International Journal of World Policy and Development Studies
ISSN(e): 2415-2331, ISSN(p): 2415-5241
Vol. 5, Issue. 8, pp: 71-81, 2019
URL: https://arpgweb.com/journal/journal/11
DOI: https://doi.org/10.32861/ijwpds.58.71.81

Academic Research Publishing Group

Original Research Open Access

Water-Demand Growth Modelling in Puerto Ayora's Water Distribution Network Using EPANET

María Reyes*
Environmental Engineering and Water Technology Department, IHE Delft (Institute for Water Education), Netherlands

Nemanja Trifunović
Environmental Engineering and Water Technology Department, IHE Delft (Institute for Water Education), Netherlands

Saroj Sharma
Environmental Engineering and Water Technology Department, IHE Delft (Institute for Water Education), Netherlands

Maria Kennedy
Environmental Engineering and Water Technology Department, IHE Delft (Institute for Water Education), Netherlands
Faculty of Civil Engineering and Geosciences, Delft University of Technology, The Netherlands, Netherlands

Abstract

This paper elaborates the hydraulic characteristics of the water supply network of the town of Puerto Ayora. First, it intends to replicate the household individual storage by simulating nodal tanks with the use of the EPANET software. Later, it uses the Pressure-Driven Approach (PDA) to develop a methodology that estimates the overflow of storage facilities, one of the main sources of wastage in Puerto Ayora. Finally, it uses the Demand-Driven Approach (DDA), with the aim of assessing the network in the future, under four population growth scenarios. With the chosen moderate growth scenario, two options are suggested in order to tackle the water supply issues at the end of the planning horizon.

1. Introduction

Water demand tends to exceed the available supply capacity, especially in many developing countries, as a result of rapid population growth (Ingeduld *et al.*, 2006). As a consequence, Intermittent Water Supply (IWS) regimes are introduced with the aim of limiting that demand. Water scarcity in arid regions is also amplified when there is a lack of conveying capacity in the distribution network, which is too deteriorated to deliver the required demands (Ameyaw *et al.*, 2013). Even though the water distribution should be equitable and efficient in such cases (Vairavamoorthy *et al.*, 2008), IWS have become a norm rather than an exemption (Seetharam and Bridges, 2005), mainly due to necessity rather than the initial design (Vairavamoorthy *et al.*, 2007).

IWS varies depending on the region and situation, ranging from a few hours per day to a few hours per week. Therefore, in order to compensate the periods of interruption, end-users need to store as much water as possible during service hours. Therefore, the water distribution is not homogenous, creating high peak factors and inadequate pressures in the distribution system (Andey and Kelkar, 2009). Consequently, the system becomes unreliable due to pumps and pipes failing to carry the required water during supply (Ameyaw *et al.*, 2013).

Individual storage facilities, usually tanks, play important role in IWS systems, since these are the solely supply at moment of unavailability. The water inflow into each household tank relies on the pressure conditions in the network and it equals to the maximum amount of water that can be collected during the supply hours (Ingeduld *et al.*, 2006). Therefore, water demand balance takes place for each individual household storage tank, "whereby replenishing of the volume behaves differently depending on water availability in the distribution network" (Trifunović and Abu-Madi, 1999). Furthermore, the water is consumed according to the demand patterns which are not necessarily influenced by the IWS regime, but rather by common household habits and activities.

1.2. Case Study Description

Tourist islands face additional pressure when tackling scarcity, while aiming to optimise the revenues from tourism and providing sufficient environmental protection at the same time. Puerto Ayora as the main tourist hub of Santa Cruz Island, has a distribution network built approximately in the 1980's, which has 2500 service connections and supplies intermittently brackish water to approximately 12,000 inhabitants (INEC, 2010). Due to the lack of proper maintenance, the network is characterized by high leakage levels whose actual figure are still not confirmed. Most premises in this town have storage facilities, mainly in the form of roof-tanks and/or cisterns, perceiving the supply as unreliable and insufficient. On the other hand, the fixed water-tariff structure seems to be the main cause of excessive water wastage within the premises. In short: the municipal supply service has not been able to cope with current tourist growth trends (Reyes Pérez, 2017).

*Corresponding Author

The aim of this paper is to analyse the performance of the current hydraulic network in Puerto Ayora, evaluating the water losses from spilling of roof tanks. During fieldwork data collection, a peculiar situation was observed regarding the overwhelming amount of wastage from the overflow of roof-tanks. The origin of this type of losses seems to be due to the absence of floating valves and the fact that the faucets are not closed when the tank is already full. Therefore, a methodology was developed for the quantification of water overflow of household-roof tanks, which further questions the currently applied IWS regime using the EPANET model. The hydraulic simulations for the current study were conducted by running the Pressure-Driven Analysis (PDA) feature of the EPANET software, while the future demand scenarios were simulated by the Demand-Driven Analysis (DDA).

2. Demand-Driven Analysis (DDA) and Pressure-Driven Analysis (PDA)

The default hydraulic solver for water network modelling commonly uses the Demand Driven Analysis (DDA). DDA assumes that nodal demands are known functions of time, and are independent of the available pressure in the distribution system (Cheung *et al.*, 2005). The hydraulic solver produces the nodal pressures and pipe/pump flows which satisfy those fixed nodal demands. The DDA under regular supply conditions presents a reasonable and close-to reality solutions, assuming that the nodal demands are always delivered. The algorithm in the modelling software is able to formulate the needed equations in order to solve the unknown nodal heads (Ozger and Mays, 2003), regardless of the pressures throughout the distribution system. However, these algorithms are unable to capture precisely the how intermittent systems behave, operating under irregular conditions.

The studies developed by Germanopoulos (1985), Martinez *et al.* (1999), Soares *et al.* (2003), Hayuti and Burrows (2004) and Tanyimboh *et al.* (2004), discuss the restrictions of DDA. On the contrary, these studies suggest that the use of the PDA, which assumes a fixed demand above given pressure threshold, zero demand below the given minimum pressure, and proportional relationship between the pressure and the demand for the pressure range between the threshold and the minimum values is a more reliable method in irregular situations (Cheung *et al.*, 2005). The PDA approach aims to replicate a pressure-demand relation in the modelling process, using the concept of orifices at system nodes. Consequently, the previous-mentioned studies suggest that PDA tend to be more effective than DDA at the moment of simulating intermittent water supply conditions.

