# Optimization techniques for leakage management in urban water distribution networks

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## ABSTRACT

Reduction of leakages in water distribution systems (WDSs) is one of the major concerns for water industries. This paper presents a leakage reduction technique using pressure management by optimizing the water level in storage tanks, along with optimized control and localization of pressure-reducing valves (PRVs) in WDSs. A new mathematical tank and pump simulation algorithm is presented for controlling pressure in WDSs, by optimizing the water storage level in the tank depending upon the demand variations. The tank is used as a decision variable for the leakage reduction model. A modified reference pressure algorithm is introduced for improving PRV localization. A multiobjective genetic algorithm (NSGA-II) is used to find the optimized operational control setting of the PRV for leakage minimization. The proposed algorithm leads to a leakage reduction of 26.51% in Anytown WDS and 20.81% in a modified benchmark WDS. This technique leads to an appreciable reduction in leakage rate, with fewer PRVs required, taking into account constraints such as maintaining a lower hydraulic failure index (<0.01), emergency storage, etc. It can be concluded that the proposed novel leakage reduction technique provides a more cost effective and efficient solution for leakage control.

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## INTRODUCTION

It is predicted that by 2050 half of the world will face water scarcity (Veolia Water 2011). Supplying a sufficient amount of water for growing populations will be a challenging task for municipalities and water industries, and searching for new water resources to fulfil the required demand is difficult. Expansion of water resources requires a lot of money for building infrastructure, hence better management of existing water resources can be seen as one of the probable solutions. A water leakage is one of the major causes of water losses (Gupta et al. 2016). Because of these leakages, extra water has to be pumped out, which increases energy consumption, and consequently increases the carbon footprint. Aged pipelines (especially if corroded) under high pressure are the major candidate for bursting, causing leakages. Many researchers and water authorities are working in the field of pressure management for leakage control.

Leakages can be controlled using structural (requires capital investment) and non-structural (through better management of existing resources) methods. Pressure-reducing valves (PRVs) have to be a promising structural technique for reducing excess pressure, but require capital investment, whereas non-structural methods such as pump and tank storage optimization do not requires any major capital investment. Water distribution systems (WDSs) which use storage tanks for water distribution remain under high pressure at low demand hours due to high water levels in storage tanks. This will increase the leakage rate. Maintaining an optimum water level in the storage tank during non-peak hours will cut down the excess pressure present in the WDS and decreases the probability of bursting the pipeline. This can be seen as an efficient method for controlling leakages without making any complex changes to pipeline

infrastructure. Demand patterns also vary according to season, causing variations in required tank water levels every hour. Maintaining optimized water storage levels in the tanks also requires optimization of pump operation to fill the tank at the desired level. The flow rate from a pump changes with respect to height, therefore the pump curve will also play an important role during pump optimization. There is a need to perfectly synchronize pump operation and tank storage optimization processes to provide efficient services such as maintaining minimum pressure at demanding nodes, and emergency storage for fire flow and recovery of tank level.

High peak hour demand requires higher storage levels, generally occurring in the middle of the day. This requires pumping operations during early low demand hours, to fill the tank to the desired level. Higher water storage levels in the tank leaves the system under high pressure during low demand hours, which come before and after peak hour demand. There is a need for time-controlled PRVs to control this excess pressure. Correct placement along with optimized PRV operational control settings is needed so that WDSs can supply water with desired efficiency. Initial optimization of water storage level in the tank will lead to reduction in the number of PRVs required for further leakage reduction. The optimization algorithm should be efficient enough to handle real-water network objective functions and variables without becoming too computationally complex to implement.

## BACKGROUND

Various researchers have studied pressure management for leakage reduction. Dynamic, linear, non-linear and mixed integrated programming are used for leakage optimization of WDSs. Mixed integrated non-linear programming (MINLP) was introduced by Hindi & Hamam (1991) for controlling pressure in pipeline infrastructures. A method has been proposed for the localization of valves for controlling pressure. This method has low accuracy and low reliability (Kraemer *et al.* 2007). MINLP was modified as mathematical programming with complimentary constraints (MPCC) by Raghunathan & Biegler (2003). MPCC also failed to optimize the valve location efficiently (Dai & Li 2014). Dai & Li (2014) proposed a non-linear program (NLP) algorithm

to solve MINLP problems. Binary variables have been proposed to accelerate the system performance. The results obtained using this algorithm were more favourable than those obtained using the existing algorithm for leakage reduction. Dai & Li (2016) proposed an extended model for calibrating optimal operational pressure values of PRVs using an NLP algorithm. The proposed algorithm was applied in EXENET WDS, leading to a leakage reduction of 5.72 l/s; however, it involves higher computational complexity. De Paola *et al.* (2016) presented a harmony search (HS) model to determine the optimum control settings across the valves. The proposed algorithm has demonstrated more favourable results with the HS algorithm compared to the genetics algorithm (GA) for pressure reduction in the WDS.

GA is a population-based technique that does not require simplification of the hydraulic model, hence GA is preferred for operational (PRV location and control, pump optimization) and design optimization (Tolson *et al.* 2004; Araujo *et al.* 2006) of WDSs when compared to classical deterministic optimization techniques. Nazif *et al.* (2010) have used GA for optimization of water storage levels in tanks for the city of Tehran, Iran. An artificial neural network model is used for hydraulic simulations. The optimization model leads to high reductions in leakage rate. Giacomello *et al.* (2012) used hybrid linear programming (LP) and a greedy algorithm for pump and tank optimization. There is a 5% reduction in the electricity bill of Anytown WDS observed after applying the proposed optimization technique. Araujo *et al.* (2006) have used a pseudo-valve insertion method for PRV installation (similar to Jowitt & Xu 1990). In this method, a PRV is inserted into the pipeline and parameters are calculated using GA. The meta-heuristic approach was used to determine the optimized control setting of the PRV. The simulations were performed on a small-scale WDS. In a real-world scenario, using this such an approach in complex pipeline infrastructures would be difficult because of the presence of a larger number of variables and computational complexity. Sometimes, the meta-heuristic approach may only be able to determine sub-optimal solutions. Creaco & Pezzinga (2014) proposed a hybrid objective algorithm (GA and LP) for valve installations and pipe replacements for optimizing the WDS. GA is used for optimized placement of valves and LP is used to search for an optimal setting of the control

valve. In addition, their study explored a trade-off between the number of PRVs and leakage volume. The proposed algorithm is quite efficient compared with the GA.

Optimization of water systems requires multiobjective functions, i.e. to determine the optimized value and location of the PRV and to minimize the leakage rate. These objectives have attracted researchers (Prasad & Park 2004; Farmani *et al.* 2005) to use a multiobjective genetic algorithm (NSGA-II) for water infrastructure management. Nicolini & Zovatto (2009) used NSGA-II to reduce leakages in the WDS using pressure management. NSGA-II was used to determine optimized locations and control settings of PRVs. This algorithm uses a population size of 100 for individual evolving for 500 generations. A higher reduction in the average leakage rate was observed compared with that in Araujo *et al.* (2006).

