Effects of Orifice Diameter and Retention Time of Local Tanks on the Reliability and Carbon Footprint of Water Distribution Networks

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By

Syed Rizvi[1]*, Rabee Rustum[2], Orazio Giustolisi[3], Grant Wright[4] , Scott Arthur[5], Luigi Berardi[6]

1. PhD Student, Institute for Infrastructure and Environment, School of Energy, Geosciences, Infrastructure and Society Heriot-Watt University, Dubai Campus, UAE, ORCID no. 0000-0002-9609-6080. Email ID: sar32@hw.ac.uk
2. Associate Professor, Institute for Infrastructure and Environment, School of Energy, Geosciences, Infrastructure and Society, Heriot-Watt University, Dubai Campus, UAE, ORCID no. 0000-0001-6938-8144
3. Professor, Technical University of Bari, Italy, ORCID no. 0000-0002-5169-7798
4. Associate Professor, Institute for Infrastructure and Environment, School of Energy, Geosciences, Infrastructure and Society Heriot-Watt University, Edinburgh, UK, ORCID no. 0000-0003-3241-1456
5. Professor, Institute for Infrastructure and Environment, Heriot-Watt University, Edinburgh, UK, ORCID no. 0000-0002-9477-911X
6. Associate Professor, D'Annunzio University of Chieti–Pescara, Italy, ORCID no. 0000-0002-6252-2467

Abstract

The role of private tanks is to provide excessive storage to the consumer to satisfy the water demand. However, they are disregarded during the design stage, in favor of simplified network analysis. This affects the accuracy of the simulation as vital components, such as tank's inlet and volume sizes, are completely ignored. Hence, the purpose of this study is to demonstrate the effectiveness of advanced modelling of WDN, encompassing the presence of local private tanks, to determine the optimum values of different parameters of private tank by conducting time and volume-based reliability analysis. Two network models are used to perform the analysis: a small sample network and a real network that resembles the area of Dubai Silicon Oasis, Dubai, UAE. The results obtained from the simulation of networks indicated that the lowest orifice and volume size(s) to achieve the required reliability of unity is the optimum values. Furthermore, it implied that any change in their optimum values would either result in tank failure or increase in the head loss and carbon footprint of the network.

Keyword(s): *Water distribution network; Reliability analysis; Private tanks; Orifice diameter; Carbon Footprint*

Introduction:

The mathematical problem related to steady-state water distribution network (WDN) modeling is governed by the two classic balance equations; momentum along pipes and mass (continuity) in internal nodes where the head is unknown. Cross (1936) proposed the first algorithm to solve WDN modeling transforming the mathematical problem to a loop analysis, thus reducing the number of equations during iterative solutions. With the advent of the first computers from the 1960s, many researchers proposed global linearization algorithms, which were characterized by the simultaneous solution of all the equations (Martin and Peters, 1963; Shamir and Howard, 1968; Epp and Fowler, 1970; Collins et al., 1978; Isaacs and Mills, 1980; Wood and Rayes, 1981; Carpentier et al., 1987; Todini and Pilati, 1988). However, the nodal demands were still considered fixed a priori in the model and relevant WDN modeling approach is designated as demand driven. All the global linearization algorithms iteratively solve a system of linear equations whose size and type depend on the adopted strategy. Of particular note, Todini and Pilati (1988) developed the Global Gradient Algorithm (GGA) that requires the iterative solution of a sparse symmetric system of linear equations whose size is the number of unknown nodal heads. Rossman (2000) developed EPANET for E.P.A (Environmental Protection Agency) US that used GGA as the hydraulic engine, and most current software packages are generally based on this "hydraulic engine".

All of the classic hydraulic simulators are created for hydraulic verification, rather than to support management decisions, and for this reason they are based on the assumption of fixed demands at nodes of the water supply network and are called demand-driven, i.e., the pressures calculated at the nodes are "driven by the demand fixed at the nodes". Therefore, during the 1980s, researchers started to investigate the dependence of demand on system pressure, initially by Bhave (1981) and later by Germanopoulos (1985), who combined a leakage term with pressure-dependent customer-demands. Similar head-demand models were soon proposed and, in particular, the generic pressure-demand model for controlled outlets, suggested by Wagner et al. (1988), is considered the most feasible to predict WDN pressure-deficient conditions with respect to customer water requests (Gupta and Bhave, 1996). However, although Wagner's pressure-demand relationship for costumer-demands is hydraulically consistent, it is not everywhere differentiable (Ackley et al., 2001); thus, several methods were developed to assure the differentiability of pressure-demand relationships (Tucciarelli et al., 1999; Tanyimboh et al., 2001; Tanyimboh and Templeman, 2004). For the same reason, Giustolisi et al. (2008a, b) introduced an adaptive over-relaxation parameter to pressure-driven analysis within GGA and Piller and van Zyl (2007, 2009) developed a pressure-driven WDN model using the “content” and “co-content” proposed by Collins et al. (1978).

Progressive urban development has produced increasingly large, complex, and age-old networks, implying that management needs with respect to water losses, reliability, water quality, energy optimization, rehabilitation, etc. The need for analyzing possible pressure-deficit scenarios showed that the classic DDA of EPANET or EPANET like engine are not suitable to support management decisions. Therefore, it was necessary introducing hydraulic simulation capable of evaluating the "actual" demand that can be provided to users when pressures are lower than the minimum pressures for correct service or the leakages outflows. The nodal demands of the advanced hydraulic simulations depend on pressure, then they perform pressure-driven analyses (PDA) as opposite to demand-driven analysis (DDA) of classic hydraulic simulation.

Giustolisi et al. (2008) developed the representation of the volumetric water losses demand component, as a function of pressures and deterioration, for each individual pipe. In this way, the accuracy of hydraulic calculations is preserved without losing information at the level of individual pipes. Also, Giustolisi et al. (2014) proposed the demand modeling accounts for local storages supplying water to customers, i.e., nodal demands depending on the filling/emptying process of the local storage, the orifice control type and the customer required demand varying over

time. Such local water storage volumes are actually very common in those countries where water supply is intermittent and not reliable. In fact, the use of local tanks, also known as private tanks, has even become mandatory in some regions of the Middle East, such as the UAE. In addition, there are some regions, e.g., in Southern Italy, where private water distribution systems are traditionally fed by private inline water storage whose filling/emptying processes are likely to affect overall system behavior. Some examples are the filling process of inline tanks in intermittent distributions (Criminisi et al., 2009; De Marchis et al., 2011) or after a pressure-deficient occurrence the prediction of the actual supplied customer-demands accounting for the existence of inline tanks. Recently, De Marchis et al. (2018) integrated the private tanks using the orifice area to show the pressure variation with flow in their practical experiment.

Whilst some research has been undertaken into the behavior simulation of local tanks in the network, very few of them have focused on the impacts of different tank parameters on the overall network model. Parameters such as orifice and volume size can significantly change the performance of the overall system in terms of carbon footprint and pressure. Therefore, the selected parameters need to be set at their optimum value using reliability indicators.

At the component level, the reliability of failure is estimated by determining the failure that occurs during a specified time interval, using a probability density function determined from historical failure data (Liserra, T et al., 2014). Nodal reliability indicators often investigate the performance of nodal junctions based on the level of service in terms of both quantity and quality. These reliability indicators are based on the total demand satisfaction at junction, the water quality or pressure at the nodes (Gargano and Pianese 2000; Tanyimboh et al. 2001), and they are often used to analyze the impact of different parameters on the whole network. For example, Martinez-Rodriguez et al., (2011) used network demand reliability indicators to evaluate the network performance using the water flow supplied to the overall demand. These values of the indicators could also be extended to compare the quality and pressure in the entire network. Then, behavior Simulation technique by McMahon et al., (2006) was to determine the reliability of water supply in presence of tanks by using mass balance technique such as time and volume-based reliability. Time based reliability is the ratio between the total number of occasions in which the full demand was met to the number of time periods in the simulation period. The Volume based reliability is the ratio of the total amount of water actually supplied to the total amount demanded. The weightings presented by these indicators allows assessment of the performance of the network under various configurations of local storage tanks (different orifice and volume sizes). However, the use

106 of reliability in this case would be limited to time and volume-based reliability; these indicators can reflect the
107 performance of the tank parameters using mass balance techniques.

108 Since, the classical modelling approach does not allow assessment of the reliability of local tanks (as they
109 are ignored), leading to possible insufficient or oversized design (Cobacho *et al.*, 2008; Criminisi *et al.*, 2009; De
110 Marchis *et al.*, 2011; De Marchis, Freni and Milici, 2016, 2018; Mohamed *et al.*, 2019). Insufficient design can lead
111 to failure where the tank cannot provide the required consumer demand and oversized design can alternatively increase
112 the cost and deteriorate the water quality. Hence, the purpose of the work reported herein is to demonstrate how
113 advanced hydraulic modeling of WDN can be used to investigate the impact of local tank parameters (orifice and
114 volume size) on reliability, pressure, and carbon footprint, thus supporting WDN management actions.

115 **Methodology:**

116 When modelling local storage, the flow through the tank's inlet depends upon orifice diameter, available
117 storage, orifice level, available pressure in the network, coefficient of discharge and pattern of consumers' water
118 demand. They are modelled as free orifice, by using PDA, whose size is controlled by storage volume. This is because
119 the node of the network is connected to the orifice of the local tank and not directly to the consumer as assumed during
120 the design phase of the network, even though the capacity-demand function of the reservoir is affected by the flow to
121 the tank which is itself affected by pressure in the network and the orifice diameter of the inlet. This leads to a change
122 in the simulation process in the network.

