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CHLORINE CONCENTRATION DECAY IN THE WATER DISTRIBUTION SYSTEM OF A TOWN WITH 50000 INHABITANTS

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A numerical model of a water distribution network designed for a town with 50,000 inhabitants was implemented in EPANET. The paper presents a methodology for computing the chlorine residual concentration decay in the above urban size water distribution system, over a 3 days period of time. The hydraulic system fits a gravity distribution scheme, consisting of a looped network with a tank, 42 nodes and 78 pipes. Chlorine is injected at the tank, with a constant concentration of 0.45 mg/l. A variable water demand over a 24 hours period is implemented with a one hour flow pattern time step. The hydraulic time step is set to 1 minute, while the water quality time step is set to 0.02 minute, to ensure predicting accurate instantaneous chlorine concentrations. Hydraulic and Water Quality analysis is performed in order to obtain the time dependent flow rate within the network, as well as the time dependent rate of reaction and chlorine residual concentration on pipes. Reactions occurring in the bulk flow, as well as pipe wall reactions are modelled with first-order decay laws, where the instantaneous rates of reaction depend on chlorine concentration. The bulk flow decay coefficient value is set to 0.85 day^{-1}. The global wall coefficient values computed on each pipe for the average daily water consumption were adopted over the whole daily water consumption pattern; those values range from 0.013 to 0.057 m/day. All reaction coefficients are inserted as negative values within EPANET, to model the disinfectant concentration decay. At peak consumption hours, the chlorine has not enough time to react while transiting the pipes. At off-peak hours, the chlorine concentration decaying process is more pronounced; but the concentration does not decrease below 0.34 mg/l.

Keywords: Chlorine, water distribution systems, EPANET, water quality.

1. Introduction

Modelling a water distribution system allows predicting the effects of operational and physical changes on both Hydraulic and Water Quality parameters. Hydraulic parameters such as flow rates and heads are commonly monitored via telemetry. Water quality parameters however, are commonly sampled monthly or annually and only at specific nodes in the water distribution system. A computer model of a water distribution system can thus reveal a great

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deal of information regarding water quality characteristics. EPANET is a widely used simulation software for Hydraulic analysis and Water Quality analysis in water distribution systems [1].

Water Quality is a term that incorporates a large number of aspects of both potable and non-potable water. In this paper, the definition of water quality is limited to disinfectant residuals, namely to Chlorine residuals; water quality is indicated by the disinfectant concentration throughout the water distribution system. Chlorine is the most commonly used disinfectant. The decay of chlorine within a water distribution system can be attributed to its reaction with organic compounds in the source water and with biofilms on pipe surfaces. The rate of decay of chlorine is often described by a first or second order reaction [2].

There are several methods used for modelling water quality in water distribution systems. The effects of turbulent diffusion are not represented in most water quality models. A comparison of two Eulerian and two Lagrangian methods conducted by Rossman and Boulos [3] showed that a Lagrangian time-driven method is the most efficient and accurate. EPANET utilises such a method in its water quality simulator [1]. A Lagrangian time-driven method requires the selection of a time step, in order to separate consecutive instantaneous water quality simulations. The water quality time step is typically much shorter than the hydraulic time step (e.g. minutes rather than hours), to accommodate the short times of travel that can occur within pipes. The selection of the water quality time step can have a significant effect on the accuracy of the model, as shown by the sensitivity analysis conducted by Kazantzis [2].

The present study investigates the evolution of chlorine residual concentration in the water distribution network of a town with 50,000 inhabitants: a hydraulic network consisting of a tank, 36 loops, 42 junctions and 78 pipes. That network has been designed by Georgescu [4], for an average daily water consumption, using a Honey Bees Mating Optimization Algorithm (HBMOA). Within this paper, a variable water demand is considered over a 24 hours period of time, upon a flow time pattern with a one hour hydraulic time step. Hydraulic and Water Quality analysis is performed over a 72 hours period of time.

