DISTRIBUTING FLOW MISMATCHES IN SUPPLY-CONSTRAINED IRRIGATION CANALS THROUGH FEEDBACK CONTROL

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ABSTRACT

The operation of main irrigation canals is complicated in situations where the operator does not have full control over the canal inflow, or where there are very long transmission distances from the point of supply, or both. Experienced operators are able to control the canal, but often supply errors are simply passed to downstream, thus creating problems further down the system. In previous work, the senior author showed that it is important to contain such errors and not let them pass downstream. With automatic upstream level control, all flow errors are passed to the downstream end of the canal. Distant downstream water level control requires full control of canal inflow. Without this, most errors will occur toward the upstream end of the canal. An alternative scheme is offered here where the canal check gates are controlled based on the relative water level error between adjacent pools. The scheme uses a simple linear model for canal pool response. The scheme is implemented as a multiple-input, multiple-output scheme and solved as a Linear Quadratic Regulator (LQR). Thus all gates respond to relative deviations from water-level set point. The scheme works to keep the relative deviations in all pools the same. If the canal has more inflow than outflow, the scheme will adjust gates so the water levels in all pools will rise together with the same deviation from set point. It thus distributes the error over the entire canal. When in equilibrium, operators will be able to judge the actual flow rate mismatch by the rate of change of these levels. The scheme acts like a combination of upstream level and distant downstream level control. It was tested on a simulation model of the Central Main Canal at the Central Arizona Irrigation and Drainage District, Eloy, AZ.

INTRODUCTION

Over the last several decades, irrigation districts have become more flexible in the service that they provide to users. Farmers need some level of flexibility in order to be efficient. This is particularly important where water is limited. However, irrigation districts are often constrained by their water supply infrastructure or by their water supplier. This hampers their ability to accommodate some requests by water users. Most districts require water users to request water ahead of time, so that they will have time to bring water to the site and arrange delivery. Order times are typically one to three days before the delivery is to begin. If the district stores water in a reservoir, it may take considerable time for the water to flow from the reservoir to the irrigated area. If the transmission time is more than a day and water order times are long, water users may feel constrained.

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In addition, water users sometimes need to change their water orders to deal with unforeseen circumstances. Irrigated farming requires a lot of adaptation in order to be successful. Thus changes in water orders are common. Districts that receive water from a water conservancy district or similar water authority are also sometimes constrained in their ability to change water orders.

Some districts use small regulating reservoirs along their canals to deal with the mismatches that inevitably occur. Other districts operate with small spills at the downstream end. In this paper, we describe a new water level control technique that takes the mismatches in supply and demand and distributes them over all canal pools. As such, the method effectively uses the canal as a storage reservoir. The method was tested on the Central Main Canal of the Central Arizona Irrigation and Drainage District through unsteady-flow simulation. Tests on the actual canal were delayed beyond the date of this publication.

EXISTING CANAL CONTROL METHODS

A common method for controlling canal check gates is to use some form of water level control, with the assumption that if the water levels are correct, then turnout flows will be correct. With upstream control, a check gate is typically adjusted to bring the water level to the target water level. If the water level is too high, the check gate opening is increased to allow more flow to pass through the structure, and thus the water level decreases. If the water level is low, the gate opening is reduced, decreasing the flow downstream. Good canal control can be achieved with this method if the correct amount of flow is supplied to the head of the canal. The operator sets the turnout gate so that when the water level is at the target level, the correct flow will pass through the turnout gate. Then, the upstream controller will pass the correct flow downstream from each gate. Upstream controllers are generally considered SISO – Single Input-Single Output (one water level – one gate). If there is an error in the canal inflow or if any of the turnout gates are set wrong, all the errors will pass downstream to the last canal pool. The operator thus must wait until these errors accumulate downstream before a reasonable correction can be made upstream. Uncorrected, these errors with either cause the last user to receive too little flow or cause a canal spill. Even if the gates and flows are initially set correctly, flow can drift over time because of weed plugs, changes in backwater downstream from turnout gates, etc.

Downstream water level control is intended to avoid the problems caused by the mismatch between supply and demand. When a water level deviates from the target value, control signals are sent to upstream gates to either increase or decrease the flow. Downstream controllers are slow relative to upstream controllers since they have to wait for flow changes to travel the length of the each pool. Downstream controllers essentially require an unlimited water supply at the canal head gate. A comparison to manual operation will give an idea of the magnitude of these changes.

