Hydraulic Performance of a Downstream Controlled Irrigation Canal Equipped with Different Offtake Types

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ABSTRACT

Regarding canal management modernization, water savings and water delivery quality, the study presents two automatic canal control approaches of the PI (Proportional and Integral) type: the distant and the local downstream control modes. The two PI controllers are defined, tuned and tested using a hydraulic unsteady flow simulation model, particularly suitable for canal control studies. The PI control parameters are tuned using optimization tools. The simulations are done for a Portuguese prototype canal and the PI controllers are analyzed and compared considering a demand-oriented-canal operation.

The paper presents and analyzes the two control modes answers for five different offtake types – gate controlled weir, gate controlled orifice, weir with or without adjustable height and automatic flow adjustable offtake. The simulation results are compared using water volumes performance indicators (considering the demanded, supplied and the effective water volumes) and a time indicator, defined taking into account the time during which the demand discharges are effective discharges.

Regarding water savings, the simulation results for the five offtake types prove that the local downstream control gives the best results (no water operational losses) and that the distant downstream control presents worse results in connection with the automatic flow adjustable offtakes.

Considering the water volumes and time performance indicators, the best results are obtained for the automatic flow adjustable offtakes and the worst for the gate controlled orifices, followed by the weir with adjustable height.

Keywords: Irrigation canal, PI controller, downstream control, irrigation offtake, water saving, performance indicators.
1. INTRODUCTION

Irrigation is the largest water user in the World, using up to 85% of the available water in the developing countries (Plusquellec et al. 1994). In the near future, irrigation will have to share the water with industrial and urban water users and to pay the same price for this scarce natural resource.

The Agriculture must be prepared for this announced competition, namely developing and implementing intelligent management and operation of the irrigation systems, in order to achieve higher water savings and better water delivery service, within a short period of time.

Due to technical and financial reasons, the large water conveyance and delivery systems are usually open-channel systems. The canal dynamics is very complex and difficult to control, especially if there is a demand-oriented-operation (Clemmens, 1987).

The main purpose of the canal control is to optimize the water supply in order to match the expected or aleatory water demands at the offtakes level. Basically, there are two canal control logics – upstream control and downstream control – respectively if the information about the real state of the hydraulics system needed by the control system arrives from upstream or downstream (control system input).

1.1 Local Upstream Control Vs. Distant Downstream Control

With the local upstream control, water depth at the downstream end of each canal pool remains relatively constant ($h_d$ controlled by the gate G2 controller, Figure 1a). It is the most used control method. The main reasons for that are: canals can be sized to convey the maximum steady flow and water depths in steady flow conditions never exceed the normal depth for the designed flow. As it is shown, the water surface profile pivots around the established constant downstream depth value ($h_d$), according to the flow. A storage wedge is created between different steady-state flows profiles (Figure 1a represents the maximum difference, between maximum and null flow surface profiles). When flow changes, the water surface and storage volume within the pool must also change in the same direction (increasing or decreasing).

Because of storage volume variations, the upstream control is particularly effective when associated with programmed delivery methods (supply-oriented-operation), like rotation (Clemmens, 1987). This method has disadvantages when combined with flexible delivery methods (demand-oriented-operation) because pool storage must change opposite to the natural tendency (Buyalski, et al. 1991). With the last kind of operation, operational water losses are always significant.

If changes in water demand can be predicted, the inflow can be changed in advance and the operation becomes more effective and efficient. For this reason, anticipation is often used to improve the control response (Goussard, 1993; Rogers et al., 1995). Distant downstream control, where gate G1 is controlled in order to keep constant $h_d$ (Figure 1b), guarantees this anticipation phase and, for this reason, can be used in order to improve upstream control and modernize the old irrigation canals. Now, the control can answer better to the aleatory outflows, but water demands can be neither abrupt nor of great amplitude, because the canal pool hydraulics remains the same as the upstream control (Buyalski, et al.1991; Goussard, 1993).
1.2 Local Downstream Control

Water depth at the upstream end of each canal pool remains relatively constant \( h_u \), controlled by the gate G1 controller, Figure 1c) – was the first control method developed to optimize the demand-oriented-operation. Now, the water surface profile pivots around \( h_u \). When the flow changes, the water surface gradient and storage volume within the pool also change, but in opposite direction, and the storage wedge (now, a real internal water reserve) can answer, instantaneously, to the outflows variations with the maximum efficiency (Buyalski, et al.1991; Goussard, 1993). Although, considering the null flow surface profile, canal bench have to be horizontal and canal building becomes much more expensive and difficult.

