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Operation and Simulation of a Water Supply System

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Abstract: Water supply systems are becoming more important, since there are increasing requirements to improve operation whilst improving the environment so that their behaviour can be fully understood and the total process is optimised. This paper presents simulation and control of water supply system of Gaziantep city in Turkey for management purposes. The main objectives are to ensure the behaviour of a water supply system and to regulate the water flow and heads by manipulating the water pumps. The system consists of a sequence of pumping stations that deliver water through pipelines to intermediate storage reservoirs. The model used is obtained using dominant system variables that represent active and passive dynamical elements. The hydraulic models also include the nonlinear coupling between flows and reservoir heads. The polynomial H_∞ optimisation method is used to design a level controller that regulates water flow and heads through the system. The whole system is simulated, and the results are presented and compared with the real-time measured data.

Key Words: Simulation, Manipulating, Hydraulic Models

Introduction

During the last two decades, water demand has increased rapidly in the developing countries as a result of high population growth, improvement of living standards, rapid urbanization, industrialization and improvement of economic conditions while accessible of sources of water is decreasing (Abderrahman, 2000). These are exerting increasing pressure on local water authorities and water planners to satisfy the growing water demands. This is even more challenging in developing countries such as Turkey.

Water supply systems are used to transfer the water from sources to consumers. The operation and control of these water supply systems is one of the most important issues to provide enough water to consistently meet the demand (Brookshire and Whittington, 1993; Mousavi and Ramamurthy, 2000 and Lee *et al.*, 2000). However, due to financial and environmental reasons, management and control of water supply systems is needed to meet the demand (Mousavi and Ramamurthy, 2000; Gieling *et al.*, 2000 and Cembrano *et al.*, 2000). Thus it is necessary to carefully manage water transfer (Cembrano *et al.*, 2000). This requires modelling and simulation of the water supply system so that its behaviour can be fully understood and the total process is optimised (Obradovic, 2000 and Elbelkacemi *et al.*, 2001). Understanding the technical side of the water supply system such as knowledge about how the system behaves if a disturbance occurs at any point of the system is also a crucial problem (Simons, 1992; Brdys and Ulanicki, 1994 and Eker and Kara, 2001a). Crucial targets also include the continuity of operation and improvement of supply systems (Mousavi and Ramamurthy, 2000). All these have involved different technical disciplines (Simons, 1992).

Populations of cities has been increased substantially due to economical conditions and environmental conditions after 1980s in Turkey. Fresh water resources for drinking purposes, however, are far away from most of the city centers and towns. Satisfying the growing water demands require further studies. For example, the water demand in Gaziantep was about a flow rate of 1.5 m^3/sec. in early 1980s and rapid industrialization and growth in population after 1980s increased the water demand in the city to almost 3.0 m^3/sec. This paper presents simulation and control of water supply system of Gaziantep city in Turkey to practice water management. The main objectives are to ensure the behaviour of a water supply system and to regulate the water flow and heads by manipulating the water pumps. The system consists of a sequence of pumping stations that deliver water through pipelines to intermediate storage reservoirs. The model used is obtained using dominant system variables that represent active and passive dynamical elements. The hydraulic models also include the nonlinear coupling between flows and reservoir heads. The whole system is simulated. The paper also defines the present challenges and the measures required to meet the water demand. Measurements obtained on the real system and simulation results are compared to show the significance of the study.

Plant Description: The water is taken from Kartalkaya dam, which is 53 km. away from the city of Gaziantep. Fig. 1 illustrates the rough diagram of the water supply system in which there are three pumping stations (PST-1, PST-2, PST-3) and three reservoirs (RS-1, RS-2, RS-3) along the supply system. The supply system is a single line system, and no water is added or distributed along the supply system. The gravity system helps in flow of water up to next reservoir. Pumps are used to elevate water to the