When a canal operator releases the flow from the canal head gate, it takes some time to travel downstream to the turnout, thus there is a delay between the head gate flow change and the turnout flow change. Operators learn this timing through experience. The flow change times the delay time represents an additional volume that is added to the canal. Suppose a sudden change in the turnout flow occurs prior to a flow change at the head gate. If the operator immediately changes the head gate flow in response, it will be too late to accommodate the initial change in flow at the turnout. The canal water levels will change. To account for this delayed response at the head gate, the operator may make a larger flow change to account for this volume. So for example, if the turnout suddenly decreases by 10 cfs, the operator may decrease the inflow to the canal by 15 cfs for a while, and then change back to the 10 cfs decrease to match flow rates.

Feedback controllers respond in the same way, although they don't know what changes occurred. They only know that the water level deviated. Thus feedback controllers often makes larger flow changes at the canal head gate than the change in flow downstream because or the delay time and volume change in the pool. Even though this occurs for a short time, such flow changes may not be acceptable, or even feasible.

CONTROL BASED ON DIFFERENCES IN WATER LEVEL ERRORS

With automatic upstream water-level control, a check gate is controlled based on the water level just upstream. With automatic downstream level water level control, a check gate is adjusted based on the water level at the downstream end of the next pool downstream, or upstream from the next check gate downstream. Control actions are based on the water level error, e_j;

$$e_j = y_j - SP_j \quad (1)$$

where y_j is actual water level, SP_j, the water level set-point and where j identifies the check gate.

In the approach proposed here, control actions are based on the difference in water level error, D_j;

$$D_j = e_j - e_{j+1} \quad (2)$$

where for example if j=1, the control of check gate 1 is based on the water level just upstream from check gate 1 minus the water level error just upstream from check gate 2. Thus this represents a combination of upstream and downstream control. This controller differs from these two methods in an important way. If for example, the water levels in both pools are say 0.1 ft above the set point, this controller takes no action since $D_j = 0$.

For upstream control, if we have 7 canal pools, we can control 7 gates; excluding the head gate, but including the furthest downstream gate. For downstream control, we also can control 7 gates, but including the head gate and excluding the most downstream gate. For this difference controller, we would only have 6 water level differences. Thus we control only 6 gates; excluding both the head gate and the most downstream gate. The net result is that this controller does not influence the inflow to the canal and it does not influence the turnout flows or spills. A diagram of this controller is shown in Figure 1. Instead, it adjusts the internal check gates to provide equal water level deviations for all pool, thus using the canal as a reservoir to mitigate inflow/outflow mismatches. It is recognized that this can only be done on a temporary basis. If the inflow and outflow are

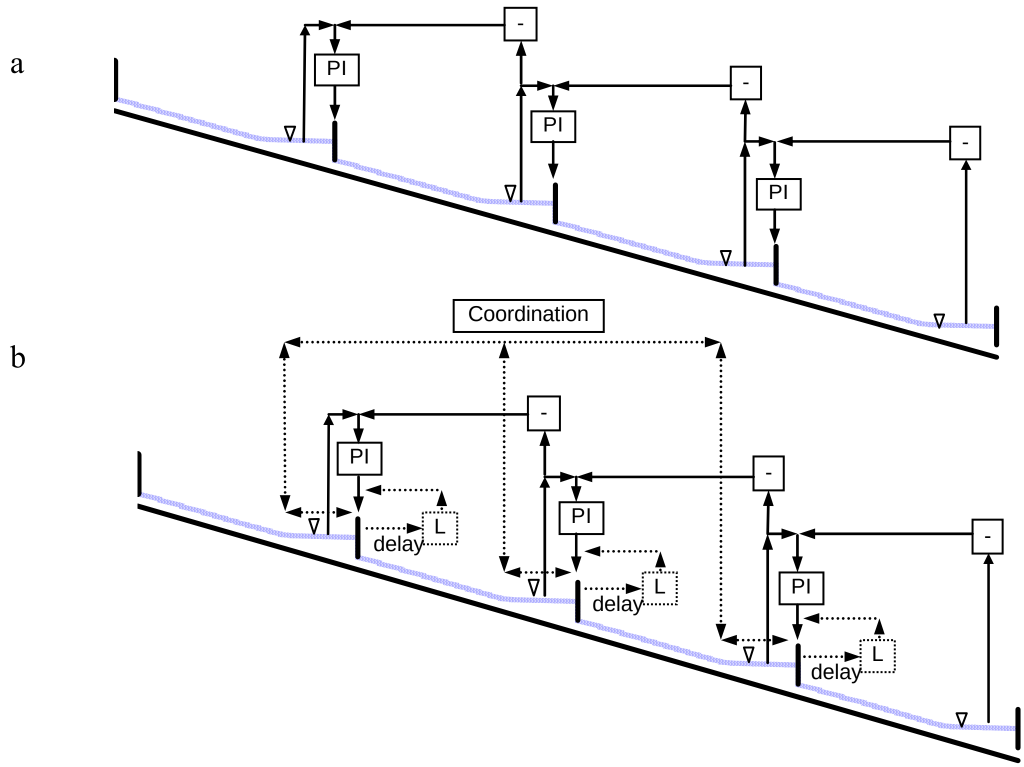

Figure 1. Schematic of Difference Controller, with varying levels of detail: a) simple PI, b) fully centralized.