2. Water Quality simulation model

The governing equations for Water Quality analysis are based on the principles of conservation of mass, coupled with reaction kinetics. While a substance (chlorine) moves down a pipe, it can undergo reaction with constituents in the water column (bulk flow reaction). While flowing through pipes, dissolved substances can be transported to the pipe wall and react with material, such as corrosion products or biofilm that are on the wall, or close to the wall; the amount of wall area available for reaction and the rate of mass transfer between the bulk fluid and the pipe wall will also influence the overall rate of this reaction (pipe wall reaction). For the present study, the following phenomena are represented [3; 5]: advective transport in pipes, bulk flow reactions and pipe wall reactions, mixing at pipe junctions and mixing in tank.

The dissolved chlorine will travel down the length L_j of a pipe j with the same average velocity $4Q_j\pi^{-1}D_j^{-2}$ as the carrier fluid (water), while at the same time reacting (decaying) at some given rate; Q_j is the flow rate and D_j is the diameter of the pipe j. The mass transport of a single chemical, herein described for chlorine decay, is the one-dimensional advection-dispersion-reaction equation; on a circular pipe j, it is represented by:

$$\frac{\partial C_j}{\partial t} = D\frac{\partial^2 C_j}{\partial x^2} - \frac{4Q_j}{\pi D_j^2}\frac{\partial C_j}{\partial x} - r_j\left(C_j\right) \tag{1}$$

where $C_j = C_j(x,t)$ is the chlorine residual concentration (mass/volume; usually in mg/l), as a function of distance $x \in [0; L_j]$ and time $t > 0$, D is the coefficient of longitudinal dispersion (length^2/time) and r_j is the rate of reaction on pipe j (mass/volume/time) as a function of concentration; the value of r_j is positive for the decaying case. Due to the general hydraulic nature of systems under turbulent flow conditions, in the water quality models of water distribution systems, the flow characteristics are generally considered steady and uniform, and the advection is assumed to be the main mass transport mechanism [6], so $D = 0$. Initial conditions of (1) are: $C_j(x,0) = 0$ and $\partial C_j(x,0)/\partial t = 0$ on all pipes. For any time $t \geq 0$, a constant concentration of chlorine (0.45 mg/l in this paper) is injected upstream, at the tank; for the set of pipes j directly connected to the tank, $C_j(0,t) = 0.45\,\text{mg/l}$ (this is the case of $j \in \{1; 2; 58\}$; see Section 3).

The rate of reaction is the sum between the rate of reaction in bulk flow and the rate of pipe wall reaction [5; 6], as:

$$r_j = k_b C_j^n + \frac{4k_w k_{f_j}}{D_j(k_w + k_{f_j})} C_j = K_j C_j \Big|_{\text{for } n=1} \tag{2}$$

where n is the reaction order, and K_j is the concentration decay rate coefficient for first-order decay reaction ($n = 1$). The closed-form solution of equation (1) is:

$$C_j\left(x+\left(4Q_j \big/ \pi D_j^2\right)\Delta t, t+\Delta t\right)=C_j\left(x,t\right)\mathrm{e}^{-K_j \Delta t} \tag{3}$$

where Δt is the water quality time step.

The rate of reaction in bulk flow on pipe j (first right-hand term of (2)) is described as a power function of concentration, where k_b is the bulk flow decay constant (time^{-1}). We will adopt the value $k_b = 9.84 \cdot 10^{-6}\,\mathrm{s}^{-1} = 0.85\,\mathrm{day}^{-1}$. Within EPANET, the bulk flow decay constant must be set negative, in day^{-1}, meaning: $k_b = -0.85\,\mathrm{day}^{-1}$, as in Dandy et al. [7]; in Georgescu et al. [8], computations were performed using $k_b = -1\,\mathrm{day}^{-1}$, as recommended by Rossman [1]. The expression of the rate of pipe wall reaction (second right-hand term of (2)) corresponds to first-order kinetics, and contains both wall decay constant k_w (length/time) and mass transfer coefficient k_{f_j} (length/time) on pipe j. In this paper, the wall decay constant will be considered $k_w = 3.53 \cdot 10^{-6}$ m/s $= 0.3$ m/day as in Axworthy et al. [5]. The mass transfer coefficient depends on the molecular diffusivity d of the reactive species (for chlorine in water, it is $d = 1.21 \cdot 10^{-9}$ m^2/s [5]) and on the Reynolds number of the flow: $Re_j = 4Q_j \big/ (\pi D_j \upsilon)$, where the cinematic viscosity of water is $\upsilon = 10^{-6}\,\mathrm{m}^2$/s at 20°C; for turbulent flow, the following empirical correlation can be used [1, page 197]:

$$k_{f_j} = 0.0149 \frac{d}{D_j}\left(\frac{\upsilon}{d}\right)^{1/3} Re_j^{0.88} \tag{4}$$

