Leakage reduction in water distribution system using efficient pressure management techniques. Case study: Nagpur, India

Aditya Dinesh Gupta and Kishore Kulat

ABSTRACT

Leakages in water distribution system (WDS) are directly proportional to its operating pressure. Pressure management is becoming an important technique for reducing leakages in the water networks. This paper presents a pressure management technique for leakage reduction in north central WDS of Nagpur City, India, using variable speed pump and pressure reducing valves (PRVs). Variable speed pump is utilized for eliminating pressure deficiency during high demand and for reducing excess pressure causing leakage reduction during lower demand, by controlling the pump speed. PRVs have been used for further leakage reduction. This paper proposes a modified reference pressure algorithm for determining the location of valves in WDS. A multiobjective genetic algorithm (NSGA-II) is used to determine the optimized control value of pressure reducing valve with respect to change in demand pattern and to minimize the leakage rate in the WDS. Proposed pressure management technique leads to leakage reduction of 16.57% to 26.30% with respect to changes in demand pattern, causing daily average saving of 5.066 Ml. Minimum required pressure is maintained on every demand nodes to avoid pressure deficiency in WDS.

Key words | multiobjective genetic algorithm, pressure management, pressure reducing valves, water distribution system (WDS)

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INTRODUCTION

Water leakages come under physical water losses and it is the main source of non-revenue water losses in water distribution systems (WDS) (Lambert 2002; Gupta *et al.* 2016). Leakages occur due to poor pipeline connections, aged pipeline bursting when operating under high pressure, etc. Recently, on 29th May 2017, a sudden pipeline burst occurred in the WDS of Ukraine, which gathered attention across the world. The pipeline burst due to high pressure and left a 6 m hole into the ground. The burst also caused damage to the road, cars and houses, thus also causing economic losses. Similar incidence of pipeline bursts due to high operating pressure was also reported in Melbourne, Australia in the year 2012. Because of these leakages, extra water has to be pumped out, which increases energy consumption, consequently, causing economic losses (Girard & Stewart 2007). Leakages are directly proportional to its operating pressure of WDS (Lambert 2001). Several researchers and experts are focusing on pressure management for reducing leakages (Araujo *et al.* 2006; Dai & Li 2014; Gupta *et al.* 2017). Pressure management has also been cited as a key factor for leakage management by specialized committees from the International Water Association (Vicente *et al.* 2015). One of the main objectives of pressure management is the reduction of background leakages, which are hard to find and to eliminate. It also helps in extending pipeline infrastructure life through reduction of new main breaks (Vicente *et al.* 2015). Therefore, pressure management has become a crucial part of leakage management in WDS.

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Pressure management can be performed by pump scheduling, tank water storage level optimization, sectorization of the WDS by using isolating valves or by inserting flow and pressure controlling valves in pipeline networks (Jowitt & Xu 1990; Gupta *et al.* 2017). Advancement in science and technology leads to emergence of pressure reducing valves (PRVs), which has improved the pressure management process. PRVs have the capability to achieve high pressure reduction rate. Thus, PRVs have become an efficient tool for managing leakages in WDS. Determining the number, locations and optimal control setting with respect to the change in water demands is required during optimization of PRVs, so that WDS can supply water efficiently, also satisfying the necessary constraints.

In the last two decades, many researchers have attempted to optimize the leakages in water network using pressure management technique (Vairavamoorthy & Lumbers 1998; Babayan *et al.* 2007; Eck & Mevissen 2012). Mixed integrated nonlinear programing (MINLP) was introduced by Hindi & Hamam (1991) for controlling pressure in pipeline infrastructure. A method has been proposed for the localization of valves for controlling pressure, but proposed algorithm 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 efficiently optimize the valve localization (Dai & Li 2014). Dai & Li (2014) proposed a nonlinear program (NLP) to solve MINLP problems. MINLP is used for finding out optimal localization and control pressure value of PRVs in a WDS. Binary variables have been proposed to accelerate system performance. The results obtained using this algorithm is more favorable than Araujo *et al.* (2006). Dai & Li (2016) have extended its previous model (Dai & Li 2014) for calibrating optimal operational pressure valve of PRV using NLP algorithm. A new mathematical equation is proposed for defining head loss at PRV. This equation can be used to calculate head loss across PRV in all the modes (open, close and active). The application of proposed algorithm leads to leakage reduction of 5.72 L/s in EXENET WDS. However, proposed algorithm includes higher computational complexity.

Araujo *et al.* (2006) had used a pseudo-valve insertion method, in which PRV is inserted in the pipeline and hydraulic parameters are calculated using the genetics algorithm (GA). The meta-heuristic approach was used to determine the optimized value of PRV. The simulations were performed on small-scale WDS. In a real-world scenario, using this approach in complex pipeline infrastructure is difficult because of the presence of a larger number of variables and computational complexity. Liberatore & Sechi (2009) proposed a scatter-search meta-heuristic approach for valve location and optimization of the pressure value across the valve. A concept of reference pressure was introduced. This proved to be an efficient PRV location technique. A scatter-search algorithm was used to determine the optimized pressure control value for PRVs. Sometimes, the meta-heuristic approach may only be able to determine sub-optimal solutions which can lead towards false results.

