Water Delivery Quality and Automatic Control Modes in Irrigation Canals. A Case Study.

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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 an 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 effectives 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 worse for the gate controlled orifices, followed by the weir with adjustable height.

Keywords. Irrigation canal, PI controller, local downstream control, distant downstream control, irrigation offtakes, water savings, performance indicators.

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 within a short period of time and better water delivery service.

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.

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.

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) – 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 flow 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, this control system 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 system response (Rogers *et al*., 1995). Distant downstream control, where gate G1 is controlled in order to keep constant h_d (Figure 1a), 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.

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 1b) – 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. Although, considering the null flow surface profile, canal bench have to be horizontal and canal building becomes much more expensive and difficult.

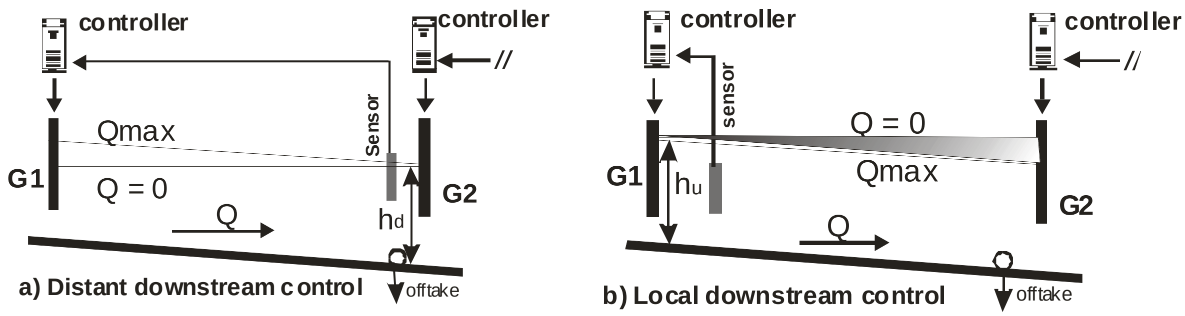


Figure 1. Distant and local downstream canal control modes.

The paper presents two *PI* controllers (Proportional-Integral), the distant and the local downstream canal control modes, developed, installed and tuned for a Portuguese canal prototype. The paper also 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 and time performance indicators for the offtakes and considering a demand-oriented-canal operation.

Hydraulics and Control Simulation Models

The study was done for the Main Canal of the Irrigation Project of Macedo de Cavaleiros (Portugal), that it is here briefly described, considering main simulations needs. The basic equations and the offtakes equations of the used hydraulics 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.

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, but the canal operates only with a maximal flow of 1,28 m^3/s (accumulation of the total offtakes flows, Table 1). Each cross structures is composed by a undershot sluice gate (gates G0….G11), with dimensions of 0,9 *m* x 1,25 *m* (width x height) and the canal has seven offtakes (T1….T7, Table 1).

Hydraulic Model

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:

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

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

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

Table 1. Gates and offtakes locations and offtakes design flows.

Gate	Offtake	Location (m)	Offtake design flow (m^3/s)
G0	--	0	
G1	T1	1694	0,048
G2	T2	3430	0,056
G3	T3	5080	0,137
G4	T4	6900	0,088
G5	T5	8360	0,084
G6	--	10026	--
G7	--	10808	--
G8	--	12527	--
G9	--	14186	--
G10	T6	15846	0,211
G11	--	17479	--
--	T7	19099	0,656

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) the upstream water elevation, Z_j (m) the downstream water elevation and W the 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 (Δt time step) and space (Δx space step) through the implicit Preissmann finite difference scheme (Cunge *et al.*, 1980).

Offtakes Equations

In the study, were considered five offtake types: gate controlled weir; gate controlled orifice; weir with adjustable height; weir without adjustable height; automatic flow adjustable offtake, weir or orifice automatic adjustable – outflows always equal to the demanded flows. Figure 2 presents schematically the first four offtake types, with the following relating flow equations:

a) gate controlled weir

$$Q = L\sqrt{2g}\left[\mu h_1^{\frac{3}{2}} - \mu_1 (h_1 - W)^{\frac{3}{2}}\right] \quad [3]$$