For the average daily water consumption, the pipe diameters and flow rate distribution reported by Georgescu [4, Table 2], for the same hydraulic system, allow to compute the mass transfer coefficient values; they range from $1.52 \cdot 10^{-7}$ m/s $= 0.013$ m/day, to $8.03 \cdot 10^{-7}$ m/s $= 0.069$ m/day. The values of the global wall coefficient, denoted as $k_{gw_j} = k_w k_{f_j} (k_w + k_{f_j})^{-1}$, range from $1.45 \cdot 10^{-7}$ m/s $\cong 0.013$ m/day, to $6.54 \cdot 10^{-7}$ m/s $= 0.057$ m/day. The values of the global wall coefficient computed on each pipe j, for the average daily water consumption, will be adopted in this paper over the whole daily water consumption pattern. Within EPANET, the global wall coefficient must be set as negative, in m/day, as Property Value of each pipe (due to variable values, it must not be defined within the Reactions Options, as the bulk flow decay constant).

At nodes receiving inflow from two or more pipes, the mixing of fluid is taken to be complete and instantaneous. Thus the concentration of the chlorine residual in water leaving the junction is simply the flow-weighted sum of the concentrations from the inflowing pipes. In this paper, there are no external source flows entering the network nodes. So, for a specific node k one can write:

$$C_k = \sum_{j \in I_k} Q_j C_j\big|_{x=L_j} \Big/ \sum_{j \in I_k} Q_j \tag{5}$$

where C_k is the concentration at the start $(x=0)$ of a pipe leaving node k, I_k is the set of pipes j with flow into node k, and $C_j\big|_{x=L_j}$ is the concentration at the end of the pipe j.

EPANET can use four different types of models to characterize mixing within storage tanks [1]. We will use the Complete Mixing model, which assumes that all water that enters a tank is instantaneously and completely mixed with the water already in the tank. It is convenient to assume that the contents of storage facilities are completely mixed, not only for the fact that the complete mixing is the simplest form of mixing behaviour to assume, but also because this is a reasonable assumption for many storage facilities operating under fill-and-draw conditions.

A detailed parametric investigation of EPANET’s Water Quality analysis module has been provided by Kazantzis [2].The effect of water quality time-step on the predicted water quality concentration produced some surprising results. Large discrepancies in the instantaneous prediction of chlorine concentration were observed while varying the water quality time step, with all other parameters held constant. These results draw attention to the limitations of the water quality simulation packages to predict accurate instantaneous disinfectant concentrations. The predicted final chlorine concentration (after 24 hours of simulation) varied dramatically and chaotically for various values of the water quality time-step. A time step of one second (the smallest and presumably the most accurate time-step) predicted a concentration of 0.9 mg/l, whereas the majority of predictions for other time steps were between 0.2 and 0.5 mg/l. Time steps over 20 minutes predicted progressively lower chlorine concentrations. Nodes at the extremities of a hydraulic network display an important variation in the predicted chlorine concentration upon the water quality time step. According to those specifications, in this paper the hydraulic time step is set to 1 minute, while the water quality time step is set to 0.02 minutes.