GA is a population-based optimization technique, which does not require any simplification of a hydraulic model. Therefore, GA is preferred for operational (PRV location and control and pump optimization) and design optimization of the WDS compared to classical deterministic optimization techniques. Creaco & Pezzinga (2014) proposed a hybrid objective algorithm (GA and linear programing (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 PRV and leakage volume. The proposed algorithm is quite efficient compared with the GA. De Paola *et al.* (2016) presented a harmony search (HS) model to determine the optimum control setting across the valves. They demonstrated more favorable results using HS algorithm compared to GA for pressure management in the WDS.

Optimization of water systems requires multiobjective functions for determining the optimized value and location of PRV and to minimize the leakage rate. These objectives have attracted researchers (Farmani *et al.* 2005; Tanyimboh & Czajkowska 2018) to use multiobjective genetic algorithm (NSGA-II) for water leakage management. Nicolini & Zovatto (2009) used NSGA-II for reducing leakages in the WDS by using pressure management. NSGA-II was used to determine optimized valve locations. This algorithm uses a population size of 100 with 1000 generations. A higher reduction in the average leakage rate was observed compared with that in Araujo *et al.* (2006). Rathnayake (2015) has utilized NSGA-II algorithm for optimal control of urban sewer networks. The pollution load of sewer system and their corresponding treatment cost are minimized for the migrating downstream and upstream storm condition by finding out the optimal orifice openings for controlling the wastewater flow to the interceptor sewer. Rathnayake & Tanyimboh (2015) have presented a multi-objective evolutionary optimization algorithm for controlling of intermittent discharges from combined sewer systems to minimize the environmental impacts, but proposed algorithm is based on single-peaked runoff hydrographs. Rathnayake & Azamathulla (2017) have extended this NSGA-II based optimization algorithm for multiple consecutive storms and obtained sets of feasible control settings that are capable of handling any kind of storm. Rathnayake (2016) shows comparison of NSGA II with an alternative constraint handling approach known as SWMM for optimal control of combined sewer systems. It is concluded that NSGA II results in a smaller treatment cost solution than that of SWMM approach. Gupta *et al.* (2017) have proposed a hybrid tank storage water level and PRV optimization for pressure management in Anytown WDS. Three PRVs have been installed for reducing pressure in WDS. Optimized pressure values across PRVs are calculated using NSGA-II. The system shows leakage reduction of 26.08%. The proposed hybrid algorithm can be used only for WDS which uses water storage tanks for water distribution.

Our study delineates the problem of leakage optimization by managing optimal pressure using variable speed pumps and the PRVs in the north central WDS of Nagpur, India, considering different seasons. Depending upon the demand variation, variable speed pump is utilized for removing pressure deficiency during high demand and for reducing surplus pressure causing leakage reduction during lower demand. PRVs have been installed in WDS for further leakage control. The entire PRV optimization operation is divided into two parts: (a) calibration of the number of PRVs required along with their localization using proposed modified reference pressure algorithm; (b) NSGA-II is used to determine the optimized control pressure value of the PRV (first objective) and to minimize the leakage rate (second objective). EPANET (Rossman 2000) is used as a simulation tool along with MATLAB R2015a for calibration, which is performed on a desktop (Intel i7 processor with 16 GB RAM).

PROPOSED METHODOLOGY

In actuality, scenario leakage is expressed as an emitter flow in terms of operating pressure. Change in leakage rate (L0 to L1) can be analyzed by $L1/L0 = (P1/P0)^{\beta}$ (Lambert & Fantozzi 2010). β is the pressure exponential varies from 0.5 to 2.3. Thus, leakages can also be controlled by reducing undesired excess pressure in WDS. The aim of pressure management is to minimize the leakages and to maintain minimum required pressure at every demand node present in pipeline network. The objective of proposed algorithm is to perform efficient pressure management in WDS by using a variable speed pump and pressure reducing valve. The flow chart of proposed algorithm is shown in Figure 1.

Mathematical formulation

Equations used for calibrating hydraulic parameters of the WDS is based on the EPANET model (adopted from Nicolini & Zovatto (2009)), to which the optimization model has been coupled. The law of conservation for WDS is represented using sets of equations.

Continuity at node is expressed as:

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

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

In order to estimate a better flow through network (including demand and water losses), a pressure driven demand model (adopted from Mahmoud *et al.* (2017) and Giustolisi *et al.* (2008)) is used here.

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

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; $[P_{\mathrm{ser}}, P_{mi}]$ is the range of intermediate working condition when actual demand is $Q_{\mathrm{req,i.}}$; and P_{mi} is the minimum pressure at which demand supplied is 0. For the pressure-driven demand model, the value $P_{\mathrm{mi}} = 0$ m is applied at all nodes (Giustolisi *et al.* 2008).

