

PUMPING STATION SCHEDULING FOR WATER DISTRIBUTION NETWORKS IN EPANET

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Pumping station scheduling for variable water supply can be set in EPANET, based on user-defined Patterns for some key parameters (i.e. water demand, head, relative speed for each pump and energy price), with a pattern time step of 1 hour, over a total duration of 24 hours. We present a methodology to design a hydraulic system and to set the pump operation schedules yielding to energy cost-savings, in order to feed a simple water distribution network. The hydraulic system consists of one reservoir, a pumping station with 3 centrifugal pumps of variable speed, 17 junctions and 20 pipes; within this system, 12 pipes belong to a looped network, which supplies 5 end-users. The proposed algorithm for the pumping scheduling is improved with respect to the classical-one (which corresponds to the pumping station operation with one pump at variable speed, and any other opened pump at constant speed, namely the nominal speed).

Keywords: Pumping station scheduling, EPANET, demand pattern, head pattern, speed pattern, price pattern

1. Introduction

One can design a water distribution network either by using a classical approach based on economic criteria, where the optimal diameter of a pipe corresponds to a certain flow rate range [1], or by using a stochastic method for combinatorial optimization, which yields a least-cost design especially for urban size networks [2].

The optimisation of pump scheduling in water distribution systems plays an important role in reducing the energy consumption, and therefore, the attached cost. This subject has been intensively studied in the last two decades using dynamic programming [3]–[4], as well as stochastic optimization algorithms [5]–[9]. All proposed solutions deal with pump operation schedules, where each pump speed is adjusted to minimize the overall energy consumption. Most studies consider hydraulic systems consisting of at least one water source, one or several pumping stations with variable speed driven pumps and at least one storage tank [10]. Additionally, for such systems, the pumping schedule can be improved by

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shifting the pumps operation from peak-hours and mid-hours, to off-peak hours, where the energy price is significantly lower [11]; thus, a tank filled during the night can be emptied during the day, to contribute to the hourly variable water demand. But the above solution based on shifting pumping hours cannot be applied for systems without a storage tank.

In this paper, we present a methodology to design a hydraulic system and to set the pump operation schedules, yielding to energy cost-savings, in order to feeding a looped water distribution network (like in figure 1), upon a variable water demand; the studied hydraulic system do not contain a storage tank. The water pipe network is designed based on economic criteria [1], [12]. After the pumps selection, the pumping station operating algorithm is set in EPANET, based on user-defined Patterns [12]–[13], for some key parameters (i.e. water demand, head, relative speed for each pump and energy price), with a pattern time step of 1 hour, over a total duration of 24 hours.

In Section 4, we set a classical pumping station operating algorithm [12], where one pump operates at variable speed, and any other opened pump operates at constant speed, namely the nominal speed. In Section 5, we propose a better pumping station operating algorithm, where all opened pumps operate, at a time step, with equal values of the speed (not necessarily the nominal speed), ensuring a minimal total power consumption [6]. Finally, both solutions are compared.

2. Input data

The hydraulic system, modelled in EPANET, consists of a reservoir that is open to the atmosphere (representing an infinite external water source [13]), a pumping station equipped with 3 identical centrifugal pumps of variable speed, 17 junctions and 20 pipes; the pipes are labelled in figure 1 by $j = 1 \div 20$. Within the studied system, 12 pipes (with $j = 9 \div 20$) belong to a looped network, which supplies 5 end-users labelled in figure 2 by the node ID $\in \{11; 12; 14; 16; 18\}$.