3. Water distribution network description

The studied water distribution network corresponds to a town with 50,000 inhabitants; that hydraulic network is flat and consists of a tank, 36 loops, 42 nodes and 78 pipes labelled by $j = 1 \div 78$; a minimum 19.5 m head is requested at each node. That network has been designed by Georgescu [4], for an average daily water consumption, using a Honey Bees Mating Optimization Algorithm. All data related to geometric and hydraulic parameters defining that network can be found in Georgescu [4; figures 1 and 2, table 2]. We recall only that for the average daily water consumption, there is an input of 39.5 m head and 249 l/s flow rate at the tank, ensuring a base demand from 5 to 23 l/s in 29 specific nodes. Within this paper, a variable water demand is considered over a 24 hours period of time, upon a flow time pattern with one hour time step, using the pattern coefficients $c(t)$ from Table 1, for each time t, starting at midnight (those coefficients are multiplying the input flow and all base demand values reported for the average daily water consumption). According to the flow pattern, the input head at the tank varies from 22.5 m at off-peak hours, to 63.5 m at peak hours.

Table 1

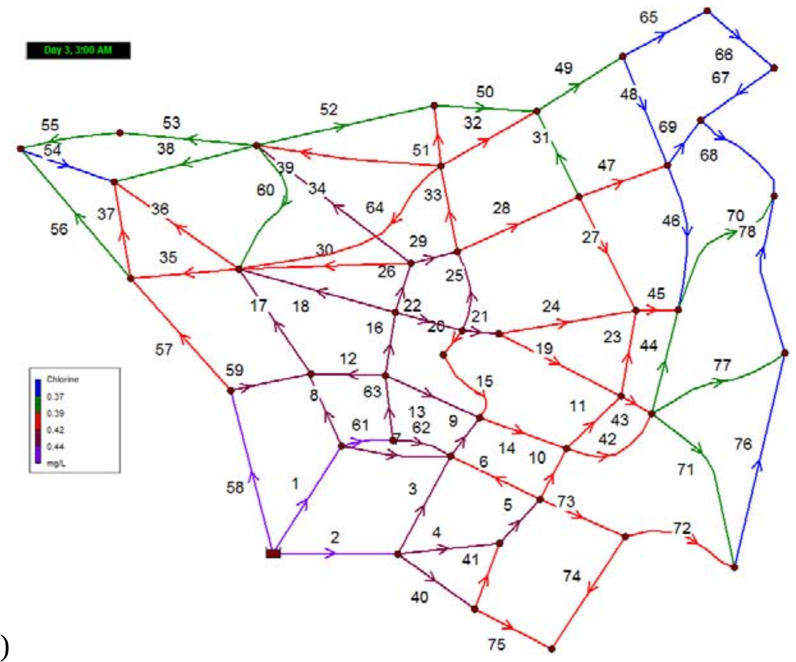
Coefficients of demand pattern *c*(*t*), at time *t* (in hours a.m.)

t	0	1	2	3	4	5	6	7	8	9	10	11
c(*t*)	0.36	0.36	0.36	0.36	0.60	0.84	**1**	1.32	1.50	1.44	1.50	1.44
t	12	13	14	15	16	17	18	19	20	21	22	23
c(*t*)	1.32	1.32	1.32	1.44	1.32	1.44	1.32	1.20	0.90	0.72	0.48	0.48

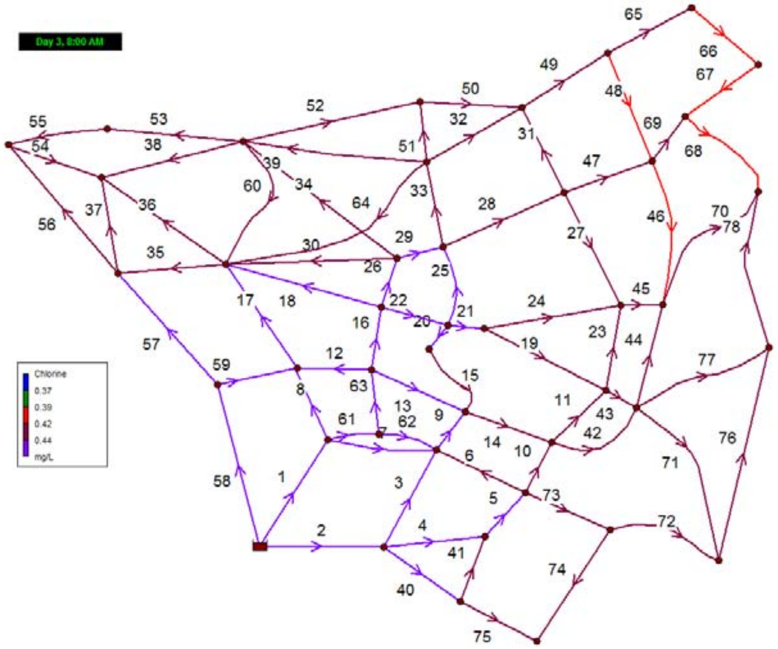
Hydraulic analysis and Water Quality analysis are performed in EPANET over a 72 hours (3 days) period of time, using a hydraulic time step of 1 minute, and a water quality time step of 0.02 minutes, as stated in Section 2.