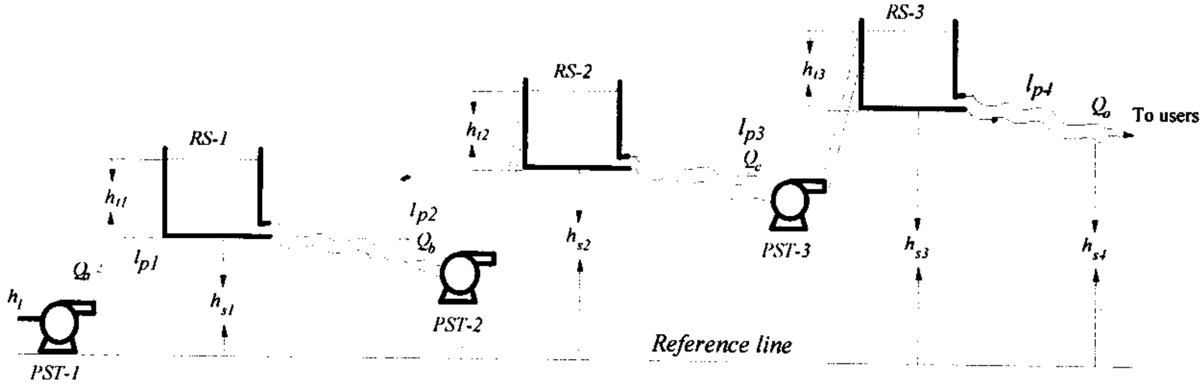

Fig. 1: Diagram of the City of Gaziantep Water Supply System

Table 1: Numerical Values of the Supply System Variables

l_{p1} =669.27 m	A_p =1.5394 m^2	D = 1.4 m
l_{p2} =13805.04 m	h_{s1} =113.4 m	G = 9.81 m/s^2
l_{p3} =20094.69 m	h_{s2} =210.4 m	N_{so} = 985 rpm
l_{p4} =4689.04 m	h_{s3} =283.4 m	$Q_{so} \cong$ 2.83 m^3/s
A_t =475 m^2	h_{s4} =279.7 m	

reservoir. The position of the PST-1 is taken as the reference point. $h_{ti}(t)$, h_{si}, l_{pi} denote the variable heads *(m)* in the reservoirs, static heads *(m)* and the lengths of pipes *(m)*, respectively. The variables $Q_a(t)$, $Q_b(t)$, and $Q_c(t)$ designate the water flow rates through pipes. The pipelines are buried underground and are assumed to be free of chemical reaction, biochemical, thermal and noise pollution, and the system does not include cavitation. Check valves are fitted on the discharge side of the each pump to maintain forward flow in the main and to prevent backflow. The air relief valves and pressure regulating valves are fitted on the piping system. The flow rates, reservoir heads and pump speeds represent deviations from the nominal steady-state operating values to obtain behaviour of the overall system. Uniform flows are assumed for the water supply system and the variations around nominal operating values do not deteriorate this generality (Douglas *et al.*, 1985). Distributed flow is not taken into account because that flow is considered for complex piping systems having high flow velocities, especially for water distribution systems, and for the systems having small and different pipe sections (Fox, 1977; McInnis and Karney, 1992). The flow disturbances in water supply systems are common (Simons, 1992; Brdys and Ulanicki, 1994; Eker and Kara, 2001b) and should be taken into account. It is assumed that the water has uniform density in the pipe. The three pumps work in parallel in each pump station with the nominal speed (N_{so}) of *985 rpm*. The pipes are concrete type with an inner diameter (D) of *1.4 meters* with a crossection area (A_p) of *1.5394* m^2 and *15* years old. The reservoirs have a cross sectional area (A_t) of *475* m^2. Bending curvatures of the pipes along the supply system were measured that those are larger than the pipe diameter (D). The numerical data about the water supply system of the city of Gaziantep are given in Table 1.

Models of Active and Passive Elements: Water supply systems are generally composed of a large number of interconnected pipes, reservoirs, pumps, valves and other hydraulic elements (Cembrano *et al.*, 2000; Brdys and Ulanicki, 1994; Eker and Kara, 2001a). These elements, that play important roles in system dynamic behaviour, can be classified into two categories: active and passive (Brdys and Ulanicki, 1994; Ermolin *et al.*, 2001; Rothe and Runstadler, 1978). The active elements are those which can be operated to control the flow of water in specific parts of the system, such as pumps and valves. The pipes and reservoirs are passive elements, insofar as they receive the effects of the active elements. Simulations of water supply systems has been an indispensable work to understand their behaviour to produce feasible control solution as well as modelling (Tillman *et al.*, 1999; Eker and Kara, 2001b; Tillman *et al.*, 2001) to generate ideas in order to develop flexible management and design schemes (Gieling *et al.*, 2000; Cembrano *et al.*, 2000; Elbelkacemi *et al.*, 2001). It also combines technical and financial viewpoints (Gieling *et al.*, 2000 and Tillman *et al.*, 2001). The first step in simulation and control is to establish a mathematical model for the plant to be controlled. Furthermore, an adequate model is an important step in determining the behaviour and producing a control algorithm (Elbelkacemi *et al.*, 2001). A successful model should be able to take advantage of system features that leads to simpler mathematical formulations and the proper choice of solution method (Mousavi and Ramamurthy, 2000). Hydraulic systems generally lead to complex models. Derivation of control strategies on the basis of the complex models is difficult (Elbelkacemi *et al.*, 2001). For these reasons, plant model should be chosen using the plant dominant variables to reflect the dynamical behaviour of the plant such as the pumps discharge, water heads in the reservoirs and flow rates through the system (Elbelkacemi *et al.*, 2001; Brdys and Ulanicki, 1994; Ermolin *et al.*, 2001). Thus, the simulation of the model that represents a water supply system proves an efficient measure to contribute to the correct transfer of water and to reduce operational cost, as well as to improve the operation (Cembrano *et al.*, 2000). It is assumed that water is an incompressible liquid and, the system and its individual components are stationary.

