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Improvement of plumbing systems performance using looped water pipe networks within buildings

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ABSTRACT

In plumbing water systems (PWSs), there are two main layouts for water pipe networks (WPNs) within buildings: branched and looped layouts. The aim of this study is to introduce an approach for improving the performance of PWSs using looped water pipe networks within buildings. This approach is applied to a case study of branched configuration that is firstly changed to a looped configuration with the same pipe diameters, secondly to a looped configuration with the same discharge and velocity, and thirdly to the same branched configuration with double the pipe diameters. All cases were analyzed for cost and hydraulic reliability in terms of resilience index, minimum surplus head, failure index and demand node's flow path. EPANET software was used for hydraulic analysis of all cases. It is concluded from this study that replacing branched configuration of PWSs by looped configuration improves the hydraulic reliability and minimizes the costs of the PWSs.
Key words | branching, costs, looping, pipe-sizing, plumbing water systems, reliability

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INTRODUCTION

Water pipe networks (WPNs) supply water from a water source or water treatment plant to the consumer's water service pipe. Plumbing water systems (PWSs) run from the water service pipe to all interior plumbing fixtures in the building. Both WPNs and PWSs can be defined as groups of multiple interconnected components (pipes, fittings, valves, etc.) arranged in different layouts and configurations. Branched and looped configurations are the main layouts for WPNs and the first is commonly used for PWSs within buildings.

While branched layouts are quite simple and have acceptable investment costs, they have low reliability, which affects all consumers located downstream of any breakdown in the pipe system, and fluctuating water demand that results in large pressure variations. This is different in looped layouts, where consumers can be supplied following many paths.

Mechanical failure can be described as system failure due to pipe breakage, pump failure, and power outages, etc. Using the troubleshooting manual for detecting, identifying, studying and solving the problems that occur in a water distribution system is essential to avoid the mechanical failure of the system and reduce the annual maintenance cost (Kanakoudis 2004a, 2004b).

Hydraulic failure can be described as system failure due to inadequate distributed flow and pressure head at one or more demand nodes. System reliability means keeping a constantly open path between the demand nodes and the supply nodes. For each demand node, these paths are independent (Kanakoudis & Tolikas 2002a; Metin 2003).

Looped layouts improve the hydraulic reliability of the pipe systems. This is very important, especially whenever a supply pipe is out of order due to cleaning or repairs (Trifunović 2002). While looped layouts are expensive and more complex to design than branched ones, the higher reliability of the looped layouts can compensate for the increase in their costs (Banosa *et al.* 2009).

Hydraulic reliability is an indicator of the performance of the water pipe network. It can be defined as the ability of the network to sufficiently meet water demands at

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adequate pressures. The estimation of hydraulic reliability for the water distribution network is subject to uncertainty due to variations in future water demand, required pressure heads, and pipe roughness (Bao & Mays 1990).

In 1977 Kapur and Lamberson were the first to calculate the hydraulic reliability of a network, when its different parts are placed in series or in parallel, using the Kapur and Lamberson equations. In 2003 Prasad *et al.* used genetic algorithm methods and introduced 'network resilience' as a new reliability index. Kanakoudis & Tsitsifli (2011) proved that the Discriminant Analysis and Classification (DAC) method is a useful tool for evaluating a water pipe network's reliability considering all pipe characteristics. Kanakoudis *et al.* introduced a new methodology concerning the hierarchical analysis of the protective maintenance actions in WPNs considering repair or replacement cost and the possible failure impacts to satisfy demand using alternative paths. Kanakoudis *et al.* presented a simulation cost-based optimization and risk analysis process using the Pipe 2000 and the EPA net codes, aiming for the minimum operating cost (Kanakoudis & Tolikas 2002a, 2002b, 2002c; Kanakoudis 2004a, 2004b; Kanakoudis & Tsitsifli 2011; Tsitsifli *et al.* 2011).