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

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

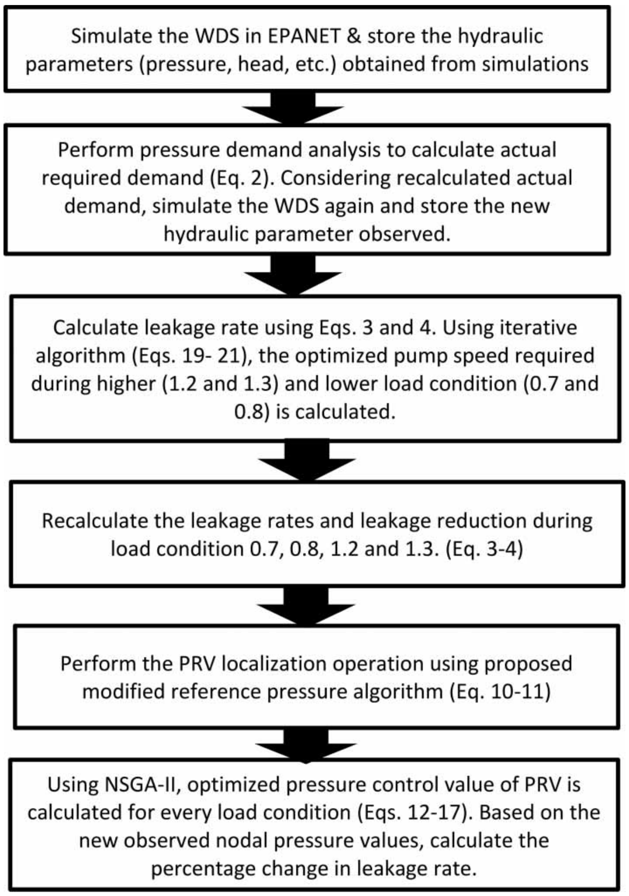

Figure 1 | Flow chart of proposed methodology.

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 γ is the leakage exponential used to define relation between flow from orifice and head difference. Leakage exponential value of 1.18 (Jowitt & Xu 1990) is observed for cracks in pipe or joints which occurs due to difference in pressure of internal and external pipe. Therefore authors have adopted same value i.e. 1.18 as leakage exponential for this study.

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

According to 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 measured at node i and j for load condition k.

Head loss for pipeline using flow values, can be calculated using 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 pipeline; l_{ij} is length of pipeline connected to nodes i and j; and $Q_{ij,k}$ is the flow across pipeline for load condition K.

Head loss across PRV, expressed as:

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

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

The limitations associated with Equation (7) are that, it cannot be used in the case of closed mode of PRV, when value of υ_{ij} becomes 0. To overcome this drawback, Dai &

Li (2016) have modified the existing model used for calculating head losses under presence of PRV. Adopting this model, $H_{ij,k}$ across PRV can be calibrated as:

$$H_{ij,k} = \max(0, \mathrm{H_{i,k}} - \mathrm{H_{j,k}}) \\ = 10.668\ C_{ij}^{-1.85} (v_{ij,k} d_{ij})^{-1.85} l_{ij} Q_{ij,k}^{1.852} \quad (8)$$

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

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

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

where Z_i represents elevation at node i.

Pressure reducing valve localization

PRV is installed in the WDS to control the pressure at the downstream end of the valve. PRV operates in three modes: active, closed, and open. The mode of operation depends on upstream and downstream pressures. Optimized localization of PRVs are desired for efficient pressure management.

Liberatore & Sechi (2009) have introduced the reference pressure algorithm to restrict candidate valve locations to a set of pipelines. Our study also uses the reference pressure technique for valve localization. Pressure is considered for an average load condition. Consider G is a set of the entire pipeline present in WDS. A subset $\mathrm{G_v}$ ($\mathrm{G_v} \in G$) is derived, which belongs to pipeline connected between nodes i and j as PRV candidate, is expressed as:

$$\text{Rule 1. if } \mathrm{N_j} > \mathrm{P_{ref}} \text{ and } \mathrm{N_i} < \mathrm{P_{ref}} \quad (10)$$

where $\mathrm{N_i}$ and $\mathrm{N_j}$ is the pressure at nodes i and j.

The reference pressure is selected during valve localization operation. The reference pressure varies over a defined pressure range to determine the different values of $\mathrm{G_{v,n}}$ ($\mathrm{G_{v,n}}$ represents the number of candidate valve locations for a current value of $\mathrm{P_{ref}}$). Pressure value corresponding to a minimum value of $\mathrm{G_{v,n}}$ is selected as the reference pressure.

The rule proposed (Equation (10)) by Liberatore & Sechi (2009) suffers the drawback that sometimes pressure difference between nodes $\mathrm{N_i}$ and $\mathrm{N_j}$ is slightly high. However, this location was not considered for PRV localization, because pressure is above $\mathrm{P_{ref}}$ for both the nodes. If able to cut this excess pressure between nodes $\mathrm{N_i}$ and $\mathrm{N_j}$, this will lead to a high reduction in leakage rate of WDS. To overcome this drawback, a new rule (Rule 2) is introduced to improve the PRV localization operation, with an idea behind it to also consider the pipeline connecting to nodes $\mathrm{N_i}$ and $\mathrm{N_j}$ as PRV candidate ($\mathrm{G_v}$), even though both the nodes have pressure value of more than $\mathrm{P_{ref}}$ and have a pressure difference (between $\mathrm{N_i}$ and $\mathrm{N_j}$) more than the predefined threshold value.

$$\text{Rule 2. if } \mathrm{N_j} - \mathrm{N_i} > 0.1 \times \mathrm{P_{ref}} \quad (11)$$

To make this threshold WDS dependable, the threshold ($\geq 0.1 \times \mathrm{P_{ref}}$) is decided in accordance with reference pressure of existing WDS. If we lower the threshold (5% of $\mathrm{P_{ref}}$) value, then number of PRV candidate will get increase. Similarly, if threshold is decided to be 20% of P_{ref}, then the number of PRV candidate can become zero. Thus, threshold is decided as 10% of reference pressure. Rule 2 is applied to WDS, after Rule 1, for localization of PRV candidate.

