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Rapport de mission CIRAD/PCSI

Hydraulic study of the Hadejia Valley Irrigation Project North Main Canal, Nigeria

Trip report 27/9 to 09/10/2000

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Abbreviations

ABU	Ahmadu Bello University, Zaria
CIRAD	Centre International de Recherche en Agronomie de Développement
CR	Cross Regulator
DC	Distributary Canal
HJRBDA	Hadejia Jama'are River Basin Development Authority, Kano
HVIP	Hadejia Valley Irrigation Project, Hadejia
NAERLS	National Agricultural Extension and Research Liaison Service, Zaria
PM	Project Manager (HVIP, Hadejia)
WUA	Water Users Association
SIC	Simulation of Irrigation Canals
STO	Sector Turn Out

1. Introduction

1.1 Hadejia Valley Irrigation Project

The Hadejia Valley Irrigation Project is located in the North of Nigeria, 200 km east of Kano. It is the second largest irrigation scheme under the responsibility of the Hadejia Jama'are River Basin Authority (HJRBDA) after the Kano River Project. A total of 12.500 ha are planned under this scheme, but so far only 2150 ha have been developed. In addition, farmers are growing an estimated 1000 ha of irrigated crops by taking water directly from the main canal of the project.
See Kuper (2000) for more information on the irrigation project's characteristics.

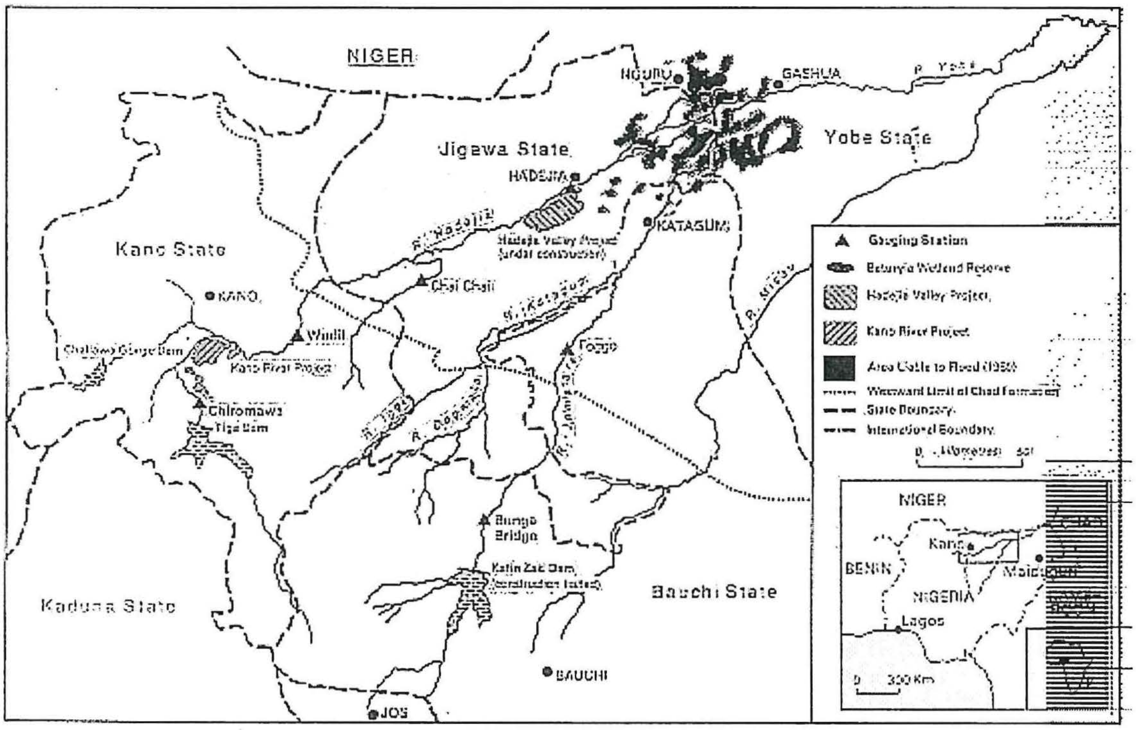

Figure 1: Location of the Hadejia Valley Irrigation Project in Nigeria

The irrigation system is taking the water from a dam on the Hadejia river, whose design capacity was estimated to be 11.4 million cubic meters. A feeder canal (FC) carries the water to the point where the two main canals separate, the North Main Division Works (NMDW) and the South Main Division Works (SMDW). Only the North Main Canal is completed, the South Main Canal is not finished. We focused our study on the North Main Canal, which is 27 km long, and a design discharge of 14.9 m^3/s. The Feeder Canal is 2.8 km long, with a design discharge of 29.8 m^3/s. See figure 2 for a sketch of the hydraulic system, with the cross structures and the Sector Turn Outs (STO) feeding the Distributary Canals (DC).

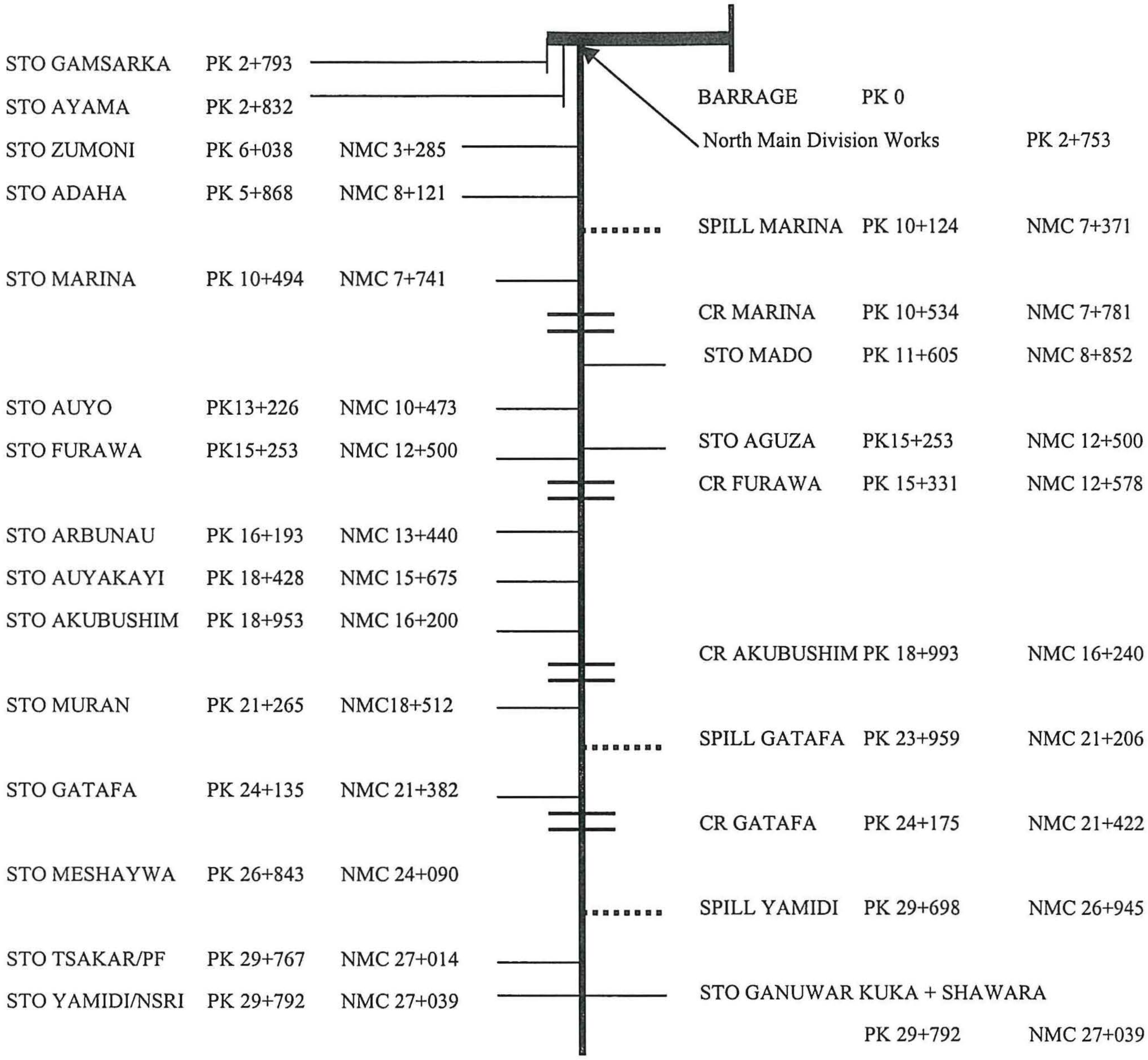

Figure 2 : Sketch of the North Main Canal hydraulic system (Quirion, 2000)

Only 8 sectors are equipped (design discharges are given in brackets when available):

- On the Feeder Canal: GAMSARKA and AYAMA sectors,
- On the North Main Canal: ZUMONI (0.951 m^3/s), ADAHA (0.645 m^3/s), MARINA (0.973 m^3/s) and AUYO (0.737 m^3/s) sectors upstream, YAMIDI and GANUWAR KUKA downstream.

The operation of the system is designed to be based on a continuous supply of water to the main canal. Irrigation is conducted during the day time, and water is stored during the night in Night Storage Reservoirs (NSR), which are used in the day time to supply secondary channels.

The canal is equipped with 4 spillways, one for the feeder canal, and 3 for the north main canal, two of which were in function during the 2 days experiment conducted during this mission. They provide a useful safety margin in the operation of the system.

There is no O&M manual for this canal, and no rating curves are available with the system managers for the hydraulic structures of the system.

The overall system is in a rather poor shape, because of the lack of maintenance, especially at the main canal level. The banks of the barrage in particular are at some places very deteriorated by erosion and would need some effective maintenance and rehabilitation.

1.2 Documentation on earlier hydraulic studies

There has apparently been no previous hydraulic measurement campaign on this canal. Available hydraulic studies (done by BRL, HASKONING and ENPLAN) were all based on design data, i.e. assuming a given value for Manning coefficients and discharge coefficients.

The functioning of automatic control gates such as the cross regulators in HVIP (flat back, or Begemann gates and round back or Vlugter gates) was studied by Vlugter in 1940, and is regaining interest in the research community, as attested by some recent publications (Burt et al. 2000, de Graaf 1998, Raemy et al. 1998).

1.3 Objectives of the mission

The objectives of the mission were as follows:
- To prepare and conduct a hydraulic measurement campaign on the Main Canal system in collaboration with NAERLS and HVIP staff,
- To calibrate a hydraulic simulation model (SIC model) with the collected data,
- To provide the system managers with reliable discharge ratings,
- To participate in the training of NAERLS staff for conducting hydraulic studies (including modelling and analysis) on an irrigation system.

The mission was carried out in the context of the common platform on irrigation systems research of CEMAGREF, CIRAD and IRD (PCSI). The mission coincided with the mission of Kuper (2000).

2. Measurement campaign

2.1 Objectives

The objectives of the hydraulic measurement campaign were as follows:
- To train HVIP/NAERLS hydraulic staff to do discharge measurements, water level measurements, establishing rating curves for hydraulic structures,
- To establish a steady state in the canal with the STO's closed,
- To propagate a wave in the canal to study its response in unsteady state,
- To study the functioning of the automatic control gates at cross regulators in real-life conditions,
- To get accurate data to enable the calibration of SIC model (water levels, discharge, gate openings).

The measurement campaign protocol is detailed in annex 1.

2.2 Discharge measurements

Five discharge measurements were conducted during the field campaign, using a current meter OTT C31 provided by Cemagref (Hydraulics Division, Lyon).
A first measurement was done on Saturday 30/09/2000 at Zumoni bridge, 3285 m after the NMDW, giving a value of 4.5 m3/s. Neglecting the seepage, this gave a first evaluation of the discharge coefficient for the first structure of the NMDW (the measuring gates were completely opened): $C_d=0.68$.

For the equation:

$$Q = C_d L w \sqrt{2g(h_2 - h_1)}$$

where Q is the discharge, L the width, w the opening, g the gravitational acceleration, h_1 the upstream water level, and h_2 the downstream water level.
Three other discharge measurements were done on Monday 02/10/2000, at Zumoni bridge, Mado bridge and Meshaywa bridge. (see section 2.4 for results).

Two discharge measurements were done on Tuesday 03/10/2000, at Zumoni bridge and Meshaywa bridge.

2.3 Water levels measurements

Water levels were recorded every 20 minutes from 07:00 to 19:00 Monday 02/10/2000 and Tuesday 03/10/2000 at 6 locations along the canal: at each cross structure (including the NMDW), upstream and downstream water levels were recorded and the downstream end of the canal.

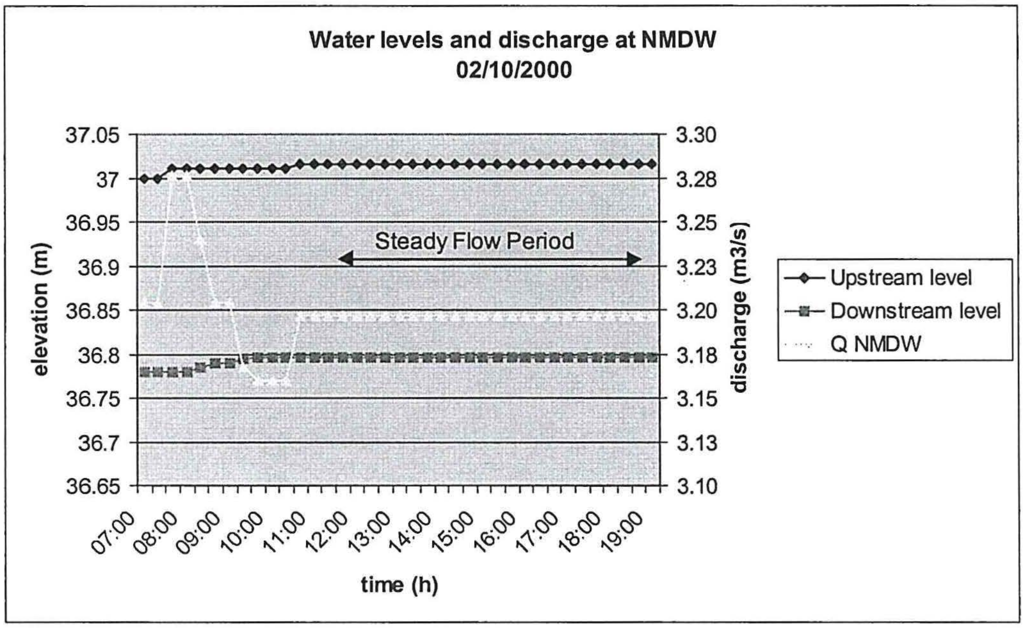

Figure 3: Water levels and discharge at the NMDW, 02/10/2000

As can be seen from the Figure 3, the discharge flowing into the system was constant from 11:00 to 19:00. Looking at the other locations shows that the system was in steady flow from 12:00 to 18:00. This steady flow period was used in order to calibrate the hydraulic simulation model SIC.