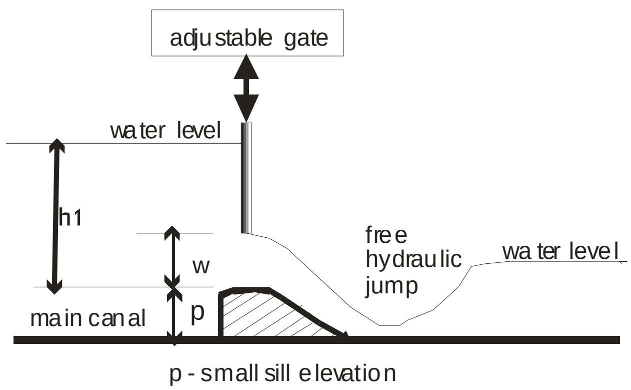
b) gate controlled orifice

$$Q = \mu L \sqrt{2g}\left[h_1^{\frac{3}{2}} - (h_1 - W)^{\frac{3}{2}}\right] \quad [4]$$

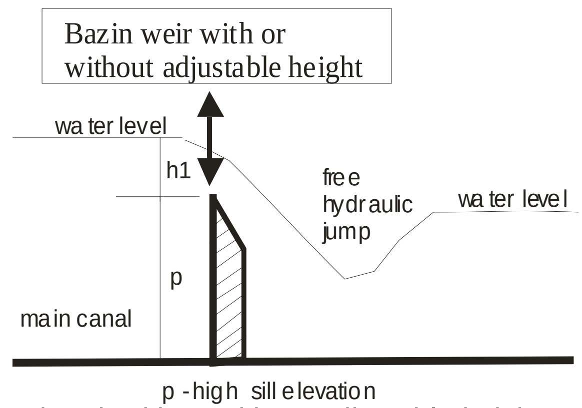
c) weir with or without adjustable height

$$Q = \mu_F L \sqrt{2g}\, h_1^{\frac{3}{2}} \quad [5]$$

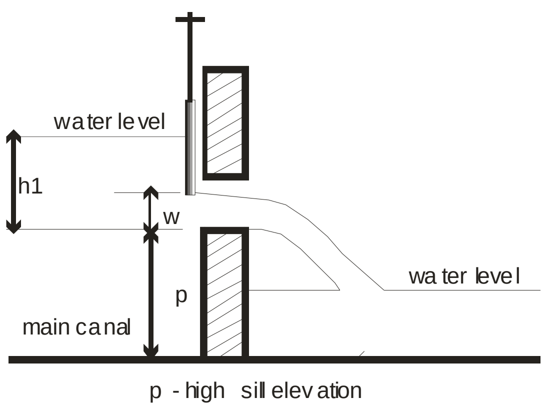
where L is the weir or gate width, p is the sill elevation, h_1 is the upstream water depth referred to the associated weir, W is the gate opening and μ, μ_1 and μ_F are discharge coefficients (SIC, 2000). The adjustable gate (offtake type a or b) or the weir with adjustable height are positioned for the demanded outflow, considering the target value for h_1; similar manual procedure is considered for the adjustable weir width (weir without adjustable height).



a) gate controlled weir



c) weir with or without adjustable height



b) gate controlled orifice

Figure 2. Canal offtake types

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 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:

$$U(t) = K_p \cdot e(t) + K_i \int edt + K_d \frac{de}{dt} \qquad [6]$$

where U is the control action (gate opening in the case), $e(t)$ 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. 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 criteria. To find the global minimum, an algorithm derived from non-linear programming (the simplex method) is used (Baume *et al*., 1999):

$$\xi = \sum_{i=1}^{n} \int_{0}^{T} \left[\left| Y_i(t) - Yr_i \right| + \delta W_i \right] \cdot dt \qquad [7]$$

where T is the length of the scenario, Y_i the measured water level and Yr_i the target water level at the pool i, δW is the gate opening variation.

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.

Performance Indicators

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

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 %):

$$\begin{cases} \text{If } (1-\frac{w}{100}) \cdot Q_D \le Q_S \le \left(1+\frac{x}{100}\right) \cdot Q_D \Rightarrow Q_{EF} = Q_S \\ \text{If } Q_S < \left(1-\frac{w}{100}\right) \cdot Q_D \Rightarrow Q_{EF} = 0 \\ \text{If } Q_S > \left(1+\frac{x}{100}\right) \cdot Q_D \Rightarrow Q_{EF} = \left(1+\frac{x}{100}\right) \cdot Q_D \end{cases}$$

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.

