CPP Consorcio Post-Panamax

ACP
AUTORIDAD DEL CANAL DE PANAMA

Conceptual design to recycle water in Post Panamax locks

HYDRAULIC PART

FINAL REPORT

R-HY- 002 - B

Tractebel Development Engineering
TECHNUM
COYNE ET BELLIER
CNR PROJETS

in association with

IMDC
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AUTORIDAD DEL CANAL DE PANAMA

Conceptual design to recycle water in Post Panamax locks

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SUMMARY

0. EXECUTIVE SUMMARY

This report deals with the conceptual design to recycle water in Post Panamax locks (3rd lane), by a pumping system. The study is only conducted for the Pacific side.
The pumping system, added to a multiple lift locks system with water saving basins, allows to save almost 100 % of the water required for the lockages.

The study has been carried out for the 3 pumping scenarios described hereafter :

- **Direct pumping system** : the water is pumped from Pacific Ocean to Gatun Lake. It is the more simple system, yet it is also the worst concerning the salt intrusion into the Gatun Lake since the pumped water is salty.

- **Semi direct pumping system** : the water is pumped from a lower reservoir to the Gatun Lake. The reservoir is located on the west bank of the locks. Its maximum water level is set under ocean level in order to be able to recover all the water spilled from the lower lock. This system also injects brackish water in Gatun Lake.

- **Pond to pond pumping system** : the water is pumped from a lower reservoir to an upper reservoir. The reservoirs are connected to the longitudinal culverts of the locks filling and emptying system. The lower reservoir is located on the west bank of the locks. Its maximum water level is set under ocean level in order to be able to recover all the water spilled from the lower lock. The upper reservoir is actually made by damming the Cocoli River. This system is the best for preventing salt intrusion into the Gatun Lake.

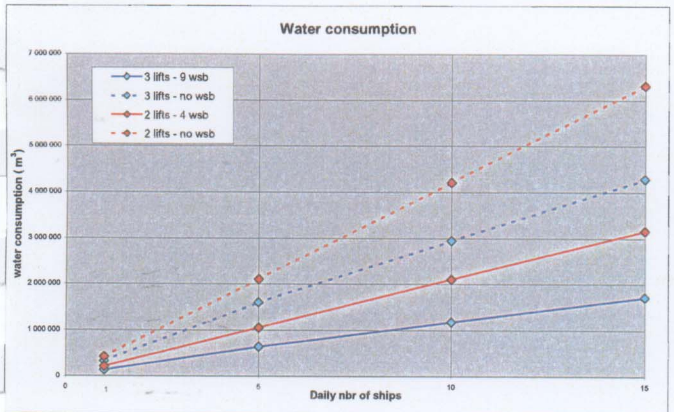
Every scenario has been tested for the 2-lift and 3-lift locks system with and without water saving basin. Moreover, study assumes daily traffic levels for Post-Panamax vessel of 1, 5, 10 and 15 lockage per day.

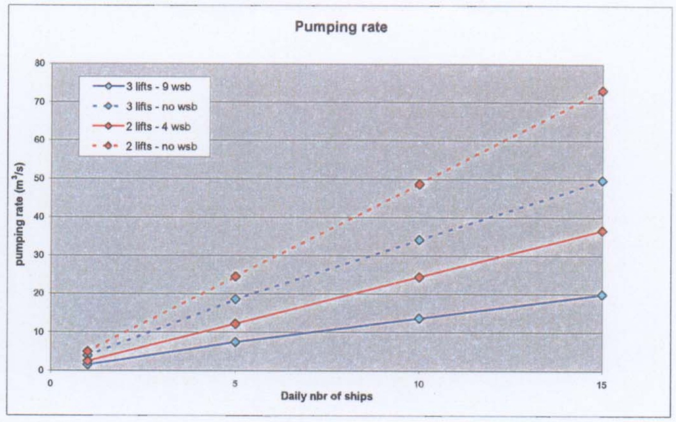
Calculations assumes a Gatun Lake level of + 26.00 m PLD and the Mean Low Water Spring ocean tide range, i.e. [-2.32 ; + 2.40] m PLD.

The first stage consists in :

- Calculating the volumes of water taken from the Gatun Lake (respectively from the upper reservoir) and spilled to the Pacific Ocean (respectively to the lower reservoir),
- Determining the pumping flow rate for different daily traffic levels (1, 5, 10 and 15 ships per day),
- Determining the water level variations in the upper and lower reservoirs.

These calculations have been carried out with the Consultant Software already used in former studies. The graphics hereafter give an overview of water consumption and pumping flow rate evolution with the daily traffic level for the 2-lift and 3-lift lock system.





Then the pumping system (pump, culvert, valve, grid) has been designed by means of the Flowmaster software. For each configuration, the pumping system is composed of three identical pumping networks designed for flow rates ranging from 0 to $Q_{\max}/3$ where $Q_{\max}$ is the pumping flow rate associated with a daily traffic of 15 ships/day.

Flowmaster has been used in order to :

- define the components of the pumping circuit between lower and upper ponds (valve, bends, length and size of culverts and grid),
- calculate regular and singular head losses in every component for the range of flow rates given by the consultant software calculations. These data are required to elaborate the network curve, $\Delta H=f(Q)$ for each configuration.

Culvert arrangements have been studied to minimize excavation works.

Tests have also been carried out to evaluate the time to fill the upper lock chamber from the upper reservoir and the time to empty the lower lock chamber to the lower reservoir. The conclusion is that the pumping system will not reduce the daily number of passing ships since filling and emptying times are shorter than times without the pumping system.

The problem of water hammer in the conduits has also been investigated. It is compulsory to install a valve upstream the pumps to avoid reversing flow if it runs out of order.

1. FOREWORD

1.1 Contract

This report is performed within the scope of the contract n° SAA-110829 awarded June 10th 2003 to the Consortium named CPP (Consorcio Post Panamax) by the client ACP (Autoridad del Canal de Panama).

This contract concerns the conceptual design study to recycle water in the 3rd lane of Post Panamax locks.

1.2 Scope of work

This study aims at developing the system requirements to recycle spilled water from Post-Panamax Locks and to prepare conceptual designs for different scenarios. This study includes capital investment costs for the project, operation and maintenance costs in order that the Autoridad del Canal de Panama can compare with other water supply alternatives.

This study is divided in two phases:

Phase I: Design of system requirements for a recycling system for the different operating scenarios at future Post-Panamax Locks. The result of this study will be provided to Delft Hydraulics for salinity intrusion study and modeling.

Phase II: Conceptual design and cost of civil, structural, electrical and mechanical system, capital investment, operation and maintenance costs.

1.2.1 Scope of Work for Phase I – Study and Design of System Requirements for a Recycling System for the Different Operating Scenarios at Future Post-Panamax Locks

In phase I, design of the system requirements for a recycling system of lockage water for future Post-Panamax Locks has been carried out. The system requirement identifies the water volumes, water levels, flows, frequency, capacities, alternative solutions, equipment size, heads, system losses, outlet to Pacific Ocean, connection to upper reservoir and operating times.

CPP has used information generated from contract no. SAA-97462 Conceptual Design for Post-Panamax Locks in the Pacific: hydraulic design, locks alignment, water saving basin arrangement, culvert size, conduit size, valve selection, gate design, operating times, connection to lock chambers.

The design of the system requirements for a water recycling system for future Post-Panamax Locks includes the following combination of scenarios:

- Study is conducted for Pacific side of the Canal only
- Study assumes a three-lift Post-Panamax locks with three water saving basins (WSB) per level and a second configuration for a two-lift locks with two WSB per lift.
- Study is conducted for locks operating with and without WSB.
- Study assumes daily traffic levels for Post-Panamax vessels of : 1, 5, 10, and 15 lockages per day.
- A baseline scenario assumes recycling water directly from the Pacific Ocean directly into Gatun Lake.
- A second scenario consists in recycling water from a lower storage reservoir directly into Gatun Lake.
- A third scenario includes water storage reservoirs at lower (ocean) end and upper (lake) end, tied into Post-Panamax locks intake and discharge system.
- Conceptual designs from this recycling study will be provided to WL/Delft Hydraulics (Delft) for numerical modeling of saltwater intrusion due to recycling lockage water in proposed Post-Panamax locks at the Panama Canal.

1.2.2 Scope of Work for Phase II – Conceptual Design and Costing of Water Recycling System in Post-Panamax Locks

In phase II the Contractor will :

- Provide conceptual level design for recycling system for Post-Panamax Locks, to include civil, structural, electrical, and mechanical engineering studies of alternatives and design options.
- Provide capital investment within 25% margin of accuracy, operation and maintenance costs, construction schedule, equipment manufacturers and data.
- Consider provisions for future 4th lane Post-Panamax locks to the west of the third lane

2. GENERAL

The actual locks of the Panama Canal are going to be saturated in less than ten years. In addition, the Canal cannot service Post-Panamax vessels, which are too large to fit in the existing locks.

The "Autoridad del Canal de Panama" is executing a study for conceptual design of a third lane Post-Panamax. Nevertheless, operating the new locks will require additional water resources, which are not currently available, hence solutions for saving water have to be found.

Several possibilities have been studied to minimize water consumption at the lock as well as to provide additional water in the Gatun Lake.
Additional water resources can be provided by deepening the existing canal, by constructing additional reservoirs in the canal area, and by new reservoirs outside the watershed of the canal area. All these possibilities are investigated by ACP.

The measures to minimize the water consumption at the locks are the following :

- multiple lift configurations,
- several water saving basin dispositions,
- spilled water recycling system.

Up to now, the multiple lift systems and the water saving basins have been considered and analyzed in a conceptual design phase. The recycling of the spilled water has been considered simultaneously to obtain a general idea of the requirements but not sufficiently in detail to compare with other possibilities in terms of investment and operating costs.

The purpose of this study is to carry out a conceptual level design of the water recycling by pumping systems, according to different scenarios.

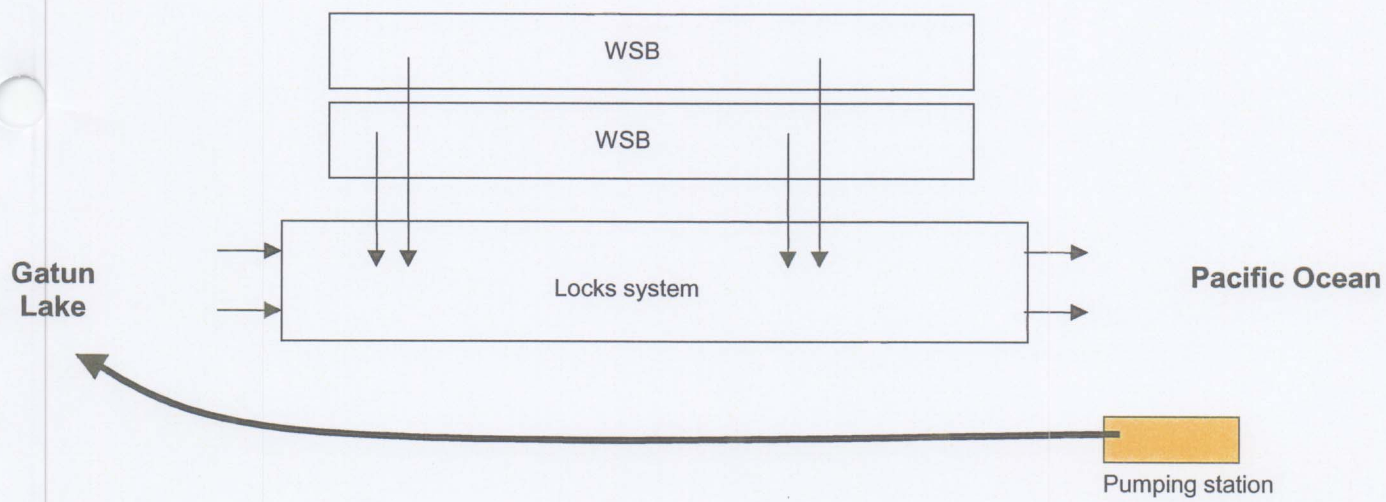
3. TERMS OF REFERENCE

3.1 Definition of the pumping scenarios

3.1.1 Direct pumping

The water is pumped from the Pacific Ocean directly to the Gatun Lake. This system is the more simple of the three systems studied both from a design and operational point of view. Nevertheless, it is also the worst for the salt intrusion in Gatun Lake.