4. Numerical results

Final data are reported only for the third day of the simulation, at three time moments, namely at 6a.m. (average daily consumption moment [4], where $c = 1$), at 3a.m. (an off-peak hour) and at 8a.m. (first peak hour). In Table 2, the following computed data are inserted for each pipe labelled by $j = 1 \div 78$: the global wall coefficient k_{gw_j} in m/day (computed for $c = 1$ and kept constant over the whole flow pattern), then the flow rate Q_j in l/s, the rate of reaction r_j (2) in $\mathrm{mg}/(\mathrm{l} \cdot \mathrm{day})$, and the chlorine concentration C_j in mg/l, at 6a.m., 3a.m. and 8a.m. Flowing directions are the same at all 3 time moments. The chlorine concentration distributions on the water network pipes at 3a.m. and 8a.m. are plotted in figure 1.



(a)



(b)

Fig. 1. Flow direction & chlorine residual concentration distribution (in mg/l) on pipes $j = 1 \div 78$, at: (a) off-peak hours, namely at 3 a.m. of day 3; (b) peak hours, namely at 8 a.m. of day 3

Table 2

Computed data at the selected representative time moments

j	$k_{gw\,j}$ [m/ /day]	t = 6a.m.			t = 3a.m.			t = 8a.m.		
		Q_j [l/s]	r_j [mg/ /l·day]	C_j [mg/l]	Q_j [l/s]	r_j [mg/ /l·day]	C_j [mg/l]	Q_j [l/s]	r_j [mg/ /l·day]	C_j [mg/l]
1	0.042	182.26	0.57	0.45	65.64	0.56	0.45	273.37	0.57	0.45
2	0.026	35.32	0.56	0.45	12.67	0.55	0.44	53.03	0.57	0.45
3	0.045	5.53	1.42	0.44	1.98	1.38	0.43	8.3	1.44	0.44
4	0.045	16.38	1.01	0.44	5.89	0.98	0.43	24.58	1.02	0.45
5	0.036	18.28	0.79	0.44	6.55	0.76	0.42	27.46	0.8	0.44
6	0.021	0.88	1.08	0.43	0.31	1.02	0.41	1.34	1.1	0.44
7	0.046	10.54	1.18	0.44	3.8	1.16	0.44	15.8	1.19	0.45
8	0.034	1.61	1.55	0.44	0.57	1.52	0.43	2.42	1.57	0.44
9	0.016	19.4	0.48	0.44	7.01	0.47	0.43	29.07	0.49	0.44