123 ***<u>Steady-state hydraulic modelling and local storages</u>***

124 For simulation of WDN, the system of momentum and continuity equations at pipes and internal nodes,
125 respectively, can be written in the matrix form as in Equation 1 (Giustolisi et al. 2012),

126
$$\begin{aligned} \mathbf{A}_{pp}(t)\mathbf{Q}_p(t) + \mathbf{A}_{pn}\mathbf{H}_n(t) &= -\mathbf{A}_{p0}\mathbf{H}_0(t) \\ \mathbf{A}_{np}\mathbf{Q}_p(t) - \frac{\mathbf{V}_n(t,\mathbf{H}_n(t))}{\Delta T} &= \mathbf{0}_n \end{aligned} \quad (1)$$

127 For each simulation snapshot t, $\mathbf{Q}_p$ is the column vector of unknown pipe flow rates; $\mathbf{H}_n$ is the column vector
128 of unknown nodal heads; $\mathbf{H}_0$ is the column vector of known nodal heads and $\mathbf{V}_n$ is the column vector of volume
129 outflows during ΔT lumped at nodes. $\mathbf{A}_{pn}$, $\mathbf{A}_{np}$ and $\mathbf{A}_{p0}$ are topological incidence sub-matrices of the general topological
130 matrix, link-node, of the network. The subscript p and n indicate the dependence of the matrices and vectors on the

number of pipes and nodes (related to unknown heads), while the subscript "0" refers to the number of reservoirs (known heads). The second part of Equation (1) represents mass-balance at model nodes and $\mathbf{V}_n = \mathbf{d}_n \cdot \Delta T$ where $\mathbf{d}_n$ are the stationary demand components (Giustolisi and Walski, 2012) in ΔT generally depending on pressure through $\mathbf{H}_n$. Equation (2) explicitly reports all demand components that are included in $\mathbf{V}_n$.

$$\mathbf{V}_n\left(t, \mathbf{H}_n(t)\right) = \mathbf{V}_n^{cons}\left(t, \mathbf{H}_n(t)\right) + \mathbf{V}_n^{priv-tank}\left(t, \mathbf{H}_n(t)\right) + \mathbf{V}_n^{orif}\left(t, \mathbf{H}_n(t)\right) + \mathbf{V}_n^{tank}\left(t, \mathbf{H}_n(t)\right) + \mathbf{V}_n^{leak}\left(t, \mathbf{H}_n(t)\right) \quad (2)$$

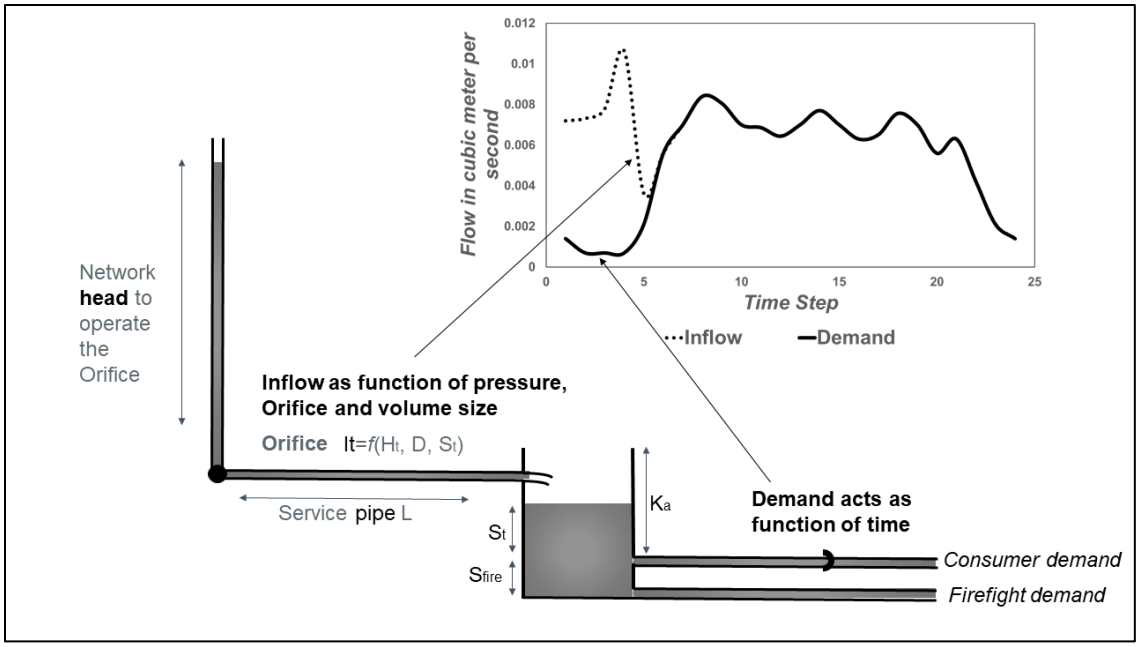
Where $\mathbf{V}_n^{cons}(t, \mathbf{H}_n(t))$ is the water demand supplied to *consumers* directly connected to the water network: if nodal pressure is higher than the minimum value to supply service to consumers (i.e. by contract) it equals the *statistical* water requests while, in case of pressure-deficient conditions, it returns the water volume supplied to consumers consistently with Torricelli law.

$\mathbf{V}_n^{priv\text{-}tank}(t, \mathbf{H}_n(t))$ represents the volume of water feeding private storage tanks in ΔT, as a in Giustolisi et al. (2014). Such function depends on tank filling conditions at the beginning of the simulation time step and on pressure at n^{th} node. Thus, filling/emptying of private tanks affects WDN hydraulic status, especially in water shortage conditions (e.g. De Marchis et al., 2018).

$\mathbf{V}_n^{orif}(t, \mathbf{H}_n(t))$ is the water volume from uncontrolled free orifices, e.g. from hydrants. The outflow coefficient of uncontrolled orifices is known for manufactured devices (e.g., hydrants, sprinkler systems), while it is unknown and might be variable with pressure for orifices representing pipe bursts.

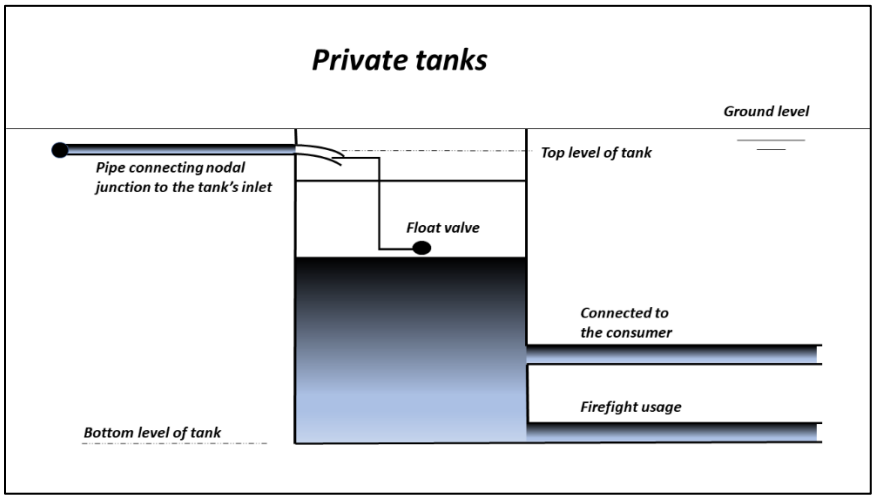
$\mathbf{V}_n^{tank}(t, \mathbf{H}_n(t))$ represents the water volume to/from *tanks* whose consistent and stable simulation is affected by the global water balance in the system (Giustolisi et al. 2012). It must remark that $\mathbf{V}_n^{tank}$ refers to urban tank in WDN, not private tanks.

$\mathbf{V}_n^{leak}(t, \mathbf{H}_n(t))$ represents leakage volume due to pressure-dependent outflows from holes, cracks, joints and fittings. Few models are available in the literature to represent pressure-leakage demand component $\mathbf{V}_n^{leak}$, e.g. Germanopoulos (1985), Germanopoulos and Jowitt (1989), May, (1994), Giustolisi, O. et al., (2008a) and, Van Zyl and Cassa, (2015).



156 *Fig. 1 Actual scenario with private tanks, modified after* (Giustolisi et al., 2014)

157 Focusing on the component $\mathbf{V}_n^{priv\text{-}tank}$, the simulation of private storages cannot be performed using DDA
158 because when the local tank is not completely filled, a process of filling/emptying occurs, and the inlet is a free orifice
159 whose size is controlled by tank volume as shown in Fig. 1. In other words, private tank simulation involves DDA
160 when the tank is filled during the snapshot ΔT, i.e., it starts filled and the discharge from the free orifice equals the
161 required demand, otherwise PDA is mandatory. A schematic of private local tank is shown in Fig. 2.

