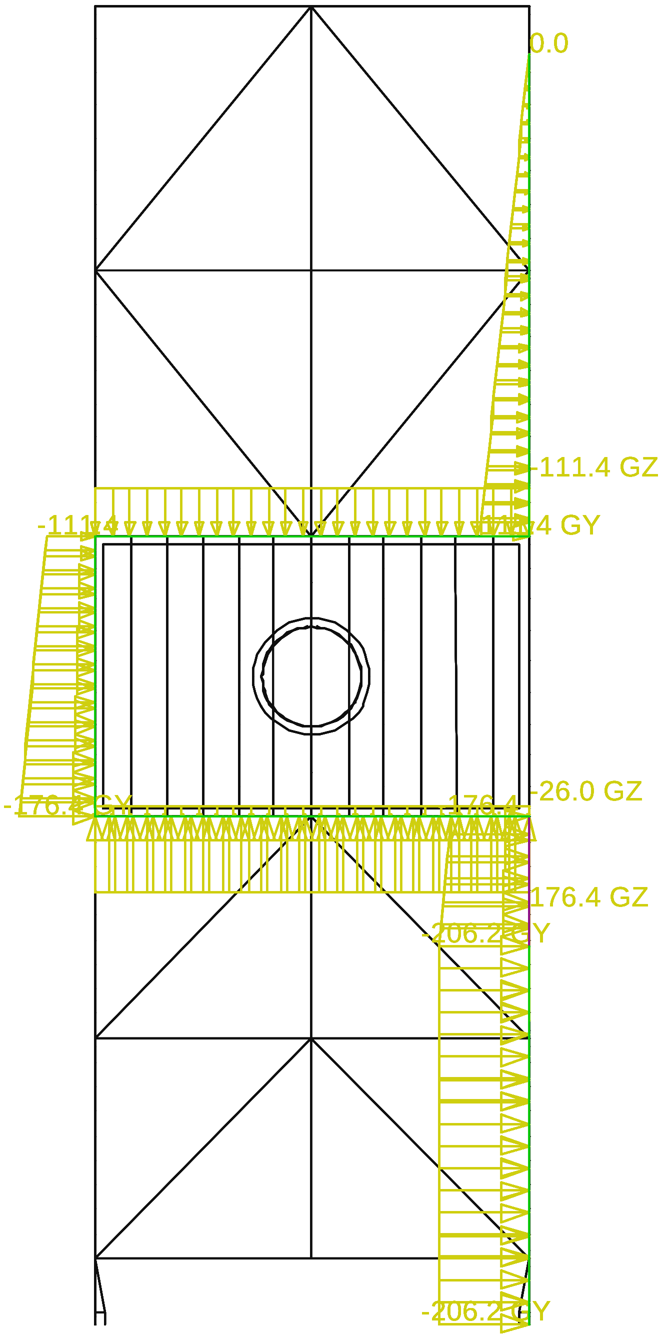
Number of nodes:	1540
Number of members:	1615
Number of 1D macros:	1605
Number of bound. lines:	1756
Number of 2D macros:	216
Number of profiles :	77
Number of cases:	6
Number of materials:	2

Material

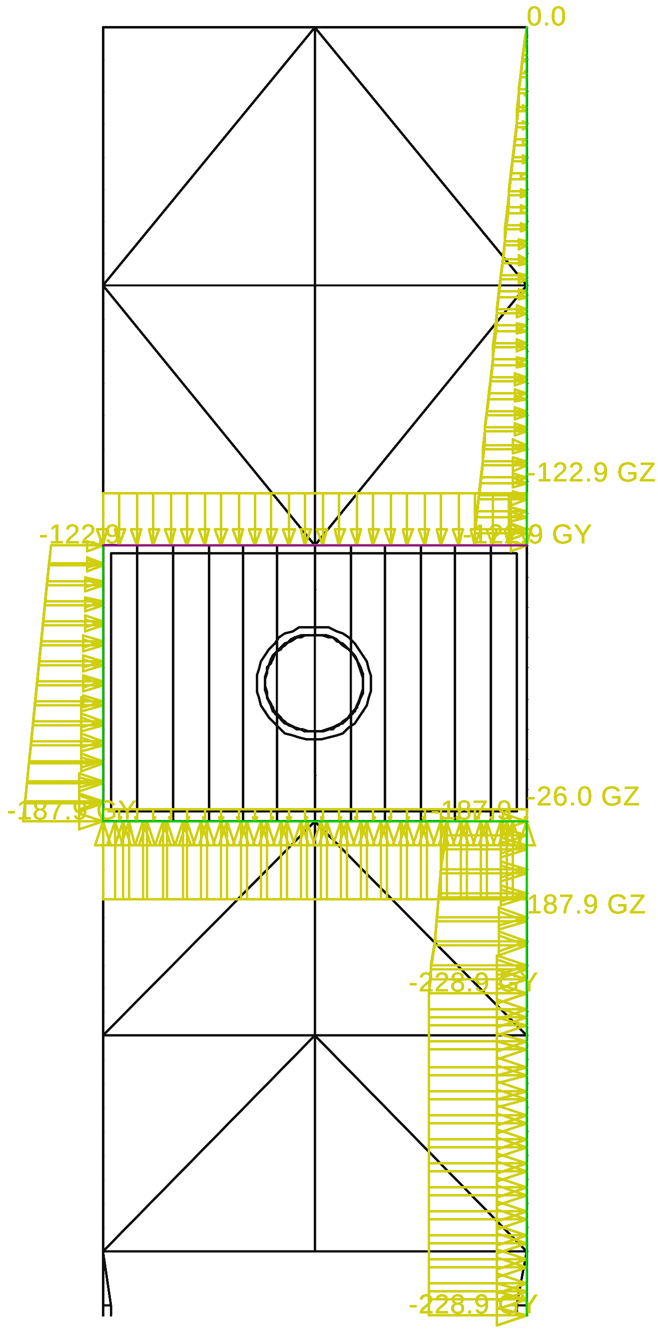
Name:		
S 355		
	Ultimate strength	510.000 MPa
	Yield design	355.000 MPa
	E modulus	210000.00 MPa
	Poisson coeff.	0.30
	Density	7850.000 kg/m^3
	Extensibility	0.012 mm/m.K
S 355		
	Ultimate strength	510.000 MPa
	Yield design	355.000 MPa
	E modulus	210000.00 MPa
	Poisson coeff.	0.30
	Density	7850.000 kg/m^3
	Extensibility	0.012 mm/m.K

Loadcases

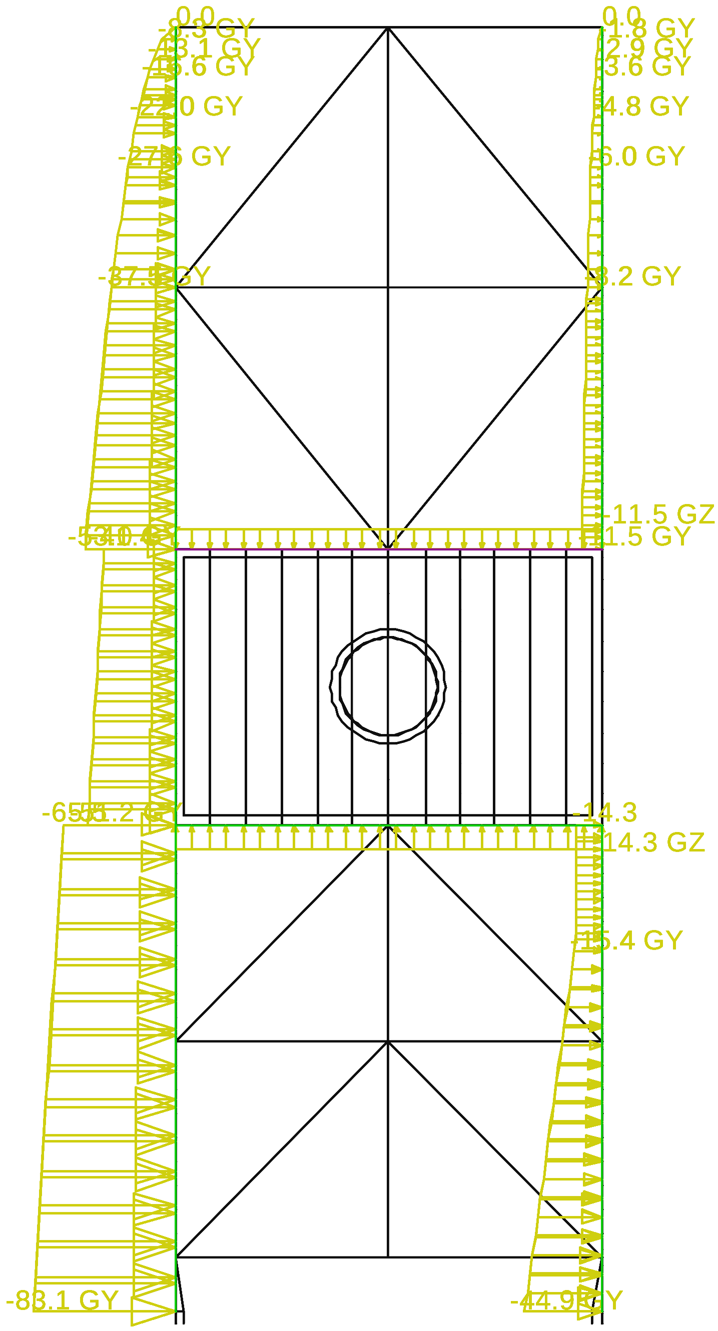
Case	Name:	Description
1	WR normal +18.30 -2.32 PLD	Variable - LC Water Excl.
2	WR ultimate +19.45 -3.44 PLD	Variable - LC Water Excl.
3	DWC (NOT)	Variable - LC Water Excl.
4	Self Weight	Self weight. Direction -Z
5	EQ : Westergaard MCE	Variable - Earth quake
6	Blow out (200 kN/m^2)	Variable - Maintenance



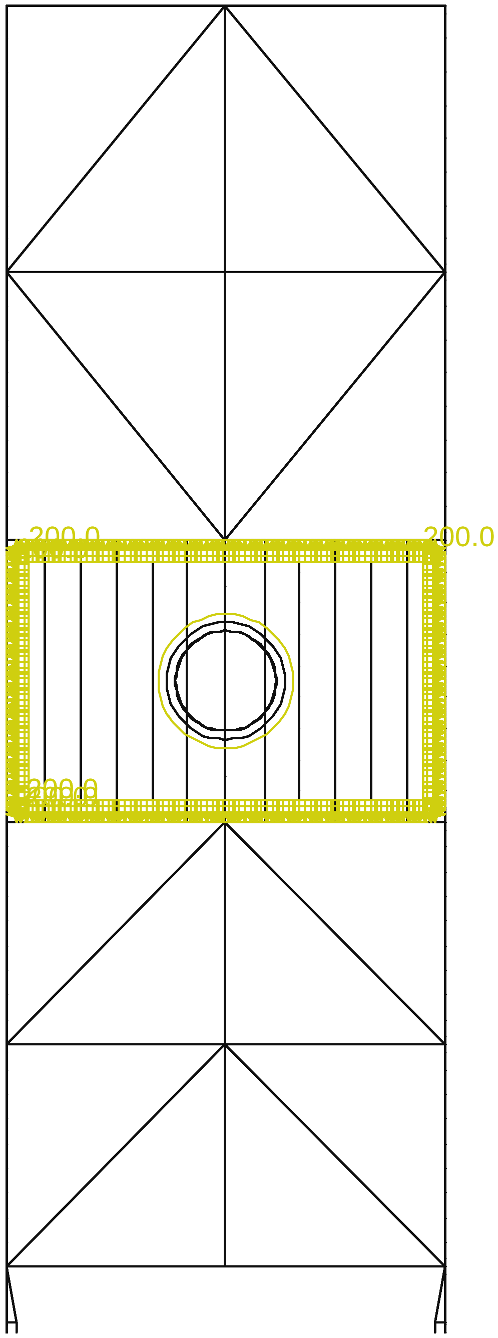
LC WRN (+18.30 -2.32 PLD)



LC WRU (+19.45 -3.44 PLD)



LC EQ : Westergaard MCE



LC Blow out (200 kN/m^2)

Combinations

Combi	Norm	Case	coeff
1.Water	User-ultimate	1 WR normal +18.30 -2.32 PLD	1.35
1.Water	User-ultimate	2 WR ultimate +19.45 -3.44 PLD	1.20
1.Water	User-ultimate	3 DWC (NOT)	0.00
1.Water	User-ultimate	4 Self Weight	1.35
2.Water+EQ	User-ultimate	1 WR normal +18.30 -2.32 PLD	1.00
2.Water+EQ	User-ultimate	2 WR ultimate +19.45 -3.44 PLD	1.00
2.Water+EQ	User-ultimate	3 DWC (NOT)	0.00
2.Water+EQ	User-ultimate	4 Self Weight	1.00
2.Water+EQ	User-ultimate	5 EQ : Westergaard MCE	1.00
3.	User-serviceability	1 WR normal +18.30 -2.32 PLD	1.00
3.	User-serviceability	2 WR ultimate +19.45 -3.44 PLD	1.00
3.	User-serviceability	3 DWC (NOT)	0.00
3.	User-serviceability	4 Self Weight	1.00
3.	User-serviceability	5 EQ : Westergaard MCE	0.00

Basic rules for generation of ultimate load combinations:
1 : 1.35*LC1 / 1.20*LC2 / 0.00*LC3 / 1.35*LC4
2 : 1.00*LC1 / 1.00*LC2 / 0.00*LC3 / 1.00*LC4 / 1.00*LC5

Basic rules for generation of serviceability load combinations:
1 : 1.00*LC1 / 1.00*LC2 / 0.00*LC3 / 1.00*LC4 / 0.00*LC5

List of extreme ultimate load combinations
1/ 2 : +1.00*LC4
2/ 1 : +1.35*LC4
3/ 2 : +1.00*LC1+1.00*LC4
4/ 2 : +1.00*LC2+1.00*LC4
5/ 2 : +1.00*LC4+1.00*LC5
6/ 1 : +1.20*LC2+1.35*LC4
7/ 1 : +1.35*LC1+1.35*LC4
8/ 2 : +1.00*LC1+1.00*LC4+1.00*LC5
9/ 2 : +1.00*LC2+1.00*LC4+1.00*LC5

List of extreme serviceability load combinations
1/ 1 : +1.00*LC4
2/ 1 : +1.00*LC1+1.00*LC4
3/ 1 : +1.00*LC2+1.00*LC4

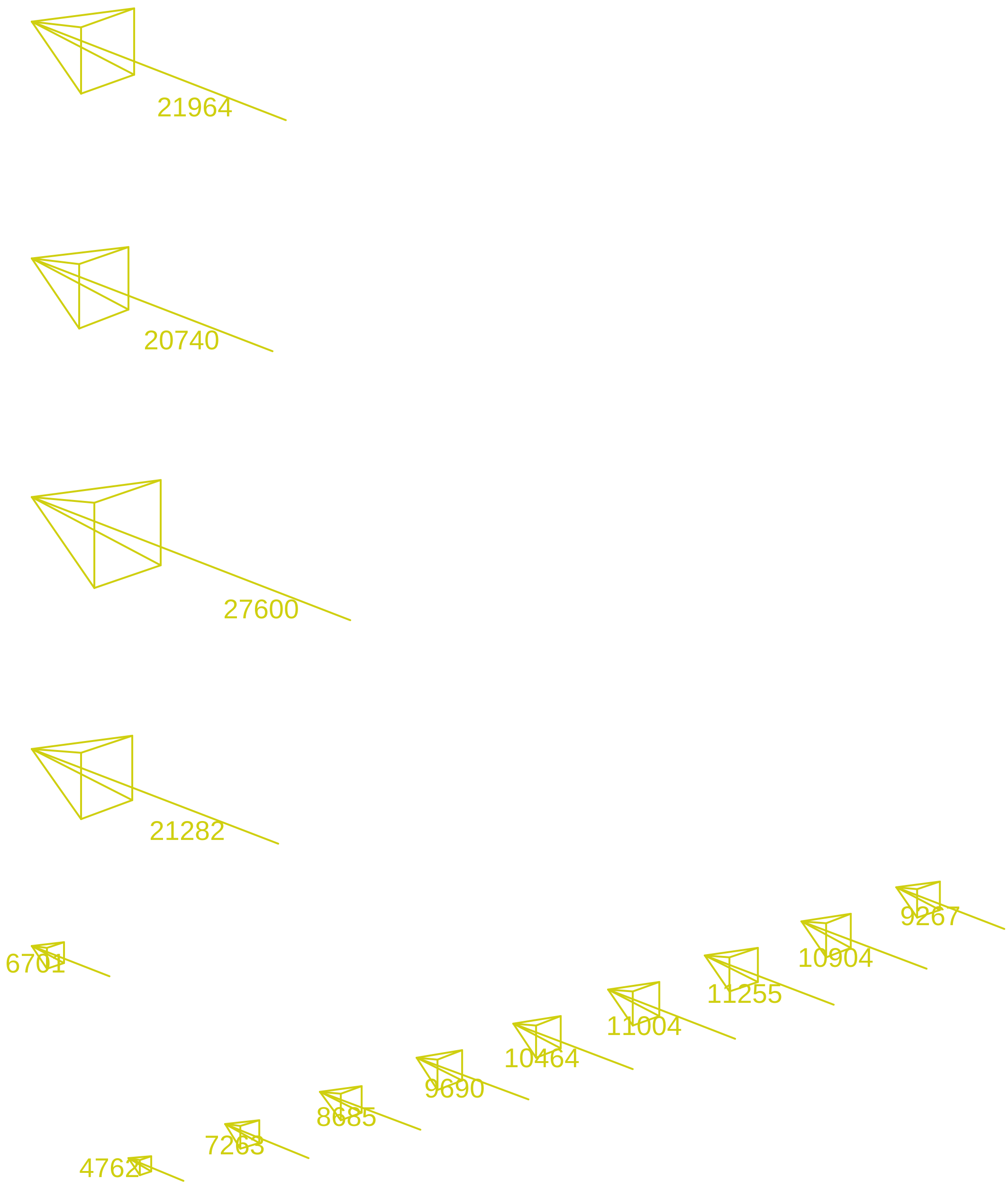
Calculation protocol.

Linear calculation

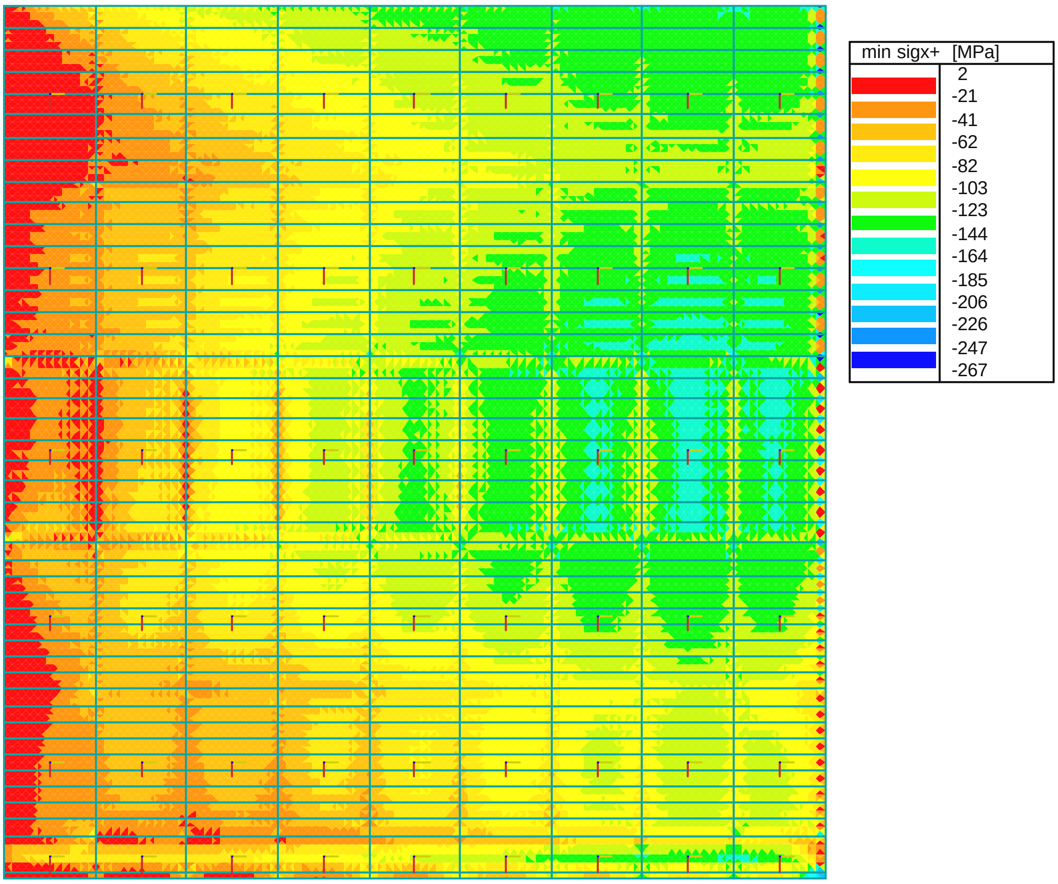
Number of 2D elements Number of 1D elements Number of mesh nodes Number of equations	45146 17294 43572 261432
Loadcases	LC 1 WR normal +18.30 -2.32 PLD
	LC 2 WR ultimate +19.45 -3.44 PLD
	LC 3 DWC (NOT)
	LC 4 Self Weight
	LC 5 EQ : Westergaard MCE
	LC 6 Blow out (200 kN/m^2)
Bending theory	Mindlin
Start of calculation	27.04.2005 19:22
End of calculation	27.04.2005 19:37

Sum of loads and reactions.