This paper proposes a pressure reduction model for optimal leakage control in a WDS. A new iterative mathematical tank and pump simulation algorithm is proposed for optimization of water storage levels in the tank (on an hourly basis), depending upon demand variations. Different water demand patterns were considered to make the algorithm practically implementable in real-world WDSs. PRVs have been installed in the WDS for further leakage reduction. A modified reference pressure algorithm is introduced for better localization of PRVs. NSGA-II is used to determine the optimized operational control setting across PRVs under different load conditions (first objective) and to minimize the leakage rate (second objective). EPANET 2.0 (Rossman 2000) is used as a simulation tool along with MATLAB R2015a for calibration purposes, which is performed on a desktop (Intel i7 processor with 16 GB RAM). The paper also focuses on minimizing the hydraulic failure index of a WDS.

## PROPOSED METHODOLOGY

The aim of pressure management is to minimize leakages and to maintain required pressure at every node. The first objective of the proposed algorithm is to find the optimum water storage level in the tank, along with the optimal location and control settings of the PRV as the second objective. The flow chart of the proposed algorithm is shown in Figure 1.
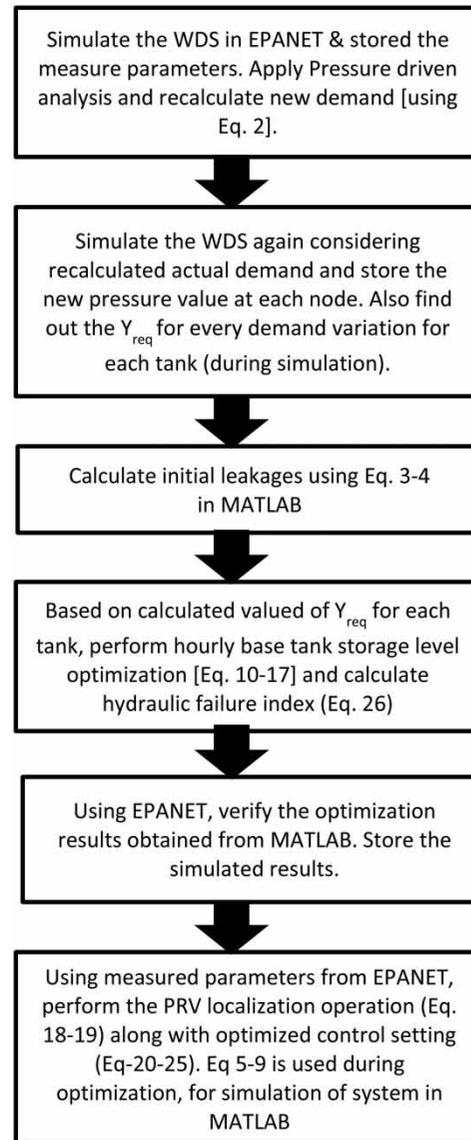


Figure 1 | Flow chart of proposed methodology.

## Mathematical formulation

Hydraulic simulation of the WDS is based on EPANET (Nicolini & Zovatto 2009), to which the optimization model has been coupled. The law of conservation is represented using sets of equations.

The continuity at a node is expressed as:

$$\sum_{j} Q_{ij,k} - K^* Q_{req,i} - L_{i,k} = 0 \tag{1}$$

where $Q_{ij,k}$ is the flow through the pipeline between nodes i and j for load condition k; $Q_{req,i}$ is the required flow through node i; and $L_{i,k}$ is the leakage associated with node i.

In order to estimate a better flow through the network (including demand and water losses) a pressure-driven demand model (adapted from Giustolisi *et al.* 2008) is used here.

$$Q_{req,i} = \begin{cases} Q_{i,des} & \text{for } P_{i,k} > P_{ser} \\ Q_{i,des} * \left( \dfrac{P_{i,k} - p_{mi}}{P_{ser} - p_{mi}} \right)^{0.5} & \text{for } P_{mi} \leq P_{i,k} \leq P_{ser} \\ 0 & \text{for } P_{i,k} \leq P_{mi} \end{cases} \tag{2}$$

where $Q_{i,des}$ is the base demand at node i; $P_{i,k}$ is the pressure of node i during load condition k; $P_{ser}$ is the minimum service pressure required for supplying demand in the network; $[P_{ser}, P_{mi}]$ is the range of intermediate working conditions when actual demand is $Q_{req,i}$; and $P_{mi}$ is the minimum pressure at which demand supplied is 0. For the pressure-driven demand model, the value $P_{mi} = 0$ is applied at all nodes (Giustolisi *et al.* 2008).

Using Equation (2), demand for each node is recalculated and, depending upon the new required demand for each node, simulation of the WDS is again performed in EPANET. Observed pressure for all nodes is stored. The leakage rate of the water network is calculated using the pressure-based leakage model using Equations (3) and (4). Assuming leakage is distributed along the pipeline network, the leakage associated with node i is:

$$L_{i,k} = C_L * L_i * P_{i,k}^{\gamma} \tag{3}$$

where $C_L$ is the coefficient of leakage per unit length of the link to service pressure; $L_i$ is the total length of pipeline associated with node i; and $\gamma$ is the leakage exponential used to define the relationship between the flow from the orifice and the head difference. A leakage exponential value of 1.18 (Jowitt & Xu 1990) is observed for cracks in pipes or joints that occur due to differences in pressure of the internal and external pipe. Therefore writers have adopted the same value, i.e. 1.18, for the leakage exponential.

The total length of pipeline associated with node i ($L_i$) can be calculated using:

$$L_i = 0.5 * \sum_j L_{ij} \tag{4}$$

where $L_{i,j}$ is the total length of pipeline connected with node i.

According to the conservation of energy, head loss can be calibrated as:

$$H_{ij,k} = H_{i,k} - H_{j,k} \tag{5}$$

where $H_{ij,k}$ is the head loss between node i and j; $H_{i,k}$ and $H_{j,k}$ are the head loss measured at node i and j during load condition k.

The head loss for the pipeline using flow values can be calculated using the Hazen-William equations:

$$H_{ij,k} = 10.668 \, C_{ij}^{-1.85} d_{ij}^{-1.85} l_{ij} Q_{ij,k}^{1.852} \tag{6}$$

where $C_{ij} = $ Hazen-William coefficient; $d_{ij}$ is the diameter of the pipeline; $l_{ij}$ is the length of pipeline connected to nodes i and j; and $Q_{ij,k}$ is the flow across the pipeline for load condition k.

The head loss across the PRV can be expressed as:

$$H_{ij,k} = 10.668 \, C_{ij}^{-1.85} (v_{ij,k} d_{ij})^{-1.85} l_{ij} Q_{ij,k}^{1.852} \tag{7}$$

where $v_{ij,k}$ is the diameter multiplier when a valve is present, during the active mode $v_{ij,k}$ varies between 0 and 1 ($0 < v_{ij,k} < 1$) and for the open mode $v_{ij,k}$ is 1; $Q_{ij,k} \geq 0$ means that reverse flow of water is not allowed.

The limitation associated with Equation (7) is that it cannot be used in the case of a closed-mode PRV, i.e. when the value of $v_{ij}$ becomes 0. To overcome this drawback, Dai & Li (2016) modified the existing model used for calculating head losses in the presence of a PRV. They have introduced new mathematical equations for calculating head loss in all three operational modes of a PRV. For this study we have also adopted the extended model proposed by Dai & Li (2016). $H_{ij,k}$ across the PRV can be calibrated as:

$$\begin{aligned} H_{ij,k} &= \max\left(0, \, H_{i,k} - H_{j,k}\right) \\ &= 10.668 \, C_{ij}^{-1.85} (v_{ij,k} d_{ij})^{-1.85} l_{ij} Q_{ij,k}^{1.852} \end{aligned} \tag{8}$$

This equation is able to describe all three modes of operation of the PRV i.e. open, closed and active, which is required during the optimization process.