roughly the same, the water levels in all pools will eventually stabilize at some level, likely close to the set points, but perhaps a bit off. If the inflow is greater than the outflow, all water levels will increase at a constant rate based the size of the flow mismatch and the backwater area upstream from each pool. If the inflow is too low, the water levels will drop at a more or less constant rate. The operator must eventually intercede to either increase or decrease the canal inflow or the demands, otherwise the canal will overtop or turnout flow will eventually decrease due to inadequate head.

MODEL FORMULATION

The canal response is described by a state-space model, where the Integrator-Delay (ID) model is used to describe canal pool response (Schuurmans et al 1999). The ID model is a simple linear model with a time delay that relates the water level to changes in the upstream and downstream gate flow.

$$
\begin{aligned}
y_j(t) &= y_j(0) - \frac{tQ_j}{A_j} && t \le \tau_k \\
y_j(t) &= y_j(0) + \frac{(t-\tau_j)Q_{j-1}}{A_j} - \frac{tQ_j}{A_j} && t > \tau_k
\end{aligned} \qquad (3)
$$

where Q_j is a change in flow rate at gate j (i.e., the gate just downstream from y_j), t is time, τ_j is delay time in pool j, and A_j is the backwater surface area of pool j. (Note that we can replace y_j with e_j since the set point would be subtracted from both sides of Eq. 3.) Applying the ID model to the difference in water level error gives:

$$D_j(t) = D_j(0) - \frac{tQ_j}{A_j} + \frac{tQ_{j+1}}{A_{j+1}} \qquad t \le \tau$$

$$D_j(t) = D_j(0) + \frac{(t-\tau_j)Q_{j-1}}{A_j} - \frac{tQ_j}{A_j} - \frac{(t-\tau_{j+1})Q_j}{A_{j+1}} + -\frac{tQ_{j+1}}{A_{j+1}} \qquad t > \tau \tag{4}$$

where the terms with (t-τ) are only included when positive. Eq. (4) is discretized over a time step Δt with the following procedure, in which the water level response to prior flow changes is distributed proportionately among prior flow changes at discrete intervals, k.

$$\tau = 0 \qquad \frac{(t-\tau_j)Q_{j-1}}{A_j} = \frac{\Delta t}{A_j} Q_{j-1}(k)$$

$$0 < \tau < \Delta t \qquad \frac{(t-\tau_j)Q_{j-1}}{A_j} = \frac{\Delta t}{A_k}[Q_{k-1}(k)\frac{(\Delta t - \tau)}{\Delta t} + Q_{k-1}(k-1)\frac{\tau}{\Delta t}] \tag{5}$$

$$\Delta t < \tau < 2\Delta t \qquad \frac{(t-\tau_j)Q_{j-1}}{A_j} = \frac{\Delta t}{A_k}[Q_{k-1}(k-1)\frac{(2\Delta t - \tau)}{\Delta t} + Q_{k-1}(k-2)\frac{(\tau - \Delta t)}{\Delta t}]$$

…

For example if the delay time is 0.3 times the time step, then 70% of the response is attributed to current time step and 30% is attributed to the previous time step. This method allows us to account for past control actions even when the delay is much longer than the time step of the controller.

Here we use the LQR method as described by Clemmens and Schuurmans (2004), which uses a state-feedback control with a control law of the form

$$\mathbf{Q}(k) = - \mathbf{K}\, \mathbf{x}(k) \tag{6}$$

where $\mathbf{Q}(k)$ is the vector of control actions at time k (one element of the vector for each control structure or gate), **K** is the controller gain matrix, and $\mathbf{x}(k)$ is the vector of states at time k. Here the control actions are changes in gate flow rates. A separate flow controller is used to adjust the gate position to provide the correct flow rate, which provides a master-slave control scenario.