The second day, a wave was created into the system in order to be able to evaluate the time lags of the canal and to validate the steady flow simulation in unsteady state.
This wave was provoked by successively opening and closing the gates at the barrage (opening took place at 7:00, and closing at 11:15). This corresponds to a positive step of discharge of about 1 m^3/s and a negative step of 1 m^3/s.

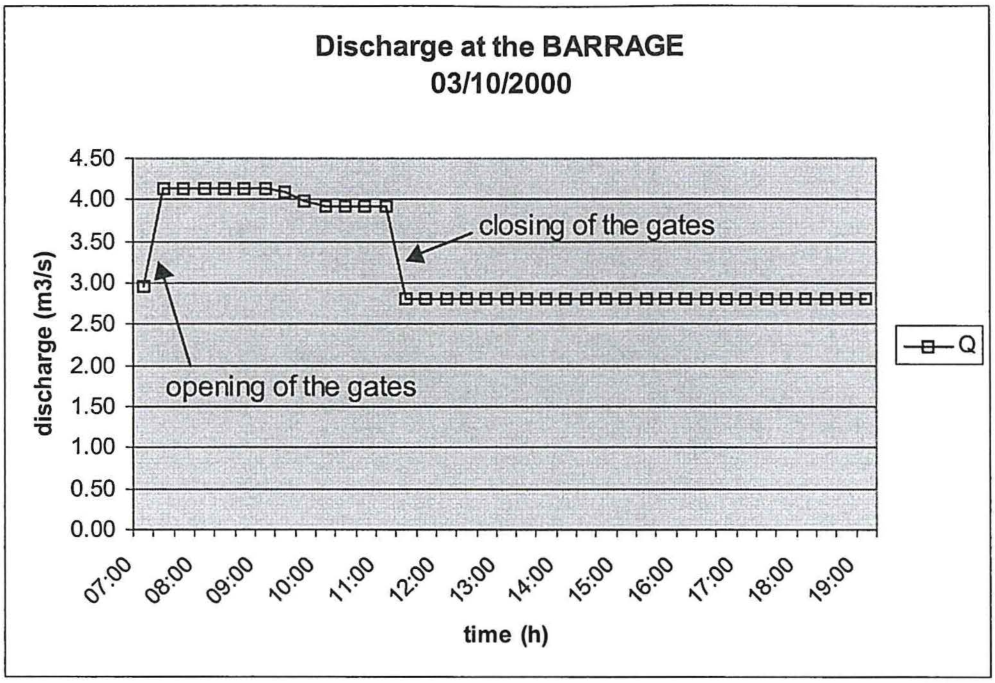

Figure 4: Discharge at the Barrage, 03/10/2000

2.4 Outputs of the campaign

- Hands-on hydraulic training (hydraulic staff HVIP/NAERLS)

The hydraulic campaign was done in collaboration with NAERLS and HVIP. The data collection was performed by HVIP technical staff and the discharge measurements and supervision were done by X. Litrico (Cemagref), M.K. Othman (NAERLS) and L.H. Umar (HVIP).

- Propagation of a wave

The second day, a wave was created at the barrage in order to be able to evaluate the time-lags of the main canal system. The following time-lags were determined from the data collected:

Table 1: Average time-lags determined from collected data of 03/10/2000

Reaches	Positive step time-lag	Negative step time-lag
Barrage-NMDW	1h	40'
NMDW-Marina	1h	1h20'
Marina-Furawa	1h	40'
Furawa-Akubushim	2h20'	2h
Akubushim-Gatafa	20'	20'
Gatafa-Yamidi	-	-

The time-lags obtained should be checked on other experiments, as the variation in upstream discharge was not large enough to get accurate results. However, one has estimates of the time-lag of the system in the condition of the experiment (all STOs closed).
We tried to find a correlation between these time-lags and other physical characteristics of the system (given in Table 2), but no significant result was obtained. Maybe the influence of the automatic gates at the Cross Regulators can explain this.

Table 2: Distances and average slope

Reaches	Length (m)	Slope
Barrage-NMDW	2810	$3.56\ 10^{-6}$
NMDW-Marina	7771	$3.35\ 10^{-5}$
Marina-Furawa	4797	$2.92\ 10^{-5}$
Furawa-Akubushim	3662	$3.55\ 10^{-5}$
Akubushim-Gatafa	5182	$5.02\ 10^{-5}$
Gatafa-Yamidi	5602	$5.70\ 10^{-5}$

- Discharge coefficients at structures (rating curves structures)

Barrage

Using the discharge measured at Zumoni bridge the 02/10/2000 at 9:30 and the corresponding water levels and openings at the barrage, the discharge coefficient for this structure is calculated as $C_d = 0.61$, for a width of gate equal to 2.16 m (this corresponds to a $C_d = 0.7$ for a width of gate equal to 1.83 m, value given by the constructor).

Some specific discharge measurements should be done on the Feeder Canal in order to calibrate precisely the structure at the barrage. As this structure seems to be submerged, it is also important for future measurement campaigns to read the levels and the gate openings at this location.

The fact that the feeder inlet is operated under submerged conditions has implications for the management of the system: the choice of an opening of the gates at the barrage does not necessarily provide the required discharge, as there is an interaction between the inlet op the Feeder and the NMDW. The Feeder Canal is rather short (2810 m) and the structure at the Barrage is influenced by the backwater curve of the Feeder Canal. The operator should therefore wait until the modified opening at the Barrage reaches the NMDW and the backwater curve is installed. He should then modify if necessary the opening in order to get the required discharge, according to the upstream end downstream levels. This point should be checked by accurate measurements of the structure dimensions.

NMDW

Using the discharge measured at Zumoni bridge the 30/09/2000 at 10:15 and the corresponding water levels and openings at the NMDW, the discharge coefficient for this structure is calculated as $C_d = 0.68$, for a width of gate equal to 1.89 m.

The NMDW consists of two hydraulic structures in series, as sketched in Figure 5. In this case, we opened completely the measuring gates, in order to calibrate the first hydraulic structure (regulating gates). The problem is that there is no gauge installed between the two structures, to measure water level h_2. Only the two other gauges are installed, enabling to measure h_1 and h_3.

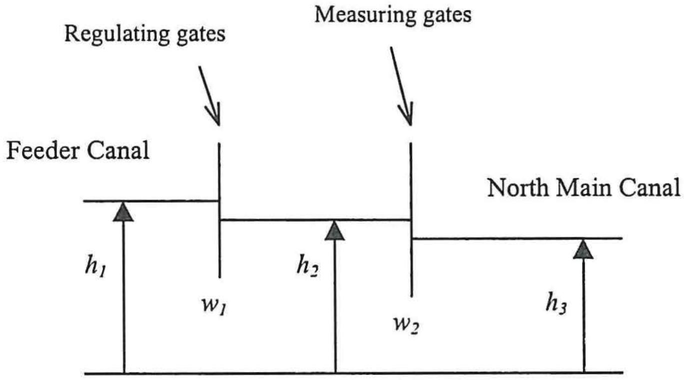

Figure 5: Series of gates at the NMDW

Opening completely the measuring gates and neglecting the remaining head loss enables to calibrate only the first structure. However, this calibration is valid only for situations where the measuring gates are opened. If it is not the case, the structure should be operated as designed, but this necessitates the installation of a measuring gauge in between of the two structures. In order to operate the structure in its intended manner, a gauge should be installed to measure water level h_2.

- Seepage evaluation

Using the discharge measurements done and the fact that the system was in steady state on Monday 02/10/2000 afternoon, it is possible to have an evaluation of the seepage of this canal.
The discharge calculated with the rating table at NMDW gives a discharge of 3.20 m^3/s and the discharge measured using the current meter at Mado Bridge is 2.78 m^3/s. These two points being distant of 24080 m, this gives a seepage of about 18 l/s/km. In comparison, the seepage calculated for the Fordwah Branch, Chishtian subdivision canal in Pakistan was about 65 l/s/km (Litrico, 1995).

- Evaluation of siltation (comparison of cross sections with the design data)

The design geometry was compared to the actual one at the locations where discharge measurements were done. It enables to evaluate the siltation along time. However, as no reference point was available, the comparison is done assuming a given reference point for the elevation of the banks. In fact, assuming the slope of the banks did not change much, one can position the two cross sections the one with respect to the other. Results are given in Figure 6. It seems that siltation is more important upstream of the system (Zumoni bridge) compared to the situation downstream.
These measures should be done together with a complete topography of the main canal system in order to assess the importance of siltation in this system.

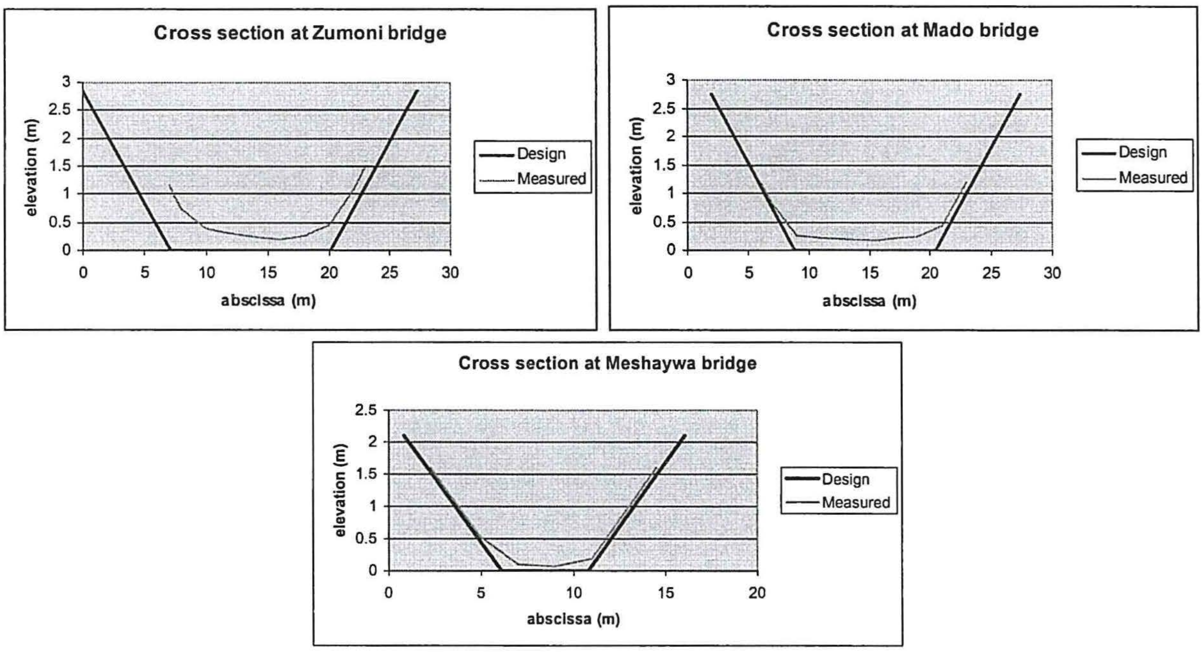

Figure 6: Comparison of designed and measured cross-sections at different locations

- Data on automatic gates at CR

The automatic gates installed at Cross Regulators are designed with a round back at Marina CR, because the head is not sufficient to ensure free flow at the structure and with a flat back at the other locations where the head is sufficient. The measured dimensions of the gates are given in annex 2.

Some experiments were done on these gates in order to be able to calibrate a regulation module to simulate their functioning in SIC.

3. SIC model calibration

The SIC model used is based on the one developed by Quirion (2000) using design data.

3.1 Description of SIC model

SIC (Simulation of Irrigation Canals) is a mathematical flow simulation package developed by Cemagref that enables to model the hydraulics of irrigation canals and simulate their functioning.
The hypothesis used in SIC are the following:

- The flow direction is rectilinear, so that the water surface can be considered horizontal in a cross section,
- Transversal velocities are negligible and the distribution of pressure is hydrostatic.

Only unidimensional and subcritical flow is be simulated.

SIC is based on 3 different units (Cemagref, 1999):

- Unit 1 (topography unit) :the topography geometry of the canal are entered and processed for the calculations. The topography data are generated in ASCII format and saved as a .tal file.
- Unit 2 (steady flow unit): the hydraulic data necessary for a steady flow computation can be entered and modified (.flu file). Water levels, discharges and openings for offtakes and cross-structures are computed using the equation of water profile for a given inflow. SIC uses special equations for gates and weirs, modifying the C_d according to flow conditions.
  The equation of the water profile in a reach is given by the equation:
  $$\frac{dH}{dx} = -S_f + (k-1).\frac{qQ}{gS^2}$$
  with :
  $$S_f = \frac{n^2Q^2}{A^2R^{4/3}}$$
  where :
  g = gravitational acceleration [m.s^{-2}]
  n : Manning's roughness coefficient ($n = 1/K$, where K is the Strickler coefficient)
  R : hydraulic radius [m]
  A : cross section area of flow [m^2]
  H : total head [m]
  q : lateral discharge per unit length ($q > 0$: inflow ; $q < 0$: outflow) [m^2.s^{-1}]
  $k = 0$ for lateral inflow ($q > 0$), $k = 1$ for lateral outflow ($q < 0$).
  S_f: friction slope
  Q : discharge [m^3.s^{-1}]
  For solving this equation, an upstream boundary in terms of discharge and a downstream boundary in terms of water surface elevation are required. In addition, the lateral inflow and the hydraulic roughness coefficient along the canal should be known. With these data, the water surface profile is integrated step by step starting from the downstream end.