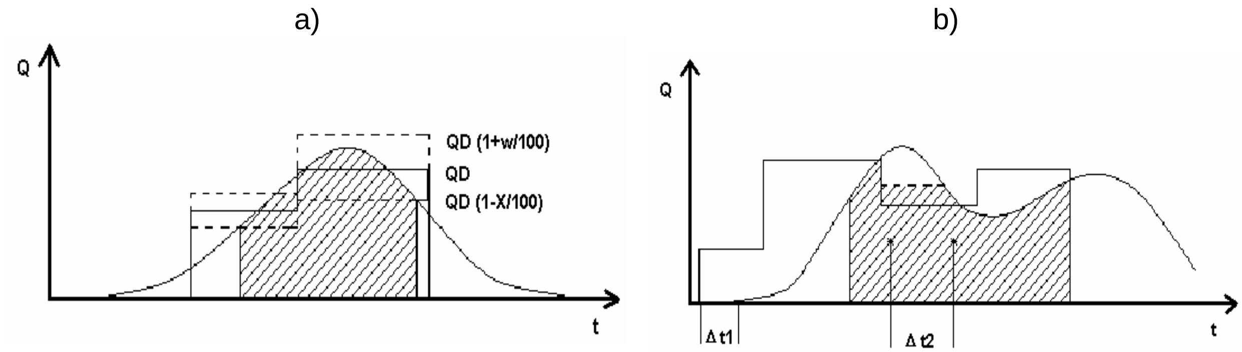


Figure 3. Definition of the effective volume (a) and time indicator (b).

Three volume indicators are defined:

$$IND_1 = \frac{V_S}{V_D}\ ;\ IND_2 = \frac{V_{EF}}{V_D}\ ;\ IND_3 = \frac{V_{EF}}{V_S}$$

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

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: ΔT_1 and ΔT_2. ΔT_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). ΔT_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.

Simulation Results

Figures 4 to 8 show the hydraulics simulation results for the offtake 2, considering the five offtake types and the two downstream control models under study. For the hydraulics simulations, it was also considered that all the others offakes were 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 offtakes are located at the downstream part of the canal pool and, for this control mode, the control section is the upstream canal pool section (Figure 1b), which is not the case for the distant downstream control (Figure 1a).

The water delivery stability is worse for the weir with or without adjustable heights (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 ouflows, 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).

Main Conclusions

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, when in association with flexible water delivery methods (demand-oriented-operation). Considering the four performance indicators, the best results are obtained for the automatic flow adjustable offtakes and the worst ones for the gate controlled orifices, followed by the weir with adjustable height.

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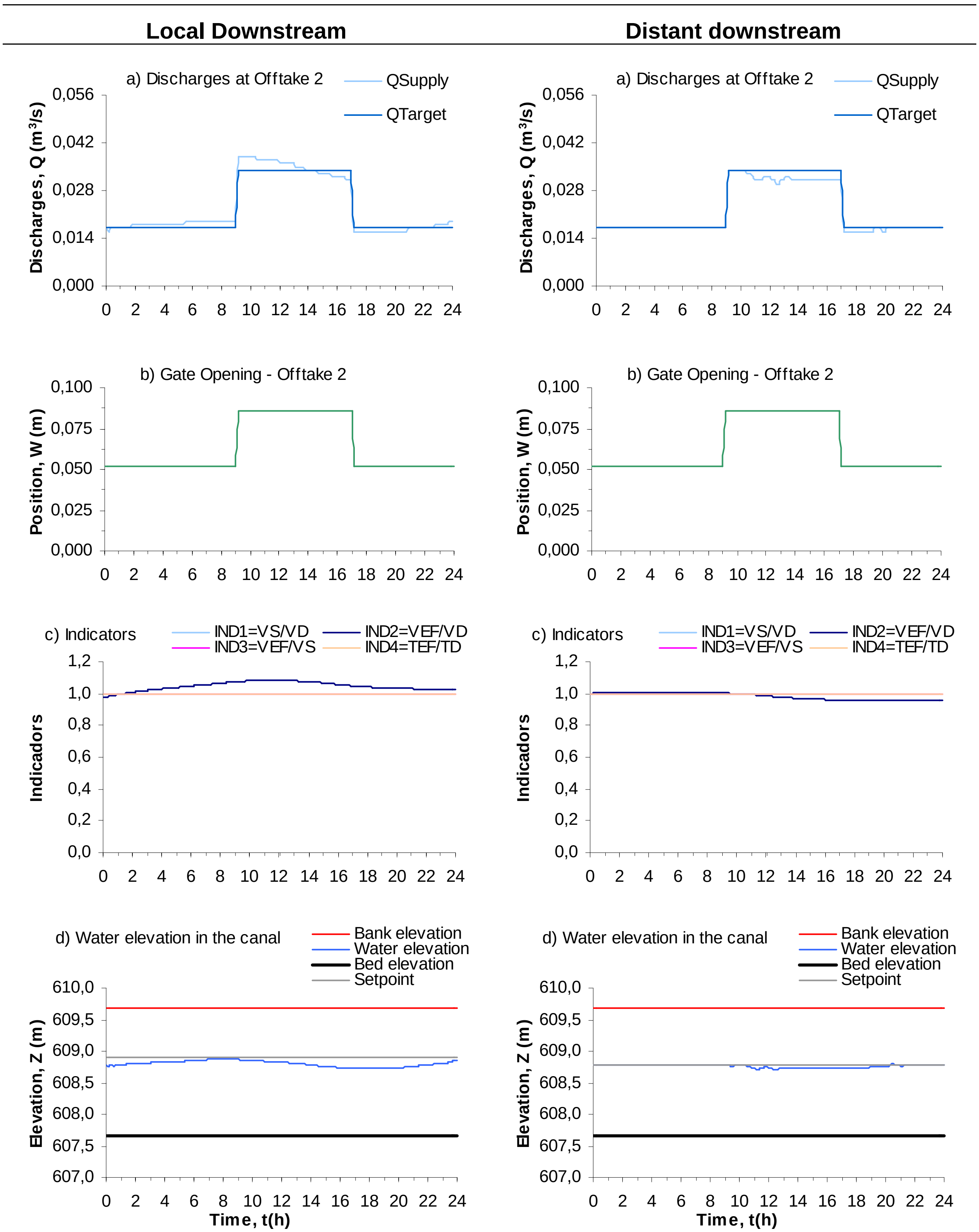