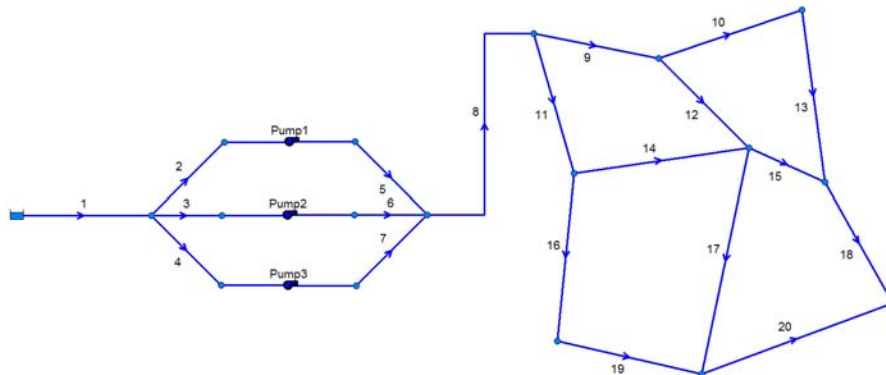


Fig. 1. Water distribution system: pipe ID-s labelled for $j = 1 \div 20$

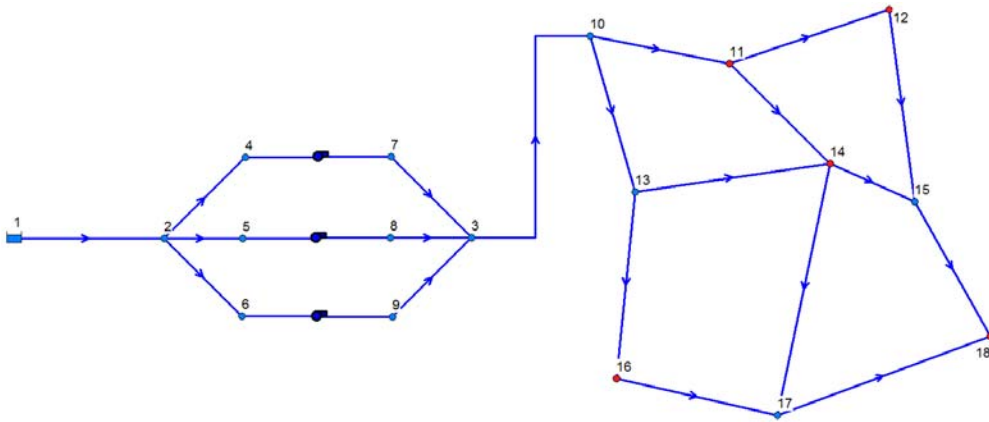


Fig. 2. Water distribution system: node ID-s labelled from 1 (at the reservoir) to 18

Table 1

Pipe's length and diameter

Pipe ID j	L_j [m]	D_j [mm]	Pipe ID j	L_j [m]	D_j [mm]
1	320	700	11	375	500
2	32	500	12	300	250
3	32	500	13	405	125
4	32	500	14	480	350
5	140	500	15	225	200
6	140	500	16	390	400
7	140	500	17	525	200
8	1080	700	18	375	250
9	300	500	19	375	250
10	375	350	20	450	300

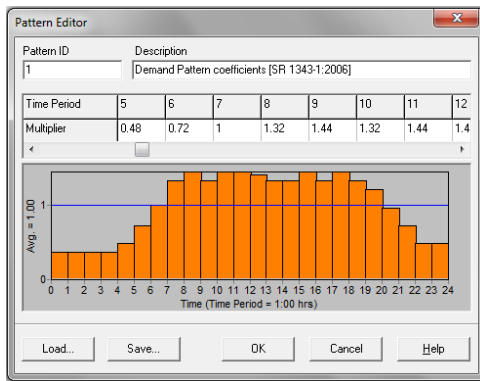


Fig. 3. Water Demand Pattern

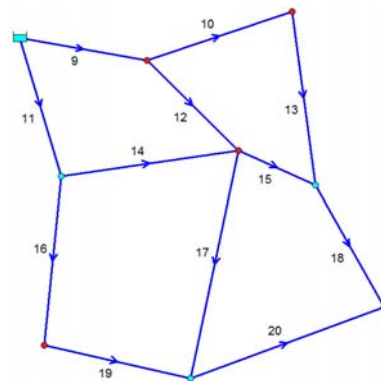


Fig. 4. Looped pipe network

Table 2

Water Demand Pattern coefficients, Energy Price Pattern coefficients and Head Pattern coefficients, starting from mid-night (e.g. $t = 1$ over the clock period from 00:00 to 01:00)