Values of the gain matrix, **K**, are determined by minimizing the penalty function, J:

$$J = \sum_{k=0}^{\infty} \mathbf{D}(k)^T \mathbf{S} \mathbf{D}(k) + \mathbf{Q}(k)^T \mathbf{R} \mathbf{Q}(k) \tag{7}$$

where $\mathbf{D}(k)$ is the vector of water level errors at time k, **S** is the penalty function for water level errors (usually an identity matrix), and **R** is the penalty function for control actions (only main diagonal elements are non zero). Standard control engineering solutions are available for computing the gain matrix **K** that minimizes J, subject to the state transition equations (Schuurmans 1997). The result is a multiple-input multiple-output (MIMO) Proportional-Integral (PI) controller where all water level errors (and some prior changes in structure flow rates) influence the recommended changes to all structure flow rates, $\mathbf{Q}(k)$.

Eq. 4, with the discretization shown in Eq. 5, is put into state space form

$$\mathbf{x}(k+1) = \mathbf{A}\mathbf{x}(k) + \mathbf{B}\mathbf{Q}(k)$$
$$\mathbf{D}(k) = \mathbf{C}\mathbf{x}(k) \qquad (8)$$

In this formulation, the state vector is in incremental form such that it includes changes in water level difference, ΔD_j; prior control actions, Q_j(k); and prior water level differences, D_j.

$$x(k+1) = [\Delta D_1(k+1) \quad Q_1(k) \quad \ldots \quad Q_1(k-n_2) \quad \Delta D_2(k+1) \quad Q_2(k) \quad \ldots \quad Q_2(k-n_3) \quad \ldots\text{for all pools} \quad D_1(k) \quad D_2(k) \quad \ldots\text{for all pools}] \qquad (9)$$

where $\Delta D_j(k+1) = D_j(k+1) - D_j(k)$ and the number of prior control actions at gate j depends on the number of delays in the next pool downstream, pool j+1. Values of **K** multiplied by ΔD terms give the proportional action and by D terms give the integral action. Values of **K** multiplied by the prior control actions allows control based on lag-time predictions (e.g., as in the Smith Predictor of Deltour and Sanfilippo 1998).

If all water levels are of equal importance, **S** is represented as an identity matrix. Values of **R** are used to tune the controller, and reflect the relative importance of water level errors and gate flow changes in Eq. (7). Here the values for the diagonal elements are adjusted according to the square of the flow rate capacity of the pool downstream. The intent is that a 1 cfs change in a 100 cfs canal should have the same penalty as a 2 cfs change in a 200 cfs canal. (See Clemmens and Schuurmans 2004 for details).

The ID model is only appropriate for canal pools where a portion of the flow is under normal depth. For pools with backwater, one must also consider reflections waves. For a simple pool under backwater, the response of the canal is influenced by the backwater surface area, A_s and the resonant frequency. However, for pools with intermediate structures, such as culverts, there can also be a delay time due to the backwater that occurs upstream from these structures. The resonant frequency depends on the locations of the structures. For upstream control, the resonant frequency can be estimated from the speed of the celerity wave from the check gate to the next structure upstream, where the celerity, $c = (gD)^{1/2}$, where D is the hydraulic depth and g is the acceleration of gravity. This frequency often dominates.

Schuurmans (1997) recommends a linear filter of the form

$$D_{Fj}(k+1) = FD_{Fj}(k) + (1-F)D_j(k+1) \tag{10}$$

where D_{Fj} is the filtered value used for control and D_j is the measured value from Eq. 2. The filter time constant, T_f, is found from (Schuurmans 1996)

$$T_f = \sqrt{\frac{A_s R_p}{\omega_r}} \tag{11}$$

where R_p is the resonance peak height and ω_r is the resonant frequency ($1/P_u$, P_u = resonance period). The filter constant is then found from:

$$F_c = e^{-T_s/T_f} \tag{12}$$

where T_s is the sample time interval. Schuurmans (1996) recommends $T_s < 0.3\ T_f$. The time delay caused by the filter can be estimated from:

$$t_{delay} = \frac{F_c}{1-F_c} T_s \tag{13}$$

Control can be improved with the use of feedforward actions. However, since inflow changes are not matched to demand changes, an alternative form of routing was devised. Since the concept is to store excess water among all pools, each known inflow or outflow is routed proportionately to all pools, based on their relative storage, as reflected by the backwater surface area. Volume compensation (Bautista and Clemmens 2005) is used to route each inflow and each outflow, individually. Here, the routing time delay is determined from

$$\Delta t_{VC} = \frac{\Delta V}{\Delta Q} \tag{14}$$

where ΔV is the volume change resulting from flow change ΔQ. The volume as a function of flow rate is found from

$$V = aQ^b + c \tag{15}$$

where *a*, *b*, and *c* are empirical constants. Values for these coefficient change with flow resistance (Manning *n*) and downstream water level.