Figure 1. Distant and local canal control modes.

Considering a demand-oriented-canal operation, the main goal of the paper is to present a comparative analysis of two PI controllers (Proportional-Integral), the distant and the local downstream canal control modes, developed and tuned for a Portuguese canal prototype using an hydraulic model. The analysis is made considering the hydraulic response of the canal (the water delivery quality) for five different offtake types – gate controlled weir, gate controlled orifice, weir with or without adjustable height and automatic flow adjustable offtake. The simulation results are compared using water volumes performance indicators (considering the demanded, supplied and the effective water volumes) and a time performance indicator, defined taking into account the time during which the demand discharges are effective discharges.

2. HYDRAULIC AND CONTROL SIMULATION MODELS

The study was performed for the Main Canal of the Irrigation Project of Macedo de Cavaleiros (Portugal), that it is here briefly described, considering main simulation needs. The basic and the offtakes equations of the used hydraulic model, model SIC “Simulation of Irrigation Canals” (SIC, 2000), are also briefly presented.
The numerical simulator SIC permits the installation and development of PI controllers and the respective control gains tuning.

2.1 Canal Description

The Main Canal of the Irrigation Project of Macedo de Cavaleiros is a lined canal with 19.1 km long, composed by twelve pools separated by gated cross structures. The usual cross section is trapezoidal, with a side slope of 1:1 (H:V) and the longitudinal bottom slope is 0.30 m/km. The canal design flow is 2.56 m$^3$s$^{-1}$, but the canal operates only with a maximal flow of 1.28 m$^3$s$^{-1}$ (accumulation of the total offtakes flows, Table 1). Each cross structure is composed by a undershot sluice gate (gates G0….G11, Table 1), with dimensions of 0.9 m x 1.25 m (width x height) and the canal has seven offtakes (T1….T7, Table 1).

2.2 Hydraulic Model

2.2.1 Basic Equations

The hydraulic model SIC uses the well known Saint-Venant equations to simulate the dynamic behavior of water within the canals. These equations are nonlinear hyperbolic partial differential equations, respectively dealing with the mass conservation and momentum conservation:

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{1}
\]

\[
\frac{\partial Q}{\partial t} + \frac{\partial Q^2}{\partial x} + gA \frac{\partial Z}{\partial x} = gA(i - J) \tag{2}
\]

where $A(x,t)$ is the watered area ($m^2$), $Q(x,t)$ is the discharge ($m^3s^{-1}$) across section $A$, $Z(x,t)$ is the water surface elevation ($m$), $i$ is the bed slope, $J(x,t)$ is the friction slope ($m/m$) and $g$ is the gravitational acceleration ($ms^{-2}$).

<table>
<thead>
<tr>
<th>Gate</th>
<th>Offtake</th>
<th>Location (m)</th>
<th>Offtake design flow ($m^3s^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G0</td>
<td>--</td>
<td>0</td>
<td>--</td>
</tr>
<tr>
<td>G1</td>
<td>T1</td>
<td>1694</td>
<td>0.048</td>
</tr>
<tr>
<td>G2</td>
<td>T2</td>
<td>3430</td>
<td>0.056</td>
</tr>
<tr>
<td>G3</td>
<td>T3</td>
<td>5080</td>
<td>0.137</td>
</tr>
<tr>
<td>G4</td>
<td>T4</td>
<td>6900</td>
<td>0.088</td>
</tr>
<tr>
<td>G5</td>
<td>T5</td>
<td>8360</td>
<td>0.084</td>
</tr>
<tr>
<td>G6</td>
<td>--</td>
<td>10026</td>
<td>--</td>
</tr>
<tr>
<td>G7</td>
<td>--</td>
<td>10808</td>
<td>--</td>
</tr>
<tr>
<td>G8</td>
<td>--</td>
<td>12527</td>
<td>--</td>
</tr>
<tr>
<td>G9</td>
<td>--</td>
<td>14186</td>
<td>--</td>
</tr>
<tr>
<td>G10</td>
<td>T6</td>
<td>15846</td>
<td>0.211</td>
</tr>
<tr>
<td>G11</td>
<td>--</td>
<td>17479</td>
<td>--</td>
</tr>
<tr>
<td>--</td>
<td>T7</td>
<td>19099</td>
<td>0.656</td>
</tr>
</tbody>
</table>
Two boundary conditions are necessary for this partial differential system, for example \( Q(0,t) = Q_0(t) \) and \( Q(X,t) = Q_X(t) \), where \( X \) is the length of the considered channel. The initial conditions are given by \( Q(x,0) \) and \( Z(x,0) \).