The aim of this study is to present an approach for improving the performance of plumbing systems using looped configuration layouts instead of branched ones, which increases the hydraulic reliability and minimizes the costs of the system. This approach was applied to a case study (Harris 1990) of a branched layout configuration as a reference. The case study configuration is changed three times to examine the reliability and corresponding costs for each variation. It may first be changed to a looped layout with the same pipe diameters (Loop I), secondly to a looped layout with the same flow and velocity (Loop II), and thirdly to the same case study configuration with double the pipe diameters (Harris 2D).

The above cases were analyzed for costs and hydraulic reliability in terms of resilience index, minimum surplus head, failure index and demand node's flow path. An EPANET hydraulic simulator was used for the hydraulic analysis of all cases. The head-dependent analysis (HDA) approach was applied, which provides even more realistic results when the case study system operates under subnormal pressure conditions.

THEORETICAL APPROACH

The following objective cost equation is used for the optimum design of a pipe network with a pre-specified layout.

$$\text{Min. } C_o = \sum_{i=1}^{N} C_i\, L_i \tag{1}$$

in which N is the number of pipes; L_i is the total length of pipe i in the network; C_i is the cost of the pipe, with certain diameter and material per unit length and C_o represents the total cost of the pipes in the network. The total cost of the network should be reduced as much as possible under an acceptable level of reliability that withstands the adverse conditions. The above optimal design is subjected to the following constraints.

Hydraulic constraints

The mass and energy conservation constraints at junctions and pipes, in the plumbing network inside the building, in terms of flow rate and head losses can be defined as follows:

$$\sum_{i\in\, in\,(k)} q_i - \sum_{i\in\, out\,(k)} q_i = Q_k \; k = 1, \ldots\ldots\ldots, J \tag{2}$$

$$\sum HL_i = 0 \; i = 1, \ldots\ldots\ldots\ldots, N \tag{3}$$

$$HL_i = \mu L_i \left(\frac{q_i}{CH_i}\right)^{\beta} d^{-\gamma} \tag{4}$$

where J and N are the number of existing junctions and loops in the system, respectively; q_i is the flow rate in pipe i; L_i is the length of pipe i; Q_k is the required discharge at demand node k; HL_i is the head loss in i^{th} pipe; CH_i is the Hazen-Williams coefficient for i^{th} pipe; and $\beta = 1.85$, $\gamma = 4.87$, $\mu = 10.5088$ for q in cubic metres per hour, and L, d in metres (Savic & Walters 1997; Boulos *et al.* 2000).

Nodal head and pipe flow velocity constraints

The pressure head and the flow velocity range constraints at junctions and pipes, in the plumbing network inside the building, can be defined as follows:

$$H_{min} \le H_k \le H_{max}\ k = 1, \ldots\ldots\ldots, J \quad (5)$$

$$V_{min} \le V_i \le V_{max}\ i = 1, \ldots\ldots\ldots, N_p \quad (6)$$

where J and N_p are the numbers of existing junctions and pipes in the pipe system, respectively; H_k is the nodal head; H_{min} and H_{max} are minimum and maximum allowable nodal heads; V_i is the pipe flow velocity; and V_{min} and V_{max} are minimum and maximum allowable flow velocities.

Pipe size availability constraints

The pipe diameters should be commercially available which can be defined as follows:

$$d_i \in d\ i = 1, \ldots\ldots., N_p \quad (7)$$

where N_p is the number of pipes in the existing network; d_i is the diameter of pipe I; and d denotes the set of commercially available pipe diameters.

Demand node flow path constraints

The reliability of a demand node can be defined as the number of independent flow paths from the source nodes to the demand nodes which can be expressed as follows:

$$R_k \ge \bar{R} \quad (8)$$

where R_k is the reliability of the demand node k and $\bar{R}$ is the required level of reliability of the optimal network.