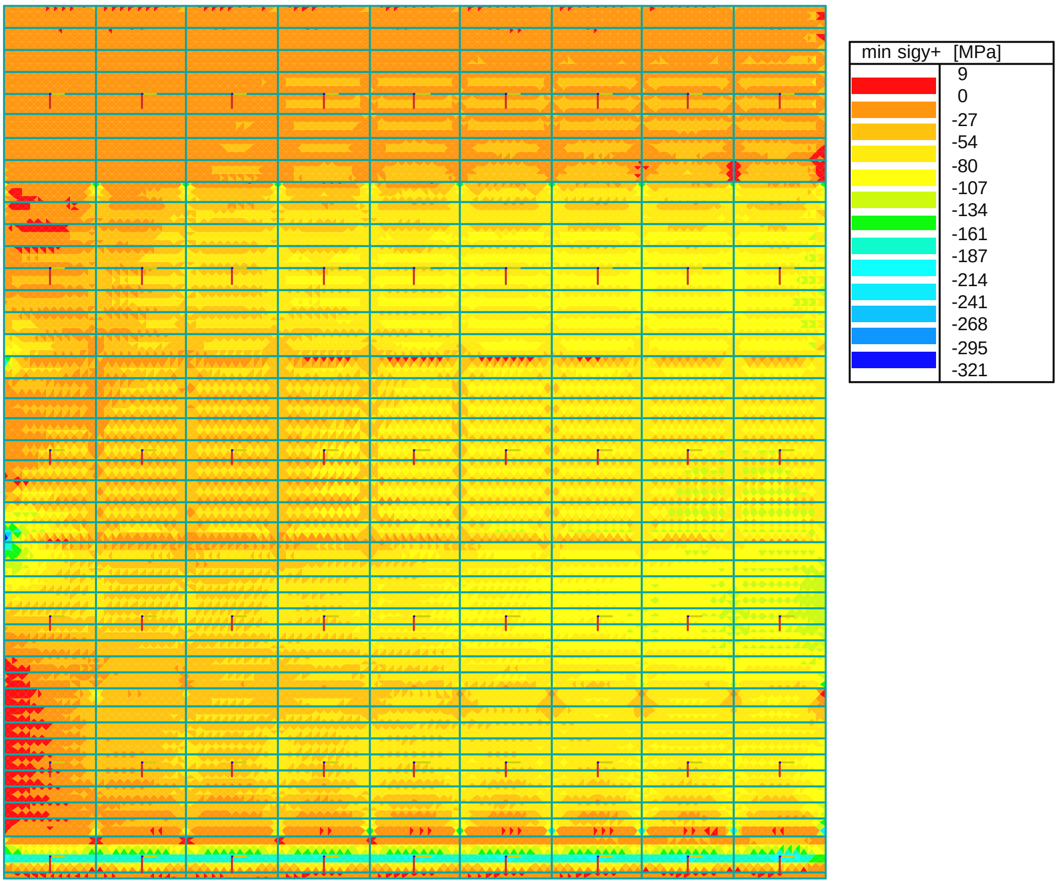
		X	Y	Z
loadcase 1	loads	0.0	-112482.0	11161.8
	reactions	0.0	112482.0	-11161.8
	contact	0.0	0.0	0.0
loadcase 2	loads	0.0	-124965.4	11161.8
	reactions	0.0	124965.4	-11161.8
	contact	0.0	0.0	0.0
loadcase 3	loads	0.0	-0.0	0.0
	reactions	0.0	0.0	-0.0
	contact	0.0	0.0	0.0
loadcase 4	loads	0.0	0.0	-11442.2
	reactions	0.0	0.0	11442.2
	contact	0.0	0.0	0.0
loadcase 5	loads	0.0	-56615.6	801.4
	reactions	0.0	56615.6	-801.4
	contact	0.0	0.0	0.0
loadcase 6	loads	-0.0	0.0	-0.0
	reactions	-0.0	-0.0	0.0
	contact	0.0	0.0	0.0



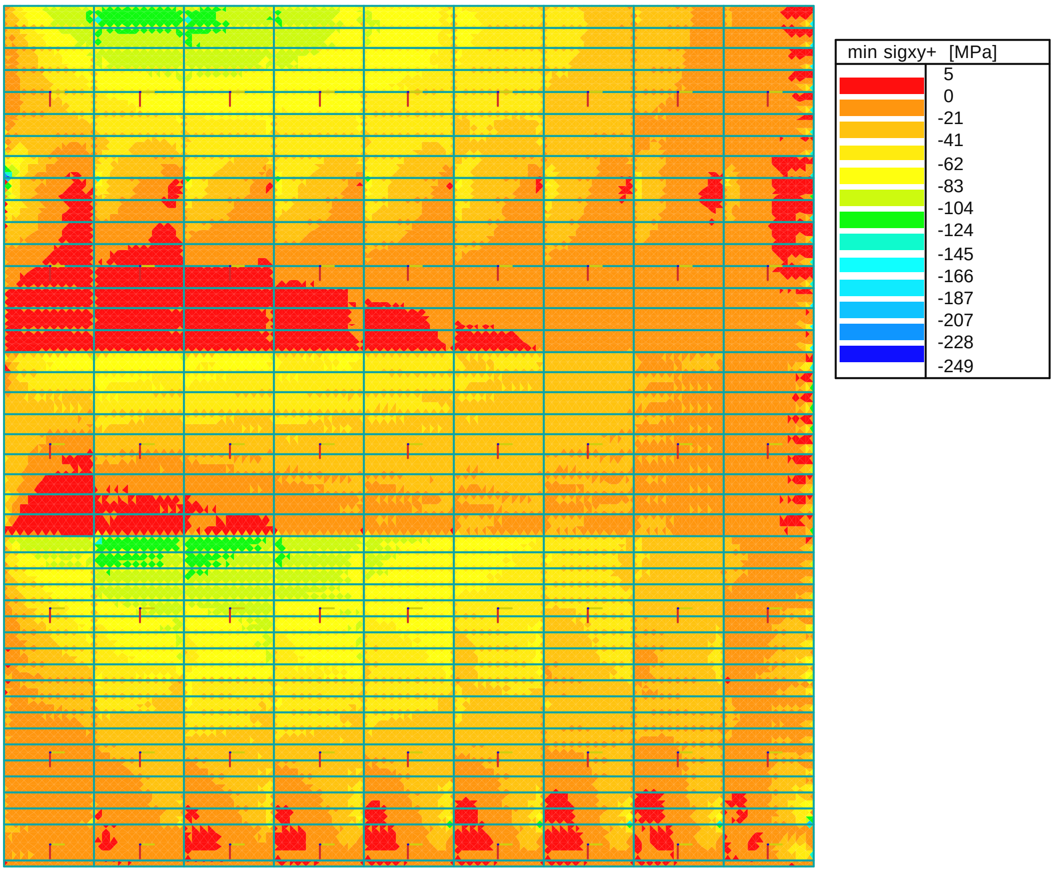
Reaction forces - ULS (SW+DWC+EQ)



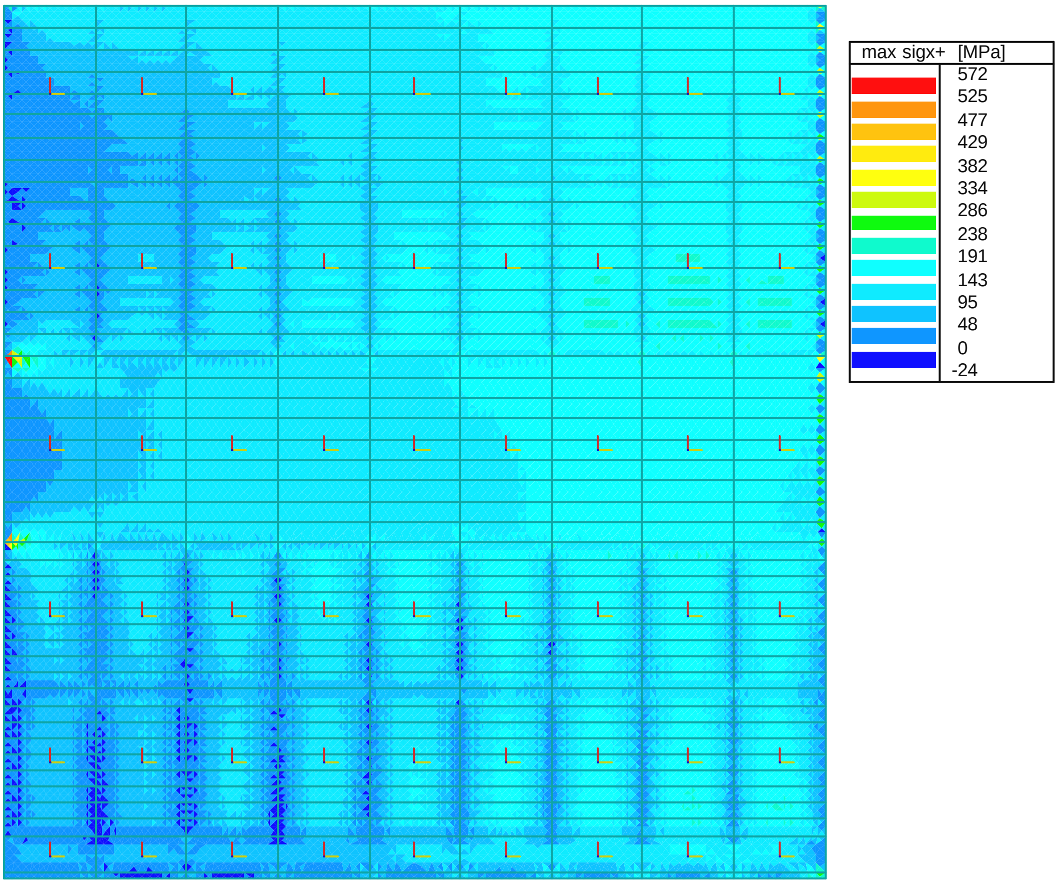
Upstream plating : normal stresses - min sigx+ - ULS



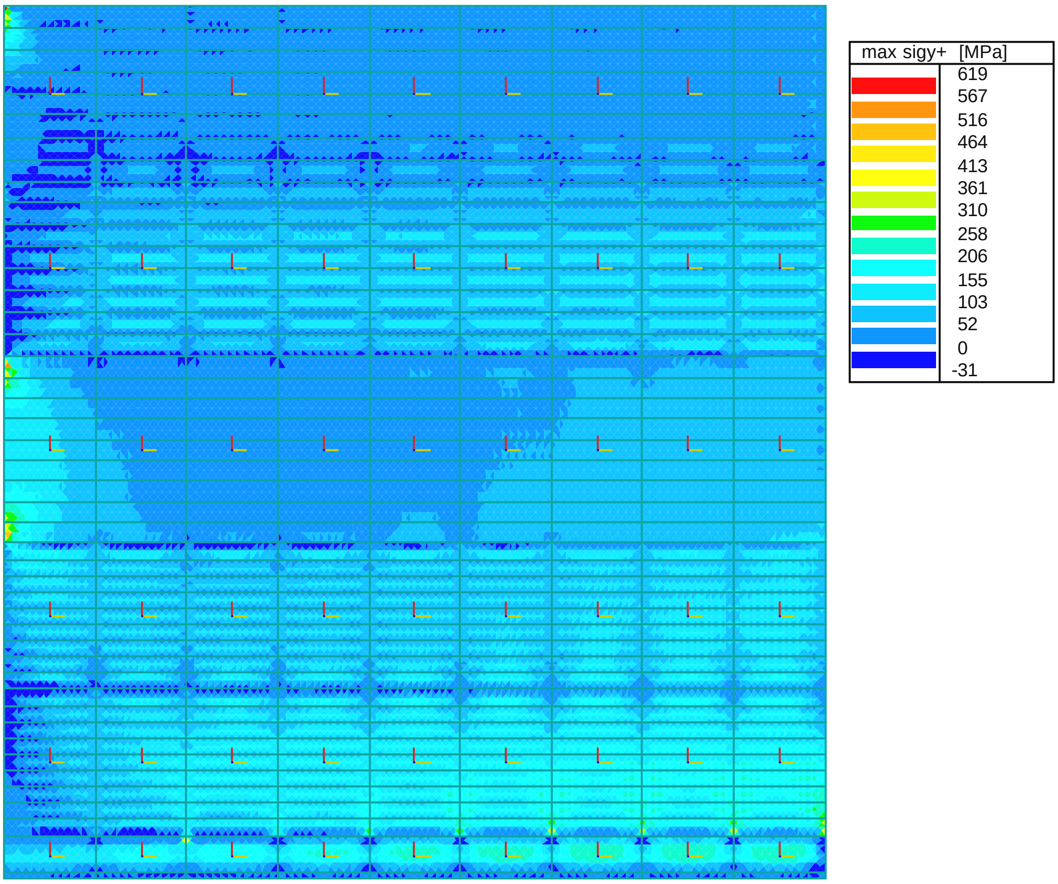
Upstream plating : normal stresses - min sigy+ - ULS



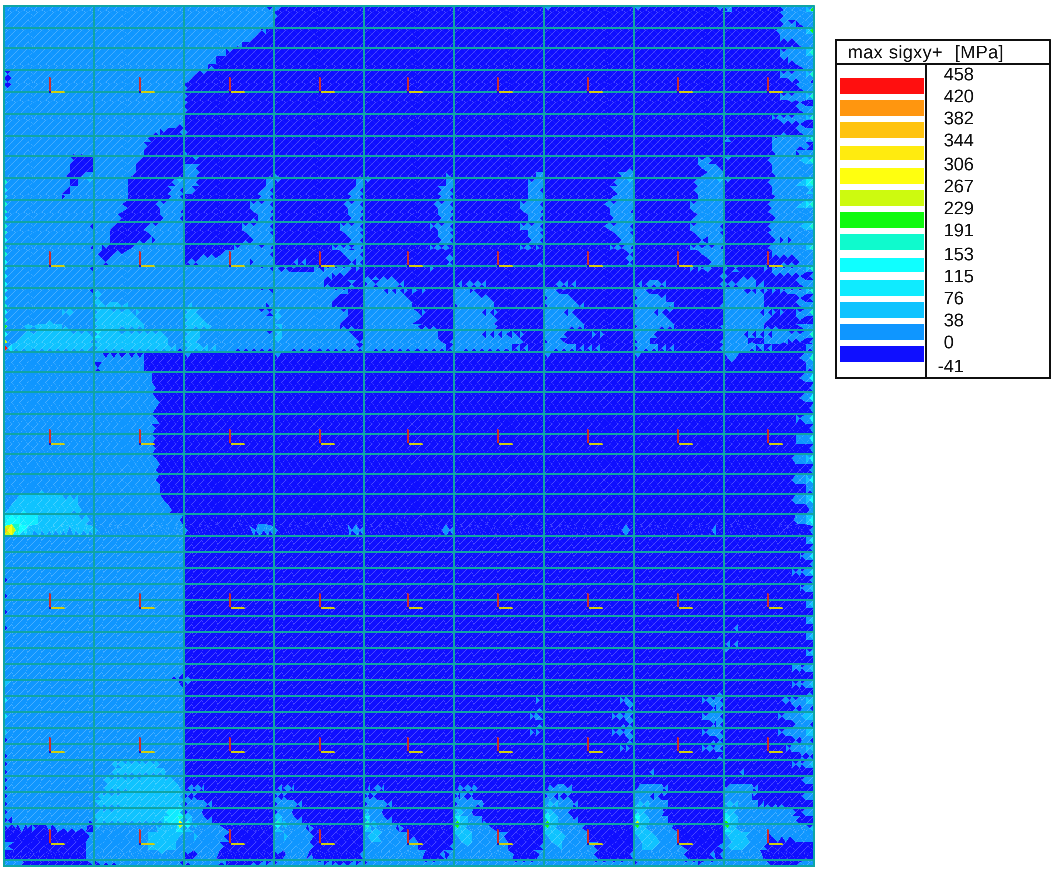
Upstream plating : shear stresses - min sigxy+ - ULS



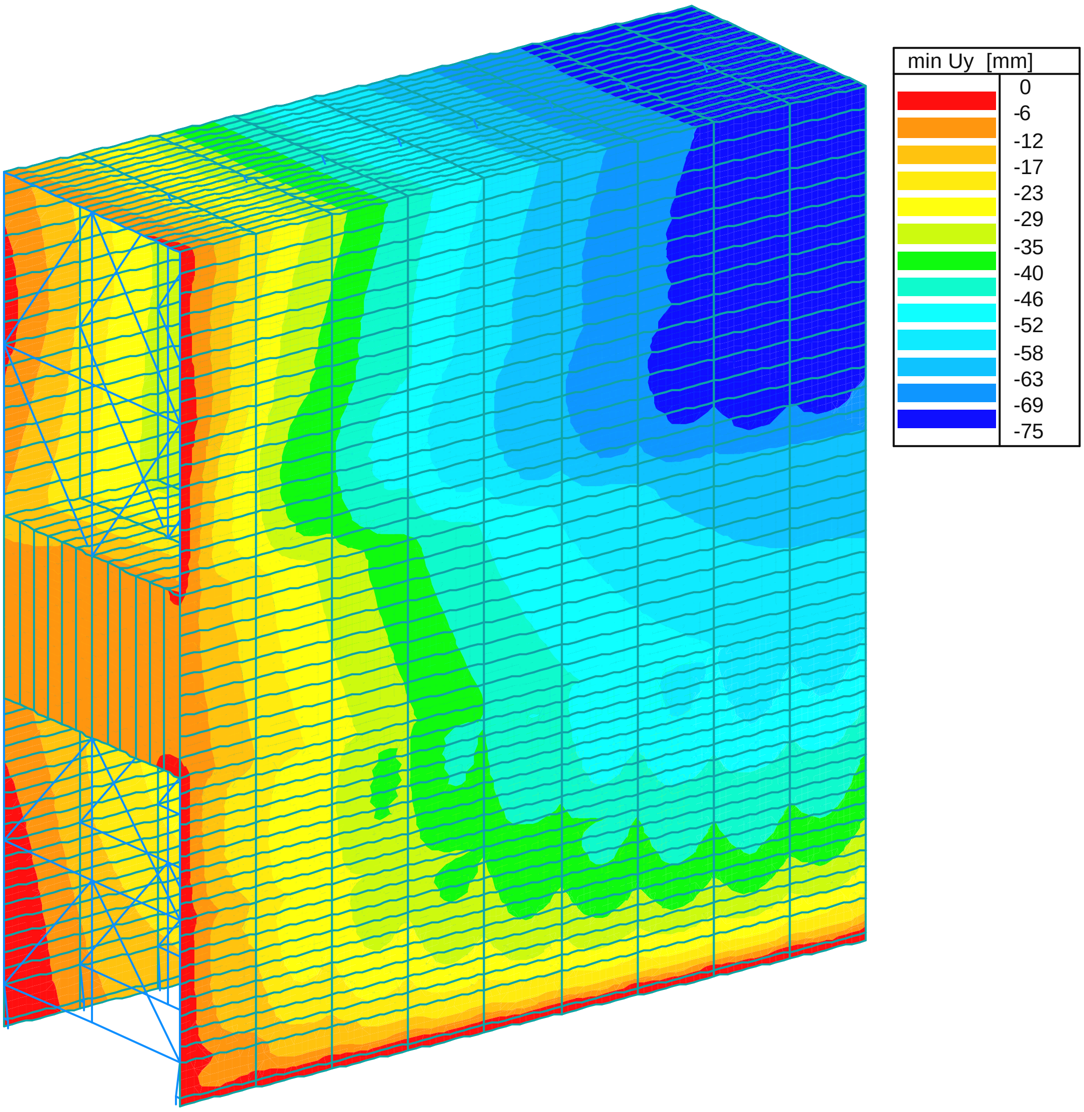
Downstream plating : normal stresses - max sigx+ - ULS



Downstream plating : normal stresses - max sigy+ - ULS



Downstream plating : shear stresses - max sigxy+ - ULS



Deformations - min Uy - SLS

CPP Consorcio Post-Panamax

GATES OF WSB OF THE MAGDEBOURG (ROTHENSEE) LOCK SYSTEM

Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual design - Contract SAA-143351

ATLANTIC LOCKS HARMONIZATION

TASK A4e – CULVERT AND WSB CONDUIT GATES

Rev B

Tractebel Development Engineering
TECHNUM
COYNE ET BELLIER
Compagnie Nationale du Rhône

in association with

SBE
PORT OF ANTWERP
FITA AOSO

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4. Cross section of a gate wheel of Berendrecht lock,
5. Typical gate structure,
6. Upstream and downstream sealing (Music note J-shape type),
7. Side and bottom seals (typical example).
8. Pictures – Typical seals view (Zandvliet lock - Belgium)

1 SUITABILTY OF DIFFERENT TYPES OF GATES

1.1 GENERAL

Throughout this report, the term "valve" will only be used in case of *butterfly valves* or of *cylindrical valves*. All other valves will be called gates. The culvert valves and conduit valves have then been replaced by *culvert gates* and *conduit gates*. The latter is also referred to as WSB gates (Water Saving Basins gates).

The analysis of the suitability of different types of gates is given in the report R4-E (Conceptual Design of Post Panamax locks – TASK 4 E - CULVERT AND CONDUIT VALVES), dated 15.11.2002.

In this report the different types of gates have been analyzed taking into account reliability, maintenance, manufacturing and construction costs, expected service life, design and construction, sensibility to cavitations and vibration.

In relation with the civil works, the overall size of the gates has also played a major role in determining the most suitable type of operating gate for filling and emptying the lock.

The types of gates/valves that have been examined are:

- Vertical-lift gates including:
  - fixed-wheel gates,
  - sliding gates
- Tainter gates including:
  - conventional tainter gates,
  - reverse tainter gates,
- Stoney gates,
- Butterfly valves,
- Cylindrical valves,
- Grid type gates.

To assess the most suitable type of gates/valves to be used for the Post Panamax locks, a comparative table has been elaborated. It is given in paragraph 1.2. below.

1.2 COMPARATIVE TABLE

The different types of lock gates/valves are listed in the table below. Several criteria are used to evaluate the gate/valve types. These criteria are linked with a weight factor, determined according to their importance.

The gates/valves are appraised on a 1 to 5 scale for each criterion. These scores are multiplied by the weight factor, resulting in a total quotation for each type of gate/valve.

The fixed wheel gate obtained the best overall quotation.

	weight factor	Fixed-wheel gate (FW)	Sliding Gate (Sl)	Grid Type Gate (GT)	Tainter Gate (Tt)	Butterfly Valve (Bt)	Cylindrical Valve (Cy)	Stoney Gate (St)
Reliability	0.20	4	4	2	4	4	3	3
Maintenance	0.20	4	4	2	3	3	3	2
Construction cost	0.15	5	4	3	3	3	3	3
Service life	0.15	4	3	2	4	4	3	2
Design and construction	0.15	4	3	2	4	3	3	3
Sensibility to vibration / cavitation	0.15	3	4	3	3	2	3	3
Total								
Total weight factors	1.00							
Total evaluation (max 5)		4.00	3.7	2.30	3.5	3.20	3.00	2.65
Total evaluation (%)		80	74	46	70	64	60	53

Note: the results of this comparative table remain valid for both flow directions through the gates/valves

1.3 CONCLUSIONS

The conclusions of the report R4E as referenced in §3 were as follows:

"Based on experience with Post Panamax locks and on engineering judgment there are only two types of gates that may realistically be used for the Post Panamax locks of the Panama Canal i.e. fixed wheel gates and sliding gates.

Nowadays vertical-lift gates are preferred for big locks because they are much cheaper to build and do not require the large space that is necessary (for example) for a tainter gate. Moreover, the hydraulic efforts are better distributed to the culvert walls and maintenance is easier.

Within this perspective the choice of fixed wheel gates seems obvious.

Another advantage of course is the actual know-how of ACP and the infrastructure for the maintenance of flat gates in use at the Panama Canal."

Moreover, the vertical lift gates have proven well for designs where sealing in both directions of water flow is required, such as between the lock chambers and the water saving basins.

2 DESCRIPTION AND DIMENSIONING

2.1 GENERAL

The analysis of the suitability of different types of gates has led to the conclusion that the most suitable type of gate is the fixed-wheel type.

For the 55 m lock chamber width (instead of 61 m previously), the dimensions of the lock culverts and water saving basins (WSB) conduits have been determined in the hydraulic study (report P4C).

The culvert dimensions are **9 (width) x 6 (height) m**.

The WSB conduit dimensions are **4 (width) x 5 (height) m.**

Redundancy (two gates for each culvert) has to be foreseen for the culvert gates, therefore the size of the culvert gates shall be **4.5 x 6m. For the WSB conduits, gate dimensions of 4 x 5m are proposed.**

Hence, on the Atlantic side culverts and WSB conduits are equipped with gates of different size.

The height to width ratio is 1.33 for the culvert gates and 1.25 for the WSB conduit gates, which is quite acceptable.

For the culvert gates, the basic principle adopted for operation reliability is to work with two gates in parallel so that any incident to any gate will not stop the operation of the locks. Furthermore, it also reduces the required gate size.

However the **risk of an asymmetrical operation** of the gates (if one gate fails to open or remains open in an intermediate position) shall have to be assessed (in the preliminary and/or final design). If required, interlocking devices shall have to be foreseen.

Each of the nine **water saving basins** is connected to the locks by four conduits. Two are connected on left hand (near to WSB) side of the corresponding lock chamber, two are connected to the right hand (far of WSB) side. **No** additional provision has been made for **redundancy** of the gates. In case of any trouble on a gate, one conduit will be out of order but the three remaining conduits of the concerned basin will be sufficient to operate almost normally.

However the **asymmetrical operation of the emptying and/or filling** of the corresponding lock chamber gates (if one gate fails to open or remains open in an intermediate position) shall have to be assessed (in the preliminary and/or final design). If required, interlocking devices shall have to be foreseen.

2.2 LAYOUT OF CULVERTS AND WSB CONDUITS

Each culvert and conduit gate is equipped upstream and downstream with bulkhead gates allowing access to the gate(s) after emptying by pumping (by movable pumps) of the space on both sides.