- Unit 3 (unsteady flow unit): the water levels and discharges are calculated using the Saint Venant equations for varying inflow and operations. The initial water surface profile is provided by unit 2 (steady flow unit).
  The water movements are described by Saint-Venant equations:
  Continuity equation: $\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q$:
  Momentum equation: $\frac{\partial Q}{\partial t} + \frac{\partial Q^2/A}{\partial x} + gA\frac{\partial z}{\partial x} = -gAS_f + kqV$
  With the same notations as above, and

t : time variable [s]
x : space variable [m]
z : water elevation [m]
V : mean velocity [m.s^{-1}]
These equations are solved numerically by discretizing them according to the Preissmann's scheme the resulting system is solved using a double sweep method.

A more detailed description can be found in the user's and theoretical guides (Cemagref, 1999).

3.2 Calibration in steady state

A steady state simulation using data collected Monday 02/10/2000 from 14:00 to 19:00 (Steady Flow Period) was performed and the Manning coefficients in reaches were adjusted in order to minimise the errors in simulated versus measured water levels.

Table 3 : Results of SIC calibration

Measuring points	Abscissa (m)	Measured in the field			Simulated in SIC		
		Water elevations		Discharge	Water elevations		Discharge
		Upstream	Downstream	m^3/s	Upstream	Downstream	m^3/s
Barrage	0	-	37.08	3.12*	-	37.03	3.26
NMDW	2810	37.015	36.80	3.27*	37.01	36.80	3.21
Zumoni	6085			3.20			3.15
Marina	10581	36.73	36.2		36.73	36.2	3.07
Mado	11652			3.17			3.05
Furawa	15378	36.11	34.92		36.11	34.91	2.98
Akubushim	19040	34.80	34.15		34.80	34.16	2.92
Gatafa	24222	33.94	32.93		33.94	32.94	2.82
Meshaywa	26890			2.78			2.78
Yamidi	29824	32.68	-		32.68	-	2.72**

* The discharges obtained at the Barrage and the NMDW are calculated using the discharge coefficients and the corresponding rating curves previously established.
** The discharge at Yamidi is the total discharge flowing out of the main system (Yamidi + Spillway + Ganuwar Kuka + Pilot Farm)
The calibration is rather good, as the maximum error in water levels is 5 cm, at the head of the system. This error is explained by the inaccurate geometry of the Feeder, which necessitate to have a low Strickler coefficient, not justified compared to the other ones in the system. This should be corrected by replacing the assumed geometry by the real one.

- Strickler and Manning coefficients

The Strickler coefficients obtained by calibration are given in table 2. The Manning coefficients are calculated as the inverse of the Strickler.

Table 4: Manning and Strickler coefficients obtained after calibration

Reaches	Manning coefficient	Strickler coefficient
Barrage-NMDW	0.067	15*
NMDW-Marina	0.04	25
Marina-Furawa	0.05	20
Furawa-Akubushim	0.055	18
Akubushim-Gatafa	0.045	22
Gatafa-Yamidi	0.036	28

*: the low value of the Strickler coefficient in the Feeder canal may be explained by the fact that the exact geometry of the feeder canal was not available: a trapezoidal geometry is assumed here, whereas the real bed geometry has two levels, the width of the lower one being smaller than the higher one. The exact dimensions of this canal should be taken during the next measurement campaign. However, this approximation does only affect the water elevation upstream of this reach, and not the rest of the canal.

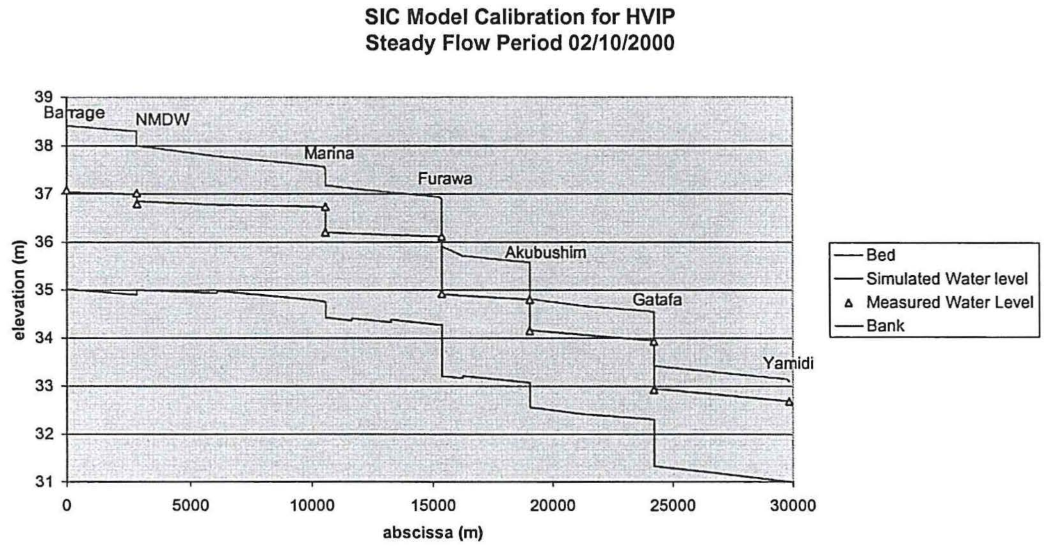

Figure 7 : Longitudinal profile of water levels for the calibration data

For this calibration in steady state, the cross structures were put in "regulator" mode, which means that the model computes the opening of the structure in order to fit a target upstream level, according to the structure law. We now have the Strickler coefficients of the reaches. The discharge coefficients at the cross-structures were determined using the data collected in the field (angle of opening, discharge, and water levels) and the Matlab files given in Annex 5.

- Cross regulators modelling

We used the work of Quirion (2000) as a starting point for the modelling of the cross regulators. The Vlugter/Begemann gates are made of a steel plate rotating around a point located above the upstream water level. In closed position, the plate closes a large crested weir. The depth of water above the sill puts a pressure on the gate, tending to open it, whereas the counterweight tends to close it. The gate is designed such that the couples due to the opening and closing forces are in equilibrium when the upstream water level is at Full Supply Depth (FSD).

This type of gates has been studied by hydraulic engineers since a long time (Vlugter, 1940) and has been the subject of recent publications (De Graaf, 1998, Raemy & Hager, 1998, Burt et al. 2000). The main problem in modelling this gate is the computation of the force exerted on the plate by the water. In closed position, the pressure distribution is hydrostatic, but this distribution becomes more complicated to compute when the gate opens. The approach followed by Raemy & Hager (1998) and Burt et al. (2000) is to derive a formula for calculating this force from experimental results. De Graaf (1998) proposes another method, using the momentum conservation to calculate the pressure forces on the plate. This expression is difficult to compute in a real situation with a complicated channel geometry. This is why we used the method proposed by Raemy & Hager (1998), modified by Quirion (2000) in order to take into account the characteristics of the Begemann gate, slightly different from the one they studied.

The maximum discharge flowing through the gates can be estimated considering the weirs without the gates. In this case, the discharge is obtained with the formula:
$Q = 1.7 \times W \times H^{3/2}$
where:
Q : discharge [m^3/s]
W : width of the weir [m]
H : upstream head [m]

With $H = 0.6$m and $W = 1.15$m, the theoretical maximal discharge for one gate is 0.91m^3/s.

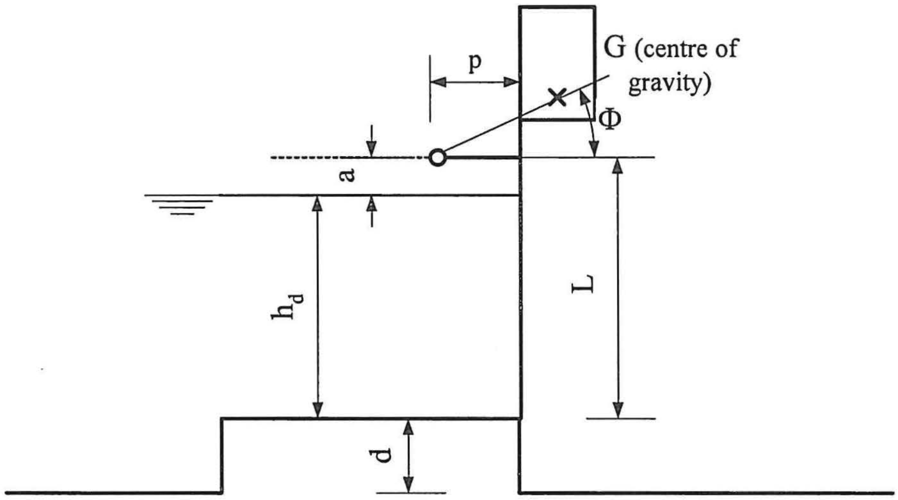

Figure 8: Important dimensions of Begemann gate (closed position)

The geometric parameters of the gate are as following (see Figure 8 and Figure 9):
- δ : angle of opening (δ=0 when closed) [rad]
- Φ : angle between the horizontal axis and the line between the hinge point and the centre of gravity of the gate in closed position [rad]
- p : horizontal distance between the hinge point and the gate [m]
- L : vertical distance between the hinge point and the bottom of the gate [m]
- X_G, Y_G : co-ordinates of the centre of gravity (with the hinge point at the origin) [m]
- L_v : width of the gate [m]
- M_{cp} : mass of the counterweight [kg]
- M_v : mass of the gate without the counterweight [kg]
- l_m : length of the wetted part of the gate [m]
- a : distance between the designed water level (FSD) and the hinge point [m]
- h_1 : upstream water level above the sill [m]
- h_2 : downstream water level above or below the sill [m]
- w^* : opening of the gate [m]
- w : vertical opening of the gate [m]

With these data, we have:

$$\Phi = \arctan\left(\frac{Y_G}{X_G}\right)$$

$$d_G = \sqrt{X_G^2 + Y_G^2}$$

where d_G is the distance between the hinge point and the centre of gravity [m].

We also have :

$$w = L(1-\cos\delta) + P\sin\delta$$

$$w^* = \sqrt{L^2+P^2}\sqrt{2-2\cos\delta}$$

$$l_m = \frac{h_1 - w}{\cos\delta}$$

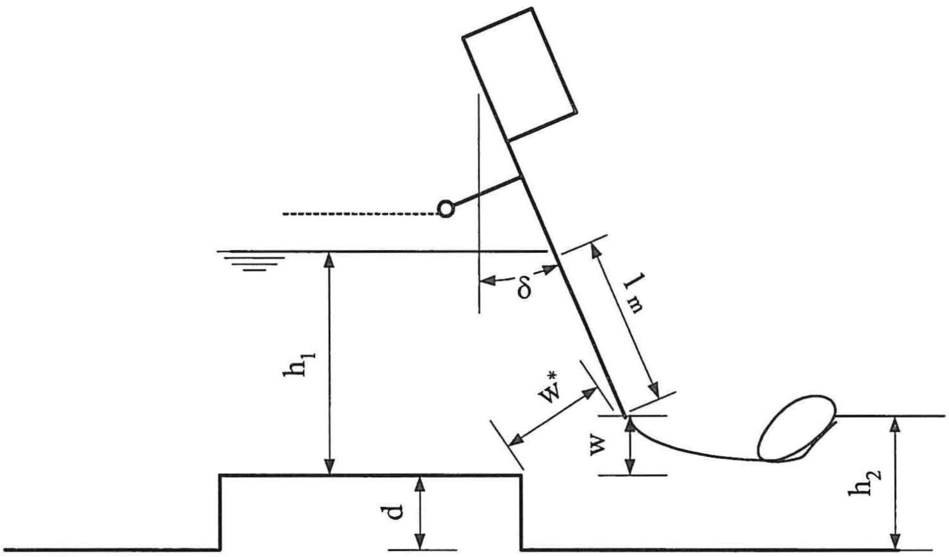

Figure 9: Dimensions of Begemann gate (opened position)

Design of the gates:
Let M_o be the opening moment due to the water pressure on the plate. In closed position, the pressure distribution is hydrostatic, therefore:

$$M_o = \left(\frac{1}{2}\rho g L_w h_1^2\right) \times \left(a + \frac{2}{3}h_1\right)$$

M_o in [N.m]
ρ is the specific mass of water [kg.m^{-3}]

The closing moment M_c due to the weight of the system gate counterweight is :

$$M_c = (M_{CP} + M_V).g.d_G$$

M_c in [N.m]
g : acceleration of gravity [ms^{-2}]

The static equilibrium is attained when $M_c = M_o$, the gate being in closed position (Q = 0).
Brouwer (1987) presents in detail the design of these gates, used in the Kano River Project in Nigeria.

Modelling of the pressure forces on the gate:
In opened position, the water pressure distribution is no longer hydrostatic, and varies with the angle of opening δ. In the area near the open surface, where the velocity is low, the pressure distribution is almost hydrostatic; but close to the bottom of the plate, velocities are larger and pressure diminishes to reach atmospheric pressure at the bottom of the plate (Raemy & Hager, 1998). Moreover, the application point of water pressure forces is no longer at 2/3 of the wetted depth, but varies with the opening of the gate.
Raemy & Hager (1998) have experimentally determined two coefficients to express the relation between static and dynamic forces.

$\sigma = F_d/F_S$ is the ratio of dynamic versus static pressure forces, given approximately by :

$$\sigma = 1 - \frac{1}{7}D.\tan\delta$$

where D is the relative length of the gate (L/h_1) and δ the angle of opening ($\delta \leq 30°$).
In the same spirit, $\mu = M_d/M_S$ is the ratio of dynamic versus static moment, given approximately by :

$$\mu = 1 - \frac{1}{4}D^{1/2}.\tan\delta$$

For $\delta = 0$, one recovers the values $\sigma = \mu = 1$. The dynamic effect of the water pressure increases with the angle of opening and the ratio D.

The moment M_c due to the weight of the gate-counterweight is given by :

$$M_c = (M_{CP} + M_V)g.d_G.\cos(\Phi + \delta)$$

In case of a hydrostatic pressure distribution, one would get (static case) :

$$M_o^{st} = \frac{1}{2}\rho g L_v l_m^{\ 2}\cos\delta \times \left(L - \frac{1}{3}l_m\right)$$

However, the corrective factors have to be taken into account. In their experiments, Raemy & Hager have used a gate where the hinge point is on the gate ($p = 0$). In our case, we cannot write directly $M_o^{dy} = \mu.M_o^{st}$. Knowing the corrective factor σ to apply on the force, we look for a general factor to apply on the lever arm.