Tanyimboh *et al.* (2001), describe the PDA relationship as follows (Reyes Pérez, 2017):

$$H_i = H_i^{\min} + K_i Q_i^n \qquad \text{(Equation 1)}$$

where H_i represents the actual head at demand node i, H_i^{min} refers to the minimum head to which below the service ends, K_i is the resistance coefficient for node i, Q_i refers to the nodal discharge flow, and n is the exponent that theoretically and usually takes the value of 2.0 (Gupta and Bhave, 1996). Furthermore, if the value of Q_i is unknown for any given nodal head, then Equation 8.1 needs to be rearranged as follows:

$$Q_i = \left(\frac{H_i - H_i^{\min}}{K_i} \right)^{1/n} \qquad \text{(Equation 2)}$$

If Q_i equals the required demand, Q_{req}, H_i should then equal the desired head in the node, named H_{des}. If the demand at that node further needs to be fully satisfied, then the head should be available as follows:

$$Q_i^{req} = \left(\frac{H_i^{des} - H_i^{\min}}{K_i} \right)^{1/n} \Rightarrow \frac{1}{K_i^{1/n}} = \frac{Q_i^{req}}{\left(H_i^{des} - H_i^{\min}\right)^{1/n}} \qquad \text{(Equation 3)}$$

Finally, Equation 4 is obtained by substituting K_i in Equation 8.2:

$$Q_i^{avl} = Q_i^{req} \left(\frac{H_i^{avl} - H_i^{\min}}{H_i^{des} - H_i^{\min}} \right)^{1/n} \qquad \text{(Equation 4)}$$

where Q_i^{avl} refers to the flow for the head available at the node (H_i^{avl}). Equation 4 has three probable situations:

1) $H_i^{avl} \leq H_i^{\min} \Rightarrow Q_i^{avl} = 0$
2) $H_i^{\min} < H_i^{avl} < H_i^{des} \Rightarrow 0 < Q_i^{avl} < Q_i^{req}$
3) $H_i^{avl} \geq H_i^{des} \Rightarrow Q_i^{avl} = Q_i^{req}$

These situations are used when balancing the flows in the pipes, which are connected to node i. The key issue is the correct definition for H_i^{min} and H_i^{des}, such as their correlation with the nodal resistance K_i, which is the one that describes the nature of the PDD empirical relationship.

The EPANET software, developed by Rossman (2000) uses the PDA concept through Emitter Coefficients (EC), which model pressure-dependant flows from sprinkler heads. The concept of EC is described by using similar relationships as in Equation 1. In this case, an emitter is modelled as a dummy pipe connected to the actual demand node, with a dummy reservoir whose nodal elevation (z) equals the initial head. Hence, $H_i^{min} = z_i$ and:

$$Q_i = \frac{1}{K_i^{1/n}} \left(H_i - z_i\right)^{1/n} \qquad \text{(Equation 5)}$$

The K-value in Equation 5 refers to the resistance of the dummy pipe, but actually it has the same meaning as in Equations 1 to 3. Finally,

$$Q_i = k_i\left(\frac{p_i}{\rho g}\right)^{\alpha} \quad ; \quad \frac{p_i}{\rho g} = H_i - z_i \quad ; \quad \alpha = 1/n \quad ; \quad k_i = \frac{1}{K_i^{\alpha}} \qquad \text{(Equation 6)}$$

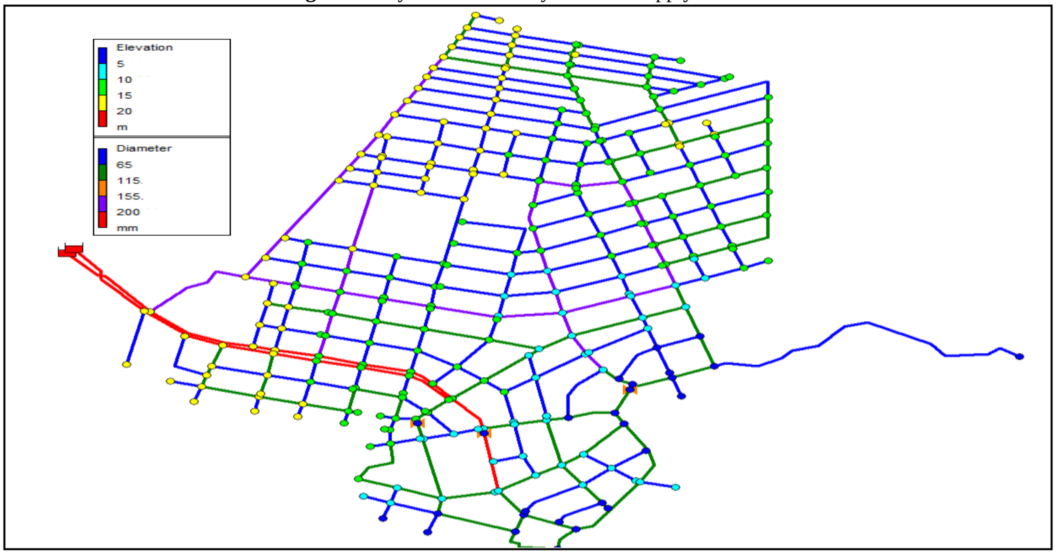
where k_i is the EC in node i and α is an emitter exponent with theoretical value of 0.5. EC was first introduced to simulate operation of fire hydrants.

In order to use the PDA accurately, it is required extensive field data collection in order to determine the relationship between nodal heads and flows (Ozger and Mays, 2003). Other PDA approaches have been based on further improvement of the EC concept of the EPANET, such as the one by Pathirana (2010).