EXAMPLE

The Central Main Canal at the Central Arizona Irrigation and Drainage District (CAIDD) is used to test the difference controller. Details of the canal are provided in Table 1. ID model properties were determined through unsteady flow simulation with Sobek (Sobek 2000). Step tests were used to determine delay times and backwater surface areas (Schuurmans et al 1999). Pools 3, 4, 5, and 6 have culverts that would influence resonance waves. Pools 1, 2, and 3 do not. The frequency of celerity waves was computed based on the entire pool length and based on the distance from the check gate to the closest culvert upstream. Then, a series of step changes in flow at those frequencies (rounded to nearest min.) were input to each pool (separately). The magnitude of the flow change was such that if flow was governed by the ID model it would cause a change in depth of ± 2 in (5 cm) [2 in = $\frac{1}{2} P_u\ \Delta Q / A_s$]. For the pools with culverts, the resonance peak height (maximum change in water level) was higher and well above the expected 2 inch deviation.

Table 1. Central Main Canal physical properties (CAIDD).
(Side slopes 1.5:1.0 Horizontal to vertical, Manning n = 0.015).
(Lengths and drops do not include all siphons. Total length 94,508 ft. total drop 27.7 ft)

	Capacity	Length*	slope	Bottom width	Depth	Drop*
Pool	cfs	ft	ft/ft	ft	ft	ft
1	900	17,119	0.00013	12	12.2	2.2
2	900	7,144	0.00013	12	12.2	0.9
3	900	7,234	0.00040	12	9.9	-5.6
4	900	17,039	0.00018	12	11.5	-3.3
5	600	20,057	0.00010	12	10.8	-1.6
6	350	14,907	0.00016	8	8.4	-3.2
7	170	10,091	0.00010	4	6.9	-1.6
	Total	93,591				-18.4

*From start of reach downstream from one check gate to canal bottom at next check downstream. So includes mid pool siphons, but not siphon or drop just downstream from check gates.

Table 2. Canal pool properties at 60% of capacity.
(2 minute observation interval, 10 minute control interval.)

	Backwater Surface Area	Area	Delay time	Water level set point	Resonance period	Resonance Peak Height	Filter constant (x/16)	Filter delay	Delay Terms
Pool	ac	%	min	ft	min	s/ft^2		min	
1	16.1	23	4.5	11.0	44	0.0029	14	14	-
2	7.4	11	0.5	11.0	18	0.0054	14	14	2
3	4.5	7	5.5	8.7	11	0.0032	13	8.7	2
4	13.9	20	10.5	9.5	16	0.0025	14	14	3
5	13.6	20	18	7.4	15	0.0047	14	14	4
6	7.7	11	12.5	7.2	11	0.0047	14	14	3
7	5.4	8	6	6.25	34	0.0051	14	14	2

Table 3. Coefficients for volume-discharge relationships, Manning n = 0.014.

	a	b	c
Pool	$ft^{3(1-b)}s^b$	-	ft^3
1	1.30	1.961	4,526,026
2	0.20	1.958	2,101,785
3	2.52	1.775	1,018,566
4	4.03	1.847	3,363,685
5	8.59	1.888	3,490,771
6	12.29	1.829	1,412,312
7	649.96	1.286	347,471

Table 4. Schedule of demand and supply changes for multiple change test.

	Initial Flow	Site of change	Flow change	Time
site	(cfs)		cfs	
CAP	459	CAP	25.8	6:00
CM-1	424	Pool 1	-17.7	10:00
CM-1	388	Pool 4	-17.7	11:00
CM-1	353	Pool 7	-17.7	12:00
CM-1	282	Pool 5	+7.1	15:00
CM-1	177	CAP	20.1	16:00
CM-1	88			
CM-1	71			

The resonant frequency was computed for each pool based on the length of the entire pool and the length of the downstream portion of the pool. The filter constants used in the many SCADA systems are express as $F = x/16$. We chose to observe water levels every two minutes. Eq. (12) was used to determine filter constants, which are shown in Table 2. The state space model (Eqs. 4, 5, 8, 9) used the sum of the pool and filter delay times. The feedback control interval was selected as 10 minutes, resulting in the number of response delays for the state vector, **x**, shown in Table 2. Eq. (6) was used to determine the gain matrix **K** based on minimizing **J** in Eq. (7) subject to the constraints in Eq. (8). (For these tests, only the fully centralized controller was studied, with full lag time prediction and upstream and downstream decoupling.) Steady flow simulation results were used to determine the constants relating volume to discharge for Eq. (15) with a Manning *n* = 0.014 (Table 3).