Multiobjective genetic algorithm for valve optimization

The optimal control setting of 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). The first objective (ƒ1) is to determine the optimized pressure value (P_{set}) of PRV, and is defined as:

$$\min f1 = \sum_{i=1}^{N_s} w_k C_L L_i \ \ P_{i,k}^{\gamma} \quad (12)$$

Subject to:

$$P_{\mathrm{i,k}} \geq P_{\mathrm{req}} \quad (13)$$

$$\mathrm{n_v} \leq \mathrm{N_v} \quad (14)$$

$$H_{i,j,k} = H_{i,k} - H_{j,k} \quad (15)$$

$$P_{\min} \leq P_{\mathrm{set}} \leq P_{\max} \quad (16)$$

where $P_{\mathrm{i,k}}$ is pressure at node i during load condition k; P_{req} is the minimum pressure that is required at each demand node; $\mathrm{n_v}$ represents number of PRV currently being used in WDS; $\mathrm{N_v}$ represents maximum number of PRV allowed to insert in WDS; $P_{\min}$ & $P_{\max}$ are the minimum and maximum value allowed across the PRV; N_s represents number of node present in the system; and W_{K} is the value of load condition (demand multiplier). The algorithm uses population of 50 for individual evolving for 200 generations. The crossover and mutation probabilities used for this optimization process are 0.65 and 0.002. The optimized value of PRV has been found out for one individual load condition at a time.

The second objective (f2) is to select the operational control value of PRVs (P_{set}) obtained from the first objective, which corresponds to the lowest leakage rate in WDS.

$$\min f2 = \sum_{i=1}^{N_s} C_i P_{i,k}^{\gamma} \quad (17)$$

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

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 PRV (P_{set}) changes, the value of the head and head loss across all 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 pressure for all the nodes corresponding to present value of P_{set} using Equations (5)–(9). The calibrated value of heads and pressure were used for calibrating f1 and f2.

RESULTS AND DISCUSSION

Case study: North central zone water distribution network, Nagpur

Proposed pressure management technique for leakage management is applied on the north central zone water distribution network of Nagpur city, India as shown in Figure 2. The WDS supplies water to the locality of 41.81 km^2, with usable command area of 25 km^2, covering total population of 7,42,399 persons. Average daily usage consumption from each consumer is 154.9 Lpcd. WDS has base demand of 1,331 L/s. Water is distributed from pumping station established at Navegaon Khairy reservoirs. Details of undertaken north central zone WDS provided by Nagpur Municipal Corporation (NMC) (Rathi 2012) is given in Figure 2.

System requires pump operation to supply water from reservoir to the distribution system. The pump installed in WDS is such that the pump will deliver the required flow of 1,331 L/s at the head of 55.43 m (Rathi 2012). The flow equation (represents pump curve) used here is:

$$\mathrm{Hb} = 73.91 - 0.00001043^{*}\mathrm{Qb}^{\wedge}2 \quad (18)$$

where Hb is the head and Qb is the flow rate from pump.

The pump installed in WDS has a flow rate variation from 0 to 2,665 L/s (Qb), with respect to head variations from 74 m to 0 m (Hb), respectively. Flow rate from pump changes with respect to head, hence there is a need to identify correct flow rate from the pump. For a given head, flow rate from pump can be calculated using Equation (18). Fixed global pump efficiency is used as pump efficiency curve.

Hydraulic simulations are performed and hydraulic parameters such as head, pressure at nodes, etc., are stored. Pressure driven analysis is performed using Equation (2). Standard EPANET results show (Giustolisi *et al.* 2008) P_{ser} of 10 m for all the nodes for supplying desired demand ($Q_{\mathrm{i,des}}$), with value of P_{mi} as 0 m. Required actual demand (Q_{req}) is recalculated using Equation (2). Depending on recalculated required demand for each node, the network is again simulated in EPANET and new pressure value for individual node is stored.

Three seasons, summer, winter and autumn, have been analyzed during this study. Demand pattern variation (demand multiplier) for every season is adopted according to data provided by NMC (Rathi 2012). Average demand pattern for winter, summer and autumn seasons are 1,197.9 L/s (0.9), 1,437.48 L/s (1.08) and 1,331 L/s (1), respectively, as shown in Figure 3.