3. Methodology

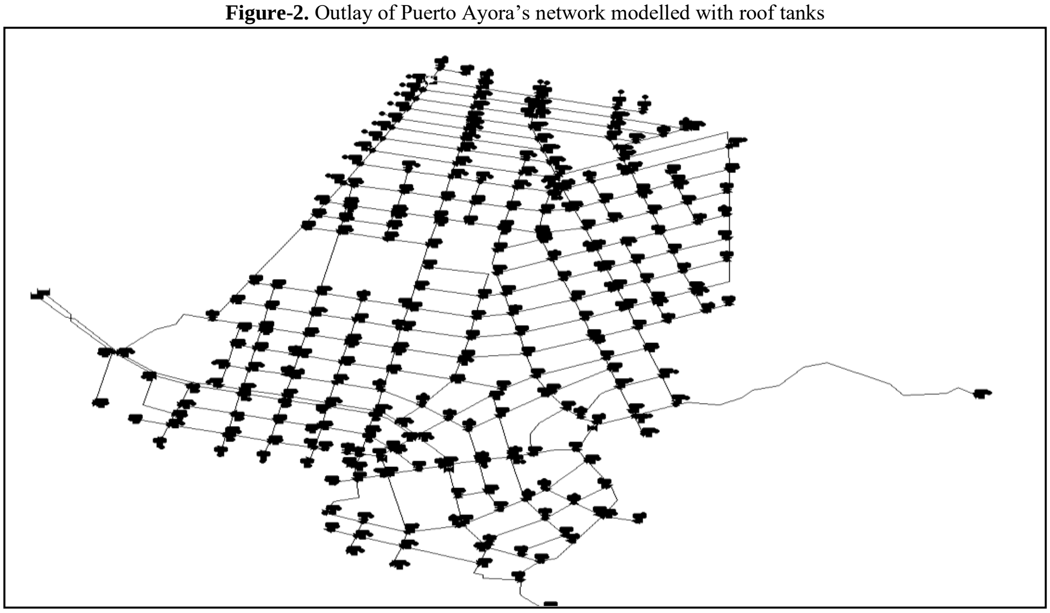
The gravity hydraulic network model was built in the EPANET software, as shown in Figure 1, which consists of two reservoirs, 284 nodes and 367 pipes. The estimation of nodal demands was done based on the total population and an estimated demography per m^2, using the demand patterns established for this particular case study found in Reyes *et al.* (2017b).

Figure-1. Layout of Puerto Ayora water supply network



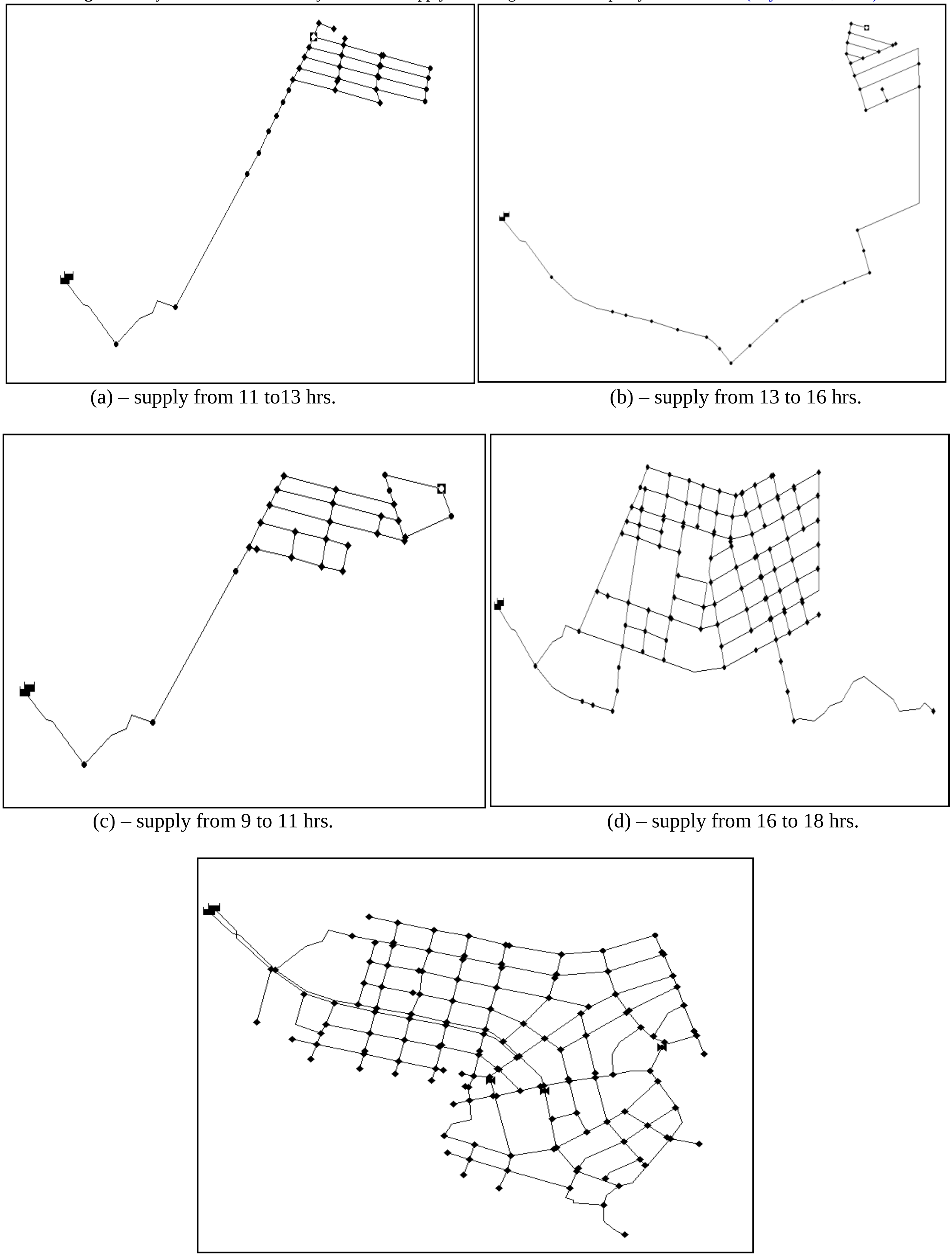
To check the numerical stability, as well as the robustness of the EPANET program, each node was modelled as being connected to a tank, which's was calculated by the number of occupants supplied at that node, and the elevation determined was that corresponding to that node; this model is shown in Figure 2. An average tank height of 1.5 m above the ground level average, as well as an average of five inhabitants per household was assumed. All tanks were assigned the initial depth of 1 m, the minimum depth of 0 m, and the maximum depth of 3 m. Also, a check-valve was modelled with a short dummy pipe, in order to prevent backflow from the storage tank (i.e simulate the inlet arrangement from the top).

