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# Optimization and reliability assessment of water distribution networks incorporating demand balancing tanks

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**Abstract**

This research aims to incorporate demand balancing tanks in network optimization and reliability assessment running extended period simulations. A tool called NORAT (**N**etworks **O**ptimization and **R**eliability **A**ssessment **T**ool) has been developed, which determines the required balancing volume, optimizes pipe diameters and tank elevations, and finally calculates the total costs. NORAT further assesses the hydraulic reliability of the network. The tool has been illustrated on a synthetic network by applying different combinations of topography, supply schemes, and locations of water sources and tanks. The results prove the ability of NORAT to employ balancing tanks, both in optimization and reliability assessment processes.


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*Keywords:* Optimization; Reliability; Demand balancing tanks; Water distribution networks.

## 1. Introduction

Water distribution networks (WDN) are vital part of urban infrastructure and require high investment, operation and maintenance costs. Their design is a complicated task due to strong interconnections between the network components and hydraulic parameters such as nodal pressures and demands. In theory, the design of a simple network consisting of only one water source and ten pipes by considering just three available pipe diameters comprises $3^{10}$ possible solutions. If the network is larger and includes other components such as pumps and tanks, the design process becomes additionally complicated. Traditional way to approach it in engineering practice is by trial and error while using rules of thumb and safety factors that usually provide non-optimal solutions. Motivated

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by these shortcomings, many optimization approaches have been proposed in the literature, such as linear and non-linear programming, heuristic approaches and global optimization algorithms. However, a completely efficient and accepted approach to optimize WDN layout is not yet available, Banos et al. (2010).

Due to high costs, optimization process of WDN usually starts by finding a least-cost solution that satisfies the required design criteria. This initially yields less attention to other aspects of WDN management, such as network reliability. Nowadays, the reliability of WDN has become a matter of concern for water utilities and researchers. Table 1 includes some of reliability definitions frequently mentioned in the literature.

Table 1: Definition of WDN reliability

| Researcher | Definition |
|---|---|
| Tung (1985) | 'Probability that flow can reach all the demand points in the network'. |
| Cullinane et al. (1992) | 'Ability of the system to provide service with an acceptable level of interruption in spite of abnormal conditions of water distribution system to meet the demand that are placed on it'. |
| Goulter (1995) | 'Ability of a water distribution system to meet the demands that are placed on it where such demands are specified in terms of the flows to be supplied (total volume and flow rate) and the range of pressures at which the flows must be provided'. |
| Xu and Goulter (1999) | 'Ability of the network to provide an adequate supply to the consumers, under both regular and irregular operating conditions'. |
| Tanyimboh et al. (2001) | 'Time-averaged value of the flow supplied to the flow required'. |
| Lansey et al. (2002) | 'Probability that a system performs its mission under a specified set of constraints for a given period of time in a specified environment'. |

Due to difficult process of both the optimization and reliability assessment of WDN, most of the researches focused only on the piping system, omitting other network components such as balancing tanks, pumps or valves. Despite the benefits that the tanks may bring, WDN are usually optimized without incorporating them into analyses. The objective of this research was to create a toll that will consider the demand balancing tanks in the optimization and reliability assessment processes to find out their influence on the total cost and service levels for given demand scenario on a typical consumption day.

## 2. Demand balancing tanks

Demand balancing tanks play an important role in WDN. They enable demand management, assure water supply during system failures and reserve water for emergency cases such as fire fighting, and allow for pump flow rate modulation. Tanks represent quite a small part of the whole network cost. Nevertheless, they have a significant impact on the overall network performance. If they are well-designed and located, they may improve the overall network performance and reduce the total cost. On contrary, they can increase the total cost of the network and reduce its performance. For instance, if the tank elevation is too high, the pressure in pipes can be also high, which increases the probability of pipe failure and water losses. If the elevation is too low, the pressure delivered can be insufficient and leads to hydraulic failure, Vamvakeridou-Lyroudia et al. (2007).