Figure 4. Hydraulics simulation results for the Offtake 2, considering all the offtakes as gate controlled weirs.

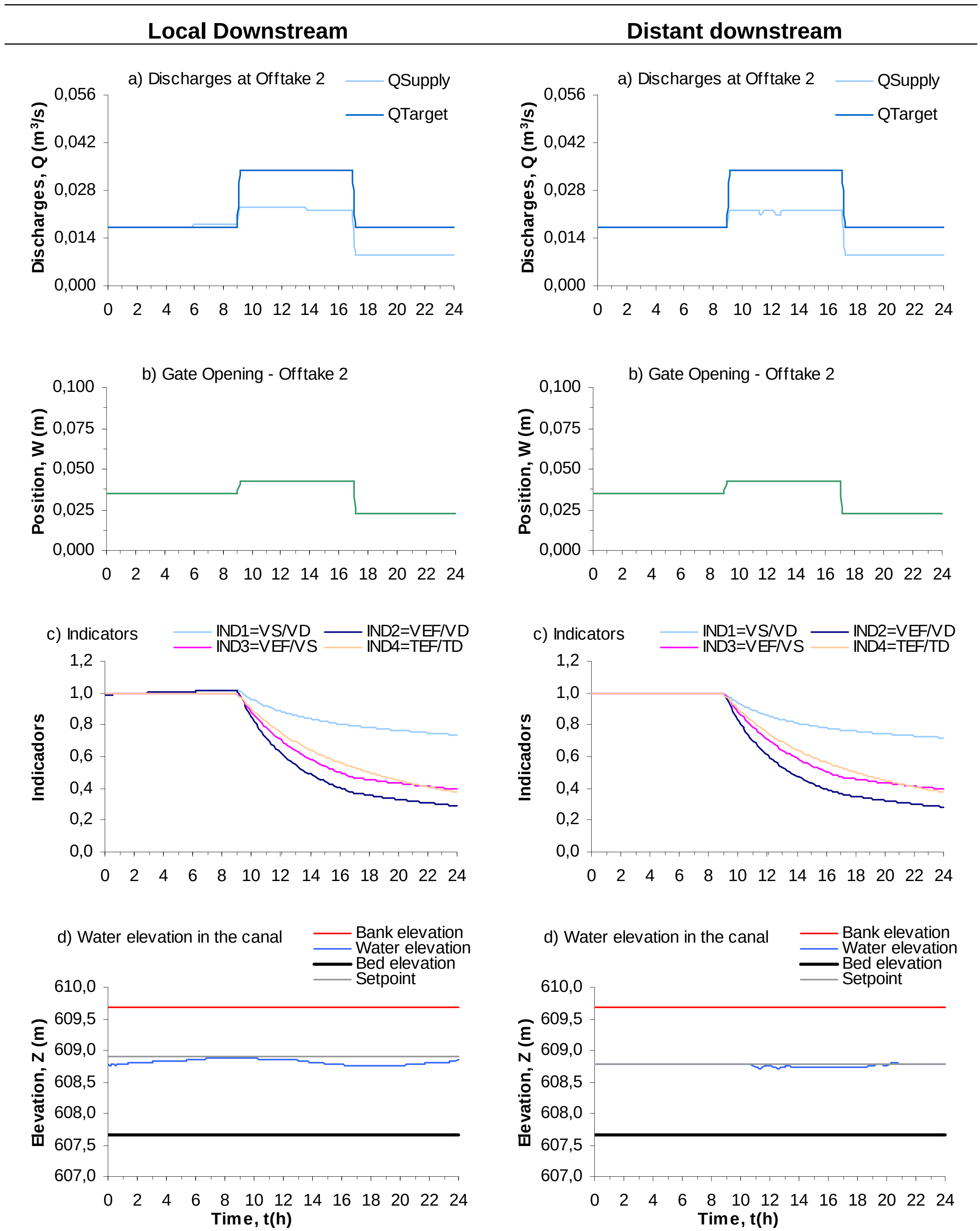

Figure 5. Hydraulics simulation results for the Offtake 2, considering all the offtakes as gate controlled orifices.