Figure-2. Outlay of Puerto Ayora's network modelled with roof tanks



Also, as an alternative, the PDA approach was simplified in order to translate the rationing as it is applied in reality. To make this possible, the network modelled with EC was divided into five small sub-systems (Figure 3), representing each distribution zone created by the municipality. In order to match the previously reported total amount of supply per day, which is approx. 5000 m^3 for the horizon 2015, including the leakage (Reyes *et al.*, 2016), different EC values were tried for each zone and supply hours, adopting an EC value of 0.5.

Figure-3. Layouts of zones divided by distributed supply according to the Municipality of Santa Cruz (Reyes Pérez, 2017).



(a) – supply from 11 to13 hrs.

(b) – supply from 13 to 16 hrs.

(c) – supply from 9 to 11 hrs.

(d) – supply from 16 to 18 hrs.

(e) – supply from 6 to 9 hrs.

The assessment of the overflow of the nodal tanks was done per zone, using several scenarios created by varying the leakage, the average volume of individual storage tanks, and the percentage of the tank level at the moment the water supply starts in a particular distribution zone.

The volumes considered in making the balance for each node of the five sub-models are further shown in Equations 7 – 10 (Reyes Pérez, 2017).

$$V_i^{avl} = n_{h,i} X_1 \qquad \text{(Equation 7)}$$

V_i^{avl} is the total available individual storage volume in node i (in m^3), $n_{h,i}$ is the number of households served from the node, and X_1 is the variable that indicates the average storage volume available per household (m^3).

$$V_i^{c24} = n_{c,i} X_2 \qquad \text{(Equation 8)}$$

V_i^{c24} is the total volume consumed in node i over 24 hours (in m^3), $n_{c,i}$ is the number of consumers served from the node, and X_2 is the variable that indicates the average specific demand per capita (lpcpd).

$$V_i^{s24} = Q_i^{EC} h_i^s \left(1 - \frac{X_3}{100}\right) \qquad \text{(Equation 9)}$$

V_i^{s24} is the total volume supplied to node i over 24 hours (in m^3); this is an IWS that occurs during h_i^s hours at the flow Q_i^{EC} (in m^3/h) based on the available pressure calculated in EPANET using the emitter coefficients. X_3 is the variable that indicates the average leakage percentage in node i.

$$V_i^{cIWS} = \sum_{j=1}^{h_i^s} \frac{V_i^{c24}}{24} pf_{j,i} \qquad \text{(Equation 10)}$$

V_i^{cIWS} is the total volume consumed in node i (in m^3) during the IWS period of h_i^s hours, at hourly peak factors $pf_{j,i}$ applied depending on the period of the day when the IWS takes place (for the diurnal patterns, see (Reyes *et al.*, 2017a). Hence, the actual volume accumulated in the tank(s) of node i during the IWS period is $V_i^{s24} - V_i^{cIWS}$. Assuming X_4 to be the variable that indicates the percentage of the total available volume V_i^{avl} already occupied at the moment when the IWS starts, the buffer of volume in the tank(s) of node i, V_i^{buf}, when the IWS stops, will be:

$$V_i^{buf} = V_i^{avl}\left(1 - \frac{X_4}{100}\right) - \left(V_i^{s24} - V_i^{cIWS}\right) \qquad \text{(Equation 11)}$$

Possible negative result in Equation 11 will indicate the overflow i.e. the spilling from the tank(s) of node i. Furthermore, when the IWS stops, the tank(s) will be discharged for the volume $V_i^{c24} - V_i^{cIWS}$ suggesting the initial volume before the IWS starts again to be:

$V_i^{avl} - (V_i^{c24} - V_i^{cIWS})$, if the overflow was taking place during the IWS ($V_i^{buf} < 0$);

$V_i^{avl} - V_i^{buf} - (V_i^{c24} - V_i^{cIWS})$, if the overflow was <u>not</u> taking place during the IWS ($V_i^{buf} \geq 0$).

In both of these cases, the result can in theory be negative, suggesting the water shortage and/or insufficient volume of the tanks. Assuming that this is not the case, some indication exists while assessing the values for X_4.

For instance, a sample calculation done for IWS Zone 1 shown in Figure 3(a) is given in Table 1, taking X_1 = 1.5 m^3, X_2 = 163 lpcpd, h_i^s = 2 hours, X_3 = 17.5%, pf_1 = 1.29, pf_2 = 1.66, and X_4 = 0. With these, the total overflow amounts at approximately 114 m^3, which is about 42% of the total volume supplied (of 269 m^3).