Batchabani and Fuamba (2012) discuss different approaches used in the design of demand balancing tanks. Accordingly, the tanks should be designed to compromise between minimizing the investment and operational costs and maximizing the network reliability. Design of any tank generally involves the following decision variables: supply volume (balancing, fire, and emergency volumes), hydraulic variables (maximum and minimum water levels), operational variables (maximum, minimum and normal operational levels) and construction variables (shape, type, location, and configuration of the outlet and inlet pipes). Tanks have been traditionally designed based on the local design guidelines (regulations) of the country. Guidelines are simple to implement, require less data collection, and useful to provide quick cost estimation during the design process. The weakness of the guidelines is that they usually focus on tank volume and not on the other variables such as elevation and location. In addition, they may cause a risk of tank over-sizing even if the WDN can operate efficiently with less storage volume.

Common optimization algorithms are suitable in principle to design the tanks considering all the decision variables mentioned above. However, increasing the number of decision variables complicates the optimization and increases the number of possible solutions exponentially. On the other hand, if too many decision variables are ignored, the optimization model can produce solutions that may be inapplicable technically and commercially.

## 3. Framework of NORAT

NORAT (**N**etworks **O**ptimization and **R**eliability **A**ssessment **T**ool) is a decision support tool able to optimize, calculate the cost and assess the reliability of WDN. The tool consists of two models; network optimization model (NetOpt model) and network reliability assessment model (NetRel model). NORAT has been developed in C++ language code with integrating the EPANET programmer's toolkit functions for hydraulic analysis. The optimization algorithm used is Evolving Objects (EO) of Keijzer et al. (2002), which is an unconstrained single-objective GA optimization algorithm.

### *3.1. NetOpt model*

NetOpt starts with calculation of the required tank volume (currently, the model can deal only with one tank), then optimizes the pipe diameters and tank elevation, and finally calculates the total cost of the network. The tank volume is calculated as mentioned in Trifunović (2006) assuming to have a cylindrical shape. This volume is consisting of the following volumes (Fig 1-a):

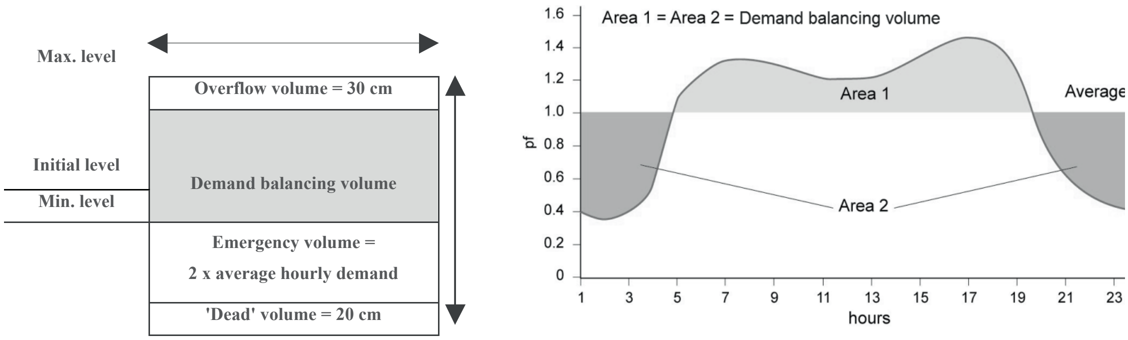


Fig. 1. (a) Required tank volume; (b) Demand balancing volume (adapted from Trifunović, 2006)

1. Demand balancing volume: equals the total water volume accumulated when the demand is below the required average value, which will be used later when the demand is above the average, Fig. 1(b).
2. Emergency volume for maintenance works, pipe failure events, fire fighting, etc.: assumed arbitrarily at twice the average hourly demand.
3. 'Dead' volume/depth to protect the tank from staying dry: assumed arbitrarily at 20 cm.
4. Overflow volume/depth protect the tank against the overflow: assumed arbitrarily at 30 cm.