Pumps, pipes and reservoirs play important roles in water supply system dynamic behaviour (Cembrano *et al.*, 2000; Brdys and Ulanicki, 1994; Eker and Kara, 2001a). It is assumed that water is an incompressible liquid, and the system and its individual components are stationary.

Pumps: Head developed by a variable-speed pump that is running in parallel with other pumps in a pump station varies nonlinearly with its speed N *(rpm)* and output water flow rate $Q_p(t)$ (m^3s^{-1}) (Brdys and Ulanicki, 1994; Eker and Kara, 2001a):

$$h_p(N,Q_p) = A_o N^2 + \frac{B_o}{n} N Q_p - \frac{C_o}{n^2} Q_p^2 \qquad (1)$$

where A_o, B_o, C_o are the constants for a particular pump depending on component characteristics and n is the number of the pumps running in parallel in a pump station. The constants A_o, B_o, C_o can also be calculated using appropriate manufacturer's specifications (Cembrano *et al.*, 2000).

Pipes: Consider a pipe section with length l_p *(m)* and of area A_p (m^2). If the head difference between two ends of a pipe section, Δh is considered, the following differential equation can be given (Eker and Kara, 2001a; Fox, 1977):

$$\frac{dQ(t)}{dt} = \frac{gA_p}{l_p}\left(\Delta h - h_{loss}(t)\right) \qquad (2)$$

where h_{loss} denotes the total head loss along the piping system, g is the acceleration due to gravity. The flow rate and head loss may be given as:

$$h_{loss}(t) = h^o_{loss} + \Delta h_{loss}(t),\ Q(t) = Q^o + \Delta Q(t) \qquad (3)$$

where $(.)^o$ denotes nominal steady-state value and $\Delta h_{loss}(t)$ designates the variable head loss caused by the variable water flow rate $\Delta Q(t)$. Friction losses and local losses are two different types of losses in hydraulic systems (Simon, 1992; Brdys and Ulanicki, 1994). There are several approaches obtained from theoretical considerations and experimental data for the friction loss in pipes (Fox, 1977 and Douglas *et al.*, 1985). The total loss in a pipe section can be given as (Eker and Kara, 2001a):

$$h_{loss}(t) = h_{loss-f_p}(t) + h_{loss-l}(t) \qquad (4)$$

where $h_{loss-fp}$ denote friction loss, h_{loss-l} denote local losses. Hazen-Williams (Brdys and Ulanicki, 1994) and Darcy-Weisbach approaches (Fox, 1977; Douglas *et al.*, 1985) have been frequently used in obtaining the head loss in piping systems (Cembrano *et al.*, 2000). The Darcy-Weisbach approach depends on the flow rate in pipes, and the head loss changes with the flow rate nonlinearly as (Eker and Kara, 2001a)

$$h_{loss-f_p}(t) = \left(\frac{f_p l_p}{DA_p^2 2g}\right) Q^2(t) \qquad (5)$$

Local losses caused by expansions, contractions, bends, valves, flow at entrance and flow at exit resulting from rapid changes in the direction or magnitude of the velocity of water are negligible for the long pipes (Eker and Kara, 200b; Douglas *et al.*, 1985).

Reservoir: When a reservoir discharges under its own head without external pressure, the continuity equation (Brdys and Ulanicki, 1994) can be applied as:

$$\frac{d(\rho V(t))}{dt} = \rho_i Q_i(t) - \rho_o Q_o(t) \qquad (6)$$

where ρ, ρ_i, ρ_o represent the water densities inside the reservoir, water inflow and outflow, respectively, that are assumed constant and equal $(\rho=\rho_i=\rho_o)$. $Q_i(t)$ (m^3s^{-1}) and $Q_o(t)$ (m^3s^{-1}) denote reservoir input and output water flow rates, respectively and $V(t)$ (m^3) is the volume of a particular reservoir.

Friction Coefficient For Pipes: There are several methods to obtain friction coefficient $f_p(\varepsilon)$ (Fox, 1977; Douglas *et al.*, 1985). The friction coefficient which depends on surface roughness under rough conditions can give accurate predictions of flows (Eker and Kara, 2001a). The Colebrook and White's empirical correlation seems to be the best for this purpose as recommended in (Fox, 1977):

$$\frac{1}{\sqrt{f_p}} = -4\log_{10}\left(\frac{\varepsilon / D}{3.71} + \frac{2.51}{2\sqrt{2f_p}N_R}\right) \qquad (7)$$

where 'ε' is the roughness of the pipe. This nonlinear equation can be solved using graphical methods or using a numerical method by using a computer program. Iterative type of solution gives more accurate result (Fox, 1977). The'*bisection*' numerical solution method was modified (Eker and Kara, 2001a) to solve the nonlinear equation (7). The algorithm needs two values initially, the upper and the lower values of friction coefficient. The developed algorithm is based iteration process by narrowing the upper (f_{pu}) and lower (f_{pl}) limit values to obtain desired friction coefficient with the defined error tolerance. Set $i=1$ initially. The steps to follow are:

step 1. Re-arrange the equation (7) to obtain as:

$$g(f_p) = 0 \qquad (8)$$

step 2. Provide initial values for the upper f_{pu} and lower f_{pl} frictions that satisfy

$$g(f_{pl}) * g(f_{pu}) < 0 \qquad (9)$$

step 3. Calculate the friction coefficient

$$f_{pi} = 0.5(f_{pl} + f_{pu}) \tag{10}$$

step 4. Check the error for a defined tolerance *(δ)*

$$g(f_{p_i}) < \delta \tag{11}$$

step 5. If the error is greater than the defined tolerance *(δ)*

$$g(f_{pi})^* g(f_{pl}) < 0 \quad f_{pu} = f_{pi} \tag{12}$$

i=i+1 and go to step 3.

step 6. If the equation (12) is not satisfied, then

$$g(f_{pi})^* g(f_{pl}) > 0 \;\Rightarrow f_{pl} = f_{pi} \tag{13}$$

i=i+1 and go to step 3.

Iteration is preceded until the defined error tolerance is achieved.

Field Tests and Simulations: Some measurements on the real system and nonlinear simulations were performed. Fig. 2 shows the output water flow rate which was obtained from the real system at an hour interval time in a day, and 24 measurements were taken using a flow meter installed at the receiving end of the water supply at the nominal speed of the pumps. Using the data obtained, the average water flow rate is about Q_{so}= *2.83 (10188 m^3/sec. m^3/hour)*, and it changes between 10175 *m^3/hour* and 10203 *m^3/hour*. Measured heads in reservoirs heads were also shown in Fig. 3 such that the heads deviate 4.2 meters for RS-1, 2.15 meters in RS-2 and 3.2 meters in RS-3, respectively.

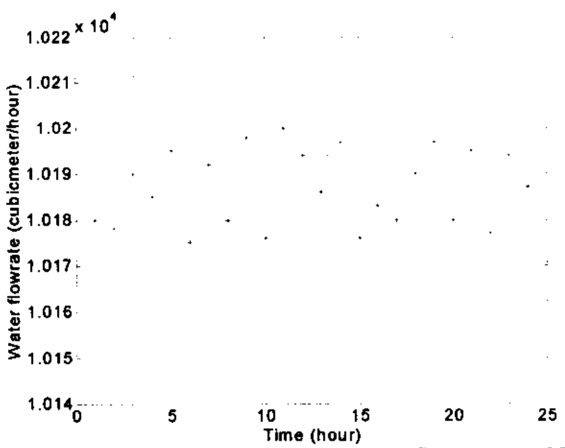

Fig. 2: Output Measured Water flow Rate (*$Q_o(t)$ m^3/hour)*

The pump characteristics were obtained from the manufacturer that was shown in Fig. 4. Head developed by the pump was calculated around the nominal operating point (N_{so}=985 rev.min^{-1}; Q_{so}=2.83 m^3.sec^{-1}) using the characteristic curve as:

$$h_p(N,Q_p) = 0.0001433N^2 + 0.005015NQ_p - 3.98Q_p^2$$

To check the zeros and poles of the system, it was linearised using Taylor Series Expansion method around the nominal operating point (N_{so}=985 rev min^{-1}; Q_{so}=2.83 m^3.sec^{-1}). The system can be represented in state space matrix form such that the reservoir

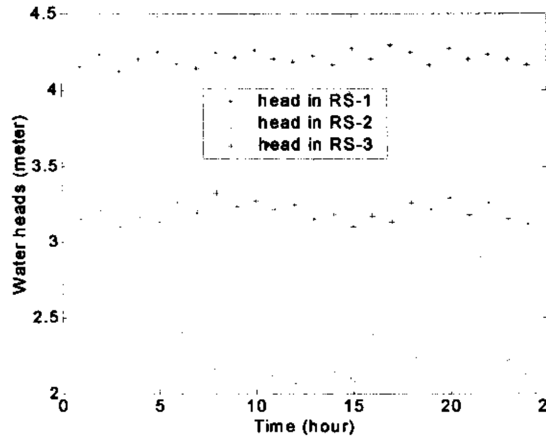

Fig. 3: Measured Water Heads in Reservoirs

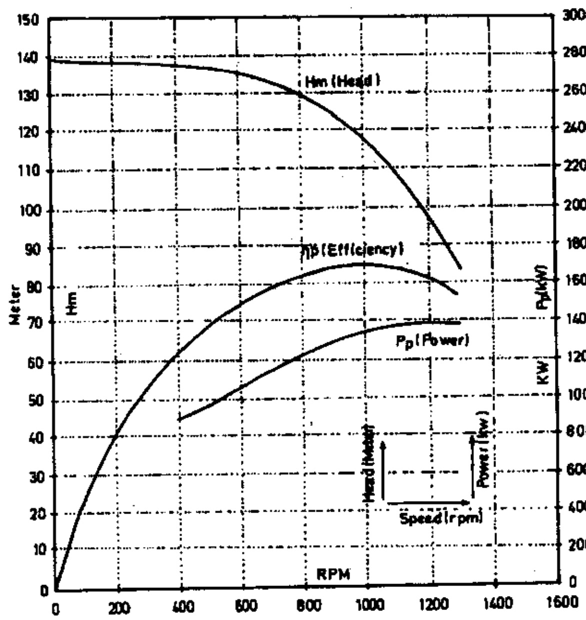

Fig. 4: Pump Characteristics

heads and flow rates can be considered as states. The manipulated input variable is the pump speed *N*, and the output variable is the water flow rate *$Q_o(t)$*. The canonical state space form is:

$$\dot{x}(t) = Ax(t) + Bu(t), \quad y(t) = Cx(t) \tag{14}$$

where *x(t)* is the state matrix, *A, B, C* are the constant system matrices, *u(t)* is the input and *y(t)* is the system output. The state matrix *x(t)*, input *u(t)* and calculated constant matrices, *A, B, C,* are

$$x(t) = [Q_o \quad h_{t3} \quad Q_c \quad h_{t2} \quad Q_b \quad h_{t1} \quad Q_a]^T \tag{15}$$

$$u(t) = N \tag{16}$$

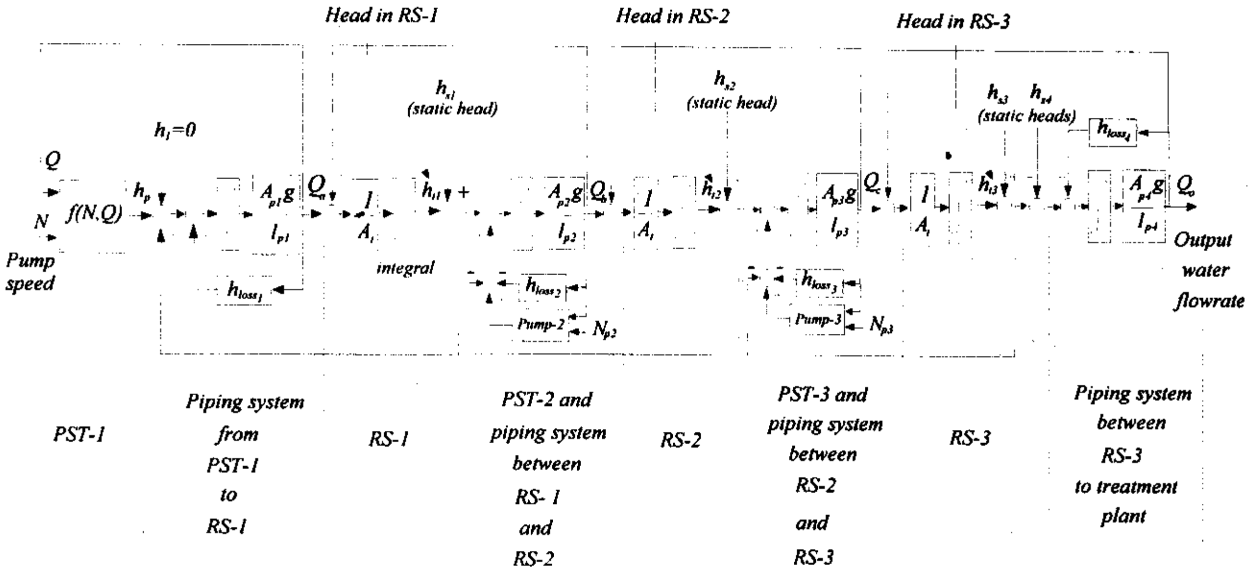

Fig. 5: Block Diagram of the Water Supply System

$$A=\begin{bmatrix} -0.0253 & 0.0032 & 0 & 0 & 0 & 0 & 0 \\ -0.0021 & 0 & 0.0021 & 0 & 0 & 0 & 0 \\ 0 & -0.0008 & -0.0398 & 0.0008 & 0 & 0 & 0 \\ 0 & 0 & -0.0021 & 0 & 0.0021 & 0 & 0 \\ 0 & 0 & 0 & -0.0011 & -0.0465 & 0.0011 & 0 \\ 0 & 0 & 0 & 0 & -0.0021 & 0 & 0.0021 \\ 0 & 0 & 0 & 0 & 0 & -0.0226 & -0.4553 \end{bmatrix}$$

$$B=\begin{bmatrix}0 & 0 & 0 & 0 & 0 & 0 & 0.0067\end{bmatrix}^T,\ C=\begin{bmatrix}1 & 0 & 0 & 0 & 0 & 0 & 0\end{bmatrix}$$

The system is 7th order. The zeros and poles of the supply system were calculated and given in Table 2 so that the system is minimum phase and stable. The last three poles are the closest to the origin with time constants of 0.86, 1.11 and 4.47 hours, respectively. The slowest dominant time constant is about 4.47 hours which dominates the response of the system.