Table-1. Estimation of tanks' volumes of overflow in IWS Zone 1

	$n_{c,i}$	$n_{h,i}$	V_i^{avl} (m^3)	X_2 (lpcpd)	h_i^s (hours)	V_i^{c24} (m^3)	V_1^{cIWS} (litres)	V_2^{cIWS} (litres)	Q_i^{EC} (l/s)	V_i^{s24} (m^3)	$V_i^{s24} - V_i^{cIWS}$ (m^3)	V_i^{buf} (m^3)
J-1	31	6	6.2	163	2	5.1	272.9	351.2	2.1	12.4	11.7	-5.5
J-2	29	6	5.8			4.7	252.1	324.4	2.0	12.1	11.5	-5.8
J-3	18	4	3.6			2.9	157.0	202.1	2.1	12.5	12.2	-8.6
J-42	15	3	3.0			2.5	133.1	171.3	2.1	12.4	12.1	-9.1
J-43	27	5	5.4			4.4	238.1	306.4	2.0	11.8	11.3	-5.8
J-44	15	3	3.0			2.5	132.2	170.1	2.0	11.8	11.5	-8.5
J-45	27	5	5.3			4.3	233.3	300.2	2.1	12.5	11.9	-6.6
J-46	39	8	7.8			6.4	342.1	440.3	2.0	11.9	11.2	-3.4
J-47	23	5	4.5			3.7	198.8	255.8	2.1	12.6	12.1	-7.6
J-48	47	9	9.3			7.6	408.9	526.2	2.0	11.6	10.7	-1.3
J-50	27	5	5.4			4.4	238.3	306.7	1.8	10.5	10.0	-4.5
J-64	28	6	5.7			4.6	249.4	321.0	2.0	11.7	11.1	-5.4
J-65	54	11	10.9			8.8	475.7	612.1	2.1	12.5	11.4	-0.5
J-66	27	5	5.4			4.4	237.6	305.8	2.1	12.5	11.9	-6.5
J-67	44	9	8.9			7.2	389.5	501.3	2.0	12.1	11.2	-2.3
J-68	28	6	5.6			4.5	244.3	314.3	2.2	12.8	12.3	-6.7
J-69	35	7	7.1			5.8	311.0	400.2	2.2	13.2	12.5	-5.4
J-70	29	6	5.8			4.7	254.1	326.9	2.0	12.1	11.5	-5.7
J-78	54	11	10.8			8.8	473.6	609.4	2.1	12.7	11.6	-0.8
J-79	47	9	9.3			7.6	407.9	524.9	2.0	11.8	10.9	-1.6
J-80	26	5	5.2			4.2	227.6	292.9	2.0	11.7	11.2	-6.0
J-81	34	7	6.8			5.5	298.0	383.5	2.3	13.5	12.9	-6.1

Source: Reyes Pérez (2017)

This assessment was carried out for the entire network, and the influence of each variable was further evaliated. Finally, the water supply network was assessed for the future demand under the same four tourist and local population growth scenarios used in Chapter 6 (Mena *et al.*, 2013). The analysis was done in the intervals of five years, as shown in Table 2. The same growth rate was used for all nodal demands, while the tourist demand was distributed amongst the nodes located in the center of the town (where most of the tourist facilities are).

Table-2. Local and tourism population growth estimations for 30 years planning horizon in Puerto Ayora

YEAR	Total growth rate	Local population (inhab.)	Demand local population (m3/day)	No. of tourists/ year	Tourist consumption (m3/day)
SLOW GROWTH					
2015	-	15,801	2,576	205,505	1,109
2020	0.05	16,607	2,707	215,780	1,165
2025	0.10	17,454	2,845	226,055	1,220
2030	0.15	18,345	2,990	236,331	1,276
2035	0.20	19,280	3,143	246,606	1,331
2040	0.25	20,264	3,303	256,881	1,386
2045	0.30	21,087	3,437	267,156	1,442
MODERATE GROWTH					
2015	-	15,801	2,576	205,505	1,109
2020	0.15	18,318	2,986	236,331	1,276
2025	0.30	21,235	3,461	267,156	1,442
2030	0.45	24,618	4,013	297,982	1,608
2035	0.60	28,539	4,652	328,808	1,775
2040	0.75	33,084	5,393	359,634	1,941
2045	0.90	37,236	6,070	390,459	2,107
FAST GROWTH					
2015	-	15,801	2,576	205,505	1,109
2020	0.25	20,167	3,287	256,881	1,386
2025	0.50	25,738	4,195	308,257	1,664
2030	0.75	32,849	5,354	359,634	1,941
2035	1.00	41,925	6,834	411,010	2,218
2040	1.25	53,508	8,722	462,386	2,496
2045	1.50	65,040	10,601	513,762	2,773
VERY FAST GROWTH					
2015	-	15,801	2,576	205,505	1,109
2020	0.35	22,174	3,614	277,432	1,497
2025	0.70	31,116	5,072	349,358	1,886
2030	1.05	43,665	7,117	421,285	2,274
2035	1.40	61,275	9,988	493,212	2,662
2040	1.75	85,987	14,016	565,139	3,050
2045	2.10	112,759	18,380	637,065	3,438

Source: (Reyes Pérez, 2017)

4. Modelling, Results and Discussion

4.1. Current Situation Simulated with Nodal Tanks

The fist simulation was conducted with the network model shown in Figure 2. This model indicates 578 nodes, 286 tanks and 940 pipes. The latter originates from the setup of the discharge from the tanks that consists of a dummy pipe attached to the node with a consumption pattern, and another dummy pipe on the upstream side of each tank, representing a check-valve; shown in Figure 4. The model setup becomes rather bulky; however it is the only way to reflect the actual reality.

After several attempts, the simulation runs did not presented any tangible result. First of all, the model appeared to be very sensitive to the selection of the tank properties and the initial water levels. As soon as the tank gets emptied or filled from the imbalance between the demand and supply, based on the EPANET settings, the tank becomes disconnected from the network. This suggests that this inflicts numerical instability, which was also experienced by significantly longer simulation times than usual, and resulting in negative pressures in the network.

Figure 4 shows the documented negative pressures in the model, while Figure 5 shows the trends of water level variation in randomly selected tanks in the same distribution area.