Now using the value of head, pressure at node i is calibrated using:

$$P_{i,k} = H_{ij,k} - Z_i \qquad (9)$$

where $Z_i$ represents elevation at node i.

## Tank water storage level optimization

A new mathematical tank simulation algorithm is proposed to achieve optimized water storage levels. Optimized tank storage levels have been formulated and can be mathematically expressed as:

$$Y_{i,t} > Y_{req,i,t} \qquad (10)$$

$$Y_{i,end} \geq Y_{i,start} \qquad (11)$$

where $Y_{i,t}$ is the current water level of tank i at time t; $Y_{req,i,t}$ is the required water level at the same instance, such that the minimum required pressure ($P_{req}$) will be maintained at demanding nodes; $Y_{i,end}$ and $Y_{i,start}$ are the water levels of tank i at the start and end of the day.

Based upon the hourly flow, the new head (water level) of tank i at time t can be calibrated using:

$$Y_{i,t} = Y_{i,t-1} - H_{i,\Delta t} \qquad (12)$$

$$H_{i,\Delta t} = \frac{V_{i,\Delta t}}{Cross_i * 10^3} \qquad (13)$$

$$V_{i,\Delta t} = Q_{i,t} * 3600 \qquad (14)$$

where $Y_{i,t-1}$ is the water level of tank i at time $t-1$; $H_{i,\Delta t}$ is the water level reduced for tank i during the previous time step ($\Delta t$ is 1 hour); $V_{i,\Delta t}$ is the total volume consumed from tank i during the previous time step (1 hour); $Cross_i$ is the cross-section of the tank, given by $\pi R^2$; $R$ is the radius of tank; and $Q_{i,t}$ is the average hourly flow from tank i in l/s.

A unit cubic metre stores 1,000 litres of water, therefore a multiplication of $10^3$ is introduced during the calculation of head loss (Equation (13)).

A pumping operation is required at time $t-N$ hours ($N = 1, 2, 3...n$) to fill the storage tank above the required level at time t. Due to pump operation, the water level of the tank will change (increases) according to:

$$Y_{i,t} = Y_{i,t-1} + \left( \frac{Q_{act*3600}}{Cross_i * 10^3} \right) \qquad (15)$$

$$Q_{act} = Q_{pump,i} - Q_{i,t-1} \qquad (16)$$

$$0 \leq Q_{pump,i} \leq Q_{pump,max,i} \qquad (17)$$

where $Q_{act}$ is the flow rate in l/s by which water is filled in the tank; $Q_{pump}$ is the flow rate of the pump; and $Q_{pump,max,i}$ is the maximum flow from pump i.

The flow rate from the pump changes with respect to head. It is important to correctly identify the flow from the pump for current water level (head) using the pump curve. Fixed global pump efficiency is used as the pump efficiency curve, which is the default pump efficiency cure used by EPANET. If the tank fails to achieve the desired storage level at time t, then the pump operation is performed at $t-1$ hour to fill the tank and the new tank level is recalculated using Equations (10)–(17). After pump operation at time $t-1$, if the tank again fails to achieve $Y_{i,t} > Y_{req,i,t}$ then the pump operation starts at time $t-2$ and once again parameters have been recalculated for time $t-1,t$ (Equations (10)–(17)). This iterative process will be continued until the condition $Y_{i,t} > Y_{req}$ (Equation (10)) is satisfied.

Optimized tank water level results obtained from the proposed algorithm are verified by simulating the WDS in EPANET. It is observed that, after tank optimization, various nodes still remain under an undesirable high pressure. Therefore PRVs can be installed in WDS pipelines to reduce this excess pressure.

## PRV localization

Liberatore & Sechi (2009) introduced the concept of reference pressure ($P_{ref}$) to restrict candidate valve locations to

a set of pipelines. Our study also uses the reference pressure technique for valve localization. Nodal pressure is considered for an average load condition. Consider G to be a set of the entire pipeline present in the WDS. A subset $G_v$ ($G_v \in G$) is derived, which belongs to the pipeline connected between nodes i and j, for the PRV candidate, and is expressed as:

**Rule – 1**. if $N_j > P_{ref}$ and $N_i < P_{ref}$ (18)

where $N_i$ and $N_j$ are the pressures at nodes i and j.

The reference pressure is selected during valve localization (using Equation (18)). The reference pressure is varied over a range to determine different values of $G_{v,n}$ ($G_{v,n}$ representing the number of candidate valve locations for a current value of $P_{ref}$). A pressure value corresponding to a minimum value of $G_{v,n}$ is selected as the reference pressure.

The rule proposed (Equation (18)) by Liberatore & Sechi (2009) suffers from drawback that, sometimes pressure difference between nodes $N_i$ and $N_j$ was slightly higher. However, this location is not considered for PRV localization because pressure was above $P_{ref}$ for both the nodes. If we able to cut this excess pressure difference between node $N_i$ and $N_j$, this will lead to high reduction in leakage rate of WDS. To overcome this drawback, a new rule (Rule-2) is introduced for PRV localization, with an idea behind is, to also consider the pipeline connecting to nodes $N_i$ and $N_j$ as PRV candidate ($G_v$), which have pressure difference more than predefined threshold value.

**Rule – 2**. if $N_j - N_i > 0.1 \times P_{ref}$ (19)

To make this threshold as WDS dependable, the threshold ($\geq 0.1 \times P_{ref}$) is decided in accordance with reference pressure of existing WDS. If we lower down the threshold (5% of $P_{ref}$) value, then number of PRV candidate will get increase, therefore pressure difference (threshold) is decided to be 10% of reference pressure, to minimize the number of PRV candidate. Rule-2 is applied for localization of PRV candidate after applying rule-1.

## Multiobjective genetic algorithm (NSGA-II) for valve optimization

The optimal control setting of a PRV is calibrated using the multiobjective GA. We aimed to determine the optimized operational control value of the PRV (first objective) with minimization of leakage rates in the WDS (second objective). After optimization, every node present in the system should maintain minimum pressure ($P_{req}$) to avoid pressure deficiency at any node.

The first objective (f1) was to determine the optimized operational pressure value ($P_{set}$) of the PRV. The objective function is as follows:

$$\min f1 = \sum_{i=1}^{N_s} w_k C_L L_i \ P_{i,k}^{\gamma} \qquad (20)$$

subject to:

$$P_{i,k} \geq P_{req} \qquad (21)$$

$$n_v \leq N_v \qquad (22)$$

$$H_{i,j,k} = H_{i,k} - H_{j,k} \qquad (23)$$

$$P_{min} \leq P_{set} \leq P_{max} \qquad (24)$$

where $P_{req}$ is the minimum pressure that is required at each node; $n_v$ represents the number of PRVs currently being used in the WDS; $N_v$ represents the maximum number of PRVs that can be placed in the WDS; $P_{min}$ and $P_{max}$ are the minimum and maximum values allowed across the PRV; $N_s$ represents the number of nodes present in the system; and $w_k$ is the value of the demand multiplier. Optimized value of PRV has been found out for individual load condition at a time.