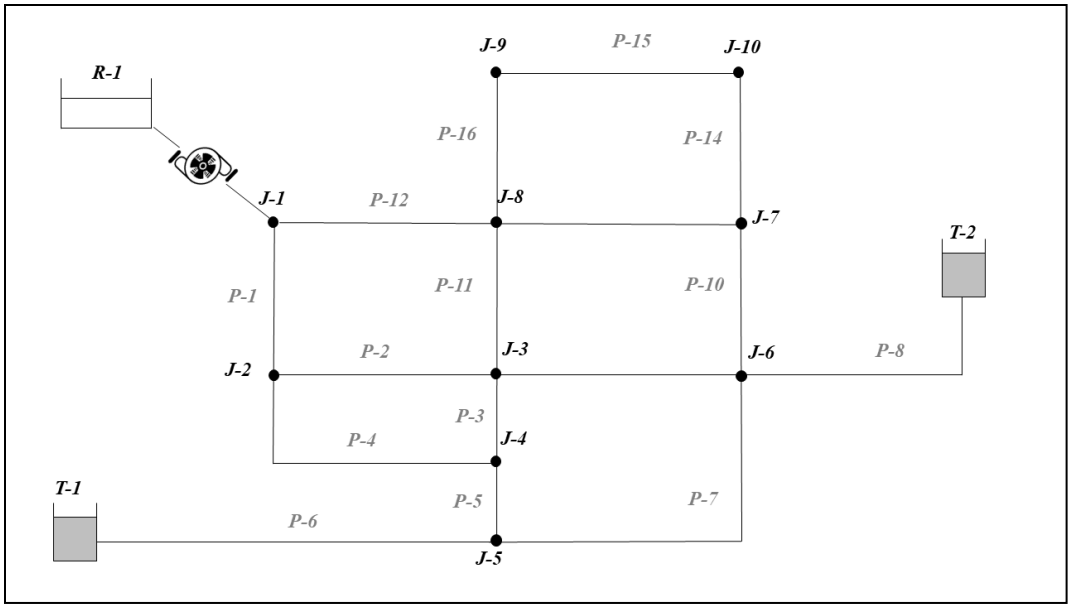


163 *Fig. 2 Layout of a common underground private tank*

Sample networks

In this study, the data of a WDN was collected from *Walski* et al.*, (2003)* and a real-time network from Dubai, UAE. For each network, every single attribute including the nodal data, base water demand, pipe characteristics, valve settings, property of water sources, additional storage(s) data, pump curve(s), type(s) and scheduling were used. From these attributes, some of the missing information for private tanks, such as the volume and orifice diameter(s), have been initially assumed based on the given standards by Dubai Electricity and Water Authority (DEWA) and general practices from the local developers in UAE. In total, two sample networks were built to show the difference in the reliability analysis of private tanks at each case. The initial assumed orifice diameter was taken as 2cm (minimum specification) and the volume size was assumed to be equivalent to 6 hours of average daily water use, i.e. retention time of 6 hours. The initial level of the tank was set to empty to account for the worst-case scenario. Furthermore, as the minimum pressure to achieve the required demand varies between 10m to 15m based on the practices from different countries, it was taken as 12.5m in this case.

Network 1(Fig. 3): WDN containing two variable tanks and a single reservoir connected to nodal junctions of the network by a single pump. The outline of the network ensures that there would be enough pressure to supply the required demand using pumps and variable tanks (Walski *et al.*, 2003). This network is used to showcase, in detail, the effect of changing orifice and volume size of private tanks using reliability indicators. Furthermore, it also shows the changes in pressure and carbon footprint with increasing the orifice and volume sizes.



182 *Fig. 3 Schematic of Network 1*(Walski *et al.*, 2003)

183 Network 2 (Fig. 4): The schematic of the WDN represents a real-time network based on the area of Dubai
184 Silicon Oasis (DSO) network in Dubai, UAE. The network model was built based on the approved master plan for the
185 development of this community. The system was built to supply a total of 45000m^3 per day to the estimated population
186 of more than 150,000 living/or working in DSO, but in reality, the current population is far lower as the area is still
187 growing. Hence, the current real time network is functioning partly where some of the pipes in the network are not
188 operational, and the running time of the pump is limited. Therefore, the model was designed from the pre-approved
189 plan that will eventually be implemented in the real network once the entire area is populated.

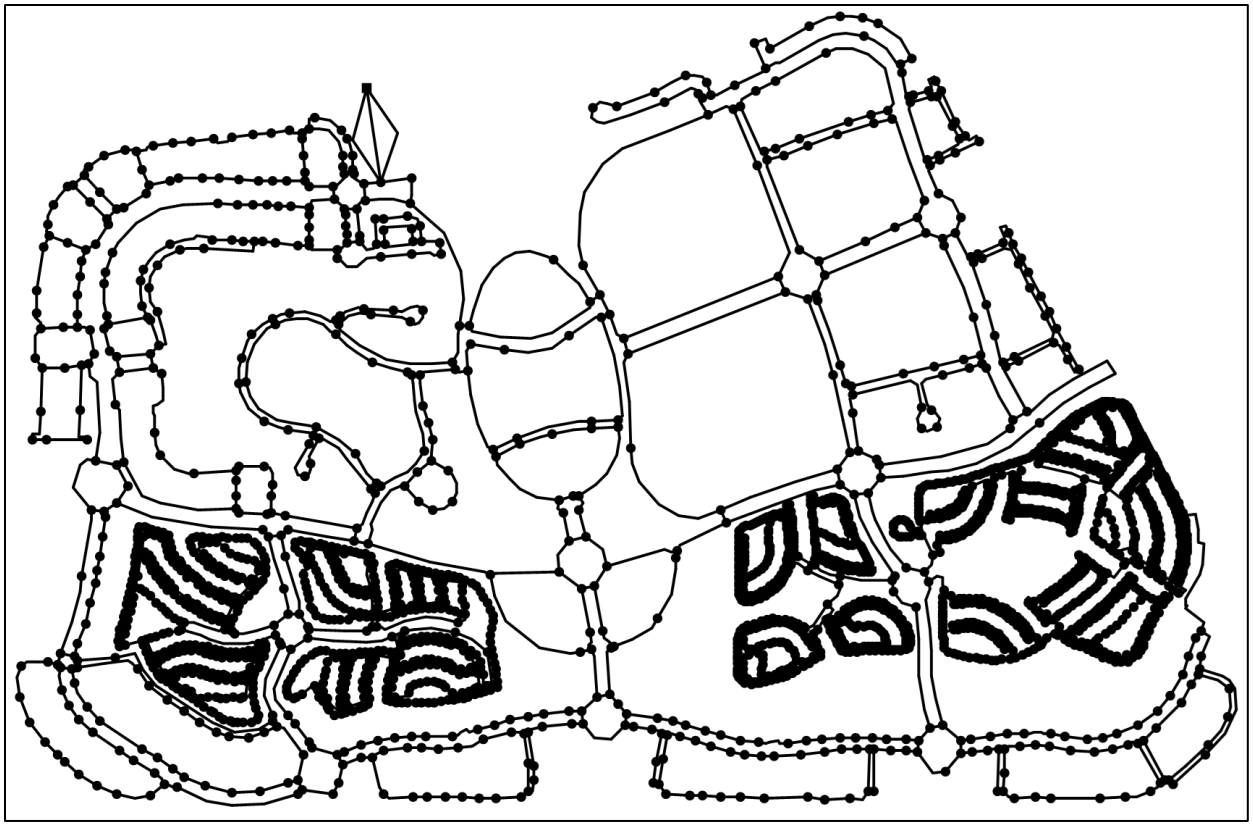


Fig. 4 Schematic of Network 2

Generally, Network 2 is a branched network, containing a total of 2550 junctions and 2750 pipe links. The water is supplied using a single source reservoir connected via three pumps running in parallel. Pipe diameters range between 150mm to 900mm, while the daily demand per capita is close to 550 liters per day. The estimated number of users is close to 165,000 capita. The elevation of the area varies, with few areas higher than the reservoir head of 30m. The nodal characteristics of the network varies as the area itself consists of a range of different residential buildings, villas, commercial buildings, educational schools, governmental offices, and hospitals. The purpose of this case study is to validate the theory based on larger real networks, with a greater number of nodes and junction. Furthermore, the analysis uses flow paths to showcase the effect on pressure by changing the properties of few junction tank nodes.

Assuming that the present analysis refers to design stage, water losses (pipe leakages) from WDN are neglected. Nonetheless, the analyses reported in this paper apply to support management decision on aged systems with known water losses, for which the parameters of the leakage model can be estimated.

Analysis and working of the simulation:

The adopted hydraulic model (i.e., Giustolisi et al., 2014) implements private tanks by using a pressure-demand function which accounts for the filling/emptying process based on orifice diameter varying with stored volume in each tank.

In case the orifice of the tank has two states only, closed or fully open (ON/OFF control), $C(t)$ is constant in ΔT if V_i^{max} is not reached. Thus, the mass balance in Equation (1) allows writing:

$$V(t+\Delta T) = d^{fill}\Delta T - d^{act}\Delta T + V(t) \text{with} d^{fill}(t) = C^{max}\sqrt{P(t) - \Delta z^{orif}} \quad (3)$$

where $d^{fill}(t)$ = average flow rate filling the tank during ΔT, $P(t)$ is the head pressure at the time t, Δz^{orif} is the elevation of the orifice inlet from the ground and C^{max} = maximum outflow coefficient of the orifice. The C^{max} was modified to incorporate the orifice diameter during the simulation and was represented by:

$$C^{max} = \frac{C_D \pi d^2 \sqrt{2g}}{4} \quad (4)$$

Where C_d is the coefficient of discharge, d is the diameter and g is acceleration due to gravity. The following formulations are adopted for $d_i^{fill}(t)$ and the delivered customer demand $d_i^{act}(t)$ at the i^{th} node (Giustolisi et al., 2014):

$$d_i{}^{fill}\big(P_i(t), V_i(t)\big) = \begin{cases} d_i{}^{act}(t) + \frac{V_i{}^{max} - V_i(t)}{\Delta T} & V_i(t+\Delta T) > V_i{}^{max} \\ C_i{}^{max}\sqrt{P(t)_i - \Delta z^{orif}} & V_i(t+\Delta T) \le V_i{}^{max} \end{cases} \quad (5)$$

$$d_i{}^{act}\big(P_i(t), V_i(t)\big) = \begin{cases} d_i^{req,hum}(t) & V_i(t+\Delta T) > V_i{}^{max} \\ C_i{}^{max}\sqrt{P_i(t) - \Delta z^{orif}} + \frac{V_i(t)}{\Delta T} & V_i(t+\Delta T) \le V_i{}^{max} \end{cases} \quad (6)$$

In the case of a tank becoming empty during Δt, the assumption $V(t+\Delta T) = 0$ holds and

$$d_i{}^{act}(t) = C_i{}^{max}\sqrt{P_i(t) - \Delta z^{orif}} + \frac{V_i(t)}{\Delta T} \quad (7)$$

where $d_i^{act}(t)$ = actual average demand supplied to customers during ΔT. This means that the tank completely empties during ΔT, although the orifice is fully opened, because the inflow summed to the initial water volume is insufficient to satisfy the customer water requests.

