




Research Article

System-wide joint-dynamic-response approach to water quality evaluation in distribution networks with multiple service reservoirs and pumps

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Received: 24 September 2020 / Accepted: 22 February 2021 / Published online: 12 March 2021
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Abstract

Rigorous system-wide aggregated water quality performance indices for water distribution networks are lacking in the literature due to complexities associated with high dimensional spatial and temporal water quality data. Water quality considerations unavoidably increase performance evaluation difficulties considerably. The formulation developed in this article addresses the post-extended period simulation high-dimensional data challenges. A system-wide joint-dynamic-response approach to water quality evaluation is introduced that accounts for spatial and temporal variations in nodal demands and the respective time-varying hydraulic and water quality properties of multiple service reservoirs. Effective comparisons of the water quality response of service reservoirs and their effects were achieved. This includes individual reservoirs and the combined effects of multiple reservoirs. Service reservoirs and the nodes they supply were particularly vulnerable from the standpoint of water quality. The role of the network's topology considering water quality risks was revealed also. The correlation between the medians and flow-weighted daily means of the water quality parameters was very strong ($R^2 \geq 0.994$) for the service reservoirs considered. Thus, the median could be useful as a practical performance surrogate in design optimization procedures. Finally, there seems to be an association between the flow-weighted daily means and overall hydraulic effectiveness of service reservoirs.

Keywords Service reservoir · Water distribution pipe network · Water quality · Residual chlorine · Disinfection by-products · Gastrointestinal illness

1 Introduction

Service reservoirs in water distribution networks are vital, as the service providers have a duty to maintain a continuous and adequate supply of water. Service reservoirs provide extra operational flexibility to cope with variations in demand and major unexpected failures, e.g. power outages, pump failures and firefighting [8, p. 120–122]. They increase resilience, hydraulic reliability and

redundancy—characterized by component failure tolerance due to surplus flow carrying capacity and alternative supply paths between the supply and demand nodes [2, 65, 66]. Also, service reservoirs can help achieve economical designs based on the life cycle costs.

Within the distribution system, service reservoir often refers to surface or underground storage while water tower or elevated tank often refers to elevated storage [63, AWWA 2013]. In this article, tanks and service reservoirs are

Supplementary Information The online version contains supplementary material available at <https://doi.org/10.1007/s42452-021-04410-0>.

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SN Applied Sciences (2021) 3:448 | <https://doi.org/10.1007/s42452-021-04410-0>

used interchangeably in an inclusive manner for reservoirs, elevated tanks and stand pipes within the distribution system (AWWA 2013). Nevertheless, all the storage facilities in the benchmark Anytown network considered here are called tanks.

Service reservoirs influence water quality by increasing the residence time, i.e. the time of travel from the treatment plant to the node under consideration. Water quality may deteriorate as a result, due to the loss of disinfectant, increase in the concentration of disinfection by-products (DBPs) and microbial re-growth [11, 17, 19]. DBPs, e.g. trihalomethanes and haloacetic acids, are produced from the reactions of chlorine with natural organic compounds in water [17, 41] and are associated with adverse health effects [6, 21, 22, 24].

Therefore, to prevent the re-growth of bacteria and other organisms, enough residual chlorine should be retained in the distribution system [68, p. 458]. However, Tinker et al. [67] observed that a disinfectant residual is not always present in all parts of a distribution system, and cases of gastrointestinal illness have been linked to contamination of water in the distribution network in a number of studies [36, 37, 54].

Attention on the distribution system as a source of water-borne illness is, therefore, increasing [45, 75]. Payment et al. [38] reported on the contribution of the distribution system to endemic gastrointestinal disease related to drinking water. Egorov et al. [14] and Levy et al. [32] considered the association between the residence time and incidence of gastrointestinal illness. Hunter et al. [27] considered the association between events leading to pressure loss in the distribution system and incidence of gastrointestinal illness [7, 15, 29, 30]. Bylund et al. [9] studied the association between gastrointestinal illness and the factors affecting drinking water quality.

The residence time is sensitive to the location of the service reservoir and operating policies adopted [16, 20, 63, p. 139]. Edwards and Maher [13] observed that the location may influence water quality more than modifying the service reservoir to improve mixing. The conventional design practice suggests that service reservoirs should be located close to the areas that have the highest water demands [34]. However, the competing requirements are very complex.

Inefficient reservoir design may increase the capital and operating costs and residence times. Kurek and Ostfeld [31] used a multi-objective optimization approach that considered water quality, the operating costs of pumps and service reservoir sizing costs. However, the locations of the tanks were not optimized. Basile et al. [5] developed a multi-criteria approach that did not incorporate water quality or optimize service reservoir design. Farmani et al. [16] considered the total design

and operation cost and locations of the service reservoirs along with water age and system resilience. Prasad and Tanyimboh [40] similarly considered the system's resilience properties including flow entropy [52, 62], but water quality was not addressed. Siew et al. [58] introduced a novel service reservoir depletion criterion. Siting of the service reservoirs was optimized but water quality was not included in the formulation. Prasad [39] and Atkinson et al. [1] addressed the total cost including service reservoir location and sizing without considering water quality in the design optimization phase. Shokoohi et al. [53] emphasized the need to address water quality at the design stage, but their optimization model did not include service reservoirs. Hallmann and Suhl [23] did not include water quality in their service reservoir optimization model or the subsequent verification by simulation.

Studies on the deleterious effects of service reservoirs and a clear understanding of the system-wide response are lacking. Optimization models for water distribution networks with multiple service reservoirs have not addressed water quality concerns adequately, and rigorous comparisons of candidate solutions are not available [26]. A quantified robust framework [26] will help to improve: (a) investment decisions [59], (b) operational control [59], and (c) the development of water quality-based design optimization models [18, 28]. While previous investigations considered the individual service reservoirs or demand nodes in isolation, this article develops and demonstrates the benefits of a system-oriented approach to the evaluation of water quality that transcends the single-event regulatory compliance paradigm. Also, water quality considerations increase the performance evaluation complexity levels enormously. The system-wide joint-dynamic-response formulation proposed addresses the challenges associated with high-dimensional data.

Indeed, the Battle of the Water Networks II [33] described vividly the difficulties of high dimensionality and computational complexity and how they were addressed variously through: engineering experience, parallel computing; reducing the number of decision variables; reducing the range of possible values for each decision variable; skeletonizing the network; sequential optimization; zone-by-zone design; and stage-wise optimization (e.g. installing backup diesel generators as the final stage of the design process). The weighted average of the water age at the demand nodes above a threshold of 48 h was used as a practical water quality indicator. Even with simplistic or no water quality considerations the hydraulic design problem was extremely complex. This illustrates the urgent need for rigorous and practical methods of water quality evaluation for decision making and design purposes.

2 Overview of water quality modelling of distribution networks

The water quality equations combine the principles of conservation of mass and reaction kinetics for the reactants under consideration. The first order reaction kinetics approach in EPANET 2 was used herein for illustration purposes. The system of equations is solved numerically in EPANET 2 using a discrete volume element method [42]. However, without loss of generality, it may be swapped with any suitable alternative as required, depending on the emphasis of the investigation at hand, i.e. EPANET-MSX or -PMX, CFD (computational fluid dynamics), etc. The focus herein is not the water quality simulation model per se but the system-wide aggregation of the results to assist with high dimensional data challenges in decision making and design optimization models that has not been addressed hitherto.