t [h]	1	2	3	4	5	6	7	8	9	10	11	12
c_q	0.36	0.36	0.36	0.36	0.48	0.72	1	1.32	1.44	1.32	1.44	1.44
c_e	0.679	0.679	0.679	0.679	0.679	0.679	0.906	1.811	1.811	0.906	0.906	0.906
c_h	0.919	0.919	0.919	0.919	0.928	0.955	1	1.069	1.099	1.069	1.099	1.099
t [h]	13	14	15	16	17	18	19	20	21	22	23	24
c_q	1.4	1.32	1.32	1.44	1.32	1.44	1.32	1.2	0.96	0.72	0.48	0.48
c_e	0.906	0.906	0.906	0.906	0.906	0.906	0.906	0.906	1.811	1.811	0.906	0.906
c_h	1.089	1.069	1.069	1.099	1.069	1.099	1.069	1.041	0.993	0.955	0.928	0.928

The input data are: the pipe length L_j , where $j = 1 \div 20$ (see table 1); the wall roughness $k = 0.5$ mm for all pipes; assuming a flat hydraulic system, all nodes have the same elevation, e.g. $z = 0$ m. The variable water demand is set upon a Demand Pattern like in figure 3 [14, Annex 2], where the demand pattern coefficients $c_q(t)$ depend on time t , in hours; $c_q(t)$ values are inserted in table 2, over a 24 hours period, starting from mid-night. Each end-user, labelled in figure 2 by the node $ID \in \{11; 12; 14; 16; 18\}$, requests a variable base demand $Q_c(t) = c_q(t)Q_{cm}$, where the value of the daily mean base demand is $Q_{cm} = 88$ l/s. The hydraulic time step is set to one hour (equal to the pattern time step). Each end-user requests a minimum gauge pressure of 40 mH₂O (or pressure head of 40 m). Head losses are computed using the Darcy-Weisbach formula.

The energy price varies upon a Price Pattern, defined hourly by the coefficients $c_e(t)$ over a 24 hours period, as in table 2. The above $c_e(t)$ values are multiplying the daily mean price of 0.065 €/kWh, to yield the hourly energy price in a working day of April 2014, for an electrical power distribution and supply company in Bucharest [15].

Based on the above known data, firstly one must choose the optimal diameter for each pipe, then select the appropriate pumps (the pump characteristic curves, namely the head versus flow rate curve, and the efficiency curve are both unknown). Secondly, one must set the operating algorithm for the pumping station, meaning the start/stop command for each pump and the corresponding pump speed (relative speed, with respect to the nominal speed), at each time step, in order to supply the variable water demand, at the requested pressure at each end-user, with a minimum energy cost per day.

3. Hydraulic system design

The design of the studied hydraulic system is performed for the daily mean water demand, which total value is $Q_m = 5Q_{cm} = 440$ l/s.

At a first step, only the looped pipe network will be designed. To do that, we detach that network from the system presented in figure 1, and we replace the supplying node (with ID=10 in figure 2), by a reservoir, as in figure 4. Based on the continuity equation, starting from the reservoir to the end-users, we admit a flow rate distribution on pipes $j = 9 \div 20$, as a start solution, then we choose the corresponding economic diameter D_j [1], [12]. We set an initial total head at the reservoir, reasonably bigger than the imposed pressure head at the end-users. Then we perform iteratively few Hydraulic Analysis in EPANET, by adjusting the total head value at the reservoir in order to obtain exactly the 40 m pressure head at the most disadvantaged end-user, which is the one with ID=18 in figure 2. Simultaneously, the pipe diameters are verified and modified if necessary, to fit the economic criteria. The final value of the total head at the reservoir is $H_m = 44.16$ m; it will be termed further as daily mean total head. The final values of the pipe diameters D_j , for $j = 9 \div 20$, are inserted in table 1.