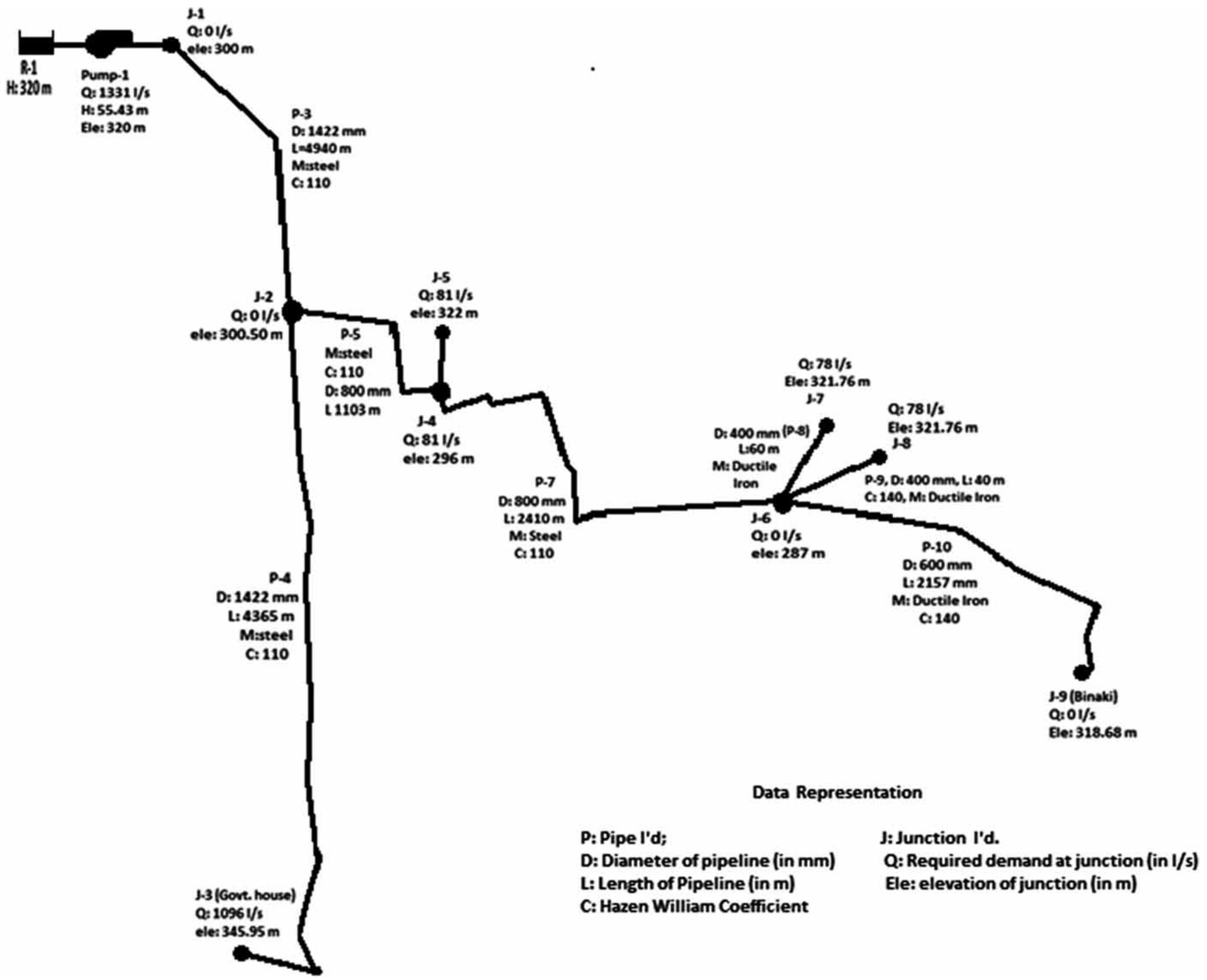

Figure 2 | North central zone water distribution network Nagpur, adapted for pressure management.

The system has physical losses of 20%, i.e. 266.2 L/s for base demand (average) of 1,331 L/s (Rathi 2012). For leakage calculations during other load conditions, C_L is calculated from Equation (3), which comes to be 1.23×10^{-4}. Leakages have been calculated for different load condition and season using Equations (3) and (4). Leakage rate for individual load condition can be identified from Table 1. Considering seasonal demand pattern, calibrated leakage rate for autumn, summer and winter are 266.15 L/s, 287.45 L/s, and 239.58 L/s, respectively. According to the regional water network design standards, minimum pressure (P_{req}) of 20 m is required at every demanding node.

During lower loads condition, i.e. 0.7 and 0.8, there is extreme increase in pressure at every node due to higher flow rate from pump. Variable speed pump can be seen as one of the solution to control the flow from the pump by reducing the pump speed, depending upon the required demand. The variable speed pump's motor is coupled with variable frequency driver to get variable speed. The relation between speed, flow and head of variable speed pump is known as the Affinity Laws (Rossman 2000), which is defined as:

$$\frac{N1}{N2} = \frac{Q1}{Q2} \text{ and } \frac{HI}{H2} = \left(\frac{N1}{N2}\right)^2 \tag{19}$$

where *N1* and *N2* are the speed of the pump for given flow *Q1* and *Q2* at head *H1* and *H2*.