The aim of the second objective (f2) is to minimize the leakage rate of the WDS, and the objective function is given as:

$$\min f2 = \sum_{i=1}^{N_s} C_i P_{i,k}^{\gamma} \qquad (25)$$

where $C_i$ ($C_i = L_i * C_L$) is the flow intensity at node i.

The leakage rate is calculated for the WDS corresponding to all the optimized operational setting values ($P_{set}$) for the PRVs generated from the first objective (f1). The operational control value of the PRV corresponding to the lowest leakage rate is selected. The head and pressure value at each node of the pipeline is required to calibrate the function f1 and f2. When the control value across the PRV ($P_{set}$) changes, the value of the head and head loss across all the nodes also changes. The value of $P_{set}$ varies from $P_{min}$ to $P_{max}$. Calculating the value of the head and pressure for each value of $P_{set}$ in EPANET is not feasible; therefore, within the objective function, another function known as **Head_cal**($P_{set}$) is used to calculate the value of the head for all the nodes corresponding to the present value of $P_{set}$ using Equations (5)–(9). The calculated values of the heads and pressure are used for calibrating f1 and f2. This algorithm uses a population size of 50 for individual evolving for 200 generations. The crossover and mutation probabilities used for this optimization process are 0.65 and 0.002.

## Hydraulic failure index

It is calculated for individual node i, after tank level optimization, when the pressure at the node becomes less than the minimum required pressure ($P_{req}$) at time *t*. Minimum and maximum pressures allowed in a WDS are 30 m and 50 m according to standard literatures (Nazif *et al.* 2010), allowing a maximum hydraulic failure index of 0.01. The hydraulic failure index ($If_{i,t}$) for any node i during load condition k can be calculated as:

$$If_{i,t} = \frac{\sum_{q_{i,t}} (P_{req} - P_{i,t})}{\sum_{q_{i,t}} * P_{req}} \qquad (26)$$

where $q_{i,t}$ is the demand at node i at time t; and $P_{i,t}$ is the pressure at node i at time t.

## RESULTS AND DISCUSSION

### Case 1: Benchmark network

The proposed hybrid optimization technique is performed on a benchmark water network as shown in Figure 2, considering four seasons. This network is used as a benchmark in Nicolini & Zovatto (2009), Araujo *et al.* (2006), etc. for pressure management in a WDS. The network contains 22 nodes and 37 links (pipelines). As the proposed algorithm is for tank storage level optimization, the reservoirs are replaced by the tanks. The tank has an elevation, diameter, minimum and maximum water level of 46 m, 20 m, 3 m and 10 m, respectively, represented by 3, 4 and 6 in Figure 2.

The WDS has a base demand of 150 l/s, and is taken from Jowitt & Xu (1990). Demand pattern variation (demand multiplier) for every season is adopted according to Nazif *et al.* (2010) as shown in Figure 2. Average demand pattern, for winter, summer, spring and autumn are 132 l/s (0.88), 171 l/s (1.14), 136.5 l/s (0.91) and 150 l/s (1), respectively, and can be identified from Figure 3.

Hydraulic simulations of WDS are performed in EPANET. Water level at each tank is maintained at 10 m (to share same water level) to benchmark our WDS with Araujo *et al.* (2006), Nicolini & Zovatto (2009), and Dai & Li (2014). After simulations, hydraulic parameters such as pressure at nodes, flow rate from pumps and tanks is stored. Pressure driven analysis is performed in WDS using Equation (2). Standard EPANET results shows (Giustolisi *et al.* 2008) $P_{ser}$ of 10 m for all nodes for supplying required demand ($Q_{i,des}$), with value of $P_{mi}$ as 0 m. Demand for each node is recalculated and depending upon the new required demand for each node, network is again simulated in EPANET and new pressure value for individual node is stored. Leakages have been calculated for different season using Equations (3) and (4). Calibrated leakage rate during autumn, summer, winter and spring are 29.3, 28.5, 29.6 and 29.5 l/s, respectively. For leakage calculations a typical value of $10^{-5}$ is taken as $C_L$ (Araujo *et al.* 2006).

Tank water storage level optimization is performed for each season using proposed algorithm. $Y_{req,i,t}$ is calculated from hydraulic simulations (EPANET), such that system will maintain minimum pressure ($P_{req}$) of 30 m at every demanding node. $Y_{req,i,t}$ gets changes on hourly basis, depending upon the variations in demand pattern. Proposed tank simulation and optimization operations are performed on MATLAB using Equations (10)–(17). Tank optimization operation also requires pump operation to fill the tank at desire level. Flow rate from pump changes with respect to
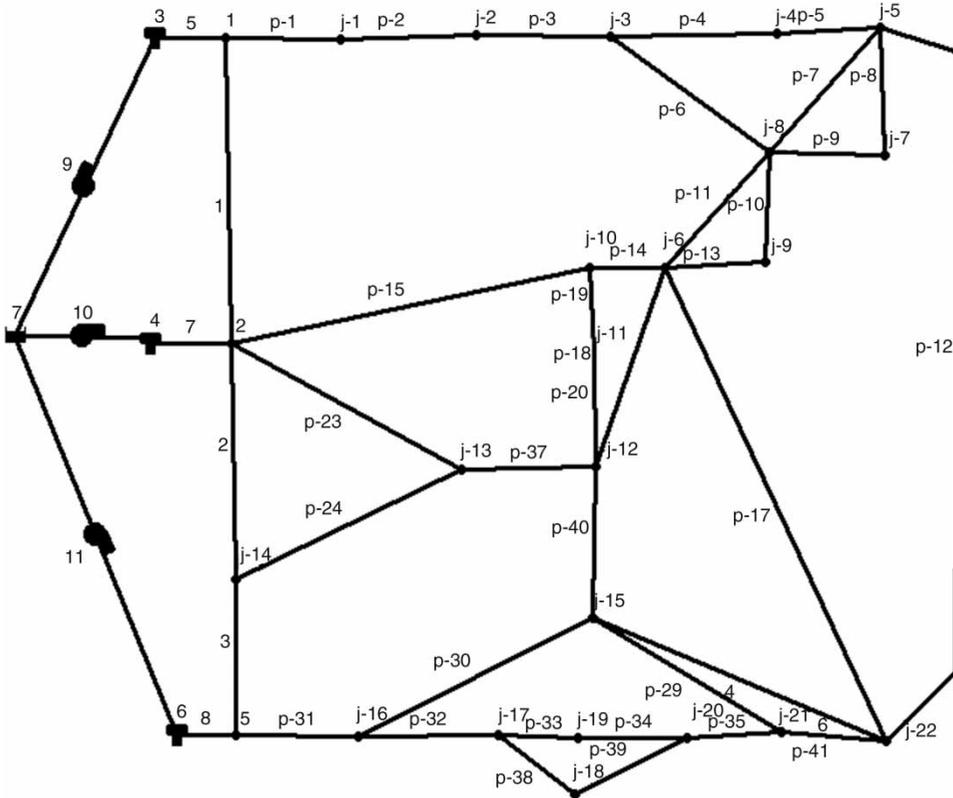
**Figure 2** | Modified WDS for pressure management adopted from Araujo et al. (2006).
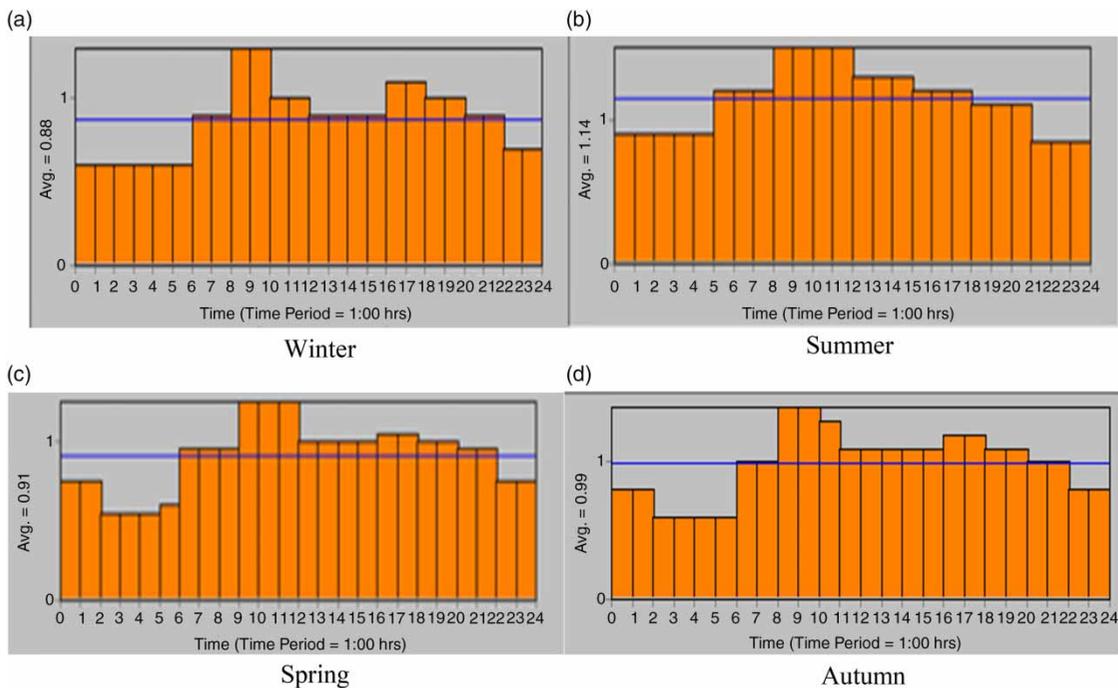