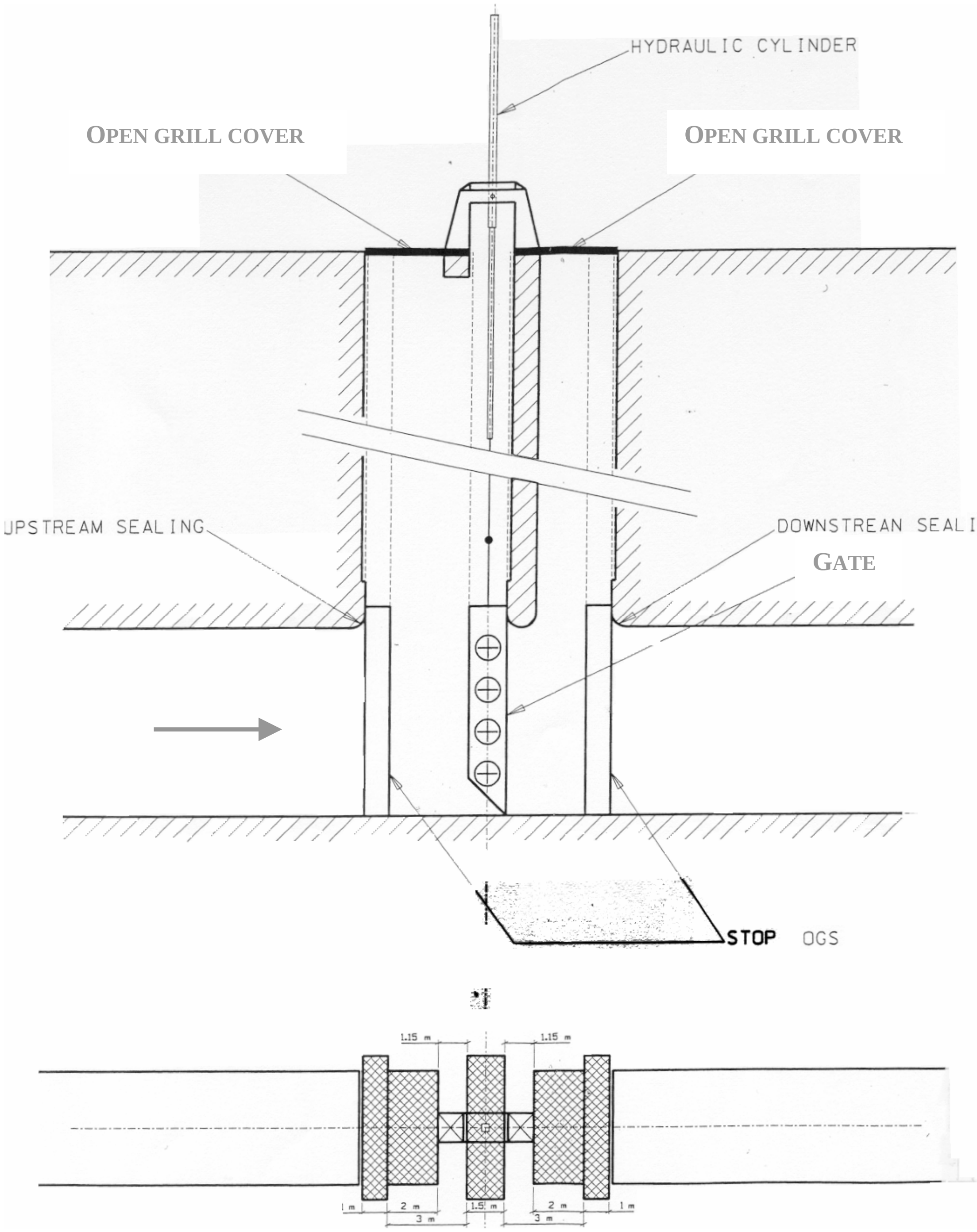
The basins conduits have been arranged two by two (in total four per WSB). The arrangement, with one conduit located on top of the other as foreseen in the initial conceptual design has been abandoned. It makes the WSB gates arrangement much easier and the operation much more reliable.

2.2.1 CULVERTS AND CULVERT GATES

There are two culverts running along each side of the locks. Their sill is at the sill level of the lock chamber. However, the bottom of the rolling gates chambers prevents the culverts from remaining horizontal. Therefore, the culverts are diverted under the rolling gates and the culvert gates are implemented between the main rolling gates.

As mentioned here above, the culvert dimensions are W x H = 9m x 6m. The culverts are locally divided into two sections of W x H = 4.5m x 6m where the culvert gates are to be installed. At full opening of the gate, the total size and thus the mean water velocity remains unchanged.

The next figure shows a basic layout for a culvert gate with two isolating bulkheads. **There is only one flow direction from the left to the right.**



For emptying both sides of the culvert gate, the **sealing conditions** are to be as follows:

- the upstream bulkhead has to be tight on its upstream side,
- the downstream bulkhead has to be tight on its downstream side,
- the gate has to be **tight on its downstream side**.

That design has the advantage (regarding civil works) that only one vertical separation wall is required.

2.2.2 WSB CONDUITS AND CONDUIT GATES

The arrangement of the gates and bulkhead gates is shown on the civil works drawings (ref D4-A-203).

The **fixed-wheel gates** are designed with **upstream and downstream sealing.**

Their leaf structures (and therefore the corresponding slots) are dimensioned to support the maximum static pressure on both sides corresponding to following pressure conditions:

- maximum lock chamber level on one side and WSB completely empty on the other side,
- maximum WSB level on one side and lock chamber completely empty on the other side.

The hydraulic cylinders operating the gates have been pre dimensioned for two cases:

- for the normal operation with the locks and basins filled with water,
- for the maximum static head.

The power required for the gate operation in the most critical case, is the one taking into account maximum static head.

The bulkhead gate (WSB side) is of the sliding type in two or three elements and is designed with a double sealing system which allows to:

- empty the WSB while keeping the locks in operation,
- empty the space between the two bulkhead gates to give access to the conduit gate and slots for maintenance.

The bulkhead gate on the lock chamber side is also in two or three pieces and is designed with a sealing system which allows to:

- empty either the lock chamber or the WSB (for the emptying of the WSB it makes a redundancy while keeping the locks in operation),
- empty the space between the two bulkhead gates to give access to the conduit gate and slots for maintenance.

The basic data for designing the gates (dimensions and maximum static head) are the same as those of the bulkhead gates.

The bulkhead elements can be lowered or removed by means of a mobile gantry crane equipped with an automatic lifting beam.

The 36 conduit gates are also the same. They are dimensioned for the maximum head of 42.52m.

2.3 BASIC DATA FOR DESIGN

The values indicated below provide, for the culvert and WSB gates as well as for the bulkhead gates, the maximum static heads of water which have been taken into account for the estimation of the weight of the moving parts.

Maximum head on sill level of culvert gates:	37.03 m
Maximum head on sill level of WSB gates:	42.52 m

The weight of the gates has also been estimated taking into account the operating heads. The values indicated below provide the maximum operating heads of water which have been taking into account.

Maximum head on sill level of culvert gates:	25 m
Maximum head on sill level of WSB gates:	10 m

For each shaft (culvert or WSB conduit), the calculation of the weight of the gate and its related bulkheads has been computed using the same water heads.

2.4 ESTIMATED WEIGHTS

A reliable determination of the moving part of a fixed-wheel gate by a comprehensive study based on preliminary data and admissible stresses is a quite long and difficult exercise. To determine an approximate weight, it is common practice to make a comparison with existing gates, of course, of the same type.

Estimation of the weight is based on the main parameters, i.e.:

- the dimensions (width and height);

- water pressure on the sill.

It can be developed by a formula based on statistical data. The weight of the slot embedded fixed parts has then to be added.

This procedure gives an acceptable approach for conceptual design.

The formula used here (see Water Power and Dam Construction by P.C. Erbiste May 1984) is a function of W, h, and H where:

- W is the span,
- h is the gate height,
- H is the static head on the gate bottom seal.

The weight of the gate leaf is given by the formula (see abacus – annex 1):

Weight of a fixed-wheel gate: = 0.706 (W².h.H)$^{0.7}$

Given the static heads are the highest ones (compared to the operating heads), only them have been taken into account for the calculation of the weights.
Span width, height, static head on seal bottom and weight of gate or bulkhead leaf are given in annex 3.

The estimated weight of the culvert gate is 25.5 tons and the estimated weight of the WSB conduit gate is 20.9 tons. The weights of the culvert and WSB conduit gates are very close to each other. At this conceptual stage, it clearly appears that the same design should be used for both gates.

The incurred costs/benefits that will result are the following:

- From the standardization point of view : same drawings, same manufacturing processes, erection procedures, …
- From the operational and maintenance point of view : reduced amount of spare parts, better material knowledge of the maintenance people, …

It is reminded that to check the procedure, a preliminary calculation of a WSB fixed-wheel gate structure has been performed (see Annex 2). The calculation has confirmed the results of the above formula.

Moreover, the weight of **one meter** of embedded fixed parts is estimated to:

- Culvert fixed-wheel gates at the bottom of the slot:	800 kg (last 12m[1])
- Culvert fixed-wheel gates at the upper part of the slot (only for guiding):	200 kg
- Culvert sliding bulkhead at the bottom of the slot:	500 kg (last 9m[2])
- Culvert sliding bulkhead (only for guiding)	200 kg
- WSB fixed-wheel gates at the bottom of the slot:	1,000 kg (last 12m)
- WSB fixed-wheel gates at the upper part of the slot:	200 kg
- WSB sliding bulkhead at the bottom of the slot:	500 kg (last 9m)
- WSB sliding bulkhead (only for guiding)	200 kg

Note: Lintel and sill embedded parts have been added separately. For the gates the weight of said parts is taken as 800 kg/m, for the bulkheads, it has been taken as 500 kg/m.

[1] Two times the height of the gates (2x6=12m)
[2] One time first leaf plus two times second leaf (3+2x3=9m)

2.5 CONSTRUCTION DETAILS

Hydraulic servomotor operated, the fixed wheel gates are equipped with wheels revolving on fixed axles cantilevered from the gate frame (see annex 4 for typical example of a sectional view of one wheel of the Berendrecht culvert gates). Wheels can be of the flat type (rolling on stainless steel tracks) or of the flanged type (rolling on rails). Tracks must withstand the bearing pressures and distribute them to the concrete structure behind. The **number of wheels** will be based on the steel characteristics. It shall **not be less than 6 wheels.**

A typical horizontal sectional view of a gate (or bulkhead) welded structure is shown in Annex 5. Horizontal plate girders or standard T or I-shape beams are the main force resisting members of the gate.

The distance between horizontal girders may vary according to the hydrostatic pressure. Diaphragm plates and intercostals are also used as reinforcement to distribute loads more uniformly.

WSB fixed-wheel gates have to resist to water pressure and **be tight in both directions** as for the locks submitted to tidal effects.

The access shaft for maintenance will be used as **surge chambers** during operation of the gates.

Tolerances must be adequate to ensure watertight seals. That is the reason why it is recommended to use very rigid U-shape steel guiding for the gates to avoid any movement during embedding of the fixed parts.

The gate and wheels are permanently under water. Maintenance of these wheels and bearings is possible by lifting out of water the moving parts. Wear of these elements can be considerably reduced by using **self lubricating** material.

SEALING SYSTEM

Seals are usually made of rubber with or without a PTFE (Teflon) overlay (**PTFE overlay is preferred**). The seals are often of the music note shape or lip type.

For the WSB gates being tight for water flowing in both directions, the lip seals adopted for Berendrecht (see sectional view of the wheel) should be convenient.

Lintel seal and side seals: can be of the upstream or downstream type (see figure - Annex 6)

Bottom seal can be flat or also of the J-shape type.(see figure – Annexes 6 and 7)

MAINTENANCE OF THE GATES AND BULKHEADS

Maintenance work on gates and bulkheads (as wheels and relevant slots) consists mainly in the replacement of rubber seals and painting. Overhaul and/or replacement of wheels could also be foreseeable. Moreover, the maintenance works will have to include the replacement of the sacrificial anodes whenever necessary.

During normal operation, any trouble with one culvert or WSB gate (blocking or incident on the oil system) will not interfere on the ship transit except concerning the operation time. Every gate can be isolated and maintenance people can reach the upstream or downstream side of the gate by use of bulkheads after emptying of the space between them.

In case of planned replacement of seals or painting, the gate will be lifted out by use of a 100 tons gantry crane moving on rails. This crane will be provided by truck, assembled and installed on the railway located above the gate slot. After dismantling of the gate, the work will be carried out in good conditions in the maintenance building. Two mobile cranes will be necessary for the 52 gates and 14 bulkheads. Rails will be installed between and outside of all the rows of WSB and culvert slots.

For the culverts, 8 (4 x 2) bulkheads are foreseen. It enables to close completely one culvert using 2 x 2 bulkheads at each of the culvert extremities.
For the WSB conduit, 6 (2 x 3) bulkheads are foreseen. It enables to close completely one conduit.

Bulkheads gates can be stored outside or suspended into the slots (one piece of bulkhead gate per slot). To remove a bulkhead gate, the cranes will be equipped with an automatic lifting beam. Planned maintenance will also be done in the maintenance building.

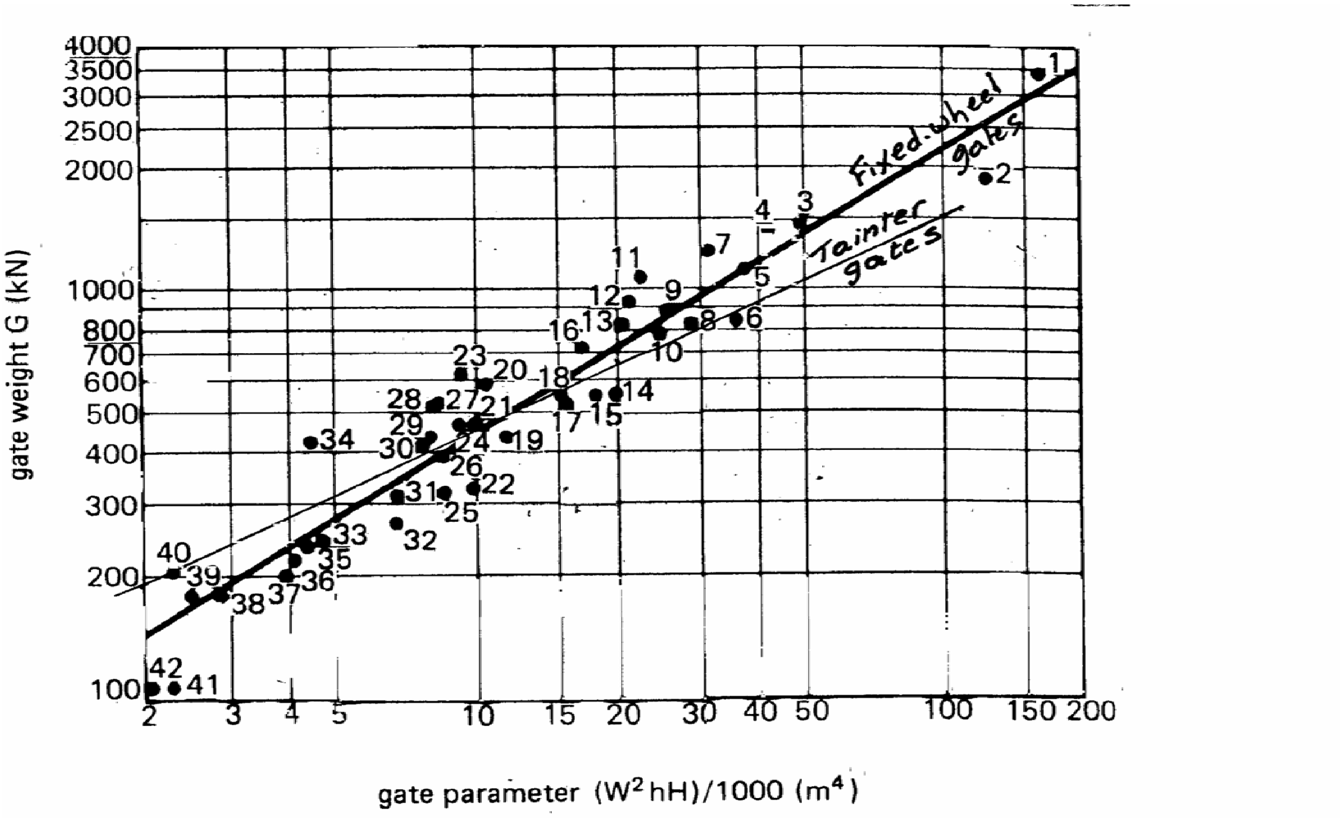
3 REFERENCES

- Hydraulic gates and valves in free surface flow and submerged outlets by Jack Lewin
- Water Power and Dam Construction (review)
- Final report of the International commission for the study of locks (PIANC)
- Engineer manuals
- CCP (2002) “Diseño conceptual de las esclusas Post Panamax – Triple Lift Lock System, Task 4”

ANNEXES

ANNEX 1

Abacus of gate weight versus gate parameter (W, h, H)



Weight of large fixed-wheel gates (W²hH > 2000 m⁴).

ANNEX 2 (Remind of report R4-E date 15.11.2002)

TYPICAL CALCULATION OF A WSB GATE (Hs = 50m)

This calculation is the same as the one included in the report mentioned at the beginning of paragraph 1.1 of this report. The only goal of this calculation is to prove that the use of the general formula (see page 2-6) is relevant for weight calculation.

SKIN PLATE

The estimated skin plate thickness corresponds to a distance of 1.5m between the horizontal I beams and 1m between the vertical T shape intercostals is 4cm

STEEL PLATE		Mesh 1.00 x 1.50 m		LOAD : 50 t/m2	
		span maximum bending moment (tm) :			3.71333
		edges and corners maximum bending moment (tm) :			5.11170
thickness	I/v	relative displacement		maximum stresses	
		span	corner	span	corner
(m)	(m3)	(mm)	(mm)	(kg/mm2)	(kg/mm2)
0.040	0.0002667	0.880	1.540	12.55	19.17
0.035	0.0002042	1.314	2.300	16.39	25.04
0.030	0.0001500	2.087	3.653	22.30	34.08
0.025	0.0001042	3.605	6.312	32.12	49.07
0.020	0.0000667	7.042	12.328	50.18	76.68

choosen thickness: 4 cm

MAIN BEAMS

The horizontal main beams size depends on the span between them and load. According to the I/v required, alternatives were investigated i.e.:

- HE 1000 A
- W 1100 x 400 x 433

MAIN GIRDERS	length (m) :	5.7	distance (m):	1.5	load (t/m):	75	Moment (tm):	304.59375
	E (t/m2):	21000000						

DISPLACEMENT vs INERTIA		STRESS vs I/v	
I (m4)	f (m)	I/v (m3)	STRESS (t/m2)
0.0005	0.098177093	0.005	60918.75
0.0006	0.081814244	0.007	43513.392857
0.0007	0.070126495	0.009	33843.75
0.0008	0.061360683	0.011	27690.340909
0.0009	0.054542829	0.013	23430.288462
0.001	0.049088546	0.015	20306.25
0.0011	0.044625951	0.017	17917.279412
0.0012	0.040907122	0.019	16031.25
0.0013	0.03776042	0.021	14504.464286
0.0014	0.035063247	0.023	13243.206522
0.0015	0.032725698	0.025	12183.75
0.0016	0.030680341	0.027	11281.25
0.0017	0.028875615	0.029	10503.232759
0.0018	0.027271415	0.031	9825.6048387
0.0019	0.025836077	0.033	9230.1136364
0.002	0.024544273	0.035	8702.6785714
0.0021	0.023375498	0.037	8232.2635135
0.0022	0.022312976	0.039	7810.0961538
0.0023	0.021342846	0.041	7429.1158537
0.0024	0.020453561	0.043	7083.5755814
0.0025	0.019635419	0.045	6768.75
0.0026	0.01888021	0.047	6480.7180851
0.0027	0.018180943	0.049	6216.1989796
0.0028	0.017531624	0.051	5972.4264706
0.0029	0.016927085	0.053	5747.0518868
0.003	0.016362849	0.055	5538.0681818
0.0031	0.015835015	0.057	5343.75
0.0032	0.015340171	0.059	5162.6069322
0.0033	0.014875317	0.061	4993.3401639
0.0034	0.014437808	0.063	4834.8214286
0.0035	0.014025299	0.065	4686.0576923
0.0036	0.013635707	0.067	4546.1753731
0.0037	0.013267175	0.069	4414.4021739
0.0038	0.012918039	0.071	4290.0528169
0.0039	0.012586807	0.073	4172.5171233
0.004	0.012272137	0.075	4061.25
0.0041	0.011972816	0.077	3955.762987
0.0042	0.011687749	0.079	3855.6170886
0.0043	0.011415941	0.081	3760.4166667
0.0044	0.011156488	0.083	3669.8042169
0.0045	0.010908566	0.085	3583.4558824
0.0046	0.010671423	0.087	3501.0775862
0.0047	0.010444372	0.089	3422.4016854
0.0048	0.01022678	0.091	3347.1840659
0.0049	0.010018071	0.093	3275.2016129
0.005	0.009817709	0.095	3206.25
0.0051	0.009625205	0.097	3140.1417526
0.0052	0.009440105	0.099	3076.7045455
0.0053	0.00926199	0.101	3015.779703
0.0054	0.009090472	0.103	2957.2208738
0.0055	0.00892519	0.105	2900.8928571
0.0056	0.008765812	0.107	2846.6705607
0.0057	0.008612026	0.109	2794.4380734
0.0058	0.008463542	0.111	2744.0878378
0.0059	0.008320093	0.113	2695.5199115
0.006	0.008181424	0.115	2648.6413043
0.0061	0.008047303	0.117	2603.3653846
0.0062	0.007917507	0.119	2559.6113445
0.0063	0.007791833	0.121	2517.303719
0.0064	0.007670085	0.123	2476.3719512
0.0065	0.007552084	0.125	2436.75
0.0066	0.007437659	0.127	2398.3759843
0.0067	0.007326649	0.129	2361.1918605
0.0068	0.007218904	0.131	2325.1431298
0.0069	0.007114282	0.133	2290.1785714
0.007	0.007012649	0.135	2256.25
0.0071	0.00691388	0.137	2223.3120438
0.0072	0.006817854	0.139	2191.3219424
0.0073	0.006724458	0.141	2160.2393617
0.0074	0.006633587	0.143	2130.0262238
0.0075	0.00654514	0.145	2100.6465517
0.0076	0.006459019	0.147	2072.0663265
0.0077	0.006375136	0.149	2044.2533557
0.0078	0.006293403	0.151	2017.1771523
0.0079	0.00621374	0.153	1990.8088235
0.008	0.006136068	0.155	1965.1209677
0.0081	0.006060314	0.157	1940.0875796
0.0082	0.005986408	0.159	1915.6839623
0.0083	0.005914283	0.161	1891.886646
0.0084	0.005843875	0.163	1868.6733129
0.0085	0.005775123	0.165	1846.0227273
0.0086	0.005707971	0.167	1823.9146707
0.0087	0.005642362	0.169	1802.3298817
0.0088	0.005578244	0.171	1781.25
0.0089	0.005515567	0.173	1760.6575145
0.009	0.005454283	0.175	1740.5357143
0.0091	0.005394346	0.177	1720.8686441
0.0092	0.005335712	0.179	1701.6410615
0.0093	0.005278338	0.181	1682.8383978

for a deformation =
span/1000 (.0057),
I must be > 860000 cm4

To keep a stress
< 15 kg/mm2, I/v
must be > 20000 cm3

Best profile matching the 2 requirements (lower weight):

W 1100 x 400 x 433		433.24	kg/m'				
I =	1125573.94	cm4	deformation :	4.3612E-11	m		
I/v =	20317.22	cm3	stress:	14.991901	kg/mm2	ok with steel yield strength 24 kg/mm2	
With a HE1000B:		314.44	kg/m'				
I =	644748.07	cm4	deformation :	7.6136E-11	m		
I/v =	12894.96	cm3	stress:	23.6211473	kg/mm2	ok with steel yield strength 36 kg/mm2	

SECONDARY BEAMS

T beams coming from HE 600 A were considered

SECONDARY GIRDERS	length (m):	1.5	distance (m):	1	load (t/m):	50	M (tm):	9.375

DISPLACEMENT vs INERTIA		STRESS vs I/v	
I (m4)	f (m)	I/v (m3)	STRESS (t/m2)
1E-05	0.00313895	0.0002	46875
2E-05	0.00156948	0.00025	37500
3E-05	0.00104632	0.0003	31250
4E-05	0.00078474	0.00035	26785.71429
5E-05	0.00062779	0.0004	23437.5
6E-05	0.00052316	0.00045	20833.33333
7E-05	0.00044842	0.0005	18750
8E-05	0.00039237	0.00055	17045.45455
9E-05	0.00034877	0.0006	15625
0.0001	0.0003139	0.00065	14423.07692
0.00011	0.00028536	0.0007	13392.85714
0.00012	0.00026158	0.00075	12500

I/v should be > 600 cm3

HE600A 1/2

	b	h	S	y	S.y	bh3/12	d	S.d2
base	30	2.5	75	28.25	2118.75	39.0625	-18.0257	24369.573848
wall	1.3	27	35.1	13.5	473.85	2132.325	-3.27575	376.64181104
plate	30	4	120	-2	-240	160	12.22425	17931.875524
			230.1	10.22425033	2352.6	2331.3875		42678.091183

I =	45009.4787			
I/v1=	3164.27774	stress:	2.962761416	kg/mm2
I/v h =	2335.03129	stress:	4.014935487	kg/mm2
f (m)=	6.974E-05			

WEIGHTS:

Steel plate 40 x 7500 x 5700 mm		13.4235 t
main girder W 1100 x 400 x 433 yield point 24 kg/mm2 total length 5 x 5.7 m		12.34734
main girder HE1000B yield point 36 kg/mm2		8.96154
secondary girders 1/2 HE600A total length 5 x 7.5		4.6725
border plate	5 cm	10.362 t
suspension		6 t
axis, wheels		2.4 t
variante 1		49.20534
variante 2		45.81954

CONCLUSION:

The estimated weight by 1st calculation is 46 or 49 tons according to the beam choice (HE 1000 A or W 1100 x 400 x 300 according to the ARBED catalogue (see extract hereunder). These values are to be compared with the 51 tons found by the above statistical formula.