The reactions of chlorine and THMs (trihalomethanes) are modelled as in Eqs. 1 and 2, respectively, for the bulk flow, and Eq. 3 for the pipe wall.

$$\partial C_{Cl} / \partial t = -k_b C_{Cl} \quad (1)$$

$$\partial C_{THM} / \partial t = k_b (C_L - C_{thm}) \quad (2)$$

$$\frac{\partial C}{\partial t} = \frac{2k_w k_f C}{R(k_w + k_f)} \quad (3)$$

where C_{Cl} is the chlorine concentration; k_b is the reaction rate constant in the bulk flow; t is the time; C_{THM} is the THM concentration; C_L is the limiting THM concentration; k_w is the wall reaction rate constant [Length/Time]; k_f is the radial mass transfer coefficient [Length/Time] that depends on the molecular diffusivity [Length²/Time] of the reactive species and the turbulence of the flow [42], R is the pipe radius; and C is the reactant concentration in the bulk flow.

The equation for transport in a pipe based on advection is

$$\frac{\partial C_i}{\partial t} = -u_i \frac{\partial C_i}{\partial x} + r_i \quad \forall i \quad (4)$$

where $C_i \equiv C(i, x, t)$ is the reactant concentration in pipe i at location x at time t ; $u_i \equiv u(i, t)$ is the mean flow velocity in pipe i at time t ; and $r_i \equiv r(C_i)$ is the rate of reaction. The equation assumes that longitudinal dispersion [69] is negligible.

Mixing in service reservoirs is assumed to occur under completely mixed conditions [11]. The mass balance equation for the sth service reservoir is

$$\frac{\partial (V_s C_s)}{\partial t} = \sum_{i \in I_s} (Qp_i C(i, x, t)_{x=L_i}) - \sum_{j \in O_s} Qp_j C_s + r_s; \quad \forall s \quad (5)$$

where $V_s \equiv V(s, t)$ and $C_s \equiv C(s, t)$ are, respectively, the volume in storage and reactant concentration at time t ; I_s and O_s represent the links with flow to and from the reservoir, respectively; and $r_s \equiv r(C_s)$ is the rate of reaction.

Similarly, mixing at the junctions of the pipes assumes that mixing is complete and instantaneous. The mass balance equation for the junctions is

$$\left(\sum_{j \in I_n} Qp_j + Q_e \right) C(i, x, t)_{x=0} = \sum_{j \in O_n} (Qp_j C(j, x, t)_{x=L_j}) + Q_e C_e; \quad \forall i \in O_n, \quad \forall n \quad (6)$$

where j and i , respectively, represent the pipes with flow entering and leaving node n . I_n and O_n , respectively, represent the sets of pipes with flow entering and leaving node n ; L_j is the length of pipe j ; Qp_j is the flow rate in pipe j ; Q_e and C_e are, respectively, the volumetric flow rate and reactant concentration of any external flow into the network, at node n .

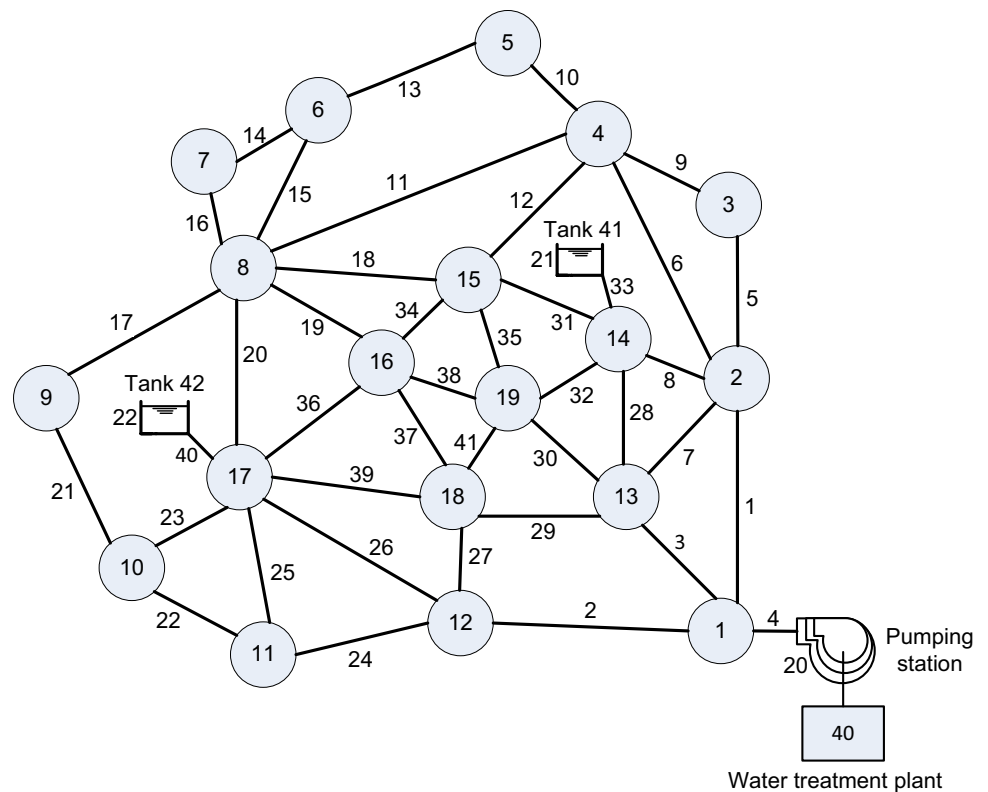
3 Upgrading requirements and the networks investigated

3.1 Network upgrading requirements considered

The 'Anytown' network shown in Fig. 1 [73] was considered for illustration purposes. Table A1 in the Supplementary Materials shows the nodal demands. The main objective in the network's specifications was to provide the most economical design. The options included cleaning and lining the existing pipes, laying new pipes, and paralleling existing pipes. Up to two new service reservoirs could be added. Each node in the network could be considered a potential location for a new reservoir, except for the nodes with the existing reservoirs. Three identical pumps in parallel supply water from the treatment plant; up to two more could be added. Two existing tanks, called Tank 41E and 42E herein, are at node 14 and 17, respectively.

All tanks were to empty and refill during the daily operating cycle. The demands included the average-day, instantaneous peak and three fire-fighting flows. The minimum residual heads at the demand nodes for the average-day, instantaneous peak and fire-fighting flows were 28.12 m, 28.12 m and 14.06 m, respectively. The instantaneous peak and two-hour fire-fighting flows would be met with the tanks operating at their low levels and one pump out of service.

Fig. 1 Network topology. Existing tanks 41 and 42 are called 41E and 42E herein. Identifiers of nodes and pipes are as indicated



3.2 Brief description of the networks assessed

Four of the best solutions in the literature for the Anytown network were assessed (Table A2 in the Supplementary Materials). Walters et al.'s [74] solution costs \$10.91 million. It has two new tanks: Tank 5 N at node 5 and Tank 12 N at node 12. Walters et al. [74] surmised that a more expensive alternative solution was preferable based on water quality considerations. Only the solution with a total cost of \$10.91 million was considered here. Prasad's [39] solution costs \$10.59 million. It has two new tanks: Tank 170 N at node 9 and Tank 150 N at node 16.