The equations (1) and (2) are not valid to model cross structure behavior. Cross structure equations are numerous and are not valid for all kind of flow (submerged, free flow...). The general form is: \( Q = f(Z_i, Z_j, W) \), with \( Z_i \) (m) as upstream water elevation, \( Z_j \) (m) as downstream water elevation and \( W \) as gate opening (m). In the case of a weir, the general form is: \( Q = f(Z_i) \), with \( Z_i \) referred to the weir crest (SIC, 2000).

The equations (1) and (2) are linearized and discretized in time (\( \Delta t \) time step) and space (\( \Delta x \) space step) through the implicit Preissmann finite difference scheme (Cunge et al., 1980).

### 2.2.2 Offtakes Equations

In the study, five offtake types were considered: gate controlled weir; gate controlled orifice; weir with adjustable height; weir without adjustable height; automatic flow adjustable offtake (for example an orifice with a motorized valve automatically controlled in order to adjust the flow to the demanded flow independently of the water level within the canal). Figure 2 presents schematically the first four offtake types, with the following relating flow equations:

- a) **Gate controlled weir**
  \[
  Q = L \sqrt{2g \left[ \mu h_i^{\frac{2}{3}} - \mu_1 (h_1 - W)^{\frac{2}{3}} \right]} 
  \]
  where \( L \) is the weir or gate width (m), \( p \) is the sill elevation (m), \( h_1 \) is the upstream water depth referred to the associated weir (m), \( W \) is the gate opening (m) and \( \mu, \mu_1 \) and \( \mu_F \) are discharge coefficients (SIC, 2000).

- b) **Gate controlled orifice**
  \[
  Q = \mu L \sqrt{2g \left[ h_i^{\frac{2}{3}} - (h_1 - W)^{\frac{2}{3}} \right]} 
  \]

- c) **Weir with or without adjustable height (p)**
  \[
  Q = \mu_F L \sqrt{2g \ h_i^{\frac{2}{3}}} 
  \]

where \( L \) is the weir or gate width (m), \( p \) is the sill elevation (m), \( h_1 \) is the upstream water depth referred to the associated weir (m), \( W \) is the gate opening (m) and \( \mu, \mu_1 \) and \( \mu_F \) are discharge coefficients (SIC, 2000). The adjustable gate (offtake type a or b) or the weir with adjustable height are positioned for the demanded outflow, considering the target value for \( h_1 \); similar manual procedure is considered for the adjustable weir width (weir without adjustable height).

### 2.3 Control Model. PI Controllers Tuning

The Proportional, Integral and Derivative (PID) control algorithm is by far the most commonly used in control engineering and its philosophy has been integrated to the number of canal control methods. The derivative term is used to anticipate the response and the integral to eliminate the static error. The PID is very often reduced to a PI controller, what happens also in the present case.
study, because it is difficult to tune it properly (Astrom, 1995) and, by the other hand, it’s used, mostly, in slow processes subjected to abrupt variations and of big amplitude, what does not happen in irrigation canals. The PID algorithm can be written as:

\[ U(t) = K_p \cdot e(t) + K_i \int e dt + K_d \frac{de}{dt} \]  

(6)

where \( U \) is the control action (gate opening in the case), \( e(t) \) is the error or deviation of the controlled variable (water level in the case) from its target value at time \( t \) and \( K_p, K_i \) and \( K_d \) are the proportional, integral end derivative gains.

