Control of Very-Large Scale Irrigation Networks: A CPS Approach in a Developing-World Setting

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Abstract: Many developing countries face severe water shortages, a large magnitude of which can be attributed to the inefficient operation of their water networks. In this paper, we study this issue from the point of view of implementing a cyber physical systems (CPS) infrastructure, in which networked embedded controllers can be used for local control of a very large-scale canal network. We take inspiration from existing literature and report on the suitability (or otherwise) of existing methods in a third world setting. We describe partial differential equation (PDE) models for open channel flows, system identification techniques to simplify the model, local control system design and strategies to implement a decentralized control network. The theoretical study is supported by simulations on large networks.

Keywords: Open channel flow; cyber physical systems; system identification; irrigation channels; water networks; smart infrastructures; networked control systems;

1. INTRODUCTION

One of the major causes related to the scarcity and distribution of water in many third-world countries is the inefficiency of their irrigation networks (For a global survey, see Seckler et al. (1999)). In countries like Pakistan, India and Egypt, the irrigation networks are extremely vast and complex, the water sources are few and drying, while the infrastructure is in a constant decay, dating back by centuries and decades (Briscoe and Qamar (2009)). In this paper, we propose that the problem of efficient distribution of available water can be effectively solved by engineering appropriate Cyber-Physical Systems (CPS) and feedback control technologies. Cyber-physical systems are a new generation of engineering paradigms, capable of deploying, monitoring and controlling large-scale infrastructures (both natural and man-made) by a tight integration of computational and physical processes (Sha et al. (2009); Wolf (2009)).

As a motivation for this study, we briefly describe the state of the affairs of Pakistan's irrigation network, which is the largest contiguous canal network in the world. Water supply to the Pakistan's agriculture base in the Indus river basin is mostly provided by this complex network (See Fig. 1). It constitutes 4 main reservoirs, 23 Barrages, 45 main canals and 14 river-to-river link canals with thousands of secondary watercourses running for over 90,000km and irrigating a total area of approximately 36 million acres (Briscoe and Qamar (2009)).

Built in the nineteenth century and having undergone a major restructuring in the 1960s, the system is being regulated by primitive methods that result in intractability of usage, wastage and disputes (Afzal (1996); Bandaragoda and ur Rehman (1995); Latif and Saleem Pomee (2003); Nakashima (1999)). With gloomy predictions of glacier melt in the Himalayas, international and inter-provincial disputes over water rights and the prohibitively large costs involved in building reservoirs (Briscoe and Qamar (2009)), the system is in a dire need of new and efficient management strategies. This paper attempts to address some critical aspects of this need by proposing an architecture towards the modernization of the current infrastructure with state of the art methods in systems engineering and sensing technologies.

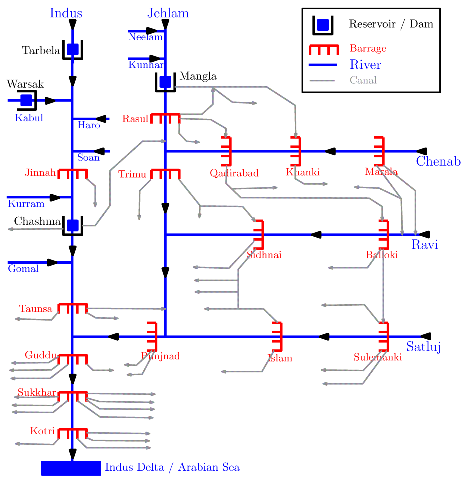

Fig. 1. Major infrastructures in the Indus River Basin Irrigation Network.

There are several levels at which control and optimization can be realized in the operation of water resources. In this paper, we restrict ourselves to the case where optimal water levels in a canal network have already been calculated by a high-level scheduler and it is required to maintain local control at the suggested set-points. In practice, the high-level scheduler is run by a regulation authority (e.g. federal & provincial irrigation departments and Punjab irrigation & drainage authority (PIDA) bodies in Pakistan), while the local distribution of water is done by either manual control (e.g. by the practice of *warabandi* in Pakistan and northern India (Bandaragoda and ur Rehman (1995))) or SCADA-like systems in developed countries. The high-level schedules are maintained by considering economic factors, public policy and international/regional water allocation accords.