In table 2, the values of the demand pattern coefficients c_q vary from 0.36 (at off-peak water demand hours), to 1.44 (at peak water demand hours). We assume that the water is supplied at off-peak by a single pump (at a certain rotational speed); at peak, it is supplied by all 3 pumps. The daily mean water demand (recorded from 06:00 to 07:00 a.m., where $c_q = 1$) can be supplied by 2 pumps (with identical duty point), the third one being closed. So, for the daily mean flow rate, based on continuity equation, we can choose the economic diameter for pipes labelled by $j \in \{1; 2; 3; 5; 6; 8\}$ in figure 1. The pipe $j = 4$ is identical with pipes $j \in \{2; 3\}$, and the pipe $j = 7$ is identical with pipes $j \in \{5; 6\}$. The values of the diameters D_j for pipes $j = 1 \div 8$ are inserted in table 1.

For a constant head $H_m = 44.16$ m at the reservoir, and the demand pattern from figure 3 attached to each end-user, at the most disadvantaged end-user we will get a pressure difference ΔH , positive or negative, with respect to the imposed pressure head of 40m. To keep a constant pressure at that end-user, over 24 hours, the total head at the reservoir must vary upon time as $H(t) = c_h(t)H_m$, where the head pattern coefficient is defined as:

$$c_h(t) = (H_m - \Delta H) / H_m . \quad (1)$$

The computed values of the head pattern coefficient are inserted in table 2; they form the Head Pattern from figure 5, which is set at the reservoir from figure 4. The time variation of the head at the reservoir (from 40.57m to 48.53m) and at the most disadvantaged end-user (equal to 40m, kept constant) is plotted in figure 6.

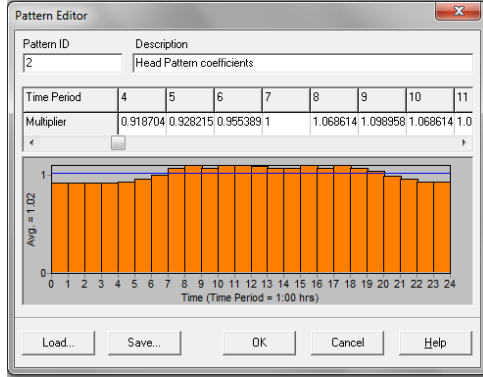


Fig. 5. Head Pattern

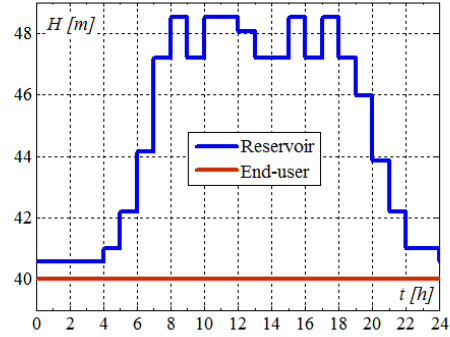


Fig. 6. Head variation at the reservoir and at the most disadvantaged end-user

The pumping station of the hydraulic system from figure 2 must ensure at the supplying node with ID=10 the same time variation of the head, as the one computed for the reservoir from figure 4, namely: $H(t) = c_h(t)H_m$.

The pumps selection is based on the assumption that at the time step from 06:00 to 07:00 a.m., where $c_q = 1$ and $c_h = 1$, only 2 pumps are working, with identical duty point. Let's assume that those pumps operate at that time step with their nominal rotational speed. So we have to look for a pump that can ensure the requested duty point, which is defined by the following pair of flow rate and head: $\{Q_m/2; (H_m + h_{r5} + h_{r8})\}$, where h_{rj} are the head losses on pipes $j = \{5; 8\}$. We found a centrifugal pump, which head – flow rate curve is defined, for the nominal speed of the pump, by the following second order polynomial equation:

$$H_i = H_i(Q_i) = 83 - 741.6 Q_i^2; \text{ [m]}, \quad (2)$$

with the flow rate in m^3/s ; the subscript $i \in \{1; 2; 3\}$ is attached to the pumps denoted in figure 1 as Pump1, Pump2 and Pump3. The efficiency curve is defined, for the nominal speed of the pump, by the following polynomial equation:

$$\eta_i = \eta_i(Q_i) = 775 Q_i - 2250 Q_i^2; \text{ [%]}, \quad (3)$$

with the flow rate in m^3/s . The pump's characteristic curves (2) and (3) are set in EPANET, for the hydraulic system from figure 1, with the flow rate in litres per second, the head in meters, and the efficiency in percents [13].