Solution 1 has a total cost of \$10.31 million, with a new tank at node 7 (Tank 7 N). Solution 2 has a total cost of \$10.41 million, with a new tank at node 6 (Tank 6 N). The new tanks in Solution 1 and 2 are identical except for their locations and emergency storage fractions [58]. The emergency storage fraction is the ratio of the emergency storage volume to the total reservoir capacity. The details of the pipes and new tanks are in Table A3 and A4, respectively, in the Supplementary Materials.

Walters et al.'s solution utilizes only two pumps that operate continuously for 24 h. Prasad's solution utilizes three pumps two of which operate continuously for 24 h while the third operates for 12 h from 06:00 a.m. to 18:00 p.m. Solution 1 utilizes three pumps, with two operating continuously for 24 h while the third operates for nine hours from

09:00 a.m. to 18:00 p.m. Solution 2 utilizes three pumps, with two operating continuously for 24 h while the third operates for nine hours from 06:00 a.m. to 03:00 p.m.

Among the above-mentioned solutions, Siew et al.'s [58] approach (Fig. 2) stands out in the literature as it achieved numerous previously unidentified least-cost designs for the Anytown and other benchmark problems [57]. Moreover, it has been shown that, computationally, it is highly competitive. Thus, Solution 1 and 2 and Prasad [39] are the best in the literature, based on the hydraulic requirements and cost. In other words, they are the most competitive feasible solutions in terms of the total cost. The maximum difference in cost between these solutions is *only* 2.6% and, to date, besides the total cost, it has proved particularly challenging to compare and rate them in a convincing way from the perspective of water quality. On the other hand, Walters et al. [74] is ranked fifth in the literature in terms of the total cost (Table A2) (Supplementary Materials). This solution was chosen because it was thought to be slightly less competitive, compared to the three best solutions. Accordingly, it was thought that Walters et al. [74] had the potential to reveal any inconsistencies in the results achieved, relative to Prasad and Solution 1 and 2.

Also, the total number of new tanks added and their locations were considered. Two solutions have two new tanks (Prasad [39] and Walters et al. [74]) and two solutions

have one new tank (Solution 1 and 2). The aim was to avoid an imbalance in the total number of tanks in each solution.

Furthermore, Vamvakieridou-Lyroudia et al. [72] whose total cost is \$10.63 million is the fourth most economical solution in the literature. It has two new tanks whose locations are identical to Prasad [39]. Accordingly, Vamvakieridou-Lyroudia et al. [72] was not selected, as Prasad [39] is less expensive. The difference in cost between these two solutions is also the smallest, in percentage and absolute terms, among the five best solutions in the literature, i.e. \$0.04 million.

3.3 Adopted water quality simulation parameters

Based on Seyoum et al. [49] and Carrico and Singer [10], the bulk and wall reaction rate constants were taken as $k_b = 0.5/\text{day}$ and $k_w = 0.1 \text{ m/day}$, respectively. The required minimum chlorine concentration at the demand nodes was 0.2 mg/L [76]. Based on Seyoum et al. [49] a constant chlorine concentration of 0.6 mg/L was set at the treatment plant. The EU and UK drinking water standards specify a maximum THM concentration of 100 $\mu\text{g/L}$ (EC 1998, HMG 2010) while the maximum concentration in the US EPA (1996) standard is 80 $\mu\text{g/L}$. Accordingly, following Seyoum et al. [49], the limiting concentration of THM in Eq. 2 was taken as 100 $\mu\text{g/L}$. An initial concentration of zero was assumed for THM at all the demand nodes and tanks. The hydraulic and water quality time steps were one minute each. The small hydraulic time step of one minute was selected to provide confirmation that the solutions are hydraulically feasible theoretically. The duration of the extended period simulation (EPS) was 3.0 days. The results presented here are based on the last 24 h. This is to avoid any unstable results near the start of the simulations.

3.4 Evaluation of water quality sub-indices based on flow-weighted arithmetic mean

Given the spatial and diurnal variations in the nodal demands and water quality, sub-indices for the quality of water at a location (demand node or service reservoir) and the entire network for a specified duration are useful, e.g. as part of a decision support system. Accordingly, aggregation based on the flow-weighted arithmetic mean was considered herein.

For the n th demand node and s th service reservoir, the overall water quality sub-indices, I_n and I_s respectively, based on the flow-weighted arithmetic mean were obtained as follows.

$$I_n = \sum_{t=1}^{N_t} w(n,t)C(n,t); \quad w(n,t) = \frac{Q(n,t)}{\sum_{t=1}^{N_t} Q(n,t)}; \quad (7a)$$

$$\sum_{t=1}^{N_t} w(n,t) = 1; \quad \forall n \in D$$

$$I_s = \sum_{t=1}^{N_t} w(s,t)C(s,t); \quad w(s,t) = \frac{V(s,t)}{\sum_{t=1}^{N_t} V(s,t)}; \quad (7b)$$

$$\sum_{t=1}^{N_t} w(s,t) = 1; \quad \forall s \in S$$

where the sets D and S comprise the demand nodes and service reservoirs, respectively, and n , s and t refer to the demand nodes, service reservoirs and time, respectively; $w(n,t)$ and $w(s,t)$ represent the weights for the demand nodes and service reservoirs, respectively; $C(n,t)$ and $C(s,t)$ represent water quality properties, e.g. water age, at the demand nodes and service reservoirs, respectively. N_t represents the number of observations. Thus, for simplicity, t refers to the observation number and corresponding time due to the one-to-one mapping. $Q(n,t)$ and $V(s,t)$ represent the nodal volume flow rates and volumes of water in the service reservoirs, respectively.

The corresponding formulation for all the demand nodes in aggregate follows similarly.

$$I_D = \sum_{n \in D} \sum_{t=1}^{N_t} w(n,t)C(n,t); \quad w(n,t) = \frac{Q(n,t)}{\sum_{n \in D} \sum_{t=1}^{N_t} Q(n,t)}; \quad (8a)$$

$$\sum_{n \in D} \sum_{t=1}^{N_t} w(n,t) = 1$$

where I_D is the water quality sub-index for all the demand nodes taken together.

Also, the sub-index I_R for all the service reservoirs in aggregate is

$$I_R = \sum_{s \in S} \sum_{t=1}^{N_t} w(s,t)C(s,t); \quad w(s,t) = \frac{V(s,t)}{\sum_{s \in S} \sum_{t=1}^{N_t} V(s,t)}; \quad (8b)$$

$$\sum_{s \in S} \sum_{t=1}^{N_t} w(s,t) = 1$$

It is worth noting that the demand nodes and service reservoirs were considered separately. Thus, the set D that represents the demand nodes does not include the service reservoirs.