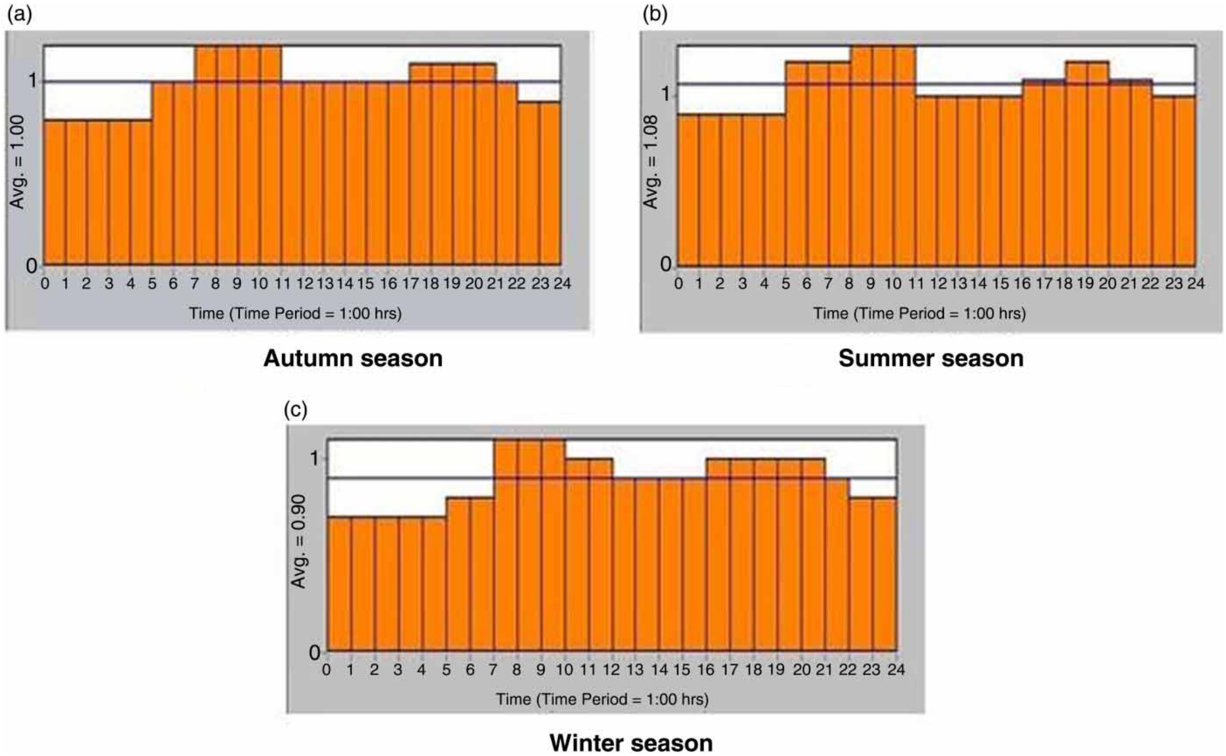

Figure 3 | Demand pattern of WDS for different seasons.

Pump speed is increased or decreased depending upon the required flow using Equation (19). The pump has flow rate variation from 0 to 2,665 L/s (Qb), with respect to head variations from 74 m to 0 m (Hb), respectively. The pump speed is increased (Equation (20)) or decreased (Equation (21)) with a speed variation of ±0.01 (ΔN), is defined as:

While increasing pump speed:

$$N_{req} = N + \Delta N \quad \text{if} \quad Q_{pump} < Q_{req,pump} \text{ at } H_{req} \tag{20}$$

Table 1 | Leakage rate (L/s) for different load condition

Demand multiplier (k)	Initial leakage rate (L/s)	After using variable speed pump is used (L/s)
0.7	316.085	244.05
0.8	301.04	241.77
0.9	284.15	
1	266.2	
1.1	245.28	
1.2	Pressure deficiency at Junc 3	258.003
1.3	Pressure deficiency at Junc 3	248.45

While decreasing pump speed:

$$N_{req} = N - \Delta N \quad \text{if} \quad Q_{pump} > Q_{req,pump} \text{ at } H_{req} \tag{21}$$

where N is the current pump speed; N_{req} is the final required pump speed; Q_{pump} is the current pump flow and $Q_{req,pump}$ is the required flow from the pump at required head (H_{req}). H_{req} is 55.43 m for this case study. This iterative process will be continued until the pump flow attains the required flow at desired head (H_{req}).

During load condition 0.8 the required demand is 1,064.8 L/s at head 55.43 m. Utilizing Equation (21), after first iteration the pump speed decreased to 0.99. Using the Affinity Laws (Equation (19)), the pump head variation becomes 72.43 to 0 for flow variation of 0 to 2,638 L/s. Pump flow becomes 1,278 L/s at head of 55.43 m, which is more than desired, thus speed is further reduced (Equation (21)) until the pump attains the required flow, i.e. 1,064.8 L/s at head 55.43 m. After certain iterations, the pump speed comes out to be $N = 0.91$ which also satisfies the desired condition. Pump curve at $N = 0.91$ is represented as:

$$\text{for } N = 0.91 \quad Hb = 61.24 - 0.00001044 * \left(Qb^{\wedge}2\right) \tag{22}$$

Similarly, during the load condition of 0.7 calculated pump speed (Equation (21)) comes out to be 0.89. The system will maintain the minimum required pressure (P_{req}) at the demand nodes for corresponding demand variations.

During load condition of 1.2 and 1.3 the pump fails to deliver the required flow, as a result pressure at junction 3 is reduced to 16.3 m and 9.61 m, respectively. This causes pressure deficiency in the WDS (<20 m). Utilizing Equations (19) and (20), desired pump speed during load condition of 1.2 and 1.3 comes out to 1.05 and 1.07 times, compared to its regular speed. This has eliminated the pressure deficiency at junction 3. If speed is further increased to 1.2 or 1.5 times its regular speed during higher load conditions of 1.2 and 1.3, then nodal pressure gets increased. This will also increase the leakage rate and can increase the bursting probability of pipelines in the network. The pump curve for variable speed pump can be identified from Figure 4.