The sketch below shows the general scheme :

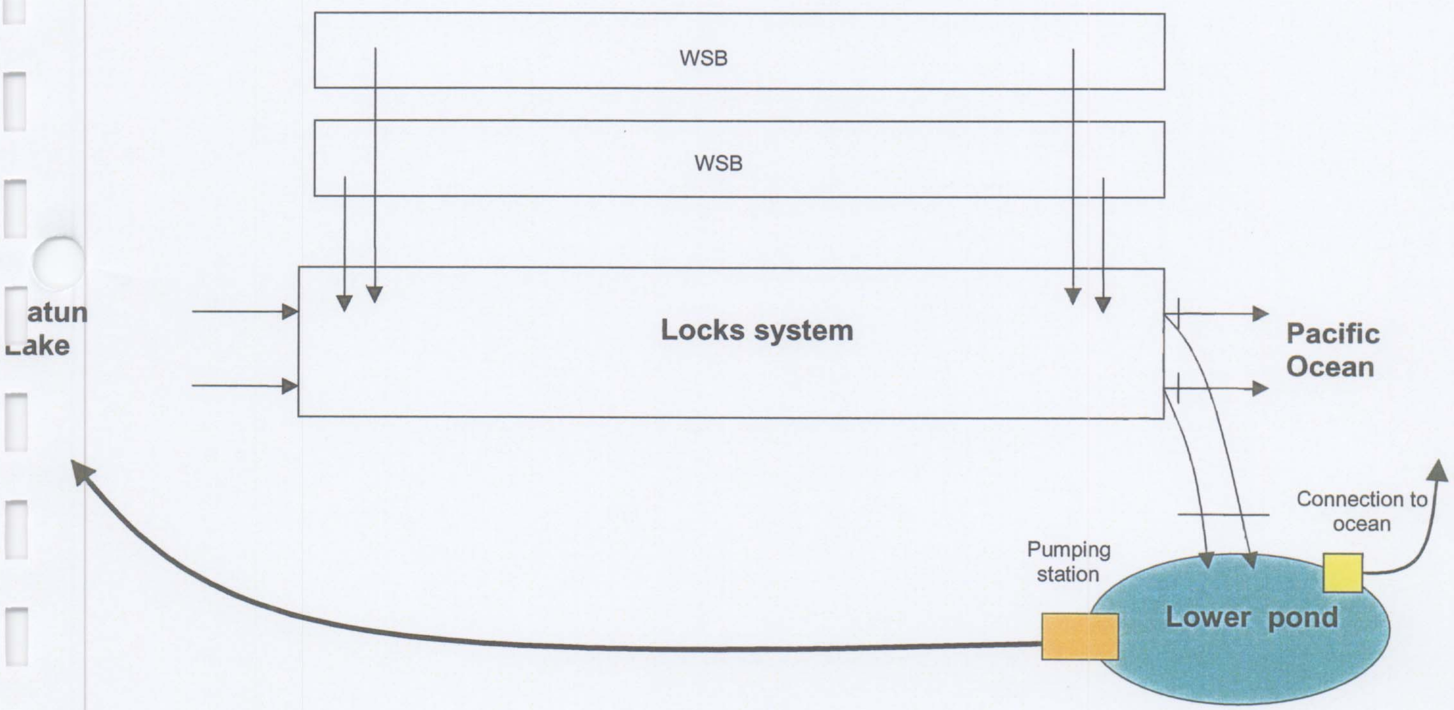


3.1.2 Semi direct pumping

The water spilled from the locks system is stored in a reservoir, independent from the Pacific Ocean, and pumped to Gatun Lake. The water flows in the lower pond by means of a culverts and valves network connected to the filling/emptying lock system.
During the "wet" season and when recycling water is not necessary, the water is spilled directly to the Pacific Ocean.

A pumping system will also allow to empty the lower pond in the Pacific Ocean for maintenance operation.

The sketch below shows the general scheme :



<u>Remark</u> : it is essential that the water level in the lower lock chamber should not be higher or lower than the Pacific Ocean level at the end of the emptying operation. Indeed a level difference (superior than 10 cm) would prevent the downstream gate from opening. This problem is highlighted in the "Design criteria and assumptions" chapter.

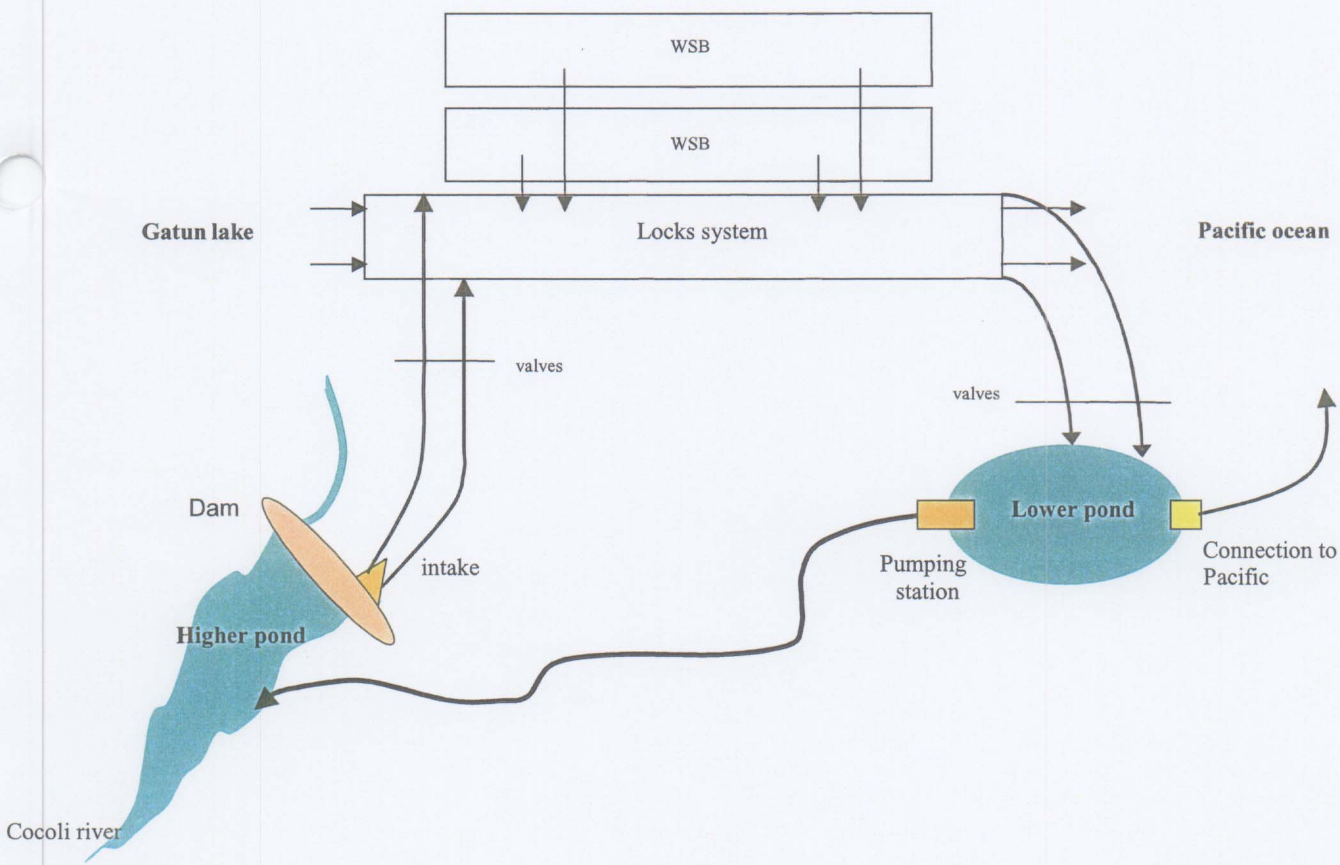
3.1.3 Pond to pond pumping

The water spilled from the locks system is stored in a lower reservoir, independent from the Pacific Ocean, and pumped to an upper reservoir on the Cocoli River. The upper lock chamber can be filled both from the Gatun Lake and the Cocoli reservoir.

The filling/emptying lock system is connected to the upper and lower ponds by means of a culverts and valves network.

During the "wet" season and when recycling water is not necessary, the water is spilled directly to the Pacific Ocean. In the same way, the upper lock chamber is filled from the Gatun Lake. A spillway on the Cocoli dam regulates the reservoir water level.

A pumping system will also allow to empty the lower pond in the Pacific Ocean for maintenance operation.



Remark : it is essential that the water level in the lower lock chamber should not be higher or lower than the Pacific Ocean level at the end of the emptying operation. In the same way, the water level in the upper lock chamber should not be higher or lower than the Gatun Lake level at the end of the filling operation. Indeed a level difference (superior than 10 cm) would prevent the gates from opening. This problem is highlighted in the "Design criteria and assumptions" chapter.

3.2 Levels

3.2.1 Lake Gatun

Maximum level : +26.67 m PLD
Minimum level : +23.90 m PLD

3.2.2 Pacific Ocean

Ranging from -3.44 to +3.60 m PLD

ACP has agreed to design the bottom levels of the chambers with the Mean Low Water Spring level, i.e. -2.32 m PLD, for all of the 3 lock systems (single, double and triple lift).

3.3 Sizes - Dimensions

3.3.1 Sizes of the chambers

- Useful length of the locks : 426.80 m
- Useful width of the locks : 61.00 m
- Draft (minimum water height over the sills) : 18.30 m
- Freeboard : 3.00 m
- Chamber area (depends on the configuration) :

configuration	wet area of the chamber (m2)
3 lifts	31 000
2 lifts	32 470
1 lift	33 800

3.3.2 Sizes of the ships

	m	f
width	54.00	180
length	386	1 265
draught	15.20	50
capacity	up to 140 000 dwt	

3.4 Operating times

Water recycling system only impacts the upper chamber filling and the lower chamber emptying times. These times will be calculated for each configuration and compared to the times given in the "Filling and emptying system design" reports.

3.5 Number of solutions to be studied

The table below gives an overview of the solutions that have been studied.

pumping configuration	configuration of locks and water saving basins	number of daily ship passages
pumping from a downstream reservoir to an upstream reservoir	3 lifts - 9 wsb	1 ship, 5 ships, 10 ships, 15 ships
	3 lifts - no wsb	
	2 lifts - 4 wsb	
	2 lifts - no wsb	
pumping from a downstream reservoir to Gatun lake	3 lifts - 9 wsb	
	3 lifts - no wsb	
	2 lifts - 4 wsb	
	2 lifts - no wsb	
pumping from Pacific Ocean to Gatun lake	3 lifts - 9 wsb	
	3 lifts - no wsb	
	2 lifts - 4 wsb	
	2 lifts - no wsb	

Theoretically, 48 scenarios have to be studied

These scenarios result from the following tables given by ACP, leading to increasing water consumptions and ship transits.

(water consumed in one locks - multiply x 2 for a complete ocean to ocean transit)

Metros cúbicos por esclusaje	porcentaje	Esclusajes Panamax	Number of water saving basins per lift				
			1 lfit convoy	1 lift alternate	2 lift locks	3 lfit locks	
714 133	100%	7.70	no wsb	x	x	x	
476 089	67%	5.13	1 wsb	x	x	x	
357 066	50%	3.85	2 wsb	no wsb	no wsb	x	opcion #1
285 653	40%	3.08	3 wsb	x	x	x	
238 044	33%	2.57	4 wsb	1 wsb	1 wsb	no wsb	opcion #2
204 038	29%	2.20	5 wsb	x	x	x	
178 533	25%	1.92	6 wsb	2 wsb	2 wsb	x	opcion #3
158 696	22%	1.71	x	x	x	1 wsb	
142 827	20%	1.54	x	3 wsb	3 wsb	x	
119 022	17%	1.28	x	4 wsb	4 wsb ?	2 wsb ?	opcion #5 ?
102 019	14%	1.10	x	5 wsb	5 wsb	x	
95 218	13%	1.03	x	x	x	3 wsb	opcion #4
89 267	13%	0.96	x	6 wsb ?	6 wsb	x	(close enough)
79 348	11%	0.86	x	x	x	4 wsb	
68 013	10%	0.73	x	x	x	5 wsb	
59 511	8%	0.64	x	x	x	6 wsb	

Water consumed per lock cubic meters per day				Total consumption both locks - cubic meters/day			
One lockage	5 daily	10 daily	15 daily	Un esclusaje	5 diarios	10 diarios	15 diarios
714 133	3 570 665	7 141 330	10 711 995	1 428 266	7 141 330	14 282 660	21 423 990
476 089	2 380 445	4 760 890	7 141 335	952 178	4 760 890	9 521 780	14 282 670
357 066	1 785 330	3 570 660	5 355 990	714 132	3 570 660	7 141 320	10 711 980
285 653	1 428 265	2 856 530	4 284 795	571 306	2 856 530	5 713 060	8 569 590
238 044	1 190 220	2 380 440	3 570 660	476 088	2 380 440	4 760 880	7 141 320
204 038	1 020 190	2 040 380	3 060 570	408 076	2 040 380	4 080 760	6 121 140
178 533	892 665	1 785 330	2 677 995	357 066	1 785 330	3 570 660	5 355 990
158 696	793 480	1 586 960	2 380 440	317 392	1 586 960	3 173 920	4 760 880
142 827	714 135	1 428 270	2 142 405	285 654	1 428 270	2 856 540	4 284 810
119 022	595 110	1 190 220	1 785 330	238 044	1 190 220	2 380 440	3 570 660
102 019	510 095	1 020 190	1 530 285	204 038	1 020 190	2 040 380	3 060 570
95 218	476 090	952 180	1 428 270	190 436	952 180	1 904 360	2 856 540
89 267	446 335	892 670	1 339 005	178 534	892 670	1 785 340	2 678 010
79 348	396 740	793 480	1 190 220	158 696	793 480	1 586 960	2 380 440
68 013	340 065	680 130	1 020 195	136 026	680 130	1 360 260	2 040 390
59 511	297 555	595 110	892 665	119 022	595 110	1 190 220	1 785 330

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 - Final report configuration 1
- R4 – C : Filling and emptying system - Final report configuration 2
- R4 – C : Filling and emptying system - Final report configuration 3

4.2 Assumptions

4.2.1 Operating modes

For each configuration, the following operating modes are selected :

- One lift lock system : alternate mode
- Two lifts locks system : convoy mode
- Three lifts lock system : convoy mode

4.2.2 Gatun Lake and Pacific Ocean levels

According to the Terms of Reference, the minimum Gatun Lake level is 23.90 m . During the meeting R1 in Panama, ACP notified that the recycling system could operate even when the Gatun Lake is higher than its lowest level (+23.90 m PLD). Consequently ACP prefers to set the lake level at + 26.00 m PLD for the simulations.

The level of the Pacific Ocean fluctuates according to the MLWS tides. The following equation has been used for carrying out the simulations :

$$Z_{Pacific} = 2.36 * \sin\left(\frac{2\pi}{12.47} * t\right) + Z_{mean}$$

with $Z_{mean} = 0.04$ m

As a result the ocean level ranges from -2.40 m PLD to +2.32 m PLD.

4.2.3 Operating the pumping station

In this hydraulic design, it is assumed to operate the pumping station during the dry season continuously during the lockage period in order to optimize power supply. In this way the power requirements for the pumping stations can be minimized.

4.2.4 Sizes of the culverts

In order to minimize the head losses, the maximum velocity in the culverts of the pumping circuit will be limited to 3 m/s.