Catalog
ProfilARBED - Beams selection program

PROFILARBED

page 1
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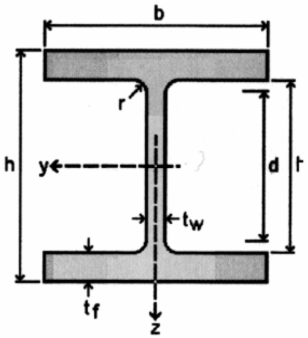


Listing with profiles according to the following rule:
I.y must be between 5000,00 cm4 and 3000000,00 cm4
W.y must be between 9000,00 cm3 and 71000,00 cm3
G ascending
Search in: IPE, IPN, HE, HL, HD, HP, HP(US), W, UB, UBP, UC, H

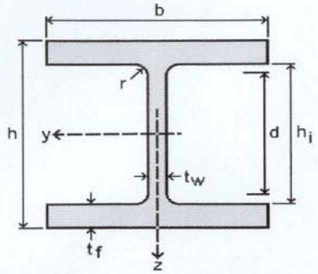
Profile						G [kg/m]	A [cm2] / A.vz [cm2]	I.y [cm4] / i.y [cm]	W.y [cm3] / W.y.pl [cm3]	I.z [cm4] / i.z [cm]	W.z [cm3] / W.z.pl [cm3]	.T [cm4]	i.T [cm] / I.omega [cm6]
W 1000 X 300 X 249	980,00	300,00	16,50	26,00	30,00	249,04	316,85	481 078,52	9 817,93	11 754,44	783,63	584,40	7,39
	928,00	*868,00*	*103,65*	*3,08*	*12,37*		*180,74*	*38,97*	*11 346,88*	*6,09*	*1 244,71*		*26 620 893*
HE 900 A	890,00	300,00	16,00	30,00	30,00	251,93	320,53	422 074,83	9 484,83	13 547,46	903,16	736,77	7,63
	830,00	*770,00*	*111,15*	*2,90*	*11,51*		*163,33*	*36,29*	*10 811,04*	*6,50*	*1 414,48*		*24 961 500*
UB 914 X 305 X 253	918,40	305,50	17,30	27,90	19,10	253,74	322,83	436 304,46	9 501,40	13 301,11	870,78	630,51	7,71
	862,60	*824,40*	*95,48*	*2,99*	*11,80*		*167,85*	*36,76*	*10 942,00*	*6,42*	*1 370,54*		*26 284 181*
W 920 X 310 X 253	919,00	306,00	17,30	27,90	19,00	254,02	323,18	437 456,16	9 520,26	13 366,25	873,61	630,91	7,72
	863,20	*825,20*	*95,36*	*2,99*	*11,80*		*167,86*	*36,79*	*10 962,73*	*6,43*	*1 374,80*		*26 449 053*
W 920 X 310 X 271	923,00	307,00	18,40	30,00	19,00	272,03	346,09	471 573,42	10 218,28	14 518,01	945,80	775,02	7,76
	863,00	*825,00*	*100,66*	*3,00*	*11,06*		*178,81*	*36,91*	*11 782,87*	*6,48*	*1 490,95*		*28 842 178*
W 1000 X 300 X 272	990,00	300,00	16,50	31,00	30,00	272,62	346,85	553 846,02	11 188,81	14 004,44	933,63	822,41	7,55
	928,00	*868,00*	*113,65*	*3,10*	*11,37*		*184,56*	*39,96*	*12 824,38*	*6,35*	*1 469,71*		*32 073 875*
HE 1000 A	990,00	300,00	16,50	31,00	30,00	272,62	346,85	553 846,02	11 188,81	14 004,44	933,63	822,41	7,55
	928,00	*868,00*	*113,65*	*3,10*	*11,37*		*184,56*	*39,96*	*12 824,38*	*6,35*	*1 469,71*		*32 073 875*
H 900 X 300 X 18 X 34	912,00	302,00	18,00	34,00	18,00	283,01	360,06	491 010,69	10 767,78	15 654,09	1 036,69	980,82	7,77
	844,00	*808,00*	*107,09*	*2,97*	*10,49*		*173,06*	*36,93*	*12 337,07*	*6,59*	*1 622,45*		*30 079 980*

ARBED GROUP

Catalog
ProfilARBED - Beams selection program

PROFILARBED

page 1
06/09/02 15:02:39



Listing with profiles according to the following rule:
I.y must be between 1125000,00 cm4 and 3000000,00 cm4
W.y must be between 10000,00 cm3 and 100000,00 cm3
G ascending
Search in: IPE, IPN, HE, HL, HD, HP, HP(US), W, UB, UBP, UC, H

Profile	h [mm] / *h.i [mm]*	b [mm] / *d [mm]*	t.w [mm] / *S.s [mm]*	t.f [mm] / *A.L [m2/m]*	r [mm] / *A.G [m2/t]*	G [kg/m]	A [cm2] / *A.vz [cm2]*	I.y [cm4] / *i.y [cm]*	W.y [cm3] / *W.y.pl [cm3]*	I.z [cm4] / *i.z [cm]*	W.z [cm3] / *W.z.pl [cm3]*	I.T [cm4]	i.T [cm] / *I.omega [cm6]*
W 1100 X 400 X 433	1 108,00	402,00	22,00	40,00	20,00	433,24	551,19	1 125 573,94	20 317,22	43 409,79	2 159,69	2 129,54	10,40
	1 028,00	*988,00*	*125,43*	*3,75*	*8,66*		*254,39*	*45,19*	*23 160,71*	*8,87*	*3 361,78*		*123 500 699*
HL 1100 M	1 108,00	402,00	22,00	40,00	20,00	433,24	551,19	1 125 573,94	20 317,22	43 409,79	2 159,69	2 129,54	10,40
	1 028,00	*988,00*	*125,43*	*3,75*	*8,66*		*254,39*	*45,19*	*23 160,71*	*8,87*	*3 361,78*		*123 500 699*
W 1100 X 400 X 499	1 118,00	405,00	26,00	45,00	20,00	499,28	635,21	1 294 059,56	23 149,54	49 984,12	2 468,35	3 134,95	10,45
	1 028,00	*988,00*	*139,43*	*3,77*	*7,56*		*300,41*	*45,14*	*26 599,48*	*8,87*	*3 870,29*		*143 405 493*
HL 1100 R	1 118,00	405,00	26,00	45,00	20,00	499,28	635,21	1 294 059,56	23 149,54	49 984,12	2 468,35	3 134,95	10,45
	1 028,00	*988,00*	*139,43*	*3,77*	*7,56*		*300,41*	*45,14*	*26 599,48*	*8,87*	*3 870,29*		*143 405 493*
W 1000 X 400 X 539	1 030,00	407,00	28,40	51,10	30,00	540,12	687,17	1 202 537,90	23 350,25	57 631,92	2 832,04	4 546,45	10,60
	927,80	*867,80*	*165,75*	*3,58*	*6,64*		*316,39*	*41,83*	*26 823,86*	*9,16*	*4 435,56*		*137 552 834*
HL 1000 X 554	1 032,00	408,00	29,50	52,00	30,00	554,76	705,81	1 232 371,55	23 883,17	59 098,19	2 896,97	4 859,98	10,61
	928,00	*868,00*	*168,65*	*3,59*	*6,47*		*328,03*	*41,79*	*27 496,21*	*9,15*	*4 546,53*		*141 326 871*
HE 1000 X 579	1 056,00	316,00	35,00	64,00	30,00	579,29	737,01	1 245 718,26	23 593,15	34 037,38	2 154,26	7 102,05	8,06
	928,00	*868,00*	*198,15*	*3,25*	*5,63*		*393,33*	*41,11*	*27 950,86*	*6,80*	*3 498,29*		*82 804 383*
W 1000 X 300 X 584	1 056,00	314,00	36,00	64,00	30,00	584,57	743,73	1 246 071,34	23 599,84	33 433,46	2 129,52	7 230,02	7,98
	928,00	*868,00*	*199,15*	*3,24*	*5,56*		*403,25*	*40,93*	*28 039,18*	*6,70*	*3 474,83*		*81 242 078*

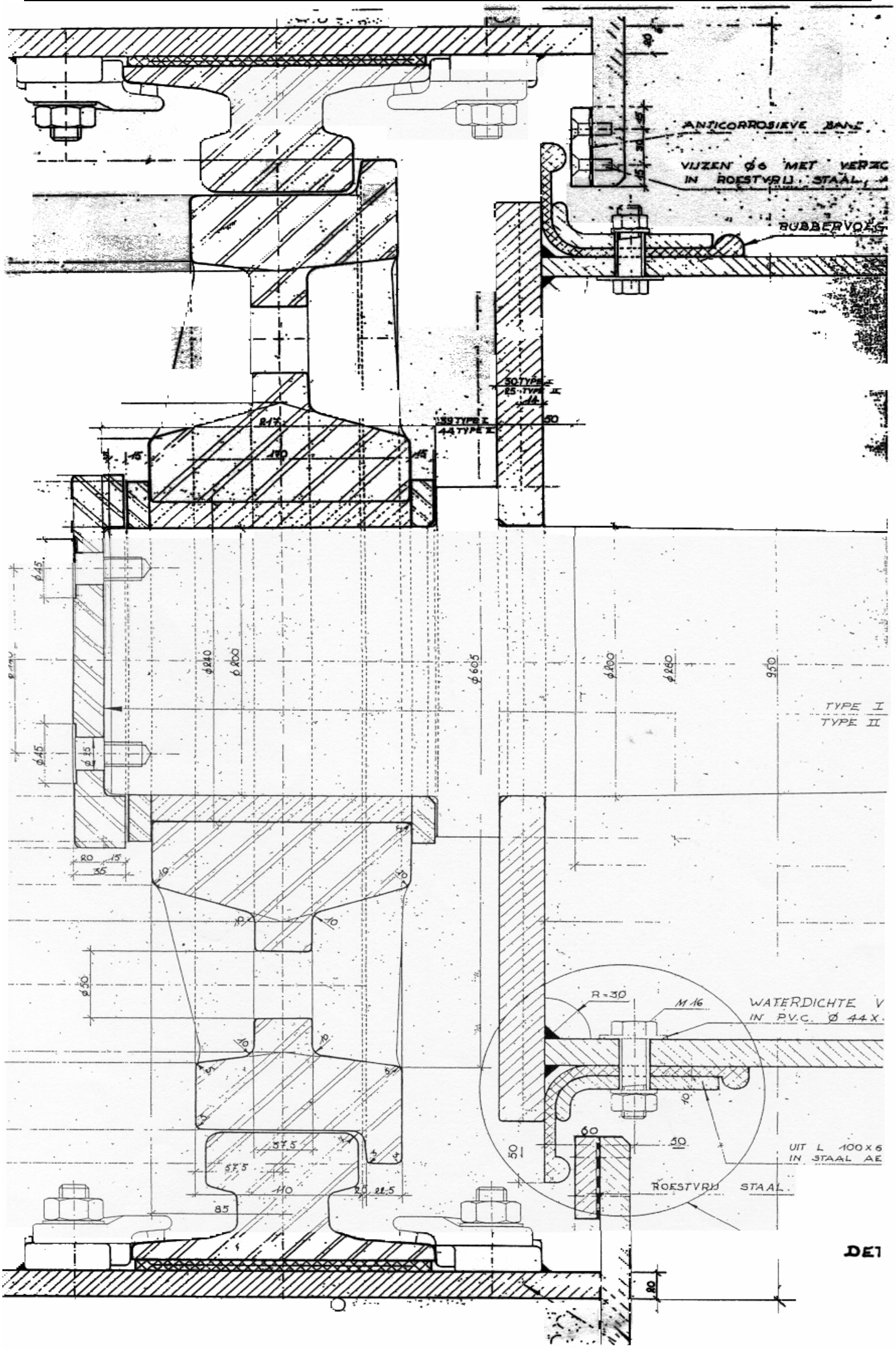
ARBED GROUP

ANNEX 3

ESTIMATION OF WEIGHT FOR CULVERT AND CONDUIT GATES TAKING INTO ACCOUNT MAXIMUM STATIC HEADS
ATLANTIC SIDE : TRIPLE LIFT (W = 55m) 3 x 3 WATER SAVING BASINS

			Width(m)	height(m)	Hmwc(m)	Hsécurité	Htot	T/m	L tot(m)	Estimated weight (T)	n	Total weight (T)
CULVERT GATES	Culvert gates		**4.5**	**6**	**37.03**					**25.5**	16	407
	Culvert gates slots	2*2gate height		12				0.8	24	19.2	16	
		2*[Htot-(2gate height)]		12	37.03	1.5	38.53	0.2	53.06	10.6	16	
		2*width	4.5					0.8	9	7.2	16	
		tot culvert gates slots										592
	Culvert bulkhead	equal to culvert gate - 3T								22.5	8	180
	Culvert bulkhead slots	2*2bulkhead height		12				0.5	24	12	16	
		2*[Htot-(2bulkhead height)]		12	37.03	1.5	38.53	0.2	53.06	10.6	16	
		2*width	4.5					0.5	9	4.5	16	
		tot culvert bulkhead slots										434
CONDUIT GATES	Conduit gates		**4**	**5**	**42.52**					**20.9**	36	754
	Conduit gates slots	2*2gate height		10				1	20	20	36	
		2*[Htot-(2gate height)]		10	42.52	1.5	44.02	0.2	68.04	13.6	36	
		2*width	4					1	8	8.0	36	
		tot conduit gates slots										1498
	Conduit bulkhead	equal to conduit gate - 3T								17.9	6	107
	Conduit bulkhead slots	2*2bulkhead height		10				0.5	20	10	36	
		2*[Htot-(2bulkhead height)]		10	42.52	1.5	44.02	0.2	68.04	13.6	36	
		2*width	4					0.5	8	4.0	36	
		tot conduit bulkhead slots										994

ANNEX 4 – CROSS SECTION OF A GATE WHEEL OF BERENDRECHT LOCK



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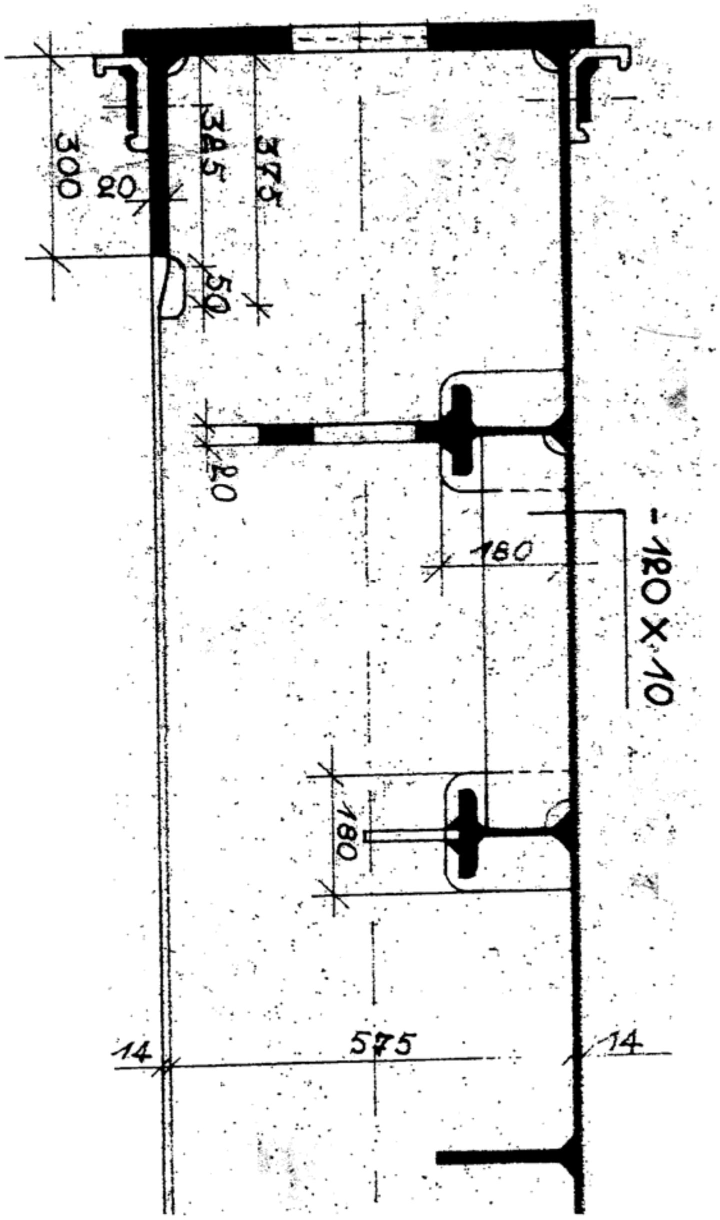
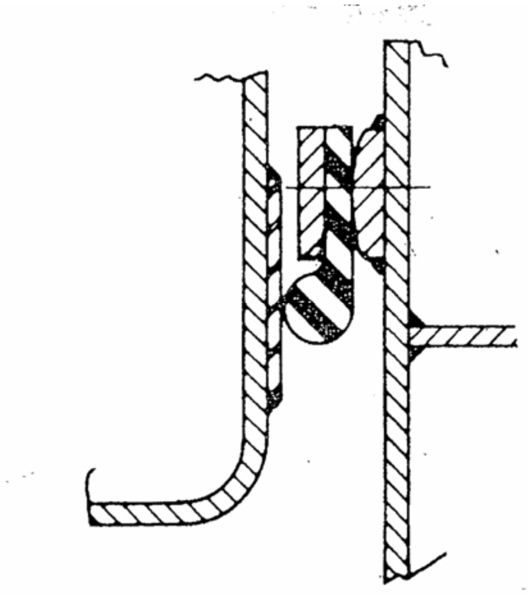
ANNEX 5

TYPICAL GATE STRUCTURE

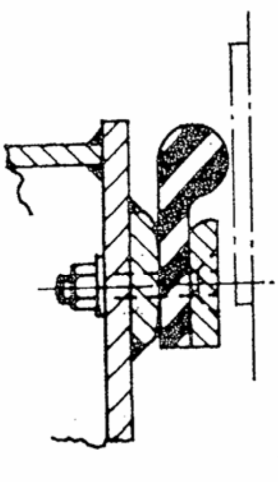


ANNEX 6

UPSTREAM AND DOWNSTREAM SEALING (Music not J-shape type)



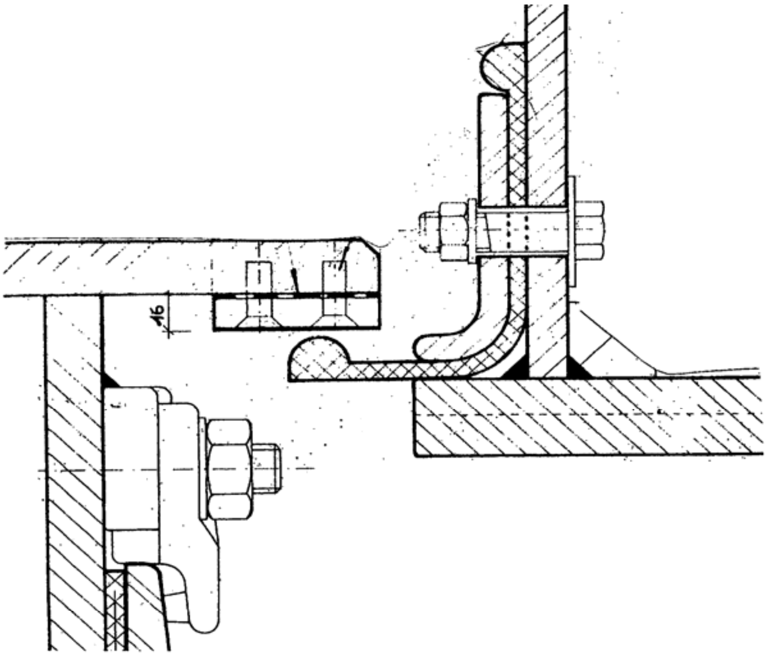
UPSTREAM SEALING

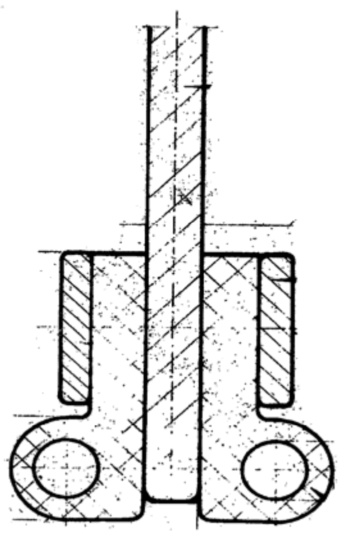


DOWNSTREAM SEALING

ANNEX 7

SIDE AND BOTTOM SEALS (BERENDRECHT)





ANNEX 8 : Pictures – typical seals view (Zandvliet lock, Belgium)

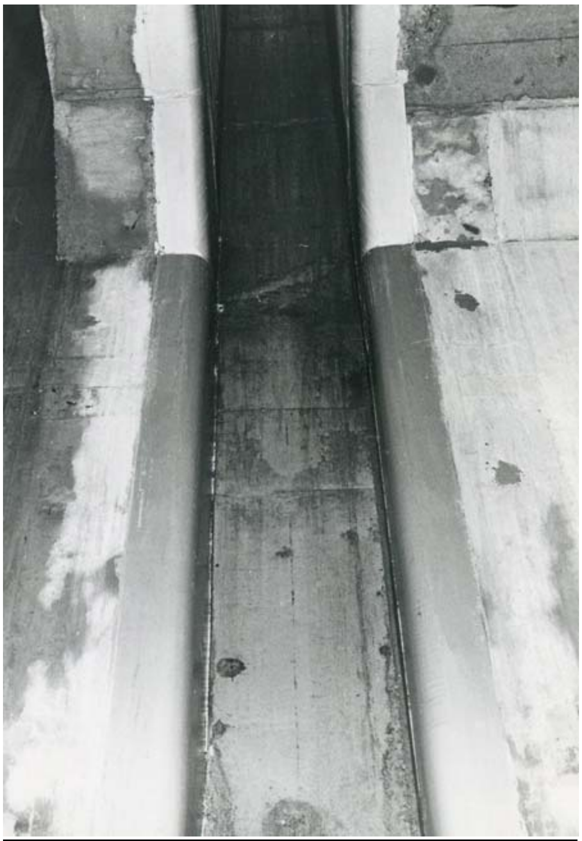
Side seal left position (angular music note type)

Front seal (simple music note seal)

Side seal right position (angular music note type)

Bended music note seal – Pressing plate and protecting device

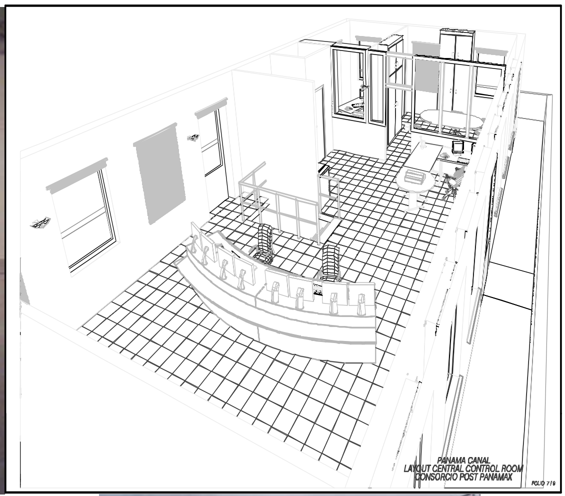
Double bottom seals



Detail of a gate slot

Handling device details

General view of culvert gate



Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual design - Contract SAA-143351

ATLANTIC LOCKS HARMONIZATION

Task A4f – OPERATING MACHINERY
Task A4g – LIGHTING
Task A4h – ELECTRICAL AND POWER REQUIREMENTS
Task A4j – OPERATING STRUCTURES
Rev B

in association with

TABLE OF CONTENTS

Annexes

1. Estimation of gate engine power for culvert and conduit gates taking into account operating heads
2. Estimation of gate engine power for culvert and conduit gates taking into account maximum static heads

1 INTRODUCTION

The Atlantic Locks Harmonization conceptual design study is based entirely on the Actualization of the Pacific Locks conceptual design.

The present document gives the harmonization of the conceptual design of the following subjects:

- the gates and valves operating machinery (Task 4 F). This corresponds to the operating machinery of the main lock gates and of the culvert and conduit gates,
- the control system architecture (including SCADA[1]), which includes the monitoring of the whole lock system as well as the monitoring of the pumping station (also included in Task 4 F),
- the lighting system (Task 4 G),
- the electrical and power requirements (Task 4 H),
- the operating structures (Task 4 J), which deals with the arrangement of the various technical buildings[2].