In the case of a tank becoming completely full during ΔT, the assumption $V_i(t+\Delta T) = V_i^{max}$ holds and, it is possible to assume a fixed flow rate filling the tank over ΔT given by:

$$C_i^{max}\sqrt{P_i(t) - \Delta z^{orif}} \geq d_i{}^{act}(t) + \frac{V_i{}^{max} - V_i(t)}{\Delta T} \quad \Rightarrow \quad d_i{}^{fill}(t) = d_i{}^{act}(t) + \frac{V_i{}^{max} - V_i(t)}{\Delta T} \tag{8}$$

In this case the customers' demand at the i^{th} node is satisfied and $d_i^{act}(t) = d_i^{req,hum}(t)$. Where $d_i^{req,hum}(t)$ is the required demand by the consumer during ΔT. It is taken from the water demand profile at specific time interval.

The above detailed explanation of the demand model for private tanks at the i^{th} node can be extended to any kind of control (Giustolisi et al., 2014). Hence, the advanced hydraulic model accounting for local tanks, allows harmonizing different design parameters (orifice diameter, orifice level and volume of local tanks) by using reliability analysis.

Reliable design can be achieved with different volume and orifice diameter combinations that satisfy the consumer demand without hydraulic failure. The time and volume-based reliability indicators could be calculated based on the result of the simulation by using Equation 9-12 (Adeloye et al., 2017).

$$R_t = 1 - \frac{\sum_{t=1}^{N} f_t}{N} \tag{9}$$

$$R_v = 1 - \frac{\sum_{t=1}^{N} f_t(d_i^{req,hum}(t) - d_i^{act}(t))}{\sum_{t=1}^{N} d_i^{req,hum}(t)} \tag{10}$$

$$f_t = \begin{cases} 1, if\ d_i^{act}(t) < d_i^{req,hum}(t) \\ 0, otherwise \end{cases} \tag{11}$$

$$d_i^{act}(t) = \begin{cases} d_i^{fill}(t) + V(t), & if\ V(t) + d_i^{fill}(t)\Delta t \leq \Delta t d_i^{req,hum}(t) \\ d_i^{req,hum}(t), & if\ d_i^{req,hum}(t)\Delta t \leq V(t) + d_i^{fill}(t)\Delta t \leq K_a \\ & Where\ V(t) is\ initial\ storage \end{cases} \tag{12}$$

where N is the total number of time periods, i.e., time steps of the extended period simulation representing the typical operating cycle, R_v is the ratio of the total amount of water actually available to the costumer and the desired amount demanded, and R_t is the ratio between the total number of time steps in which the full demand was met to the number of time steps in the extended period simulation. The required reliability to achieve the consumer consumption

is equal to 1 for each local tank. Therefore, the sizing of the orifice is an important tool as it can restrict the incoming flow to the tank. For example, if the diameter of the opening is too small then the local tank might not be able to achieve the required reliability even if there is enough pressure to satisfy demand as shown in Fig. 5.

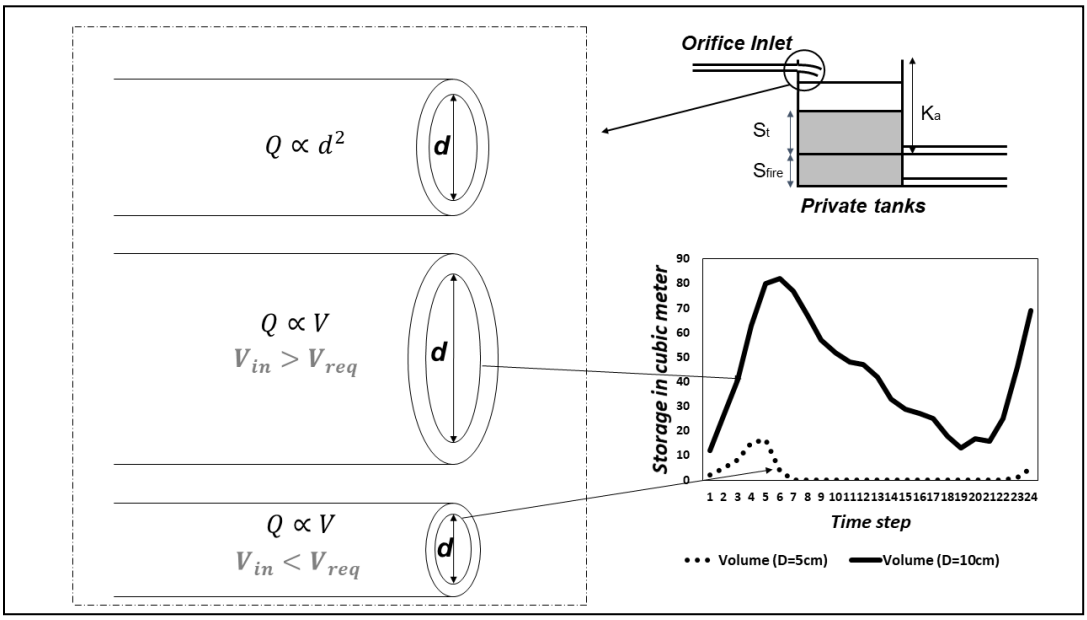


Fig. 5 Impacts of orifice sizing on Volume (V is the volume, Q is the flow and d is diameter)

Carbon footprint:

Carbon footprint depends on the power required by the pump that is operating the network. Often power is calculated by the flow, total head, density of the water being pumped, mechanical efficiency and acceleration due to gravity (Walski, T., 2003).

$$P = \frac{\rho g H Q}{\eta} \quad (13)$$

where P stands for power in kW, H is total head (suction head-discharge head) in m, Q represent the discharged flow rate m^3/s, ρ refers to the density of water being discharged, and η is the efficiency factor. A simple factor is applied to the obtained power of the pump(s) that differs from country to country to obtain the carbon footprint. A factor of 0.43 was applied on the overall total power obtained from the pump to obtain the carbon footprint.

Hence, the investigation itself would be a trial-and-error approach to achieve the required reliability of 1 by the following steps:

1. Obtaining the demand data, volume and orifice diameter of the tank based on the codes and selecting the time period of the simulation,
2. Perform the simulation which will give the number of failures by using advanced hydraulic modelling encompassing Equations 1-8,
3. Calculate the time and volume-based reliability based on equation 9-12 for multiple orifice and volume size,
4. The optimum size is chosen once the lowest orifice and volume parameters achieves the required reliability,
5. Steps 1 to 4 are repeated with different orifice inlet level to obtain the best parameters in terms of low carbon footprint and pressure.

The above steps are summarized in a flow chart in Fig. 6. Therefore, the idea is to investigate the reliability of the tanks with different retention times and orifice diameter, without changing the geometry of the network.

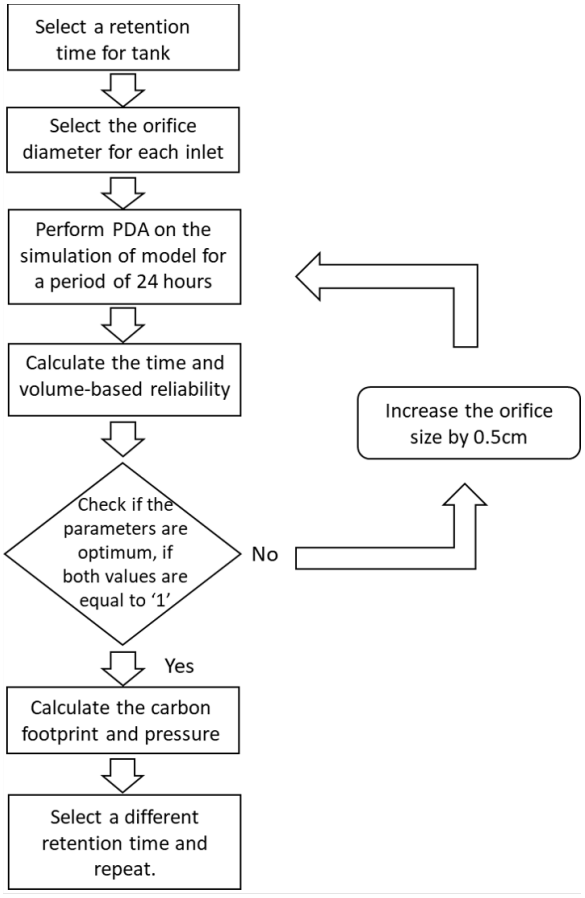


Figure 6 Flow chart of the entire process

Results and Discussion:

Network 1: WDN containing two tanks and single pump supplied from a source.

For network 1, Fig. 7 showcases storage at each time interval for every tank in the network. The comparison between each plot is drawn based on different retention times to show the impact of volume capacity. The area of success and failure is based on the storage level at different times; where reaching to zero instantly means that the tank is unable to satisfy the consumer demand.