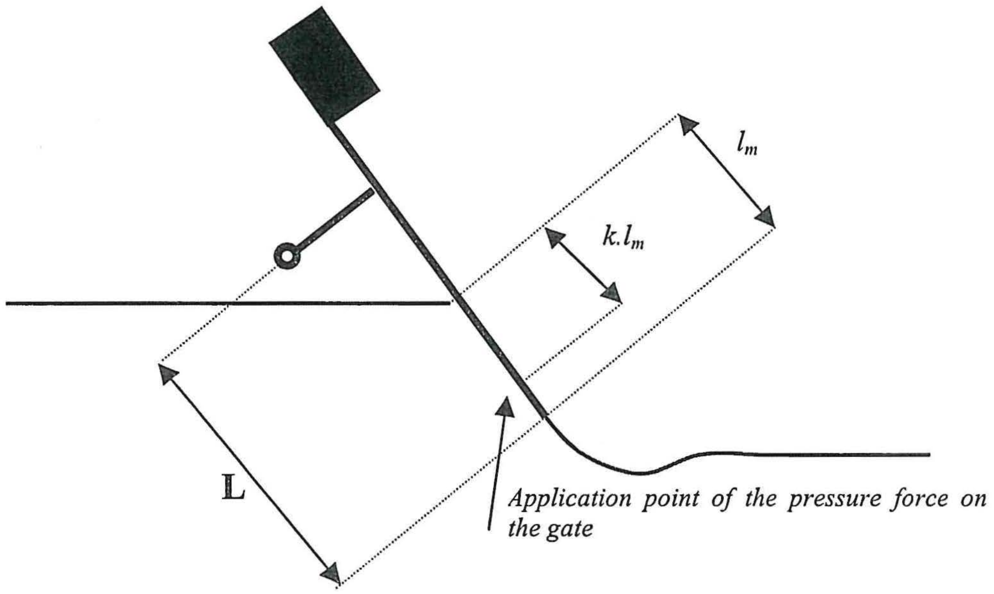

Figure 10: Begemann gate – calculus variables

Supposing the application point of the pressure force resultant on the gate is located at k times l_m (in static, $k = 2/3$), one gets :

$$d_{Hy} = L - (1-k).l_m$$

with d_{Hy} the lever arm (distance between the application point and the hinge point) [m]

As the moment is corrected by a factor μ, the dynamic pressure has to be taken into account in k:

$$k = \frac{2}{3}\left(\frac{\mu}{\sigma}\right)$$

and finally :

$$k = \frac{2}{3}\left(\frac{1-\frac{1}{4}\sqrt{\frac{L}{h_1}}\tan\delta}{1-\frac{1}{7}\left(\frac{L}{h_1}\right)\tan\delta} \right)$$

Using this equation, the angle of opening δ is calculated by dichotomy until $M_o^{dy} = M_c$, h_1 being given by the hydraulic computations performed by the hydraulic model (SIC).

The regulation module included in SIC computes the opening of the gate this way. The computed opening is used in SIC equations to compute the discharge.

Remark 1: the same type of results has been obtained by Burt et al. (2000). In their case, the pressure distribution is calculated fitted with a parabolic curve, leading to the following formula:

$$M_o = \frac{1}{2}\rho g L_v l_m (1-0.024\delta) \times \left(L - \frac{h_1}{3}(1+.009\delta) \right)$$

where the angle δ is expressed in degrees.

Remark 2: the specific situation encountered when the downstream level influences the equilibrium of the gate has not been taken into account in this preliminary work (Vlugter gates at Marina CR). This will be an important subject for future studies.

Calibration of the regulation module:
Some few experiments were done on site in order to be able to calibrate the regulation module simulating the Begemann gates. A dynamometer was used in order to measure the force necessary to open the gate at a given level (see Figure 11 and Figure 28).

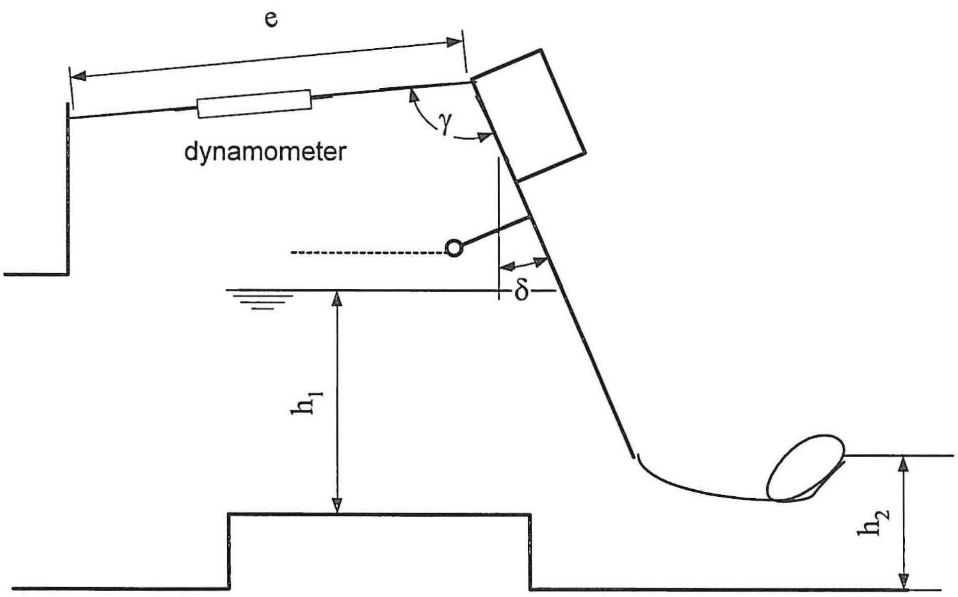

Figure 11: Begemann gate with dynamometer

These measurements were done on two places, but unfortunately the discharge was not measured at the same time (this should be done in the next measurement campaign). The objective was to identify the characteristics of the gates (weight of the gate and counterweight, discharge coefficient of the gates). The measurements were first done at Marina CR, but only on one gate. The problem with this experiment is that the discharge flowing into the gate is not constant.
Then, the same type of measurements were done at Akubushim CR, on all the gates. It is quite tedious, as there are 6 gates to move all together in order to get the same opening. Then, measuring the

upstream and downstream levels, the angle γ, the distance e, the angle of opening δ enables to check different things:

- Assuming the weight of the gate and counterweight is known, we can check whether the formulations for the pressure forces are in accordance with the measured data, and especially how they vary with the opening,
- If the weight of the structures is not known exactly, it is possible to calculate the mass of the gate and counterweight, by using a given formula for the pressure force distribution,
- If the discharge is measured along with the other variables, a discharge formula can be established for the structure.

With the data collected, only the second point proved to be feasible (the first one giving rather bad results). However, there is a need for more detailed measures of this type in order to model more accurately this gate.

This has enabled us to compare the two formulas of Burt et al. (2000) and of Raemy & Hager (1998). It was found that the results of Raemy & Hager were more consistent with the data available (see Figure 12, obtained with the Matlab file Mass_`calculus.m`, given in annex 5). It should however be stressed that this constitutes only a starting point for further research. The results of de Graaf (1998) were difficult to apply because they necessitate the computation of the energy of the flow downstream of the structure, which is quite difficult with a real geometry, and was not possible to test in limited time.

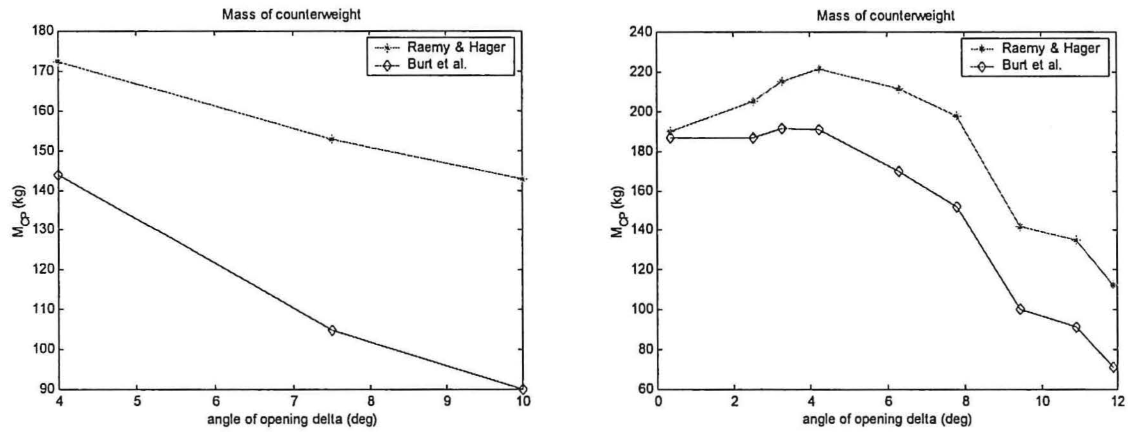

Figure 12: Calculated mass of counterweight for Akubushim and Marina CRs as functions of the opening angle δ

The calculated mass of counterweight (assuming a mass of the gate equal to 256 kg) are different for different opening because of the differences in upstream levels, that were not measured accurately (especially for large openings). But these experiment enable to have a rough idea of the mass of the counterweight rather accurate for small openings, which are of the same order as the values given in table 5.

Simplified procedure for the calibration of the regulation module:

We derived a simplified method for calibrating the model, using the data collected during the steady flow measurement campaign. The calibration of the cross regulators is done by assuming that the gate has a mass equal to 256 kg (data provided by the constructor), and computing the corresponding counterweight mass such that the upstream level is in accordance with the discharge flowing into the gates at this point. This discharge is computed by the hydraulic model using the discharge measurements and the seepage determined earlier. The Matlab program `calage_MCP.m` given in annex 4 computes this for the four cross regulators. The counterweight is calculated by dichotomy in order to get an upstream level equal to the measured one, using the formulas above for the pressure force distribution (see Figure 13).

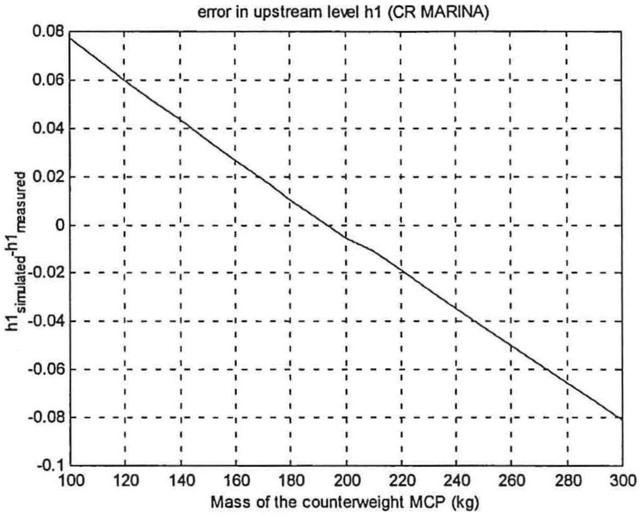

Figure 13: Error in upstream level h_1 as a function of the counterweight M_{CP}

Then, the discharge coefficient to put in SIC is calculated by the program `calage_Cd.m` also given in annex 4. This program determines the opening corresponding to the upstream level measured (using the formulas given above), and calculates the C_d such that the measured discharge flows through the gate.
This C_d is then fed back into SIC for steady state and unsteady state computations.

The values were obtained by these calculations are given in Table 5:

Table 5: Counterweight mass and discharge coefficients for the CRs

CR	M_{CP} [kg]	C_d
Marina	193	0.58
Furawa	233	0.62
Akubushim	132	0.64
Gatafa	93	0.67

The values obtained are rather conform to the usual ones (C_d) and indicate that the calibration is at least not unrealistic. The method of calibration of the counterweight should however be modified in order to take into account real data as the mass of the gates and counterweights in case they can be measured. This will be the subject of future research.

The unsteady state simulation will be used as a validation for the model of the cross regulators of this canal.

3.3 Validation in unsteady state

An unsteady state simulation is done using the data collected during the second day of the measurement campaign (gate openings and discharge delivered at the barrage on Tuesday 03/10/2000).
The results of simulation are given in the following figures (Figure 14 to Figure 19), together with the measured data in order to validate the simulation model.
The results are rather good, as the maximum error in water level occurs at the NMDW, where the upstream level is underestimated by the model (7cm). This may be explained by the rough calibration of this structure (and by the fact that the structure was not operated according to its design intentions),

when the second series of gates was not interacting with the flow. Some more measurements should be done at this point to get better simulation results.
One important feature is that the general behaviour of the canal is well represented, such as time-lags, variation of water levels at the cross regulators and discharge propagation.
This model should be a good starting point for future detailed research on this system. In fact, the Ph.D. research of Mr. S.Z. Abubakar, focused on the management of the irrigation scheme, could for its water management component be based on the simulation of different operational scenarios using SIC (Abubakar, 2000).