We will denote a pump relative speed as $r_i = n_i/n_0$, for $i \in \{1; 2; 3\}$, where n_i is the rotational speed at time t and n_0 is the nominal speed of the pump. So the relative speed depend on time t in hours, as: $r_i = r_i(t)$.

With the pipe ID-s from figure 1, we will denote by $M_s = (M_1 + M_8)$ the global hydraulic resistance modulus on main pipes $j=1$ and $j=8$, and by M_{p_i} the hydraulic resistance modulus on suction and discharge pipes connected at each pump, where $i \in \{1; 2; 3\}$, meaning: $M_{p_1} = (M_2 + M_5)$, $M_{p_2} = (M_3 + M_6)$ and $M_{p_3} = (M_4 + M_7)$. The hydraulic resistance modulus is defined for a pipe j as:

$$M_j = 0.0826 \lambda_j L_j / D_j^5, \quad \text{for } j = 1 \div 20, \quad (4)$$

where λ_j is the Darcy coefficient. The values of M_s and M_{p_i} are computed at every time step. The flow regime is turbulent and the coefficient λ_j is computed, for $Re > 4000$, using the Swamee and Jain explicit formula [12–13]:

$$\lambda_j = 0.25 \left[\lg \left(\frac{k}{3.7D_j} + \frac{5.74}{Re_j^{0.9}} \right) \right]^{-2}, \quad (5)$$

where the Reynolds number is defined as: $Re_j = 4Q_j / (\pi D_j \nu)$, and the water cinematic viscosity is $\nu \cong 10^{-6} \text{ m}^2/\text{s}$.

The energy balance can be written from the suction reservoir (with ID=1 in figure 2) to the supplying node with ID=10 in figure 2, by passing through each pump. Taking into account in (2) the similitude criteria between a pump operating at a certain speed n_i and the same pump operating at the nominal speed n_0 , the energy balance and the continuity equation form the following system:

$$\begin{cases} Q_1 + Q_2 + Q_3 = c_q Q_m \\ 83 r_1^2 - 741.6 Q_1^2 = c_h H_m + M_{p_1} Q_1^2 + M_s (Q_1 + Q_2 + Q_3)^2 \\ 83 r_2^2 - 741.6 Q_2^2 = c_h H_m + M_{p_2} Q_2^2 + M_s (Q_1 + Q_2 + Q_3)^2 \\ 83 r_3^2 - 741.60 Q_3^2 = c_h H_m + M_{p_3} Q_3^2 + M_s (Q_1 + Q_2 + Q_3)^2 \end{cases} \quad (6)$$

For $Q_m = 0.44 \text{ m}^3/\text{s}$ and $H_m = 44.16 \text{ m}$, the system (6) can be rewritten as:

$$\begin{cases} f_1 = Q_1 + Q_2 + Q_3 - 0.44 c_q = 0 \\ f_2 = 83 r_1^2 - 741.6 Q_1^2 - 44.16 c_h - M_{p1} Q_1^2 - M_s (0.44 c_q)^2 = 0 \\ f_3 = 83 r_2^2 - 741.6 Q_2^2 - 44.16 c_h - M_{p2} Q_2^2 - M_s (0.44 c_q)^2 = 0 \\ f_4 = 83 r_3^2 - 741.6 Q_3^2 - 44.16 c_h - M_{p3} Q_3^2 - M_s (0.44 c_q)^2 = 0 \end{cases} \quad (7)$$

The nonlinear system (7) has 4 equations and 6 unknowns, namely the pumped flow rate Q_i and the relative speed r_i of the pumps $i \in \{1; 2; 3\}$; we recall that the hydraulic resistance modulus in (7) depend on the unknown flow rates as: $M_{p_i} = M_{p_i}(Q_i)$ and $M_s = M_s(Q_1, Q_2, Q_3)$. To solve the system (7), we need to impose some rules for the relative speed of each pump. We will assume that the pumping station operates upon the following rules:

- Rule 1: for $c_q \leq 0.5$, the Pump1 is opened ($r_1 \neq 0$ and $Q_1 \neq 0$), while the Pump2 and Pump3 are closed ($r_2 = r_3 = 0$ and $Q_2 = Q_3 = 0$);
- Rule 2: for $0.5 < c_q < 1$, the Pump1 and Pump2 are opened ($r_1 \neq 0$; $r_2 \neq 0$ and $Q_1 \neq 0$; $Q_2 \neq 0$), while the Pump3 is closed ($r_3 = 0$ and $Q_3 = 0$);
- Rule 3: for $c_q = 1$, as stated at the pump selection, Pump1 and Pump2 are opened with $r_1 = r_2 = 1$ ($Q_1 = Q_2 \neq 0$), and Pump3 is closed ($r_3 = 0$; $Q_3 = 0$);
- Rule 4: for $c_q > 1$, all 3 pumps are opened ($r_i \neq 0$ and $Q_i \neq 0$, for $i \in \{1; 2; 3\}$).

Following the Rule 1, one get from (7) the following relative speed values for the Pump1: $r_1 = 0.848$ at $c_q = 0.36$, and $r_1 = 0.951$ at $c_q = 0.48$.

The above form of the Rules 2 and 4 are not sufficient to yield solutions, so in Sections 4 and 5, additional statements are added to the Rules 2 and 4, to compute the relative speed of each pump for $0.5 < c_q < 1$ and $c_q > 1$; finally, the nonlinear system (7) is solved in GNU Octave (using the built-in function *fsolve*), and the resulting values of $r_i = r_i(t)$ at every time step t are inserted in the EPANET model from figure 1. The Hydraulic Analysis gives the duty points of the pumps: the pumped flow rate $Q_i = Q_i(t)$ and the pump head $H_i = H_i(t)$, where $i \in \{1; 2; 3\}$. The pump efficiency $\eta_i = \eta_i(t)$ at any duty point is computed in EPANET using solely the efficiency curve, as described by (3), for any speed of the pump [12–13], equal or different from the nominal speed. To overcome this

limitation, we compute separately the pump efficiency $\eta_i = \eta_i(t)$ at each duty point applying the similitude criteria to (3), which gives the following equation:

$$\eta_i = \eta_i(Q_i) = 775(Q_i/r_i) - 2250(Q_i/r_i)^2; \text{ [\%]}. \quad (8)$$

Further, the power of each pump can be computed at every time step t , as $P_i = P_i(t)$, where $P_i = \rho g Q_i H_i / \eta_i$; the water density is $\rho = 1000 \text{ kg/m}^3$ and the gravity is $g = 9.81 \text{ m/s}^2$. Finally, knowing the power of all pumps at all time steps, we can compute the daily energy consumption, denoted E . In accordance with the energy price, which varies upon a Price Pattern [15], defined hourly by the coefficients $c_e(t)$ as in table 2, where the daily mean price is 0.065 €/kWh , we can compute the energy cost for a working day of April 2014.

4. Classical Pumping Station scheduling

As stated in Section 1, here we will set a classical pumping station operating algorithm [12], where at a certain time step t , a single pump operates at variable speed, and any other opened pump operates at constant speed, namely the nominal speed. Accordingly, the Rules 2 and 4 defined in Section 3 will be modified here as following:

- Rule 2/classic: for $0.5 < c_q \leq 1$, Pump1 is opened with $r_1 = 1$ ($Q_1 \neq 0$), Pump2 is opened with $r_2 \neq 0$ ($Q_2 \neq 0$), while Pump3 is closed ($r_3 = 0$ and $Q_3 = 0$);
- Rule 4/classic: for $c_q > 1$, Pump1 and Pump2 are opened with relative speed $r_1 = r_2 = 1$ ($Q_1 = Q_2 \neq 0$), while Pump3 is opened with $r_3 \neq 0$ ($Q_3 \neq 0$).