**Figure 3** | Demand pattern of WDS for different seasons.

height, hence there is a need to identify correct flow rate from the pump. The pump used here had a flow rate that varied from 0 to 160 l/s ($Q_{pump,max,i}$), with respect to head variations from 67 m to 0 m, respectively. The flow equation (represents pump curve) used here is:

$$Hb = 66.67 - 0.00258*Qb^2 \tag{27}$$

For a given head (water level), flow rate from a pump can be calculated using Equation (27). The flow equation is selected such that, at the head of 50 m the pump will deliver flow of 90 l/s which is double the average flow from the tanks during autumn. Table 1 describes the calibrated optimized pump operation using proposed algorithm for different seasons.

Tank storage level and pump optimization is performed for each season. It can be observed from Table 1 that flow rate from pump is maximum for spring and winter, where required storage level in the tank is less when compared to summer season, which requires higher storage level in the tank. Optimized tank water storage level for summer season is shown in Figure 4.

Optimized tank water storage level results obtained from the proposed algorithm (Figure 4) are verified by simulating the WDS (Figure 5) in EPANET, using the optimization results such as pump operation, calibrated initial water level, etc. from the proposed algorithm. It can be observed from Figures 5(c) and 4 that simulated results from EPANET and from proposed algorithm are almost similar, thus proves the accuracy of proposed tank and pump simulation algorithm. Figure 5(a)–5(d) show the verified results of optimized tank water storage level for autumn, winter, summer and spring, respectively. In Figure 5(a)–5(d) red, green and pink lines represent the optimized water level of tank 3, 4 and 6, respectively. After optimization the leakages have been reduce to **26.3 (10.2%), 26.15 (8.3%), 26.4 (10.8%) and 26.33** l/s (**10.84%**) for autumn, summer, winter and spring, respectively.

High water level requirement in tank during summer causes long pump operation, resulting in comparatively less reduction in leakage rate. This can be improved by using the variable speed pump having speed of 1.1 times in summer, causing better flow rate of 115 l/s (double than

**Table 1** │ Optimal pump flow rates (l/s) obtained on using proposed algorithm for different seasons

| Time | Autumn | Summer | Spring | Winter |
|------|--------|--------|--------|--------|
| 0:00 | 0 | 0 | 0 | 0 |
| 1:00 | 0 | 85.79 | 0 | 0 |
| 2:00 | 0 | 84.45 | 0 | 0 |
| 3:00 | 0 | 83.15 | 0 | 0 |
| 4:00 | 88.55 | 81.87 | 91.25 | 89.59 |
| 5:00 | 86.6 | 81.05 | 89.3 | 87.75 |
| 6:00 | 84.82 | 80.31 | 87.43 | 85.92 |
| 7:00 | 83.53 | 79.6 | 86.13 | 84.55 |
| 8:00 | 82.32 | 79.34 | 84.88 | 83.24 |
| 9:00 | 81.14 | 79.15 | 83.65 | 82.53 |
| 10:00 | 0 | 78.99 | 0 | 81.91 |
| 11:00 | 0 | 0 | 0 | 0 |
| 12:00 | 0 | 0 | 0 | 0 |
| 13:00 | 0 | 0 | 0 | 0 |
| 14:00 | 83.31 | 85.4 | 90.5 | 0 |
| 15:00 | 81.8 | 84.6 | 88.92 | 87.07 |
| 16:00 | 81.65 | 83.89 | 87.71 | 85.73 |
| 17:00 | 82.08 | 83.1 | 86.58 | 84.7 |
| 18:00 | 82.67 | 82.2 | 85.49 | 83.72 |
| 19:00 | 83.29 | 81.28 | 84.35 | 82.62 |
| 20:00 | 83.94 | 80.39 | 83.22 | 0 |
| 21:00 | 84.56 | 0 | 0 | 0 |
| 22:00 | 0 | 0 | 0 | 0 |
| 23:00 | 0 | 0 | 0 | 0 |