***

[1] ***SCADA = System Control And Data Acquisition***
[2] ***Electrical rooms, Maintenance building, Rolling gates technical rooms, WSB technical building, Culvert technical building, Emergency Diesel Room and (Main) Control room***

2 Operating machinery (Task A4f)

2.1 MACHINERY OF THE (MAIN) ROLLING GATES

Reference is made to Task P4f (Pacific Actualization).

2.2 MACHINERY OF THE CULVERT AND WSB CONDUIT GATES

The calculation of the rated output of the motor of the main oil pumps mounted on the hydraulic power pack is enclosed in Annexes 1 and 2, respectively for operating and maximum static heads. This calculation takes into account the actual dimensions of the culvert and WSB conduit gates (see report A4e).

A summary of the output for different options is given hereafter:

55 m (operating heads)	culvert gates:	53kW,
	WSB conduit gates:	15kW.

(see annex 1 – Estimate of the gate engine power taking into account operating heads)

55 m (maximum static heads)	culvert gates:	77kW,
	WSB conduit gates:	60kW.

(see annex 2 – Estimate of the gate engine power taking into account maximum static heads)

Regarding the two last values, standardization of the servomotors is possible if we consider the operation under maximum static heads.
But another alternative could be envisaged in the next step of the studies:

Regarding the design of the motors (two per gate):

- for the operating heads, one motor will operate the gate, one will remain on stand-by (one redundancy degree).
- operation under maximum static heads should be with the two motors in operation (no redundancy).

Of course the power output of the motors will have to be slightly adapted to fit the above operation procedures.

2.3 CONTROL SYSTEM ARCHITECTURE

Reference is made to Task P4f (Pacific Actualization).
For the drawings, reference is made to the reports R4-F from the 2002 original conceptual design study and its drawings.

3 Lighting (Task A4g)

3.1 OUTSIDE LIGHTING

3.1.1 LOCK CHAMBER WALLS LIGHTING

Reference is made to Task P4g (Pacific Actualization).

3.1.2 LIGHTING POLES

The location of the lighting poles is slightly easier without the locomotive tracks.
The philosophy of the lighting is to have a lighting level along the lock chamber (both side) and decreasing lighting level after the fictive line running along the dead end of the main rolling gates recesses.

The length of the entrance walls is shorter than for the Pacific locks. The number of lighting poles is 48 instead of 61.

Side WSB – Gatun lake entrance :
- 2 lighting poles.
- 60m between two LP
- 6 floodlights of 1000 W

Side WSB – Chamber locks :
- 3 x 5 lighting poles.
- 93m between two LP
- 12 floodlights of 1000 W

Side WSB – Atlantic Entrance :
- 3 lighting poles.
- 60m between two LP
- 6 floodlights of 1000 W

Other side :

- 28 lighting poles
- 59m between two LP
- 6 floodlights of 1000 W

The number of lighting poles has decreased as compared to the Pacific side.
The estimated budget price for 48 high masts, 378 floodlights, lock chamber and gallery lighting amounts: USD 2 million.

Reference is made to the report R4-G from the 2002 original conceptual design study. For the layout, reference is made to drawing D4-A-203.

3.2 INTERNAL LIGHTING

Reference is made to Task P4g (Pacific Actualization).

4 Electrical and power requirements (Task A4h)

Reference is made to Task P4h (Pacific Actualization).
For the general layout, reference is made to drawing D4-A-203.
The arrangement of the electrical substations is shown on drawing D4-H-206.

5 Operating structures (Task A4j)

Reference is made to Task P4j (Pacific Actualization).

6 References

- CCP (2002). Diseño conceptual de las Esclusas Post Panamax. Triple Lift Lock System, task 4.
- PACIFIC LOCKS ACTUALIZATION, Tasks P4f, P4g, P4h and P4j.

ESTIMATION OF THE GATE ENGINE POWER
ATLANTIC SIDE : TRIPLE LIFT (W=55m) 3 X 3 WATER SAVING BASINS
TAKING INTO ACCOUNT OPERATING HEADS

	LOCK CULVERT GATE	WSB CONDUIT GATE
Maximum effort (T)	96	33
Oil pressure (bar)	200	200
Stroke (m)	6,00	5,00
Opening time (min)	2,00	2,00
Cylinder section (m²)	0,048	0,017
Cylinder oil volume (m³)	0,288	0,084
Oil flow (m³/min)	0,144	0,042
mechanical efficiency	0,9	0,9
POWER (kW)	**53**	**15**

Calculus of the forces on the gate

		LOCK	WSB
	Gate width (m)	4,5	4
	length of horizontal seal (m)	4,9	4,4
	Gate heigth (m)	6	5
	length of vertical seal (m)	6,3	5,3
	width of seal (cm)	3	3

OPENING

		LOCK	WSB
Sealing friction forces Fs			
Fs = f x 1,5 x p x A	f (friction coefficient)	0,15	0,15
	p (hydraulic pressure on the gate) (bar)	2,5	1,0
	A (Area of sealing contact) (m²)	0,525	0,45
	Fs (kg)	2953	1013
Wheel friction Fw			
Fw = Q x (fd x d + fr) / D	Q (max load on the gate) (kg)	771750	233200
(six wheels have been foreseen)	fd (friction coeff of the wheel bushings)	0,12	0,12
	fr (friction coeff of wheels rolling on slot rails)	0,2	0,2
	d (diameter of wheel shaft) (cm)	20	20
	D (wheel diameter) (cm)	80	80
	Fw (kg)	25082	7579
Hydraulic load F1 on the top seal of the gate			
F1 = p x l x ls	p (hydraulic pressure on the gate) (bar)	2,5	1,0
	l (width of the seal) (m)	0,08	0,08
	ls (length of the seal) (m)	4,9	4,4
	F1 (kg)	9800	3520
Hydraulic load F2 on the top of the gate			
F2 = p x gt x ls	p (hydraulic pressure on the gate) (bar)	2,5	1,0
	gt (gate thickness) (m)	1	1
	ls (length of the seal) (m)	4,9	4,4
	F2 (kg)	122500	44000
Hydraulic load F3 under the gate			
F3 = F2 x dlc	F2 (kg)	122500	44000
	dlc (dynamic load coefficient)	0,8	0,8
	F3 (kg)	98000	35200
Weight W (under water)			
W = rw x 6.85/7.85 x 1.05	rw (real weight) (kg)	19345	7603
	W (weight under water) (kg)	17725	6966
Maximum opening load			
F = Fs + Fw + F1 + F2 - F3 + W	**F (T)**	**80**	**28**

CLOSING

		LOCK	WSB
Sealing friction forces F's			
F's = 0.1 x p x A	p (hydraulic pressure on the gate) (bar)	2,5	1,0
	A (Area of sealing contact) (m²)	0,525	0,45
	Fs (kg)	1313	450
Wheel friction F'w			
F'w = Q x (f'd x d + f'r) / D	Q (max load on the gate) (kg)	771750	233200
	f'd (friction coeff of the wheel bushings)	0,08	0,08
	f'r (friction coeff of wheels rolling on slot rails)	0,1	0,1
	d (diameter of wheel shaft) (cm)	20	20
	D (wheel diameter) (cm)	80	80
	F'w (kg)	16400	4956
Hydraulic load F'1 on the top seal of the gate			
F'1 = 0.5 x F1	F1 (kg)	9800	3520
	F'1 (kg)	4900	1760
Hydraulic load F'2 on the top of the gate			
F'2 = 0.9 x F2	F2 (kg)	122500	44000
	F'2 (kg)	110250	39600
Hydraulic load F'3 under the gate			
F'3 = 0.5 x F3	F3 (kg)	98000	35200
	F'3 (kg)	49000	17600
Weight W'			
real weight of the gate	W' (kg)	19345	7603
Maximum braking force			
B = W' + F'1 + F'2 - F'3 - F'w - F's	**B (T)**	**68**	**26**

ESTIMATION OF THE GATE ENGINE POWER
ATLANTIC SIDE : TRIPLE LIFT (W=55m) 3 X 3 WATER SAVING BASINS
TAKING INTO ACCOUNT MAXIMUM STATIC HEADS

	LOCK CULVERT GATE	WSB CONDUIT GATE
Maximum effort (T)	139	130
Oil pressure (bar)	200	200
Stroke (m)	6,00	5,00
Opening time (min)	2,00	2,00
Cylinder section (m²)	0,069	0,065
Cylinder oil volume (m³)	0,416	0,324
Oil flow (m³/min)	0,208	0,162
mechanical efficiency	0,9	0,9
POWER (kW)	**77**	**60**

Calculus of the forces on the gate

		LOCK CULVERT GATE	WSB CONDUIT GATE
	Gate width (m)	4,5	4
	length of horizontal seal (m)	4,9	4,4
	Gate heigth (m)	6	5
	length of vertical seal (m)	6,3	5,3
	width of seal (cm)	3	3

OPENING

		LOCK CULVERT GATE	WSB CONDUIT GATE
Sealing friction forces Fs			
Fs = f x 1,5 x p x A	f (friction coefficient)	0,15	0,15
	p (hydraulic pressure on the gate) (bar)	3,7	4,3
	A (Area of sealing contact) (m²)	0,525	0,45
	Fs (kg)	4374	4305
Wheel friction Fw			
Fw = Q x (fd x d + fr) / D	Q (max load on the gate) (kg)	1143116	991566
(six wheels have been foreseen)	fd (friction coeff of the wheel bushings)	0,12	0,12
	fr (friction coeff of wheels rolling on slot rails)	0,2	0,2
	d (diameter of wheel shaft) (cm)	20	20
	D (wheel diameter) (cm)	80	80
	Fw (kg)	37151	32226
Hydraulic load F1 on the top seal of the gate			
F1 = p x l x ls	p (hydraulic pressure on the gate) (bar)	3,7	4,3
	l (width of the seal) (m)	0,08	0,08
	ls (length of the seal) (m)	4,9	4,4
	F1 (kg)	14516	14967,04
Hydraulic load F2 on the top of the gate			
F2 = p x gt x ls	p (hydraulic pressure on the gate) (bar)	3,7	4,3
	gt (gate thickness) (m)	1	1
	ls (length of the seal) (m)	4,9	4,4
	F2 (kg)	181447	187088
Hydraulic load F3 under the gate			
F3 = F2 x dlc	F2 (kg)	181447	187088
	dlc (dynamic load coefficient)	0,8	0,8
	F3 (kg)	145158	149670,4
Weight W (under water)			
W = rw x 6.85/7.85 x 1.05	rw (real weight) (kg)	25468	20940
	W (weight under water) (kg)	23335	19186
Maximum opening load			
F = Fs + Fw + F1 + F2 - F3 + W	**F (T)**	**116**	**108**

CLOSING

		LOCK CULVERT GATE	WSB CONDUIT GATE
Sealing friction forces F's			
F's = 0.1 x p x A	p (hydraulic pressure on the gate) (bar)	3,7	4,3
	A (Area of sealing contact) (m²)	0,525	0,45
	Fs (kg)	1944	1913
Wheel friction F'w			
F'w = Q x (f'd x d + f'r) / D	Q (max load on the gate) (kg)	1143116	991566,4
	f'd (friction coeff of the wheel bushings)	0,08	0,08
	f'r (friction coeff of wheels rolling on slot rails)	0,1	0,1
	d (diameter of wheel shaft) (cm)	20	20
	D (wheel diameter) (cm)	80	80
	F'w (kg)	24291	21071
Hydraulic load F'1 on the top seal of the gate			
F'1 = 0.5 x F1	F1 (kg)	14516	14967,04
	F'1 (kg)	7258	7483,52
Hydraulic load F'2 on the top of the gate			
F'2 = 0.9 x F2	F2 (kg)	181447	187088
	F'2 (kg)	163302	168379,2
Hydraulic load F'3 under the gate			
F'3 = 0.5 x F3	F3 (kg)	145158	149670,4
	F'3 (kg)	72579	74835,2
Weight W'			
real weight of the gate	W' (kg)	25468	20940
Maximum braking force			
B = W' + F'1 + F'2 - F'3 - F'w - F's	**B (T)**	**97**	**99**

Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual design - Contract SAA-143351

ATLANTIC LOCKS ACTUALIZATION

TASK A4i – ENTRANCE WALLS

Rev A

Tractebel Development Engineering

TECHNUM

COYNE ET BELLIER

Compagnie Nationale du Rhône

in association with

SBE

PORT OF ANTWERP

FIPA AOSO

1 General considerations

The entrance walls to the lock complex are the transitional part between the wider canal and/or Lake (trapezoidal section) and the lock (rectangular section). They are also required to guide the ship when entering the lock and, in case of emergency, for mooring of a ship.

The proposed layout of the entrance walls is shown on drawing D4-I-201.

At the Atlantic entrance, it is strongly recommended to provide a quay wall on the east side of the canal, which can be used to moor vessels whenever there is a problem entering the locks. Such situation can be caused by a failure of the lock gate operation, a problem with tug boat assistance, a vessel engine problem, unexpected weather conditions, etc… However, due to the rather poor soil conditions on this side of the lock (Atlantic Muck), it would be very difficult and costly to construct a quay wall structure along the entire entrance. Therefore it is proposed to reduce the quay wall length to a minimum, still seated in sound rock of the Gatun formation, and to install a row of flexible dolphins for the part of the entrance situated in Atlantic Muck. Like this, mooring of the ships in case of emergency will be possible against a row of flexible dolphins at regular spacing.

At the Lake entrance to the locks, it is in our opinion not really necessary to provide a long entrance wall for emergency mooring along the east side. This is illustrated in drawing D3-0-200. There is sufficient space in the Lake area to keep vessels waiting, if necessary by anchoring.

The west side forms the future center wall between third and fourth lane, and will only be a guiding wall for vessels entering the lock. If necessary, the access to the third locks complex can be separated from the fourth lane by means of heavy dolphins placed in the future center line of the two locks.

2 Design criteria

Design criteria for the new lock structures have been given in the report of Task 2 - "Design Criteria" of the Pacific Locks Actualization and the Atlantic Locks Harmonization study.

3 Technical description of entrance structure

Below, a detailed description is given of the type of structures adopted for the entrance to the lock complex. Reference is made to the following drawings:

- D4-B-201 Longitudinal view on the left bank lock wall
- D4-B-202 Longitudinal view on the right bank lock wall
- D4-B-203 Plan view lock chamber 1
- D4-B-205 Plan view lock chamber 3
- D4-I-201 Cross section of entrance walls

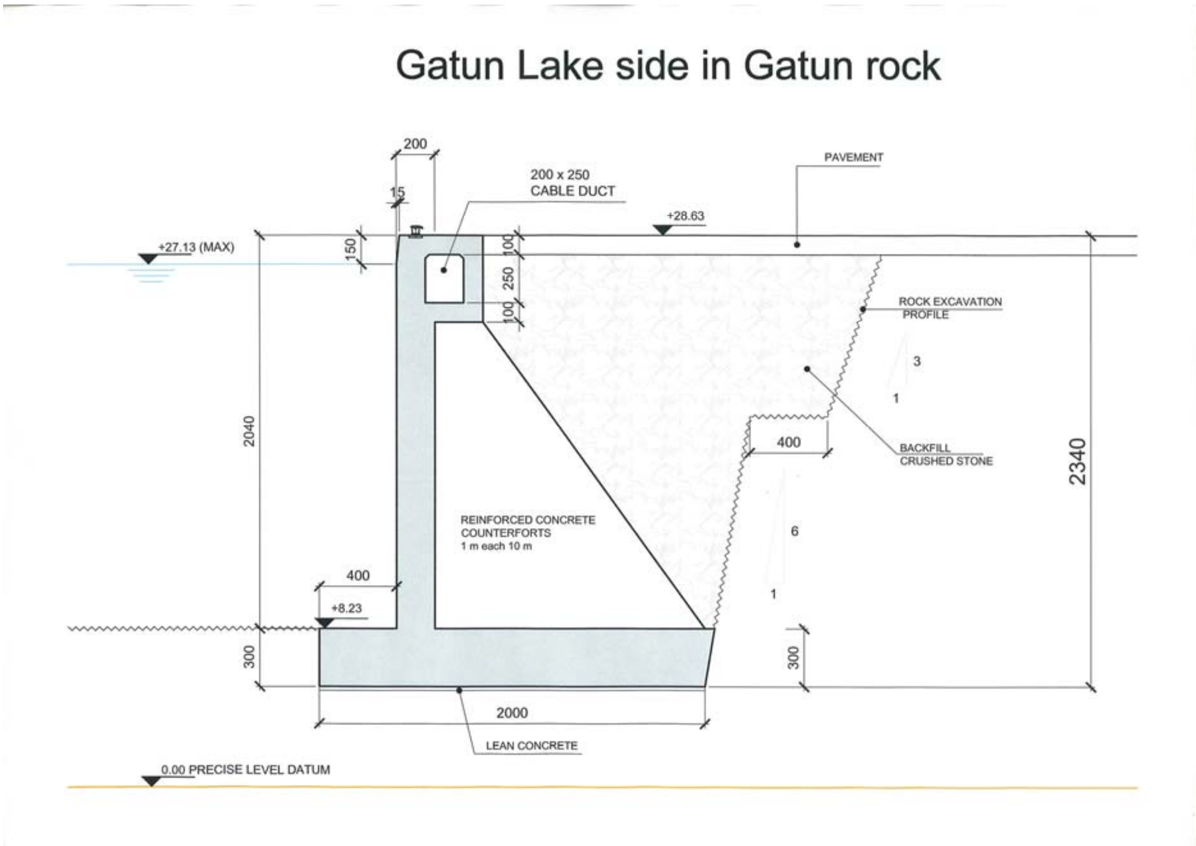
3.1 TYPE 1 – FOUNDATION IN GATUN ROCK

As already mentioned, the entrance walls will be situated in the "Gatun" rock formation. Although the characteristics of this formation can be considered as fairly good corresponding to a rock mass type IV, with UCS-values ranging from 2-20 MPa, they are considerably lower than those of the basalt formation (RMT I-II, UCS-values 40-100 MPa) which is found at the Pacific side of the Canal.

Furthermore, it is also clear that the deformation behavior of the "Gatun" formation will be different than the one of the basalt formation under vertical loading. (Deformation modules ranging from 1–1,3 GPa in Gatun rock, compared to 8-12,5 GPa in basalt).

As the lock walls are counterfort retaining walls due to merely geotechnical considerations, it is considered logical to retain the same wall type for the entrance walls. From the practical point of view this is also considered as an advantage, as the contractor will be using the same techniques and methods as for the main construction work.

The main difference with the lock walls is the fact that there is no longitudinal culvert integrated in the structure. Furthermore, the entrance walls are exposed to less severe loadings than the lock walls because the water levels are not so much fluctuating as in the lock chambers.



At the top surface of the wall, additional structural concrete is required to install a cable duct and other quay side equipment, including bollards, ladders and lighting recesses.

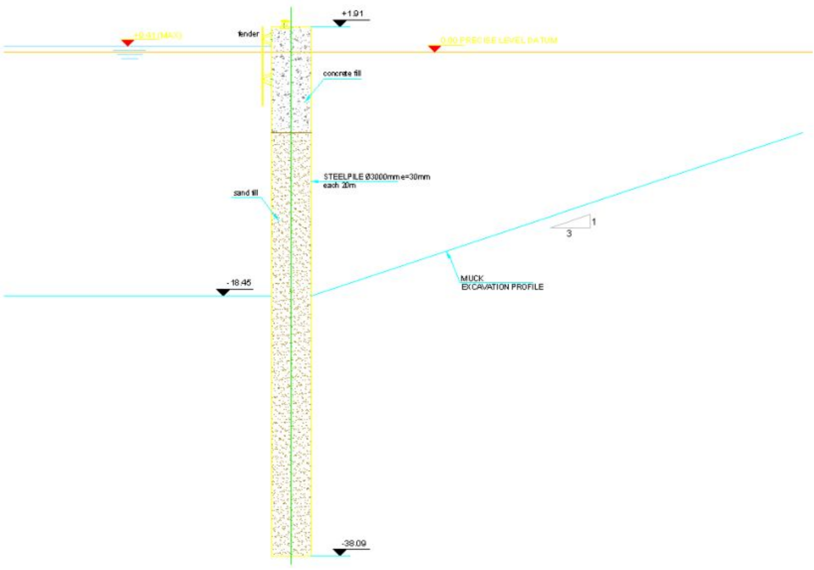
Re-use of excavated material, fragmented to the proper size, for backfilling purposes, will lead to economical benefit. Besides, the high frictional properties of the broken rock allow for reduction of the horizontal pressure, exerted by the fill on the vertical retaining wall, leading to structural optimization.

The width at the foundation level has been determined in function of required safety against overturning; sliding effects being negligible due to the embedment in solid rock.

3.2 TYPE 2 – FOUNDATION IN ATLANTIC MUCK

Flexible dolphins are tubular high quality steel piles, driven in the soft soil by means of powerful driving equipment and from a jack-up platform equipped with a heavy crane.

Atlantic entrance in muck



4 Type 1 – Foundation in Gatun rock

4.1 ANALYSIS

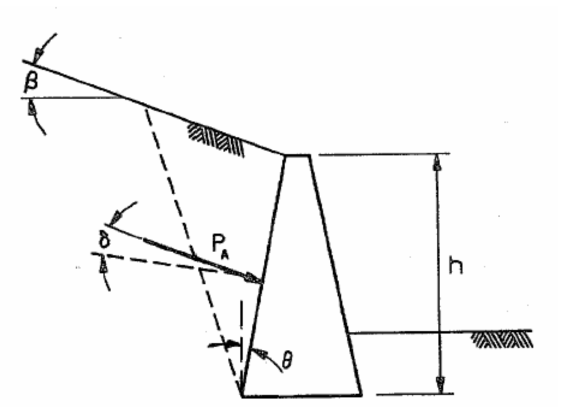
4.1.1 LOADS

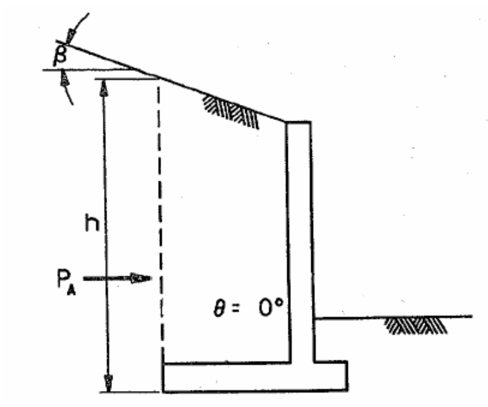
A. Self Weight (LC 1)

Concrete	γ= 25.0 kN/m³
Wet backfill	γ= 20.0 kN/m³
Dry backfill	γ= 18.0 kN/m³

B. Earth Pressure (LC 2)

As the counterfort retaining walls nears geometrically to a cantilever wall type, the active lateral pressures will be calculated for a Rankine situation:





$$K_a = \frac{\cos^2(\varphi'-\theta)}{\cos^2\theta\cos(\theta+\delta)\left[1+\sqrt{\dfrac{\sin(\varphi'+\delta)\sin(\varphi'-\beta)}{\cos(\delta+\theta)\cos(\beta-\theta)}}\right]^2}$$

with $\delta = 0°$; $\beta = 0°$ and $\theta = 0°$
The angle of friction in the filling of crushed stone is 45°

C. Water Pressure (LC 3)

Inside the lock, the minimum water level will be applied.
At the backside of the wall the maximum water level of the lock chamber will be applied.

D. Vertical live load

On the surface a vertical live load p=10 kN/m² is applied, this load case is included in normal earth pressure load case (LC2). In seismic conditions, only half of the live load is taken into account (5 kN/m²) and is included in the seismic earth pressures load case (LC6).

E. Bollards (LC 4)

EAU1996 (Recommendations of the Committee for Waterfront Structures, Harbours & Waterways) gives recommendations for layout and loading of bollards for seagoing vessels (sections 5.12 – p. 143). Accordingly, bollards of 1500 kN working load have been retained with a spacing of 30 m. The wall is divided into segments of about 30m, so the total horizontal load applied to one meter of quay is 50 kN/m'.

F. Seismic Loads (LC 5, 6, 7 and 8)

F.1 Earthquake level

ACP stated in its Memorandum of 20 Jan 2005 on 'Seismic Design Criteria' that an earthquake with a return period of 5000 years should be considered as Maximum Credible Earthquake (= MCE) and taken equal to the Maximum Design Earthquake (= MDE).

F.2 Performance Grade

The highest performance grade (Grade S) is applicable for

- critical structures with potential for extensive loss of human life and property
- key structures that are required to be serviceable for recovery from earthquake disaster
- critical structures that, if disrupted, devastate economic and social activities

A catastrophic failure of the locks may cause flooding in the terminal cities of the canal.
This means that the level of damages is:

- Structural: minor or no damage
- Little or no loss of serviceability

for a MCE-event.