There exists a vast body of literature on modeling the flow of water in open channels (Chaudhry (2008); Szymkiewicz (2010); Schuurmans et al. (1999); Malaterre and Baume (1998)) and controlling water levels in an irrigation canal network using methods of feedback control (Litrico and Fromion (2009); Cantoni et al. (2007); Rabbani et al. (2009)). Various control theoretic problems have been addressed and solved by researchers related to modeling and controller design (Ooi and Weyer (2008)), decentralization (Gómez et al. (2002); Negenborn et al. (2009); Cantoni et al. (2007)), security (Amin et al. (2010)) and practical implementation issues (Litrico et al. (2005); Ooi and Weyer (2008); Stefanovski and Dimirovski (2001)).

Based on fundamental results in these recent studies, we take the first steps towards the feasibility and performance of a CPS for the control of irrigation networks, suitable for deployment in a developing world setting, such as the Indus river basin in Pakistan. We give below the various factors in choosing or developing particular techniques.

(1) Applicability to a very large-scale, geographically extent network.
(2) Design and testing in a low-cost (preferably virtual) laboratory setting
(3) Decentralization to allow flexibility in governance and scalability.
(4) Fitness for unreliable energy and telecommunication infrastructures.

We first develop and simulate dynamical models for open channel hydraulics in irrigation networks. We then study boundary control of these PDE models via appropriate sensing and feedback. This theoretical framework is then analyzed in the light of implementation issues, i.e the translation into appropriate real-time control technologies via embedded digital controllers that regulate channel flows in various sections of the irrigation network by actuating canal gates. A distributed control scheme is proposed by linking these local controllers via a low bandwidth communication network. Aspects of network topology, information feedback, robustness, system identification and energy consumption are at the heart of this investigation.

2. OPEN CHANNEL FLOW MODELS

Open channel flow, or *free surface flow* is an example of fluid flow in which there is a free surface subject to

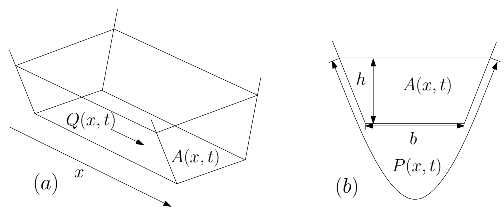

Fig. 2. (a) open channel flow along a longitudinal axis indexed by abscissa x. (b) cross-section of an open trapezoidal channel

atmospheric pressure. Most canals and rivers fall in this category. Free surface flows can be classified into *steady flows* and *unsteady flows*. An open channel flow is steady when the flow velocity does not change with respect to time at a given location. Most transient conditions such as opening of canal gates or off-takes at farms give rise to unsteady flows in open channels before settling to a steady one.

2.1 Saint Venant Equations

Describing unsteady flow of water in open channels has remained of a great interest among hydraulic engineers since ancient times. In the nineteenth century, a set of hyperbolic nonlinear partial differential equations were developed by the French engineer *Saint Venant* which accurately describe the one-dimensional dynamics of unsteady open channel flows (Chaudhry (2008); Szymkiewicz (2010)). These equations are derived from a mass and momentum balance, and are given by

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0,$$

$$\frac{\partial Q}{\partial t} + (\frac{gA}{B} - \frac{Q^2}{A^2})\frac{\partial A}{\partial x} + \frac{2Q}{A}\frac{\partial Q}{\partial x} + gA(S_f - S_0) = 0. \quad (1)$$

Here x is the distance coordinate; t is time; A is the cross-sectional area of the channel; B is the width of water surface; Q is the flow (discharge); g is the gravitational constant (taken as $9.81m/s^2$); S_f and S_0 are frictional and bed slope of the channel respectively. The frictional slope is modeled with the classical Manning formula i.e. $S_f = \frac{Q^2 n^2}{A^2 R^{4/3}}$. where, n is the Manning coefficient, R is the hydraulic radius, defined by $R = A/P$ with P as the wetted perimeter. For a section of a channel, these variables have been described in Fig. 2. These models have been used extensively by civil and hydraulic engineers and describe the flows fairly accurately if supplied with the geometry and operating conditions of the channels (Chaudhry (2008); Litrico and Fromion (2009)).

2.2 Numerical Solution of St. Venant PDEs

Since the St. Venant model is highly nonlinear, it is both difficult and impractical to find analytical solutions. Therefore, most practitioners prefer to solve the PDEs numerically for a solution. Semi-analytical methods such as the *method of characteristics* have also been popular, but *finite-difference method* have received most acceptance due to their ease of implementation, and availability of cheap computational resources (Chaudhry (2008)). The

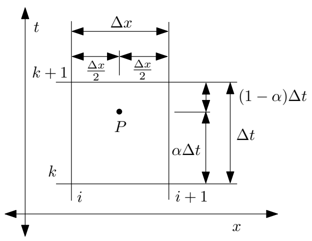

Fig. 3. Grid point for the Preissmann scheme

Preissmann's Scheme is commonly accepted to be the most robust finite-difference scheme for solving the system of St. Venant equations (Szymkiewicz (2010)).