The intent for operation of the Central Main Canal is to have all lateral head gates under flow control such that all errors in flow settings must be absorbed by the main canal. Canal inflow is determined by water orders to the Central Arizona Water Conservancy District (CAP) which are made the previous day and not under control by CAIDD. The first set of tests was made with a simulation model of the canal with the unsteady-flow simulation software, Sobek (Sobek 2000). Prior to running a test of the controller, a steady-state condition was set up with a flow of 459 cfs (13 m^3/s) at the headgate, dropping to 71 cfs (2 m^3/s) at the downstream end, with laterals taking the flow in between, as shown in Table 4. Then at 10:00 outflow from the canal was increased by 10 cfs without a corresponding change in canal inflow. Three tests were run with extra outflow in one pool at a time in pools 1, 4 and 7. The full centralized difference controller was run for all tests with all lateral flows held constant. This should cause all canal pools to drop, eventually by a constant rate since the turnout structures are under flow control. The results are shown in Figures 2, 3 and 4.

In Figure 2, note the initial drop in the level in pool 1. However, the controller eventually brought it back in line with the other water levels. In Figure 3, the water level deviates in pool 4, but recovers a little more quickly. In Figure 4, the water level in pool 7 drops significantly before recovering. These results are reasonable since pool 4 has two neighboring pools from which is can get recovery; while pool 7 is at the end of the canal where the flow change is a much larger fraction of capacity, the downstream gate is not adjusted, and there is a significant delay time in changes from gate 6.

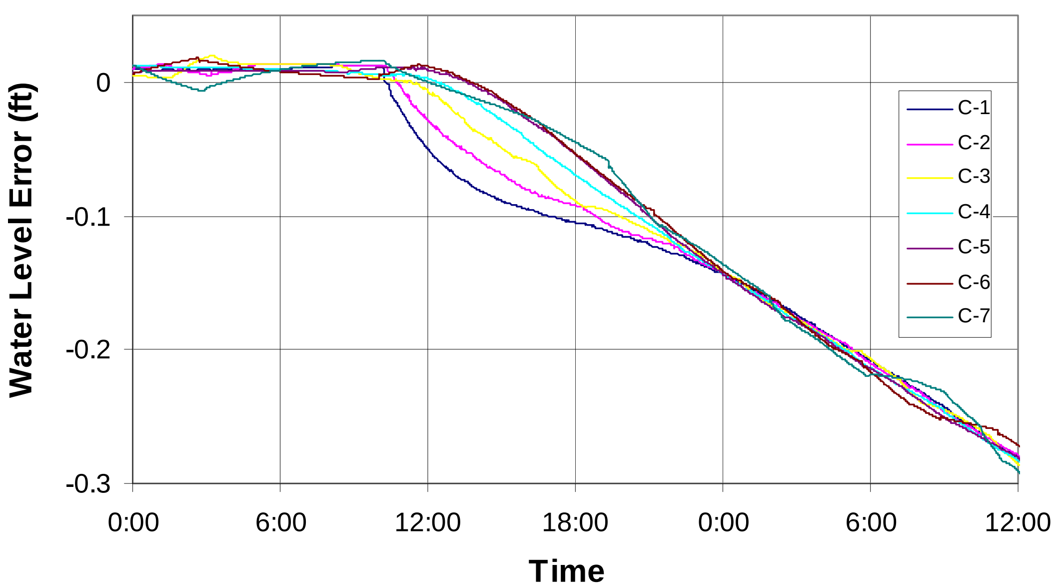

Figure 2. Water level errors for example problem with -10 cfs change in pool 1 (C-1).