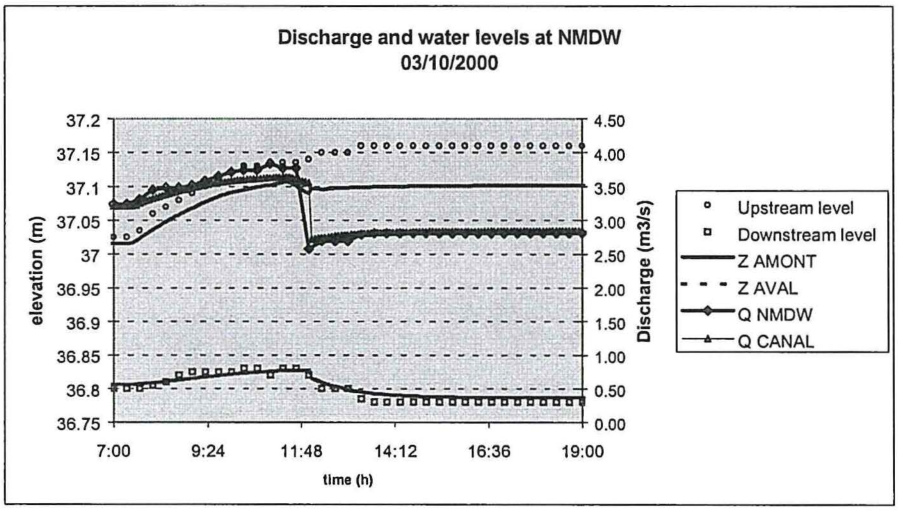

Figure 14: Simulated vs. measured discharge and water levels at NMDW

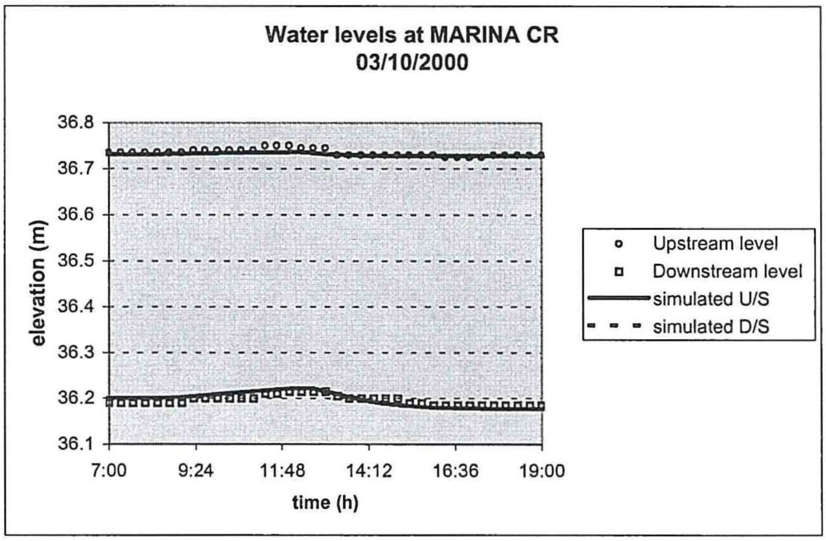

Figure 15: Simulated vs. measured water levels at Marina

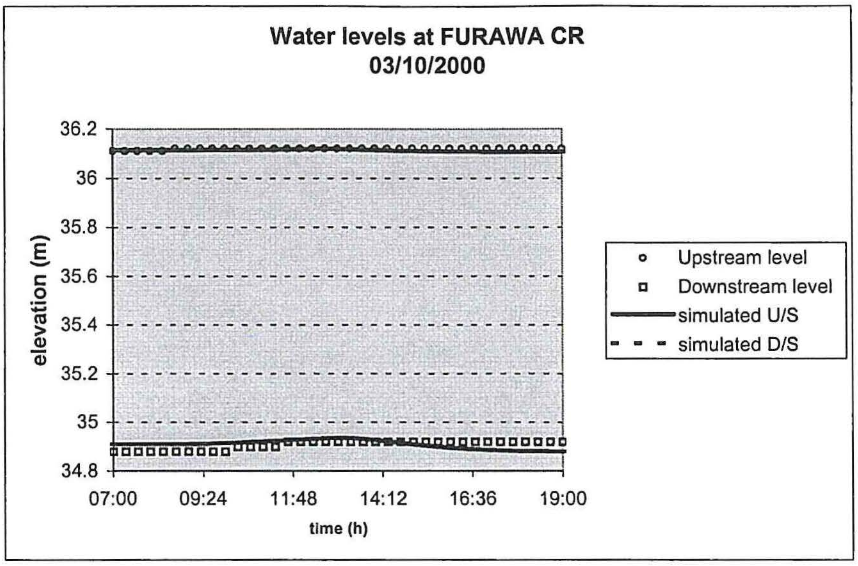

Figure 16: simulated vs. measured water levels at Furawa

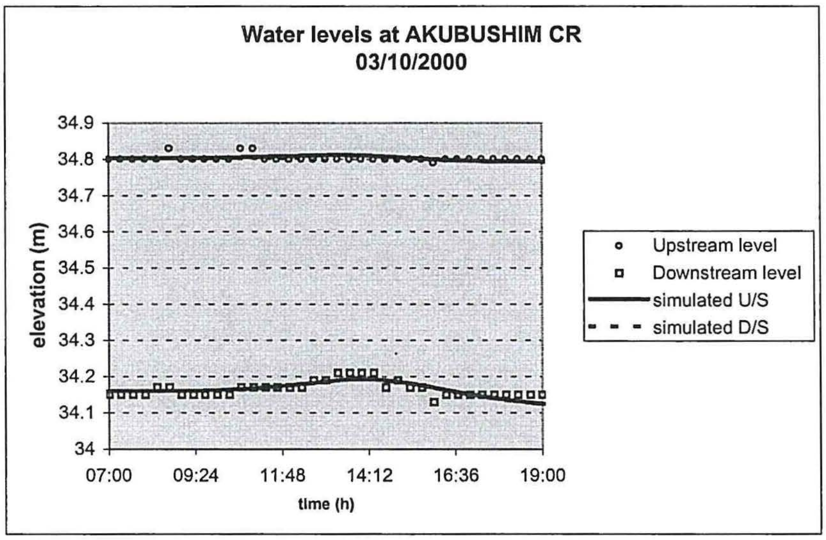

Figure 17: simulated vs. measured water levels at Akubushim

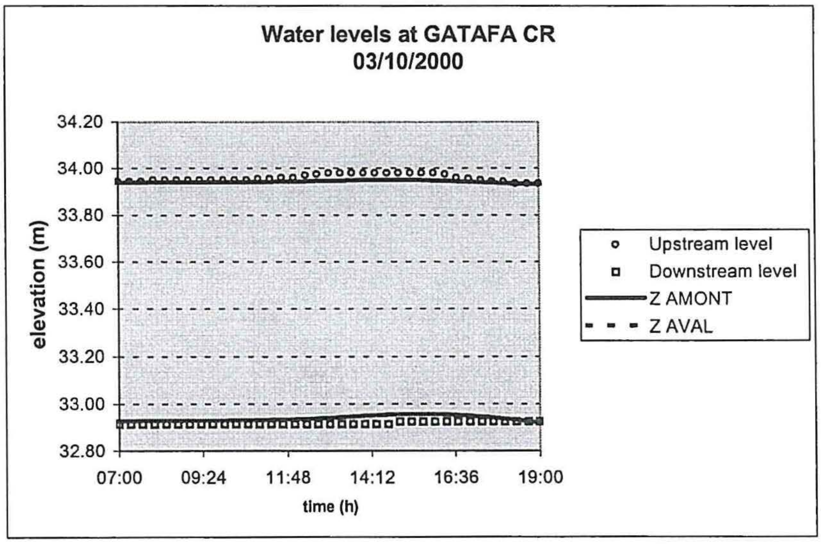

Figure 18: Simulated vs. measured water levels at Gatafa

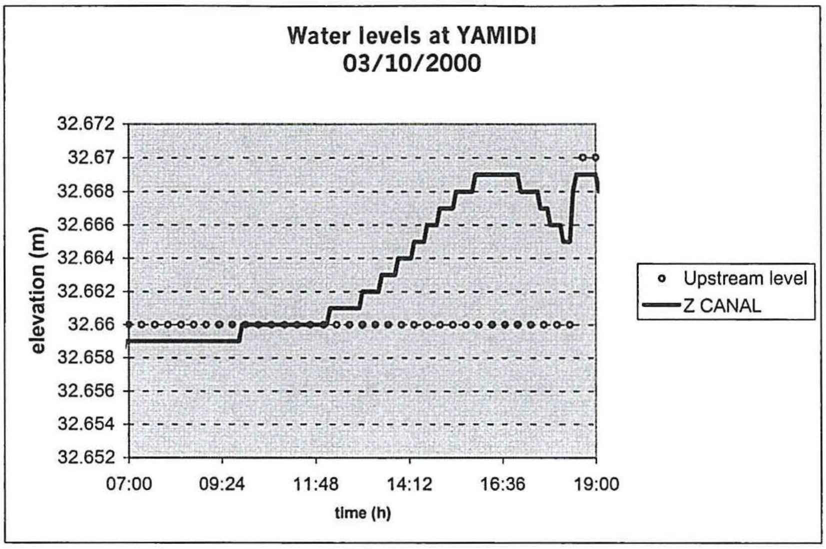

Figure 19: Simulated vs. measured water level at Yamidi

3.4 Conclusion on the simulation model

Many hydraulic simulations can now be done with the model developed. However, there is some work to do before using the model for the whole system. We present here some possibilities that were not tested for the moment, but that will take part of the objectives for the continuation of the collaborative program:

- Simulation of the system at full capacity (with STOs opened)
- Maximal conveyance capacity
- Propagation times at different flow regimes
- Check the effect of some maintenance work on the capacity of the canal
- Test different management rules

The model as presented was calibrated using the data provided by the measurement campaign conducted during this 10 days mission. It is therefore validated for the canal with STO closed, which is a very specific situation. However, the main characteristics of the canal should not be modified by the opening of the STO; the STO hydraulic structures should be studied in order to get a complete model of the system.

There is a need for more modelling at :

- the cross-structures (study of their static/dynamic behaviour, flat back/round back, method for calibrating the structure)
- the offtakes (STO, model of the CHO to be included in SIC,)
- the secondary level (Distributary Canals with their tertiary offtakes)

A Ph.D. is underway on the impact of management rules on the water distribution in HVIP (Abubakar, 2000), benefiting directly from the work on the simulation model. A proposal for another DEA/Ph.D. programme (Mr. M.K. Othman) has been formulated to investigate the hydraulic functioning of the cross-regulators and STOs.

4. Conclusions and recommendations

4.1 Conclusions

This report is the output of a 10 days mission on the HVIP main canal. This is a preliminary work that should be followed by more detailed ones in order to get practical results for a better management of the system (two programmed Ph.D. theses Abubakar 2000 and Othman 2001). These conclusions are followed by some recommendations for the future.

The results of the campaign were discussed with the field staff, the system manager, the Ass. General Manager O & M of HJRBDA, and NAERLS. The following main issues emerged:

- The measurement campaign was a useful tool to test the reactivity and robustness of the system. It has significantly increased the knowledge of the system manager and field staff of their system. It is recommended that a similar exercise be carried out in 2001, this time including the secondary outlets and better involving the field staff. The campaign can be carried out by the system manager with the support of the hydraulics engineer from NAERLS.
- The campaign yielded useful outputs, such as rating curves for the inlets of the Feeder Canal and North Main Canal, and determination of the time lags. The rating curves at these two structures should be checked by other discharge measurements, and a gauge should be installed immediately downstream the first series of gates at the NMDW.
- It is recommended to install gauges at the different secondary inlets (STO's).
- There is at present little potential for the operational use of SIC at HVIP. SIC may be more useful for research purposes (NAERLS) or diagnosis purposes (HJRBDA), for example to determine the impact of maintenance activities on water deliveries.

4.2 Recommendations

- On the measurement campaign: some detailed measurements should be done at the STO in order to correctly model the CHO (which are not included as an outlet device in SIC); the Cross Regulators should also be the subject of careful study in order to model as precisely as possible their functioning. The few measurements undertaken during this campaign should be done in a systematic way on all the CR (with concomitant measures of discharges) in order to have a better estimation of their characteristics. Some more work is also needed on the theoretical investigation of the functioning of these gates in real situations (as in HVIP or KRP). The dimensions of the Feeder Canal should also be measured, in order to get a more realistic Manning coefficient. During next measurement campaigns, the water levels at the Barrage should also be recorded together with the other water levels along the system.

- On the modelling part: the gates at the barrage should be included in the model; for the moment, only a sluice discharge is given as the upstream limit condition. This solution is inaccurate when the gates at the barrage are submerged, as the discharge depends on the downstream water level, itself influenced by the downstream limit condition at the NMDW (which is only 2 km downstream). This would require to give a water level in the reservoir as the upstream limit condition, and the corresponding gate openings.

- On the canal operations: the impact of actual and alternative operational rules on water deliveries can be studied by formalising and modeling these rules (Abubakar, 2000). In doing so, the scope for improvement in the manual operation of irrigation scheme can be determined (even if no O&M manual is available, there are some oral rules that can be derived in actual operations).

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Annex 1: Measurement campaign protocol for Hadejia Valley Irrigation Project, North Main Canal, October 1-2, 2000

This measurement campaign is designed to meet three objectives:

1) make discharge measurements in the canal when it is in steady state (constant discharge, and no gate movements).
2) follow the propagation of a wave. A step in upstream discharge will be provoked in order to measure the water levels in unsteady state (upstream discharge from 2.5 m^3/s to 4 m^3/s).
3) Study the hydraulic behaviour of the Vlugter gates.

Things to check before the beginning of the measurement campaign :

- Personal requirements: 6 gauge-readers trained to read and note water levels. They will be placed at several locations along the canal:
- The discharge should be constant at the head of the channel 1 day before the beginning of the campaign,
- There should be no gates movement during measures (except if foreseen in the campaign),
- The STO should be closed in order to limit the uncertainties in discharge flowing out of the system.

2 options for the hydraulic model calibration:

1. get a steady flow regime during 1 or 2 days.
2. Follow a step between the initial steady flow regime to another steady flow regime (with a change in the upstream discharge between the two, and gate movements to check)

For calibration of the regulation module representing the Vlugter gates, one should check upstream and downstream levels, the discharge, and the openings of the gates for different equilibrium positions (which correspond to different discharges). One possibility would be to study a particular gate (for example one of the 4 gates at the end of the canal, at Cross regulator GATAFA, with the other 3 closed, if possible, in order to correctly estimate the discharge in this gate).

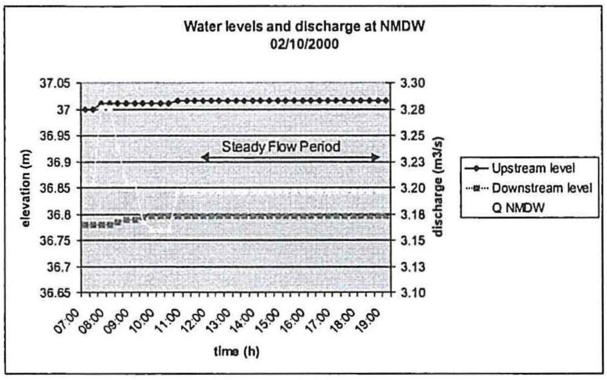
As we only have a limited time for this measurement campaign, we could try to meet the two objectives in one campaign, by choosing option 2. This would enable us to calibrate the hydraulic model for one (or two) steady flow regime, and to observe the Vlugter gates behaviour for two different discharges.