average flow for summer) at 50 m head, the leakage rate is reduced to 25.5 l/s (10.52%) for the summer season, which is 2% more than previous reduction rate. The pump operational time is reduced to 13 hours, which is 4 hours less when compared to earlier pump operational time. Although energy consumption in both the cases remains almost same, that is 956 Kwatts. If there is further increase in pump speed, i.e. 1.2 or 1.5 times its regular speed, then pump operation times may be reduced, but tank will have higher head level, which will increase the leakage rate. During emergency condition such as fire flow, the system can deliver flow rate of 40 l/s (considering single node) for 18 hours other than usual demand. Minimum water level of 3 m is maintained at all 3 tanks. But during this can cause pressure deficiency at some of the nodes. The system has a hydraulic
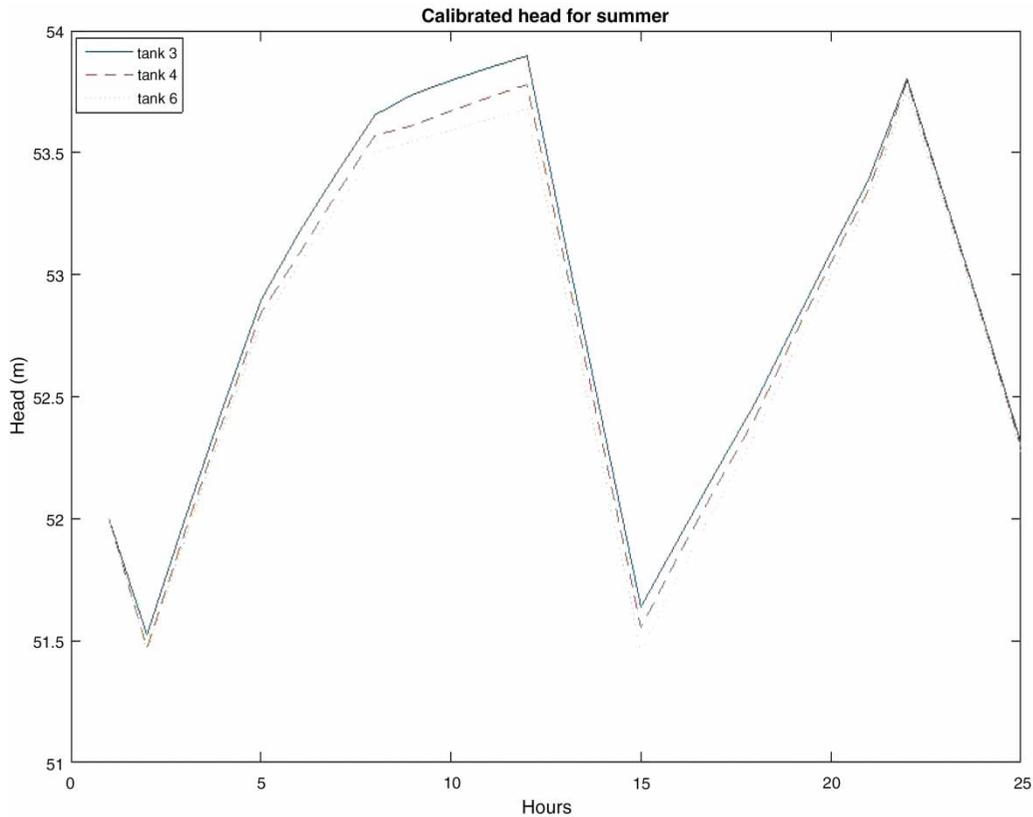
**Figure 4** | Calibrated tank water storage level optimization during summer (using MATLAB).

failure index (using Equation (26)) of 0.00425, 0.00489, 0.00412 and 0.00391 respectively for autumn, summer, winter and spring, respectively.

Higher water storage level in tank leaves system under high pressure during low demand hours comes before and after peak hour demand. Hence, time controlled PRV has been installed to reduce excess pressure during low demand hours. PRV locations are calculated for average value of load condition during autumn season using modified reference pressure algorithm i.e. Equations (18) and (19) (rule-1, 2). Value of reference pressure was selected using Equation (18). The value of pressure references varied from 32 to 40 m. Figure 6 shows that the number of PRV is minimum for the reference pressures of 36 m and 37 m. The concept of reference pressure ($P_{\mathrm{ref}}$) is introduced to restrict valve candidate locations to a small set of pipelines, therefore, 36 m is selected as value of reference pressure. After this, rule 2 (Equation (19)) has been applied for finding out any additional PRV localization to remove

the drawback of rule-1. One additional location is observed for PRV candidate after applying new rule. The maximum value of the number of PRV was fixed to 5 ($N_{\mathrm{v}}$). The locations of PRV are at pipe 1, 31, 19, 18 (from rule-1), and 40 (from rule-2).

**NSGA-II** is used for finding out optimized operational setting of PRV using two objective functions and constrains mentioned in Equations (20)–(25). System takes hydraulic parameters such as nodal pressure, head, etc as input. The operational control pressure of PRV ($P_{\mathrm{set}}$) will vary between 30 m ($P_{\mathrm{min}}$) and 36 m ($P_{\mathrm{max}}$). The value of minimum desired pressure ($P_{\mathrm{req}}$) at every node is 30 m. The valve for optimal operational value of PRV varies from 30.97–32.23 m for low demand hours and from 32.23–34.87 m during high demand hours for different seasons. GA uses random population for generation of optimal solution. Every time this system generates different optimal solution. Window is already provided for population generation, which minimizes the variations in final optimal solution by ±0.15 m. Therefore NSGA-II
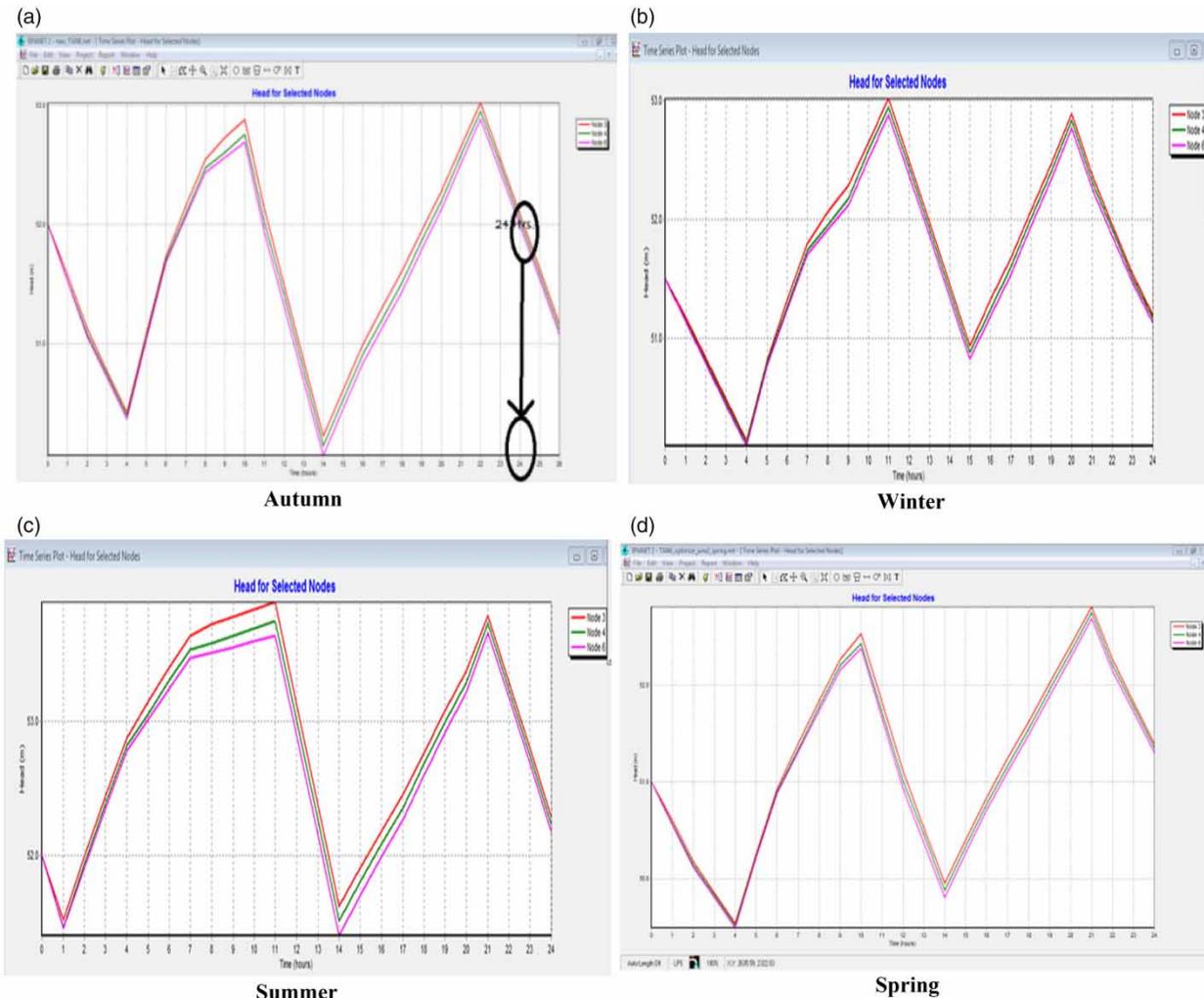
**Figure 5** | Optimized water storage level of tanks for different seasons. Please refer to the online version of this paper to see this figure in colour: http://dx.doi.org/10.2166/ws.2017.064.