Table 1 Water quality results of demand nodes

	Worst instance											
	Solution 1			Solution 2			Prasad (2010)			Walters et al. (1999)		
	Value	Time	Node	Value	Time	Node	Value	Time	Node	Value	Time	Node
Chlorine—min (mg/L)	0.22	0600	7	0.31	2300	9	0.25	0400	9	0.18	0400	5
THM—max (µg/L)	49.8	0600	7	28.6	1100	6	36.8	0300	16	51.7	0400	5
Water age—max (hours)	30.8	0600	7	19.4	1100	6	25.2	0300	16	34.2	0400	5
	Flow weighted daily mean											
	Solution 1			Solution 2			Prasad (2010)			Walters et al. (1999)		
	Value	Time	Node	Value	Time	Node	Value	Time	Node	Value	Time	Node
Chlorine (mg/L)	0.48			0.48			0.49			0.49		
THM (µg/L) ^a	8.53			8.48 ^a			8.10 ^a			7.97 ^a		
Water age (hours)	4.17			4.16			3.81			3.84		

^aSome tanks in the solutions indicated did not have a continuous or sufficient supply of chlorine throughout the 24-h operating cycle

4 Results

Water age, chlorine and THMs were simulated for the networks in Sect. 3.2. Tables 1, 2 and 3 show the results for the demand nodes, existing and new tanks, respectively. Table 4 shows the aggregated performance of the tanks in each network, based on water quality. The results include the worst instances, medians, flow-weighted daily means and correlations between the medians and flow-weighted daily means. The average durations of the 72-h simulations in EPANET 2 were 2.6 s for water age, 4.6 s for residual chlorine and 4.9 s for THMs on a laptop (Intel Core TM i5-2430 M, 2.4 GHz CPU, 8.0 GB RAM). The results obtained are discussed for the new and existing tanks plus demand nodes in turn. To add some perspective, a survey of US water utilities reported average and maximum water ages in distribution networks of 1.3 days (31.2 h) and 3.0 days (72 h), respectively (AWWA and AwwaRF 1992).

4.1 Water quality in the new tanks

4.1.1 Water age

The flow-weighted water ages for the new tanks in Solution 1, 2 and Prasad [39], i.e. Tank 150 N and Tank 170 N, were 31.4, 31.5, 37.2 and 45.9 h, respectively. The corresponding maximum water ages were 37.5, 39.3, 46.5 and 53.5 h. These results show that the new tanks in Solution 1 and 2 outperformed the new tanks in Prasad (Fig. 3). The

maximum water age in Solution 2 was probably greater than Solution 1 because Solution 2 has a larger emergency storage fraction (Table A3) (Supplementary Materials). Figure 3 also shows that the maximum water ages in Solution 1 and 2 were lower than in Walters et al. [74], i.e. 42.2 h in Tank 5 N.

4.1.2 Residual chlorine

The lowest chlorine concentrations were 0.23 mg/L for Solution 1 (14:00–18:00) and 0.22 mg/L for Solution 2 (14:00–15:00). The lowest values in Prasad [39] were mostly below 0.2 mg/L. In Walters et al. 58% of the concentrations in Tank 5 N were below 0.2 mg/L.

4.1.3 Disinfection by-products

The flow weighted THM concentrations of Solution 1, 2 and Prasad (Tank 150 N and 170 N) were 43.8, 43.8, 50.5 and 59.2 µg/L, respectively. In Walters et al. the values were 49.8 and 28.3 µg/L in Tank 5 N and 12 N, respectively. Tank 12 N is relatively close to the pumping station and water treatment works, while Tank 5 N is at the opposite end of the network. This explains the large difference between the THM concentrations in favour of Tank 12 N.

Table 2 Water quality results of existing tanks

	Solution 1			Solution 2			Prasad (2010)			Walters et al. (1999)						
	Tank 41E		Tank 42E	Tank 41E		Tank 42E	Tank 41E		Tank 42E	Tank 41E		Tank 42E				
	amt	hr	amt	hr	amt	hr	amt	hr	amt	hr	amt	hr				
<i>Chlorine (mg/L)</i>																
Minimum	0.34	09:00–15:00	0.32	09:00	0.00	20:00	0.32	12:00	0.35	09:00	0.33	09:00	0.00	24 h	0.00	24 h
Median	0.38	-	0.34	-	0.39	-	0.37	-	0.42	-	0.37	-	0.00	-	0.00	-
Flow weighted	0.40	-	0.35	-	0.39	-	0.38	-	0.42	-	0.38	-	0.00	-	0.00	-
<i>THM (µg/L)</i>																
Maximum	35.3	15:00	39.8	15:00	41.8	20:00	41.7	12:00	33.8	09:00	39.0	09:00	77.7	24:00	77.7	24:00
Median	28.5	-	37.3	-	26.0	-	34.2	-	21.2	-	32.1	-	71.7	-	71.7	-
Flow weighted	23.9	-	36.2	-	21.1	-	31.6	-	20.7	-	31.6	-	71.4	-	71.4	-
<i>Water age (hours)</i>																
Maximum	22.2	15:00	27.7	15:00	25.7	20:00	27.9	12:00	20.4	09:00	25.8	13:00	72.0	24:00	72.0	24:00
Median	17.4	-	25.2	-	14.2	-	22.7	-	12.2	-	21.3	-	60.5	-	60.5	-
Flow weighted	14.2	-	24.8	-	11.7	-	20.8	-	11.8	-	20.7	-	60.5	-	60.5	-
<i>Linear correlation of median vs. flow weighted daily mean</i>																
Parameter												Water Age				
R ²												0.998				
												THM				
												0.994				