HYDRAULIC RELIABILITY EVALUATIONS

Penalized objective function

A penalty method is commonly used to introduce the optimal design of a pipe system as an unconstrained optimization problem. Including pressure head, in which head and discharge constraints in the objective function, leads to a new problem defined by the minimization of the following penalized objective function subject to the constraints defined in Equation (7).

$$C_p = \sum_{i=1}^{N_p} C_i L_i + \alpha_p \left\{ \sum_{i=1}^{N_p} (V_i/V_{min} - 1)^2 + \sum_{i=1}^{N_p} (V_i/V_{max} - 1)^2 + \sum_{k=1}^{J} (H_k/H_{min} - 1)^2 + \sum_{k=1}^{J} (H_k/H_{max} - 1)^2 \right\} \quad (9)$$

where α_p is the penalty parameter equal to a high value when the constraints are violated, and equal to zero when the constraints are not violated. Here, the cost of the most expensive pipe system is used as the penalty parameter. The hydraulic constraints are satisfied by using a simulation program that solves the set of hydraulic constraints for nodal heads (Afshar 2001).

Including the demand node flow path constraints expressed in Equation (8) into the penalized objective function of Equation (9) leads to the final form of the objective function (Afshar 2005).

$$\text{Minimize}\, C_j = C_p + C_r \quad (10)$$

with

$$C_r = \alpha_p \sum_{k=1}^{J} (\bar{R} - R_k)^2 \text{ if } R_k < \bar{R} \text{ and } C_r = 0 \text{ otherwise} \quad (11)$$

where α_p is the cost of the most expensive network.

Resilience index (I_r)

Todini (2000) proposed the resilience index based on the concept that the power input into a network equals the power lost internally to overcome friction plus the power that is delivered to the demand nodes (Todini 2000). The resilience index (I_r) of PWSs can be defined as:

$$I_r = \frac{\sum_{j=1}^{n_d} (Q_j^{avl} H_j) - \sum_{j=1}^{n_d} (Q_j^{req} H_j^{des})}{\sum_{k=1}^{n_r} Q_k H_k - \sum_{j=1}^{n_d} (Q_j^{req} H_j^{des})} \quad (12)$$

where Q_k and H_k are the discharge and head, respectively, relevant to each reservoir k, and n_r is the total number of reservoirs connected to the PWSs, Q_j^{req} and H_j^{des} are the required demands and the head desired at the demand nodes j, and n_d is the total number of demand nodes with the real flow (Q_j^{avl}) draws-off, and real head (H_j).

Minimum surplus head (I_m)

The minimum surplus head (I_m) is defined as:

$$I_m = \min\{H_j - H_j^{des}\}\ j = 1, 2, \ldots\ldots\ldots.., n_d \tag{13}$$

This surplus head indicates the available energy for dissipation during failure conditions with regards to distributed flow and pressure head. Maximization of the available surplus head at the most depressed demand node improves the reliability of the PWSs to some extent.

Failure index (I_f)

Under failure conditions of PWSs, the provided power at some demand nodes may decrease to a value less than the requested power to satisfy the design demand. In this study, the constraint violation for any solution of a PWS can be calculated using a failure index (I_f) which can be defined as (Gad & Abd-Elaal 2016):

$$I_f = \frac{\sum_{j=1}^{n_d} e_j}{\sum_{j=1}^{n_d} (Q_j^{req} H_j^{des})} \tag{14}$$

$$e_j = \begin{bmatrix} 0 & \forall_j : Q_j^{avl} H_j \geq Q_j^{req} H_j^{des} \\ (Q_j^{req} H_j^{des} - Q_j^{avl} H_j) & \forall_j : Q_j^{avl} H_j < Q_j^{req} H_j^{des} \end{bmatrix} \tag{15}$$

CASE STUDY

Reference example of Harris branched configuration (Harris)

Harris (1990) provides an example of a typical plumbing water distribution system of a one-family residential home. Figure 1