Figure-4. Layout of Puerto Ayora's network with nodal tanks

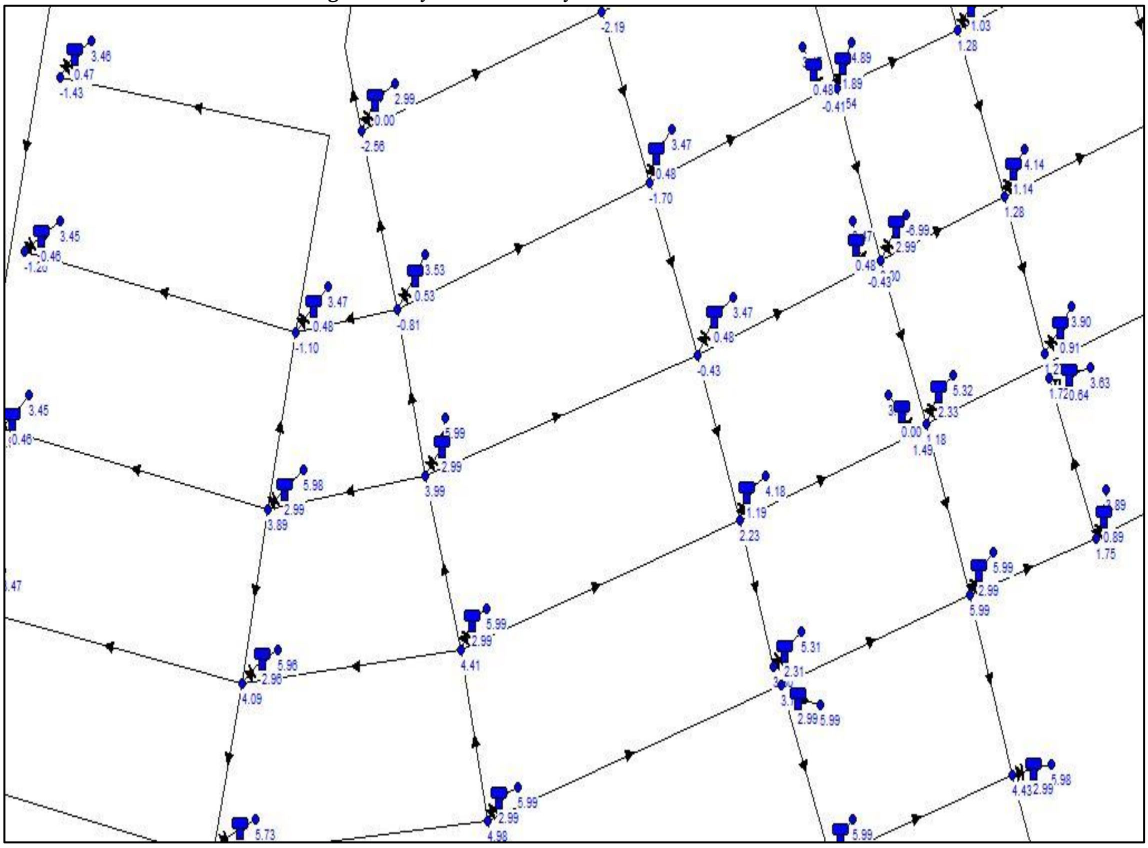
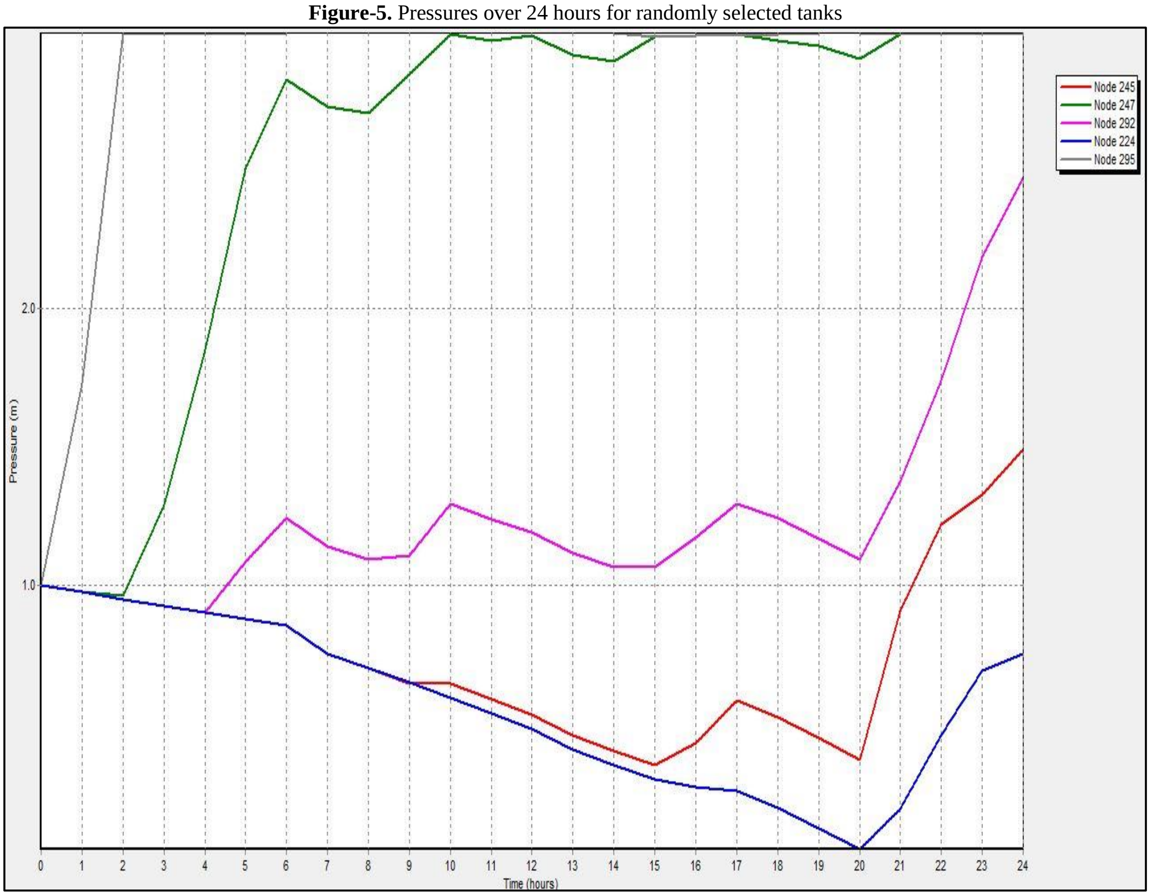


Figure-5. Pressures over 24 hours for randomly selected tanks



Therefore, the calibration of such a model appeared to be very complex and it suggested that the amount of tanks influenced somehow; the model could work only if these are not entirely emptied or completely full. The model becomes unstable and might only work if all the tanks have sufficient amount of water throughout the entire day. In that case, analyses of irregular water distribution schemes become very challenging, and the other approach was tested.