Table 3 Water quality results of new tanks

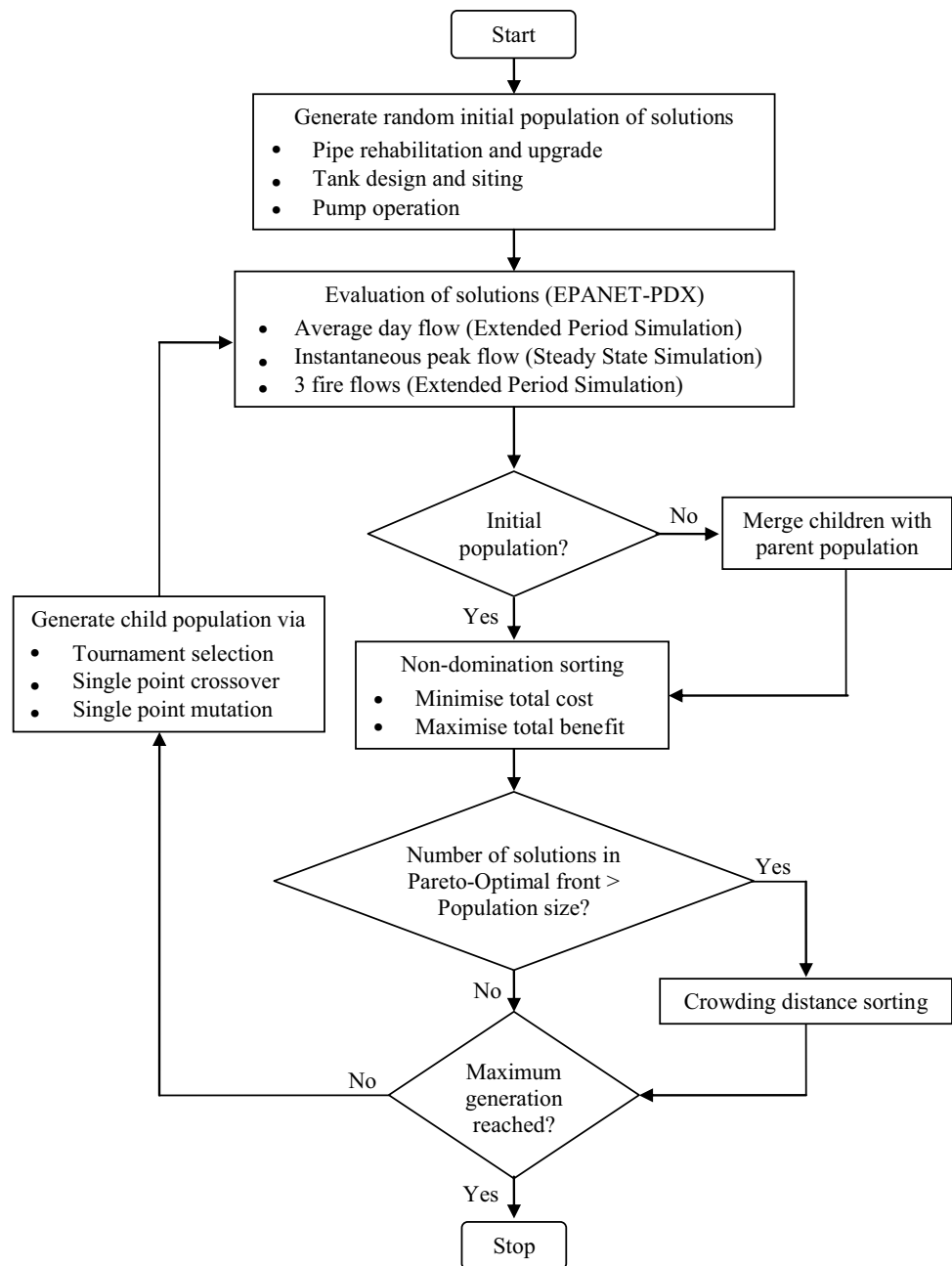
	Solution 1		Solution 2		Prasad (2010)				Walters et al. (1999)			
	Tank 7 N		Tank 6 N		Tank 150 N		Tank 170 N		Tank 5 N		Tank 12 N	
	Value	Time	Value	Time	Value	Time	Value	Time	Value	Time	Value	Time
<i>Chlorine (mg/L)</i>												
Minimum	0.23	14:00–18:00	0.22	14:00–15:00	0.15	18:00	0.09	17:00	0.16	14:00	0.31	15:00
Median	0.25	–	0.26	–	0.19	–	0.11	–	0.19	–	0.38	–
Flow weighted	0.26	–	0.27	–	0.20	–	0.11	–	0.20	–	0.41	–
<i>THM (µg/L)</i>												
Maximum	50.7	16:00	53.2	15:00	60.1	18:00	65.5	17:00	56.4	15:00	43.7	15:00
Median	45.3	–	45.2	–	51.3	–	59.3	–	50.7	–	32.7	–
Flow weighted	43.8	–	43.8	–	50.5	–	59.2	–	49.8	–	28.3	–
<i>Water age (hours)</i>												
Maximum	37.5	15:00	39.3	18:00	46.5	18:00	53.5	17:00	42.2	15:00	29.3	15:00
Median	32.3	–	32.1	–	37.1	–	46.9	–	36.2	–	20.8	–
Flow weighted	31.4	–	31.5	–	37.2	–	45.9	–	36.1	–	18.5	–
<i>Linear correlation of median vs. flow weighted daily mean</i>												
Parameter	Chlorine				THM				Water Age			
R^2	0.999				0.998				0.994			

Table 4 Aggregated water quality results of all tanks

Solutions	Criteria	Flow weighted daily means for the tanks individually and collectively ^a				
		Tank 41E	Tank 41E	Tank 7 N	Network ^a	
Solution 1	Water age (hours)	14.23	24.78	31.39	28.00	
	Chlorine (mg/L)	0.4	0.35	0.26	0.30	
	THM (µg/L)	23.88	36.18	43.78	39.86	
Solution 2	Water age (hours)	11.74	20.75	31.5	27.53	
	Chlorine (mg/L)	0.39	0.38	0.27	0.30	
	THM (µg/L)	21.10	31.56	43.81	39.26	
Walters et al.(1999)	Water age (hours)	60.5	60.5	36.06	18.42	43.11
	Chlorine (mg/L)	0	0	0.20	0.41	0.15
	THM (µg/L)	71.36	71.36	49.79	28.34	55.14
Prasad (2010)	Water age (hours)	11.17	20.65	38.08	45.89	31.37
	Chlorine (mg/L)	0.42	0.38	0.20	0.11	0.26
	THM (µg/L)	20.66	31.58	50.45	59.18	43.02

^aThe tank identifiers show the results for individual tanks while “Network” refers to the tanks in aggregate

Fig. 2 Penalty-free multi-objective genetic algorithm for Solution 1 and 2 [55]. EPANET-PDX (pressure-dependent extension) [56] is embedded in the genetic algorithm for fitness evaluation. EPANET-PDX simulates solutions using pressure-driven analysis that accounts realistically for any deficiencies in residual pressures at demand nodes



4.2 Water quality in the existing tanks

4.2.1 Water age

The respective flow-weighted water ages of Solution 1, 2, Prasad and Walters et al. for Tank 41E and 42E, respectively, were: (14.2, 24.8), (11.7, 20.8), (11.8, 20.7) and (60.5, 60.5) hours. The respective maximum water ages of Solution 1, 2, Prasad and Walters et al. for Tank 41E and 42E, respectively, were: (22.2, 27.7), (25.7, 27.9), (20.4, 25.8) and (72.0, 72.0) hours. These results indicate that Prasad's existing tanks had the best performance.

4.2.2 Residual chlorine

Figure 4 shows that the chlorine concentrations of Tank 41E in Solution 2 (19:00 and 20:00) and both existing tanks in Walters et al. (00:00 to 24:00) were unsatisfactory, i.e. $C_{Cl} < 0.2$ mg/L. The rest of the concentrations were satisfactory. This result for Solution 2 contradicts Siew et al. [58] and Seyoum et al. [49] who did not investigate Tank 41E in detail. They focused only on Tank 42E that, hitherto, had been considered particularly challenging [39, 72] and for which Solution 2 is better as Table 2 shows.

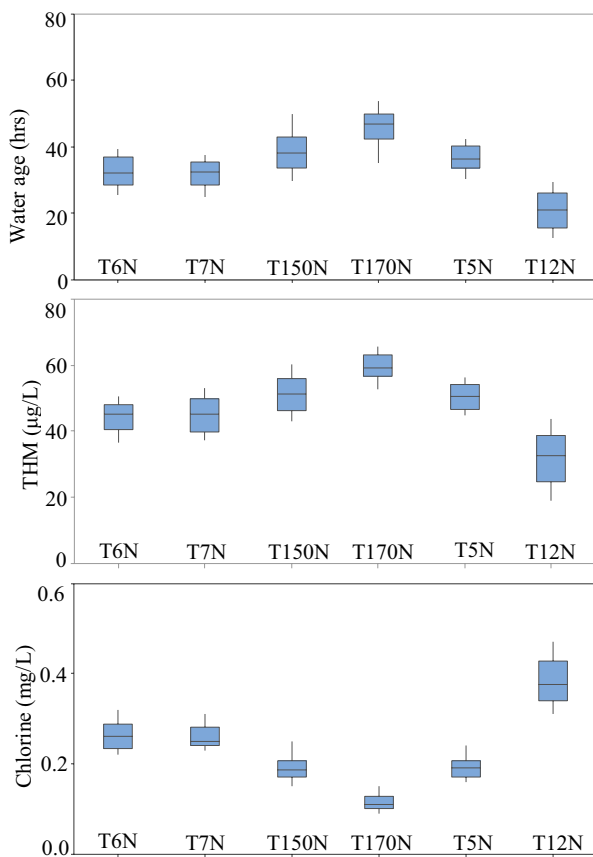


Fig. 3 Water quality of new tanks. T5N, T6N, T7N, etc. denote new tanks

On the other hand, the medians and flow-weighted means of water age, chlorine and THMs for Solution 2 are better than Solution 1 for Tank 42E. This outcome is consistent with Siew et al. [58] in which it was stated that only approximately 40% of the operational volume of Tank 42E of Solution 1 was utilized during the average day.