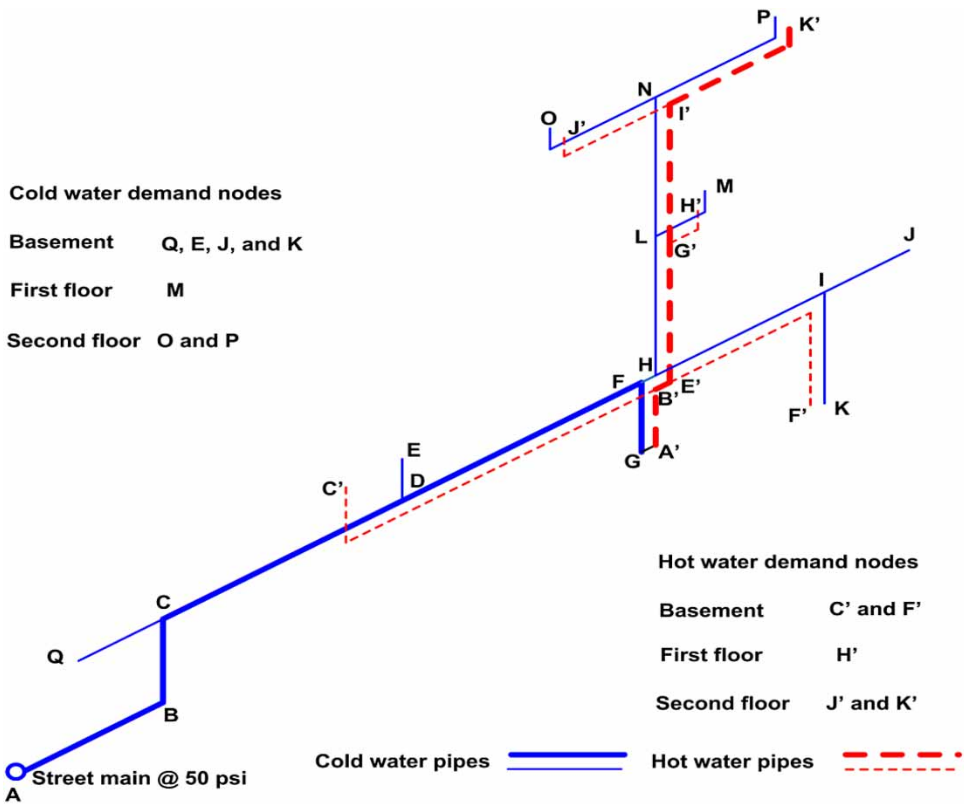

Figure 1 | Typical one-family residential plumbing system (Harris 1990).

represents a schematic based on Harris's example (Harris 1990). The street main is assumed to provide daily service pressure of 50 psi (344.7379 kPa) as in Harris (1990). The system includes fixture groups located on three floors (basement, first floor, and second floor). Cold water pipes are designated by solid blue lines with cold water demand nodes Q, E, J, and K on the basement floor; M on the first floor; and O and P on the second floor. Hot water pipes are designated by dashed red lines with hot water demand nodes C' and F' on the basement floor; H' on the first floor; and J' and K' on the second floor.

A worst-case loading scenario upon the system with all fixtures operated simultaneously was introduced. Although this situation is not realistic, it does provide a basis for analyzing the behavior of the system (Ladd 2005). An EPANET hydraulic simulator program was used for hydraulic analysis of Harris's example plumbing water network and the other looped and branched cases explained below.

A pressure-driven solution approach was taken with the use of emitter coefficients to represent the pressure-dependent nature of plumbing fixtures. Emitter coefficient values (k) were derived for each fixture group. K-values are based on the minimum required pressure for the group to supply the required flow according to the International Plumbing Code (IPC 2000) and it can be calculated from (Ladd 2005):

$$K = C_d\, A \sqrt{\frac{2g}{\gamma}} \tag{16}$$

where C_d is the flow rate coefficient, A is the pipe cross-sectional area, g is the acceleration due to gravity and γ is the unit weight.