Preissmann's Scheme. As is the case in all finite-difference methods, the time, t, and spatial variable, x, are discretized in the Preismann's scheme onto a grid on which the St. Venant equations are approximated using approximations of partial derivatives. Consider the point P in (x, t) plane of Fig. 3. P is placed at the middle of the interval Δx_i while P can move along t axis controlled by a weighting parameter α. The value of a function at P, denoted by $f_P(x, t)$, and its partial derivatives ($\frac{\partial f}{\partial t}$ and $\frac{\partial f}{\partial x}$) are given by the approximations presented in (Lyn and Goodwin (1987)).

Rectangular Channel. For ease of explanation, we restrict ourselves to rectangular channels in this paper. We emphasize though that this is not a restriction and it is fairly easy to generalize the scheme to canals of more complex geometry Chaudhry (2008); Szymkiewicz (2010). We assume a rectangular channel of bottom width B. At any time t, and distance, x, the height of water is given by $h(x, t)$ and the cross sectional area of water channel is $A(x, t) = Bh(x, t)$. Also assume the bottom slope of the channel S_0 to be approximately zero. Eq. 1 can be rewritten as below, where we have also substituted the Manning formula.

$$\frac{\partial h}{\partial t} + \frac{1}{B}\frac{\partial Q}{\partial x} = 0,$$

$$\frac{\partial Q}{\partial t} + gBh\frac{\partial h}{\partial x} + \frac{\partial(\frac{Q^2}{Bh})}{\partial x} + \frac{gn^2Q^2}{BhR^{\frac{4}{3}}} = 0. \qquad (2)$$

The system is solved for $0 \leq x \leq L$ and $t \geq 0$. Initial and final boundary conditions are given by

(1) $h(x, t = 0)$ and $Q(x, t = 0)$ for $0 \leq x \leq L$.
(2) $Q(x = 0, t) = Q_0(t)$ and $Q(x = L, t) = Q_L(t)$ for $t \geq 0$.

Applying the Preissmann's scheme to Eq. 2, we have Eqs. 3 and 4.

$$\begin{aligned}&\frac{1}{2}\Big(\frac{h_i^{k+1} - h_i^k}{\Delta t} + \frac{h_{i+1}^{k+1} - h_{i+1}^k}{\Delta t}\Big) + \\ &\frac{1}{B}\Big((1-\alpha)\Big(\frac{Q_{i+1}^k - Q_i^k}{\Delta x}\Big) + \alpha\Big(\frac{Q_{i+1}^{k+1} - Q_i^{k+1}}{\Delta x}\Big)\Big) = 0.\end{aligned} \qquad (3)$$

$$\begin{aligned}&\frac{1}{2}\Big(\frac{Q_i^{k+1} - Q_i^k}{\Delta t} + \frac{Q_{i+1}^{k+1} - Q_{i+1}^k}{\Delta t}\Big) + gBh_P\Big((1-\alpha)\Big(\frac{h_{i+1}^k - h_i^k}{\Delta x}\Big) \\ &+\alpha\Big(\frac{h_{i+1}^{k+1} - h_i^{k+1}}{\Delta x}\Big)\Big) + (1-\alpha)\Big(\frac{(\frac{Q^2}{Bh})_{i+1}^k - (\frac{Q^2}{Bh})_i^k}{\Delta x}\Big) + \\ &\alpha\Big(\frac{(\frac{Q^2}{Bh})_{i+1}^{k+1} - (\frac{Q^2}{Bh})_i^{k+1}}{\Delta x}\Big) + \Big(\frac{gn^2Q^2}{BhR^{\frac{4}{3}}}\Big)_P = 0.\end{aligned} \qquad (4)$$

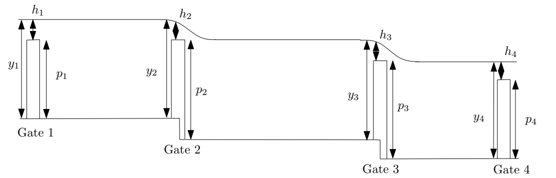

Fig. 4. Side view of pools and gates in an open channel.