4.2.3 Disinfection by-products

The THM concentrations were consistent with the water ages; Prasad had the best performance, accordingly.

4.3 Water quality at the demand nodes

4.3.1 Walters et al. solution

The chlorine concentration at demand node 5 had values of 0.19 mg/L and 0.18 mg/L, at 03:00 and 04:00, respectively. Node 5 is remote relative to the treatment plant.

Also, the new Tank 5 N, whose chlorine concentration was less than 0.2 mg/L from 03:00 to 16:00, i.e. 13 h duration, supplies node 5 directly.

The concentration of chlorine was zero in the existing tanks throughout the 24 h. However, the concentrations at all the other demand nodes were satisfactory, even though the aggregated chlorine concentration of all the tanks was only 0.15 mg/L (Table 4). *The discrepancy between the tanks and demand nodes indicates that, hydraulically, the tanks are not well integrated in the network.* The nodal flow-weighted daily means in Table 1 reinforce this idea. Even though the two existing tanks were full throughout, which is highly undesirable from a water quality perspective, the overall water quality for the demand nodes was the joint best for chlorine, second best for water age and best for THMs. Thus, the flow-weighted means of the water quality parameters would appear to provide insights on the *hydraulic connectivity effectiveness* of the tanks in a network, besides the emergency and equalization storage roles.

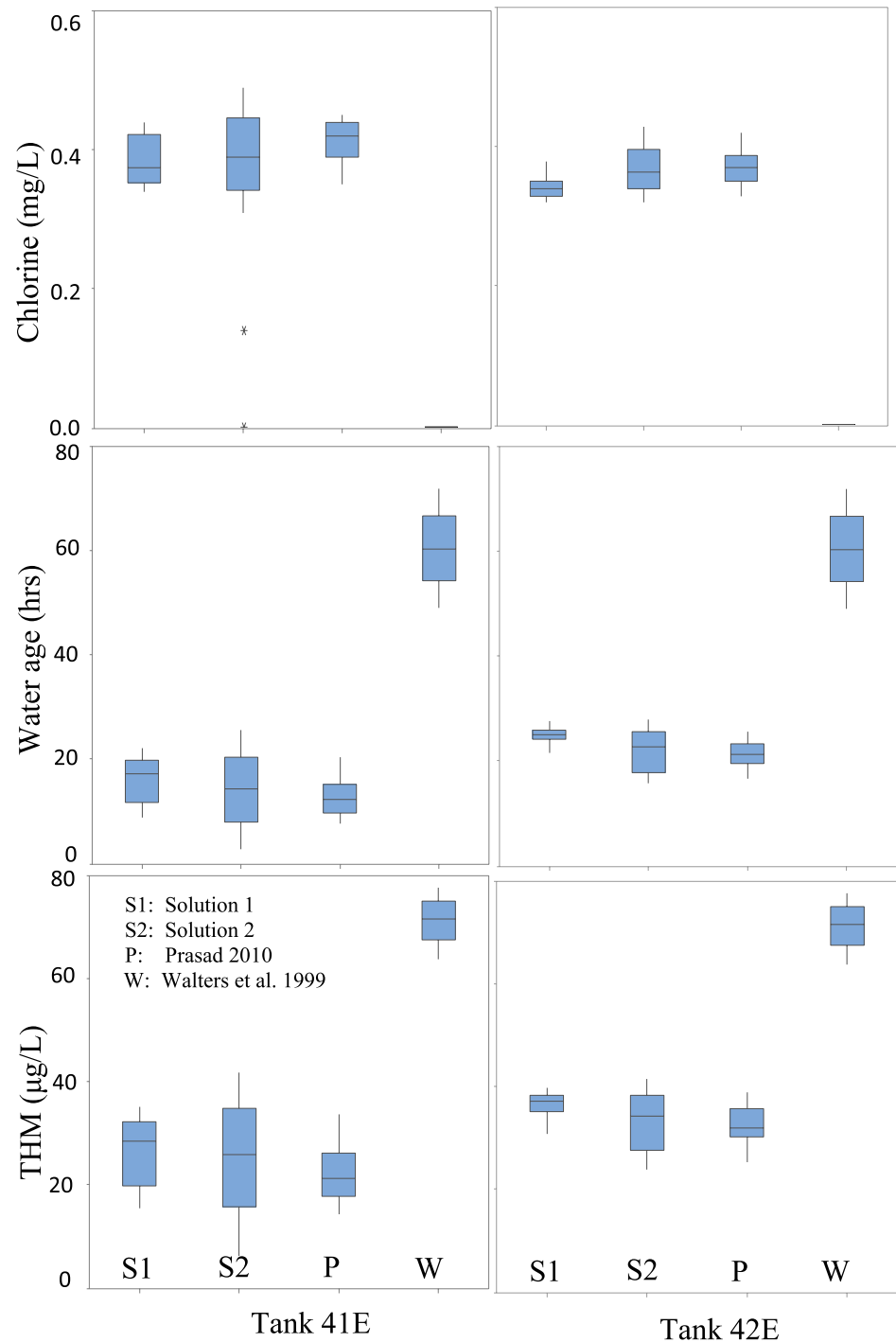
4.3.2 Solution 1

The chlorine concentrations at demand nodes 5, 7 and 9 were generally lower than the rest of the nodes. These nodes are remote relative to the treatment plant and have only two incident pipes. The lower values and larger fluctuations at node 7 are attributable to the new Tank 7 N at node 7 (Figs. 1, 5). The maximum daily fluctuation in the chlorine concentration at node 7 was 0.30 mg/L.

4.3.3 Solution 2

The maximum daily fluctuation in the chlorine concentration was 0.14 mg/L at node 6, due to the new Tank 6 N. It is interesting that the lowest daily concentration of 0.31 mg/L occurred elsewhere, i.e. at node 9. Node 9 is remote and has only two incident pipes. The maximum daily fluctuation of 0.14 mg/L for Solution 2 represents a reduction of 53.3% relative to Solution 1 whose maximum fluctuation was 0.30 mg/L. The improvements in Solution 2 relative to Solution 1 include: (a) a significant increase in the *minimum daily concentration* of chlorine from 0.22 mg/L to 0.31 mg/L, i.e. 40.9%; and (b) a significant reduction of 53.3% in the *maximum daily fluctuation* of chlorine. These improvements can be attributed to the innovative reservoir (tank) depletion measure that was included in Solution 2's formulation.

Fig. 4 Water quality of existing tanks. S1, S2, P and W denote solutions



4.3.4 Prasad solution

The lowest chlorine concentration of 0.25 mg/L was at demand node 9. The daily fluctuations at node 9 and 16 that have the new tanks were considerably more than the other demand nodes. The chlorine concentration in Tank 170 N (at node 9) was below 0.2 mg/L throughout the 24-h

cycle (Fig. 3). Similarly, the chlorine concentration in Tank 150 N (at node 16) was mostly below 0.2 mg/L. However, the chlorine concentrations at demand node 16 and 9 did not fall below 0.2 mg/L, as those nodes have alternative supply paths besides the new tanks. Nevertheless, the lowest concentrations at demand node 9 and 16 were considerably below the other demand nodes.