The computed values of the relative speed, namely $r_i = r_i(t)$ of each pump, where $i \in \{1; 2; 3\}$, are inserted in table 3. In figure 7, we present the variation of the pumped flow rate, over 24 hours, for each pump, together with the variation of the total flow rate on the main discharge pipe, labelled by $j = 8$ in figure 1.

Table 3

Relative speed of pumps, issued for the classical operating algorithm, starting from mid-night (e.g. $t = 1$ over the clock period from 00:00 to 01:00)

t [h]	1	2	3	4	5	6	7	8	9	10	11	12
r_1	0.848	0.848	0.848	0.848	0.951	1	1	1	1	1	1	1
r_2	0	0	0	0	0	0.770	1	1	1	1	1	1
r_3	0	0	0	0	0	0	0	0.941	1.077	0.941	1.077	1.077
t [h]	13	14	15	16	17	18	19	20	21	22	23	24
r_1	1	1	1	1	1	1	1	1	1	1	0.951	0.951
r_2	1	1	1	1	1	1	1	1	0.959	0.770	0	0

r_3	1.029	0.941	0.941	1.077	0.941	1.077	0.941	0.835	0	0	0	0
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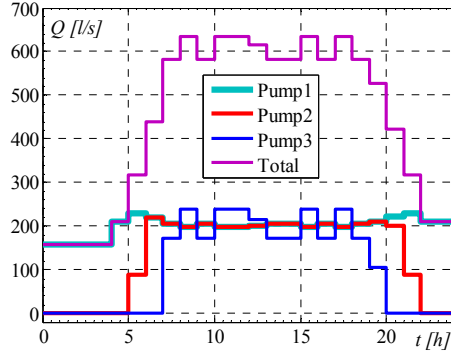


Fig. 7. Variation of the: flow rate per pump and total discharge, for the classical algorithm

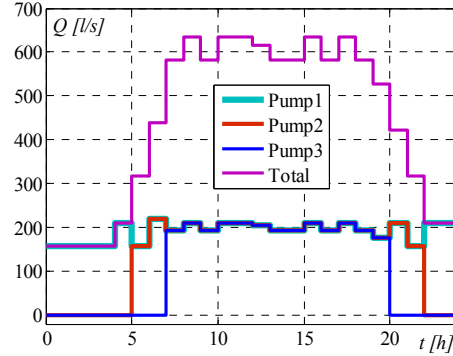


Fig. 8. Variation of the: flow rate per pump and total discharge, for the improved algorithm

5. Improved Pumping Station scheduling

Here we will set a better pumping station operating algorithm, where all opened pumps operate, at a certain time step, with equal values of the speed (not necessarily the nominal speed), ensuring a minimal total power consumption [6]. The Rules 2 and 4 defined in Section 3 will be modified as following:

- Rule 2/improved: for $0.5 < c_q \leq 1$, Pump1 and Pump2 are opened with $r_1 = r_2 \neq 0$ ($Q_1 = Q_2 \neq 0$), while Pump3 is closed ($r_3 = 0$ and $Q_3 = 0$);
- Rule 4/improved: for $c_q > 1$, all 3 pumps are opened with $r_1 = r_2 = r_3 \neq 0$ (and $Q_1 = Q_2 = Q_3 \neq 0$).

The computed values of $r_i = r_i(t)$ are inserted in table 4. The flow rate per pump and the total discharge on pipe $j = 8$ are plotted over 24 hours in figure 8.