Moreover, in order to allow an easier construction of the culverts and/or transportation of the pipes, the size of the conduits will also be limited to 3.20 m.

Consequently the size and the number of the culverts will be adjusted to fit in with these criteria.

4.2.5 Reservoirs : relationships area versus level and volume versus level

The *annex 1* shows the variation of the area and capacity of the Cocoli reservoir with the level in PLD meters.

The lower reservoir is shown on the drawing in *annex 2* "General layout". It has been situated on the west side of the locks, near a former ground excavation. Its useful wet area, 240 000 m^2, does not fluctuate with the water level.
This reservoir size is the same for the "semi-direct" and "pond to pond" pumping scenario.

4.2.6 Reservoir water levels

Water levels in the reservoirs depend on :

- reservoir area,
- pumping flow rate,
- flow rate between upper pond and upper lock chamber (chamber filling phase),
- flow rate between lower pond and lower lock chamber (chamber emptying phase).

Concerning the Cocoli reservoir, it does not take into account the water supply coming from the Cocoli River, which can be evacuated (regulated) by the spillway.

For each configuration, i.e. for each combination of locks and water saving basins, we can assume that :

- Given the pond area and the pumping flow rates, the variation of the Cocoli reservoir water level is very small (less than 0.80 m).
- The variation of the lower reservoir depends on the supply from the locks, the pumping rate and the pond area.

The pumping rate is set to limit the variation of water levels in the upper and lower ponds. Indeed, if the flow rate is higher than the "balance" flow rate, the lower pond will empty in time, on the contrary, if the flow rate is lower than the "balance" flow rate the lower pond will be overfilled. The pumping rate is continuous and mainly proportional to the number of lockages.

The variation of the levels of the reservoirs can be calculated by means of the following equations :

Upstream reservoir:

$$Z_{up}(t+\Delta t) = Z_{up}(t) - \frac{Q_{filling} * \Delta t}{S_{up}} + \frac{Q_{pump} * \Delta t}{S_{up}}$$

Downstream reservoir :

$$Z_{down}(t+\Delta t) = Z_{down}(t) + \frac{Q_{emptying} * \Delta t}{S_{down}} - \frac{Q_{pump} * \Delta t}{S_{down}}$$

with Q_{filing} : flow rate between upper pond and upper lock chamber

Q_{filing} : flow rate between lower lock chamber and lower pond

Q_{pump} : pumping flow rate

S_{up} : upper pond area

S_{down} : lower pond area

Particular attention has to be paid to the "lower lock chamber to lower pond" emptying phase and to the "upper pond to upper lock chamber" filling phase. At the end of these phases, the water level in lower lock chamber (respectively upper lock chamber) must be as close as possible to the Pacific Ocean level (respectively Gatun Lake level) in order to be able to open the gate.

For that purpose, the consultant proposes :

- to install level gauges in lock chamber and ponds in order to fit the valves opening ratio in with level variations and reach the expected water level.
- to close the pond valves before required levels have been reached in the chamber (when the chamber level is 50 cm above or under the required value for example) and to let the chamber level equalize with the lake or the ocean level. Nevertheless this operation will not permit to spare all the water.
- to install butterfly valves in the gates to equalize the levels if necessary. This operation will certainly increase the filling / emptying times.

5. SIMULATIONS

The study has been carried out in two phases :

- a first series of simulations have been made with the Consortium Software used in "filling and emptying system" studies,
- a second series have been made with the Flowmaster Software, using the results achieved during the first phase (pumping heads, pumping flow rates).

5.1 *Consortium software*

The Consortium software developed to calculate the water level in lock chamber during lockage operations has been adapted for the recycling study in order to :

- Calculate the volumes of water taken from the Gatun Lake (respectively from the upper reservoir) and spilled to the Pacific Ocean (respectively to the lower reservoir),
- Determine the pumping flow rate for different daily traffic levels (1, 5, 10 and 15 ships per day),
- Determine the water level variations in the upper and lower reservoirs. The equations used to calculate the level are given in paragraph 4.2.6. and recalled hereafter :

Upstream reservoir :

$$Z_{up}(t+\Delta t) = Z_{up}(t) - \frac{Q_{filling} * \Delta t}{S_{up}} + \frac{Q_{pump} * \Delta t}{S_{up}}$$

Downstream reservoir :

$$Z_{down}(t+\Delta t) = Z_{down}(t) + \frac{Q_{emptying} * \Delta t}{S_{down}} - \frac{Q_{pump} * \Delta t}{S_{down}}$$

5.1.1 Calculation assumptions

The data entered in the software to carry out calculations are the following :

- Gatun Lake level : +26.00 m PLD,
- Pacific Ocean tide range : MLWS range, i.e. from -2.32 m PLD to +2.40 m PLD,
- Lock chamber and WSB area : 3 lift lock system 31 000 m^2
  2 lift lock system 32 470 m^2
- Residual filling depth : 0 m
- Operating times

Times are given in minutes		**3 lift locks system**		**2 lift locks system**	
operation		**3 WSB**	**no WSB**	**2 WSB**	**no WSB**
gate opening		5'	5'	5'	5'
gate closing		5'	5'	5'	5'
filling/emptying of WSB		5'		5'	
filling/emptying of lock chamber		7'	9'	9'	14'
1st ship displacement	5 ships/day	73'	85'	95'	100'
	10 ships/day	29'	42'	37'	43'
	15 ships/day	11'	25'	18'	22'
ship displacement	5 ships/day	73'	85'	90'	100'
	10 ships/day	29'	42'	37'	43'
	15 ships/day	11'	24'	17'	22'

Simulations have been run for 60 days period (100 days for the 1 ship / day configuration).

The present software developed by CPP is based on regular transits of ships over a day. Thus the times figuring in the table were adapted to fit the exact number of ship passages during the period (night and day). In other words, the time needed to pass a ship is the same in each configuration and was lengthened (the maximum number of passages being nearly 15 in the previous studies)

5.1.2 Results

The chart hereafter gives the main results of the simulations :

		Pumping flow rate in m^3/s	Dowstream pond water level in m PLD		Upstream pond water level in m PLD		Pumping head in m		$Z_{up}(t=0)$ in m PLD	$Z_{down}(t=0)$ in m PLD
			min	max	min	max	min	max		
3 lifts - 3 WSB	1 ship / day	2.10	-5.3	-2.1	27.4	27.9	29.7	32.9	-4.5	27.5
	5 ships/day	7.45	-5.7	-3.9	27.3	27.8	31.3	33.4	-4.5	27.5
	10 ships/day	13.75	-6.1	-4.1	27.4	28.1	31.5	34.2	-4.5	27.5
	15 ships/day	20.00	-5.9	-4.0	27.3	27.9	31.3	33.8	-4.5	27.5
3 lifts - no WSB	1 ship / day	4.67	-6.1	-3.8	27.0	27.9	31.2	33.7	-4.5	27.5
	5 ships/day	18.95	-6.6	-3.8	27.0	28.1	30.8	34.6	-4.5	27.5
	10 ships/day	34.50	-6.8	-3.8	27.0	28.2	30.8	35.0	-4.5	27.5
	15 ships/day	50.10	-6.9	-4.0	27.2	28.2	31.2	35.1	-4.5	27.5
2 lifts - 2 WSB	1 ship / day	2.45	-7.0	-3.4	27.4	28.3	31.0	35.1	-4.5	27.5
	5 ships/day	12.45	-6.6	-4.1	27.2	28.1	31.3	34.7	-4.5	27.5
	10 ships/day	24.80	-5.9	-3.8	27.3	28.1	31.1	34.0	-4.5	27.5
	15 ships/day	36.30	-6.1	-3.8	27.4	28.2	31.2	34.3	-4.5	27.5
2 lifts - no WSB	1 ship / day	4.90	-6.8	-3.5	27.7	28.9	31.4	35.6	-4.5	28
	5 ships/day	24.30	-7.7	-3.8	27.2	29.2	31.0	36.9	-4.5	28
	10 ships/day	49.05	-6.7	-3.3	27.4	29.0	30.7	35.7	-4.5	28
	15 ships/day	73.75	-6.9	-3.8	27.2	28.6	31.0	35.5	-4.5	27.5

The water level range in upstream and downstream pond is given considering a continuous pumping flow rate all along the simulation period ; a non continuous pumping flow rate could reduce this range.

The graphics given in *annex 3* present the pond levels variation for the 3-lift locks system simulations.

5.2 Hydraulic design of the pumping system with Flowmaster

5.2.1 Pumping networks arrangement

Pumping network layout and longitudinal profile for each pumping scenario are given *in annexes 2, 5, 6 and 7*. The layout and profile have been established in accordance with the topographical map. Culverts arrangements have been studied to minimize both excavation work and singular head losses.

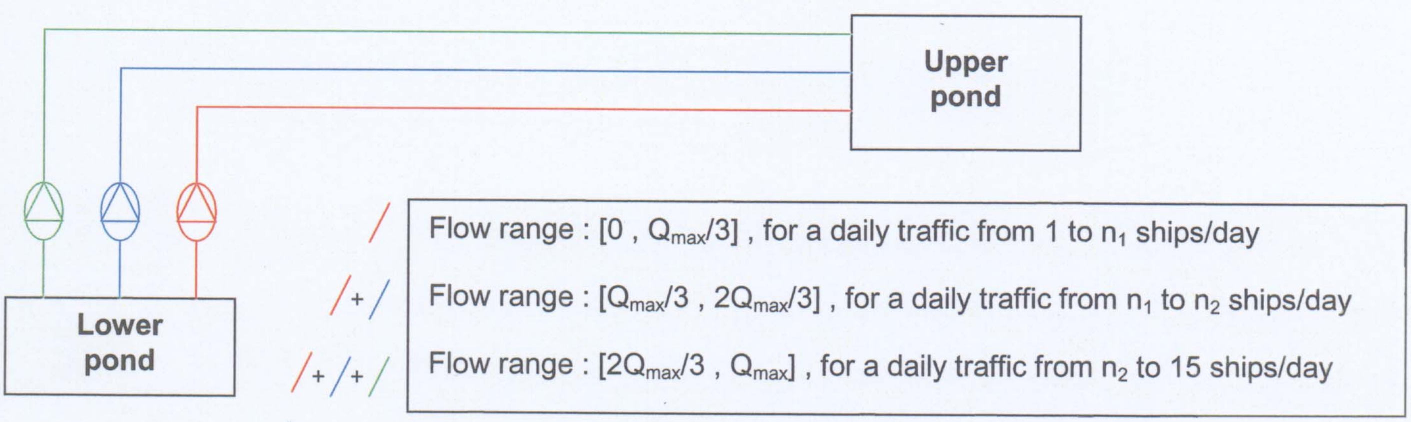
5.2.2 Principle of calculations

The pumping system has been studied with the Flowmaster 2. software already used for the design of the locks filling and emptying system.

Flowmaster has been used in order to :

- define the components of the pumping circuit between lower and upper pond (valve, bends, length and size of culverts and grid),
- calculate regular and singular head losses in every component for the range of flow rates given by the consultant software calculations. These data are required to elaborate the network curve, $\Delta H = f(Q)$ for each configuration.

For each configuration, the pumping system is made of three identical pumping networks (see sketch hereafter) designed for flow rates ranging from 0 to $Q_{max}/3$ where Q_{max} is the pumping flow rate associated with a daily traffic of 15 ships/day. Q_{max} values have been calculated during the first phase of the study with the consultant software.



For each pumping network, 5 or 6 simulations, with flow rates distributed over the range $\left[0; {Q_{\max}}/{3}\right]$, have been run with Flowmaster in order to draw the network curves. Regular and singular head losses have been calculated for all simulations.

Remark : The simulations show that regular head losses are much higher than singular head losses. This is due to the great length of the culverts in the circuit (2 400 m, 2 500 m and 3 500 m depending on the scenario). Consequently, the design of the culverts (diameters, roughness) is essential in order to reduce as far as possible the total head loss.

The paragraphs hereafter describe more precisely the hydraulic design of the pumping network for the 3 scenario proposed in the TOR and for the 2-lift and 3-lift locks system.

5.2.3 Results for "pond to pond" pumping scenario (from lower reservoir to Cocoli reservoir)

5.2.3.1 Three lift lock system with 9 water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond when the pump runs out of order.
All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 1) and *annex 5* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 24 m^3/s
Pumping network flow rate range : from 0 to 8 m^3/s
Culvert diameter : 1.80 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m^3/s	*Total head losses in m*	*Geometric head in m*			*Total manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.2	31	32.5	34	31.2	32.7	34.2
3	1.3	31	32.5	34	32.3	33.8	35.3
5	3.6	31	32.5	34	34.6	36.1	37.6
7	6.9	31	32.5	34	37.9	39.4	40.9
9	11.3	31	32.5	34	42.3	43.8	45.3

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.1 m/s for a flow rate of 8 m^3/s.
The network curves are given in *annex 8.1.1*. The reference daily traffics are indicated on the curves.

- Alternative : 2 pumping networks

It can be seen on network curves given in *annex 8.1.1* "3-lift locks system with WSB - 3 pumping networks" that 2 networks are unable to handle a daily traffic of 15 ships per day. Furthermore, the third network is not used at its full capacity when the daily traffic raises to 15 ships per day. The pumping system is not fully optimized for that configuration.
That is the reason why additional simulations with only 2 pumping networks have been run to improve the system in general.

The network structure is kept unchanged excepted for the culvert diameter which is increased. Consequently, the maximum flow rate per network is also increased :

Pumping station flow rate range : from 0 to 22 m^3/s
Pumping network flow rate range : from 0 to 11 m^3/s
Culvert diameter : 2.10 m

The chart below shows the results achieved with Flowmaster :

Flowrate in m^3/s	*Total head losses in m*	*Geometric head in m*			*Total manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.1	31	32.5	34	31.1	32.6	34.1
3	0.6	31	32.5	34	31.6	33.1	34.6
6	2.4	31	32.5	34	33.4	34.9	36.4
9	5.2	31	32.5	34	36.2	37.7	39.2
12	9.2	31	32.5	34	40.2	41.7	43.2

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.2 m/s for a flow rate of 11 m^3/s.
The network curves are given in *annex 8.1.1*. The reference daily traffics are indicated on the curves. It appears that two pumping networks are sufficient to handle a daily traffic from 1 to 15 ships per day in this configuration.