Here, subscript i represents the spatial grid point and superscript k represents the grid point in time. Δx and Δt are grid intervals along x and t axes respectively. In Eqs. 3 and 4, the water levels at node $k + 1$ i.e. h_i^{k+1} and h_{i+1}^{k+1} and the flow rates at node $k + 1$ i.e. Q_i^{k+1} and Q_{i+1}^{k+1} are unknown. This pair of equations can be written for all spatial grids i.e. $i = 1, 2, 3, ..., M - 1$. We therefore obtain $2(M - 1)$ algebraic equations with $2M$ unknowns. Two boundary conditions of flow rates complete the system as, $Q_1^{k+1} = Q_0(t_{k+1})$. and $Q_M^{k+1} = Q_L(t_{k+1})$. Finally, a system of $2M$-dimensional non-linear algebraic equations in $2M$ variables is obtained. In a compact matrix form we have, $\mathbf{F(X)} = 0$, where, $\mathbf{X^T} = [\, h_1 \; Q_1 \; h_2 \; Q_2 \; . \; . \; . \; h_M \; Q_M \,]$ and this is a vector of $2M$, unknowns and, $\mathbf{F^T} = [\, F_1 \; F_2 \; F_3 \; . \; . \; . \; F_M \,]$ is a vector of $2M$, equations. Equations of this system are of the following form

$$F_1(Q_1^{k+1}) = Q_1^{k+1} - Q_0(t_{k+1}), \qquad (5)$$

$$F_{2i-1}(h_i^{k+1}, Q_i^{k+1}, h_{i+1}^{k+1}, Q_{i+1}^{k+1}) = 0, \qquad (6)$$

$$F_{2i}(h_i^{k+1}, Q_i^{k+1}, h_{i+1}^{k+1}, Q_{i+1}^{k+1}) = 0, \qquad (7)$$

$$F_{2M}(Q_{2M}^{k+1}) = Q_M^{k+1} - Q_L(t_{k+1}). \qquad (8)$$

where, $i = 1, 2, ...M - 1$. First and last equations represent boundary conditions and the rest capture Eq. 3 and 4. This system of equations must be solved using some iterative method. Newton-Raphson method can be used here. The iterative process has form $\mathbf{J}^{(j)}.\Delta\mathbf{X}^{(j+1)} = -\mathbf{F}^{(j)}$. where, j, is the index of iteration, $\Delta\mathbf{X}^{(j+1)} = \mathbf{X}^{(j+1)} - \mathbf{X}^{(j)}$ is a correction vector, $\mathbf{J}^{(j)}$, is a Jacobian matrix.

2.3 Channel Geometry and Physical Parameters

Let us consider a (rectangular) channel with many interconnected pools with bottom slope nearly zero. Each pool has an upstream and downstream gate associated with it to control the amount of water flowing into it from its upstream pool. A three pool system is depicted in Fig. 4. In Fig. 4, y_m, represents height of water in pool m at gate position, h_m, depicts water head over the gate, $p_m = y_m - h_m$, shows height of the m-th gate. For a sharp-edged rectangular gate, flow, Q_m, over any gate can be described by an empirical formula: $Q_m = 0.6\sqrt{g}b_m h_m^{3/2}$, where, g is gravitational constant in SI units and b_m is the gate width. This relation can be used in describing inflows and outflows depending upon head-overs. We represent $0.6\sqrt{g}b_m h_m$ by a constant c. So each pool has constants c_{in} and c_{out} corresponding to its upstream and downstream gates. Boundary conditions are derived from relations:

$$Q_{in} = c_{in}h^{3/2}, \qquad Q_{out} = c_{out}h^{3/2}. \qquad (9)$$

where, $h = y - p$ is used in calculating head over the gate. Over flows are set to zero if $h_m < 0$.

3. GATE CONTROLLER DESIGN

While the nonlinear PDE model of St. Venant is accurate for most practical applications, it is difficult to use it for deriving a boundary control law. There are two approaches in current literature to overcoming this difficulty. In one approach, the nonlinear model can be linearized at a steady flow solution to get a linear PDE model. This (infinite dimensional) linear model can accurately describe small fluctuations near the steady flow (Litrico and Fromion (2009)). Unfortunately, in a water scarce country like Pakistan, the flows are seldom steady for a long period and the system undergoes wide and frequent fluctuations. Moreover, controller design is still quite challenging and complex. The other approach is to use a finite-dimensional lumped-parameter ODE model approximation as discussed in Ooi and Weyer (2008); Cantoni et al. (2007). As a result, the controller design becomes quite simple and easy to implement. In this paper, we follow this second approach very closely.