By assuming the height (maximum level) of the tank, the model determines the required pipe diameters, minimum water level and initial water level at the beginning of the simulation. After calculating the required tank volume, NetOpt starts the optimization process, performed by using the EO algorithm. The optimization includes the pipe diameters and tank elevation based on minimizing the total cost. The optimization is constrained by minimum nodal pressure, maximum pipe unit head-loss and tank inflow/outflow that preserve the demand balance in the network.

The optimization process is done by generating many solutions (populations) and then selecting the best solution based on the objective function. In each population generation, the objective function firstly calculates the total cost of the pipes. Then, the function checks if the optimization constraints are satisfied or not with this population. If the constraints are not satisfied, the objective function will add a proportional penalty cost to the total cost. Finally, the objective function returns the total cost (including the penalty costs) to be evaluated. The optimal solution will be in this case the least-cost one that satisfies the optimization constraints. The whole process of the objective function is summarized in Fig. 2. There: $D$ and $L$ are the diameter and length of $m$ pipes, respectively; $Q_{avl/req}$ is the available/required tank flow (inflow/outflow), where the required tank flow is determined during calculating the demand balancing volume; $P_{avl/min}$ is the available/required minimum nodal pressure; $H_{avl/max}$ is the available/maximum pipe unit head-loss. $F1$, $F2$ and $F3$ are factors determined in advance by trial and error approach until reaching the best values that guide to the optimal solution.

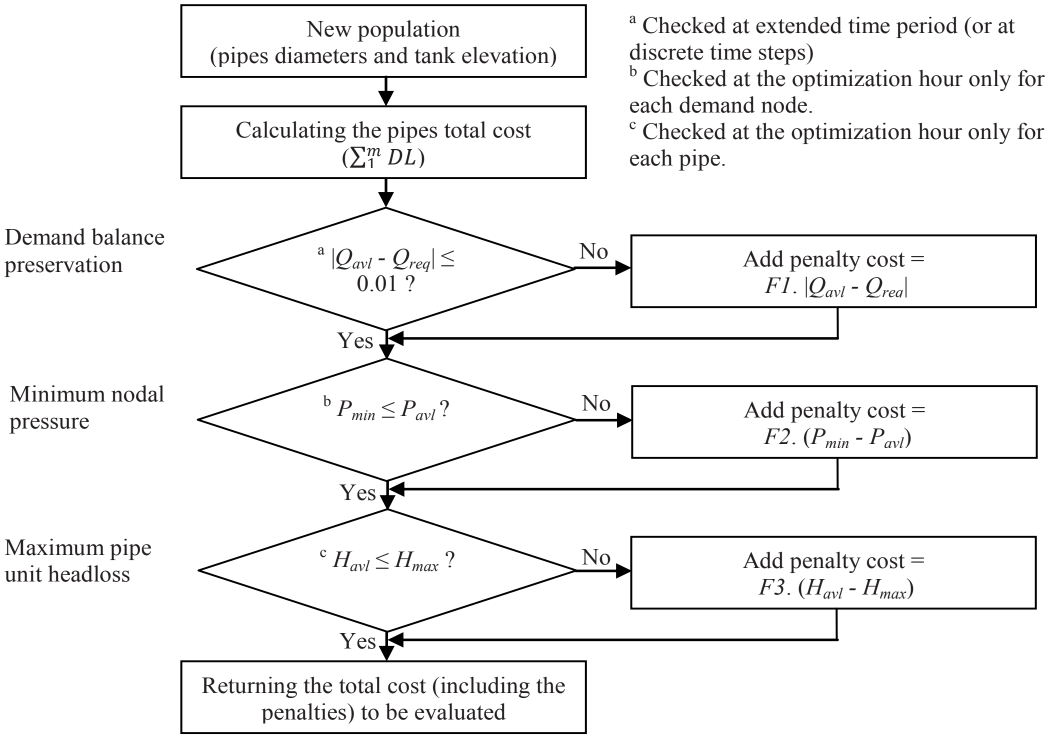


Fig. 2. Objective function in EO algorithm (NetOpt model)