4.2 Results of Pressure-Driven Approach with Emitter Coefficients

The alternative way of modelling was therefore adopted using the approach illustrated by Equations 7 to 11 and in Table 1. The 16 selected scenarios based on the values X_1 to X_4 and the results showing the buffer volume are shown in Table 3. Two specific demand scenarios (X_2) correspond to the figures obtained by the field survey described in Reyes *et al.* (2017b). Similarly, the leakage scenarios (X_3) were set by assuming the physical leakage to be 50% of the total NRW levels (assessed at 35 and 50%, respectively). The negative buffer indicates the spilling from the tanks with the 2nd figure showing the value reduced for 30%, which is believed to be the percentage of the households having the float valves installed on their roof tanks.

Table-3. Scenarios for the estimate of tank overflows in Puerto Ayora

X_2 (lpcpd)	X_3 (%)	X_1 (m^3)	X_4 (%)	V_i^{buf} (m^3)	V_i^{buf} (2) (m^3)	% total daily supply
163	**Scenario 1**					
	17.5	1	0	-1327	-929	18.6
	Scenario 2					
	17.5	1	50	-2301	-1611	32.2
	Scenario 3					
	17.5	2	0	-423	-296	5.9
	Scenario 4					
	17.5	2	50	-709	-496	9.9
	Scenario 5					
	25	1	0	-1053	-737	14.7
	Scenario 6					
	25	1	50	-1957	-1370	27.4
	Scenario 7					
	25	2	0	-287	-201	4.0
	Scenario 8					
	25	2	50	-1053	-737	14.7
195	**Scenario 9**					
	17.5	1	0	-1285	-900	18.0
	Scenario 10					
	17.5	1	50	-2239	-1567	31.3
	Scenario 11					
	17.5	2	0	-411	-287	5.7
	Scenario 12					
	17.5	2	50	-1285	-900	18.0
	Scenario 13					
	25	1	0	-1016	-712	14.2
	Scenario 14					
	25	1	50	-1900	-1330	26.6
	Scenario 15					
	25	2	0	-298	-208	4.2
	Scenario 16					
	25	2	50	-1016	-712	14.2

Source: Reyes Pérez (2017)

In terms of the percentage of daily supply, the results show a wide range between 4.2 and 32.2 %, which indicates sensitive input data. Based on the field observations at the case study area, it is believed that Scenario 6 may be the closest to the reality, with remark that the tank levels at the beginning of the IWS next day may vary in filling percentages, which depends on the consumption pattern of the previous day; hence, the values of X_4 need specific validation compared to the other three variables.

Based on the results in Table 3, an assessment carried out to calculate the additional supply in case the overflow from the tanks could have been prevented. Table 4 shows the additional population that could be supplied for each of the 16 chosen scenarios, assuming two specific demands used in the analysis of the tank buffers. As can be observed, in the most extreme situation of Scenario 2, almost 60% of additional population could have been supplied at the lower specific demand of 163 lpcpd, and nearly 50% at the higher one of 195 lpcpd. These results put a valid hypothesis about the actual necessity of the roof tanks, since they may not be easing the current intermittent situation, but actually boosting it, resulting from the negligence of local population.

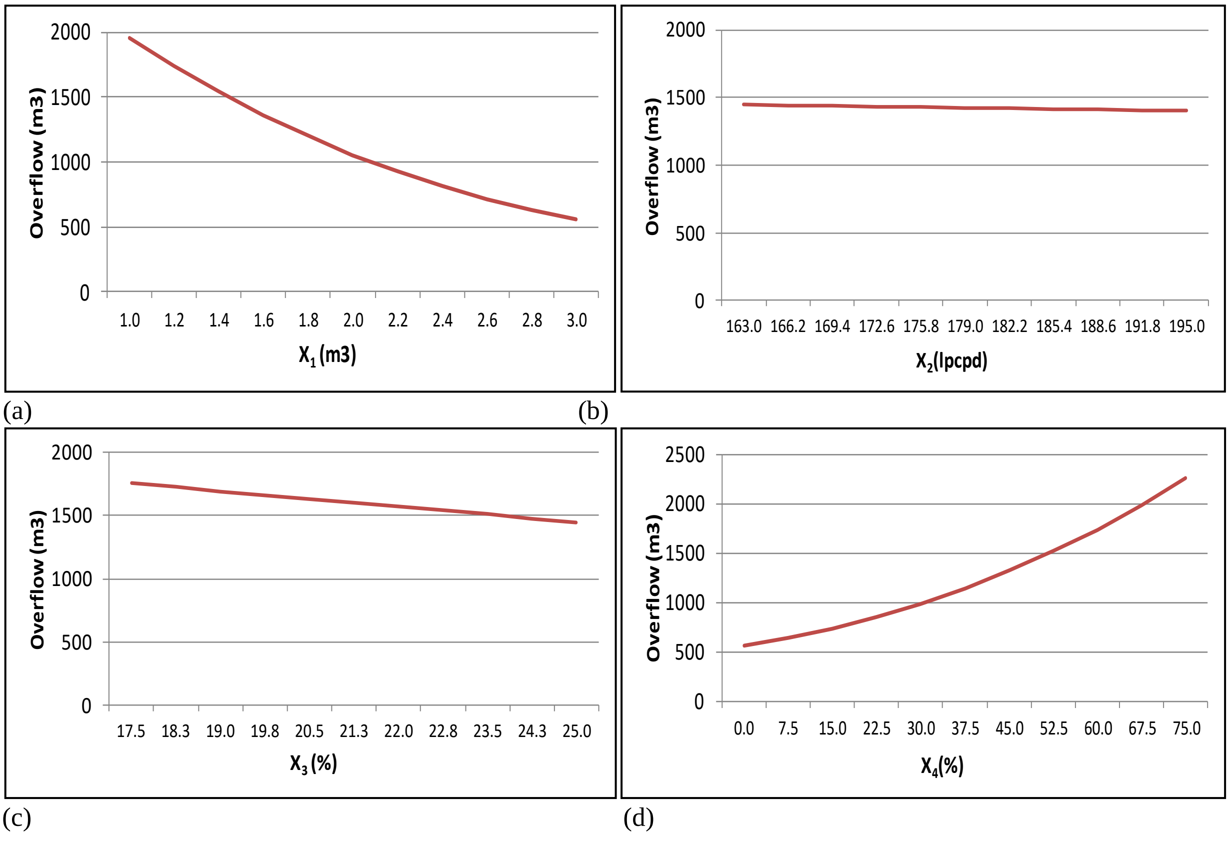
Table-4. Additional population to be supplied based on different overflow scenarios