has been run for three times and optimal solution is selected by calculating the average of these solutions. For every change in optimal control setting ($P_{set}$) there is change in nodal pressure and head. It is not possible to calibrate this change in EPANET. A function, Head_cal ($P_{set}$) is used within the NSGA-II to calculate the value of head and pressure (using Equations (5)–(8)) at each node corresponding to current operational control setting of PRV at $P_{set}$. On performing a random check, the values of pressure calculated using Equations (5)–(9) varied from actual simulated values (from EPANET) with a maximum error of ±1.3% (i.e. ± 0.5 $m$).

Table 2 provides a comparison of leakage rate and hydraulic failure index for each season. As expected

leakage rate during the summer season is less due to high water consumption. It is observed that, there is noticeable leakage reduction is achieved on using proposed optimization method (Tank storage level and PRV optimization), with reduced number of required PRV. Leakage rate have been reduced from 29.3 l/s to 23.2 l/s after pressure management for autumn season, having same demand pattern as that of benchmark WDS by Jowitt & Xu (1990). Leakage rate is reduced by 20.81% (6.1 l/s). After adding fifth PRV at Pipe-19 the PRV remains open during peak hours. Whereas during low demand hours, it leads to nominal additional leakage reduction of 0.24 l/s, which is less than 1% of initial leakage rate. Addition of fifth PRV will only increase the infrastructure cost of WDS, thus only
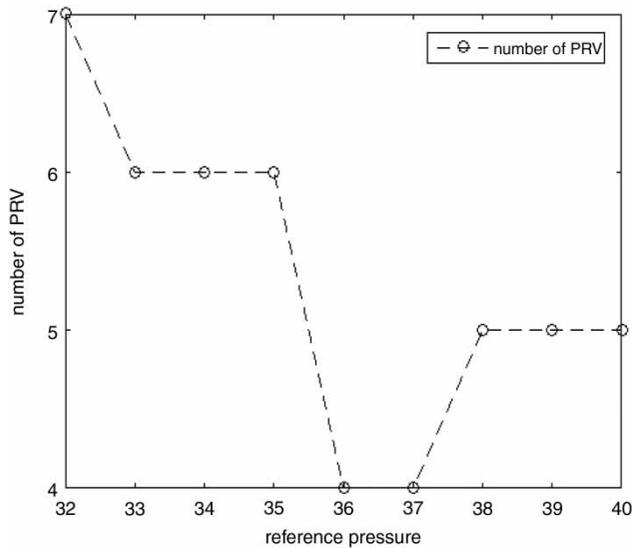
**Figure 6** | Numbers of PRV' verses. 'P$_{ref}$'.

**Table 2** | Comparison of leakage rate (l/s) after pressure management for different seasons

| After optimization | Autumn (l/s) | Summer (l/s) | Winter (l/s) | Spring (l/s) |
|---|---|---|---|---|
| **Initial leakage** | **29.3** | **28.5** | **29.6** | **29.5** |
| Tank water storage level optimization | 26.3 | 26.15 | 26.4 | 26.33 |
| Hydraulic failure index (after tank water storage level optimization) | 0.00425 | 0.00489 | 0.00412 | 0.00391 |
| After PRV (Pipe-1,31,18,40) | 23.2 | 23.05 | 24.2 | 23.4 |
| **% reduction of leakage** | **20.81%** | **19.1%** | **18.2%** | **20.67%** |

four PRVs are used for pressure management, which is lower than that used in previous literatures. The proposed methodology will reduce the capital investment required for pressure management. The time taken to calibrate the optimal operational value of PRV is 3–5 sec. Hence proposed algorithm can be implemented in real time. A vast reduction in pressure is observed after using PRV at pipe number 40, which is not placed in any earlier literature (as per authors' knowledge). This is due to the new rule (rule-2, Equation (19)) introduced for localization in this paper. This proves the robustness of proposed algorithm. Minimum required pressure (Equation (21)) of 30 m is maintained at every node to avoid pressure deficiency at any demand node.

## Case 2: Anytown network

The proposed algorithm is also applied to well-known Anytown network which is a benchmark by Walski et al. (1987). The system consists of 17 demanding nodes, 4 tanks, 3 pumps and 1 reservoir. System has base demand of 504.72 l/s. Water is supplied to consumers from reservoirs (through 3 different pumps 79, 78, and 80) and four tanks. Distribution of water only using reservoir will cause pressure deficiency in WDS, this will drastically increase the hydraulic failure index. There is need to supply water from storage tanks also. Four pumps (14, 15, 16 and 25) are used to fill these water tanks (one each), as modification of WDS as shown in Figure 7. Pump used here have flow variation from 0 to 250 l/s ($Q_{pump,max,i}$) with head variation from 120 m to 0 m. These pumps have pump curve such that pump will deliver flow of 126.18 l/s for head of 90 m given by:

$$Hb = 120.00 - 0.00189*Qb^2 \qquad (28)$$

The proposed leakage reduction methodology is applied to optimized network given and solved by Vamvakeridou-Lyroudia et al. (2005). To calculate initial leakage of WDS, tank water level is maintained around initial level throughout the day. Pressure driven analysis is performed using Equation (2). Using Equations (3) and (4), initial leakage rates have been calculated. WDS has average leakage rate of 81.4 l/s. Tank water storage level optimization (Equations (10)–(17)) is performed for the given WDS. Desire water storage level in the tank is taken, such that minimum pressure of 30 m will be present at every node in accordance with demand variation on an hourly basis. Optimized tank storage results obtained from proposed algorithm are implemented back in EPANET for verification of results. The optimized tank water storage level is shown in Figure 8. Leakage rate of WDS has been reduced to 72.4 l/s. There is a reduction in leakage rate of 11.05% is observed. From Figure 8 it can be observed that, at the end of the day the head of tank is either equal or more than initial head
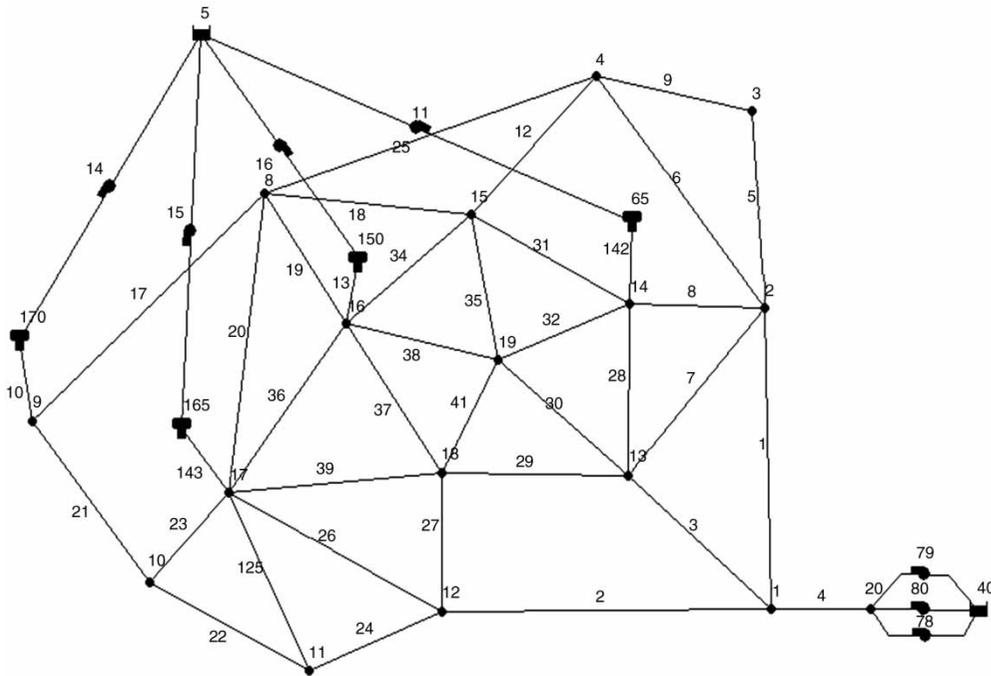
**Figure 7** | Modified Anytown network.

value of tank (12:00 AM). Minimum volume of water is always stored at each tank so that it can be used at a time of emergency operations like fire flow, if required. It is observed that nodes 9, 10 and 11 are the only nodes that remain under pressure for some period of time. Hydraulic failure index (using Equation (26)) corresponding to nodes

9, 10 and 11 are 0.0395, 0.0074, and 0.00169, respectively. Hydraulic failure index for overall WDS is calculated as 0.00283, which is quite less than allowed hydraulic failure index (<0.01).

It is observed that, after tank water storage level optimization some of the nodes still remain under high pressure,
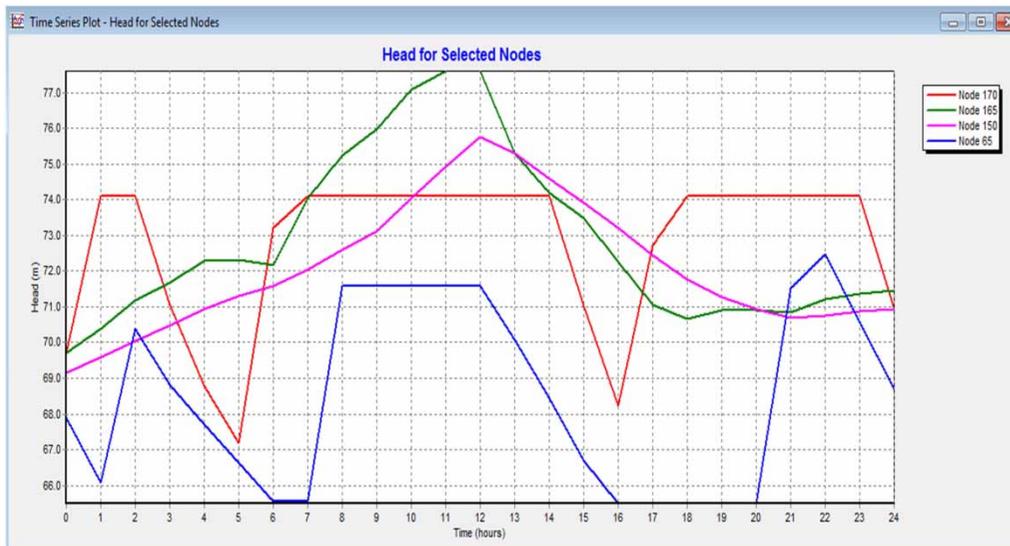


**Figure 8** | Hourly optimized water level in the storage tank of each tank using the proposed algorithm.

thus PRV has been installed in WDS to reduce this excess pressure which will lead to further leakage reduction. Number of candidate for value localization is selected using modified reference pressure algorithm, for average load condition. Value of reference pressure comes out to be as 38 m (Equation (18)). The number of PRV candidate after using Equation (18) comes out to be three. The number of PRV candidate after applying rule-2 (Equation (19)) is one. The maximum number of PRV candidate is restricted to 4 ($N_v$). PRV has been installed at the pipe no 1, 3 and 142. NSGA-II (Equations (20)–(25)) has been used to find out the optimized control setting across PRV for average load condition, which comes out to be 33.75 m. It is observed that the leakage rate is reduced to 59.82 l/s. There is a reduction in leakages of 26.51% is observed after using proposed optimization algorithm.

## CONCLUSION

This paper presents a leakage reduction technique for optimizing water storage level in the tanks along with optimized control and localization of PRV. Water storage level in the tank is maintained such that system will maintain minimum pressure of 30 m at each node. A new iterative mathematical algorithm is presented for optimal water storage level in the tank causing leakage reduction of 10–12% for both the case studies without adding any cost in infrastructure of WDS. Constrains such as maintaining minimum pressure at demanding node, emergency storage for fire flow, etc. is also considered during tank level optimization for providing efficient services. Proposed algorithm has shown successful results for multiple tank optimization, hence can be used in real world WDS. To make proposed algorithm more practically implementable, different demand pattern representing different season is also considered. PRVs have been used for further pressure reduction during low demand hours. Earlier proposed reference pressure algorithm is modified by introducing new rule (Equation (19)), which has revealed new location of PRV (Modified benchmark WDS) resulting in more favourable leakage reduction due to the reduction of surplus pressure present in the WDS. NSGA-II is implemented with an aim to determine (1) optimal control setting of PRV under

different load conditions and (2) minimization of leakages. There is reduction in leakage rate of 26.51% is observed for Anytown WDS, with hydraulic failure index of 0.00283 (during tank storage optimization), which is very much less than allowed hydraulic failure index (<0.01). Proposed algorithm is also implemented in another well-known modified benchmark WDS, results in leakage reduction of 6.1 l/s (20.81%), having 0.00429 as hydraulic failure index (during tank storage optimization). The time taken to calibrate the optimal operational value of PRV is 3–5 sec. Hence proposed algorithm can be implemented in real time. Pump operational time especially during summers can be reduced by using variable speed pump. Performing tank water level optimization before PRV installation provides much more cost effective and efficient solution for leakage reduction. The proposed system has shown successful results for small, medium water networks. Future research on its application in complex networks is warranted.

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