5.2.3.2 Three lift locks system without water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.

All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 1) and *annex 5* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 60 m^3/s
Pumping network flow rate range : from 0 to 20 m^3/s
Culvert diameter : 2.90 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m^3/s	*Total head losses in m*	*Geometric head in m*			*Manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.0	31	33	35	31.0	33.0	35.0
5	0.3	31	33	35	31.3	33.3	35.3
9	1.1	31	33	35	32.1	34.1	36.1
13	2.2	31	33	35	33.2	35.2	37.2
17	3.8	31	33	35	34.8	36.8	38.8
21	5.7	31	33	35	36.7	38.7	40.7

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.0 m/s for a flow rate of 20 m^3/s.
The network curves are given in *annex 8.1.1*. The reference daily traffics are indicated on the curves.

5.2.3.3 Two lift locks system with 4 water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4. Annex 2* also presents a general layout of the circuit (solution 1) and annex 5 shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 39 m^3/s
Pumping network flow rate range : from 0 to 13 m^3/s
Culvert diameter : 2.30 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m3/s	*Total head losses in m*	*Geometric head in m*			*Manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.0	31	32.5	34	31.0	32.5	34.0
3	0.4	31	32.5	34	31.4	32.9	34.4
6	1.5	31	32.5	34	32.5	34.0	35.5
9	3.4	31	32.5	34	34.4	35.9	37.4
12	5.9	31	32.5	34	36.9	38.4	39.9
15	9.1	31	32.5	34	40.1	41.6	43.1

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.1 m/s for a flow rate of 13 m^3/s.
The network curves are given in *annex 8.1.2*. The reference daily traffics are indicated on the curves.

5.2.3.4 Two lift locks system without water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 1) and *annex 5* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 75 m^3/s
Pumping network flow rate range : from 0 to 25 m^3/s
Culvert diameter : 3.20 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m3/s	***Total head losses in m***	***Geometric head in m***			***Manometric head in m***		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.0	31	33.5	36	31.0	33.5	36.0
5	0.2	31	33.5	36	31.2	33.7	36.2
10	0.8	31	33.5	36	31.8	34.3	36.8
15	1.8	31	33.5	36	32.8	35.3	37.8
20	3.2	31	33.5	36	34.2	36.7	39.2
25	5.0	31	33.5	36	36.0	38.5	41.0

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.1 m/s for a flow rate of 25 m^3/s.
The network curves are given in *annex 8.1.2*. The reference daily traffics are indicated on the curves.

5.2.4 Results for semi direct pumping scenario (from lower reservoir to Gatun Lake)

5.2.4.1 Three lift lock system with 9 water saving basin

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 2) and *annex 6* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 24 m^3/s
Pumping network flow rate range : from 0 to 8 m^3/s
Culvert diameter : 1.80 m

The chart below shows the results achieved with Flowmaster :

Flowrate in m^3/s	Total head losses in m	Geometric head in m			Total manometric head in m		
		minimum	mean	maximum	minimum	mean	maximum
1	0.2	30	31	32	30.2	31.2	32.2
3	1.7	30	31	32	31.7	32.7	33.7
5	4.7	30	31	32	34.7	35.7	36.7
7	9.0	30	31	32	39.0	40.0	41.0
9	14.8	30	31	32	44.8	45.8	46.8

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.1 m/s for a flow rate of 8 m^3/s.
The network curves are given in *annex 8.2.1*. The reference daily traffics are indicated on the curves.

5.2.4.2 Three lift lock system without water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.

All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 2) and *annex 6* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 60 m^3/s
Pumping network flow rate range : from 0 to 20 m^3/s
Culvert diameter : 2.90 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m3/s	*Total head losses in m*	*Geometric head in m*			*Total manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.0	30	31.5	33	30.0	31.5	33.0
5	0.4	30	31.5	33	30.4	31.9	33.4
9	1.4	30	31.5	33	31.4	32.9	34.4
13	2.9	30	31.5	33	32.9	34.4	35.9
17	4.9	30	31.5	33	34.9	36.4	37.9
21	7.3	30	31.5	33	37.3	38.8	40.3

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.0 m/s for a flow rate of 20 m^3/s.
The network curves are given in *annex 8.2.1*. The reference daily traffics are indicated on the curves.

5.2.4.3 Two lift lock system with 4 water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 2) and *annex 6* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 39 m^3/s
Pumping network flow rate range : from 0 to 13 m^3/s
Culvert diameter : 2.30 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m^3/s	*Total head losses in m*	*Geometric head in m*			*Total manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.1	30	31	32	30.1	31.1	32.1
3	0.5	30	31	32	30.5	31.5	32.5
6	2.0	30	31	32	32.0	33.0	34.0
9	4.4	30	31	32	34.4	35.4	36.4
12	7.7	30	31	32	37.7	38.7	39.7
15	11.9	30	31	32	41.9	42.9	43.9

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.1 m/s for a flow rate of 13 m^3/s.
The network curves are given in *annex 8.2.2*. The reference daily traffics are indicated on the curves.

5.2.4.4 Two lift lock system without water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4. Annex 2* also presents a general layout of the circuit (solution 2) and *annex 6* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 75 m^3/s
Pumping network flow rate range : from 0 to 25 m^3/s
Culvert diameter : 3.20 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m3/s	*Total head losses in m*	*Geometric head in m*			*Total manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.0	30	31.5	33	30.0	31.5	33.0
5	0.3	30	31.5	33	30.3	31.8	33.3
10	1.1	30	31.5	33	31.1	32.6	34.1
15	2.4	30	31.5	33	32.4	33.9	35.4
20	4.1	30	31.5	33	34.1	35.6	37.1
25	6.4	30	31.5	33	36.4	37.9	39.4

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.1 m/s for a flow rate of 25 m^3/s.
The network curves are given in *annex 8.2.2.* The reference daily traffics are indicated on the curves.

5.2.5 Results for direct pumping scenario (from Pacific Ocean to Gatun Lake)

5.2.5.1 Three lift lock system with water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4. Annex 2* also presents a general layout of the circuit (solution 3) and *annex 7* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 24 m^3/s
Pumping network flow rate range : from 0 to 8 m^3/s
Culvert diameter : 1.80 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m^3/s	***Total head losses in m***	***Geometric head in m***			***total manometric head in m***		
		minimum	***mean***	***maximum***	***minimum***	***mean***	***maximum***
1	0.2	24	26	28	24.2	26.2	28.2
3	1.3	24	26	28	25.3	27.3	29.3
5	3.4	24	26	28	27.4	29.4	31.4
7	6.6	24	26	28	30.6	32.6	34.6
9	10.8	24	26	28	34.8	36.8	38.8

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.1 m/s for a flow rate of 8 m^3/s.
The network curves are given in *annex 8.3.1*. The reference daily traffics are indicated on the curves.

5.2.5.2 Three lift lock system without water saving basins

- Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 3) and *annex 7* shows the longitudinal profile of the penstock.

- Network features

Pumping station flow rate range : from 0 to 60 m^3/s
Pumping network flow rate range : from 0 to 20 m^3/s
Culvert diameter : 2.90 m

The chart below shows the results achieved with Flowmaster :

Flow rate in m^3/s	*Total head losses in m*	*Geometric head in m*			*total manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.0	24	26	28	24.0	26.0	28.0
5	0.3	24	26	28	24.3	26.3	28.3
9	1.0	24	26	28	25.0	27.0	29.0
13	2.1	24	26	28	26.1	28.1	30.1
17	3.6	24	26	28	27.6	29.6	31.6
21	5.5	24	26	28	29.5	31.5	33.5

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.0 m/s for a flow rate of 20 m^3/s.
The network curves are given in *annex 8.3.1*. The reference daily traffics are indicated on the curves.

5.2.5.3 Two lift lock system with water saving basins

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 3) and *annex 7* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 39 m^3/s
Pumping network flow rate range : from 0 to 13 m^3/s
Culvert diameter : 2.30 m

The chart below shows the results achieved with Flowmaster :

Flowrate in m^3/s	*Total head losses in m*	*Geometric head in m*			*total manometric head in m*		
		minimum	*mean*	*maximum*	*minimum*	*mean*	*maximum*
1	0.0	24	26	28	24.0	26.0	28.0
3	0.4	24	26	28	24.4	26.4	28.4
6	1.4	24	26	28	25.4	27.4	29.4
9	3.2	24	26	28	27.2	29.2	31.2
12	5.7	24	26	28	29.7	31.7	33.7
15	8.8	24	26	28	32.8	34.8	36.8

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

The maximum velocity reached in the network is 3.1 m/s for a flow rate of 13 m^3/s.
The network curves are given in *annex 8.3.2*. The reference daily traffics are indicated on the curves.

5.2.5.4 *Two lift lock system without water saving basins*

➢ Network description

Pumping networks are mainly composed of 1 pump, 1 valve, 1 grid and a long culvert (reversing culvert). The grid protects the pump from sucking any debris in the lower pond. The valve is situated upstream the pump and prevents the emptying of the upper pond in case of the pump runs out of order.
All the components are described more precisely in *annex 4*. *Annex 2* also presents a general layout of the circuit (solution 3) and *annex 7* shows the longitudinal profile of the penstock.

➢ Network features

Pumping station flow rate range : from 0 to 75 m^3/s
Pumping network flow rate range : from 0 to 25 m^3/s
Culvert diameter : 3.20 m

The chart below shows the results achieved with Flowmaster :

Flowrate in m^3/s	Total head losses in m	Geometric head in m			total manometric head in m		
		minimum	mean	maximum	minimum	mean	maximum
1	0.0	24	26	28	24.0	26.0	28.0
5	0.2	24	26	28	24.2	26.2	28.2
10	0.8	24	26	28	24.8	26.8	28.8
15	1.8	24	26	28	25.8	27.8	29.8
20	3.1	24	26	28	27.1	29.1	31.1
25	4.9	24	26	28	28.9	30.9	32.9

The geometric head is the difference between upper pond and lower pond water levels.
The manometric head is equal to the geometric head plus head losses.

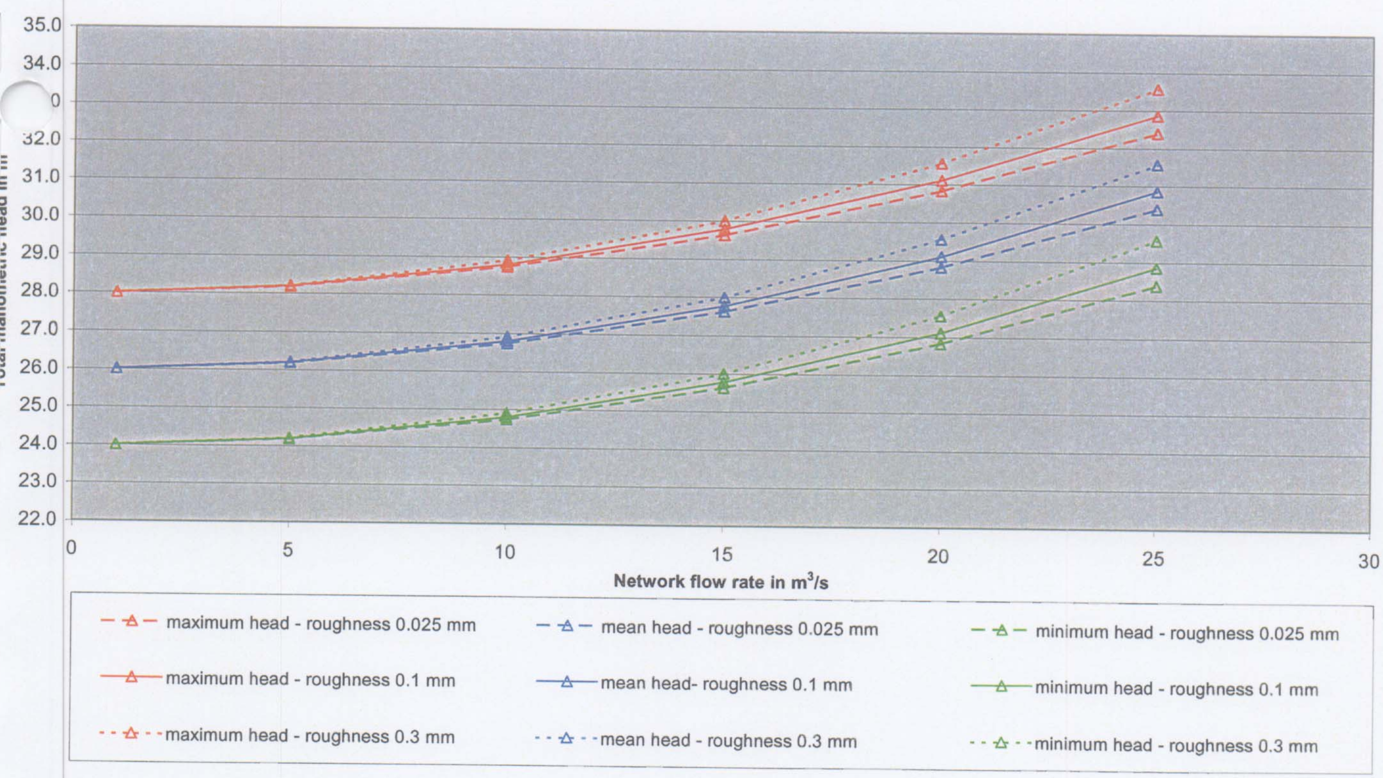
The maximum velocity reached in the network is 3.1 m/s for a flow rate of 25 m^3/s.
The network curves are given in *annex 8.3.2*. The reference daily traffics are indicated on the curves.