Local Downstream	Distant downstream

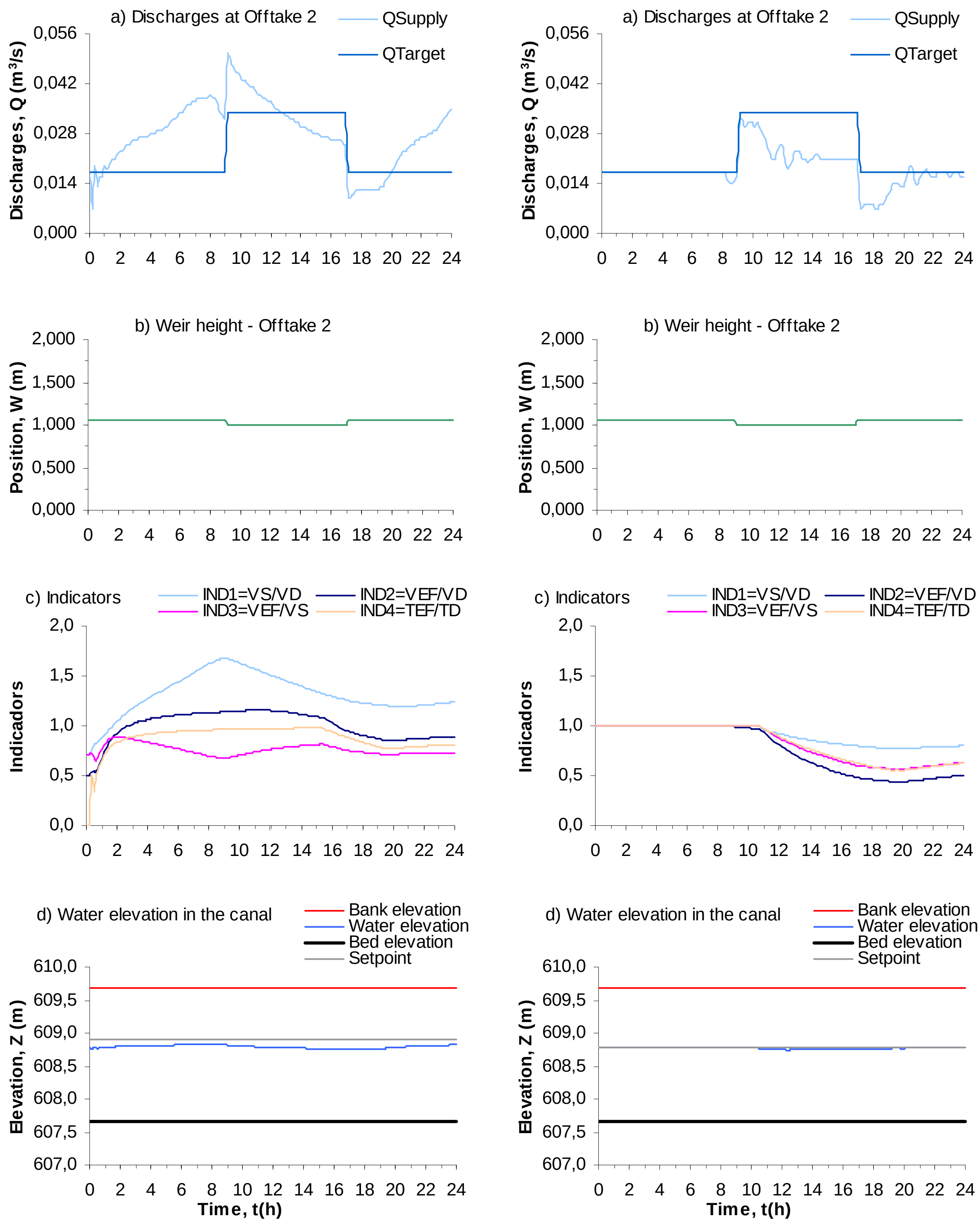

Figure 6. Hydraulics simulation results for the Offtake 2, considering all the offtakes as weirs with adjustable heights.