Conceptual looped configuration (Loop I)

As shown in Figure 2, a conceptual looped (Loop I) configuration is introduced in which the main supply pipeline (CDFHI) on the basement floor level is replaced by the loop of pipes ($Cn_1n_2Dn_3Fn_4Hn_5n_6In_7$) with an equivalent entrance area and the same discharge flow at the same velocity as in the Harris example.

$$Q_{branch} = A_{branch}\, V_{branch} = A_{loop\ I}\, V_{loop\ I} \tag{17}$$

$$A_{branch} = A_{loop\ I} \tag{18}$$

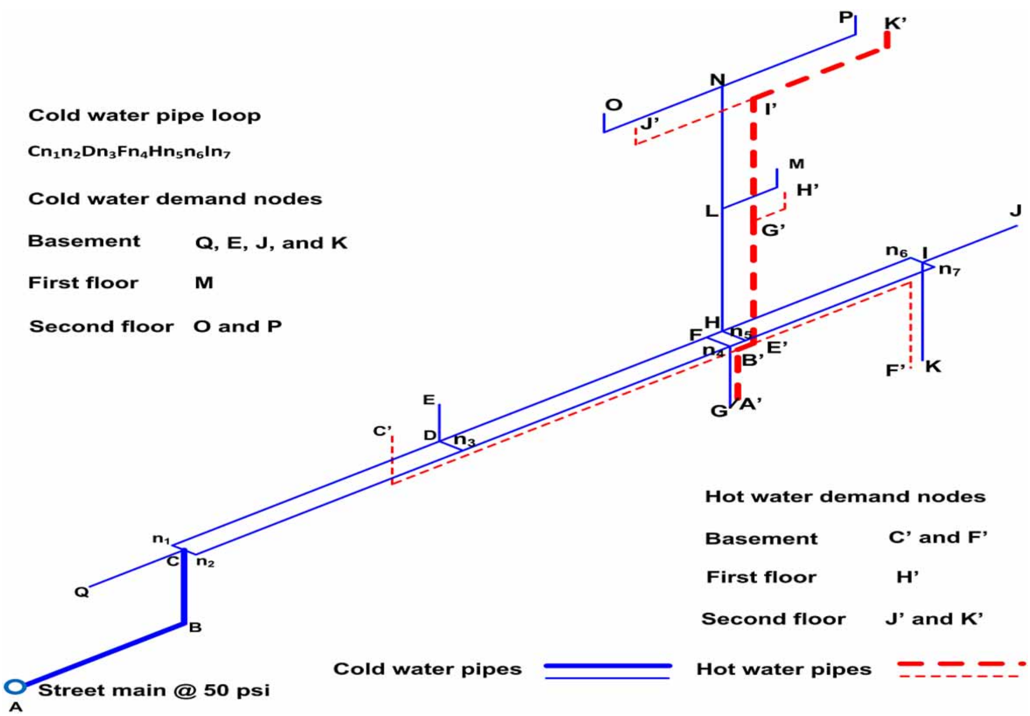

Figure 2 | Looped I configuration layout.

Table 1 | Values of I_r, I_m, and I_f for the four configurations

	I_r			I_m (m)			I_f		
Layout	**Basement**	**First**	**Second**	**Basement**	**First**	**Second**	**Basement**	**First**	**Second**
Harris	0.443	0.4	0.0	10.51	7.49	−4.26	0.0	0.0	0.223
Loop I	0.339	0.355	0.0	7.39	5.76	−5.77	0.0	0.0	0.281
Loop II	0.658	0.5702	0.0	15.23	9.79	−2.56	0.0	0.0	0.121
Harris 2D	0.76	0.605	0.0	17.66	9.71	−2.62	0.0	0.0	0.082

where Q, A, and V are flow rate, pipe cross-section areas, and flow velocity, respectively.