Table 2: Zero and Poles of the Water Supply System

Zeros	Poles (Second^{-1})
-0.0396805	-0.4551564
-0.0255868	-0.0396796
-0.0250794	-0.0255573
-0.0002201±0.0000598*i*	-0.0250177
	-0.0003240
	-0.0002504
	-0.0000621

The nonlinear block diagram of the system is shown in Fig. 5. The system is simulated to obtain its response to input changes and load changes with the head losses on the system. The output response ($Q_o(t)$) was illustrated in Fig. 6 for N=985±20 rpm square wave speed variations with a frequency of 5.10^{-6} Hz in the PST-1. The speed variation was applied at $t=10^5$ *secs.* (t=27.7 hours) after starting. The steady-state output water flow rate was obtained at about 2.835 m^3s^{-1}. The friction losses for the pipe section 3, (l_{p3}) were also obtained as shown in Fig. 7. Approximately *45* meters head loss in steady-state was obtained from the Darcy-Weisbach's method that was shown in solid line. The smaller head loss in magnitude (dashed line) was obtained from the Hazen-Williams's method, about *44.5* meters in steady-state operating conditions.

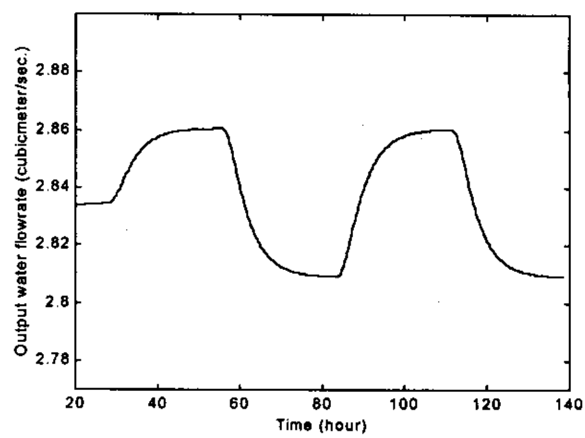

Fig. 6: Output Flow Rate $Q_o(t)$ for *N=985±20 rpm* Square Wave Speed ariations

The nonlinear friction coefficient equations for different methods (Douglas *et al.*, 1985) were solved using improved *bisection* numerical solution algorithm that was implemented to run in Matlab environment. The algorithm was very simple providing fast convergence and is easy to use. The calculated values obtained for different methods were given in Table 3. The methods of Davies and White, the Prantdl and von Karman and the Blasius were developed for smooth pipes, and that is why the calculated friction coefficients from these methods were different and smaller than those values obtained from the other methods. The results obtained from the methods developed for the rough pipes show that all values are very close to each other. The results are consistent with the fact that RS-1 needs level control, which has been mentioned by the system operators such that the level is just at the overflow limit (4.30 meters).

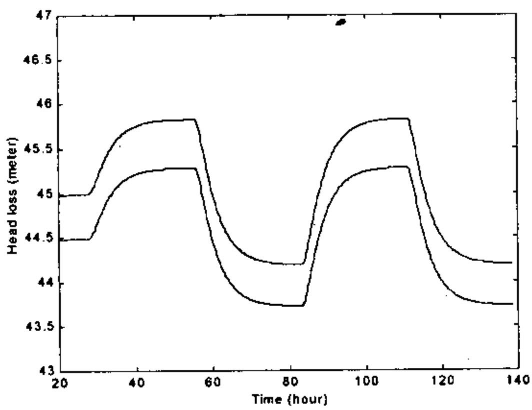

Fig. 7: Head Losses Obtained from the Hazen-Williams's Method (dashed) and Darcy-Weisbach's Method (solid) for the Pipe Section 3, l_{p3}.

Table 3: Calculated Friction Coefficients for Different Methods

Methods	f_p (ε)
Davies and White	0.00873830
Blasius	0.00862908
The Prantdl and von Karman	0.01055540
Gainguillet and Kutter	0.01754690
Nikuradse	0.01810300
Colebrook and White	0.01833740
Colebrook	0.01834350
Moody	0.01901630

Robust H_∞ Optimisation and a Level Controller Design: The feedback control is needed to provide more stable operation and to improve the robustness margins. Since the water head was just at the overflow limit in the RS-1, a level controller should be designed. The feedback control diagram to regulate the water head in the RS-1 is given in Fig. 8. The speed of one of the pumps is manipulated to control the water head in RS-1. Because manipulating the speed of the three pumps simultaneously has higher costs than that of a single pump, most of the water supply systems work in that case in Germany (Eker and Kara, 2001b). This reduces the cost of operation and saves several kW electricity per day. N_1 represents the manipulated input variable as the pump speed. The nominal speed is assumed for the two pumps in the PST-1 ($N_2=N_3=985$ *rpm*) and for the other pumps in the PST-2 and PST-3. h_t denotes the output variable as the water level at the RS-1 to control, Q_t is the total water flow rate pumped by the first pump station, Q_{di} and Q_{do} denote input and output water flow rate disturbances, respectively. h^{ref} is the setpoint for h_t (4 meters). The linearised time-domain state-space system is transferred into the s-domain transfer function. *C(s)* denotes the controller, $G_1(s)$ and $G_2(s)$ represent the linearised pumps and $G_t(s)$ denotes the linearised reservoir, $G_{p1}(s)$ and $G_{p2}(s)$ are the linearised pipe transfer functions including pipe losses, and h_{c1} and h_{c2} are the static heads.