10	0.027	2.86	0.98	0.43	1.01	0.93	0.41	4.31	1	0.44
11	0.021	27.76	0.51	0.43	10.01	0.48	0.41	41.62	0.52	0.44
12	0.027	1.23	1.3	0.44	0.43	1.26	0.43	1.85	1.32	0.44
13	0.038	8.11	1.04	0.44	2.92	1	0.43	12.16	1.05	0.44
14	0.017	31.18	0.47	0.43	11.24	0.44	0.41	46.74	0.47	0.44
15	0.033	3.67	1.12	0.43	1.31	1.04	0.4	5.51	1.14	0.44
16	0.041	130.32	0.58	0.44	46.91	0.57	0.44	195.49	0.59	0.45
17	0.016	20.39	0.48	0.44	7.36	0.47	0.42	30.57	0.49	0.44
18	0.038	8.26	1.04	0.44	2.99	1	0.43	12.37	1.05	0.44
19	0.024	1.03	1.17	0.43	0.36	1.09	0.4	1.56	1.2	0.43
20	0.029	13.67	0.7	0.44	4.91	0.67	0.42	20.51	0.71	0.44
21	0.033	16.6	0.75	0.44	6.01	0.72	0.42	24.87	0.76	0.44
22	0.018	61.24	0.45	0.44	22.03	0.44	0.43	91.87	0.46	0.44
23	0.031	1.42	1.39	0.42	0.51	1.3	0.4	2.14	1.43	0.43
24	0.046	10.57	1.15	0.43	3.85	1.07	0.4	15.81	1.17	0.44
25	0.034	30.97	0.67	0.44	11.12	0.64	0.42	46.49	0.67	0.44
26	0.037	55.83	0.64	0.44	20.09	0.62	0.43	83.75	0.64	0.45
27	0.036	4.12	1.16	0.42	1.48	1.09	0.4	6.18	1.19	0.43
28	0.02	16.42	0.54	0.43	5.91	0.51	0.41	24.63	0.55	0.44
29	0.021	41.54	0.49	0.44	14.97	0.48	0.42	62.3	0.5	0.44
30	0.02	0.83	1.05	0.43	0.29	1	0.41	1.26	1.06	0.44
31	0.013	0.52	0.79	0.42	0.18	0.72	0.39	0.79	0.81	0.43
32	0.024	7.23	0.68	0.42	2.6	0.65	0.41	10.85	0.7	0.43
33	0.019	51.09	0.46	0.43	18.38	0.44	0.41	76.65	0.47	0.44
34	0.019	3.45	0.69	0.43	1.23	0.67	0.42	5.2	0.71	0.44
35	0.018	3.17	0.67	0.43	1.13	0.63	0.41	4.76	0.68	0.44
36	0.032	6.49	0.9	0.43	2.34	0.85	0.41	9.72	0.92	0.44
37	0.028	1.25	1.3	0.42	0.44	1.21	0.4	1.89	1.33	0.43
38	0.03	1.41	1.34	0.42	0.5	1.21	0.38	2.12	1.38	0.43
39	0.017	22.16	0.47	0.42	7.96	0.45	0.4	33.27	0.49	0.43
40	0.031	3.41	1.09	0.44	1.2	1.06	0.43	5.15	1.1	0.44
41	0.019	1.9	0.79	0.43	0.66	0.75	0.41	2.88	0.81	0.44