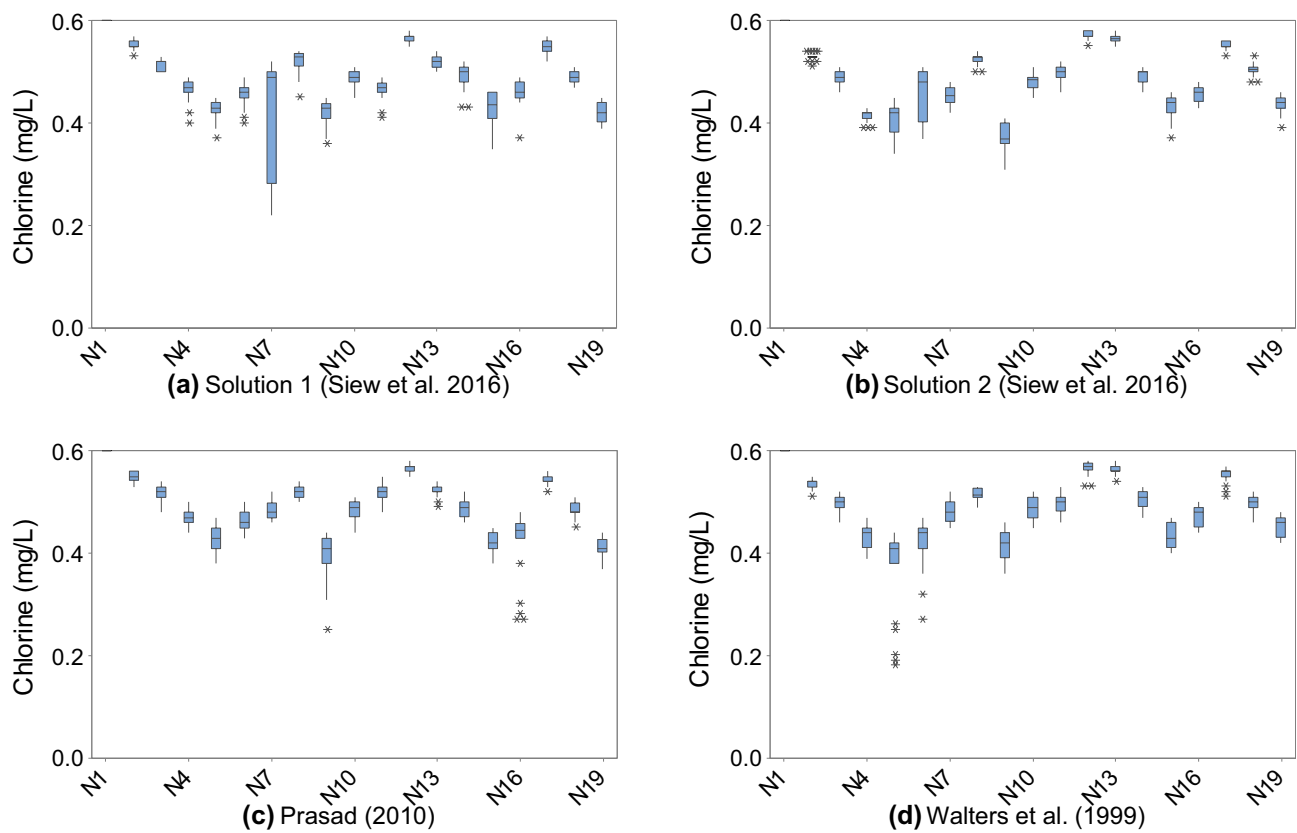


Fig. 5 Daily variations of chlorine concentrations of demand nodes. N1, N2, N3, ..., N19 represent nodes 1 to 19

These results illustrate the importance of the *reservoir location and network topology*. Table 1 shows that the aggregated performance of the demand nodes was the joint best for chlorine, best for water age and second best for THMs. However, the combined performance of the tanks is the second worst in Table 4. This shows the influence of the tanks is somewhat limited and may be an indication that, besides storage, the tanks are hydraulically less integrated and effective compared to Solution 1 and 2.

5 Discussion

The spatial distributions of the water age and concentrations of chlorine and THMs based on the daily demand node arithmetic means are shown in Fig. 6. They do not seem to reveal any dominant solutions. The simple arithmetic mean (US EPA 2006), inherently, lacks robustness, as any differences in the flows or volumes that the individual concentrations pertain to are not considered. This may be one of the reasons that satisfactory integrated service reservoir design optimization models are not available yet. Huang et al. [26] stated that research on water quality reliability has generally been very simplistic and

comprehensive measures and protocols were required. For example, Atkinson et al. [1] discussed the maximum and “average system water age”. Nevertheless, the demand node daily arithmetic means appear to indicate *possible performance outliers and gross trends* in the system’s response (Fig. 6).

The medians and flow-weighted means in Tables 2 and 3 suggest that the previously mentioned weaknesses in the Walters et al. and Prasad solutions are deep-seated. Furthermore, Table 4 shows that the tanks in Solution 2 have the best *overall performance*, followed by Solution 1, Prasad and Walters et. al. Also, Table 1 shows that Solution 2 has the best water quality at the *demand nodes*. Thus, the flow weighted water quality means would appear to be able to reveal the relative efficiency of the tanks in a network.

However, Table 2 also shows that the minimum concentration of chlorine in Tank 41E in Solution 2 was zero. Therefore, Solution 2 seems *marginally infeasible*. Figure 4 supports this observation; the two chlorine concentration values (out of 24) that are less than 0.2 mg/L are outliers. Accordingly, Solution 2 seems to be an exemplar of a *cost-effective virtually feasible* solution (with respect to water quality) that could thrive in a penalty-free multi-objective