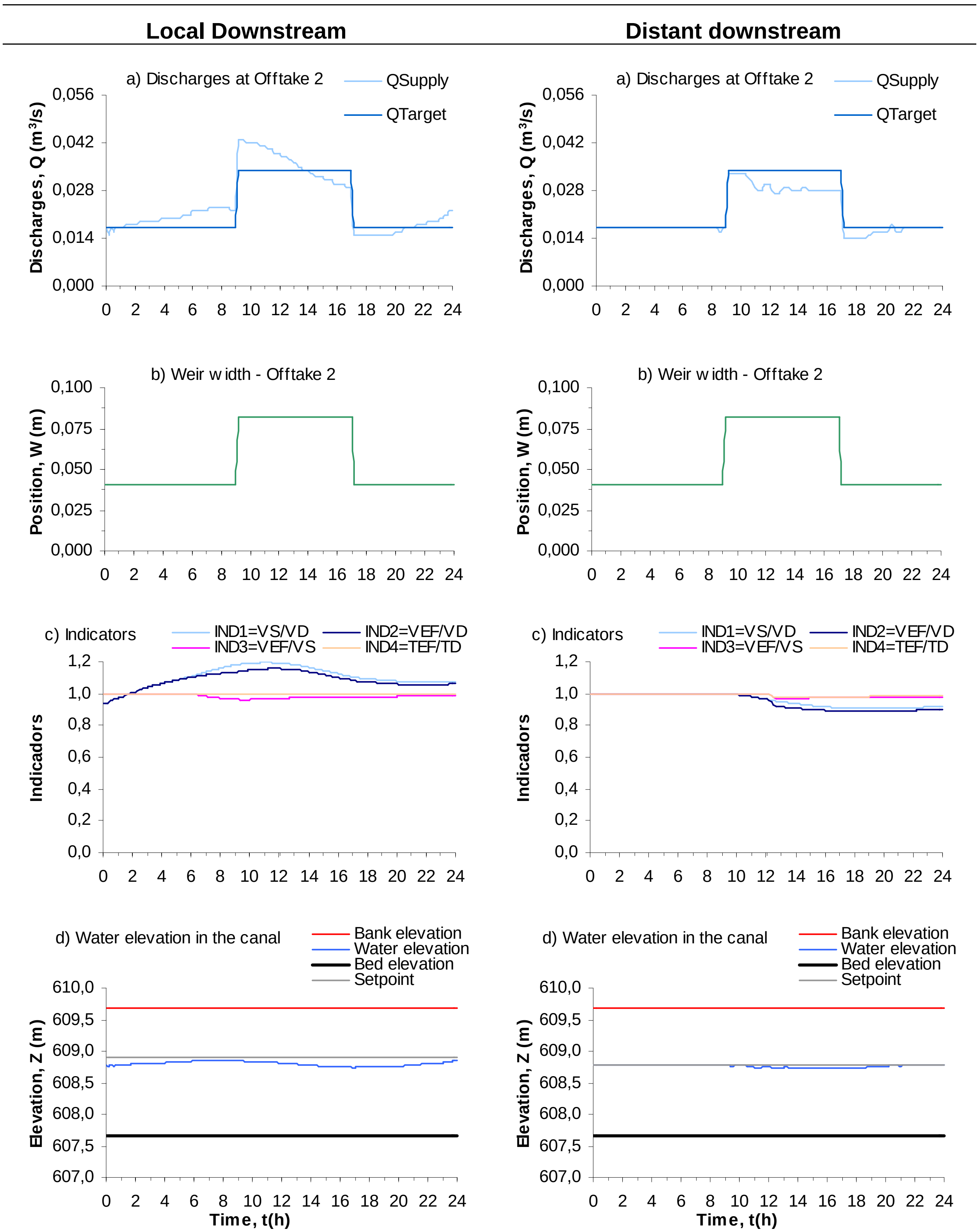

Figure 7. Hydraulics simulation results for the Offtake 2, considering all the offtakes as Weirs without adjustable heights (variable widths).