5.2.6 Additional simulation 1 : roughness sensitivity

All the results given above have been achieved considering a roughness of 0.1 mm for the inner surface of the culverts. This value corresponds to the roughness of a concrete pipe with carefully made joints. This roughness can be associated either to a concrete pipe structure built on site or to an assembly of prefabricated concrete pipes. The equivalent Strickler coefficient value (noted K_s) is about 85. Nevertheless original roughness can be modified according to the building site conditions, moreover culvert aging also contributes to the increase of roughness.
Consequently, some additional simulations have been run with ranging roughness in order to assess the effects of a modification of this parameter on the hydraulic design.

The graph hereafter shows the network curves for the "2-lift locks system without WSB" configuration, direct pumping scenario. Each curves has been drawn for a roughness of 0.025 mm (smooth concrete pipe, K_s = from 90 to 100), 0.1 mm (concrete pipe, $K_s \approx 85$) and 0.3 mm (rough concrete pipe, $K_s \approx 75$).

Network curves for one pumping network
Culvert diameter 3.20 m



It obviously appears that a rougher pipe leads to higher manometric head. Nevertheless, The difference is not significant since it represents only 2 % of **the total manometric head** for a flow rate of 20 m^3/s and 4 % of **the total manometric head** for a flow rate of 25 m^3/s (considering the mean head).

All data are given in *annex 9*.

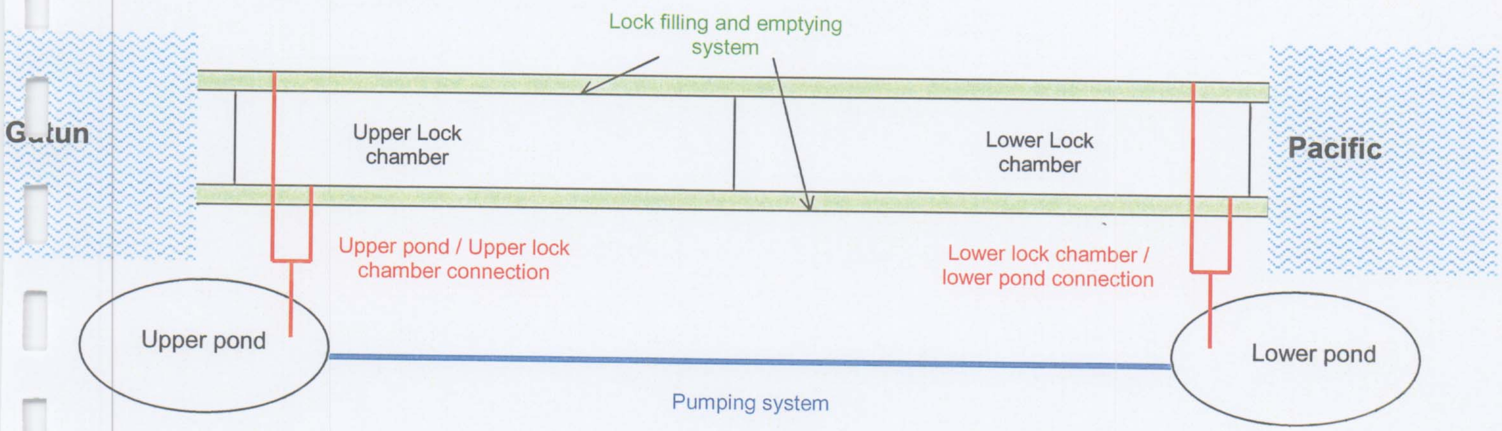
5.2.7 Additional simulation 2 : number of pumps per network

Simulations have also been run with 2 pumps per network. The network curves are not modified. This parameter will be studied more precisely in the next stage of the design.
Graphics and result charts are given in *annex 10.*

5.3 *Hydraulic design of the connections between ponds and lock chambers*

5.3.1 General description

This part aims to dimension at a conceptual design level, the circuit between upper pond and upper lock chamber and between lower lock chamber and lower pond.
These system have to be connected to the lock filling and emptying system, designed in previous studies, as shown on the following sketch :



Both connections (upper and lower) are composed of two culverts, 7.50 m high and 9.00 m wide. The general layout is shown in *annex 2.* They pass under the locks and meet the filling and emptying locks culvert inside the lock chamber as shown on section view in *annex 11.*

Water intakes in upper pond and outflow in the lower pond are larger than the main culvert, a smooth transition will limit the head losses. Water intake and outflow are protected by grids. Some reservations are foreseen for installing stoplogs. The valves controlling the flow rate are located on the west side of the 3rd line of locks. Their positions take into account the fourth lane of locks.

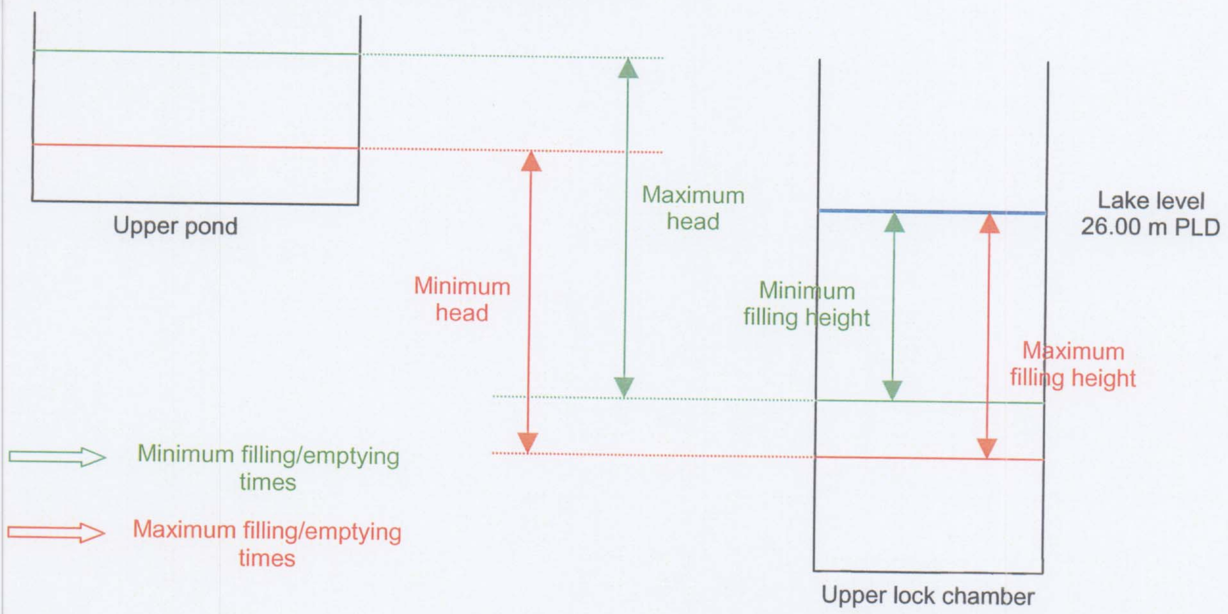
Further accurate design will be made in the next stage of the study

5.3.2 Flowmaster calculations

Once the filling and emptying circuit have been designed, Flowmaster has been used to calculate the minimum and maximum times for filling and emptying operation. The minimum and maximum times are defined as follow :

- Minimum time : minimum filling time of the upper lock chamber or minimum emptying time of the lower lock chamber is reached when the "filling height" is minimum while the initial head is maximum
- Maximum time : maximum filling time of the upper lock chamber or minimum emptying time of the lower lock chamber is reached when "the filling height" is maximum while the initial head is minimum

The sketch hereafter illustrates the definition :



The maximum and minimum head or "filling height" have been calculated by means of the Consultant software previously used to set the minimum and maximum water level reached in upper and lower ponds (cf. paragraph 5.1).

Flowmaster simulations have been performed with the following data :

- Culverts size : 7.5 * 9 m^2
- Culverts absolute roughness :
  0.1 mm for the culvert connecting pond to lock chamber ($Ks \approx 85$)
  0.025 mm for the culvert of filling/emptying system ($Ks \approx 90$)
- Lock chambers area : 32 400 m^2 for the 2-lift locks system
  31 000 m^2 for the 3-lift locks system
- Ponds area : 500 000 m^2 for the upper pond
  240 000 m^2 for the lower pond

5.3.3 Results

All the times given in the charts hereafter correspond :

- for upstream lock chamber, to the time when chamber water level reaches the Gatun Lake level (i.e. 26 m PLD)
- for downstream lock chamber, to the time when chamber water level reaches the Pacific Ocean level (i.e. [-2.32 ; +2.40] m PLD)

Simulations did not make allowance for valve closure operation. It is yet essential to anticipate the valve closure (i.e. starting closing the valve before the water level reaches the required level). However this operation should not affect too much the filling or emptying times as it has been demonstrated in the "design of filling/emptying system" studies.

Graphics presenting the results, i.e. flow rate versus time and water level versus time, are given in *annex 12.*

5.3.3.1 Connection between upper pond and upper lock chamber

The first chart gives the maximum filling time, as defined above, the second chart gives the minimum filling times.

configuration	upstream chamber		upstream reservoir		Height to fill (m)	Initial head (m)	**Maximum filling time (s)**
	Level at beginning of filling phase (m)	Level at end of filling phase (m)	Level at beginning of filling phase (m)	Level at end of filling phase (m)			
2 chambers without WSB	11.95	26.00	28.55	27.65	14.05	16.60	**765**
2 chambers with WSBs	19.35	26.00	27.90	27.45	6.65	8.55	**490**
3 chambers without WSB	16.70	26.00	28.10	27.50	9.30	11.40	**565**
3 chambers with WSBs	22.55	26.00	27.75	27.50	3.45	5.20	**315**

configuration	upstream chamber		upstream reservoir		Height to fill (m)	Initial head (m)	**Minimum filling time (s)**
	Level at beginning of filling phase (m)	Level at end of filling phase (m)	Level at beginning of filling phase (m)	Level at end of filling phase (m)			
2 chambers without WSB	14.15	26.00	28.45	27.70	11.85	14.30	**685**
2 chambers with WSBs	19.75	26.00	27.85	27.45	6.25	8.10	**470**
3 chambers without WSB	17.95	26.00	28.00	27.50	8.05	10.05	**520**
3 chambers with WSBs	22.55	26.00	27.80	27.55	3.45	5.25	**310**

5.3.3.2 Connection between lower pond and lower lock chamber

The first chart gives the maximum emptying time, as defined above, the second chart gives the minimum emptying times.

configuration	Downstream chamber		Downstream reservoir		Height to empty (m)	Initial head (m)	**Maximum emptying time (s)**
	Level at beginning of emptying phase (m)	Level at end of emptying phase (m)	Level at beginning of emptying phase (m)	Level at end of emptying phase (m)			
2 chambers without WSB	14.00	-2.25	-7.60	-5.40	16.25	21.60	**590**
2 chambers with WSBs	7.50	-2.15	-6.70	-5.35	9.65	14.20	**400**
3 chambers without WSB	9.80	-2.10	-6.55	-5.00	11.90	16.35	**435**
3 chambers with WSBs	4.50	-1.70	-5.90	-5.10	6.20	10.40	**290**

configuration	Downstream chamber		Downstream reservoir		Height to empty (m)	Total head (m)	**Minimum emptying time (s)**
	Level at beginning of emptying phase (m)	Level at end of emptying phase (m)	Level at beginning of emptying phase (m)	Level at end of emptying phase (m)			
2 chambers without WSB	11.90	0.25	-5.50	-3.95	11.65	17.40	**470**
2 chambers with WSBs	5.65	1.50	-4.45	-3.90	4.15	10.10	**225**
3 chambers without WSB	7.00	2.40	-5.15	-4.55	4.60	12.15	**205**
3 chambers with WSBs	2.50	0.20	-4.40	-4.10	2.30	6.90	**160**

5.3.3.3 *Comparison with the times calculated in former "design of filling/emptying system" studies*

The results of simulations have also been compared to the filling and emptying times stemming from the former "design of filling and emptying system" studies in order to assess the impact of the pumping system on the filling and emptying times of the lock chambers.
All data are presented in the chart hereafter, the time calculated in previous studies are given in columns "Emptying of lower lock chamber to Pacific Ocean" and "Filling of upper lock chamber from Gatun Lake".

		Emptying of lower lock chamber					Filling of upper lock chamber				
		To downstream reservoir			To Pacific Ocean		From upstream reservoir			From Gatun lake	
		Height to empty (m)	Total head (m)	Time (s)	Head (m)	Time (s)	Height to fill (m)	Total head (m)	Time (s)	Head (m)	Time (s)
2 chambers without basin	maximum	16.23	21.57	589			14.04	16.59	765		
	minimum	11.65	17.42	468	15	630	11.83	14.27	685	15	950
	mean	13.94	19.50	529			12.94	15.43	725		
2 chambers with basins	maximum	9.64	14.16	400	10.20	475	6.65	8.54	487	7.00	630
	minimum	4.17	10.10	223	3.15	260	6.23	8.08	469	5.60	560
	mean	6.91	12.13	312	6.68	368	6.44	8.31	478	6.30	595
3 chambers without basin	maximum	11.93	16.37	432			9.28	11.35	566		
	minimum	4.63	12.15	205	10	451	8.04	10.04	517	10	658
	mean	8.28	14.26	319			8.66	10.70	542		
3 chambers with basins	maximum	6.22	10.43	290	6.50	344	3.47	5.18	313	3.50	370
	minimum	2.25	6.89	160	5.10	304	3.46	5.23	310	3.20	355
	mean	4.235	8.66	225	not available		3.47	5.21	312	not available	

Remarks : filling and emptying times do not include valves closing time
filling and emptying times for the 3 step configurations have been recalculated with a 31 000 m^2 area for the chambers (instead of 26 035 m^2 in the first stage)

It can be seen that the pumping system improves the filing and emptying times of the upper and lower lock chambers. Nevertheless, there is no point in reaching too small filling and emptying times since the daily number of ships transits depends on the other filing and emptying operations. Consequently, it should be interesting, from an economical point of view to optimize culvert sizes (by reducing their dimensions).