The polynomial H_∞ controller design method (Grimble, 1987) is considered, since this method has several advantages and applications (Eker and Johnson, 1996). The cost function to be minimised, the parameterised dynamic weighting functions and the optimal controller are:

$$J_\infty = \left\| \Phi_{\varphi\varphi}(s) \right\|_\infty \tag{17}$$

$$P_e = \frac{P_{en}}{P_{ed}} = \frac{s+k_{e1}}{k_{e2}s+\delta}, F_c = \frac{F_{cn}}{F_{cd}} = \beta\frac{s+k_{c1}}{k_{c2}s+k_{c3}} \tag{18}$$

$$C(s) = \frac{F_{cd}G}{P_{ed}H} \tag{19}$$

where $\Phi_{\varphi\varphi}(s)$ is the spectral density of the weighted sum of the error and control signals, P_e and F_c are dynamic weighting functions, k_{e1}, k_{e2}, k_{c1}, k_{c2}, k_{c3}, β and δ are positive tuning parameters. In the Laplace transform domain, $\varphi(s)=P_e e(s)+F_c u(s)$. The dynamic weighting functions, P_e and F_c are to achieve good disturbance rejection and attenuation at high frequencies and good tracking. The polynomials G and H are the solutions of a couple of diophantine equations in the H_∞ design algorithm. The design algorithm and more information about selection of dynamic weightings and their parameters can be found in references (Eker and Johnson, 1996). The normalised linear models used for the controller design are calculated as:

$$G_1(s) = 5.15195 \qquad G_2(s) = 0.01759$$

$$G_{p1}(s) = 44.3225s + 1.0473$$

$$G_t(s) = \frac{1}{475s}$$

$$G_{p2}(s) = \frac{330.047}{13805.04s + 330.047}$$

The dynamic weighting functions were chosen as

$$P_e(s) = \frac{1}{(1000000s)} \quad F_c(s) = \frac{(0.01s+0.001)}{1}$$

The optimal H_∞ controller was calculated as:

$$C(s) = \frac{0.154s^3 + 0.0732s^2 + 0.0017s + 0.18\cdot 10^{-6}}{s(s^3 + 0.575s^2 + 0.0586s + 0.001)}$$

The integral control eliminates steady-state error, input and output disturbances. The high frequency roll-off eliminates high frequency noise, measurement disturbance and unwanted signals.

The nonlinear system was simulated using the linear feedback controller in Simulink. $Q_{do}=\pm 5\%$ *(± 0.142 m^3/sec.)* square wave output flow disturbance is applied with a frequency of *0.000005 Hz.* at $t=10^5$ *secs.* (t=27.7 hours) after starting. The water heads in the reservoirs were illustrated in Fig. 9. The output flow disturbance caused very small changes in the head of RS-1 (solid line). Because the disturbance is applied at a point after the RS-1. However, significant changes occurred in the head of RS-2 (dotted line) such that the water head was increased to *3.8 meters* for the increased cycle of the flow disturbance changes and decreased to 0.25 meters for the decreased cycle of the flow disturbance. Because the disturbance was applied just before the RS-2. Almost *0.5 meters* in peak-to-peak water head changes were obtained in the RS-3.