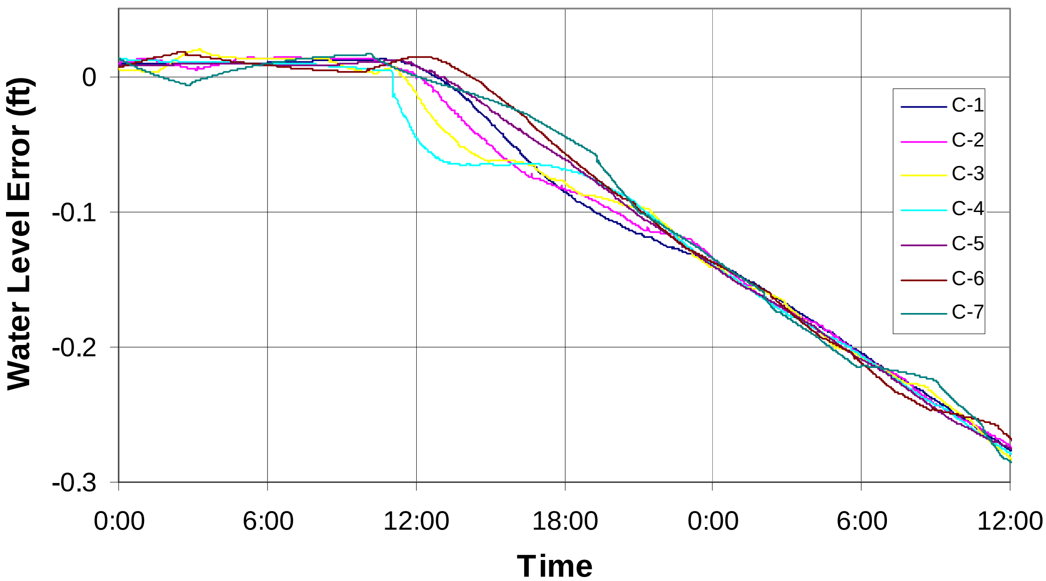

Figure 3. Water level errors for example problem with -10 cfs change in pool 4 (C-4).

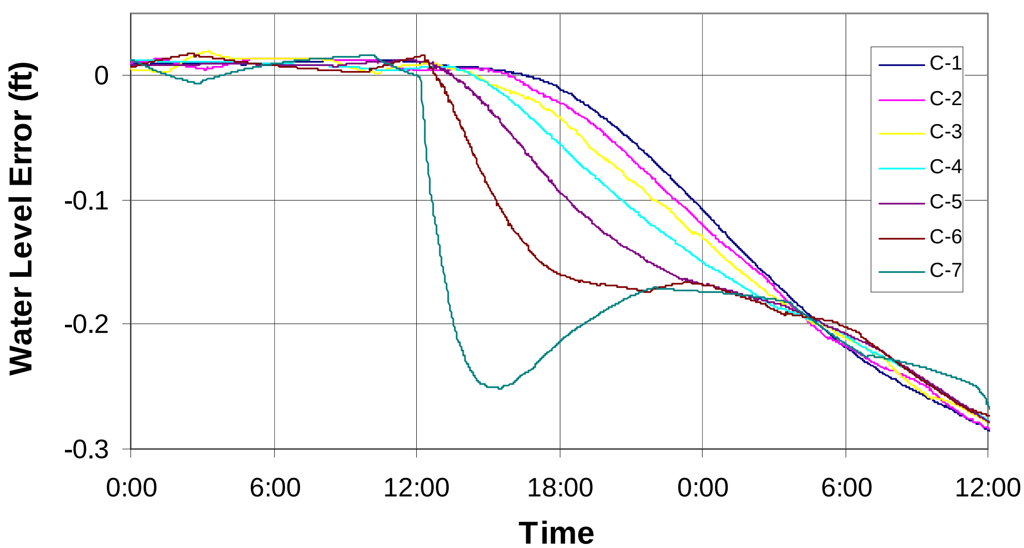

Figure 4. Water level errors for example problem with -10 cfs change in pool 7 (C-7).

The second test was meant to deal with the incompatibility between the water supply schedule and the schedule of water demands to laterals, and farmers downstream. Starting at the same initial condition, changes in the canal inflow and to the laterals for this test are shown in Table 4. Eq. 14 was used to determine the routing of flow changes to distribute each inflow or outflow to all pools. Table 5 shows the flow changes and how the flow change for each was distributed to the pools (negative pool flow is turnout increase). Note the bold time represents the scheduled change. These schedules were overlapped.

Table 5. Schedule of check gate changes for multiple change test.

	Q	Time	Q	Time	Q	Time	Q	Time	Q	Time	Q	Time
	(cfs)		(cfs)		(cfs)		(cfs)		(cfs)		(cfs)	
CAP	25.8	**6:00**									20.1	**16:00**
CM-1	19.7	6:15	4.1	9:59	4.1	10:31	-13.5	**10:00**	-1.7	13:50	15.4	16:16
CM-2	16.9	6:17	6.1	10:01	6.1	10:33	-11.6	10:02	-2.4	13:52	13.2	16:18
CM-3	15.2	6:25	7.2	10:09	7.2	10:41	-10.4	10:09	-2.9	14:00	11.9	16:26
Cm-4	10.0	6:43	10.8	10:27	-6.8	**11:00**	-6.8	10:28	-4.3	14:18	7.8	16:44
CM-5	4.9	7:24	14.3	11:10	-3.3	11:42	-3.3	11:10	1.3	**15:00**	3.8	17:26
CM-6	2.0	7:51	16.3	11:38	-1.4	12:11	-1.4	11:38	0.6	15:28	1.6	17:54
CM-7		8:41		**12:00**								18:47

The results are shown in Figure 5. Note that from 6:00 to 11:00 supply exceeded demand such that all the water levels rose. After 12:00, demand was more than supply such that the water levels dropped. The last supply flow change at 16:00 matched the inflow to the outflow (values in Table 4 don't add due to round-off error). The final error in water levels results from a volume mismatch between inflow and outflow timing. Of importance is that all the water levels eventually tracked each other. The volume associated with the difference in levels at the end of this test represents roughly 25 cfs for 1 hour. The small size of Pool 7 causes more deviation in the water level there. This method provides a convenient method for overcoming the mismatch in timing between supply and demand, while at the same time providing reasonable water level control.