5.4 Water hammer in the pumping system

A water hammer in the recycling network might be the consequence of a sudden pump stop.

Simulations have been run with the software FLOWMASTER, which is able to perform the calculation of water hammers in whatever hydraulic network.

5.4.1 Calculation hypothesis

5.4.1.1 Configuration of the recycling system

The indirect configuration for the recycling system was chosen (lower reservoir to Cocoli reservoir)

The configuration chosen for the calculations of the water hammer is the two lift lock system without WSB, which requires the highest discharge of the recycling pumping station. Thus the estimation of the water hammer will be maximized.

The longitudinal profile of the culvert corresponds to the topography.

The recycling system consists in 3 parallel and identical networks of 0 to 25 m^3/s each.

The characteristics of each network are as follows:

Total network discharge (m3/s)	*Total head loss (m)*	*Geo. Height (m)*			*Total manometric height (m)*		
		min	*aver.*	*max*	*min*	*aver.*	*max*
1	0.0	31	33.5	36	31.0	33.5	36.0
5	0.2	31	33.5	36	31.2	33.7	36.2
10	0.8	31	33.5	36	31.8	34.3	36.8
15	1.8	31	33.5	36	32.8	35.3	37.8
20	3.1	31	33.5	36	34.1	36.6	39.1
25	4.9	31	33.5	36	35.9	38.4	40.9

The corresponding graph is given in annex 13.

Max network represents the highest geometric difference between the levels of the Cocoli reservoir and the lower one.

5.4.1.2 Characteristics of the pumping station

The pumping station is designed for a maximal discharge of 75 m^3/s. It consists in 9 axial pumps of 8.33 m^3/s each.

The culvert diameter of each network is 3.2 m resulting in a maximum speed of 3 m/s.

The pipe diameter of each pump is 1.7 m.

The diameter of the butterfly valves and pipes surrounding the pump from the intake to the connection to the network is 1.7 m as well.

Pump characteristics were provided by ALSTOM. These characteristics only give the normal pump quadrant (annex 14.1); consequently, the characteristics have been completed for the turbine mode (head losses and resistant torque) using the characteristics of the standard radial electro-pump of FLOWMASTER that presents the closest specific speed (annex 14.2 and 14.3). The real characteristics should be provided for further studies by the manufacturer in order to perform detailed calculations.

The nominal functioning point of the pumps corresponds to their greatest efficiency:

Efficiency:	79% (= motor efficiency x pump efficiency)
Nominal pump efficiency	88 %
Motor efficiency	90%
Discharge:	8 m^3/s
TMH :	37.60 m
Motor inertia momentum:	1950 kg.m^2
Pump inertia momentum:	200 kg.m^2
Nominal friction torque:	9 602 N.m (10% of absorbed torque)
Nominal speed:	372 rpm
Specific speed:	69
Hydraulic power:	2 951 kW
Absorbed power:	3 735 kW
Absorbed torque:	96 015 N.m

The corresponding arrays are:

Electro-pump discharge (m3/s)	TMH (m)	Hydraulic power of electro-pump (kW)	Pump efficiency	Motor efficiency	Electric absorbed power (kW)	Electro-pump absorbed torque (N.m)
0	59.5	0	0	0.90	0	0
1	59	579	0.24	0.90	2 680	68 884
2	58	1 138	0.44	0.90	2 907	74 722
3	56.7	1 669	0.59	0.90	3 143	80 785
4	53.8	2 111	0.71	0.90	3 304	84 930
5	50.7	2 487	0.79	0.90	3 520	90 487
6	46.7	2 749	0.84	0.90	3 636	93 469
7	42.9	2 946	0.87	0.90	3 762	96 719
8	37.6	2 951	0.88	0.90	3 726	95 779
8.33	36.1	2 950	0.88	0.90	3 746	96 299
9	32.4	2 861	0.87	0.90	3 674	94 460

5.4.1.3 Hydraulic hypothesis

The average water level in the lower pond is -5.50 m PLD and +28.00 m PLD for the upper reservoir (Cocoli dam). The average output height is 33.50 m.

The functioning point of the electro-pumps is determined from the average height of output by crossing the network characteristics and the pumping station characteristics.

See graphs in annex 15.

Consequently, the maximum discharge for one electro-pump is 7.9 m^3/s and the maximum discharge for each network is 23.7 m^3/s. The maximum speed in the main culvert of the network is 2.9 m/s. Those figures match the former hypothesis.

5.4.1.4 Simulation principles

The celerity of the waves in the concrete culverts is taken as 1200 m/s (maximum value, according to Flowmaster's data tables).

Considering the previous functioning characteristics, the pumps are suddenly cutout. This will induce a pressure wave that will travel through the network thanks to the elasticity of the conduits.

Due to the inertia of the flow through the system from the lower pond to the upper reservoir, the cutout will first generate a negative pressure wave that will transform into a positive one by reflecting itself into the upper reservoir.

On every following graph, the cutout happens at Time = 10s.

The graph definition of the different nodes of the model is given in annex 16 (these are only nodes, the calculation points are much more numerous).

5.4.2 The simulations run

From the list above, only the most relevant calculations have been commented in the report.

Simulations without any protection	
1	valve opened, without cavitation simulation
2	valve opened, with cavitation simulation
3	Pre-dimensioned closure delay of the valve (30s)
4	Larger closure delay of the valve (60s)
Simulations of surge shafts	
5	Surge shaft - inclined - short (520 m downstream the station, point D) - diameter 3.2 m
6	Surge shaft - inclined - short (520 m downstream the station, point D) - diameter 6.6 m - without restriction at the connection
7	Surge shaft - inclined - short (520 m downstream the station, point D) - diameter 5.7 m - restriction at the connection point, diameter 1.65 m
8	Surge shaft - inclined - long (connected to te pumping station) - diameter 3.2 m
9	Surge shaft - inclined - long - optimized - diameter 2.2 m - linear closing of the valve (30s) - without equilibrium chamber
10	Surge shaft - inclined - long - optimized - diameter 2.2 m - linear closing of the valve (30s) - with equilibrium chamber 40 m²
11	Surge shaft - inclined - long - optimized - diameter 2.2 m - linear closing of the valve (30s) - with equilibrium chamber 100 m²
12	Surge shaft - inclined - long - optimized - diameter 2.2 m - 2 stage closing law (30s) - without equilibrium chamber
13	Surge shaft - inclined - long - optimized - diameter 2.2 m - variable volume/height law[1] - linear closing of the valve (30s) - without equilibrium chamber

[1] As Flowmaster does not allow to simulate a partly filled conduit, the inclined conduit was first simulated by a vertical surge shaft of same global volume. In the present simulation, the volume/height law was calculated in order to represent the slopes of the different parts of the inclined surge shaft.

14	Surge shaft - inclined - long - optimized - diameter 2.2 m - variable volume/height law - linear closing of the valve (30s) - with equilibrium chamber 100 m²
15	Surge shaft - inclined - long - optimized - diameter 2.2 m - variable volume/height law - new closure law of the valve (30s) - with equilibrium chamber 100 m²
16	Surge shaft - vertical - linear closing of the valve (30s) - without restriction at the connection
17	Surge shaft - vertical - linear closing of the valve (30s) - with restriction at the connection (3.2 m diameter)
Simulations of hydraulic accumulators	
18	Hydraulic accumulators alone
19	Hydraulic accumulator with a vertical surge shaft of little diameter (3.2 m)
20	Hydraulic accumulators with a vertical surge shaft of great section (100 m²)

5.4.3 Simulations of the water hammer without any protection (base scenario)

The first simulations were run in order to determine the pressure evolution and the possible occurrence of water hammer in the different parts of the recycling station, without any protection.

These simulations have no real physical signification since the resulting volumes of cavities are too high. The interest of these first series of calculations is to underline the necessity of protection devices.

The pre-dimensioned valve features a closing time of 30 sec. The pumps stop on their proper inertia.

The calculations show that:

- The pump stop is responsible for the vapor cavity that forms mainly in point A (annex 17.2) and reaches the volume of 530 m3. This cavity gets its maximum at T=40 sec and implodes at T = 80 sec. That collapse creates an over-pressure of about 40 bar in point A.
- In all the other points downstream (annexes 17.3 to 17.8), cavities appear at a much lower level, with volumes of 1 m^3 or less. The collapse of the main cavity around point A is responsible for over-pressures of about 40 bar during a few seconds in the whole downstream conduit
- Downstream the valve (annex 17.1), it can be observed that the discharge reverses after 3/4 sec till the entire closure. The pressure remains close to the initial one of 5 bar all the time long, till the collapse of the cavity occurs; then the pressure raises up to 70 bar, which is much more than in the former simulation, due to the presence of the closed valve when the cavity collapses.

5.4.4 Simulations of the water hammer with a protection of the network

5.4.4.1 Types of protections envisaged

The different types of protections are:

- Surge shaft : this device permits to transform the pressure wave into a mass movement, thus compensating the negative effects on the network elements: cavitation on the pump, depression on the pipes and on the butterfly valve. This device looks like a great vertical chimney that has to be placed very near from the departure point of the pressure perturbation and has to be high enough to absorb the level oscillations that appear without overflowing.

- Inclined surge shaft : its principle is approximately the same as the previous one :it consists in a conduit with a reduced diameter, thus resulting less efficient than a chimney but much cheaper

- Hydraulic accumulator: it consists in balloons filled with air, placed immediately downstream the valve. Their proper volume and air pressure allows to compensate the variations of pressure due to the cutout

- Wheel of inertia: this device is implemented on each pump in order to increase its inertia momentum, thus reducing the effect of the cutout (positive torque loss) due to a greater stop delay.

5.4.4.2 General considerations about protections tested

Surge shafts.

In order to first test an alternative to a high surge shaft immediately downstream the recycling station, the solution of a horizontal / inclined surge shaft was first tested. This solution was recently adopted at Potrerillos (Argentina).

Choosing the connecting point of the conduit according to the shortest possible conduit length will minimize the investment. See plan in appendix 25.

The main drawback of this solution is to give little protection to the first 560 meters of the main conduit. The depression wave is still provoked by the rapid pump stop and will travel as far as the connection point with the chimney where it will be compensated thanks to the chimney's emptying.

The downstream part of the conduit is protected but not the upstream one containing the most fragile devices.

Thus long shafts, connected as close as possible to the pumps and the valves have also been tested.

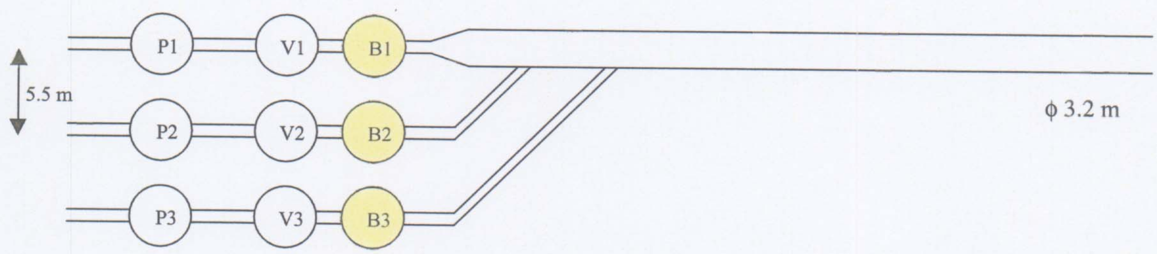
Last, a vertical surge shaft placed 50 m downstream the pumping station in point A, was tested.

Hydraulic accumulators

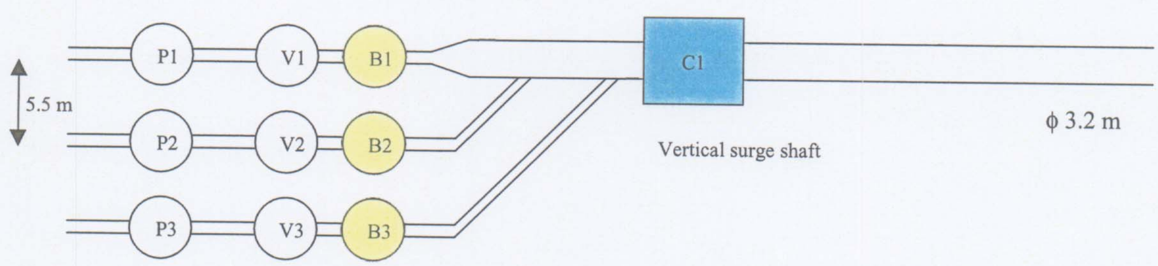
Considering the length of the culvert, the great discharges and the velocities, the hydraulic accumulators have been tested both alone as a means of protection or together with a surge shaft.

The following cases have been tested:

Case 1: hydraulic accumulators only



Case 2: Hydraulic accumulators with a vertical surge shaft connected to point A



The hydraulic accumulators include a rubber bladder in order to prevent from air dissolution in the water, that would oblige to implement a costly air compressor system to permanently reestablish the adequate air volume in the accumulator.

Wheel of inertia

The addition of wheels of inertia to the pumps is another way to reduce the depressions in the exhaust culvert by reducing the instantaneous variation of energy transmitted to the flow. It is however only a complementary means to another device: due to the characteristics of the station, the wheels of inertia should be very heavy to cancel the water hammer by their own.