Finally, NetOpt model calculates the total cost of the network based on the unit cost of each component, interest rate and design/repayment period. Cost calculations include calculating the total investment cost, operation and maintenance costs (O&M). The annual loan repayment is calculated using the annuity equation:

$$A = \frac{r(1+r)^n}{(1+r)^n - 1} P \tag{1}$$

where $A$ is the equivalent annual cost of the present investment cost $P$, $n$ is the project life in years, and $r$ is the interest rate.

### *3.2. NetRel model*

NetRel model assesses the hydraulic reliability of WDN by using the following three indices; Available Demand Fraction, *ADF*, of Ozger and Mays (2003), Network Buffer Index, *NBI*, of Trifunović (2012) and Network Resilience, $I_n$, of Prasad and Park (2004).

*ADF* is a reliability index that expresses the demand proportion still available in the network after pipe failure events. The calculation starts with non-failure condition and then by failing the pipes in sequence, calculating the network ADF for each one (Equation 2), and finally calculating the average value of *ADF* for the entire network (Equation 3).

$$ADF_{net} = \frac{\sum_{all\ nodes} Q^{avl}}{\sum_{all\ nodes} Q} \tag{2}$$

$$ADF_{avg} = \frac{1}{m}\sum_{j=1}^{m} ADF_{net,j} \tag{3}$$

In the above equations: $Q^{avl}$ is the available demand after pipe failure, $Q$ is the demand under normal conditions, $ADF_{net,j}$ is the $ADF_{net}$ corresponding to the failure of pipe $j$, and $m$ is number of pipes.

*NBI* of Trifunović (2012) is derived graphically from his hydraulic reliability diagram (HRD). By adding the weighting proportional to the pipe flows under regular supply conditions, NBI can be calculated as in Eq. (4).

$$NBI = 1 - \frac{\sum_{j=1}^{m}\left(Q_{tot} - Q_{tot,j}\right)}{\sum_{j=1}^{m} Q_j} \tag{4}$$

In the above equation, $Q_{tot}$ is the total demand in the network under normal supply, $Q_{tot,j}$ is the total demand in the network after the failure of pipe $j$, $Q_j$ is the flow in pipe $j$ under normal condition, and $m$ is the total number of pipes.

To determine the available demand in both *ADF* and *NBI* considerations, the hydraulic analysis should be performed by the pressure-driven demand simulation (PDD). The PPD simulation in NetRel is done by using the algorithm of Pathirana (2010). This algorithm considers three demand conditions:

1. Full demand: if $P_i \geq ECUP$, $Q_{i,PDD} = Q_{i,DD}$
2. Partial demand: if $0 < P_i < ECUP$, $Q_{i,PDD} = k_i P_i^{\alpha}$
3. No demand: $P_i \leq 0$, $Q_{i,PDD} = 0$

$P_i$ is the pressure at node $i$, $ECUP$ is threshold pressure, $Q_{i,PDD/DD}$ is the demand at node $i$ which is calculated by PDD and DD simulation respectively, and $k$ is the emitter coefficient which can be estimated by Equation 5:

$$k_i = \frac{Q_{i,DD}}{ECUP^{\alpha}} \tag{5}$$

Prasad and Park (2004) upgraded the resilience index $I_r$ of Todini (2000) to their network resilience $I_n$, based on the concept of the power balance:

$$I_n = \frac{\sum_{i=1}^{n} C_i Q_i \left(H_i - H_i^*\right)}{\sum_{s=1}^{l} Q_s H_s + \sum_{p=1}^{k} Q_p h_p - \sum_{i=1}^{n} Q_i H_i} \tag{6}$$