In order to develop a local gate controller, we follow a sequence. We first run our full model on some test input data to generate representative trajectories that reveal system dynamics. We then run standard system identification techniques to extract out parameters for a simpler ODE model. Finally, we use standard control design techniques to derive the controller.

3.1 ODE Model Extraction

For controller design, we consider a simple volume balance model, $\frac{dV}{dt} = Q_{in}(t) - Q_{out}(t)$. ,where, V, is the volume of a particular pool and, Q_{in}, Q_{out} describe inflow and outflow. As the cross sectional area of the pool remains constant in a rectangular channel, we can map volume to water level. Using Eq. 9 we have a more elaborated equation:

$$\dot{y}_{i+1}(t) = c_{i,in} h_i^{3/2}(t - \tau_i) - c_{i+1,out} h_{i+1}^{3/2}(t), \qquad (10)$$

where index i represents the i_{th} gate of the pool. τ_i represents the time delay for water in reaching the downstream gate starting from the upstream gate. For simulation and digital control design purposes, we may consider a sampling time T to get a discrete time model

$$y_{i+1}((k+1)T) = y_{i+1}(kT) + T c_{i,in} h_i^{3/2}(kT - \tau_i) \\ - T c_{i+1,out} h_{i+1}^{3/2}(kT). \qquad (11)$$

Hence, we need to estimate the delay τ_d and constants c_{in} and c_{out} through parameter estimation technique. Later, we will also use a fourth parameter, the wave period for controller design.

Parameter Estimation by Step Response. We run an example of three concatenated pools to elaborate the system identification procedure. Table 1 shows the physical parameters of the three pools.

In this table, b_u and b_d describe upstream and downstream gate widths respectively. To determine our first set of parameters (delay and wave period), we compute the step response of each pool by raising its input gate instantaneously by a large jump. All simulations are done using $\Delta x = 20m$ and $\Delta t = 60sec$, both satisfying the Lax stability condition for numerical simulation. Step tests are performed in each pool by stepping the head, h_1, h_2 and h_3 over the upstream gates p_1, p_2 and p_3 respectively from 0 to $0.2m$, at $t = 10min$ and all the tests were run separately on each pool. Fig. 5 shows step test response of the pool 1. Both time delay and wave period can be computed by a direct visual inspection of the step responses. Table 2 shows both of these measured parameters for all three pools.

Table 1. Physical data of Pools 1-3

Pool	1	2	3
Length, $L(m)$	420	700	300
Bottom Width, $b_0(m)$	5	5	5
Bottom Slope, S_0	0	0	0
Gravity, $g(m/s^2)$	9.81	9.81	9.81
Manning coefficient, n	0.02	0.02	0.02
$cin = 0.6\sqrt{g}b_u$	3.6	4.8	4.8
$cout = 0.6\sqrt{g}b_d$	4.8	4.8	3.6

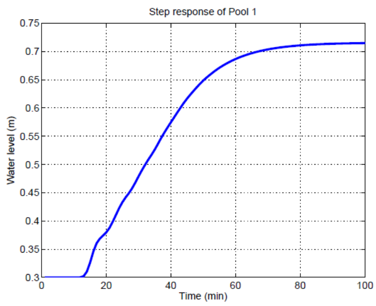

Fig. 5. Simulated step response of Pool 1.

Table 2. Time delays and wave periods of Pools 1 − 3

Pool	1	2	3
Time delay, τ_d, (min)	2	3	1
Wave period, (min)	8	13	7

3.2 System Identification by Error Minimization

Parameters $c_{i,in}$ and $c_{i+1,out}$ can be estimated by other standard *System Identification* techniques. In this case, simulations are performed using Saint Venant equations with specially designed input i.e simultaneously varying height of upstream gate by increasing or decreasing head over that gate and also varying downstream gate height. Fig. 6 represents variation of water head and height of upstream and downstream gate respectively and finally simulated water level in pool 1. A predictor associated with the model of Eq. 11 for pool 1 is given by:

$$\hat{y}_2((k+1)T; C) = \hat{y}_2(kT; C) + T c_{1,in} h_1^{3/2}(kT - \tau_i) \\ - T c_{2,out} h_2^{3/2}(kT).$$

where $C = \{c_{1,in}, c_{2,out}\}$. The parameter set C is estimated by minimizing the average squared prediction error by an optimization over C. Estimated water level along with original water level at distance, $340m$ on Pool 1 is plotted in Fig. 7. In a similar fashion, parameters are estimated for Pool 2 and 3. Table 3, shows estimated parameter values of all the pools.