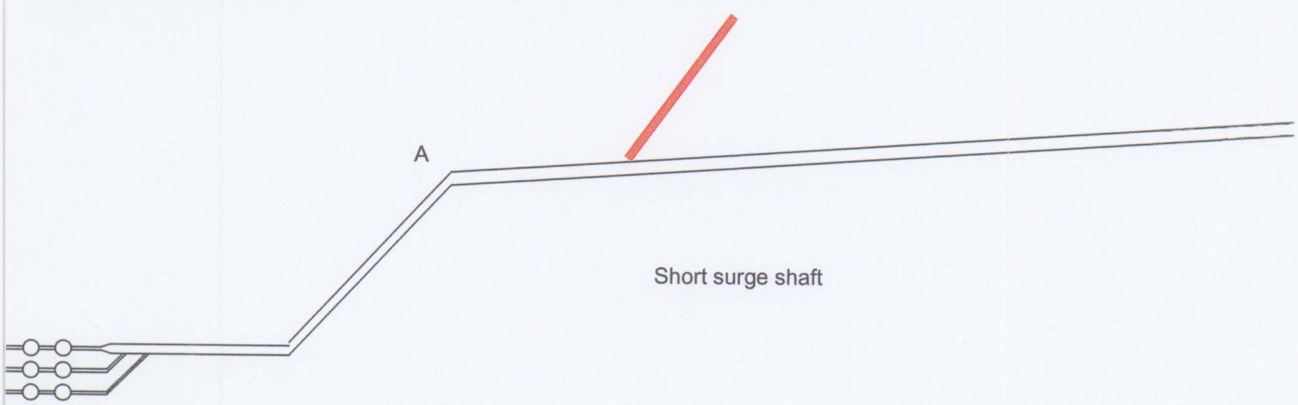
Moreover, because of the great number of pumps into the station (9 in the chosen configuration of a two-lift lock), the room left between the pumps wouldn't be sufficient to allow the implementation of big inertia wheels with a sufficient momentum.

For those reasons, this system has not been analyzed yet.

5.4.5 Simulations of the protected networks

5.4.5.1 Short surge shaft -optimization

The short surge shaft has an equivalent diameter of 5.7 m, and is connected 560 m downstream of the valve and lying on the hill situated on the west side of the conduit.



Its optimization, has consisted in creating a hydraulic head loss at the connecting point between the network and the surge shaft, by means of a restriction that allows to reduce the surge shaft diameter without provoking the total emptying of the surge, the formation of a vapor cavity at the connection and providing the protection of the whole downstream culvert (max acceptable pressure and no cavitation). The head loss corresponding to this brutal restriction is modeled as follows:

$hj=K*V^2/2g$

with $K = 1$ in the direction culvert – surge shaft
$K' = 0.5$ in the direction surge shaft– culvert
V = flow speed through the contraction

The dimensions of the short optimized surge shaft are:

- diameter: 5.72 m ($S = 25.71\ m^2$ and $V = 1800\ m3$) instead of 6.60 m (first calculations, $S = 34.3\ m^2$ and $V = 2400m3$)

- restriction diameter: 1.65 m ($s = 2.14m^2$)

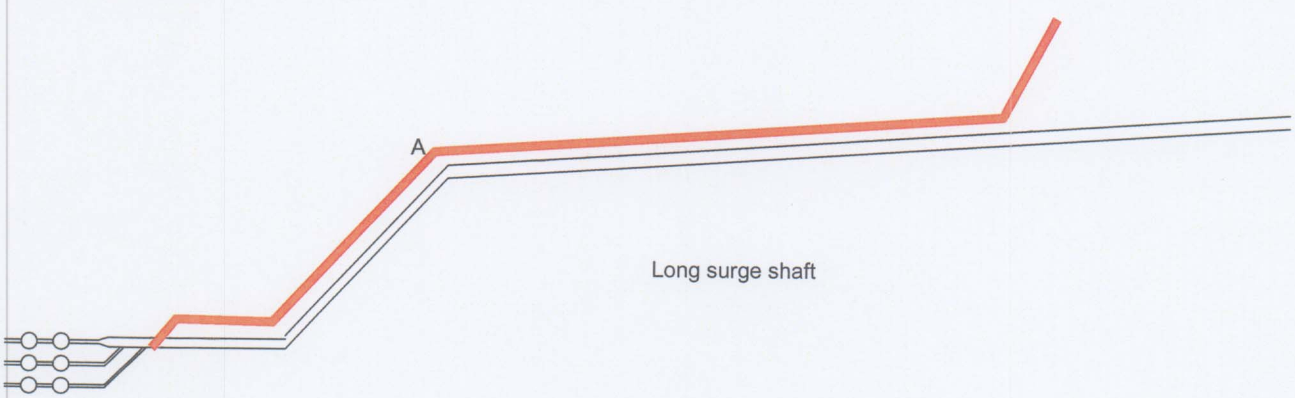
The results show that this optimization allows to reduce the pressure that reaches 15 bar (annex 18.2). This cavity collapse at t = 28 sec, but it is however responsible for unacceptable pressure perturbations in the upstream part of the culvert (annexes 18.3 to 18.8).

5.4.5.2 Long surge shaft -optimization

In order to protect the upstream part of the conduit, the pump and the valve, a surge shaft connected directly after the valves (close to the connecting point of the 3 exhaust culverts) was tested.

Its optimization after a few simulations has consisted in reducing its diameter while verifying that it still protected the whole culvert (max acceptable pressure and no cavitation).

No additional restriction is needed at the entrance of the surge shaft as its lower diameter creates its proper head loss.



The dimensions of the long optimized surge shaft are:

diameter: 2.20 m ($S = 3.8\ m^2$ and $V = 1900\ m3$)

The simulations show that good results are obtained with this protection, results are as good as in the case of a long surge shaft of 3.2 m diameter.

The results are shown in annex 19. The water level in the surge shaft decreases from 38 to 23 m, then increases rapidly till 75 m which is much too high and would require an excessively long pipe. The pressure at the connection reaches almost 8 bar, which is high towards the penstock resistance.

The low-amplitude remaining oscillations will not cause any damage to the pumps, nor to the valves or the culverts.

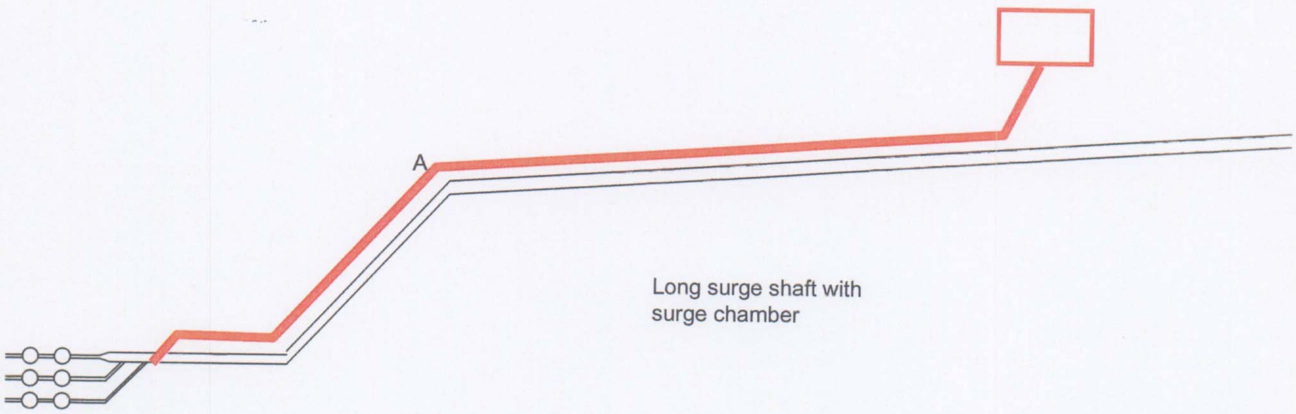
According to those results, a configuration was tested with a surge chamber placed at the top of the surge shaft in order to reduce the high levels.

5.4.5.3 Long surge shaft - with surge chamber

The former protection was improved thanks to a surge chamber placed at the end of the surge shaft. The characteristics of the surge chamber are:

- surface: 100 m^2 per each surge shaft
- height: 9 m
- bottom level: 32 m PLD
- roof level: 41 m PLD

The bottom level corresponds to the permanent water level while the pumping station functions. The height has been calculated considering the maximum water oscillations produced by the pump cutout.



The results in annex 20 demonstrate the efficiency of the surge chamber: the levels reach a maximum of 47 m PLD instead of 75 m PLD and the max pressure is 5.6 bar.

The minimum pressure downstream the valve reaches 3.3 bar.

The discharge graph shows that almost one fourth of the surge shaft's volume is lost through the valve as it flows back, which leads to the over dimensioning of the surge shaft.

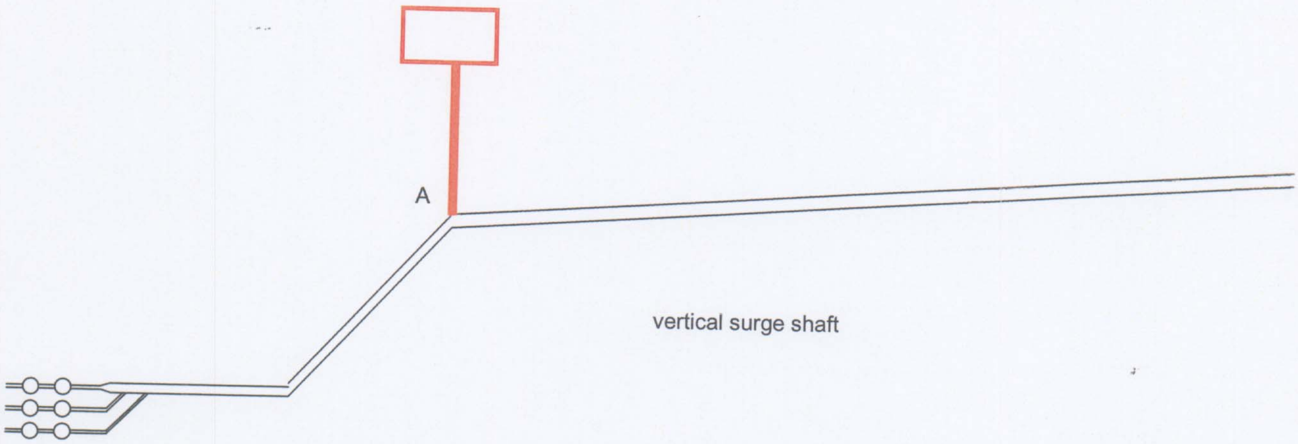
5.4.5.4 Long surge shaft - with surge chamber - Valve law modification

In order to try to reduce the volume lost through the penstock, a new closing diagram was tested on the former protection; it allows to close the valve quicker at the beginning, getting an opening ratio of 30% in 10 seconds and shutting down in another 20 seconds.

The graphs in annex 21 show little differences with the former configuration, in terms of pressures and discharges.

5.4.5.5 Vertical surge shaft

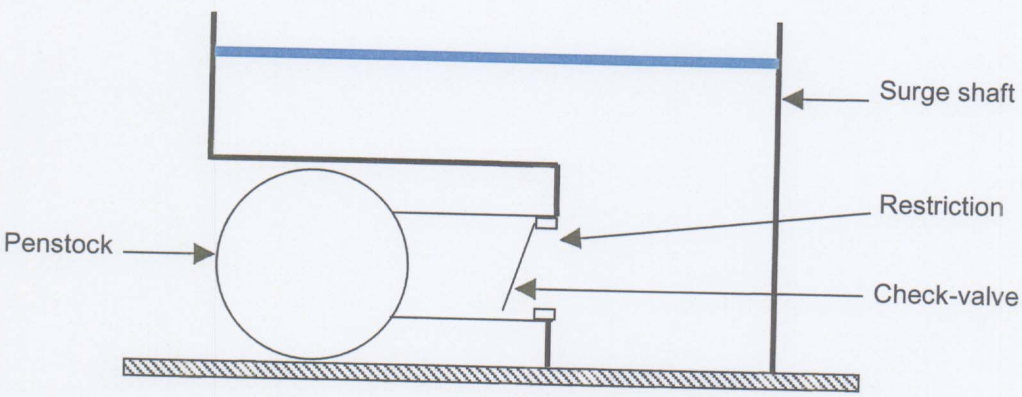
In order to compare costs and efficiency with the former protections, a vertical surge shaft was tested. It was placed on point A in order to reduce its height and because there wouldn't be enough space close to the pumping station.



The characteristics of the surge shaft are:

- surface: 100 m² (10x10 m or 11.3 m diameter)
- height: 21 m
- connection point: A, level 13.90 m.

The shaft is connected to the main penstock through a restriction of 8.8 m² (2 squares of 2.1x2.1 m).



The height of the shaft was calculated in order to avoid the overflow due to the mass oscillations created by the pump cutout. The maximum level of the water level being 20.70 (36.2 PLD), the roof of the surge shaft should be at least 21.70 (37.2 PLD).

The results in annex 22 show that the surge shaft drains in the conduit during the first 100 seconds, the final water level reaches 1 m above the bottom of the surge shaft. The consumed volume is 1500 m^3.

The pressure at point A drops gently from 2.6 to 1.2 bar while at the pump a quick drop can be observed in the first second, from 4.6 bar down to 2.7 bar. The pressure at the pump reaches a peak of 5.3 bar and decreases till 4.4 bar where it follows the pressure dictated by the surge shaft.

The oscillations observed at the pump after 40 seconds are due to the closure of the butterfly valve

5.4.5.6 Hydraulic accumulator

In order to protect the very near perimeter of the pumps, a simulation was done on the basis of the vertical short surge shaft completed with hydraulic accumulators (one per pump) immediately downstream the valves. This solution was implemented in Morocco for a desalinization plant (see annex 26).

Placing the vertical surge shaft on point A would reduce its height and its cost, while Hydraulic accumulators placed immediately after the valves would protect the mechanical devices and the conduit till its connection with the surge shaft.

In order not to loose too much volume though the valves during their closure, which would oblige to over-dimension the surge shaft and the accumulators, the consultant in hydraulic protections CHARLATTE recommends that a check-valve should be placed between each pump and its valve. This implementation is common in pumping stations and has proven very efficient.