Table 4

Relative speed of pumps, issued for the improved operating algorithm, starting from mid-night (e.g. $t = 1$ over the clock period from 00:00 to 01:00)

t [h]	1	2	3	4	5	6	7	8	9	10	11	12
r_1	0.848	0.848	0.848	0.848	0.951	0.867	1	0.979	1.025	0.979	1.025	1.025
r_2	0	0	0	0	0	0.867	1	0.979	1.025	0.979	1.025	1.025
r_3	0	0	0	0	0	0	0	0.979	1.025	0.979	1.025	1.025
t [h]	13	14	15	16	17	18	19	20	21	22	23	24
r_1	1.009	0.979	0.979	1.025	0.979	1.025	0.979	0.936	0.979	0.867	0.951	0.951
r_2	1.009	0.979	0.979	1.025	0.979	1.025	0.979	0.936	0.979	0.867	0	0
r_3	1.009	0.979	0.979	1.025	0.979	1.025	0.979	0.936	0	0	0	0

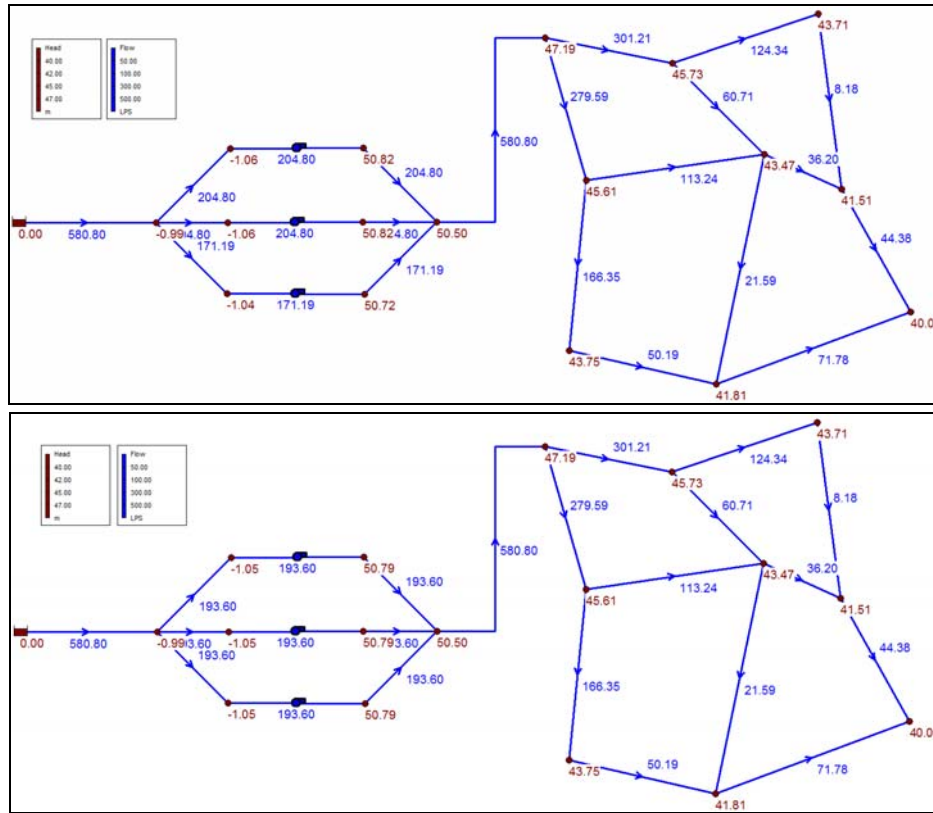


Fig. 9. Flow rate distribution for $c_q = 1.32$: classical algorithm (upper), improved-one (lower)

At the same time step, the values of the total discharge are the same for both algorithms (figures 7 and 8). The flow rate distribution on the hydraulic system's pipes is presented in figure 9, for both classical algorithm and improved-one, at a time step where the demand pattern coefficient is $c_q = 1.32$.

6. Discussions and Conclusion

For the classical pumping station operating algorithm, where one pump works at variable speed, and any other opened pump works at its nominal speed (table 3), the computed daily energy consumption is $E \cong 8186$ kWh; for the considered energy price pattern, the energy cost is about 560 EUR/day. For the improved algorithm, where all opened pumps operate, at a time step, with equal values of the speed (table 4), the computed daily energy is $E \cong 8084$ kWh, which yields 102 kWh energy-saving; the attached energy cost is about 553 EUR/day.

The above comparison shows that the algorithm proposed in Section 5 is better than the classical-one, since it reduces the energy consumption and the cost.

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