In the above equation, $H_{s/i}$ indicates the piezometric heads at $l$ sources (which includes all the reservoirs and tanks that supply the network), and the piezometric heads at $n$ nodes (which includes all the demand nodes and tanks supplied from the network) respectively. $H_i^*$ is the minimum piezometric head required to satisfy the demand at node $i$. Furthermore, $Q_{s/p/i}$ is the corresponding supplying flow ($s$), pump flow ($p$), and nodal demand flow ($i$), respectively. The adaptation of the Todini's index includes the index $C_i$ which takes care about the nodal uniformity, according to Equation 7 where $D_j$ is the diameter of $m$ pipes connected to node $i$:

$$C_i = \frac{\sum_{j=1}^{m,i} D_j}{m_i \max\left\{D_j\right\}} \tag{7}$$

## 4. NORAT Application

The case study selected to apply NORAT is Safi town network demonstrated as a design exercise in Trifunović (2006). The layout of the network is shown in Fig 4(a) including the baseline demands (l/s) and pipe lengths (m). All the nodes follow the diurnal demand pattern shown in Fig 4(b), except the factory that is working from 7 am to 7 pm with constant consumption. The seasonal peak factor is 1.406, which includes 10% water loss.

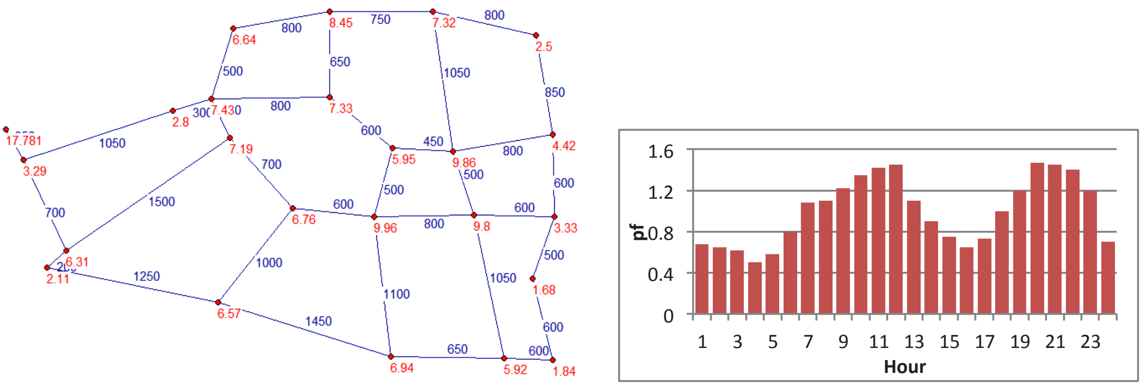


Fig 4. (a) Layout of Safi town network (including nodal base demands and pipes lengths); (b) Domestic demand pattern.

### 4.1. Scenarios

The original network layout has been transformed into 12 variants by combining:

1. Three topographic terrains; flat, hilly and valley, as shown in Fig 5.
2. Two pipe configurations: fully-looped and quasi-looped, shown in Fig 6(a) and (b). The quasi-looped cases have adapted total demand for the factory node is removed. To avoid possible confusion, the fully-looped and quasi-looped schemes are further named as 'looped' and 'branched', respectively.
3. Two source locations; at the edge and in the middle of the network, shown in Fig 6(a) and (b).

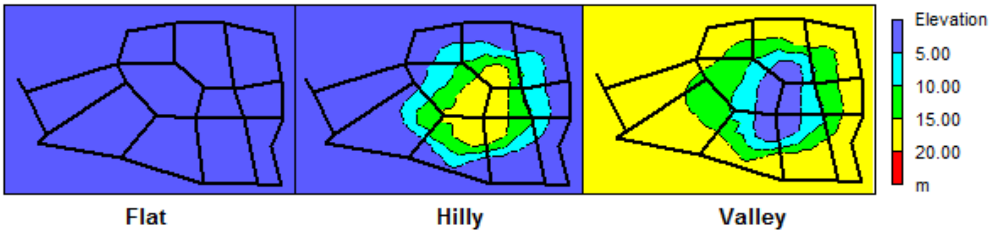


Fig 5. Topographic terrains.