F.3 Analysis type

As this report concerns a conceptual design, a simplified analysis will be used considering equivalent static forces to apply the seismic loads on the structure.

F.4 Representative ground acceleration values

Based on the review of probabilistic seismic hazard analysis by Winter (2005), the following PGA-values can be taken as representative for the site at Miraflores:

Return period [years]	Representative PGA	Level	Load case
100	0.06		
500	0.15		
1,000	**0.22**		
2,500	0.32		
5,000	**0.41**	MDE =MCE	LC6
10,000	0.51		

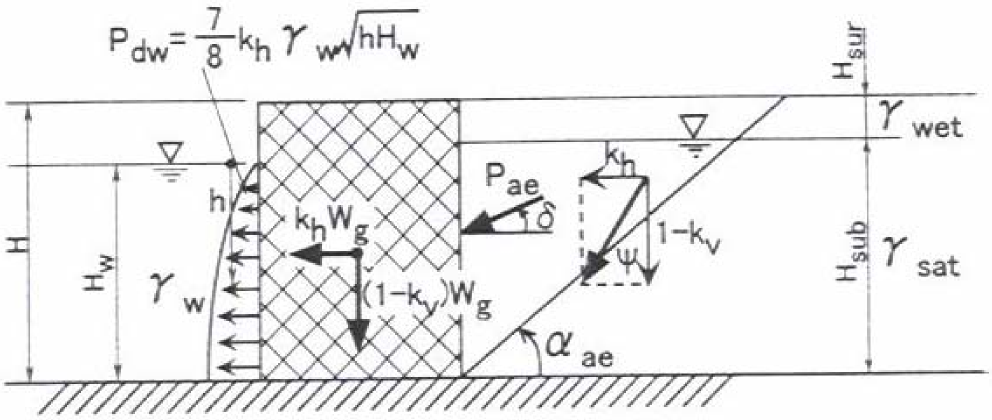
The seismic coefficient k_e for use in retaining structures is defined as follows for Special Class Structures:

$$k_e = PGA/g \quad \text{for } PGA < 0.2\ g$$

$$= \frac{1}{3}\left(\frac{PGA}{g}\right)^{\frac{1}{3}} \quad \text{for } PGA \geq 0.2\ g$$

With according to Eurocode design (CEN 1994)

$k_{h\ (horizontal)} = k_e$
$k_{v(vertical)} = ½k_e$



(a)

The seismic loads are calculated separately for earth pressure (LC 6), equivalent horizontal (LC 7) and vertical (LC 8) reaction forces on structural masses and water bodies (LC 5).

4.1.2 LOAD COMBINATIONS

According to ROSA 2000

A. Quasi Permanent Load Combination

QP1 1*LC1 + 1*LC2 + 1*LC3 + 1*LC4

B. Fundamental Load Combinations

F1 1.20*LC1 + 1.20*LC2 + 1*LC3 + 1.20*LC4
F2 0.90*LC1 + 1.20*LC2 + 1*LC3 + 1.20*LC4

C. Accidental Load Combination - MCE

MCE 1*LC1 + 1*LC3 + 0.25*LC5 + 1*LC6b + 0.25*LC7 + 0.12*LC8

4.1.3 CRITERIA

According to ROSA 2000

A. Sliding along the base

The factor of safety against sliding is given by the relation between the sum of the horizontal resisting forces and the sum of the horizontal driving forces:

$$FS_{(sliding)} = \frac{\sum F_{R'}}{\sum F_d} = \frac{\sum V \tan\delta + Q_{p,h}}{\sum H}$$

In which:
δ = the angle of friction between the soil and the base slab (=2/3φ')
Adhesion between the soil and the base slab is neglected

According to ROSA2000, the value of the safety factor the accidental load combination, MCE, is taken equal to 1.1. ΣV and ΣH are then given by:

$\Sigma V = (1-k_v)*G - P_{w,upl} - Q_{p,v}$

$\Sigma H = k_h*G + P_{w,h} + P_{w,sesm} + Q_{a,h}$

B. Turn over

Load Combination	Criterion
QP1	Compression zone A' > 90% A
F1 to F2	Compression zone A' > 10% A
MCE	Compression zone A' > 10% A

C. Load Bearing Capacity

$\gamma_d \cdot q_{ref} < q_u$
γ_d . = 1.4 for load combination F1 to F2
γ_d . = 1.0 for load combination MCE

q_u = ultimate load bearing capacity
q_{ref} = reference pressure

D. Deformations

The deformations will be checked for frequent, rare and accidental load combinations QP1 and MCE.

E. Concrete Stresses

Fundamental load combinations: F1 tot F2: 1.125 x $\sigma_c < 0.85\ f_{ck}/1.5$
Accidental load combination: MCE: $\sigma_c < 0.85\ f_{ck}$

The factor 1.125 is a result of combination of the different factors in the load combinations between ROSA 2000 and Eurocode

F. Global Stability

Will be checked for the wall in La Boca formation only.

4.1.4 SOFTWARE

The calculations are made by means of “Esa-PrimaWin 3.60.381”, a finite element program. The gravity wall is modeled as a 2D WALL structure, allowing forces acting in its plane.

A non-linear analysis is performed to allow for compression only supports.

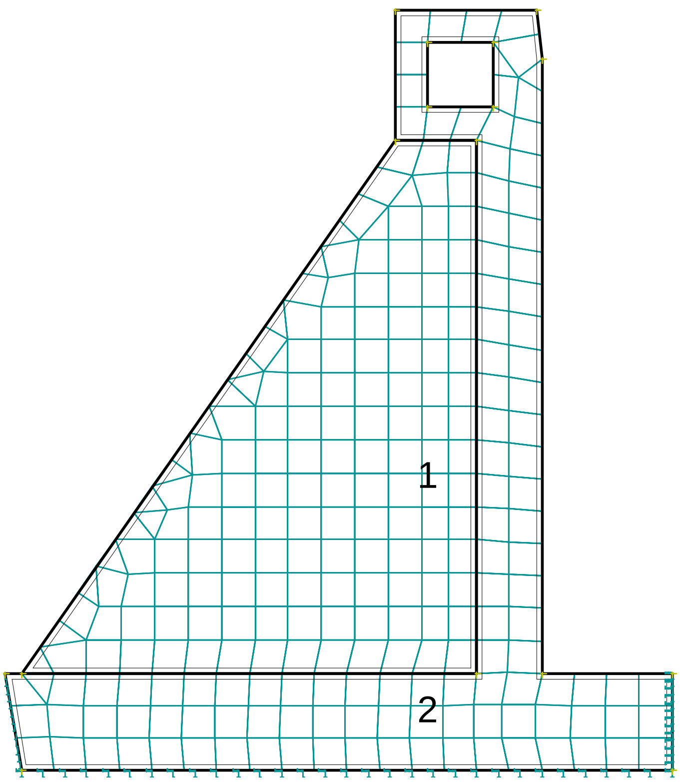
4.2 RESULTS

4.2.1 INPUT DATA

A. Geometry

The analysis below is performed for the entrance wall at Gatun Lake side, corresponding to the highest retaining height and the highest water level difference of respectively:

- $\Delta H = (+28.63m) - (+5.23m) = 23.4m$
- $\Delta WL = (+27.13m) - (+24.99m) = 2.14m$



The FE model consists of the following three 2D macro structural elements:

No.	Element	Thickness
1	Reinforced concrete counterforts	0.1 m (each 10 m)
2	Retaining wall	1 m

B. Materials

Name:		
C25/30-cracked		
	E modulus	20,000.00 MPa
	Poisson coefficient.	0.20
	Density	2,500.00 kg/m^3
	Expansion coefficient	0.01 mm/m.K

For the material corresponding to the reinforced concrete counterforts, the submerged weight of 15 kN/m^3 is taken into account, as the water pressure of LC3 are immediately applied to the main wall element.

C. Supports

The rock base is modeled by means of linear supports.

- Vertical to the base of the gravity wall and perpendicular to the boundary of the back toe, the bedding on the Gatun rock formation is characterized by a linear support of, K= 100 MN/m/m’, working in compression only.
- Parallel to the base, sliding is restricted through a frictional resistance of 30% of the elasticity of the bedding.

4.2.2 LOADS

A. Self Weight (LC 1)

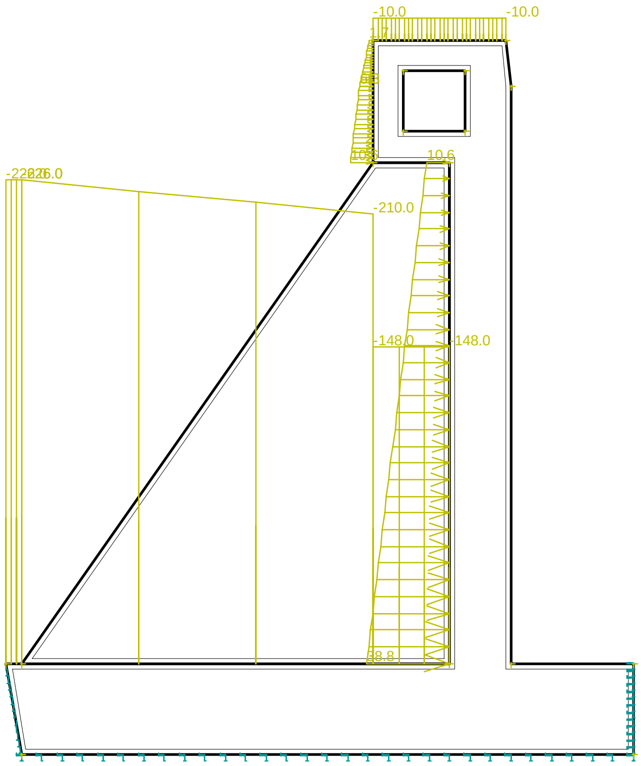
The total weight of the structure: 302.4 ton/m'.

B. Rock fill Pressure (LC 2)

from	28.63		27.13	
			λav =	λah =
λa =	0.1716		0.0000	0.1716
ϕ =	45	0.79	δ =	0
θ =	0	0.00		
γ =	18			
z	p		**pv**	**ph**
28.63	10		0.000	1.716
27.13	37		0.000	6.348

from	27.13	to	24.63	
			λav =	λah =
λa =	0.1716		0.0000	0.1716
ϕ =	45	0.79	δ =	0
θ =	0	0.00		
γ =	10			
z	p		**pv**	**ph**
27.13	37		0.000	6.348
24.63	62		0.000	10.638

from	24.63	to	8.23	
			λav =	λah =
λa =	0.1716		0.0000	0.1716
ϕ =	45	0.79	δ =	0
θ =	0	0.00		
γ =	10			
z	p		**pv**	**ph**
24.63	62		0.000	10.638
8.23	226		0.000	38.775

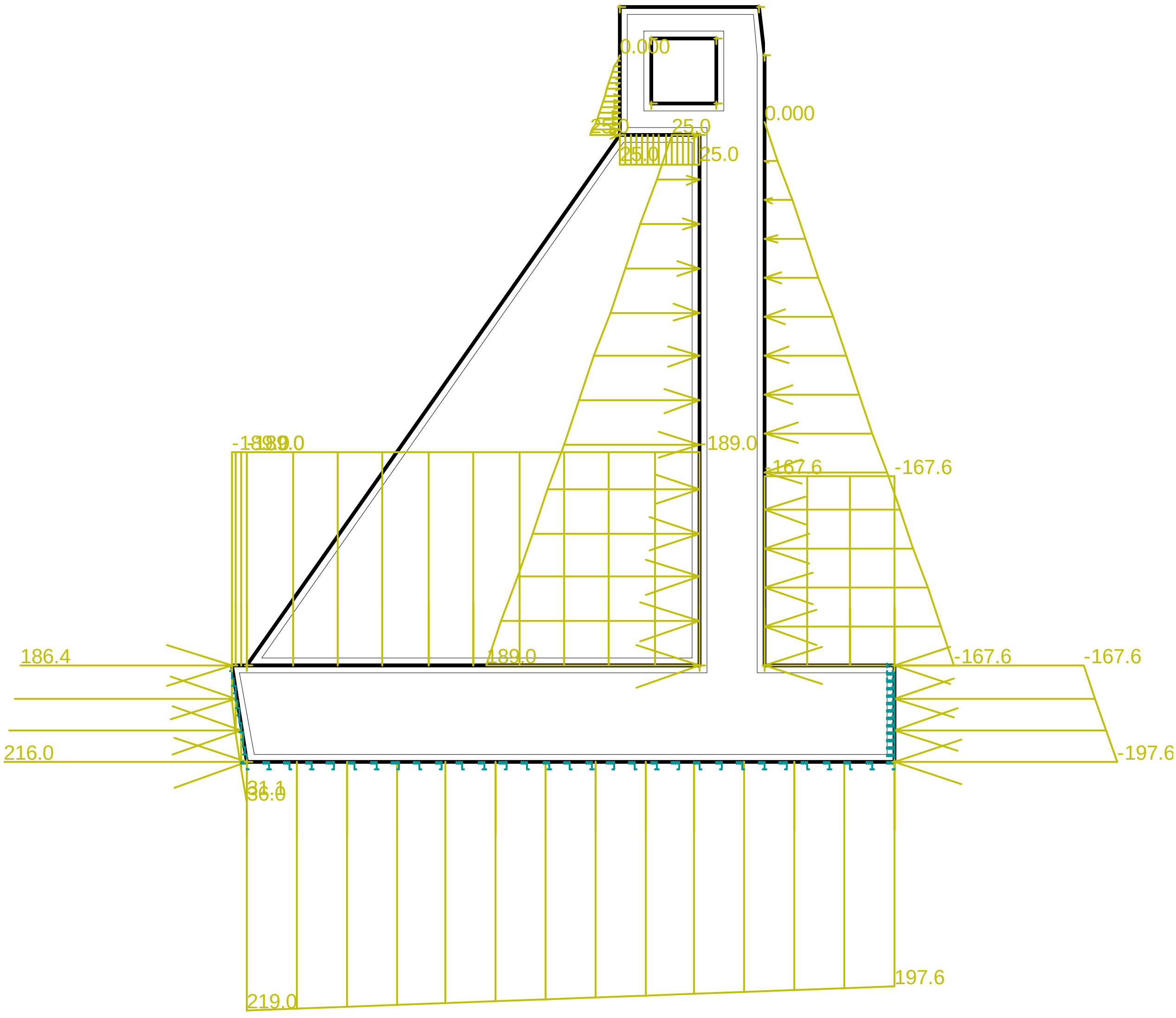


Soil weight on culvert block

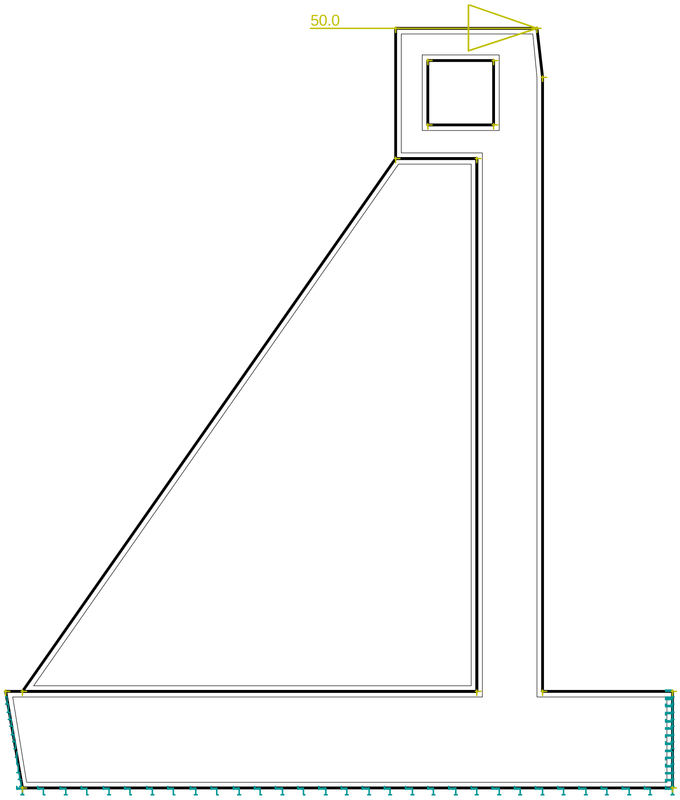
X	pv
-3	226
0	226
11.5	210
11.5	148
14	148

C. Water Pressure (LC 3)

Water level inside the lock: +24.99 m
Water level outside the lock: +27.13 m



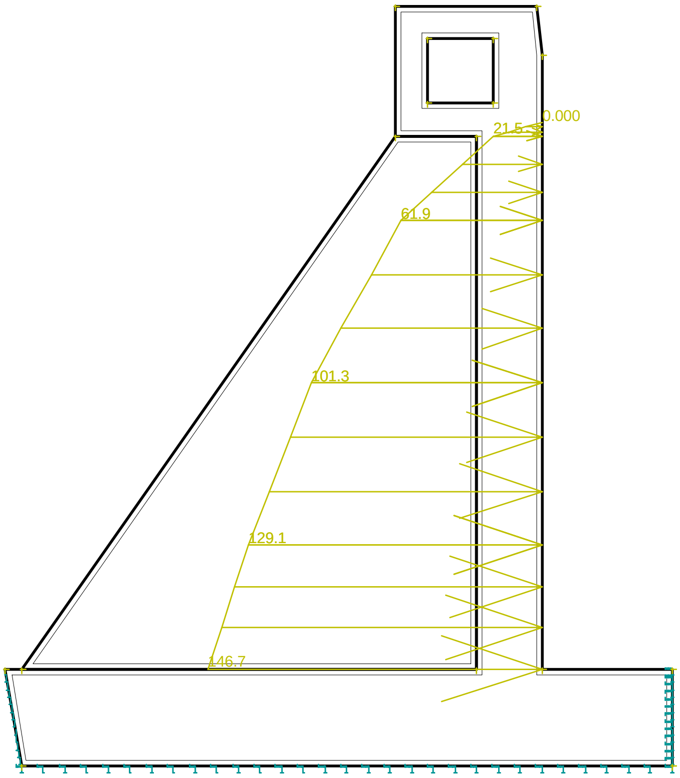
D. Bollard pull (LC 4)



E. Water Pressure - Seismics (LC 5)

Water level inside the lock: +24.99 m
Water level outside the lock: +27.13 m

Water suction forces at the outside face of the wall according to Westergaard :



The seismic water pressure load case is valid for a unit value of k_e, and is multiplied by the respective k_e value in the corresponding load combinations.

F. Seismic Loads (LC 6, 7 and 8)

LC 6

The active lateral earth thrust under seismic conditions is calculated based on the Mononobe-Okabe (M-O) methodology. The M-O method is an extension of Coulomb's theory, wherein the M-O method takes into account the inertial forces acting on the soil mass during earthquake loading. It was developed to assess the stability of massive gravity walls, assuming that the retaining wall and the failure wedge act as rigid bodies.

The inertia forces are then accounted for by considering a seismic inertia angle, Ψ = atan ($k_h/(1-k_v)$), in which k_h represents the horizontal seismic coefficient or the modified horizontal seismic coefficient for dry and submerged layers respectively.

As the counterfort retaining wall under consideration is geometrically near to cantilever wall type, a Rankine situation is assumed for the calculation of the seismic active earth pressures on the vertical through the rear edge of structure. Consequently, the weight and inertia forces of the soil masses above the structure's rear base have to be taken into consideration.

Remark:

- The active lateral pressures and the horizontal inertia components of the soil masses are transferred to the front wall elements for ease of implementation.
- Half of the vertical live load is taken is accounted for in the seismic earth pressures.

LC 7 and 8

The inertia forces on the structural weight are calculated as follows:

$G_v = (1-k_v)\ G$

$G_h = k_h\ G$

With G the weight of the filling

G_v the vertical component of the weight in seismic conditions

G_h the horizontal component of weight in seismic conditions

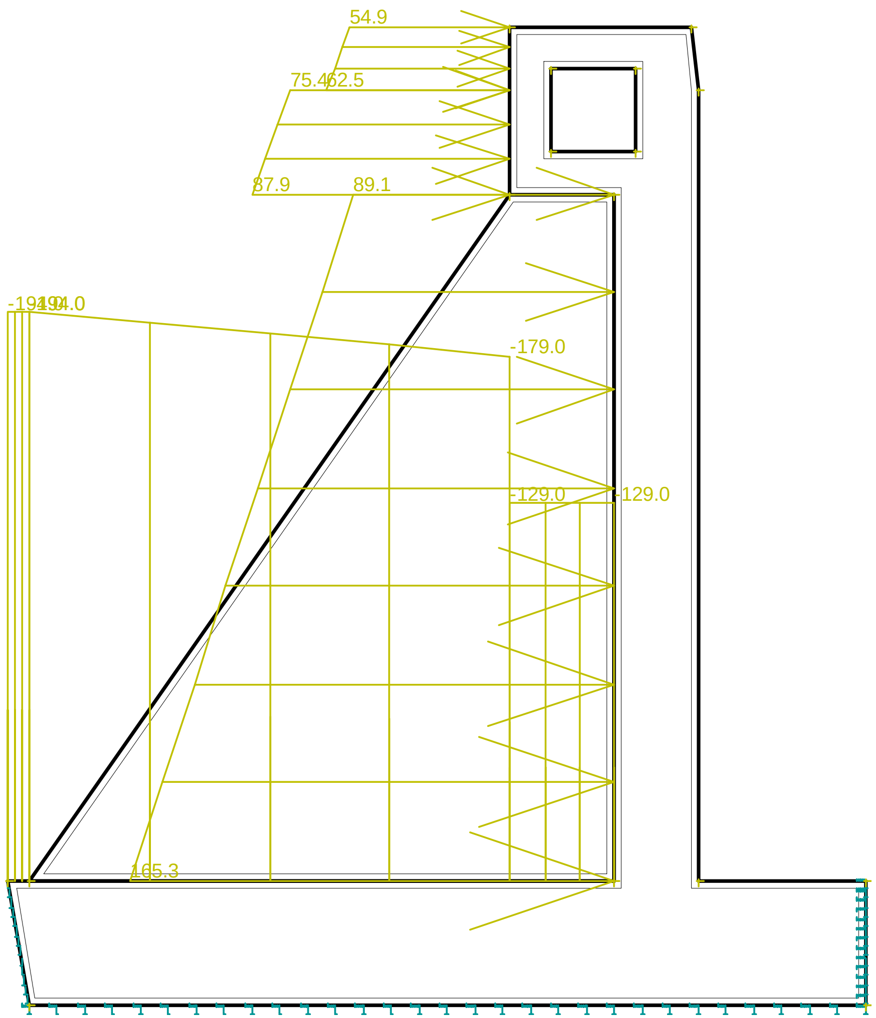
PGA	0.41	g	k_e	0.248					CFR	0.1
k_h	0.248		k_v	0.124					Width	16.5

from	28.63		27.13		1.50				Width =	12
			λav =	λah =		k_h'	0.248		b =	4.5
λa =	0.3218		0.0000	0.3218						
ϕ =	45	0.79	δ =	0	0.00	ψ	15.8	0.28		
θ =	0	0.00								
γ =	18								**ph2**	53.489
z	p		**pv**	**ph1**		**Q1v**	**0.00**			**ph**
28.63	4.381		0.000	1.410		**Q1h**	**7.82**			54.898
27.13	28.038		0.000	9.021		**Q1**	**7.82**			62.510

from	27.13	to	24.63		2.50				Width =	12
			λav =	λah =		k_h'	0.495		b =	4.5
λa =	0.5698		0.0000	0.5698						
ϕ =	45	0.79	δ =	0	0.00	ψ	29.5	0.51		
θ =	0	0.00								
γ =	10								**ph2**	59.432
z	p		**pv**	**ph1**		**Q2v**	**0.00**			**ph**
27.13	28.038		0.000	15.976		**Q2h**	**55.54**			75.408
24.63	49.942		0.000	28.458		**Q2**	**55.54**			87.889

from	24.63	to	8.23		16.40				Width =	14.5
			λav =	λah =		k_h'	0.495		b =	2
λa =	0.5698		0.0000	0.5698					b'=	16
ϕ =	45	0.79	δ =	0	0.00	ψ	29.5	0.51		
θ =	0	0.00								
γ =	10								**ph2**	
z	p		**pv**	**ph1**		**Q2v**	**0.00**			**ph**
24.63	49.942		0.000	28.458		**Q2h**	**1138.11**			89.128
8.23	193.637		0.000	110.336		**Q2**	**1138.11**			165.311

1201.48

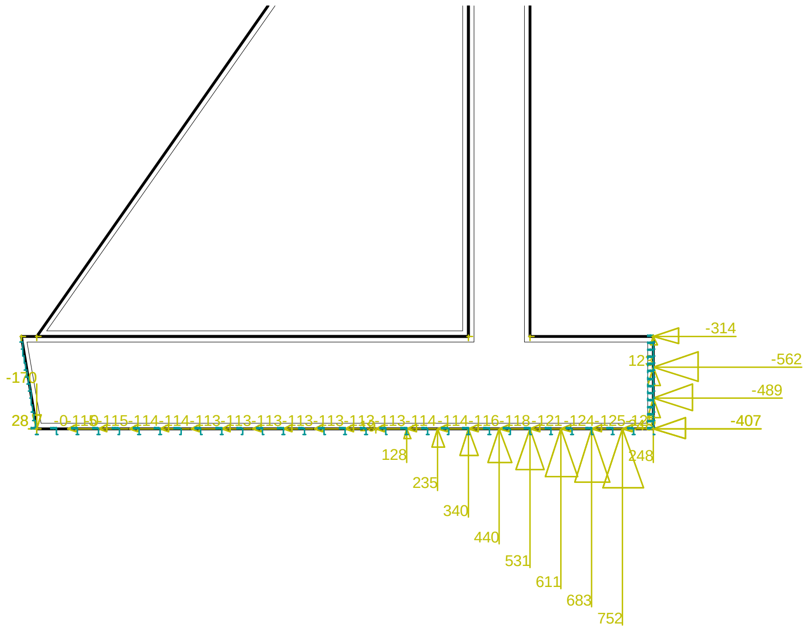


Soil weight on culvert block

X	**pv**
-0.5	194
0	194
11.5	179
11.5	129
14	129

4.2.3 SLIDING

A. Accidental Load Combination – Earth Quake MCE



Analytically, it is found that :

$\Sigma V = 4{,}843$ kN

$\Sigma H = 4{,}686$ kN

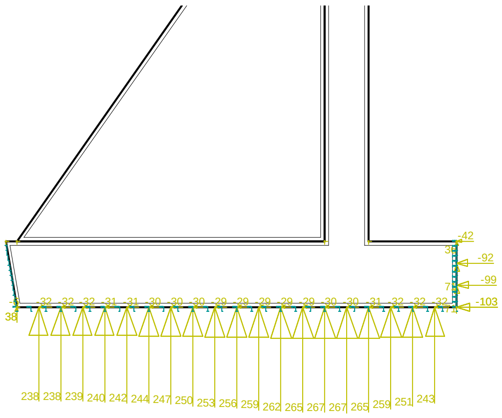
To obtain the required safety (γ_d = 1.1) against sliding, a passive lateral pressure of 2,358 kN has to be taken on the side face of the inner toe over a height of 3 m. This results in a pressure of 0.79 MPa, which is lower than the uni-axial compressive strength of the “Gatun” formation and concrete. The resistance to sliding is thus fulfilled.