Harris branched configuration double supply pipe diameter (Harris 2D)

This is the same configuration as shown in Figure 1 but the main supply pipe on the basement floor level is replaced by a pipe with a diameter (D) equal to twice the original Harris diameter.

$$D_{Harris\ 2D} = 2D_{Harris} \tag{19}$$

Conceptual looped configuration (Loop II)

This is the same configuration as shown in Figure 2 but the looped pipe diameters are set equal to twice the original Harris diameter.

$$D_{loop\ II} = 2D_{Harris} \tag{20}$$

RESULTS AND DISCUSSION

Table 1 shows the values of I_r, I_m, and I_f for the four configurations considered in this study for the three floors: basement, first floor, and second floor.

These results are shown in Figures 3–6 and are discussed in the following sections.

Resilience index (I_r)

Figure 3 represents comparisons of resilience index (I_r) values for the three floors among the four alternative configurations: Harris, Loop I, Loop II, and Harris 2D. It is clear from this figure that, for the three floors, the value of resilience index (I_r) is highest for the Harris 2D configuration and lowest for the Loop I configuration. In addition, it is observed that the Loop II configuration results in I_r better than that of Harris.

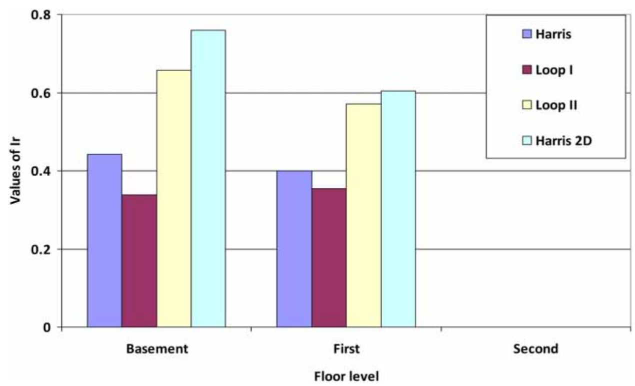

Figure 3 | Values of the resilience index (I_r) at the different floors and configurations.

Minimum surplus head

Figure 4 represents comparisons of minimum surplus head (I_m) values for the three floors with the four alternative layouts: Harris, Loop I, Loop II, and Harris 2D. It is clear from this figure that, for the three floors, the value of I_m is highest for the Harris 2D configuration and lowest for the Loop I configuration and I_m has a negative value on the second floor which means that the pressure head at the demand nodes on this floor are lower than the desired head to meet the demand. The layout of the Harris 2D provides the maximum performance with the highest value of I_m and Loop I configuration provides the minimum performance with the lowest value of I_m. In addition, the Loop II configuration provides better performance than the Harris configuration.

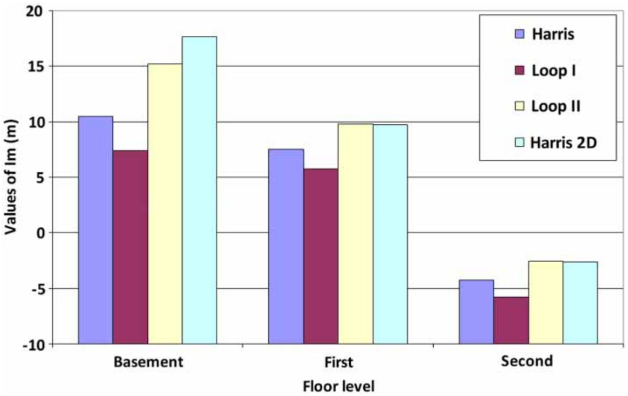

Figure 4 | Variations of the minimum surplus head (I_m) at the different floors and configurations.