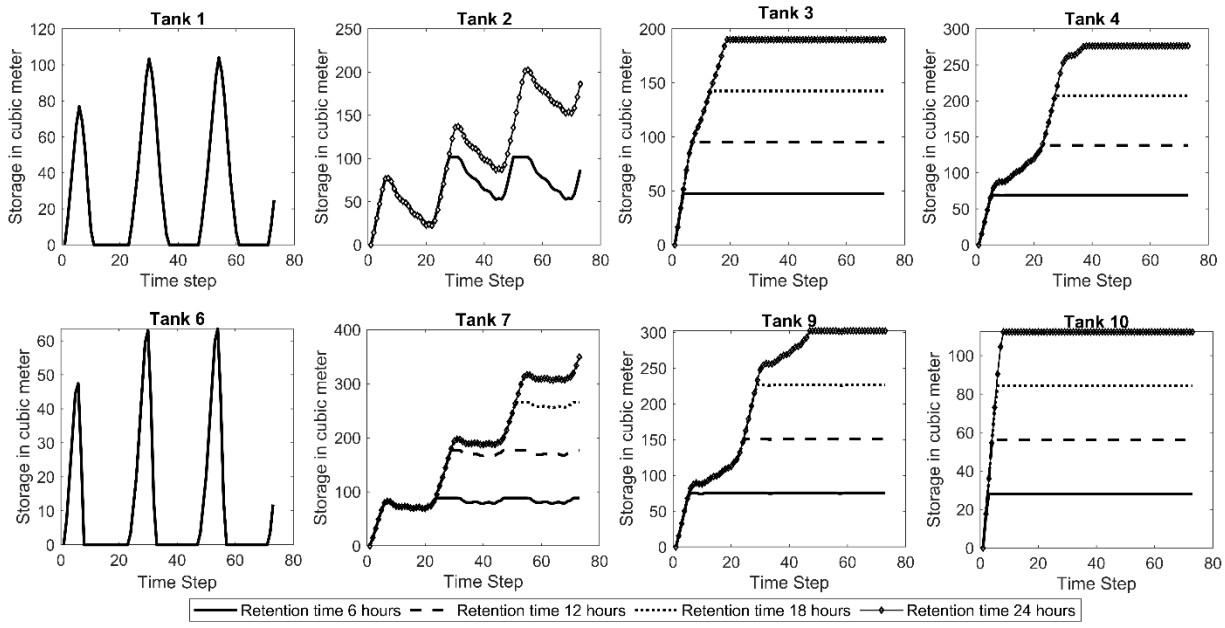


Fig. 7 Simulated storage for different retention times (orifice diameter constant at 2cm)

One of the key things noted in Fig.7 is that increasing the volume size does not always lead to improvement in reliability. In fact, it has been seen that changing the retention time for tank '1' and '6' did not made any significant impact on the reliability of the tank; this is because the orifice diameter of the tank is too small to fulfill the consumer demand at relevant time periods. This is also justified as the base demand at the nodal junction for each tank is relatively higher than at any other nodes. On the contrary, increasing the volume size for other tanks increases the storage capacity as the tank regularly fills without depleting as seen for tank '10'. However, the common concern with large tanks is that the water quality drops as water stays in the tank for too long without being renewed. It indicates that the volume of the private tanks is larger than required which can cause water stagnation. This could be improved by decreasing or restricting the orifice diameter which changes the actual inflow coming into the tank. Therefore, to

reflect on the impact of the orifice diameter, simulations were repeated while keeping the retention time constant and changing the orifice diameter for every local tank in the network.

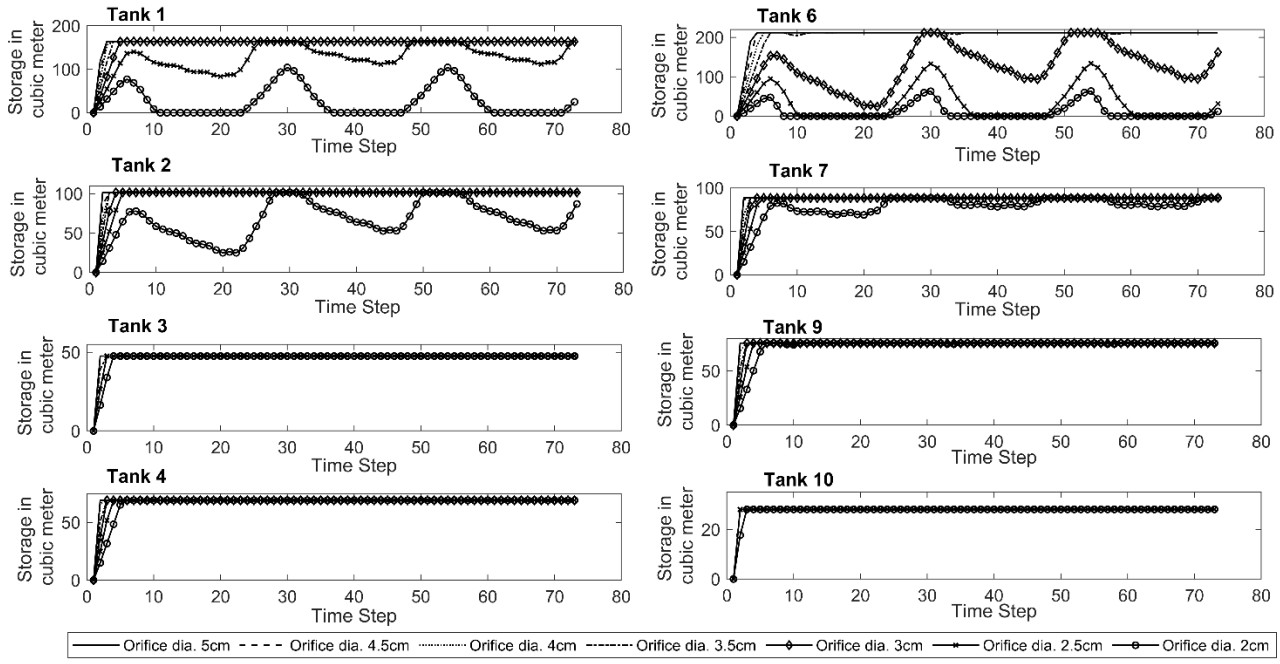
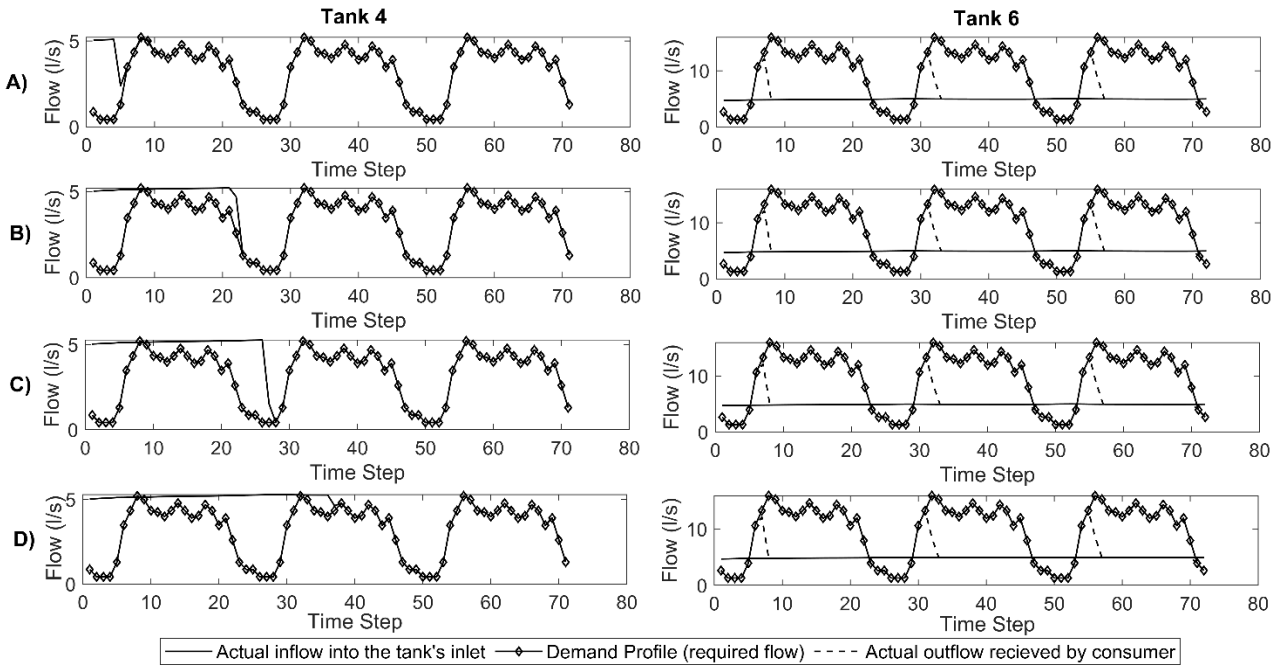


Fig. 8 Simulated storage for different orifice diameter of the private tanks (Volume size equivalent at retention time of 6 hours)

The results in Fig. 8 show that increasing the orifice diameter for tank '1' and '6' allows them to achieve the required demand. Based on the storage level, it is observed that shifting from 2cm to 2.5cm allows tank '1' to fulfill the demand without being completely empty (tank failure). While for tank '6', the orifice diameter of 3cm is sufficient to satisfy the user's water requirement. In fact, this also shows that fewer changes in orifice diameter are as impactful as increasing tank volume. Furthermore, for the remaining tanks, increasing the orifice diameter allows them to reach their capacity in a much shorter time as the inflow increases from the tank's inlet. To illustrate this effect in detail, Fig. 9 and 10 shows the comparison of inflow with outflow for different retention times and orifice diameter, respectively.