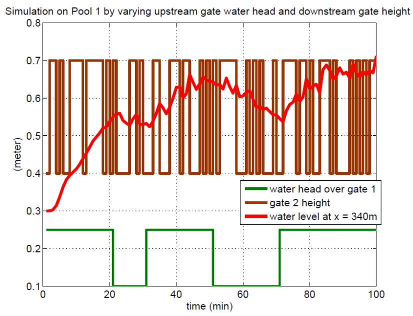

Fig. 6. Simulated water level in Pool 1 for a given scenerio.

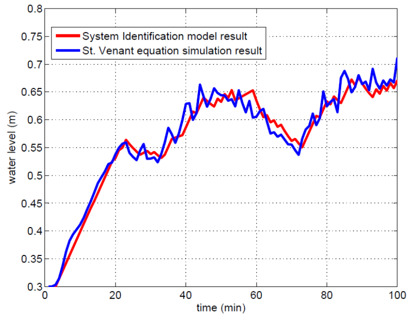

Fig. 7. Water level simulated using St. Venant equation and system identification model in Pool 1 at distance $340m$

Table 3. Estimated parameters of Pools $1-3$

Pool	1	2	3
$c_{i,in}$	0.1090	0.1010	0.2340
$c_{i+1,out}$	0.1460	0.0910	0.2010

3.3 Controller Design

Single pool control From the ODE models above, a transfer function model can be derived as

$$Y_{i+1}(s) = \frac{c_{i,in}e^{-\tau_i s}}{s}U_i(s) = P_i s U_i(s). \tag{12}$$

Making controller for such models is much easier then finding one for a PDE model. Following, the scheme in Ooi and Weyer (2008), we design a compensator for this model in which a PI controller is augmented with a low pass filter. Such a controller $C_i(s)$ is given by

$$C_i(s) = (\frac{K_i}{T_{i,c}s})(\frac{1+T_{i,c}(s)}{1+T_{i,f}(s)}). \tag{13}$$

Parameters K_i, $T_{i,c}(s)$ and $T_{i,f}(s)$ are tuned based on the procedure given in Ooi and Weyer (2008). For simulation, the designed controller has been digitized using a sampling time equal to the time step in St. Venant simulations i.e. $60s$ and a zero order hold (ZOH). Fig. 8 shows the controlled behavior of water in Pool 2. Set point is $0.55m$ and the controller aims to stabilize the pool height at this level.

3.4 Control of Multiple Pool Channels

In most canal channels, there are number of pools. Since the operation of each gate affects the water levels in all pools downstream, the local controller can be made more

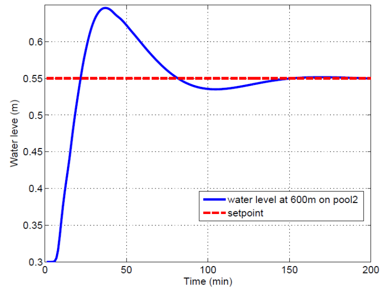

Fig. 8. Water level controlled on set point $0.55m$ in Pool 2

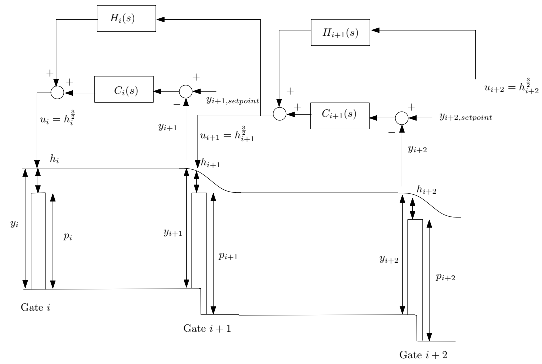

Fig. 9. Controller configuration in multiple pool

robust against disturbances by supplying a feed-forward path. To implement intelligent control of water level in all the pools and in addition to the local controller designed in the last section, there should be another controller which passes scarcity of water in a pool to its upstream pools. This makes sense because information about the upstream control $u_{i+1}(t)$ can be made available (using a fast communication method) since it acts as a disturbance on i^{th} pool. Thus, this control information is added to the local control information of i^{th} pool and it can help in compensation of affects caused by $(i+1)^{th}$ pool. The combined control strategy is depicted in Fig. 9. The feed forward path can be given as