The most usual procedure for tuning PI controllers is the iterative method (Astrom, 1995). Its disadvantage is that several interconnected optimal controllers do not guarantee a globally optimal one. So, in the present study, an optimization method was used to determinate the globally best tuning of the PI controllers for a given set of perturbations at the offtakes level (Rijo, 2003). Optimal values for the gains are found by minimizing a performance criterion. To find the global minimum, an algorithm derived from non-linear programming (the simplex method) was used (Baume et al., 1999):

\[ \xi = \sum_{i=1}^{n} \int_{0}^{T} [Y_i(t) - Y_{ri}] + \delta W_i \cdot dt \]  

(7)

where \( T \) is the length of the scenario (s), \( Y_i \) is the measured water level (m) and \( Y_{ri} \) is the target water level at the pool \( i \) (m) and \( \delta W \) is the gate opening variation (m).

For water levels within irrigation canals, large deviations from the correspondent target values and oscillations are dangerous. So, the performance criteria used was based on the integral of the water level errors and the integral of the gate opening variations, in order to avoid large variations of gate opening (Baume et al., 1999).
3. PERFORMANCE INDICATORS

For the water delivery quality analysis it was considered the following performance indicators, permitted by the hydraulics model SIC.

3.1 Water Volume Indicators

The volume indicators relate three kinds of water volumes:

- The demand volume \( V_D \), which is the target volume at the offtakes;
- The supply volume \( V_S \), which is the volume supplied to the offtakes;
- The effective volume \( V_{EF} \), which is the really usable part of the supplied volume.

The definition of the effective volume depends on two coefficients: the upper limit \( w \) and the lower limit \( x \) (in %):

\[
\begin{align*}
\text{If } (1 - \frac{w}{100}) \cdot Q_D & \leq V_S \leq (1 + \frac{x}{100}) \cdot Q_D \Rightarrow Q_{EF} = V_S \\
\text{If } Q_S & < \left(1 - \frac{w}{100}\right) \cdot Q_D \Rightarrow Q_{EF} = 0 \\
\text{If } Q_S & > \left(1 + \frac{x}{100}\right) \cdot Q_D \Rightarrow Q_{EF} = \left(1 + \frac{x}{100}\right) \cdot Q_D
\end{align*}
\]

and \( V_{EF} = \int Q_{EF} \cdot dt \)

The effectiveness parameters, \( w \) and \( x \), were considered 20%.

Only the supply discharge close to the water demand is thus taken into account. In Figure 3, the effectiveness volume is shaded.
Three volume indicators are defined as:

\[ IND_1 = \frac{V_S}{V_D} \quad ; \quad IND_2 = \frac{V_{EF}}{V_D} \quad ; \quad IND_3 = \frac{V_{EF}}{V_S} \]

These indicators can be defined for a single offtake or for an offtake set.

### 3.2 Time Indicator

Defining \( T_D \) as the total period of time during which the demand discharge is non-zero and \( T_{EF} \) as the total period of time during which the effective discharge is non-zero, the time indicator:

\[ IND_4 = \frac{T_{EF}}{T_D} \]

Compares the duration of delivery of the effective volume with that of the demand volume. This indicator is dimensionless and can only be calculated for individual offtakes, because it doesn’t have any significance for all the offtakes taken together. For the \( IND_4 \) establishment, two time lags were defined: \( \Delta T_1 \) and \( \Delta T_2 \). \( \Delta T_1 \) is the time separating the start of water demand and the start of the effective discharge. This time is positive if the effective discharge arrives after the demand discharge (Figure 3b). \( \Delta T_2 \) is the time lag between the centers of gravity of the demand hydrograph and the effective delivery hydrograph.

This indicator can be calculated for any particular period of the simulation the user wants to focus on. In the present study, it was considered the entire simulation period.

Unity is the best value for the four performance indicators.

### 4. SIMULATION RESULTS

Figures 4 to 8 show the hydraulic simulation results for the offtake 2, considering the five offtake types and the two downstream control models under study. For the hydraulic simulations, it was also considered that all the others offtakes were of the same type as the offtake 2.

The lower part of all figures shows that the water level variations are bigger for the local control. This happens because the offtake is located at the downstream part of the canal pool and, for this control mode, the control section is the upstream canal pool section (Figure 1c); by contrary, for
the distant downstream control, the control section and the offtake have the same location – downstream end of the pool (Figure 1b).

The water delivery stability is worse for the weir with or without adjustable heights (upper part of Figures 6 and 7). However, the water volume and time indicators are worse for the gate controlled orifice (Figure 5) and for the weir with adjustable height (Figure 6). As it was expected, best results are obtained with the automatic flow adjustable offtakes (Figure 8).