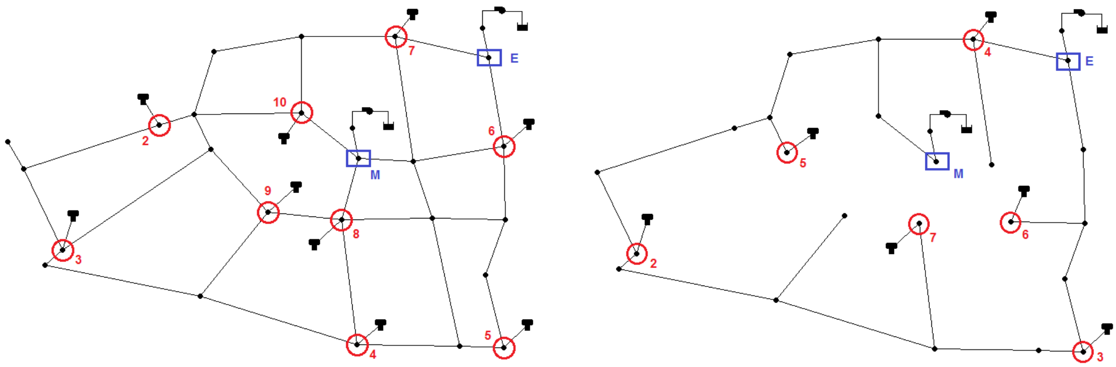


Fig 6. (a) Looped scheme with source and tank locations; (b) Branched scheme with source and tank locations.

In each of the 12 cases, the network is firstly analyzed with the existing water source and then by incorporating demand balancing tank at different locations in the network; nine locations in the looped networks and six locations in the branched networks. The locations of the tanks are selected at the border and in the middle of the network, to be close/far from the water source. The sources and tanks are connected to the by 300 and 500 m pipe respectively, and have elevations equal to the network nodes they connect. In total, 102 network variants are generated from the original network.

Each network is named as XYZN, where X denotes the topography (F - flat, H - hilly, and V - valley), Y is the network scheme (L - looped and B - branched), Z is the source location (E - edge and M - middle), and N is the tank location number as in Fig 6(a) and (b) (N is 1 if there is no tank incorporated in the network).

*4.2. Application steps*

Each network variant has been a subject of the following steps:

1. The required pumping capacity: number of pumps, duty heads and flows and pumping operation schedule have been specified and based on the pump efficiency pattern as shown in Table 2.

Table 2: Pumps efficiency

| Flow (Q) | 0.25 $Q_{duty}$ | 0.5 $Q_{duty}$ | 1.0 $Q_{duty}$ | 1.5 $Q_{duty}$ | 1.75 $Q_{duty}$ |
|---|---|---|---|---|---|
| Efficiency (%) | 20 | 60 | 75 | 65 | 30 |

2. NetOpt optimisation based on the settings shown in Table 3.

Table 3: NetOpt model settings

| Parameter | Value |
|---|---|
| Minimum nodal pressure constraint | 20 m |
| Maximum pipe unit head-loss constraint | 5 m/km |
| Optimization hour | Maximum demand hour |
| Maximum tank elevation | 30-40 m |
| Maximum tank volume height | 7 m |
| Pipe laying costs, in USD/m: | |
| D = 80 mm | 60 |
| D = 100 mm | 70 |
| D = 150 mm | 90 |
| D = 200 mm | 130 |
| D = 300 mm | 180 |
| D = 400 mm | 260 |
| D = 500 mm | 310 |
| D = 600 mm | 360 |
| Investment cost pumping station (USD) | $5\times10^{3}\times Q^{0.8}$ , Q = maximum installed flow ($m^3$/h) |
| Constructing tanks (USD) | $35\times10^{4}+150\times V$, V = tank volume ($m^3$) |
| Elevated tank, supporting structure (USD) | $3\times H\times V$, V = volume ($m^3$), H = elevation (m) |
| O&M cost of pipes | 0.5 % of total pipe investment cost |
| O&M cost of pumping stations | 2.0 % of pump investment cost |
| O&M cost of tanks | 0.8 % of tank investment cost |
| Energy cost | 0.15 USD/kWh |
| Interest rate | 8 % |
| Design period | 20 years |