304 *Fig. 9 Simulated flow for tanks 4 and 6 with retention time of a) 6 hours b) 12 hours c) 18 hours and d) 24 hours*

305 In Fig. 9, the actual inflow is referred as the incoming flow to the tank's inlet (i.e., $d^{fill}(t)$). During the periods
306 when tank is close to full, the actual inflow curve becomes similar to the desired demand. The desired demand is the
307 maximum required water consumption obtained from the Water Demand Profile. Finally, the actual outflow is referred
308 as the outgoing flow coming out from the tank. The actual outflow will be equal to the desired demand unless there is
309 insufficient flow coming into an empty tank. Any gap between the desired demand and outflow would be indicated as
310 tank failure. The area between the outflow and desired demand is taken as the required storage to prevent tank failure.
311 Moreover, the private tank fills up when the actual inflow is higher than the desired demand and begins to empty when
312 the desired demand is more than the inflow. With above description laid out, the results in Fig. 9 take in account of
313 different retention times for two tanks '4' and '6', with orifice diameter kept constant at 2cm. As seen before, tank '4'
314 was able to satisfy the demand as the inflow in the tank's inlet is higher than the required consumption profile. With
315 lower retention time (6 hours), the tank can reach its full capacity as the inflow and outflow curves become similar
316 from the time 6hrs, due to the closure of inlet. Increasing the retention time further, means the inflow curve takes more
317 time to become equivalent to the water demand curve (tank is close to full) as the water keeps on filling the tank. In
318 fact, when using a very high retention time of 24 hours, it can take up to 40 hours for the tank to reach its maximum
319 capacity. However, for tank '6', increasing the retention time has no effect on the inflow as the control is the restricted

orifice diameter. Hence, the outflow curve does not coincide with the required demand indicating that the tank is empty.

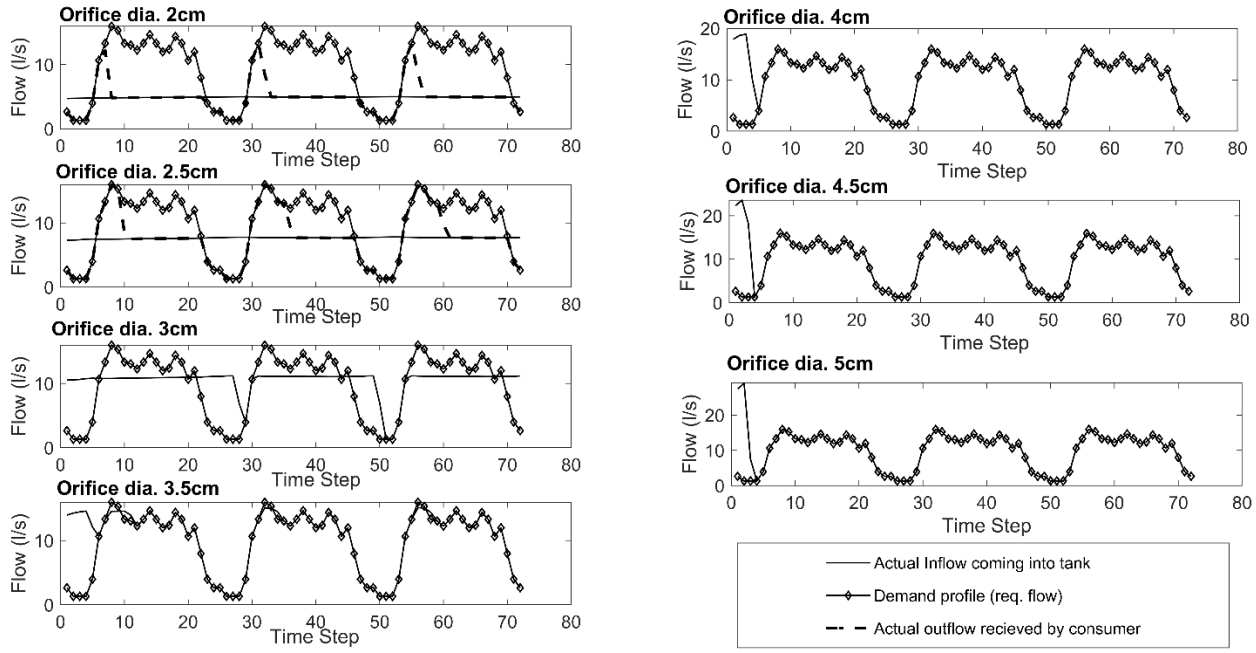


Fig. 10 simulated flow for tank 6 with changing orifice diameters

In Fig. 10, tank '6' with increased orifice diameters; the results show that the inflow curve increased as the orifice diameter increased. The incoming flow (actual inflow) is increased when the orifice diameter changed from 2cm to 2.5cm, gradually reducing the required storage, area between the desired demand and outgoing flow. Yet, the tank fails due to insufficient storage to satisfy the desired demand. Hence, when the diameter is increased further to 3cm, the tank can fulfil the required demand (no gap between desired demand and outflow). The tank also reaches it maximum capacity as indicated from the inflow curve for the orifice diameter of 3cm at time 30 and 50 hours. The inflow drops and becomes equal to the demand at those times, which shows that the pressure has dropped due to closure of the orifice. Additionally, any increase in the orifice diameter above 3cm would be unnecessary and might reduce the pressure at downstream nodes due to increased inflow.

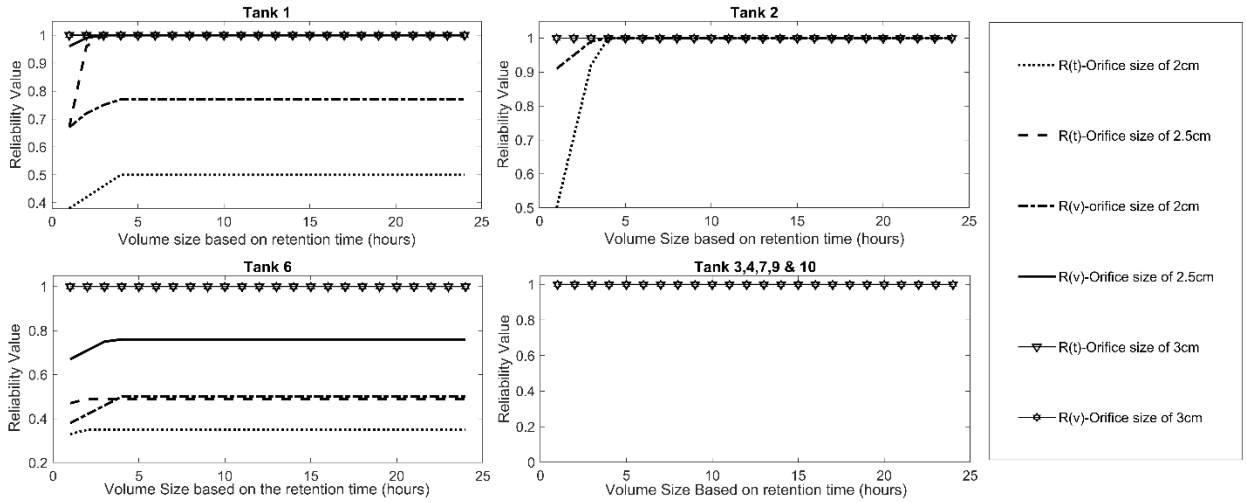


Fig. 11 Reliability analysis for each tank in the network

Finally, Fig. 11 shows volume and time-based reliability indicators for different tank retention time and orifice diameter. It is also worth noting that both the reliability indicators have variations in their value due to the difference in outgoing flow and desired demand. The volume-based reliability indicators tend to give more weightage to the values of partial flow that satisfy a small portion of the demand. On the other hand, the time-based reliability indicators only analyze performance of the tank based on satisfying the full requirement of the consumer, therefore it does not give any weightage to partial flows. Their graphical representation in Fig. 11 indicates that the reliability value increases with higher retention time, to a certain extent, for tanks '1' and '6'. It also shows that none of the retention times can achieve the required reliability of 1. However, the other tanks are able to achieve their required reliability with minimal volume sizes and hence, increasing the retention time does not impact the indicators. Similarly, the reliability indicators show the impact of changing the orifice diameter for different local tanks. For tank 3,4,7,9 and 10, increasing the orifice diameter did not have any impact on the reliability as it is already at maximum. For tank 1, increasing the orifice diameter just by 0.5cm allows it to reach the required reliability. Finally, for tank '6', increasing the diameter does increase the reliability value for both indicators, however, it needs to be raised till 3cm to achieve the required reliability.