$$H_i(s) = G_f F_i(s)\frac{c_{i+1,out}}{c_{i,in}}.$$

G_f is taken as 0.75. $F_i(s)$ is a low pass filter having cutoff frequency equal to half of wave frequency can be measured from wave period given in Table 2. $c_{i+1,out}$ and $c_{i,in}$ can be used from Table 3. Simulations were run on three pools with the controllers inserted as described above. Keeping in mind Fig. 4, gates $1, 2$ and 3 were controlled by PIL controllers in the way Fig. 9 shows. Gate 4 was kept manual. At time, 0 min, all the pools were at steady state at, $0.4m$, water level. Gate 4 was opened as shown in Fig. 11, at, $t = 5$ min, from, $0.5m$ to $0.1m$, It created deficiency of water level first in, Pool 3, and then it traversed toward most upstream gate i.e. Gate 1. Fig. 10, describes how water level is controlled in all the pools and it shows pool 1 has taken longest time to get settled back on steady state as it was most responsible in fulfilling deficiency of downstream pools first.

Simulations were also run by adding more feedback paths from other downstream pools. But performance seemed to have gotten worse with the addition of more feed-forward loops. This is a bit counter intuitive but finding

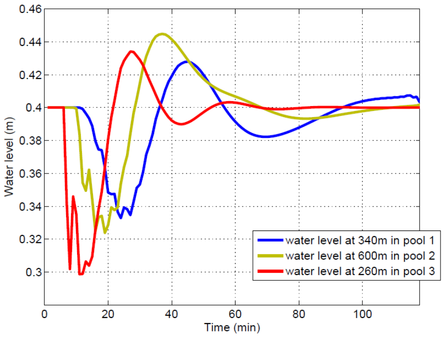

Fig. 10. Water level controlled in Pool 1,2 and 3.

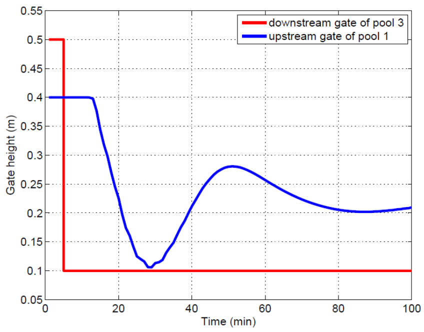

Fig. 11. Gate 1 behaviour during control action.

the optimal topology for interaction between controllers is still an area of future investigation by us.

4. LARGE-SCALE SIMULATIONS

In order to gain confidence about our scheme for a very large scale system, we simulated the controllers on many networks of varying sizes. Inspired by the local nomenclature and terminology, we setup our simulations which resembled a typical canal network in central Punjab (Pakistan).

In our simulations, we have canals, secondary canals and *khala* (colloquial term for small watercourses). One such example is depicted in Fig. 12(a). It shows a main canal emerging from a river, feeding into secondary canals. The secondaries extend to khala's. A local controller is put on each gate having feed forward paths established by radio communication links. The radio links can easily be established using Zigbee or Dash7 sensor network technologies. Physical parameters of pools in Fig. 12(a) is shown in Table 4, which closely resemble similar canal parameters in central Punjab region. A typical canal network in the Indus river basin has been reproduced in Fig. 12(b).

For simulation purposes, we use the simpler ODE model instead of full-blown St. Venant equations. This confidence is gained by extensive simulations that seem to suggest that the system identification procedure runs fairly effectively in typical cases. All the pools were assumed to be empty initially. Control actions of whole network of controllers brought water levels to set points, which was set, 1.5m, in each pool. All the pools are controlled by embedded digital controllers at the gates using water level meters as sensors to compute control actions. The last gate

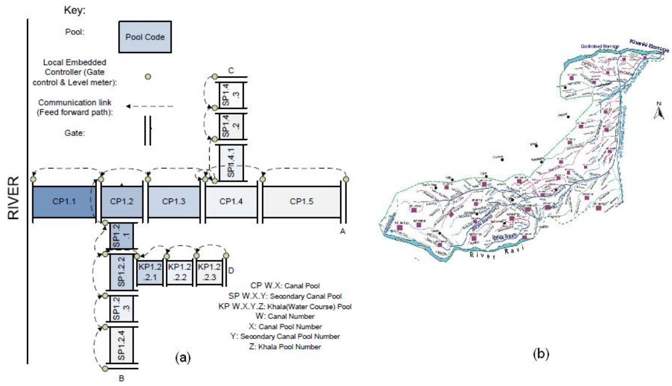

Fig. 12. (a) An example of irrigation network. (b) LCC canal network in Punjab, Pakistan. (Courtesy. Irrigation Department, Govt of Punjab)

of each stem (labeled A, B, C and D in Fig. 12(a)) are manually controlled. The operation of these manual gates creates disturbance in water levels, which traverses back to root pool of whole irrigation network which in this case is CP1.1.