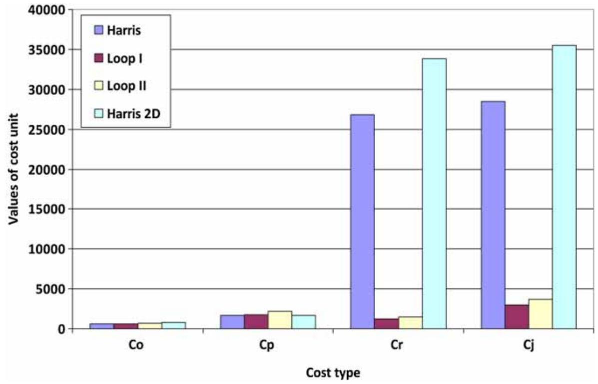

Figure 6 | Variations of costs for various network configurations (C_o), (C_p), (C_r), and (C_j).

Failure index

Figure 5 represents the variations of the failure index (I_f) values for the three floors with the four alternative models of Harris, Loop I, Loop II, and Harris 2D. It is clear from this figure that values of I_f vanish for the basement and first floor. On the second floor the value of I_f is highest for the Loop I configuration and lowest for the Harris 2D configuration. Loop I configuration reflects the worst performance, accompanied by the biggest value of I_f. In addition, it is observed that the Harris 2D configuration results in I_f better than that of Harris.

Cost functions

Figure 6 shows variations of the different cost functions, the actual cost (C_o), the cost related to the hydraulic constraints (C_p), the cost related to the number of independent flow routes to the demand joint (C_r), and the total cost function (C_j) for the four alternative models Harris, Loop I, Loop II and Harris 2D.

It is clear from the figure that the values of actual costs (C_o) of both Harris and Loop I configurations are smaller than those for Loop II and Harris 2D configurations. The values of costs related to the hydraulic constraints (C_p) of Harris, Loop I and Harris 2D configurations are smaller than that of the Loop II configuration.

Similarly for the cost related to the number of independent flow routes to demand joints (C_r) Loop I and Loop II configurations entail much lower costs than the Harris and Harris 2D configurations. The value of the total cost function (C_j) is lowest for the Loop I configuration followed by the Loop II configuration with a little higher cost.

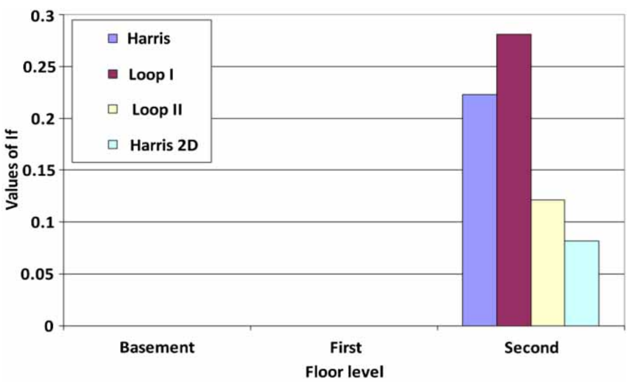

Figure 5 | Variations of the failure index (I_f) for different floors and configurations.

CONCLUSIONS

An approach for improving the performance of PWSs using looped WPNs within buildings is introduced in the present study. This approach is applied to a case study of branched layout configuration which was changed to a looped configuration with different pipe diameters and velocities. All cases were analyzed for cost and reliability, in terms of resilience index, minimum surplus head, failure index, and demand node's flow path using EPANET software. It is concluded from this study that using a looped configuration for PWSs inside buildings greatly improves the performance of the system since the overall hydraulic reliability of the system is increased in terms of resilience index, minimum surplus head, failure index and demand node's flow path. Using a looped configuration for PWSs inside buildings also decreases the pressure variation at plumbing fixtures, supplying sufficient demand at each of them since every demand node has many demand paths that do not affect each other which provides a good service to householders.

In addition, using a looped configuration for PWSs inside buildings minimizes the total cost function of the system which is the summation of actual cost, cost related to the hydraulic constraints and the cost related to the number of independent flow paths to demand nodes.

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