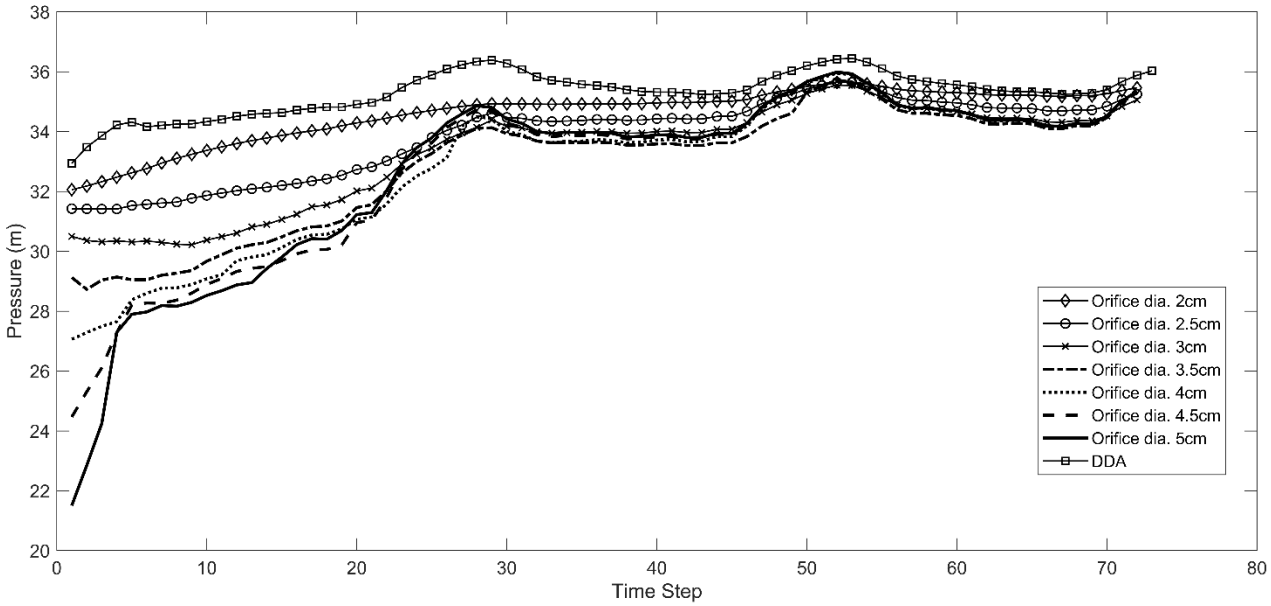
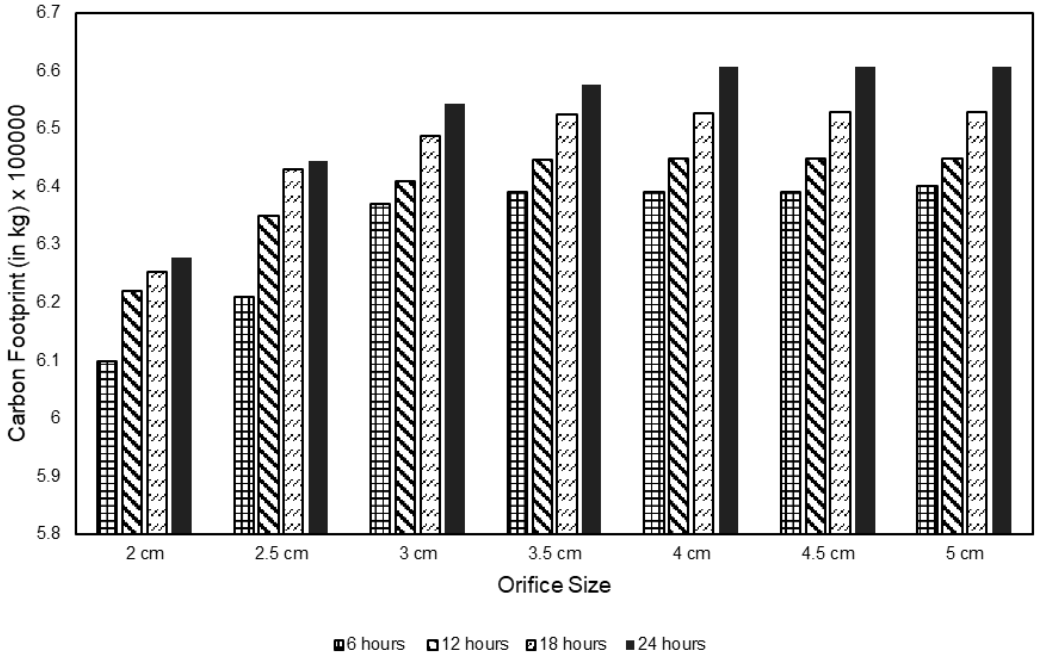


Fig. 12 Pressure variation with different orifice diameters for tank 6 and from DDA (without the inclusion of tanks)

For every scenario mentioned above, Fig. 12 represents the fluctuation in pressure to show the impact of changing orifice for tank '6', as similar results were subjected in other tanks. Fig. 12 also contains the pressure pattern obtained without the integration of local tank (as commonly practiced by engineers) to show the effects of integrating local tanks. Firstly, it is noted that increasing the orifice diameter, decreases the pressure, as decreasing diameter leads to an increased inflow which itself results in a higher head loss and hence a lower pressure. Secondly, the result does indicate that the pressure values, as certain time periods become similar to that of DDA. More commonly, this happens once all tanks in the network is completely full. For example, the pressure fluctuations of DDA and orifice diameters, that is above or equal to 3cm are the same which indicates that the tank reaches its full capacity. In fact, increasing the diameter allows the tanks to be filled up much sooner, therefore, the pressure curve becomes much more similar to DDA. Lastly, it should be noted that the variation in pressure can differ quite drastically especially if the comparison is drawn between large and smaller orifice diameters with higher retention times. In fact, the pressure drop of 12m was noted in this case (when comparing orifice diameter of 2cm and 5cm), which indicates very high head loss without changing any feature of the network (e.g., pipe hydraulic resistance, pressure control devices).



365 *Fig. 13 Carbon footprint comparison with different orifice and volume size for all tanks*

366 The carbon footprint of any system is directly related to the power utilized by the pumps. Fig. 13 shows the
367 impact of changing orifice diameter and tank volume on the production of carbon footprint. It shows that the carbon
368 footprint increases with orifice diameter, as the flow rate increases which causes the pumps to utilize more power to
369 distribute water in the network. The carbon footprint eventually starts to become constant when using large orifices
370 between 4cm to 5cm (for every retention time), as all tanks remains full in the network model. Similarly, having a
371 larger retention time increases the carbon footprint as more water is pumped again into the network to fill these large
372 local tanks. In fact, the carbon footprint differs vastly when comparing the small orifice diameter of 2cm at a low
373 retention time of 6 hours, with the large orifice diameter of 5cm at a higher retention time of 24 hours.

374 ***Network 2: Silicon Oasis network***

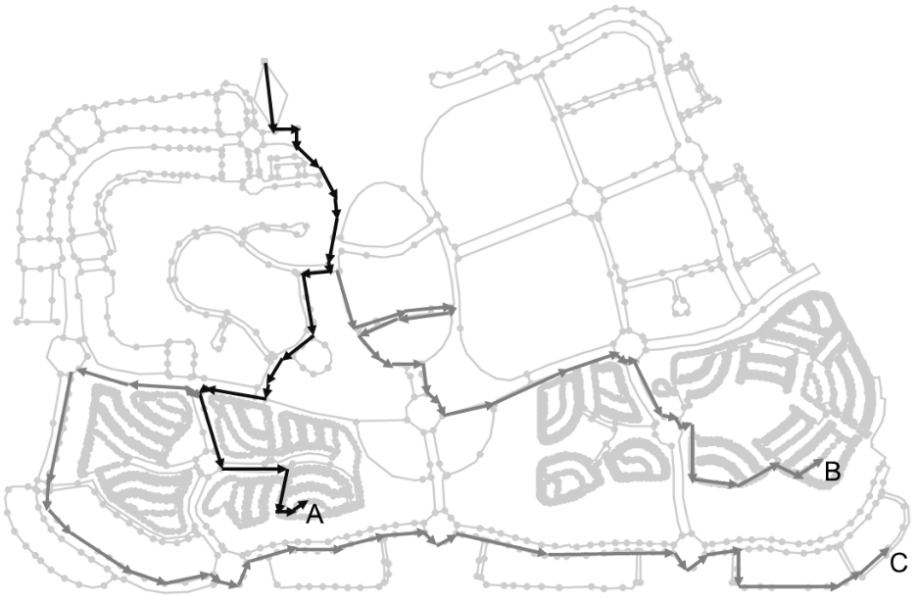
375 Table 1 Optimum setting of the orifice and volume size for each type of node

	Percentage of nodes (%)	Demand (m^3/d)	Optimum Orifice diameter (cm)	Optimum Volume size (in hours)
Villas	58	2-5	2	6
Low-rise buildings	18	50-70	2	6

Medium-rise buildings	9	80-150	2-3	6
High-rise buildings	9	400-650	5	6
Offices	3	90-100	2-2.5	6
Governmental building	1	60	2	6
Schools	1	220	5	6
Hospitals	1	450	5	6

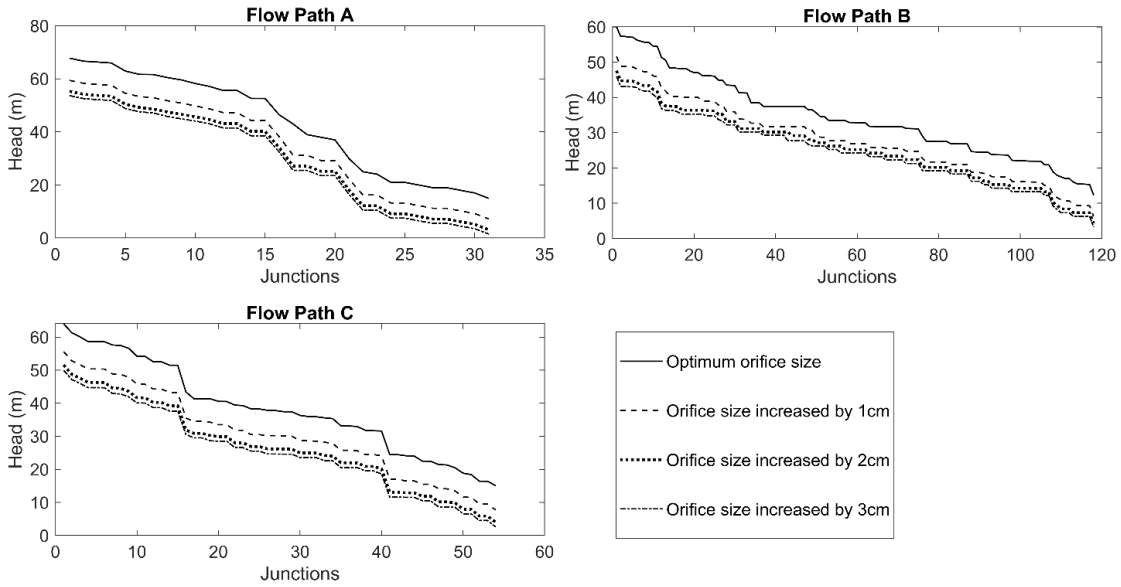
Table 1 shows the optimum (lowest) orifice diameter and tank volume to achieve a reliability of 1. It should be noted that, due to the high number of the nodal junctions, Table 1 does not include every single node, but rather those with similar characteristics based on their base demand. It also includes the percentage of each nodal type to showcase the characteristic of the network.