Figure 9 presents the water volumes and time indicator results for the canal offtake set. According to the logic of the local downstream control, there are no water operational losses (null tail end outflows), what is confirmed with the present study (Figure 9). However, it is not the case for the distant downstream control (also as expected), where the operational water losses are significant, mainly for the automatic flow adjustable offtakes, what is understood because there are no variations between the supplied and demanded outflows, and also for the gate controlled orifices (Table 2).

Figure 9 also shows that: the best performances are obtained for the automatic flow adjustable offtakes \((IND_1=IND_2=IND_3=IND_4=1)\); for the manual controlled offtakes (all the other four types), the best results are obtained for the gate controlled weir, because the associated sill elevation is small and, for this reason, the outflows are less sensitive to the head variations; there are no variations of the indicators \(IND_1, IND_2\) and \(IND_3\) for the weir with or without adjustable height and gate controlled orifice with the local control mode; only more or less 65\% of the demand flow are effective for the weir with or without adjustable height and the gate controlled orifices for the two control modes under study (with the exception of the weir without adjustable height connected with the distant control mode).

For the indicator \(IND_4\), Figure 9 shows that best results are obtained for the automatic flow adjustable offtakes for the two control modes \((IND_4=1\) , what means that the total period of time during which the demand discharge is non-zero is also effective discharge), followed by the weir without adjustable height \((IND_4\ close to the unity, with the exception of the offtakes 1 and 6 for the local control mode). The same figure also shows that the worst results for the same indicator are obtained for the gate controlled orifice (below 40\% for the offtakes 1, 2, 5 for the two control modes, followed by the weir with adjustable height).

Table 2 shows the outflow water volumes at the downstream section of the canal for the five offtake types and considering the distant downstream control. These water volumes are control operational water losses. As already mentioned, for the local downstream control there are no operational water losses because each canal pool is a real online water reserve. As can be seen, the automatic flow adjustable offtake guarantee the best performance, but the tail end water loss presents the biggest value.

5. MAIN CONCLUSIONS

Considering a demand-oriented-canal operation, the main conclusions are:

i) Control water losses (outflows at the downstream end of the canal)
   - Downstream local control guarantee no operational water losses;
   - Distant downstream control always presents water losses; worst results are obtained with the automatic flow adjustable offtakes;

ii) Water volumes and time indicators
- Time indicator – the gate controlled orifices and the weirs with adjustable heights present long time periods where the demanded outflows are not effective flows (IND4 ≠ 1);
- Best performance indicators are obtained for the automatic flow adjustable offtakes (IND1=IND2=IND3=IND4=1); worst results are obtained for the gate controlled orifices, followed by the weir with adjustable height.

6. ACKNOWLEDGEMENTS

The present study was supported by the “Fundação para a Ciência e Tecnologia” through the Research Project POCTI/GG/44060/2002.
Figure 4. Hydraulics simulation results for the Offtake 2, considering all the offtakes as gate controlled weirs.

Figure 5. Hydraulics simulation results for the Offtake 2, considering all the offtakes as gate controlled orifices.
Figure 6. Hydraulics simulation results for the Offtake 2, considering all the offtakes as weirs with adjustable heights.

Figure 7. Hydraulics simulation results for the Offtake 2, considering all the offtakes as Weirs without adjustable heights (variable widths).
Figure 8. Hydraulics simulation results for the Offtake 2, considering all the offtakes as automatic flow adjustable offtakes (gate controlled orifices).
Figure 9. Global water volumes and time indicators for the two control modes.
Table 2. Tail end water volumes for the distant downstream control.

<table>
<thead>
<tr>
<th>Offtake type</th>
<th>Tail end water availability (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gate controlled weir</td>
<td>419</td>
</tr>
<tr>
<td>Gate controlled orifice</td>
<td>572</td>
</tr>
<tr>
<td>Weir with adjustable height</td>
<td>210</td>
</tr>
<tr>
<td>Weir without adjustable height</td>
<td>298</td>
</tr>
<tr>
<td>Automatic flow adjustable offtake</td>
<td>585</td>
</tr>
</tbody>
</table>

7. REFERENCES