Table 2 (continuation)

42	0.014	1.27	0.68	0.43	0.44	0.62	0.4	1.93	0.69	0.43
43	0.021	27.37	0.5	0.43	9.87	0.48	0.4	41.03	0.51	0.44
44	0.025	1.12	1.17	0.41	0.4	1.05	0.38	1.7	1.21	0.43
45	0.024	11.11	0.63	0.42	4.03	0.58	0.39	16.63	0.64	0.43
46	0.023	1.01	1.06	0.4	0.35	0.96	0.36	1.52	1.12	0.42
47	0.033	6.78	0.91	0.42	2.45	0.86	0.4	10.16	0.94	0.43
48	0.025	4.93	0.74	0.4	1.76	0.66	0.36	7.43	0.77	0.42
49	0.019	15.41	0.5	0.41	5.52	0.46	0.38	23.15	0.52	0.43
50	0.021	17.65	0.53	0.42	6.34	0.49	0.39	26.5	0.55	0.43
51	0.023	19.15	0.56	0.43	6.92	0.53	0.41	28.69	0.57	0.44
52	0.013	3.51	0.52	0.42	1.22	0.48	0.38	5.31	0.54	0.43
53	0.013	15.07	0.44	0.42	5.44	0.4	0.38	22.59	0.45	0.43
54	0.02	0.85	0.99	0.41	0.31	0.89	0.37	1.27	1.03	0.42
55	0.042	15.07	0.9	0.41	5.44	0.82	0.38	22.59	0.93	0.43
56	0.019	0.77	0.99	0.42	0.26	0.91	0.39	1.18	1.01	0.43
57	0.034	3.86	1.15	0.44	1.37	1.1	0.42	5.81	1.17	0.44
58	0.034	31.42	0.68	0.44	11.33	0.67	0.44	47.1	0.68	0.45
59	0.057	22.56	1.18	0.44	8.16	1.12	0.43	33.79	1.19	0.45
60	0.016	0.62	0.87	0.41	0.21	0.78	0.37	0.94	0.9	0.43
61	0.049	165.11	0.63	0.45	59.47	0.62	0.44	247.65	0.63	0.45
62	0.052	12.45	1.29	0.45	4.52	1.26	0.44	18.64	1.3	0.45
63	0.046	152.66	0.61	0.45	54.95	0.6	0.44	229.01	0.61	0.45
64	0.024	2.55	0.9	0.42	0.9	0.85	0.4	3.84	0.92	0.43
65	0.023	10.48	0.59	0.41	3.76	0.52	0.36	15.72	0.62	0.42
66	0.028	5.48	0.79	0.4	1.96	0.69	0.36	8.22	0.82	0.42
67	0.013	0.48	0.7	0.38	0.16	0.65	0.35	0.72	0.77	0.41
68	0.026	1.18	1.1	0.38	0.42	0.97	0.34	1.79	1.19	0.41
69	0.017	0.71	0.89	0.41	0.25	0.8	0.37	1.07	0.93	0.42
70	0.028	13.24	0.66	0.41	4.78	0.6	0.38	19.85	0.68	0.43
71	0.029	5.87	0.83	0.42	2.09	0.75	0.38	8.82	0.86	0.43
72	0.024	1.05	1.16	0.42	0.37	1.07	0.39	1.59	1.18	0.43
73	0.03	14.54	0.71	0.43	5.23	0.67	0.41	21.82	0.72	0.44
74	0.019	3.49	0.68	0.42	1.26	0.63	0.4	5.23	0.69	0.43
75	0.032	1.51	1.46	0.43	0.54	1.37	0.41	2.27	1.48	0.44
76	0.019	1.92	0.75	0.4	0.66	0.67	0.36	2.91	0.78	0.42
77	0.041	21.65	0.81	0.42	7.82	0.74	0.39	32.45	0.84	0.43
78	0.015	0.57	0.82	0.4	0.2	0.73	0.36	0.86	0.86	0.42

In figure 2, we have plotted the temporal variation of chlorine residual concentration, in mg/l, on 5 pipes labelled by $j = \{30; 44; 54; 60; 68\}$. Since computations start with the initial condition $C_j(x,0) = 0$ on all pipes, there is a transit time needed by the chlorine to spread through the network. The concentration variation starts to follow a path that agrees with the flow pattern, only after the first 24 hours of the simulation; to be sure that reported data are correct, in Table 2 we reported only data attached to the 3rd day of the simulation.

The effect of water quality time-step on the predicted chlorine residual concentration, signalled by Kazantzis [2], can be seen on the zoomed frame of figure 2. Such tiny variations of the chlorine concentration upon time would be smoothed by a greater value of the water quality time step. In our computations, the water quality time step (of 0.02 minutes) is 50 times smaller than the hydraulic time step (of 1 minute), and it is 3000 times smaller than the flow pattern time step (of 60 minutes). Thus, in our opinion, the decaying process described by equation (3) is trustworthy.