The equations for pump curves during various speeds are defined as:

$$\text{for } N = 0.89 \quad Hb = 58.40 - 0.00001042 * \left(Qb^2\right) \tag{23}$$

$$\text{for } N = 1.05 \quad Hb = 81.28 - 0.00001041 * \left(Qb^2\right) \tag{24}$$

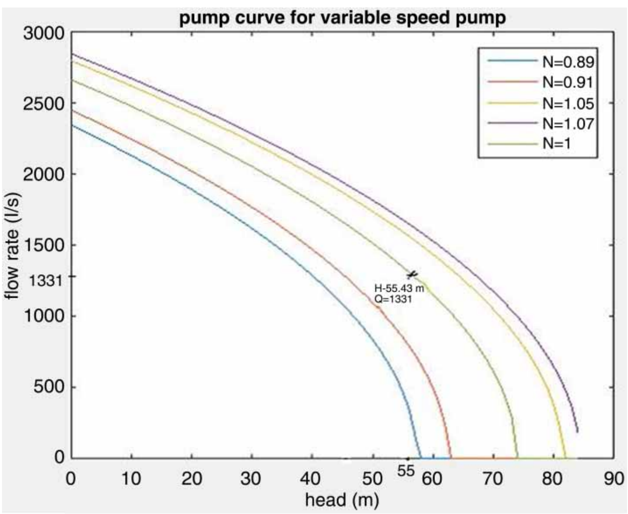

Figure 4 | Pump curve for variable speed pump.

$$\text{for } N = 1.07 \quad Hb = 84.41 - 0.00001041 * \left(Qb^2\right) \tag{25}$$

After using variable speed during load condition of 0.7, 0.8, 1.1 and 1.3, leakage rate has been recalculated using Equations (3) and (4). Recalculated leakage rate can be identified from Table 2. After using variable speed pump during load conditions 0.7 and 0.8; there is reduction in surplus pressure by 138.27 m and 114.3 m, respectively. The leakage rates have been reduced from 316.085 L/s and 301.04 L/s to 244.05 L/s and 241.77 L/s during load conditions of 0.7 and 0.8, respectively. The value of leakage rate lessens during load conditions of 0.7 and 0.8 when compare to 1 and 0.9. Therefore, in winter, the leakage rate is less when compared to the autumn season, even though the demand is less. Similarly, in summer, during higher load conditions (1.3 and 1.2), the pump speed is increased causing more pressure in WDS, which leads to increase in leakage rate during this season.

Even after use of variable speed pump, some of the nodes still remain under high pressure. Time controlled PRVs have been installed to reduce the excess pressure present in the WDS. PRV locations are calculated for average value of load condition (i.e. 1) during the autumn season using proposed modified reference pressure algorithm, i.e. Equations (10) and (11) (Rules 1 and 2). 20 m is the minimum required pressure hence initial value of P_{ref} is selected as 20 m and varies until it doubles, i.e. 40 m. The number of PRV location comes out to be zero, after using only Rule 1 (Equation (10)), proposed by Liberatore & Sechi (2009). Their proposed algorithm fails to localize any PRV for this WDS. To remove this drawback associated with Rule 1, Rule 2 (Equation (11)) has been applied for finding out any additional PRV localization. Two locations, i.e. pipes 3 and 5 are observed as PRV candidate, which can be identified from Figure 5. The value of the number of PRV candidate is 2, which remains the same throughout the process (Figure 5); therefore, any pressure value that lies between 20 and 40 m can be selected as P_{ref}. In this case we have used 30 m as P_{ref}. The maximum value of the number of PRV is fixed to 2 (N_v). The proposed modified reference pressure algorithm used for localization of valves is computationally simple when compared to GA (Araujo *et al.* 2006) and MINLP (Dai & Li 2014).

Table 2 | Optimal pressure value of PRV (P_{set}) in meters, obtained after applying proposed algorithm considering different load conditions

PRV no.	0.7	0.8	0.9	1	1.1	1.2	1.3
1 (P-3)	63.8 m	65.0 m	65.12 m	65.79 m	66.52 m	67.62 m	68.52 m
2 (P-5)	46.10 m	46.14 m	46.20 m	46.27 m	46.34 m	46.45 m	46.51 m

NSGA-II is used here for finding out optimized operational pressure value of PRV using two objective functions and constrains mentioned in Equations (12)–(17). The system takes hydraulic parameters such as nodal pressure, head, etc., as input. The operational control pressure value of PRV (P_{set}) will vary between 20 m (P_{min}) to 72 m (P_{max}) which is the minimum and maximum value of pressure observed at nodes of WDS. Hence the optimize pressure setting across PRV will definitely be the intermediate vales between P_{min} and P_{max}.

GA uses random population for the generation of optimal solution. Due to random population generation every time GA generates different optimal solution. The window is already provided for population generation, which causes minimization in variation of final optimal solution (within ±0.15 m). Hence, NSGA-II runs thrice and the optimal solution is selected by calculating the average of three solutions. Every change in optimal control setting (P_{set}) causes change in nodal pressure. It is not possible to calibrate this change in EPANET. A function, Head_cal (P_{set}), is used within the NSGA-II to calculate pressure (using Equations (5)–(8)) at each node, corresponding to current operational control pressure value of PRV (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*). The optimal operational pressure value across PRVs during different load condition is given in Table 2.

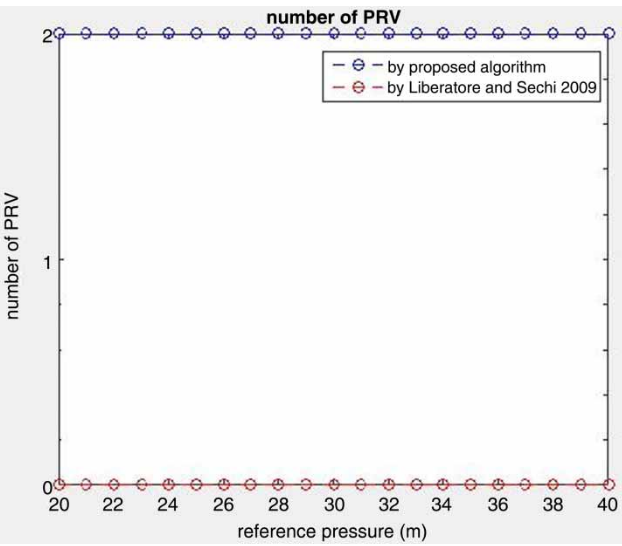

Figure 5 | Numbers of PRV' versus 'P_{ref}'.