It is clear that the selection of orifice diameter depends on the base demands as higher orifice diameters are normally chosen for higher demands in the network. In general terms, most of the optimum orifice diameters in this network range between 2-3cm followed by the orifice diameter of 5cm. The optimum tank volume is that to achieve a retention time of 6 hours, for each case signifying the use of smaller tanks.



386 *Fig. 14 Flow path A, B and C for the DSO network*

387 To add further, the diameter of the orifice is now increased to showcase the impact on a real-time network.
388 In total, 3 flow paths are selected to capture the most significant parts of the network based on the pressure difference,
389 as shown in Fig. 14. Most of the nodes in flow path A and B consist of villas with low base demands while flow path
390 C consists of low to high rise buildings with high base demand. The resulting pressure variation established from these
391 flow paths for different orifice diameters under constant retention time are shown in Fig. 15.



393 *Fig. 15 Head (pressure) changes on nodal junctions with increasing orifice diameters for each flow path*

394 For the given flow path in Fig. 15, the results indicated that the pressure drops with increasing orifice
395 diameter. In fact, for flow path A, an increase in orifice diameter of 1cm results in a pressure drop close to 9m. Further
396 increasing orifice diameter to 3cm for each nodal tank shows a change of approximately 15m. The drop in pressure is
397 also higher for the junctions that had a larger optimum orifice diameter obtained from the reliability analysis (Table
398 1). This is observed from flow path A and B where the difference in pressure decreases at the end of the flow path
399 where the associated nodes type is villa. For flow path C, the nodal junction's types where relatively similar based on
400 the demands, hence, the pressure drop with increasing orifice diameter remains constant along the path. Finally, it is
401 worth noting the few discrepancies found in the flow path lines particularly when increasing the orifice diameter. It is
402 observed that the curve in flow path A, B and C, do not follow a smooth pattern (pressure rises for some nodes
403 downstream) when increasing the orifice diameters. This is because the orifice diameter for adjacent junction (that are
404 not present in the flow path) increases. Hence, it changes direction of flow slightly and shows these few anomalies.

405 The final comparison is conducted by changing the retention time and orifice diameter of local tanks to
406 observe the effect on the carbon footprint. The reliability analysis required both indicators to reach the value of '1' for
407 the tanks to have an acceptable orifice and volume size. As such, when the orifice and volume size increases, the
408 reliability values for both indicators would increase. This means the flow into the system would increase further which
409 would increase the pump power consumption that would increase the carbon footprint. Any further increase in the

optimum orifice and volume size would not be reflected in the reliability indicator since the maximum value obtained is '1'. However, logically, if the size of the parameters is being increased, then it would only result in more pump flow that would utilize more carbon footprint.

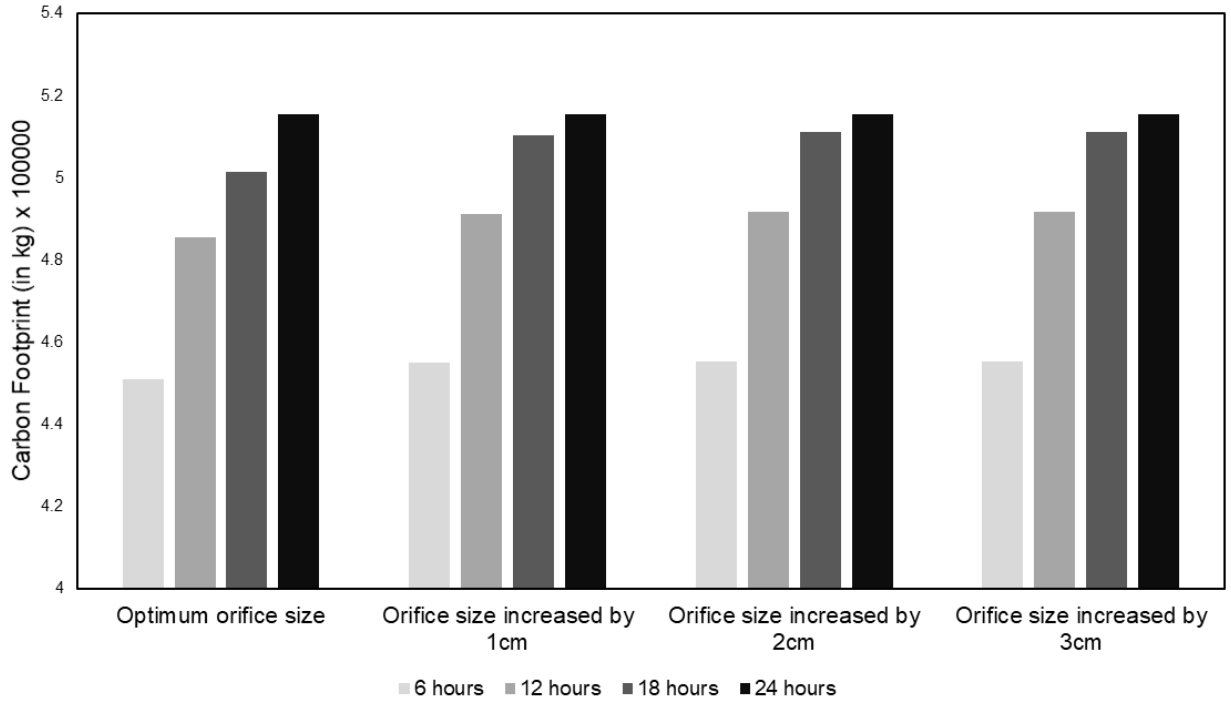


Fig. 16 Effect on carbon print with different orifice and volume sizes (retention time)

In this analysis (Fig. 16), it was observed that increasing the orifice diameter with constant retention time (6 hours) did not had a significant impact on carbon footprint. This is because the tank quickly reaches it full capacity within a few hours which causes the flow to become similar in all cases with different orifice diameter. On the contrary, increasing the retention time boosted the amount of carbon footprint produced as the tanks took relatively longer to fill up. Approximately, 10% of the carbon footprint has increased when the retention time is changed from 6 hours to 24 hours. The results confirm that the footprint is proportional to the energy used by the pumps (i.e., a function of the power supplied by the pumps). Indeed, it had been observed previously that the power consumption increases with retention time, the carbon footprint also increases further. The result was accepted, but it should be noted that the actual carbon footprint will also account for the lifestyle of the area, temperature and greenhouse emission rate. It further suggests that the retention time should be kept low to avoid the production of larger carbon footprint.

Conclusion:

The strategy to find the optimum volume and orifice diameter would allow the engineers to improve the current design of private tanks. As most of the associated code of practice recommends a 24-hour retention time for local tanks is essential, this study shows that in fact the orifice diameter is much more significant parameter to be optimized. Moreover, associating relatively high volume is not feasible as tank volume with 6 hours retention time are observed to have enough capacity to satisfy the demand.

The results have shown the importance of reliability indicators as they can assess the performance of the private tank by combining different factors that are affecting it. It is also found that there is a significant difference between the two indicators (time-based and volume-based). The difference in their value is due to the weightage provided by them. Nonetheless, each of them needs to reach the required reliability of 1 to satisfy the demand. These indicators were effective in this case since they were able to predict the correct parameters of tank that would not lead to failure by insufficient supply to the consumer, where others are not able to. However, they were not able to give weightage to the parameters of pressure and quality, which can be the essence of developing new indicators that can take these factors into account.

Overall, the results implied by the reliability indicators showed that the orifice and volume size of the tank is influenced by the base demand and available pressure. In all the networks, where the tanks were initially empty, the reliability value increases with the orifice diameters while changing the retention time did not had much impact. It further indicated that the selection of the orifice and volume size should always be based on the worst-case scenario (tanks are empty at the start of the simulation). In terms of carbon footprint and pressure, the results indicated that the optimum orifice and volume size is the lowest value that reaches the required reliability. Any further increase in the value (orifice and volume) would only increase the production of carbon footprint and head loss in the system. However, these results do not include the impact on the water quality which should be analyzed for future studies.

Data Availability:

- Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request. This includes data for the sample network 1 with all its characteristics.

- Some or all data, models, or code generated or used during the study are proprietary or confidential in nature and may only be provided with restrictions. This includes the data for sample network 2 which is a real-time network that includes sensitive and personal details of the consumer.

Acknowledgements

WDNetXL system used to run the simulation of all WDN models was provided by IDEA-RT s.r.l. (www.idea-rt.com).

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Table Captions:

Table 1 Optimum setting of the orifice and volume size for each type of node

Figure Captions:

Fig. 1 Actual scenario with private tanks, modified after (Giustolisi et al., 2014)

Fig. 2 Layout of a common underground private tank

Fig. 3 Schematic of Network 1, data from (Walski et al., 2003)

Fig. 4 Schematic of Network 2

Fig. 5 Impacts of orifice sizing on Volume (V is the volume, Q is the flow and d is diameter)

Fig. 6 Flow chart for the performed analysis

Fig. 7 Simulated storage for different retention times (orifice diameter constant at 2cm)

Fig. 8 Simulated storage for different orifice diameter of the private tanks (Volume size equivalent at retention time of 6 hours)

Fig. 9 Simulated flow for tanks 4 and 6 with retention time of a) 6 hours b) 12 hours c) 18 hours and d) 24 hours

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Fig. 14 Flow path A, B and C for the DSO network

Fig. 15 Head (pressure) changes on nodal junctions with increasing orifice diameters for each flow path

Fig. 16 Effect on carbon print with different orifice and volume sizes (retention time)