Rock strength of Gatun:
Uni-axial Compressive Strength: >2 MPa
It can be concluded that the sliding criterion is fulfilled.

4.2.4 TURN OVER

A. Quasi Permanent Load Combination (QP1) – Lock in service

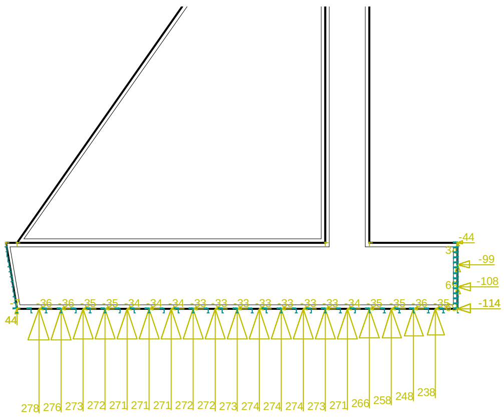
Criterion: Compression zone A'(= 100 %) > 90% A



The magnitude of the vectors shown represent the support reaction in each node and not the soil pressure.

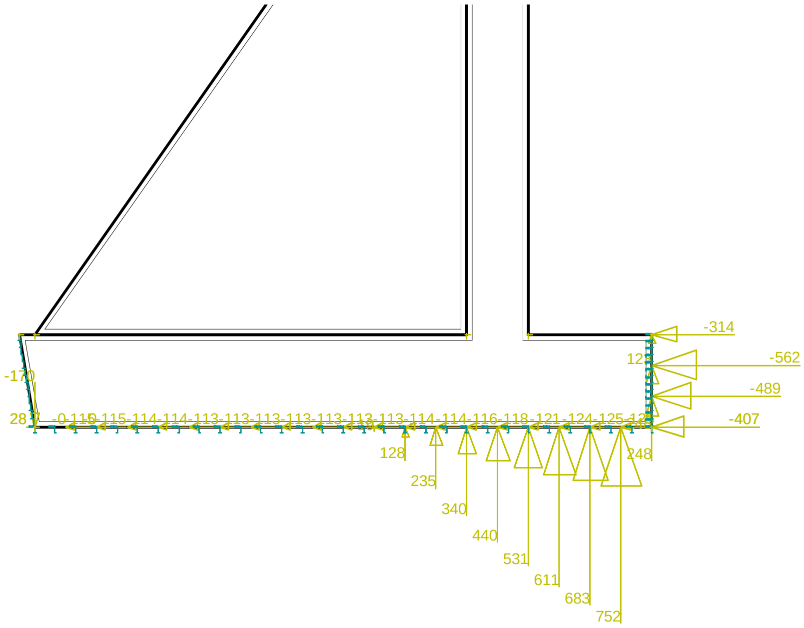
B. Fundamental Load Combinations F1 to F2

Criterion: Compression zone A' (= 100 %) > 10% A



C. Accidental Load Combination – Earth Quake MCE

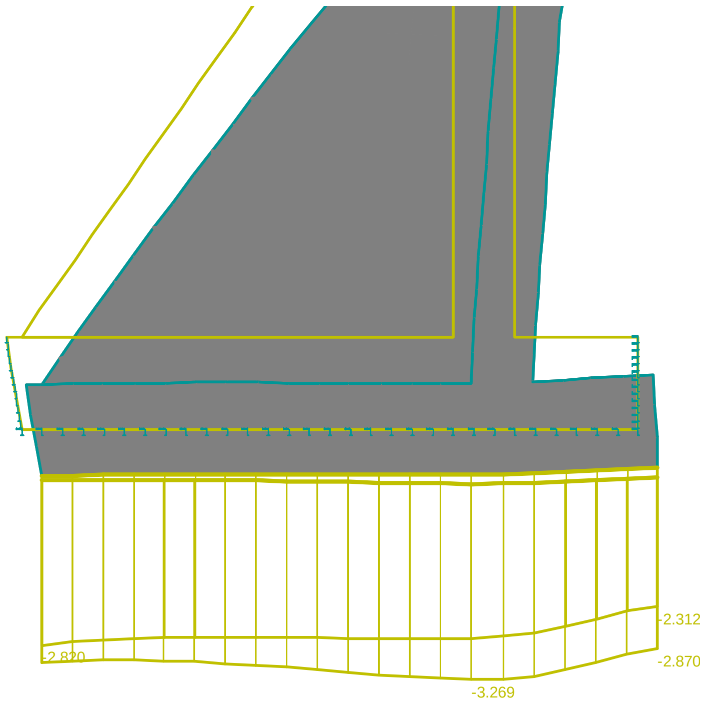
Criterion: Compression zone A’ (= 40 %)> 10% A



4.2.5 LOAD BEARING CAPACITY

A. Fundamental Load Combinations F1 to F2

Criterion: $\gamma_d \,.\, q_{ref} < q_u$
with $\gamma_d \,. = 1.4$



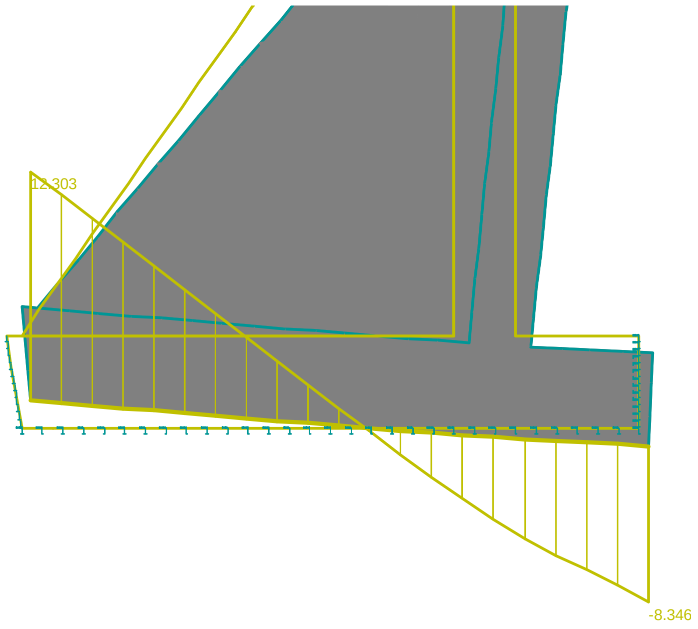
The maximum vertical displacement at the bottom of the wall is 3.269 mm, which is negligible.
This corresponds to a maximum vertical pressure of 0.0033 x 100,000 kN/m² = 327 kN/m² = 0.33 Mpa.
Consequently:

$$\gamma_d \,.\, q_{ref} = 1.4*0.327 = 0.46 < q_u = 2\text{MPa}$$

(According to report R2-A the Uni-axial Compressive Strength of the Gatun rock is 2 MPa).

B. Accidental load combination MCE

Criterion: $\gamma_d \cdot q_{ref} < q_u$
with γ_d . = 1.0 for accidental load combinations



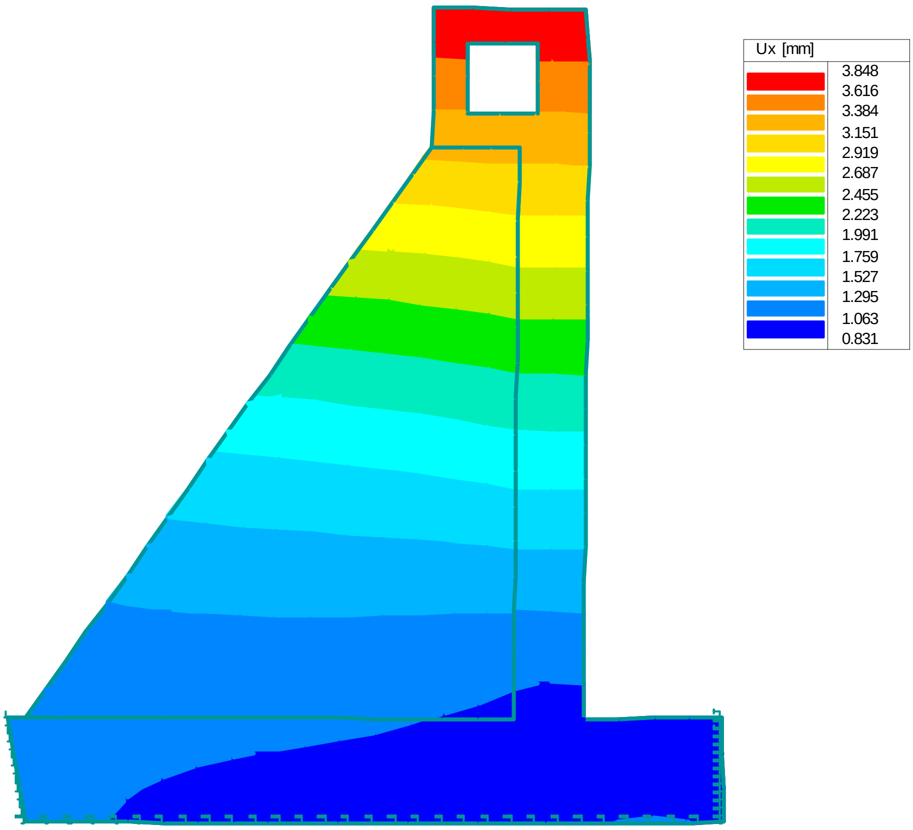
The maximal vertical displacement at the bottom of the wall is 8.346 mm
This means a vertical pressure of 0.0083 x 100,000 kN/m² = 835 kN/m² = 0. 83 MPa

(According to report R2-A the Uni-axial Compressive Strength of the Gatun rock is 2 MPa).

4.2.6 DEFORMATIONS

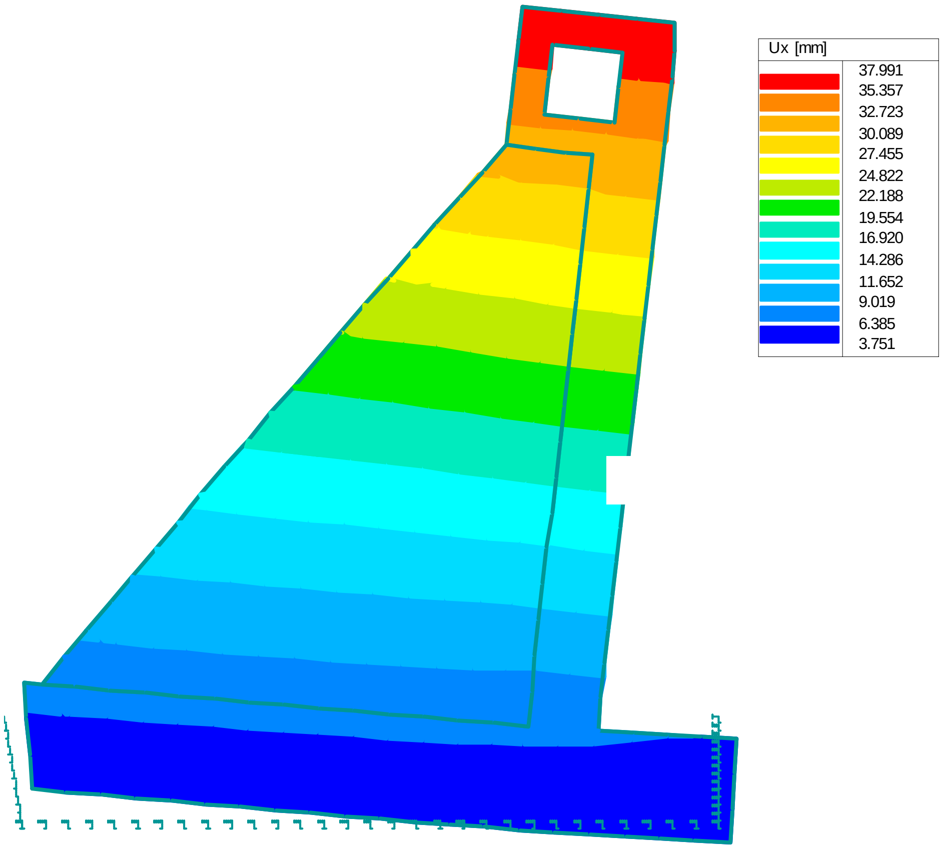
A. Load Combination QP1 (Lock in Service)

Maximal horizontal displacement, $u_{x,max}$ = 3.8 mm



B. Load Combination MCE (Earth Quake)

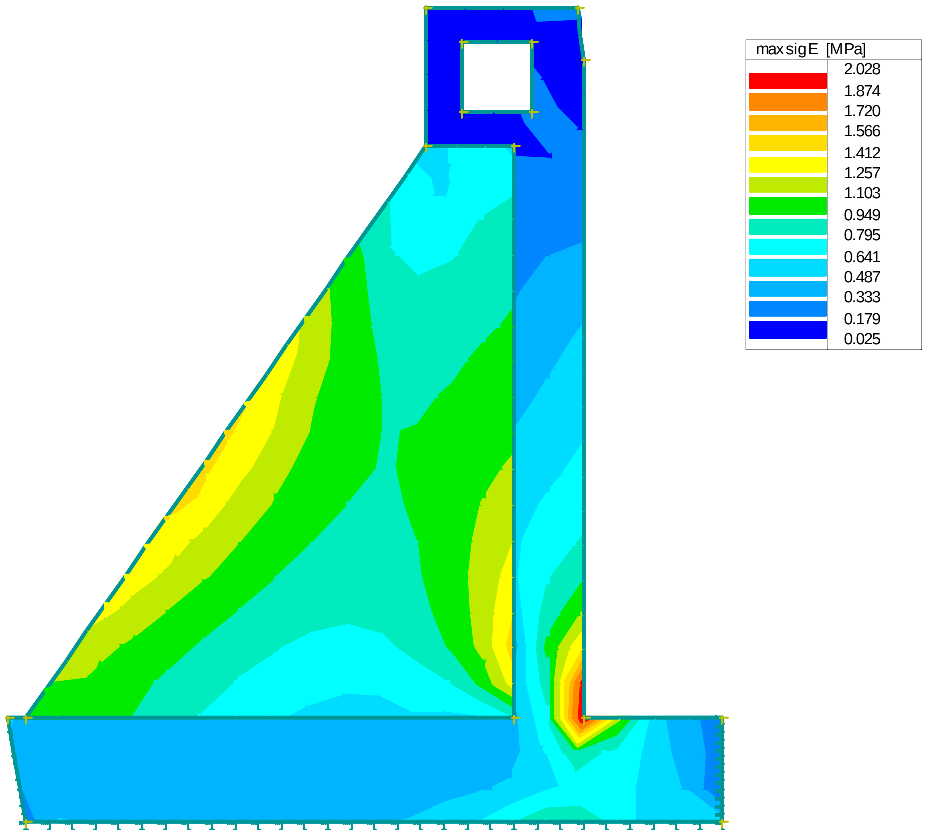
Maximal horizontal displacement, $u_{x,max}$ = 38.0 mm



4.2.7 CONCRETE STRESSES

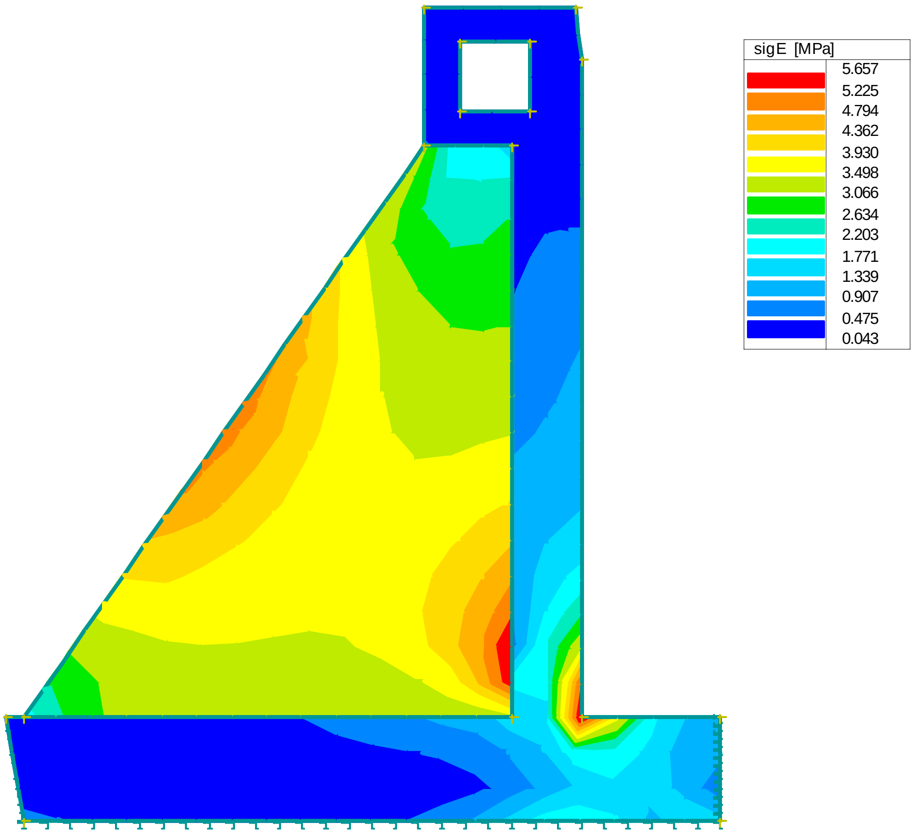
A. Fundamental load combinations F1 tot F2:

$1.125 \times \sigma_c < 0.85\ f_{ck}/1.5$
with $f_{ck} = 25N/mm^2$:
$\sigma_c = 2.03\ N/mm^2 < 12.5\ N/mm^2$



B. Accidental load combination MCE:

$\sigma_c < 0.85\ f_{ck}$
with $f_{ck} = 25N/mm^2$:
$\sigma_c = 5.66\ N/mm^2 < 21.25\ N/mm^2$



4.2.8 SUMMARY

Criterion	QP1 (Quasi Permanent Load Combination - Service)	F (Envelope of Fundamental Load Combinations)	MCE (Accidental Load Combination - MCE)
Compression zone A' > 90% A	100%	-	-
Compression zone A' > 75% A	-	-	
Compression zone A' > 10% A	-	100%	40%
Load Bearing Capacity			
1.4 $q_{ref} < q_u$ (MPa)	-	0.458	-
$q_{ref} < q_u$ (MPa)	-	-	0.835
Deformations			
maximal displacement (mm)	3.8	-	22.2
Concrete stresses			
1.125 x $\sigma c < 0.85$ fck/1.5	-	16%	-
$\sigma c < 0.85$ fck	-	-	27%
Deep Seeted Sliding			
	-	-	OK

5 Type 2 – Foundation in Atlantic Muck

5.1 ANALYSIS

Flexible dolphins absorb the vessel berthing energy by deformation. Deformation of a vertical tubular pile is due to the elastic deflection of the pile and the soil during the horizontal impact when the ship hits the pile.

The ship's berthing energy can be calculated by following relation:

$E = 0,5*M_d*v^2*C_m*Ce*Cs*Cc$

With:

- M_d = vessel displacement tonnage, taken as 200,000tons
- V = vessel approach velocity, taken as 0.15m/sec
- C_m = added mass coefficient = 1+2D/B = 1+2*15.3/49 = 1.625
- Ce = excentricity factor = 1 (conservative)
- Cs = softness coefficient = 1
- Cc = configuration coefficient = 1 for open structures

More information can be found in PIANC Report of WG 33 from 2002 "Guidelines for the Design of Fender Systems".

The vessel's berthing energy is consequently determined at E = 3,656kNm.

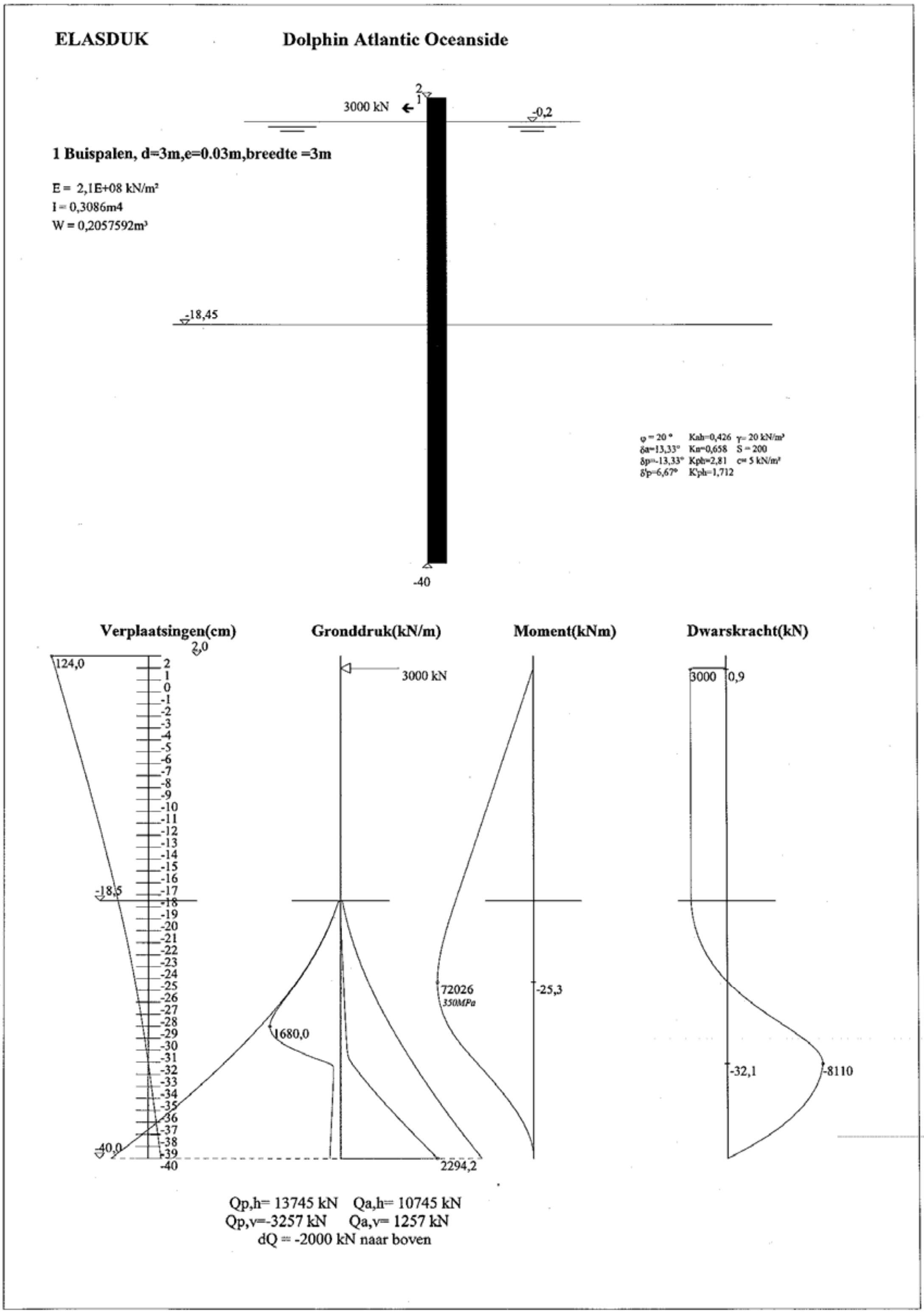
The distance between two adjacent dolphins has been set at 20m, as a consequence it can be assumed that the vessel's berthing energy will be taken by two dolphins (1,828kNm each)..