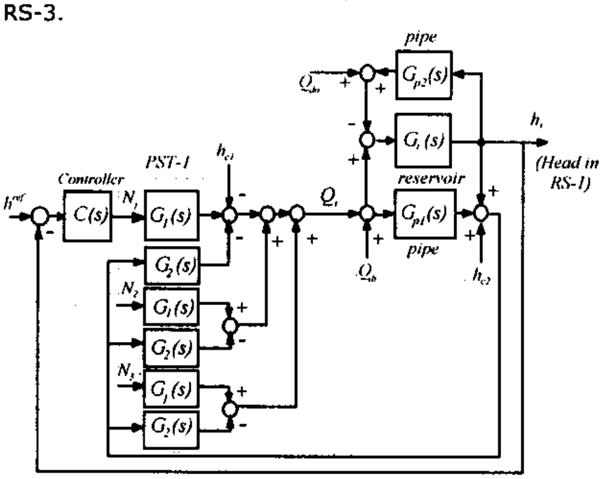

Fig. 8: Feedback Control Diagram of the System

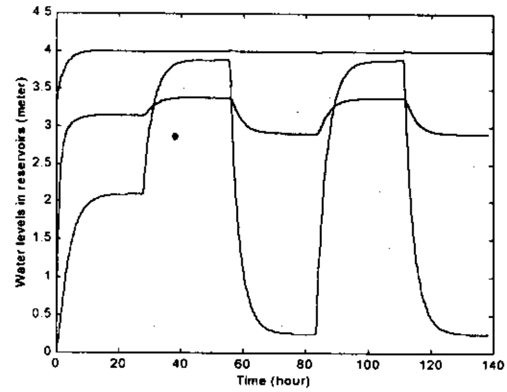

Fig. 9: Water Heads in the Reservoir for ±5% Output Load flow Disturbance Q_{do} (0.1415 m^3/s), (solid Line= water head in the RS-1; Dotted line= water Head in the RS-2;dashed line=Water Head in the RS-3.)

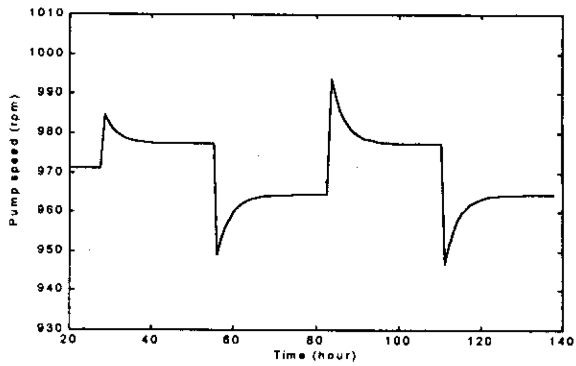

Fig. 10: Speed Change of the Variable Pump, N_1 for *±5%* Output Load Flow Disturbance, Q_{do}

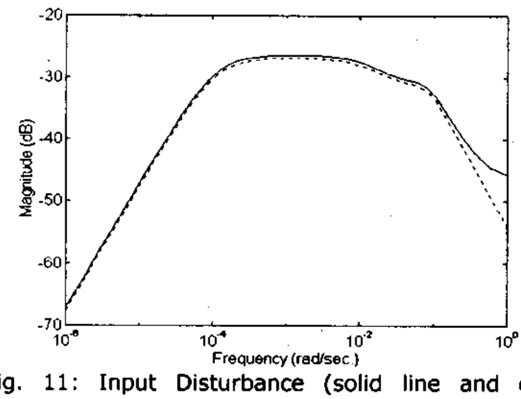

Fig. 11: Input Disturbance (solid line and output Disturbance (dotted line)

The changes in the speed of the pump in the PST-1 were illustrated in Fig. 10 that occurred as a result of the square wave output load flow disturbance changes. The speed of the variable pump was reduced to a *972 rpm* at steady-state when the feedback control was used. The speed increased or decreased according to the applied disturbance cycle. The system performance obtained from the closed-loop frequency response was improved by the feedback control so that *38 dB* gain margin, *82°* phase margin, and *0.0027 rad/sec.* of operation frequency were obtained. The robustness to disturbances are achieved such that $\|h_t / Q_{do}\|_\infty < 1$ and $\|h_t / Q_{di}\|_\infty < 1$ should be satisfied (Eker and Johnson, 1996). Fig. 11 illustrates frequency response of the input and output disturbance sensitivities, ($h_t(s)/Q_{di}(s)$ and $h_t(s)/Q_{do}(s)$), respectively. The largest gains of the disturbances were about *-26.5 dB* and *-26 dB*, respectively that satisfy the disturbance rejection conditions given.

Conclusion

Simulation and control of water supply systems is an efficient means for water transfer operation to achieve management goals such as flow regulation and cost minimisation. The study presented here seems to be effective in tackling the behaviour of the supply system and to generate feasible and reasonable solution to improve system operation. It was chosen a modelling approach to enhance our understanding of the observed system. This was not primarily to provide definitely accurate representation, but to enrich our understanding of fundamental behaviour of the system to produce a control mechanism. Models of active and passive elements were incorporated in obtaining behaviour of the system. The hydraulic models, in particular, included the nonlinear coupling between flows and reservoir heads. The study revealed the dynamics of behaviour and interactions among active and passive elements in the water supply system. The type of advanced control design method applied can improve control of the water supply system by limiting unacceptable variations in flow and level. There is also the possibility of building in requirements to limit energy consumption and even taking into account hard constraints in future work.

The case study presented in the paper showed that operation can be improved by using an optimal robust control method to control water transfer operation. Gaziantep city water supply system in Turkey was considered. A good approximation for pumps was obtained using appropriate manufacturer's specifications. The whole system was simulated and the results were presented in both the time and frequency domains. A robust H_∞ control approach was used to keep the flow and reservoir heads at a fixed level. This concept has the intrinsic ability to compensate for changes in water disturbance that occurred at any point of the water supply system. The model that was considered allowed to simulate the different scenerios of the system to compare the results with observed phenomena of the real-time system. The improved bisection numerical solution method was used to solve nonlinear equations for the friction coefficient numerically using iterative type of solution.

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