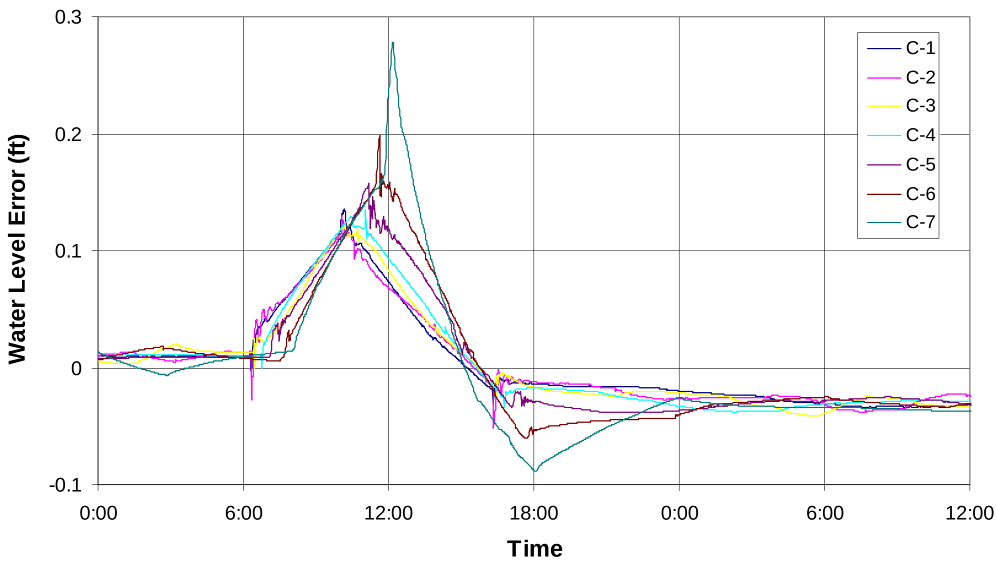

Figure 5. Water level errors for numerous, uncoordinated flow changes.

DISCUSSION

Control of water level differences among pools appears to be an effective way to deal with the complexities of main canal control. It allows an easy method to account for short-term demand/supply mismatches. It is a good mix of upstream control to maintain water levels and downstream control to avoid spills. Obviously this controller will not adjust supply and demand and will eventually lead to control failure. So it is up to the operator to work toward matching supply and demand through water supply ordering and interaction with water users. The intent of the method is to overcome the problem of distributing flow errors through the canal system, and instead concentrates them in the main canal.

It is possible to design these controllers as simple PI devices, so that they can be implemented with peer to peer communications between PLCs, but this has not been tested. It is also possible to put more weight on water level errors in some pools than others, thus avoiding deviations where pools have tighter constraints, but this method has also not been tested.

REFERENCES

Bautista, E. and Clemmens, A. J. 2005. Volume compensation method for routing irrigation canal demand changes. Journal of Irrigation and Drainage Engineering 131(6):494-503.

Clemmens, A. J. and J. Schuurmans. 2004. Simple optimal downstream feedback canal controllers: Theory. Journal of Irrigation and Drainage Engineering. 130(1), 26-34.

Deltour, J-L and F. Sanfillipo. 1998. Introduction of Smith predictor into dynamic regulation. Journal of Irrigation and Drainage Engineering. 124(1), 47-52.

MathWorks 2003 Matlab User Guide. The MathWorks, Inc., Natick, Massachusetts.

Schuurmans, J. 1997. Control of water levels in open channels. Ph.D.-dissertation Delft University of Technology, The Netherlands.

Schuurmans, J., A. J. Clemmens, S. Dijkstra, R. H. Ahmed, O. H. Bosgra, and R. Brouwer. 1999. Modeling of irrigation and drainage canals for controller design. Journal of Irrigation and Drainage Engineering 125(6), 338-344.

Sobek 2000 Manual and Technical Reference. WL|Delft Hydraulics, Delft, The Netherlands.