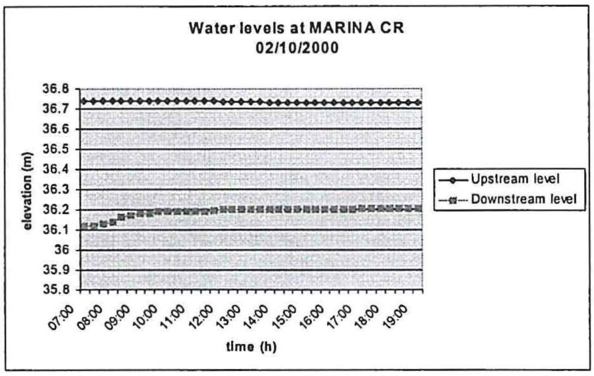
Question: what about the STO discharges during the campaign? The better would be to have them closed during the campaign. Is it possible?

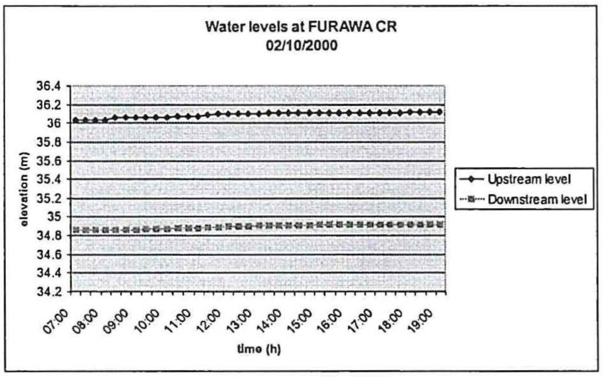
The programme would then be as follow:

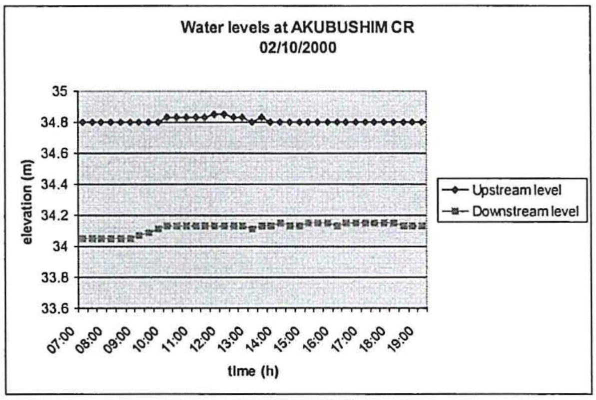
dates		action
Sunday 1/10	Morning	Constant discharge at the dam and at the NMDW (North Main Division Works)
	Afternoon	Discharge measurement(s) at the NMDW (to check the rating curve)
Monday 2/10	Morning	Discharge and water levels measurements at all cross structures + measure of Cross Regulators opening angles
	Afternoon	Id.
Tuesday 3/10	Morning	At 6h : change of upstream discharge from 2.5 to 4 m^3/s.
	Afternoon	Id. + Discharge measurement at CR GATAFA
Wednesday 4/10		Study of the Vlugter gate at GATAFA Cross Regulator
Thursday 5/10		Measurement campaign evaluation, and new measures if necessary

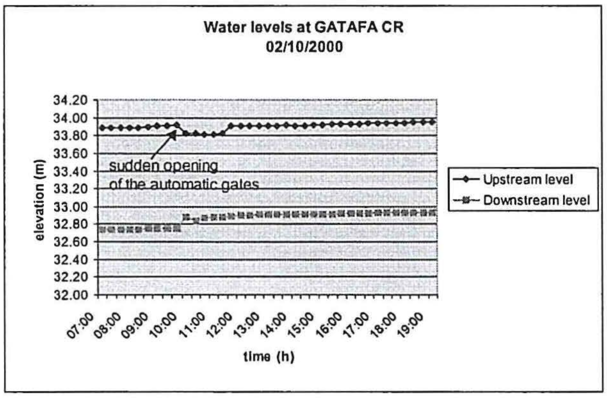
Annex 2: Measurement campaign data

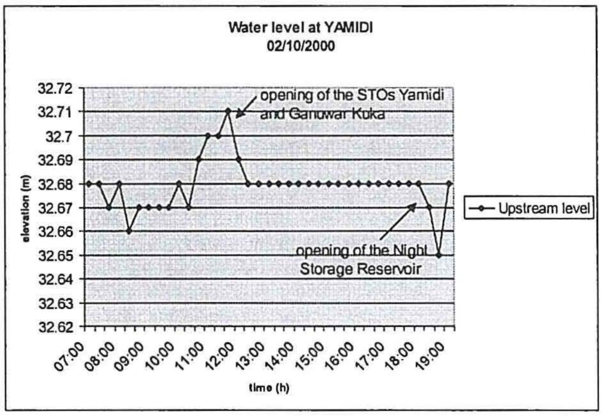


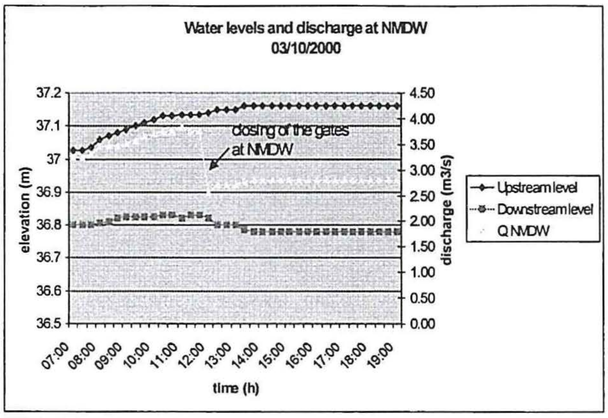


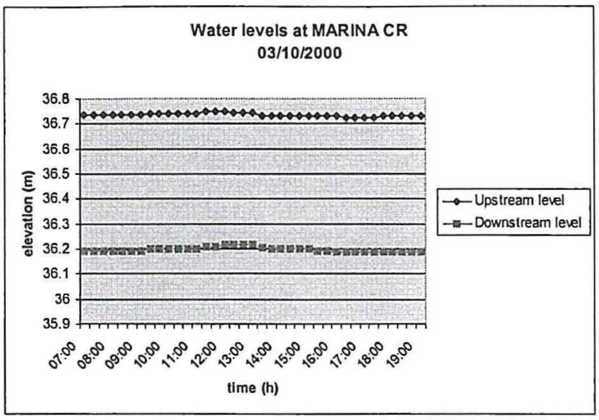


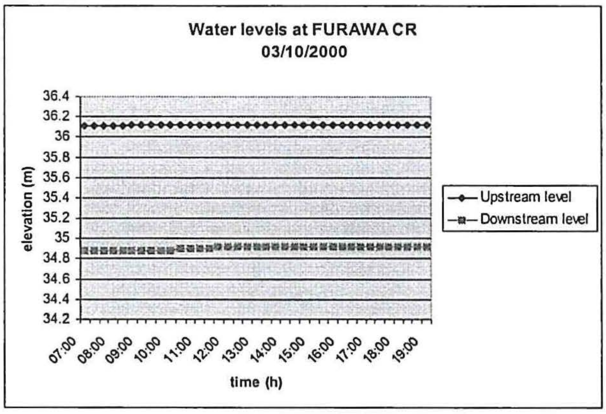


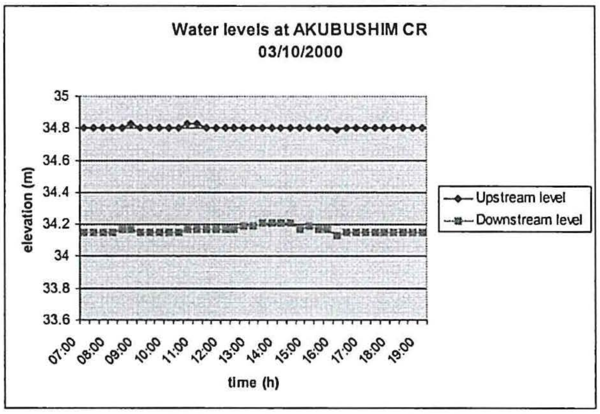


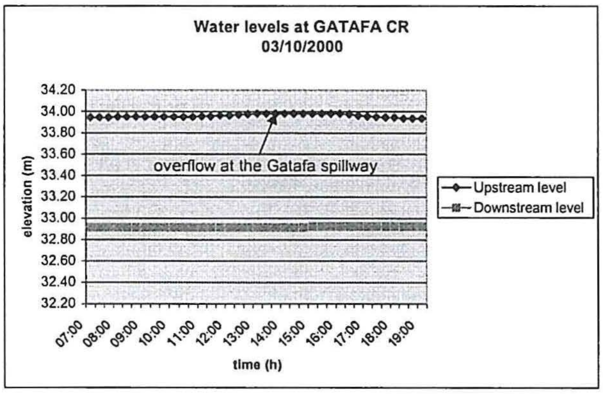


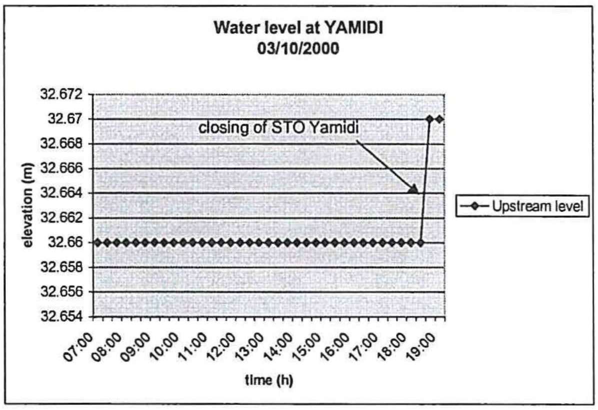

Water levels and discharge at NMDW
03/10/2000
closing of the gates at NMDW
elevation (m)
discharge (m3/s)
time (h)
Upstream level
Downstream level
Q NMDW


Water levels at MARINA CR
03/10/2000
elevation (m)
time (h)
Upstream level
Downstream level


Water levels at FURAWA CR
03/10/2000
elevation (m)
time (h)
Upstream level
Downstream level


Water levels at AKUBUSHIM CR
03/10/2000
elevation (m)
time (h)
Upstream level
Downstream level

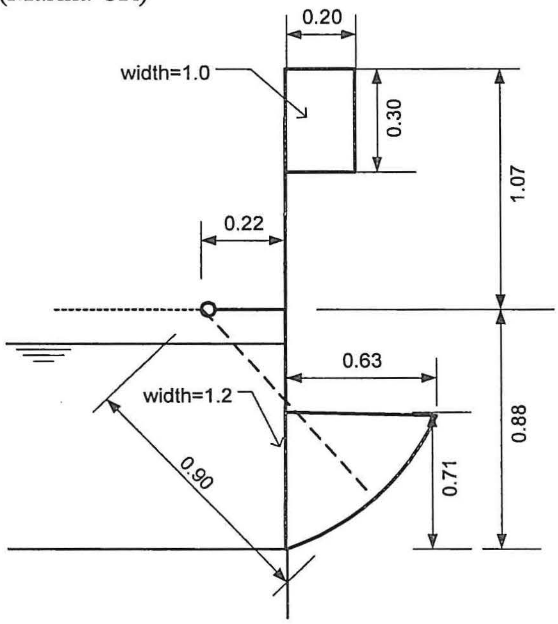

Water levels at GATAFA CR
03/10/2000
overflow at the Gatafa spillway
elevation (m)
time (h)
Upstream level
Downstream level


Water level at YAMIDI
03/10/2000
closing of STO Yamidi
elevation (m)
time (h)
Upstream level

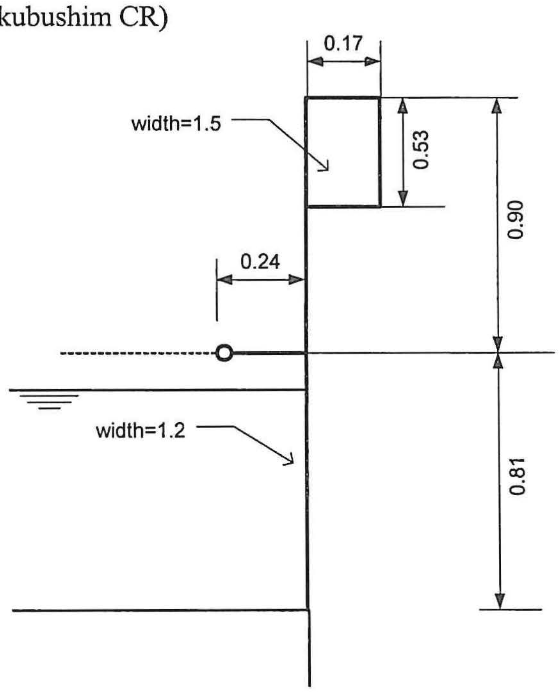
Annex 3: Dimensions of the hydraulic structures measured

	Width (m)	Number of gates
Gates at the Barrage	2.16	15
Gates at NMDW	1.89	8
Marina CR	1.2	14
Furawa CR	1.2	10
Akubushim CR	1.2	6
Gatafa CR	1.2	4

Round back automatic gates (Marina CR)



Flat back automatic gates (Akubushim CR)



Annex 4: Listing of Matlab files used

File : calcul_MCP.m

```
% Calcul de la masse des contrepoids pour caler le module de régulation
% sur les données de terrain
% on suppose que la formule donnée par Raemy et Hager est la bonne
% pour le calcul du moment ouvrant
% XL 26/10/2000

L = 0.862;
P = 0.225;
LV = 1.2;
XG = 0.326;
YG = 0.026;
MV = 256;
TAB = [];
METH=2;      % 2: Raemy et Hager

% débits mesurés aux régulateurs en travers
Q=[3.07 2.98 2.917 2.82];
% nombre de vannes aux régulateurs en travers
NBE=[14 10 6 4];
%Qo=Q./NBE; %débit unitaire par vanne
% cote du niveau d'eau mesuré (amont)
z = [36.73 36.11 34.80 33.95];
% cote du fond
zf= [34.74 34.28 33.07 32.30];
% hauteur du radier
radier=[1.35 1.15 1.1 1.0];
% hauteur d'eau au dessus du radier (amont)
h1=z-zf-radier;

MCP0=50;
MCP1=400;

for i=1:4
   % calcul du contrepoids par dichotomie
   ecartC = 100;
   n=0;
   while (abs(ecartC)>0.01)&(n<=50)
      n=n+1;
      C = (MCP0+MCP1)/2;
      ecartC = ecart2(Q(i),h1(i),L,P,LV,XG,YG,MV,C,NBE(i),METH);
      ecart0 = ecart2(Q(i),h1(i),L,P,LV,XG,YG,MV,MCP0,NBE(i),METH);
      TMP = ecartC*ecart0;
      if TMP > 0
         MCP0 = C;
         MCP1 = MCP1;
      else
         MCP1 = C;
         MCP0 = MCP0;
      end
   end
   MCP(i)=C;
   if n>=50
      display('maximum number of iterations = 50')
   end
end
MCP
```