After pressure management, reduction in leakage rate during different load conditions varies from 16.57% (40.7 L/s) to 26.30% (74.74 L/s), which can be identified from Table 3. Higher leakage reduction is achieved due to reduction of surplus pressure on every load condition after using variable speed pump and two PRVs. The proposed algorithm leads to leakage reduction of 20.47% (57.49 L/s), 23.35 (55.95 L/s) and 22.3% (59.45 L/s) during summer, winter and autumn seasons, respectively.

It can be observed from the result that during load condition of 0.7 and 0.8 the leakage rate after pressure management is lower than that of 0.9, 1 and 1.1. This is due to the fact that use of variable speed pump during the load condition of 0.7 and 0.8 has already lower down the leakage rate; therefore, even though the demand is less, still lower leakage rate is achieved. Thus, during winter, even though the demand is less, the reduction of leakage rate is less than in autumn. Similarly, during summer, due

Table 3 | Reduction in leakage rate (l/s) after insertion of PRV in WDS

Load factor (k)	Leakage rate (L/s) after pressure management	% reduction	Surplus pressure reduction (m)
0.7	200.42	17.88	157.25
0.8	201.70	16.57	152.13
0.9	209.41	26.30	223.99
1	206.14	22.56	190.9
1.1	204.56	16.60	150.42
1.2	213.16	17.39	174.34
1.3	207.18	16.61	156.19

to the use of variable speed pump to avoid pressure deficiency during higher load conditions (1.3 and 1.2) has increased the pressure, so as the leakage rate in WDS. Therefore, even though the demand is high in summer, more leakage reduction is observed as compared to that in autumn.

The time taken to calibrate the optimal operational pressure value of PRV, considering single load condition at a time is 4–6 s. Hence proposed algorithm can be implemented in real time. A vast reduction in pressure is observed after using PRV at pipes 3 and 5, which is due to the additional rule proposed as a modified reference pressure algorithm (Rule 2, Equation (11)) for localization in this paper, removing earlier drawback of Liberatore and Sechi (2009). This proves the robustness of the system. Minimum required pressure (Equation (13)) of 20 m is maintained at every demand node to avoid any pressure deficiency in the water network.

CONCLUSION

Our study presents a pressure management technique for leakage reduction in North-central WDS, Nagpur, India. Variable speed pump has been utilized for eliminating pressure deficiency during higher water demand, although this has increased the leakage rate in WDS. During lower load condition decrease in pump speed leads to reduction of access pressure by decreasing the pump speed. This has also lowered the leakage rate. Variable speed pump can be seen as effective tool for efficient pressure management. Pressure reducing values (flow-controlled) are installed for further pressure reduction. Two rules were implemented (Equations (10) and (11)) for localization of PRVs known as modified reference pressure algorithm. Existing reference pressure algorithm (Equation (10)) is unable to find any valve location (Liberatore & Sechi 2009). The newly proposed Rule 2 (11) has overcome this drawback and has revealed two locations of valves (i.e. P-5 and P-3). Proposed modified reference pressure algorithm used for localization of valves is computationally simple when compared to GA (Araujo *et al.* 2006) and MINLP (Dai & Li 2014). NSGA-II (12–17) is implemented with an aim to determine (a) optimal pressure value of PRVs under different load conditions and (b) minimization of leakages in WDS. Minimum required pressure (P_{req}) is maintained at every demanding node to avoid pressure deficiency (lower pressure than desired), thus allows water network to provide water distribution services efficiently. Proposed pressure management model leads to leakage rate reduction variation from 16.57% (40.7 L/s) to 26.30% (74.74 L/s) considering different load condition. There is leakage reduction of 20.47%, 23.35% and 22.3% observed during the summer, winter and autumn season, respectively. There is average leakage reduction of 30.34 L/s. This leakage reduction causes daily water saving of 5.066 mL, which can feed 32,689 people more than the existing population in WDS. For this case study, pressure driven analysis is perform separately (Equation (2)) as EPANET 2 is not capable to perform such analysis. EPANET-PDX (Siew & Tanyimboh 2012) is pressure driven extension of EPANET 2, which can be utilized in future for performing pressure driven analysis of WDS. EPANET-PDX is more efficient and accurate than EPANET 2 while analyzing both normal and pressure deficient conditions. This model can also perform both steady state and dynamic hydraulic simulations. EPANET-PDX (Seyoum & Tanyimboh 2017) is also capable of performing water quality modeling of WDS. Tanyimboh & Seyoum (2017) have proposed parallel evolutionary optimization algorithms by dividing the fitness evaluations equally among the eight processors. The adaption of proposed parallel algorithm can decrease the execution time by 15 times when compared to serial operation. Thus, such parallel evolutionary optimization algorithms can be used in future for speeding up the optimization process. The proposed system showed successful results for medium water networks. Future research on its application in complex networks is warranted.

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