Table 4. Physical parameters of irrigation network

Pool	Nature	Length	Width
CP1.1	Canal pool	1000m	6m
CP1.2	Canal pool	500m	6m
CP1.3	Canal pool	900m	6m
CP1.4	Canal pool	950m	6m
CP1.5	Canal pool	1300m	6m
CP1.1	Canal pool	1000m	6m
SP1.2.1	Secondary pool	420m	5m
SP1.2.2	Secondary pool	700m	5m
SP1.2.3	Secondary pool	300m	5m
SP1.2.4	Secondary pool	700m	5m
SP1.4.1	Secondary pool	700m	5m
SP1.4.2	Secondary pool	420m	5m
SP1.4.3	Secondary pool	300m	5m
KP1.2.2.1	Khala pool	300m	4m
KP1.2.2.2	Khala pool	300m	4m
KP1.2.2.3	Khala pool	300m	4m

Results of water level in each pool of all levels of canals in Fig. 12(a) are shown in Fig. 13. Fig. 13, shows each pool controller bring water level to the set point which is set at 1.5m in all the pools. Note that negative values of Water level is an artifact of using the ODE model. But it may also be interpreted as an indication (see Eq. 10) that water entering the pool is somehow leaving through its downstream gates to first fulfill the need of downstream pools. Keeping Fig 12(a), in mind, we can see in Fig. 13, that this effect is more obvious in the case when pools have a large subnetwork down the stream.

These simulations clearly indicate global network stability, propagation of errors, predictable delays in reaching the set-points, the propagation of fluctuations due to downstream disturbances and all similar behaviors that can be expected from a real canal network.

5. CONCLUSIONS AND FUTURE WORK

In this paper, first steps have been taken to study the feasibility of a cyber physical system approach towards the

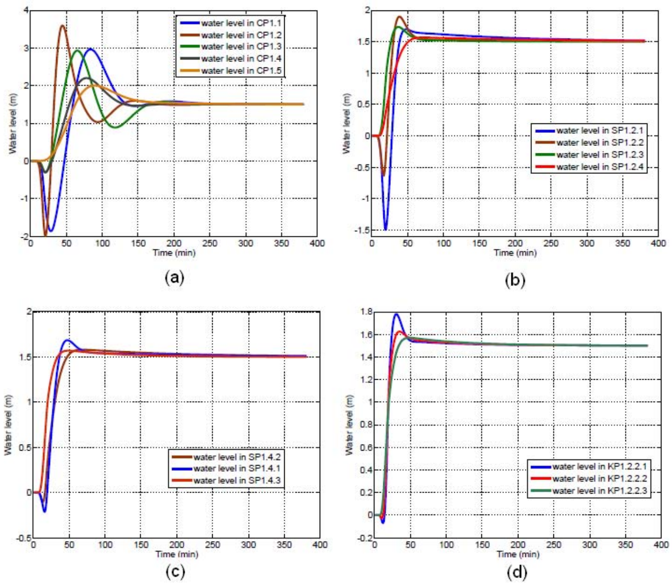

Fig. 13. (a) Water levels in main canal pools, (b) Water levels in secondary canal pools emerging from CP1.2, (c) Water levels in secondary canal pools emerging from CP1.2, (d) Water levels in khala pools emerging from SP1.2.2.

control of irrigation canal networks in a developing world setting. We have verified that:

(1) Design and simulation of a very large network is possible without very elaborate physical measurements or lab techniques.
(2) The proposed methodology is suitable for a devolved governance model, suitable in developing world environments where failures happen in both governance and infrastructure.
(3) The methodology is applicable, but would need an elaborate backbone of communication and energy infrastructure.

In future work, we aim to address the following issues.

(1) Vulnerability of the system to failures.
(2) Actual field tests on a real canal network.
(3) Integration of high-level planning with local canal command.
(4) Calculation of actual improvement in efficiency over existing system.
(5) Energy and communication budgets for a full-scale operation, and ways of optimizing them.

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