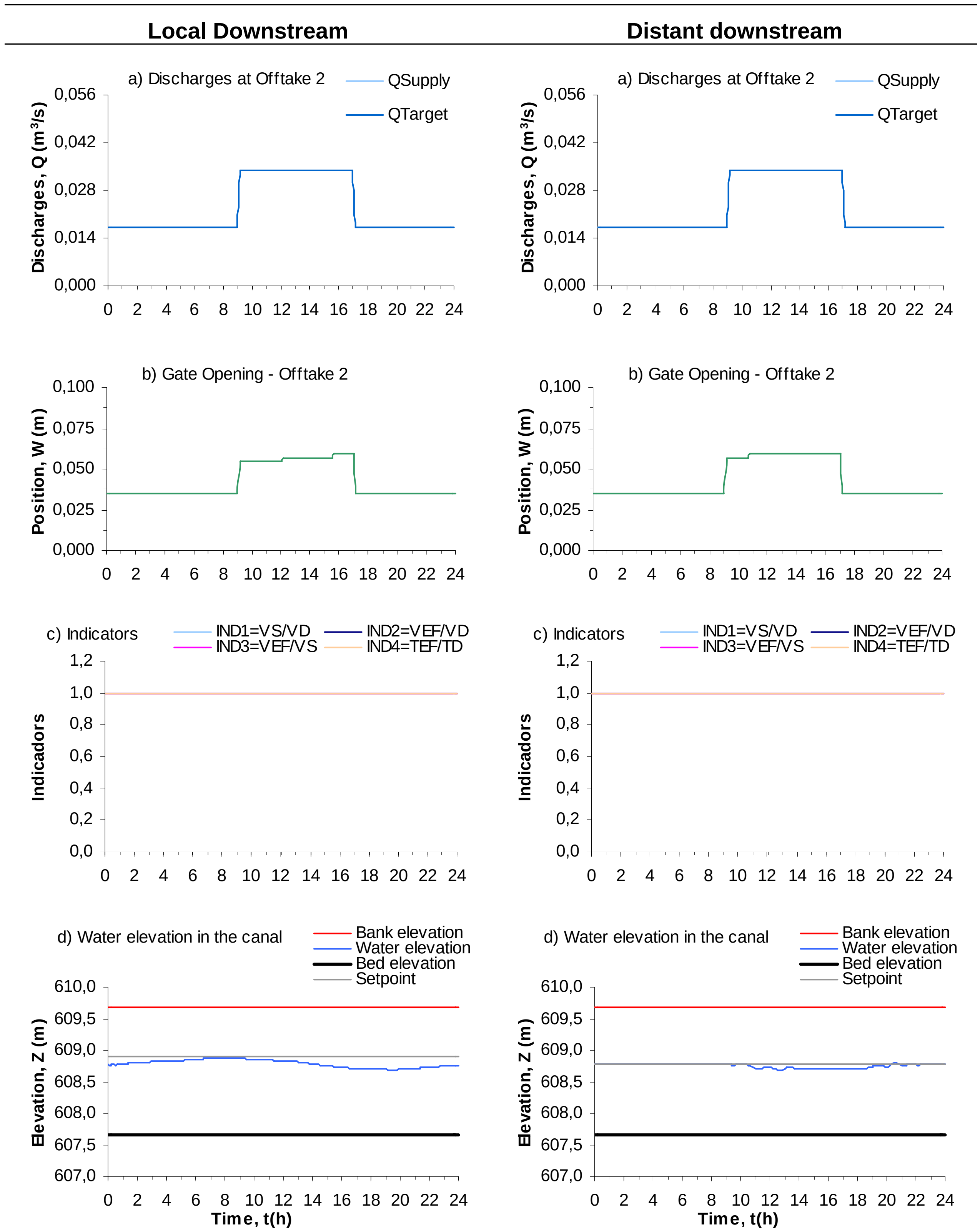

Figure 8. Hydraulics simulation results for the Offtake 2, considering all the offtakes as automatic flow adjustable offtakes (gate controlled orifices).