CLASAR check-valve

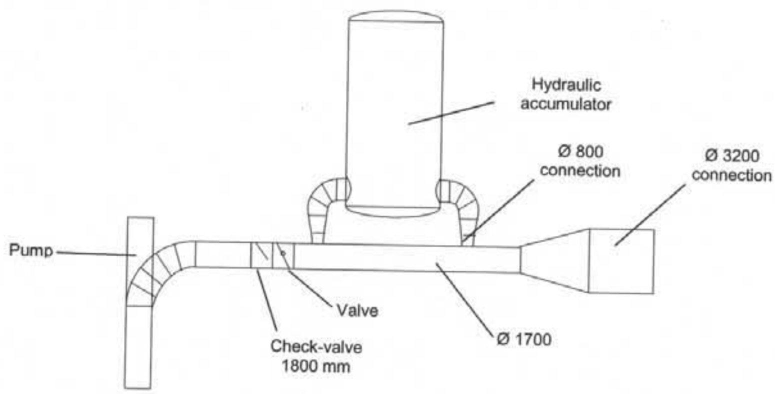
The check-valves would add some hydraulic losses, about 1 m, but would prevent from having:

- a too great volume lost through the pumps during the valve closure
- the pumps rotating backwards, which could be damaging (to be specified with Alstom)

Implementing check-valves would also allow:

- either to reduce the motorization of the butterfly valve: without any check-valve, the butterfly valve should resist to a quick closure that would produce another water hammer in the pump
- or to replace it with a simple valve just for the maintenance of the pumps and the check-valves

Charlatte recommends the CLASAR type of check-valve (TYCO-SAPAG), which allows to cut the discharge very quickly without any check-valve blow. Its diameter could be 1800 mm in order to fit the 1700 mm conduit (see annex 28 for more details).



The implementations of the short and long surge shafts are shown in annex 25.

Hydraulic accumulators alone.

The simulations made indicate that to prevent from any water hammer, it is necessary to implement a volume of 500 m^3 for each pump (1500 m^3 for each network). This volume would be reached thanks to 4 accumulators of 125 m^3 each, connected to the exhaust conduit of each pump. The dimensions of each accumulator would be 3100 mm diameter and 16 m length, which is probably too much to fit the available room.

The results in annex 23.1 show that in the very first moments, the pressure downstream the pump at the connection with the accumulators decreases similarly to the pump's side, till the closure of the check-valve. From that point (pressure of 26 m of water), the energy of the accumulators is provided to the conduit and the minimum pressure of 10 m Water Column is reached after 45 sec. Then the pressure in the conduit raises till its hydrostatic level, while the internal pressure of the accumulators raises quietly thanks to the check-valve place at their connection with the conduits: this device prevents from having oscillations due to the exchange between the conduit and the accumulators during a while.

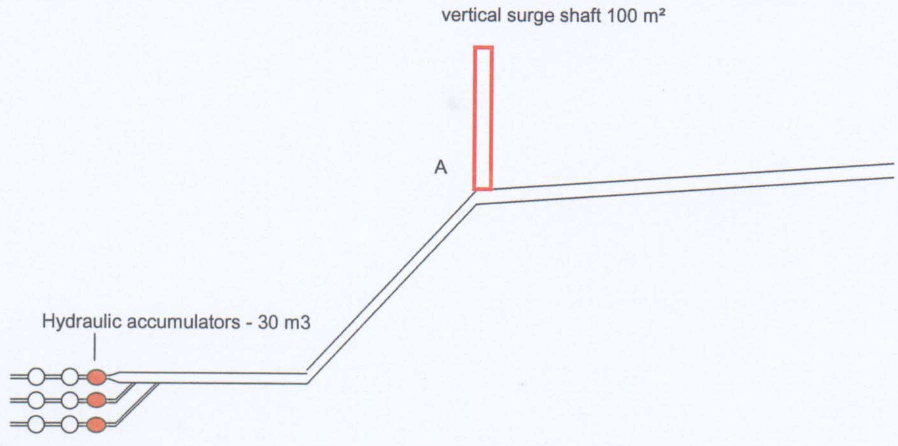
The total volume given by the accumulators for a network of 3 pumps is 1100 m^3 (see annex 23.2).

The longitudinal pressure profile on annex 23.3 show that, supposing that the minimum absolute pressure of 0.2 bar could be acceptable (if the conduit would be made of steel or concrete with sheet heart), that is to say -8 m WC beyond the roof of the conduit, this minimum pressure would almost be reached in point A. It demonstrates that this dimensioning of the hydraulic accumulators is a minimum: the volume of the accumulators should be greater than 1500 m^3 to increase the security.

The annex 23.4 shows the evolution of the discharges in the conduit and at the outlet of the hydraulic accumulators.

Hydraulic accumulators in combination with vertical surge shafts.

The protection of the vertical surge shaft can be bettered in the very near perimeter of the pumps by the adjunction of hydraulic accumulators which react instantaneously after the pump cutout.



The graphs of the annex 24.1 show that the vertical surge shaft, being 50 m downstream the pumps, has a reaction delay of around 1 second. The hydraulic accumulators react instantly and allow the pressure in the pumps not to decrease under 21.5 m of water column at time = 1.5 sec. At that moment, the check-valve closes and the pressure goes on decreasing in the pump while the suction pipe empties. On the other side, downstream the pump, the accumulators release their energy , maintaining a pressure superior to 19 m. The minimum is reached as the surge shaft relays the accumulator to provide the conduit with water.

The general graph of the pressures (annex 24.2) shows that within the first 30 seconds, potential energy is exchanged between the accumulators and the surge shaft. After 30 seconds, the mass movements in the conduit make the pressures in both devices vary similarly.

The oscillations traduce the fact that no check-valve at the entrance of the surge shaft nor at the entrance of the accumulators have been implemented. This would not be convenient only in the case the pump or the whole pumping station would have to be started again while the pressures in the network would still be oscillating.

To prevent this phenomenon, check-valves should be implemented at the connection of the surge shaft and at the connections of the accumulators.

The graph of the discharges (annex 24.3) show that the accumulators provide up o 20 m^3/s and the surge shaft up to 32 m^3/s.

The volume of the compressed gas in the accumulators vary from 51 m^3 for the network of 3 pumps to 71 m^3 to absorb the water hammer in the first instants. (annex 24.4).

The longitudinal profile in annex 24.5 show that the piezometric height keep superior to the roof level of the conduit, which means that this dimensioning is very safe towards the risk of under-pressures in the conduit. The upper level reached in the surge shaft is around 35 m but could be reduced thanks to check-valves at its connection.

The annex 26 shows the configuration of a similar protection system used in Morocco, as a combination of both a hydraulic accumulator and a vertical surge shaft.

The annex 27 shows the pumping station of Marsanne, France, equipped with 2 accumulators of 30 m3 each. The second photograph shows an accumulator of 75 m^3.

5.4.6 Water hammer protection : conclusion

The implementation of check-valves at the outlet of the pumps should be used in order not to allow the discharge to flow backwards and to damage the pumps. The configuration of a vertical surge shaft and hydraulic accumulators would then provide a perfect protection of the whole network

We think this is the best technico-economical solution for the water hammer protection. This leads to the following investments.

5.4.7 Investments

The water hammer protection consists in :

- CLASAR check valves, located right after the pumps. Description and photos are in the annexes
- Hydraulic accumulators, located downstream the CLASAR check valves
- Vertical surge tanks located as close as possible to the pumping station, on each penstock. The ground level was determined in order to reduce the height of the shafts, thus the investments. The project foresees to locate them at the slope change in the longitudinal profile of the penstocks (see drawings).

  The vertical surge shafts consist in concrete boxes embedded in thick slabs. The structure of the walls are strengthened by grids of beams in order to reduce their thickness. The penstocks run through the shafts. Simple movable valves with holes are located at the connection to the shafts.

The table below shows the amount of the investments, for each configuration

5.4.8 Recommendations for further studies

- The exact characteristics of the pumps on the 4 quadrants (pumps + turbine) would be required to precise the simulations

- The nominal power of the pumps was chosen according to the minimum geometric head. Consequently, the discharge of the station doesn't reach 25 m^3/s for the average geometric head of 33.5 m. The check-valves placed at the outlet of the pumps in the configurations including hydraulic accumulators, are responsible for an additional head loss that will have to be considered in the final dimensioning of the pumps, in order to increase the discharge from 23.55 m^3/s to 25 m^3/s for a network of 3 pumps.

INDIRECT	3 lifts + 9 WSB			3 lifts + no WSB			2 lifts + 4 WSB			2 lifts + no WSB		
	1 network	2 networks	3 networks	1 network	2 networks	3 networks	1 network	2 networks	3 networks	1 network	2 networks	3 networks
CLASAR check valves	100 000	200 000	300 000	210 000	420 000	630 000	200 000	400 000	600 000	405 000	810 000	1 215 000
hydraulic accumulators	56 500	113 000	169 500	158 100	316 200	474 300	109 000	218 000	327 000	180 000	360 000	540 000
surg tanks	205 779	364 228	524 736	311 267	550 943	793 731	268 716	475 628	685 227	375 901	665 346	958 549
valves and links	1 908	3 816	5 724	4 953	9 906	14 859	3 116	6 232	9 348	6 615	13 230	19 845
TOTAL	**364 187**	**681 044**	**999 960**	**684 320**	**1 297 049**	**1 912 890**	**580 832**	**1 099 860**	**1 621 575**	**967 516**	**1 848 576**	**2 733 394**

SEMI DIRECT	3 lifts + 9 WSB			3 lifts + no WSB			2 lifts + 4 WSB			2 lifts + no WSB		
	1 network	2 networks	3 networks	1 network	2 networks	3 networks	1 network	2 networks	3 networks	1 network	2 networks	3 networks
CLASAR check valves	100 000	200 000	300 000	210 000	420 000	630 000	200 000	400 000	600 000	405 000	810 000	1 215 000
hydraulic accumulators	56 500	113 000	169 500	158 100	316 200	474 300	109 000	218 000	327 000	180 000	360 000	540 000
surg tanks	193 330	342 194	492 991	299 267	529 703	763 132	249 816	442 175	637 031	344 748	610 203	879 106
valves and links	1 908	3 816	5 724	4 953	9 906	14 859	3 116	6 232	9 348	6 615	13 230	19 845
TOTAL	**351 738**	**659 010**	**968 215**	**672 320**	**1 275 809**	**1 882 291**	**561 932**	**1 066 407**	**1 573 379**	**936 363**	**1 793 433**	**2 653 951**

DIRECT	3 lifts + 9 WSB			3 lifts + no WSB			2 lifts + 4 WSB			2 lifts + no WSB		
	1 network	2 networks	3 networks	1 network	2 networks	3 networks	1 network	2 networks	3 networks	1 network	2 networks	3 networks
CLASAR check valves	100 000	200 000	300 000	210 000	420 000	630 000	200 000	400 000	600 000	405 000	810 000	1 215 000
hydraulic accumulators	56 500	113 000	169 500	158 100	316 200	474 300	109 000	218 000	327 000	180 000	360 000	540 000
surg tanks	318 634	563 983	812 517	484 928	858 323	1 236 566	408 669	723 345	1 042 107	558 871	989 202	1 425 122
valves and links	1 908	3 816	5 724	4 953	9 906	14 859	3 116	6 232	9 348	6 615	13 230	19 845
TOTAL	**477 042**	**880 799**	**1 287 741**	**857 981**	**1 604 429**	**2 355 725**	**720 785**	**1 347 577**	**1 978 455**	**1 150 486**	**2 172 432**	**3 199 967**

6. CONCLUSIONS

CPP was asked to study several water recycling systems in order to save the water that is otherwise spilled into the Pacific Ocean.

The present report demonstrates that recycling of the spilled water is technically feasible from a hydraulic point of view. The pumping system, added to a multiple lift lock system with or without water saving basins, allows to save almost all the water required to operate the locks.

Three kinds of pumping system have been designed :

- **Direct pumping system** : the water is pumped from Pacific Ocean to Gatun Lake. It is the simplest system yet it is also the worst in terms of the salt intrusion in the Gatun Lake since the water pumped is salt water.
- **Semi direct pumping system** : the water is pumped from a lower reservoir to the Gatun Lake. The reservoir is situated on the west side of the locks. Its maximum water level is set below ocean level in order to be able to recover all the water spilled from the lower lock. This system also injects brackish water in Gatun Lake.
- **Pond to pond pumping system** : the water is pumped from a lower reservoir to an upper reservoir. The reservoirs are connected to the locks filling and emptying system by means of conduits. The lower reservoir is situated on the west side of the locks. Its maximum water level is set below ocean level in order to be able to recover all the water spilled from the lower lock. The upper reservoir is actually made by a dam on the Cocoli river. This system is the best to prevent salt intrusion in Gatun Lake.

A first stage consisted in the calculation of the water volumes spilled downstream and pumping flow rates for several daily traffic levels for the 2-lift and 3-lift lock system with and without water saving basins. These calculations have been carried out with the Consultant Software already used in former studies.

Then the pumping system (pump, culvert, valve, grid) has been designed by means of the Flowmaster software. Culvert arrangements have been studied to minimize excavation works.

Simulations have been performed to calculate the network head losses for several flow rates. The results of the simulations have been used to set the network curves.

Tests have also been carried out to evaluate the time to fill the upper lock chamber from the upper reservoir and the time to empty the lower lock chamber to the lower reservoir.

The conclusion is that the pumping system will not reduce the daily number of passing ships since filling and emptying times are shorter than those without pumping system.

The problem of water hammer in the conduits has also been investigated. The calculations performed with Flowmaster have highlighted the necessity to implement a protecting system to avoid creating unacceptable cavitation inside the pumps, unacceptable vapor cavities inside the culvert and unacceptable pressure variations when a pump cutout occurs.

The protection can be reached either by a surge shaft connected immediately downstream the valves (implemented after the pumps) or by a shorter surge shaft connected to the main culvert 560 m downstream the valves, in order to reduce its length, together with hydraulic accumulators with consequent volumes placed immediately downstream the valves.