The tubular piles will be situated entirely in the "Atlantic Muck" soil, and as a conservative value for the physical characteristics of the soil following values will be retained for the analysis:

- $\Phi = 20°$ (angle of internal friction)
- $c = 5kN/m^2$ (cohesion)

5.2 RESULTS

The dolphin analysis is an elasto-plastic pile – soil interaction model. It is based on the relation between soil displacement and soil pressure (active or passive).



The model has been run for different pile diameters, penetration depths, and as a feasible result it was found that a pile with diameter of 3000mm and a wall thickness of 30mm, driven to approximately 22m penetration depth, can take a horizontal deformation of 1,24m at the top at a horizontal force of 3,000kN.
At that deformation the corresponding steel stress is reaching 350MPa, which is allowable for a high quality steel grade, such as X60.

The results of the calculation are shown on the figure below.

The corresponding absorbed energy equals :

$E_{dolphin} = ½ * 1,24 * 3,000 = 1,860kNm > E_{vessel} = 1,828kNm$

CPP Consorcio Post-Panamax

Update of Pacific Locks Conceptual Design and Harmonization of Atlantic Locks Conceptual Design – Contract SAA-143351

ATLANTIC LOCKS HARMONIZATION

TASK A4L – Construction Plan and Schedule

Rev A

Tractebel Development Engineering | TECHNUM | COYNE ET BELLIER | Compagnie Nationale du Rhône

in association with

SBE | PORT OF ANTWERP | FITA AOSO

TABLE OF CONTENTS

0 Introduction

This Chapter aims to detail how the harmonized (2005) Atlantic Post Panamax Locks will be constructed and put into operation. The purpose of this document is to show a feasible construction plan and schedule, compatible with today's modern construction techniques.
The following paragraphs will therefore describe the Construction Plan together with a general organization of the Works, distribution of main construction features within the project area, and the corresponding Construction Schedule of the harmonized (2005) Atlantic Locks.
The Construction Works analyzed in the present chapter are limited on the South by the extremity of the side-approach-wall from Gatun Lake , and on the North by the extremity of the Atlantic side-approach-wall.

1 Construction Plan

1.1 GENERAL

- Construction planning is an essential activity during the design of the Project. Therefore, we have developed a Construction Plan foreseeing the use of the most effective construction techniques. Access space as well as availability of resources has been taken into account in the selection of procedures and equipment.

The Construction Plan does the following:

- Defines the work tasks.
- Describes the technology and resources required to perform the work and the manner in which these must be handled to ensure their availability in a timely manner.
- Identifies any interactions among the different work tasks.

1.2 Description of Work to be performed

1.2.1 Activities

Land Clearing
- Equipment
- Disposal

Excavation
- Rock
  - Drilling
  - Blasting
  - Use for fill
  - Pile for use in aggregate production
  - Haul away to deposit site
  - Stabilization
- Soil
  - Use for fill
  - Haul away to deposit site
  - Stabilization

Tunnels and Trenches

Drainage Ditching and Dewatering

Fill and Compaction
- Rock
  - Transportation and Depositing
  - Compaction
- Soil
  - Transportation and Depositing
  - Compaction

Access Roads and Structures
- Permanent
- Temporary
- Bridges and Culverts

Forms
- Production
  - Wood
  - Steel
- Transportation
- Storage
- Placement

Concrete Production
- Cement handling and storage
- Aggregate production and stockpiling
- Additives
- Batching plant

Temperature control
Aggregates

Ice
Transportation
Placement
Transit mixers and transportation vehicles
Pumping
Pipelines and conduits
Pumps

Shotcrete

Cement handling and storage
Aggregate production and stockpiling
Additives
Transportation
Equipment storage and maintenance facility

Utilities

Power supply and distribution system
Water supply and distribution system
Sewerage and waste disposal
Communications
Lighting

Offices

Project management and engineering

Shops

Machine
Electrical
Vehicle
Welding
Pipe
Carpentry

Warehousing and Storage

Materials
Parts
Explosives
Fuel and lubricants

Personnel Facilities

Dressing rooms
Sanitary Facilities
Dining Facilities
Security

Construction Equipment

Storage
Servicing and repair

Firefighting facilities

1.2.2 Facilities

1. Concrete

It is foreseen that the bulk of the concrete will be transported by conveyor belt, although other alternatives may also be used for transporting concrete to special areas. A decision whether to pump or to transport concrete in buckets will directly affect the cost and duration of tasks involved in building construction. A decision between these two alternatives should consider the relative costs, reliability, and availability of equipment for the two transport methods. Unfortunately, the exact implications of different methods depend upon numerous considerations for which information may be sketchy during the planning phase. These are:

Quality Control laboratory
Cement storage silos
Loading and conveying equipment
Water storage tanks
Aggregate handling and stockpiling facilities
Batching Plant
Concrete transportation equipment
Transportation equipment cleaning facilities
Concrete temperature control equipment and facilities

2. Aggregate Processing Plant

Rock crushing and classification equipment, including crushers, screens, conveyors, waterpumps, fuel storage tanks, hoppers and stockpiles
Crushers, screens, washers and their feeding and discharge mechanisms
Loading, moving and hauling equipment
Emission Control Equipment

3. Metal and Reinforcing Steel Shop

Deliveries of structural and reinforcing steel will be scheduled in accordance with the execution of the work in order to reduce on-site storage to a minimum. Fabrication of rebar will be performed at this location whenever feasible and transported for placement at the worksite when required. Pre-assembly of structural steel sections will also be done here and transported to the location when required.

Unloading, classification and storage of steel sections and reinforcement bars
Welding area
Fabricating area
Transportation and handling equipment
Bridge crane

4. Concrete Form Shop

Unloading, classification and storage of metal and wood materials
Fabrication facility

Repair facility
Form storage and assembly area
Handling, loading, and transportation equipment

5. Shotcrete Facilities

Equipment storage and repair facility
Cement storage and transportation equipment
Aggregate stockpiling, loading and transportation equipment

6. Drilling and Blasting Facility

Drill and bit storage and repair shop

7. Powder Magazine (located and managed in coordination with Canal Protection and Safety Department).

Equipment storage warehouse
Loading, handling and transportation equipment

8. Pipe Shop

Receiving and storage facilities
Handling equipment
Shop equipment

9. Fuel and Lubricants Facilities

Storage tanks and warehouses
Service stations
Lubricant disposal

10. Compressed Air

Compressors
Storage
Distribution

1.2.3 PRODUCTION CAPACITY BY ACTIVITIES

The work production capacity for the different activities required to build the harmonized (2005) Atlantic Post Panamax Locks Project can be established using information obtained from similar projects, as shown in the following Table:

REQUIRED WORK PRODUCTION CAPACITY

Type of Activity	Production Adopted for the Locks m³/month	Factor	Production Capacity Required m³-month
Rock Excavation	300,000	2	600,000
Soil Excavation	500,000	2	1,000,000
Fill	200,000	1.5	300,000
Conventional Concrete	50,000	1.5	75,000
Roller Compacted	80,000	1.5	120,000

"Production capacity" required is the "peak" assumption for Contractor's equipment to be available, in order to achieve a production which is a "mean" production, so called "Production adopted".
Setting up of works time schedule is based on "Production adopted".

2 Construction Schedule

2.1 GENERAL

In order to establish realistic Schedules based on feasible work production capacities, an analysis of large construction Projects carried out in different countries has been carried out. The analysis is presented below.

2.2 WORK PRODUCTION CAPACITY

The Table 3.1, presented on the next page, sums up the information gathered from various large Construction Projects undertaken during the last ten years.

The different Projects are presented in columns. The top line summarizes the main characteristics and dimensions of each Project
The left column lists the different types of work considered. Each project column details the corresponding total quantity of work, the monthly peak and mean production capacities (highlighting with a red circle the relevant specific work of each project), and the extreme right column gives the selected production capacity for the harmonized (2005) Atlantic Post Panamax Locks.
The following comments can be made:

- The ACP Cut-Widening Project shows a mean production for rock excavation in the range of 100.000 m3/month. It has been verified (source: ACP) that the Contractors involved in those works were not working at full capacity. Tripling that production capacity is a necessary target, because of the large quantities of WSB excavations, in Gatun rock formation.

WORKS PRODUCTION CAPACITY
References in large Construction Projects and recommendation for the Post Panamax Locks project

PROJECT		Garafiri	Al Wahda	Birecik	Katse	Tha Dan	ACP Cut Widening	ACP Cut Widening
Description Type		Earth dam	Earth dam	Composite dam Earth & concrete	Arch dam	RCC gravity dam	Cartagena Soil Excavations	Gold Hill Rock excavations
Height		75m	90m	62m	185m	95m		
Length		725m	2700m	2507m	710m	2600m		
Year of construction		1994-1998	1992-1996	1997-1999	1991-1997	1996-1998	1999-2001	1997-1999
Country		Guinea	Morocco	Turkey	Lesotho	Thailand	Panama	Panama
Excavation	**Unit**							
Rock								
Total Volume	m3				975 300			1 485 000
Monthly Mean Production	m3/month				80 000			**100 000**
Monthly Peak Production	m3/month							170 000
Soil								
Total Volume	m3	4 805 132		11 000 000	278 000		2 250 000	
Monthly Mean Production	m3/month	190 000		**500 000**			125 000	
Monthly Peak Production	m3/month	400 000						
Dredging								
Total Volume	m3							
Monthly Mean Production	m3/month							
Monthly Peak Production	m3/month							
Fill								
Total Volume	m3	5 275 757	27 000 000	9 000 000				
Monthly Mean Production	m3/month	290 000	**482 000**	**500 000**				
Monthly Peak Production	m3/month	575 000	1 500 000	750 000				
Concrete								
Conventional Concrete								
Total Volume	m3	108 528	600 000	1 660 000	2 360 000			
Monthly Mean Production	m3/month	4 930	13 000	**70 000**	**75 000**			
Monthly Peak Production	m3/month	9 000	28 000	104 000	115 000			
Roller Compacted Concrete								
Total Volume	m3					5 000 000		
Monthly Mean Production	m3/month					**130 000**		
Monthly Peak Production	m3/month					170 000		
Source of information		*Coyne et Bellier*					*ACP*	

- The concrete placement capacity of the Birecik and Katse projects, reaching roughly 70.000 m3/month, are quite high because of the project type involving mainly large mass concrete structures without any significant reinforcement constraints. For the harmonized (2005) Atlantic Post Panamax Locks, it is considered that the geometry is to some extent more complicated than for a gravity dam, which justifies the chosen monthly mean production capacity for concrete placement of 50.000 m3/month

- The same comment made above applies for RCC placement. As a matter of fact, the working area of Tha Dan dam is wide and long and, above all, the RCC placement is continuous; whereas, for the harmonized (2005) Atlantic Post Panamax Locks the areas where RCC is required are of smaller dimensions, and the rhythm of placement is governed by the conventional concrete construction progress. An RCC monthly mean capacity placement of 80.000 m3/month seems more appropriate at this stage for the harmonized (2005) Atlantic Post Panamax Locks.

2.3 DESCRIPTION OF THE CONSTRUCTION SCHEDULE

2.3.1 QUANTITIES

The quantities given in the construction schedule are the quantities, as computed from the drawings, without any mark-up for contingencies.

2.3.2 CONSTRUCTION HYPOTHESIS

The construction schedule presented below is based on a certain construction sequence. The sequence of work adopted supposes that:

- The work starts from Lockhead 1 located on the Gatun Lake side, and progresses northward, except for the Gatun side-approach-wall which progresses southward.

- Only a single Lockhead can be built at a time.

- The concreting of one Chamber can start only if all the excavation is completed in the area, including the corresponding WSB´s culverts excavation close to the Lockwall or below the Chamber itself. It is assumed that one third of the culvert excavation has to be done together with the chamber excavations.

- The rest of the WSB´s culvert excavation and the proper WSB´s excavation will be done after chamber and lockwall excavations are completed. As a matter of fact, the WSB´s excavations may even be completed when the Lock structure is filled with water.

- The filling of the lock from the Atlantic side can only start when all the civil works and electromechanical equipment erection of the Lock is completed Of course the valves of the WSB shall also be erected, tested and dry commissioned because, at that time, the WSB construction will still be going on and must be isolated from the Locks.

- Once the Locks are opened on the Atlantic side, and filled up to the Atlantic level, all the rolling gates erected in a shipyard will be transported by flotation up to the locks and stored in Chamber 3 (Atlantic side).

- The two downstream rolling gates are then shifted and suspended into the Lockhead 4 recesses. The same will be sealed with bulkhead gates and pumped out. The rolling gate will be fitted with wooden seals, wagons, etc. (finishing operations). Once ready for operation, the gates will be shifted into the Lock.. The same procedure can then be repeated for the Lockhead 3, 2, and 1

2.3.3 KEY DATES

The Construction Schedule details the following Key Dates:

- Completion of the Civil Work of each Lockhead
- Connection of the locks to the Atlantic and filling the Chambers 3 and 2 up to the Atlantic level
- Connection of the Locks to the Gatun lake
- Locks and WSB starting date for commercial operation

2.3.4 CRITICAL PATH

The excavation works are continuously on this Critical Path, with a necessary excavation production rate much higher than what was required for the Pacific side.
The concreting activities have been scheduled assuming a mean rate of concrete placement of 50,000 m3 per month. Some float has been left for instance between the end of excavation of Chamber (N)-Lockhead (N+1) and the concreting of Lockhead (N+1).

3 Conclusion

The Construction Schedule shows that the total construction time for the Locks and associated WSB is coming to six years. It is certainly possible to compact a little bit more the Construction Schedule by increasing the mean production for rock excavation which is already 300,000 m3 per month, however the Consultant considers that at the present Conceptual Design Stage it is recommended to leave some float in the critical path.
The Construction Schedule assumes that the works progress from South to North. A reverse progression could have been analyzed considering that the flooding of the Locks starts from the Atlantic side. Some construction time may be saved assuming that the flooding takes place when the Gatun side of the Lock remains under construction finishing. However the corresponding saving will be limited to about two months (time needed for the rolling gate finishing works in the Lockhead 4) as the main longitudinal culverts and the Gatun water intakes will rapidly be needed to fill the lock chamber 3 and raise the water level in order to float and install the rolling gates in the next Lockhead 3.

ANNEXES

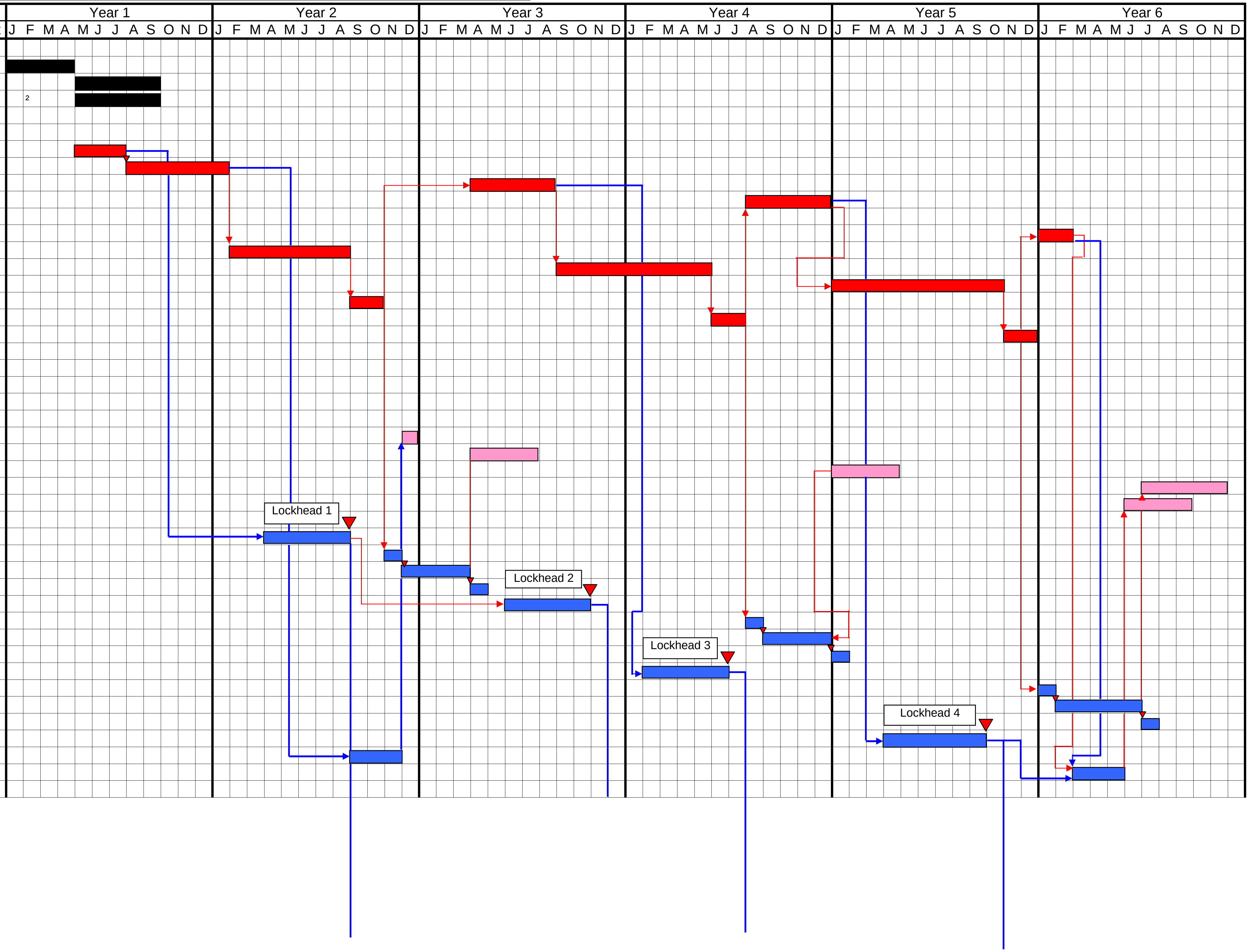
CONSTRUCTION SCHEDULE
HARMONIZATION (2005) ATLANTIC POST PANAMAX LOCKS (W = 55M / 3 WSB)

Tractebel Development Engineering — TECHNUM — Compagnie Nationale du Rhône — COYNE ET BELLIER

Excavations
Backfill
Concreting

Rolling Gate
Electromechanical equipment
RCC

Item No	ITEMS	Qty	Unit
1	**CIVIL WORKS FOR LOCKS**		
1,1	Mobilization		
1,2	Contractor installation		
1,3	Access roads		
1,4	Excavation for Locks		
1.4.1	Lockhead 1	582.000	m3
1.4.2	Chamber 1 and lockhead 2	1.900.000	m3
1.4.3	Chamber 2 and lockhead 3	1.420.000	m3
1.4.4	Chamber 3 and lockhead 4	1.527.000	m3
1.4.5	Gatun approach walls	0	m3
1.4.6	Atlantic approach wall	152.400	m3
1,5	Excavation for WSB		
1.5.1	WSB Chamber 1	2.138.000	m3
1.5.2	WSB Chamber 2	2.770.000	m3
1.5.3	WSB Chamber 3	2 850 00	m3
1.5.4	Culvert 11 and 12	424.000	m3
1.5.5	Culvert 21 and 22	424.000	m3
1.5.6	Culvert 31 and 32	424.000	m3
1,6	Fill for WSB		
1.6.1	WSB Chamber 1	0	m3
1.6.2	WSB Chamber 2	0	m3
1.6.3	WSB Chamber 3	0	m3
1,7	Backfilling for Lockwalls		
1.7.1	Gatun approach walls	135.000	m3
1.7.2	Lockhead 1 and Chamber1	600.000	m3
1.7.3	Lockhead 2 and Chamber 2	621.000	m3
1.7.4	Lockhead 3 and Chamber 3	715.000	m3
1.7.5	Lockhead 4 and Atlantic approach wall	418.000	m3
1,8	Concreting Locks		
1.8.1	Lockhead 1 and U/S transition segments	122.000	m3
1.8.2	Chamber 1 bottom culvert	8.500	m3
1.8.3	Chamber 1 Lockwall East and West	201.700	m3
1.8.4	Chamber 1 Bottom Concrete Slabs/filling	24.800	m3
1.8.5	Lockhead 2	140.700	m3
1.8.6	Chamber 2 bottom culvert	8.500	m3
1.8.7	Chamber 2 Lockwall East and West	200.000	m3
1.8.8	Chamber 2 Bottom Concrete Slabs/filling	24.650	m3
1.8.9	Lockhead 3	140.400	m3
1.8.10	Chamber 3 bottom culvert	8.500	m3
1.8.11	Chamber 3 Lockwall East and West	210.100	m3
1.8.12	Chamber 3 Bottom concrete filling	26.400	m3
1.8.13	Lockhead 4 and D/S transition segments	166.300	m3
1.8.14	Gatun approach walls	40.400	m3
1.8.15	Atlantic approach wall	59.300	m3



CONSTRUCTION SCHEDULE
HARMONIZATION (2005) ATLANTIC POST PANAMAX LOCKS (W = 55M / 3 WSB)

Tractebel Development Engineering — TECHNUM — Compagnie Nationale du Rhône — COYNE ET BELLIER

Excavations
Backfill
Concreting
Rolling Gate
Electromechanical equipment
RCC

Item No	ITEMS	Qty	Unit
2	**LOCKS ROLLING GATES**		
2,1	Detailed design		
2,2	Construction		
2.2.1	Lock gates 11 and 12	3.460	t
2.2.2	Lock gates 21 and 22	5.670	t
2.2.3	Lock gates 31 and 32	5.670	t
2.2.4	Lock gates 41 and 42	5.470	t
2,3	Transport		
2.3.1	Lock gates 11 and 12		
2.3.2	Lock gates 21 and 22		
2.3.3	Lock gates 31 and 32		
2.3.4	Lock gates 41 and 42		
2,4	Erection		
2.4.1	Lock gates 11 and 12		
2.4.2	Lock gates 21 and 22		
2.4.3	Lock gates 31 and 32		
2.4.4	Lock gates 41 and 42		
2,5	Testing and comissioning		
2.5.1	Lock gates 11 and 12		
2.5.2	Lock gates 21 and 22		
2.5.3	Lock gates 31 and 32		
2.5.4	Lock gates 41 and 42		
3	**ELECTROMECHANICAL EQUIPMENT FOR LOCKS**		
3,1	Detail design		
3,2	Valve construction	1.610	t
3,3	Transport		
3,4	Erection		
3,5	Comissioning		
4	**WATER SAVING BASINS**		
4,1	Lining WSB 1	300.000	m2
4,2	Concreting		
4.2.1	Culverts, Intakes and valve block WSB 11	50.860	m3
4.2.2	Culverts, Intakes and valve block WSB 12	50.860	m3
4.2.3	Culverts, Intakes and valve block WSB 21	50.860	m3
4.2.4	Culverts, Intakes and valve block WSB 22	50.860	m3
4.2.5	Culverts, Intakes and valve block WSB 31	50.860	m3
4.2.6	Culverts, Intakes and valve block WSB 32	50.860	m3
4.2.7	Walls WSB 1	30.600	m3
4.2.8	Walls WSB 2	30.600	m3
4.2.9	Walls WSB 3	30.600	m3
4,3	RCC culvert fill	222.500	m3
4,4	Electomechanical for WSB		
4.4.1	Valve and Stoplog Gate Detail Design		
4.4.2	Valve and gate Construction	3.400	t
4.4.3	Transport		
4.4.4	Erection		
4.4.5	Tests and comissioning		

Year 1 | Year 2 | Year 3 | Year 4 | Year 5 | Year 6 — each: J F M A M J J A S O N D