File : ecart2.m

```
function ecart=ecart2(Q,h1m,L,P,LV,XG,YG,MV,MCP,NBE,METH)

%-----Cette fonction calcule la différence entre la hauteur mesurée h1m
% et la hauteur calculée h pour un débit mesuré Q

% Procédure qui permet de déterminer le coefficient de débit de SIC
% pour un régime permanent donné. Les valeurs obtenues dans TABLEAU
% sont delta (ouverture [degré] ), h1 (cote amont [m]), CdSIC, QSIC, U

TAB=[];
i=0;
for deltadeg = 0.5:0.5:31
```

```
   i=i+1;
   h10 = 0.1;
   h11 = 1.25;
   delta = deltadeg*pi/180;
   ecartC = 100;
   n=0;
   while (abs(ecartC)>0.1)&(n<=50)
      n=n+1;
      C = (h10+h11)/2;
      ecartC = ecart1(delta,C,L,P,LV,XG,YG,MV,MCP,NBE,METH);
      ecart0 = ecart1(delta,h10,L,P,LV,XG,YG,MV,MCP,NBE,METH);
      TMP = ecartC*ecart0;
      if TMP > 0
         h10 = C;
         h11 = h11;
      else
         h11 = C;
         h10 =h10;
      end
      X = [deltadeg C];
   end
   if n>=50
      warning('maximum number of iterations = 50')
   end
   TAB(i,:) = X;
end
deltadeg = TAB(:,1);
h1 = TAB(:,2);
TABLEAU=[TAB calcul_gent1(deltadeg,h1,L,P,LV,XG,YG,MV,MCP,NBE)];

debit=TABLEAU(:,4);
k=min(find(debit>Q));
h=TABLEAU(k,2);

ecart=h1m-h;
```

File : calage_Cd.m

```
% Calcul du coefficient de débit de SIC pour caler le module de régulation
% sur les données de terrain
% on suppose que la formule donnée par Raemy et Hager est la bonne
% pour le calcul du moment ouvrant
% XL 26/10/2000

L = 0.862;
P = 0.225;
LV = 1.2;
XG = 0.326;
YG = 0.026;
MV = 256;
TAB = [];
METH=2;     % 2: Raemy et Hager

Q=[3.07 2.98 2.917 2.82];
NBE=[14 10 6 4];
Qo=Q./NBE; %débit unitaire par vanne
z = [36.73 36.11 34.80 33.95];
zf= [34.74 34.28 33.07 32.30];
radier=[1.35 1.15 1.1 1.0];
h=z-zf-radier;

MCP = [193.1 233.07 131.98 93.44];

for i=1:4
   % calcul du Cd et de l'ouverture correspondant au débit mesuré
   TAB=[];
   j=0;
   for deltadeg = 0.5:0.5:31
      j=j+1;
      h10 = 0.1;
      h11 = 1.25;
      delta = deltadeg*pi/180;
      ecartC = 100;
      n=0;
      while (abs(ecartC)>0.1)&(n<=50)
         n=n+1;
         C = (h10+h11)/2;
```

```
        ecartC = ecart1(delta,C,L,P,LV,XG,YG,MV,MCP(i),NBE(i),METH);
        ecart0 = ecart1(delta,h10,L,P,LV,XG,YG,MV,MCP(i),NBE(i),METH);
        TMP = ecartC*ecart0;
        if TMP > 0
           h10 = C;
           h11 = h11;
        else
           h11 = C;
           h10 =h10;
        end
        X = [deltadeg C];
     end
     if n>=50
        warning('maximum number of iterations = 50')
     end
     TAB(j,:) = X;
  end
  deltadeg = TAB(:,1);
  h1 = TAB(:,2);
  TABLEAU=[TAB calcul_SIC(deltadeg,Q(i),h(i),L,P,LV,XG,YG,MV,MCP(i),NBE(i))];
  debit=TABLEAU(:,4);
  k=min(find(h1>=h(i)));
  deltam(i)=TABLEAU(k,1);
  Qc(i)=TABLEAU(k,4);
  hc(i)=TABLEAU(k,2);
  Cd(i)=TABLEAU(k,3);
  U(i)=TABLEAU(k,5);
end

RES=[deltam',hc',Cd',Qc',U']
```

File: Cacul_SIC.m

```
function [TAB1]= calcul_SIC(deltadeg,Q,h1,L,P,LV,XG,YG,MV,MCP,NBE)

% function [TAB1]= calcul_SIC(deltadeg,Q,h1,L,P,LV,XG,YG,MV,MCP,NBE)
% h1  : cote amont [m]
% deltadeg   : angle d'ouverture (degré)
% L   : distance verticale pied de plaque et pivot
% P   : distance horizontale pivot plaque
% LV  : largeur d'une vanne
% XG,YG : position du centre de gravité du sytème vanne contrepoids p/r au pivot
% MV,MCP : masse de la vanne/contrepoids
% NBE : nombre de vannes montées en parallèle
% Cd  : coefficient de debit
% Q   : debit

% Initialisation -------------------------------------------
R2G = sqrt(2*9.81);
R32 = 3*sqrt(3)/2;

delta = deltadeg*pi/180;
w = L-(L*cos(delta)-P*sin(delta));
U=sqrt((L^2+P^2)*2*(1-cos(delta)));

% Vanne - Denoye -------------------------------------------
CdSIC=3/2*((Q./(LV*NBE*R2G)+0.08.*U.*(sqrt(h1)-sqrt(h1-U)))./(h1.^(3/2)-(h1-U).^(3/2)));
mu0=(2/3)*CdSIC;
mu=mu0-0.08./(h1./U);
mu1=mu0-0.08./(h1./U-1);
QSIC=LV*NBE*R2G*(mu.*h1.^1.5-mu1.*(h1-U).^1.5);

CdSIC=CdSIC(:);
QSIC=QSIC(:);
U=U(:);
TAB1=[CdSIC QSIC U];
```

File: ecart1.m

```
function ecart=ecart1(delta,H1,L,P,LV,XG,YG,MV,MCP,NBE,METH)

%-----Cette fonction calcule la différence entre le moment fermant
%-----(système vanne contrepoids) et le moment ouvrant (dû a la
%-----pression exercée par l'eau sur la vanne

%-----Valeur de l'ouverture verticale
```

```
      w=L-(L*cos(delta)-P*sin(delta));
      w1=sqrt((L*L+P*P)*2*(1-cos(delta)));
      LM=(H1-w)/cos(delta);

%==========Méthode Raemy et Hager modifiée (2)==================
%---Calcul du moment ouvrant
         k=2./3.*(1-0.25*tan(delta)*sqrt(L/H1))/(1-(1./7.)*tan(delta)*(L/H1));
         MO=0.5*1000*9.81*(L-(1-k)*LM)*(1-(1./7.)*(L/H1)*tan(delta))*LV*NBE*LM*LM*cos(delta);
%==========Fin méthode Raemy et Hager modifiée (2)==============

%---Calcul du moment fermant
      DG=sqrt(XG*XG+YG*YG);
      PHI=atan(YG/XG);
      MF=9.81*(MV+MCP)*NBE*DG*cos(PHI+delta);
%---Calcul de la différence des deux moments
      ecart=MO-MF;
```

File: Mass_calculus.m

```
% test avec les différentes formules pour le calcul des moments
% sur les données du CR AKUBUSHIM et MARINA
% XL 23/10/2000

load akubu.mat
%load marina.mat

gamma=gammadeg*pi/180;
% levier
r=sqrt(l1^2+P^2)*sin(gamma-epsilon);
% moment de la force supplémentaire
MF=r.*F;
delta = deltadeg*pi/180;
w=L*(1-cos(delta))+P*sin(delta);
U=sqrt((L^2+P^2)*2*(1-cos(delta))); % valeur de l'ouverture selon de Graaf
LM = (h1-w)./cos(delta);

%====================== Calcul Raemy et Hager =======================
% calcul du moment ouvrant
k = (2/3)*(1-(1/4)*sqrt(L./h1).*tan(delta))./(1-(1/7)*(L./h1).*tan(delta));
Mo(1,:) = (L-(1-k).*LM).*(1-1/7)*(L./h1).*tan(delta))*0.5*1000*9.81*LV.*LM.^2*NBE.*cos(delta);
% calcul du moment fermant
DG = sqrt(XG.^2+YG.^2);
PHI = atan(YG/XG);
Mc(1,:) = 9.81*(MCP+MV)*NBE*DG.*cos(delta+PHI);
Mt(1,:)= Mo(1,:)+MF*NBE;
err_M(1,:)=Mt(1,:)-Mc(1,:);
Masse(1,:)=(Mo(1,:)+MF*NBE)./cos(PHI+delta)/DG/9.81/NBE;
%-------------------------------------------------------------------

%====================== Calcul Burt et al. ==========================
% calcul du moment ouvrant
Mo(2,:) = 0.5*1000*9.81*LV*NBE*LM.*h1.*(1-0.024*deltadeg).*(L-(1+0.00919*deltadeg).*h1/3);
% calcul du moment fermant
DG = sqrt(XG.^2+YG.^2);
PHI = atan(YG/XG);
Mc(2,:) = 9.81*(MCP+MV)*NBE*DG.*cos(delta+PHI);
Mt(2,:)= Mo(2,:)+MF*NBE;
err_M(2,:)=Mt(2,:)-Mc(2,:);

Masse(2,:)=(Mo(2,:)+MF*NBE)./cos(PHI+delta)/DG/9.81/NBE;
%-------------------------------------------------------------------

Masse=Masse-MV

figure:plot(deltadeg,Masse(1,:),'r*-',deltadeg,Masse(2,:),'bd-');
xlabel('angle of opening delta (deg)');
ylabel('M_{CP} (kg)');
legend('Raemy & Hager','Burt et al.');
title('Mass of counterweight');

figure:plot(deltadeg,err_M(1,:),'r*-',deltadeg,err_M(2,:),'bd-');
xlabel('angle of opening delta (deg)');
ylabel('Moment error (Nm)');
legend('Raemy & Hager','Burt et al.');
title('Mo+MF-Mc');
```

Annex 5: Regulation module (FORTRAN listing)

```
      SUBROUTINE SUSER1
C-----Routine de supervision eventuelle
      END

      SUBROUTINE LUSER1(CHAINE,PARA)
C-----Lecture des parametres specifiques pour la methode USER1.
C-----Exemple de lecture d'un reel sur une ligne PS=12. par exemple
C-----Cette valeur est mise dans PARA pour une utilisation future dans
C-----le regulateur correspondant. Le vecteur PARA est en effet
C-----transmis a CUSER1.
C-----A maximum of 10 parameters can be read and stored in the PARA
C-----array variable. If you need more than 10 parameters, you must
C-----store them in another common.
      CHARACTER CHAINE*(*)
      COMMON/P1/ILNPS,INURE,INUAP(100),IREGS
      COMMON/F1/IERROR
      COMMON/BEGEMAN/UOLD(100)

      REAL L,P,LV,XG,YG,MV,MCP,NBE,RADIER,METH,DMAX
      INTEGER I
      DIMENSION PARA(*)
      READ (CHAINE,900,ERR=100)(PARA(I),I=1,10)

      L=PARA(1)
      P=PARA(2)
      LV=PARA(3)
      XG=PARA(4)
      YG=PARA(5)
      MV=PARA(6)
      MCP=PARA(7)
      NBE=PARA(8)
      RADIER=PARA(9)
      METH=2
      DMAX=PARA(10)
      RETURN
 100  CONTINUE
      IERROR=2
 900  FORMAT(9(F6.0,1X),F4.0)
      END

      REAL FUNCTION ECART(DELTA,H1,L,P,LV,XG,YG,MV,MCP,NBE,METH)
C-----Cette fonction calcule la différence entre le moment fermant
C-----(système vanne contrepoids) et le moment ouvrant (dû a la
C-----pression exercée par l'eau sur la vanne
      REAL W,LM,K,DG,PHI,MO,MF,L,LV,METH
      PARAMETER (PI = 3.14159)
C-----Valeur de l'ouverture verticale
      W=L-(L*COS(DELTA)-P*SIN(DELTA))
      W1=SQRT((L*L+P*P)*2*(1-COS(DELTA)))
      LM=(H1-W)/COS(DELTA)
C===========Méthode Raemy et Hager modifiée (2)==================
C-----Calcul du moment ouvrant
          K=2./3.*(1-0.25*TAN(DELTA)*SQRT(L/H1))/
     S          (1-(1./7.)*TAN(DELTA)*(L/H1))
          MO=0.5*1000*9.81*(L-(1-K)*LM)*(1-(1./7.)*(L/H1)*TAN(DELTA))
     S         *LV*NBE*LM*LM*COS(DELTA)
C===========Fin méthode Raemy et Hager modifiée (2)==============

C-----Calcul du moment fermant
      DG=SQRT(XG*XG+YG*YG)
      PHI=ATAN(YG/XG)
      MF=9.81*(MV+MCP)*NBE*DG*COS(PHI+DELTA)
C-----Calcul de la différence des deux moments
      ECART=MO-MF
      END

      SUBROUTINE CUSER1(U,Y,YT,Z,PARA)
C-----Calcul de la commande U à partir des variables controlées Y
C-----des consignes correspondantes YT et des variables mesurées Z
C-----Exemple d'utilisation de CUSER1 pour imprimer des variables sur
C-----le fichier .LST
      COMMON/TEMPS/FIC1,TDEB,FIC2,DT,T,TFIN,FIC3
      COMMON/P1/ILNPS,INURE,INUAP(100),IREGS
```

```
      COMMON/BEGEMAN/UOLD(100)
      DIMENSION U(*),Y(*),YT(*),Z(*),PARA(*)
      REAL A,B,C,L,P,TMP,LV,METH,DMAX,Q
      PARAMETER (PI = 3.14159)
      L=PARA(1)
      P=PARA(2)
      LV=PARA(3)
      XG=PARA(4)
      YG=PARA(5)
      MV=PARA(6)
      MCP=PARA(7)
      NBE=PARA(8)
      RADIER=PARA(9)
      METH=2
      DMAX=PARA(10)
      NMAX=100
C-----Y(1) est en tirant d'eau (mode TYS obligatoire sur le fichier .REG)
      H1=Y(1)-RADIER
C-----Z(1) est en absolu (débit)
      Q=Z(1)
C-----Z(2) est en tirant d'eau (mode TYS obligatoire sur le fichier .REG)
      H2=Z(2)-RADIER
C-----Valeur max d'ouverture delta= 35 degrés
      A=35*PI/180
C-----Valeur min d'ouverture delta = 0
      B=0.0
      N=0
      DH1=H1-YT(1)
      DH2=H2-H1
C-----Initialisation de Zold au premier appel
      IF(INUAP(INURE).EQ.1) UOLD(INURE)=U(1)
C-----calcul de l'ouverture
C       IF (ABS(DH1).LT.0.01) THEN
C         U(1)=UOLD(INURE)
C       ELSE
C-------Calcul de l'ouverture par dichotomie
        DO WHILE((ABS(ECART(C,H1,L,P,LV,XG,YG,MV,MCP,NBE,METH)).GT.5.)
     S            .AND.(N.LT.NMAX))
           N=N+1
           C=(A+B)/2
           TMP=ECART(A,H1,L,P,LV,XG,YG,MV,MCP,NBE,METH)*ECART(C,H1,
     S               L,P,LV,XG,YG,MV,MCP,NBE,METH)
           IF (TMP.GT.0.) THEN
              A=C
              B=B
           ELSE
              B=C
              A=A
           ENDIF
        END DO
        IF(C.GT.DMAX) THEN
           C=DMAX
        ENDIF
C-----  U(1)=L-(L*COS(C)-P*SIN(C))
        U(1)=SQRT((L*L+P*P)*2*(1-COS(C)))
        IF (N.GT.NMAX) THEN
         MESSR=1
      ENDIF
C       END IF
C       WRITE(IREGS,*) 'N = ',N
C       WRITE(IREGS,*) 'Calculated opening U(1)= ',U(1)
      WRITE(IREGS,15) T,INURE, U(1),H1,DH1,DH2,Q,N
   15 FORMAT(F9.0,I3,5F9.3,I5)
C-----On met un filtre d'ordre 1 : S/E = (1-a)Z-1 / 1-aZ-1 (FILTRE = a)
      FILTRE=0.0
      U(1)=FILTRE*UOLD(INURE)+(1.0-FILTRE)*U(1)
C-----On stocke Zold filtré dans PARA(10) pour le filtre
      UOLD(INURE)=U(1)
C-----The following lines are just to prevent warning F4202 of the
C-----Fortran compiler
      Y(1)=Y(1)
      YT(1)=YT(1)
      Z(1)=Z(1)
      END
C=============================================================
```