3. Checking if the optimization constraints are satisfied. If not, the source pumping capacity is not sufficient and steps 1 and 2 must be repeated with new pumping capacity. The penalty cost factors can also provide unsatisfactory results, but these were tested in advance.
4. The demand balancing pattern analyzed four times; at every 1, 2, 4 and 6 hours (the network will be optimized four times), where the least cost design that satisfies the constraints will be selected.
5. The reliability of the optimized network assessed by NetRel based on the settings in Table 4. The minimum tank volume in the optimized network file generated from NetOpt model includes the dead and emergency volumes. The minimum volume in the reliability analyses is reduced to the dead volume to enable NetRel to consider the emergency volume.

Table 4. NetRel model settings

| Parameter | Value |
|---|---|
| Assessment hour | Maximum demand hour |
| Minimum pressure | 20 m |
| Emitter exponent | 0.5 |

6. Analyzing the output results of both NetOpt and NetRel models.

*4.3. Results*

The optimizations based on running EPS took anything between five (FBM3) and 64 minutes (FLE7) depending on the variant, and using a standard laptop computer. All 102 simulations resulted in the minimum pressures in the networks ranging from 19.38 m (HBM5) up to 21.78 m (HLM3), the vast majority being in the vicinity of the targeted minimum pressure of 20 m. The range of maximum unit head-loss was between 1.55 m/km (FBE6 and FBM7) and 5.02 m/km (HLM9). Based on the same demand in all the cases per category, the calculated volume of balancing tanks was the same. Regardless the location, the satisfactory balancing pattern was maintained in all the cases, which is illustrated in Fig. 7 showing HLE and VBE categories. Expectedly, in virtually all the cases the costs of branched configurations were lower than the costs of loped configurations, which is illustrated in Fig. 8 showing the same categories as in Fig. 7.

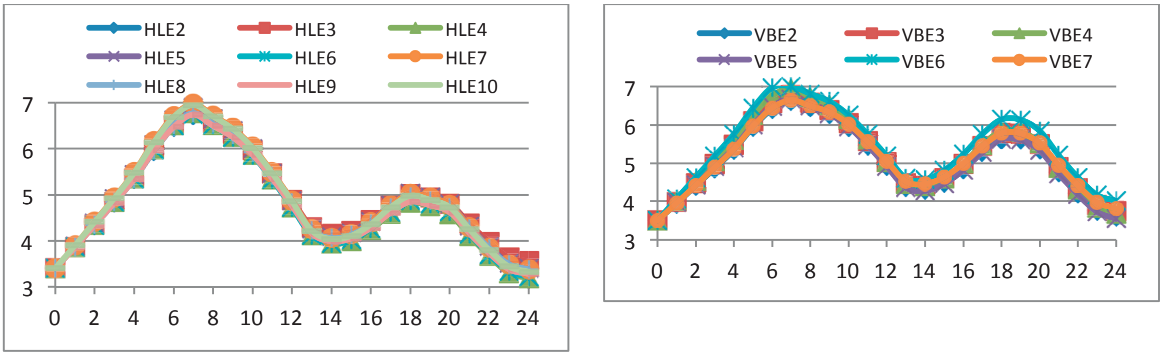


Fig 7. Tank volume variation: Y-tank depth (m), X - time (hours)