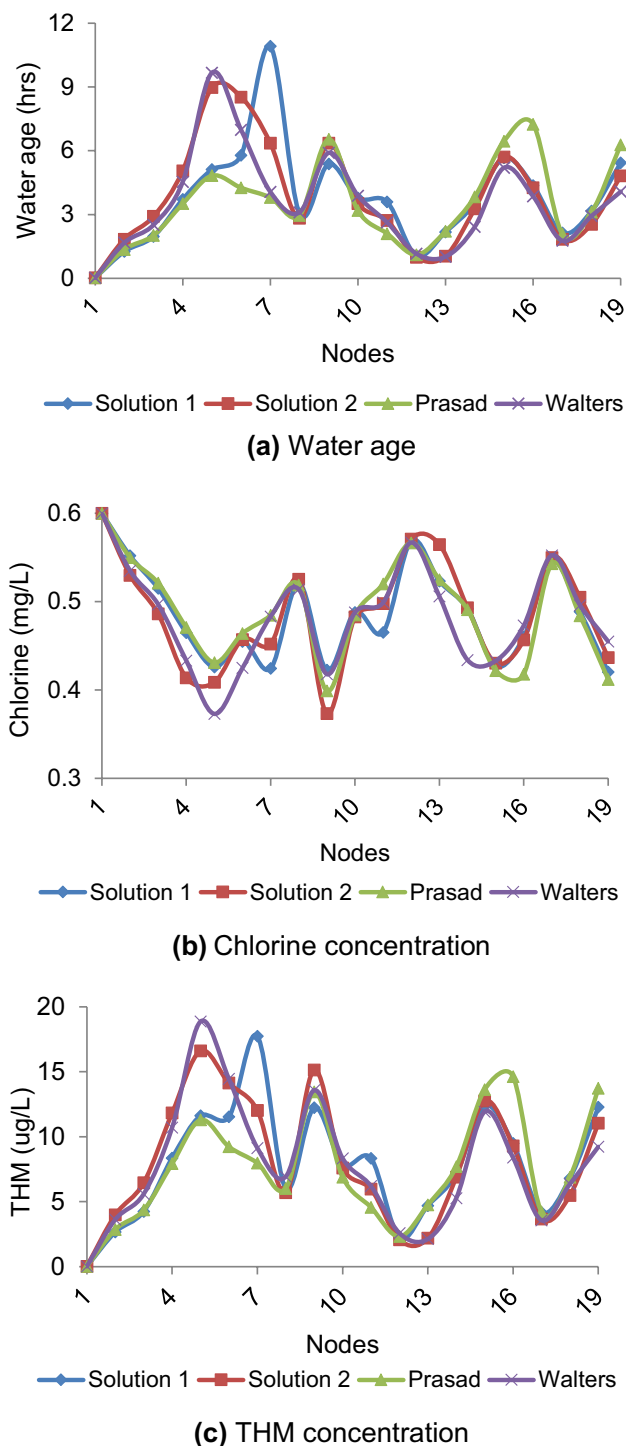


Fig. 6 Daily averages of water quality parameters of demand nodes

evolutionary optimization framework that co-evolves sub-populations of efficient feasible and infeasible *non-dominated* solutions [64]. Solution 2, therefore, highlights the vital need for robust performance indicators that reveal the *intrinsic properties of the system, as a unit*, to complement the regulatory compliance approach based purely

on the worst instances. The above-mentioned penalty-free evolutionary optimization approach has been shown to be highly effective, based on the solution quality and computational efficiency [43, 44, 57]. Overall, Solution 1 is the best in terms of the hydraulic requirements of nodal flows and pressures, water quality regulatory compliance and total cost.

Also, the water quality results achieved are consistent with the current distribution practice, in which booster chlorination is employed at the inlets of some reservoirs (tanks) in some extensive distribution systems [68, p. 458]. Taking all the tanks and demand nodes into consideration, the worst performing nodes overall for the Walters et al. solution are the two existing tanks (Tank 41E and 42E, Table 2) where the lowest chlorine concentrations occurred. The lowest chlorine concentration in Prasad's solution occurred in Tank 170 N (Table 3). The minimum chlorine concentration in Solution 1 occurred at node 7 (Table 1), where Tank 7 N is located, while the minimum chlorine concentration in Solution 2 occurred in Tank 41E (Table 2).

Solution 1 and 2 have the least expensive new tanks in the literature (Table A2) (Supplementary Materials). It is, therefore, reasonable to attribute at least part of the water quality weaknesses of other solutions to the built-in, albeit hidden, surplus capacity of the new tanks. Located in an area of relatively high demand and diametrically opposite to the water treatment works, the new tanks in Solution 1 and 2 would likely improve the hydraulic capacity reliability also. The reason is that the water would be supplied from two opposite ends of the network. The foregoing observation notwithstanding, aspects relating to hydraulic capacity reliability and failure tolerance were not considered as they fall outside the scope of the research reported here.

Also, it is thought that the criterion that promoted reservoir depletion also improved the cost effectiveness by removing surplus capacity [58]. This is an indication that the novel reservoir design methodology developed by Siew et al. [58] is highly effective. The operating water levels in the new tanks in Solution 1 and 2 were 5.80 m and 6.42 m respectively, compared to 4.32 m and 4.69 m, respectively, in Tank 150 N and 170 N in Prasad [39], for example. This aspect was not investigated, and additional research that includes the flow patterns in the tanks is worth considering in future.

Several minor inconsistencies in the simulation results were observed, which may be due to the single-species water quality simulation approach used. Thus, the worst instances based on water age, THM and chlorine did not coincide always. For example, an apparent phase shift of one hour for Tank 5 N could be because the results represent *independent* single-species simulations. The apparent

anomaly could be addressed easily using a multi-species simulation model [61], for example, EPANET-MSX (multi-species extension) [50]. However, the CPU (central processing unit) simulation times would increase significantly [47, 48].

Furthermore, a disadvantage of the EPANET 2 THM model used in the research is that the limiting value of the THM concentration should be provided in advance at the data input or initialization stage (e.g. the nodal chlorine concentrations were all set to zero to initiate the water quality simulations). A maximum THM concentration of 100 µg/L was assumed herein, based on the EU and UK drinking water standards (EC 1998; HMG 2010). It was observed that the THM concentrations in the tanks did not exceed 100 µg/L. Sohn et al. [60] proposed an alternative THM model that does not require a limiting concentration, which could be used instead, as in e.g. Seyoum and Tanyimboh [46].

Similarly, while the advection-reaction simulation model employed here neglects longitudinal dispersion, it can be substituted readily, if necessary. The methodology developed is generic and any suitable reaction kinetics approach besides the first order model in EPANET 2 may be used instead. An appraisal of various reaction kinetics approaches considering multiple species (including six species of haloacetic acids) is available in Seyoum and Tanyimboh [48] in which EPANET 2 achieved consistently good results.

6 Conclusions

A system-wide joint-dynamic-response approach to water quality evaluation in distribution networks with multiple service reservoirs was developed to account for the temporal and spatial variations in the nodal demands together with hydraulic and water-quality response of each service reservoir. More meaningful and instructive comparisons were thus achieved. This includes comparisons between different service reservoirs in individual solutions and across multiple solutions, and similarly the overall performance of competing solutions.

The results revealed that the correlation between the medians and flow-weighted daily means of the water quality parameters was very strong ($R^2 \geq 0.994$), for the 14 (fourteen) service reservoirs in the networks considered. Therefore, subject to further verification, it seems that the median could be useful as a practical performance surrogate in design optimization algorithms. Also, the results would appear to indicate that the flow-weighted daily means of the water quality parameters could help provide additional insights on the hydraulic integration and effectiveness of the service reservoirs

in a network, besides the emergency and equalization storage roles.

Nodal demands are non-deterministic in practice and the hyperplane that separates the system performance space into the safe and failure zones cannot be ascertained reliably from purely deterministic simulations [77]. There is, therefore, a need for robust performance indicators that reflect the system's underlying performance, to underpin those essential investigations that transcend routine regulatory compliance. There are significant differences in the four alternative networks considered, including the number of service reservoirs and their capacities, locations, modes of operation and operating water levels. Furthermore, comparisons of entirely different networks or distribution systems may be required in certain circumstances, for example, for investment planning purposes. Rigorous comparisons of high-dimensional water quality results would be required, as demonstrated here.

Finally, it was observed that the new service reservoir design methodology in Siew et al. [58] yields tanks that are both economical and hydraulically efficient. It is thus recommended for further evaluation including the explicit use of water quality design objectives. While the discussion herein considered only three water quality parameters and four Pareto-optimal candidate solutions for illustration purposes, the population size of the candidate solutions requiring fitness evaluations *in each generation* in an evolutionary optimization framework could be thousands or more, in an optimization problem having multiple service reservoirs and thousands of demand nodes. This is the thrust of the next phase of the research.

Funding This research did not receive any specific grant from funding agencies in the public, commercial, or not-for-profit sectors.

Availability of data and materials All the essential data are in the manuscript and supplementary materials.

Code availability EPANET 2 is free software in public domain (<https://www.epa.gov/water-research/epanet>).

Compliance with ethical standards

Conflict of interest The authors declare that there is no conflict of interest.

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