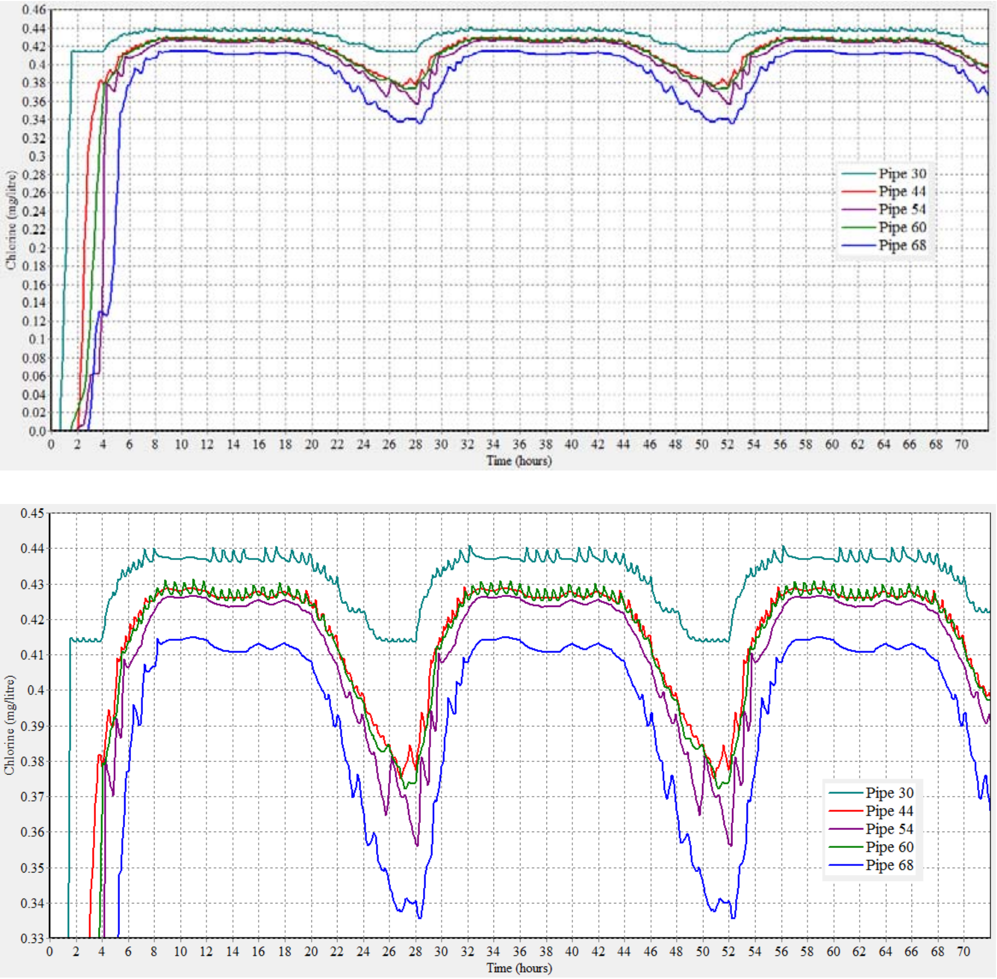


Fig. 2. Temporal variation of chlorine residual concentration (in mg/l) on 5 pipes, over the whole computational period of time (72 hours). The above pipes are labelled by $j = \{30; 44; 54; 60; 68\}$. The same variation is plotted at full scale (upper frame), and at zoomed scale (lower frame)

A chlorine residual concentration with a minimal value of 0.34 mg/l has been recorded at off-peak consumption hours on the pipe no. 68 (see figure 1.a). In Table 2, the rate of reaction values range from 0.4 to 1.57 $\mathrm{mg}/(\mathrm{l}\cdot\mathrm{day})$.

5. Conclusions

The paper presents a methodology for computing the chlorine residual concentration decay in a water distribution network designed for a town with 50,000 inhabitants (a gravity distribution scheme with a tank, 36 loops, 42 nodes and 78 pipes). Chlorine is injected at the tank, with a constant concentration of 0.45 mg/l. Reactions occurring in the bulk flow, as well as pipe wall reactions are modelled with first-order decay laws. Assuming a variable water demand over a 24 hours period, Hydraulic and Water Quality analysis is performed in EPANET, over a 3 days period of time, to obtain the time dependent flow rate, as well as the time dependent rate of reaction and chlorine residual concentration on network pipes. Final data are reported for the third day of the simulation, at three representative time moments, namely: the average daily consumption moment, an off-peak hour and a peak consumption hour.

At peak hours, when consumption is high (e.g., from morning to late afternoon), the water transits more rapidly the network pipes and the chlorine has not enough time to react; thus chlorine residual concentration values remain quite high (≥ 0.41 mg/l). At off-peak hours, when consumption is low (e.g. during the night), water velocity decreases and the chlorine concentration decaying process is more pronounced; thus chlorine residual concentration decreases down to 0.34 mg/l. For the considered water distribution network, all chlorine residual concentration values computed during the last two days of the simulation remain greater than the minimum admissible range, which is 0.1÷0.3 mg/l [10].

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