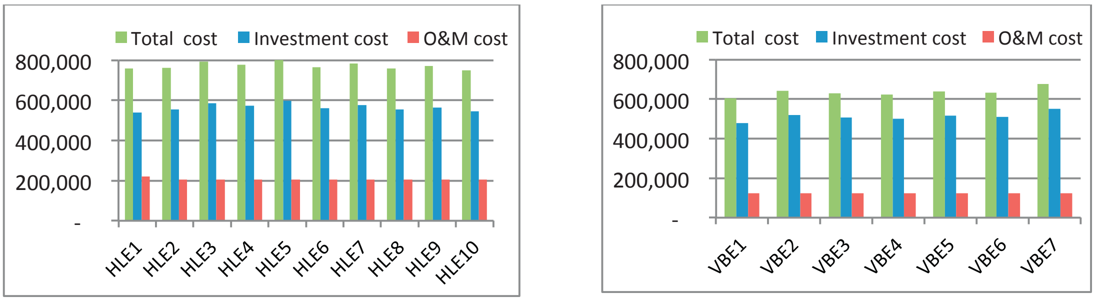


Fig 8. Total costs per variant (USD)

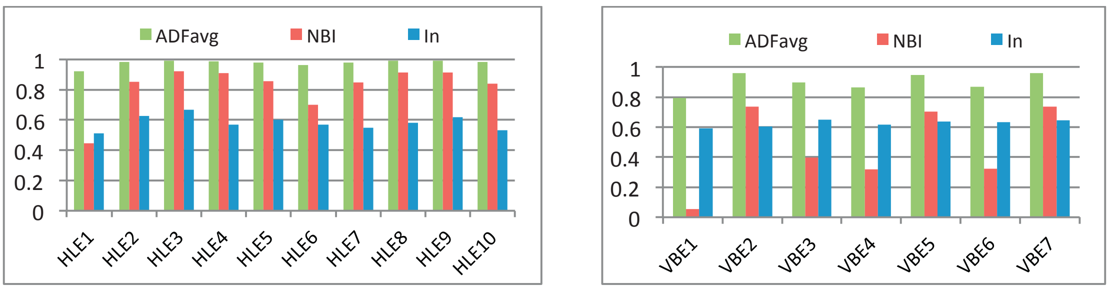


Fig 9. Reliability measures per variant

Finally, the overview of the reliability indices for the given variants is shown in Fig. 9. Despite the discrepancies between the values of $I_n$ on one side, and $ADF_{avg}$ and $NBI$ on the other side, it is visible in all the categories that the variant without tank (the first in the bar charts), is less reliable than the rest. Significantly different values in Fig. 9 originate from the nature of the indices, which has been discussed by Trifunović (2012).

## 5. Conclusions

The presented results show a synthetic network to illustrate the NORAT performance and output results. Thus, drawing more precise i.e. general conclusions on the benefits from having a balancing tank in the network and its impact on the reliability is much more complex task. By all means, incorporating the demand balancing tank at an appropriate location can decrease the total cost and increase the reliability of the network. On contrary, the ill tank location can increase the total cost and decrease the reliability. No general trend can be proposed to determine where the best location of the tank is in the network. The best tank location can be only determined by generating several design scenarios with placing the tank at different locations based on the land availability and finally selecting the best scenario by the compromise between the total cost and reliability of the network. Each case in practice will be specific on its own, and the tool like NORAT should help to assess it thoroughly and make design choices with higher degree of confidence; not offer a readymade optimal solution.

Nevertheless, NORAT proved able to consider the demand balancing tanks in the optimization and reliability assessment processes. It also proved to be a robust decision support tool providing full assessment and helping to trade off between the available design alternatives and draw the conclusion about the best one by the compromise between the reliability and the total cost. Finally, further steps in the development of NORAT are needed to include multiple tanks, as well as tanks which have multiple purposes in the network, such as water towers.

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