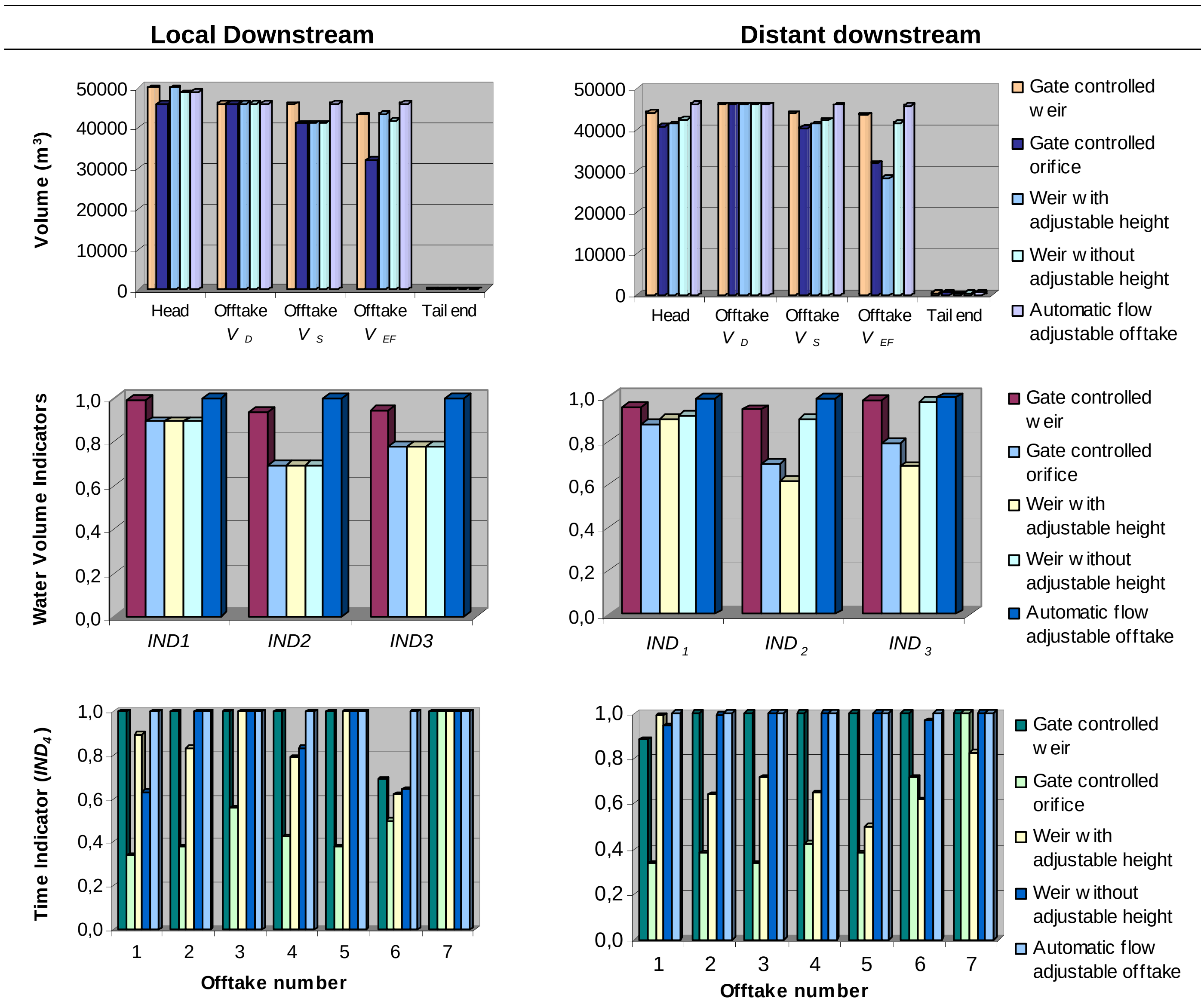

Figure 9. Global Water Volumes and Time Indicators for the two control modes.

Table 2. Tail end water volumes for the distant downstream control.

Offtake type	**Tail end water availability (m^3)**
Gate controlled weir	418,655
Gate controlled orifice	571,54
Weir with adjustable heigth	209,648
Weir without adjustable heigth	297,895
Automatic flow adjustable offtake	584,29

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