Annex 6: Trip schedule

Date	Activity	Location	Persons/institutions
23-9	Travel Kuper	Montpellier-Paris	
24/9	Travel Kuper Preparation meeting	Paris-Ams-Kano Kano	 Mission team, G. Christophe (FE)
25/9	Meeting BEC Trip Kano-Auyo Field trip Meeting WUA Ayama Discussion French Evaluation Team	Kano HVIP HVIP Auyo	M. Le Man Mission team, G. Christophe Field mission team, G. Christophe J. McRay, A. Ramalan
26/9	Appraisal DS '99/'00, WS 2000 Visit to HVIP	Auyo Hadejia	Field mission team, G. Christophe
27/9	Appraisal DS '99/'00, WS 2000 Orientations DS 2000/2001 Travel Litrico	Auyo Auyo Montpellier-Paris	Field mission team
28/9	Preparation hydraulic measurement campaign Analysis information system Travel Litrico	Auyo Paris-Ams-Kano	Kuper, Othman, Langen Kuper/Chaussenot
29/9	Seminar information system Meeting Information System Monitoring Committee Field preparation of the hydraulic measurement campaign	Auyo Auyo Hadejia	Chaussenot Field mission team Hydraulic team
30/9	Work plan Chaussenot, TOR CSN Planning meeting 2001 Planning meeting 2001 Preparation of the hydraulic measurement campaign	Auyo Auyo Auyo Hadejia	Kuper, Chaussenot Kuper, NAERLS Kuper, HJRBDA/HVIP Hydraulic team
1/10	Definition of performance indicators Define proposal 2001	Auyo Auyo	Field mission team Field mission team
2/10	Wrap-up meeting Measurement campaign	Auyo Hadejia	Field mission team Hydraulic team
3/10	Travel Auyo-Kano Meeting HJRBDA Measurement campaign	 Kano Hadejia	Kuper/Chaussenot MD, ED Services, ED O&M Hydraulic team
4/10	Travel Meeting NAERLS Measurement campaign	Kano-Zaria Zaria Hadejia	Kuper/Chaussenot Director Hydraulic team
5/10	Travel Zaria-Abuja Meeting French Embassy Analysis/evaluation campaign Travel Auyo-Kano Travel Abuja-Kano	 Abuja Hadejia	Kuper/Chaussenot Attaché SCAC Hydraulic team Litrico, Othman Kuper/Chaussenot
6/10	Review meeting	Kano	Mission team
7/10	Orientations and work plan Chaussenot Field trip to Tiga dam (KRIP)	Kano	Kuper, Litrico Kuper, Litrico
8/10	Synthesis, trip report Meeting BEC	Kano	Kuper, Litrico M. Le Man
9/10	Travel	Kano-Ams-Paris-MPL	Kuper, Litrico

The mission team consisted of: M.U. Kura (project co-ordinator HJRBDA), Y.D. Kazaure (HJRBDA), Y.D. Yiljep (acting project co-ordinator NAERLS), G.B. Murtala (NAERLS), M.K. Othman (NAERLS), N. Chaussenot (FTAC), M. Kuper (CIRAD).

The field mission team consisted of the mission team as well as the following staff from HVIP: Z.Z. Abubakar, U.I Langen, B.K. Hussaini, H. Garba, M. Imrana, M. Shuaibu.

The hydraulic team consisted of U.I. Langen (HVIP), M.K. Othman (NAERLS) and X. Litrico (CEMAGREF).

The addresses of the contact persons of the different partners are presented below:

Institution	Address	Telephone	Fax	E-mail
CIRAD-TERA	TA 60/15 73, rue J.-F. Breton 34398 Montpellier cedex 5	(33) [04] 67615639	(33) [04] 67614415	marcel.kuper@cirad.fr
CEMAGREF	Division Irrigation, Domaine de Lavalette, 361 rue J.F. Breton, BP 5095, 34033 Montpellier cedex 1	(33) [04] 67046300	(33) [04] 67635795	xavier.litrico@cemagref.fr
French Embassy	32 Udi Street, off Aso Drive, Maitam, Abuja	(234) [09] 5235088 (234) [09] 5235076	(234) [09] 5235482 (234) [09] 5235072	guy.christophe@diplomatie.gouv.fr
HJRBDA	No. 38, Hotoro, Maiduguri road, P.M.B. 3168, Kano	(234) [064] 668183 (234) [064] 663528	(234) [64] 666631	Yahayadk@yahoo.com hjrbda@infoweb.abs.net
NAERLS	Ahmadu Bello University, P.M.B. 1067, Zaria	(234) [069] 550589 (234) [069] 551435	(234) [069] 552198	NAERLS@abu.skannet.com yiljep@abu.edu.ng

Annex 7: Photographic documentation

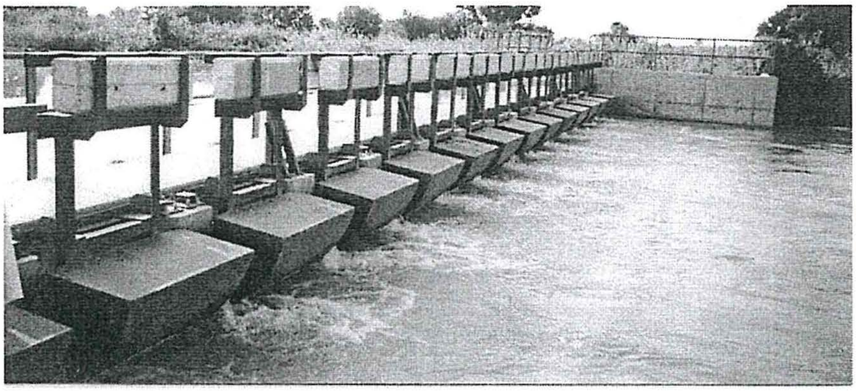


Figure 20: Cross regulator at Marina: round back gates

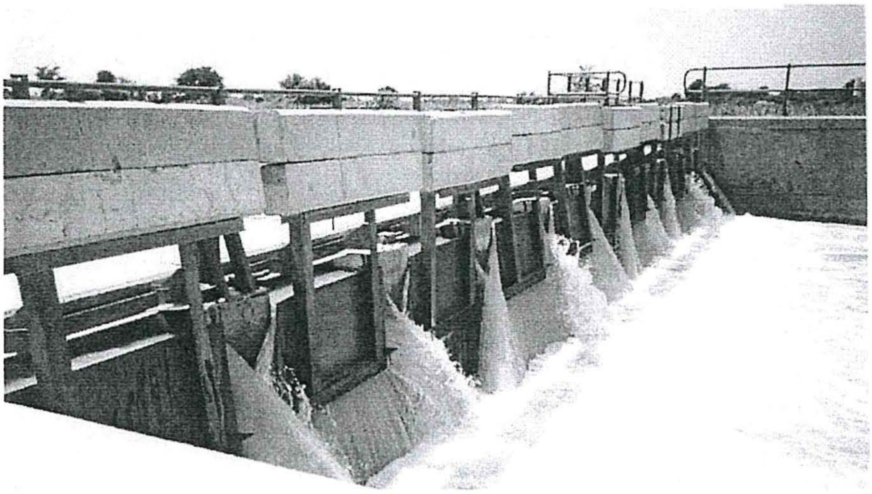


Figure 21: Cross regulator at Furawa: flat back gates

Figure 22: Erosion of the North Main Canal banks

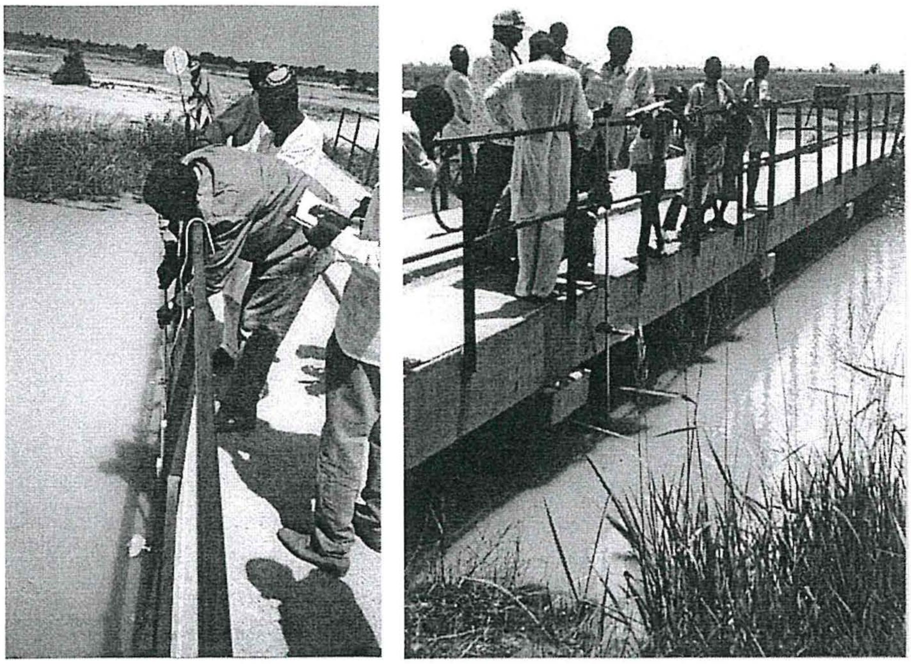


Figure 23: Hydraulic team undertaking a discharge measurement at Meshaywa bridge

Figure 24: Water flowing over the side spillway at Yamidi 02/10/2000

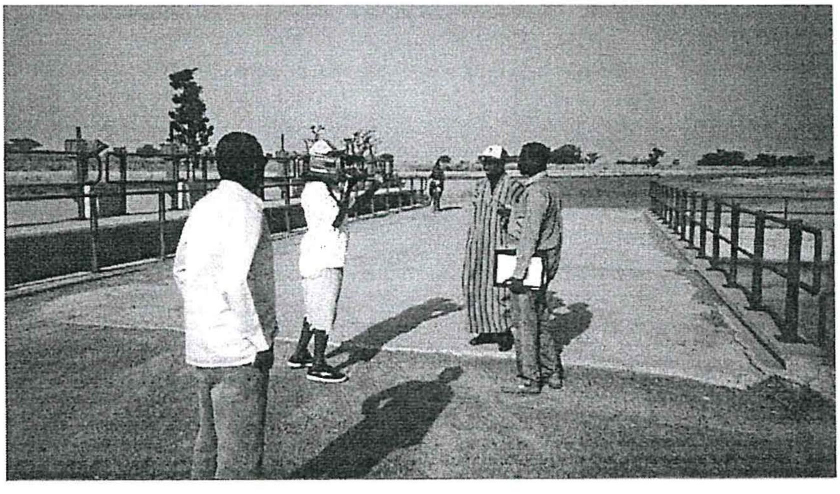


Figure 25: Interview of the gauge readers by the hydraulic team

Figure 26: Deterioration of the STO gates at Marina

Figure 27: Output of the measurement campaign at Gamsarka STO

Figure 28: Dynamometer measurements at Marina CR

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