Scenarios	Specific Demand of 163 lpcpd	Percentage of population	Specific Demand of 195 lpcpd	Percentage of population
1	5697	33.5	4762	28.0
2	9882	58.1	8261	48.6
3	1817	10.7	1519	8.9
4	3043	17.9	2543	15.0
5	4521	26.6	3779	22.2
6	8403	49.4	7024	41.3
7	1234	7.3	1032	6.1
8	4521	26.6	3779	22.2
9	5519	32.5	4614	27.1
10	9614	56.6	8036	47.3
11	1763	10.4	1474	8.7
12	5519	32.5	4614	27.1
13	4365	25.7	3649	21.5
14	8158	48.0	6819	40.1
15	1278	7.5	1068	6.3
16	4365	25.7	3649	21.5

Source: (Reyes Pérez, 2017)

The sensitivity of the four *X*-variables was further analysed against the tank overflow in somewhat wider range than the one applied in the 16 scenarios. Figure 6 shows that the highest sensitivity belong to the variables X_1 and X_4, which underpins the need for further fieldwork in order to validate the available volume (individual household storage), and the initial level of the tanks before the IWS kicks-off every day. On the other hand, the specific demand, as well as the average leakage percentage, influence the overflow volume to a lesser extent.

Figure-6. Sensitivity analysis based on different (a) individual tank volume, (b) specific demand, (c) leakage level and (d) tank level in total overflow estimation



5. Conclusions

Three modelling approaches illustrated in this paper, included nodal tanks, the PDA and the DDA, for the current demand in Puerto Ayora. In the first case, it was very complicated to replicate the actual individual storage in the model. The EPANET software becomes unstable due to different water level patterns in each tank, sometimes full and sometimes empty. More detailed information would be needed for a proper calibration of such a model but even then it is possibly that the model could perform only regular demand scenarios. Unfortunately, EPANET disconnects empty tanks which distorts its numerical stability in multiple occurrence of this situation.

The PDA of present demand reflects partially the situation with the household storages. The aim was to estimate the overflow of roof tanks under several scenarios. As observed from the results, it can be concluded that there may be a significant amount of water lost due to the overflow of the roof tanks. The local authorities in Puerto Ayora, as well as local residents, have the erroneous idea of the "need" for individual storage to compensate the lack of water in the hours of no supply, which to the large extent may result from the negligence.

The sensitivity analysis suggests which input data should be verified with priority. The result show the size of individual household tanks and the initial level when the supply begins have more impact on the overflow, than the leakage percentage or per capita demand. Despite the fact that many assumptions were made, this analysis provides a practical approach to measure the volume that might be spilled from household tanks, which seems to be the main source of water wastage. The municipality would need to monitor and record individual characteristics of the households' storage facilities, in order to assess more accurately the extent of this problem. Also, the model should be further calibrated by adequate choice of emitter coefficients.

Finally, the research points that the hydraulic modelling of distribution networks in tourist islands poses quite a complex problem due to: (1) numerical instability caused by multiple tanks existing in the model, and (2) difficult calibration from lots of unknown and inaccurate data needed to build a reliable model.

References

Ameyaw, E. E., Memon, F. A. and Bicik, J. (2013). Improving equity in intermittent water supply systems. *Journal of Water Supply: Research and Technology-AQUA,* 62(8): 552-62.

Andey, S. P. and Kelkar, P. S. (2009). Influence of intermittent and continuous modes of water supply on domestic water consumption. *Water Resources Management,* 23(12): 2555-66.

Cheung, P., Van Zyl, J. and Reis, L. (2005). Extension of EPANET for pressure driven demand modeling in water distribution system. *Computing and Control for the Water Industry,* 1(1): 311-16.

Germanopoulos, G. (1985). A technical note on the inclusion of pressure dependent demand and leakage terms in water supply network models. *Civil Engineering Systems,* 2(3): 171-79.

Gupta, R. and Bhave, P. R. (1996). Comparison of methods for predicting deficient-network performance. *Journal of Water Resources Planning and Management,* 122(3): 214-17.

Hayuti, M. and Burrows, R., 2004. "Sequential solution seeking dda based hda (sss-dda/hda) approach." In *Proc. of Decision Support in the Water Industry under Conditions of Uncertainty-ACTUI 2004.*

INEC, 2010. "Censo de población y vivienda del ecuador 2010." In *Ecuador, Instituto Nacional de Estadísticas y Censos.*

Ingeduld, P., Svitak, Z., Pradhan, A. and Tarai, A., 2006. "Modelling intermittent water supply systems with EPANET." In *8th Annual Water Distribution Systems Analysis Symposium, Cincinnati.*

Martinez, F., Signes, M., Savall, R., Andrés, M., Ponz, R. and Conejos, P. (1999). *Construction and use of dynamic simulation model for the valencia metropolitan water supply and distribution network.* Water Industry Systems: Modeling and Optimization Applications, In: Savic, DA and Walters, GA. 1: 155-74.

Mena, C., Walsh, S., Pizzitutti, F., Reck, G., Rindfuss, R., Toral-Granda, V. D. O., Valle, C., Quiroga, D., García, J., Vasconez, L., Guevara, A., Sanchez, M., Frizelle, B. and Tippett, R. (2013). *Determination of social, environmental and economical relations which allow the development based on different processes of modeling, potential scenarios of sustainability of the socio ecological system of the Galapagos Islands with emphasis on the dynamic of the flux of tourist visitors.* Ministry of Environment and Galapagos National Park. 1: Galapagos.

Ozger, S. S. and Mays, L., 2003. "A semi-pressure-driven approach to reliability assessment of water distribution networks." In *Proceedings of the Thirtieth Congress.*

Pathirana, A., 2010. "EPANET2 desktop application for pressure driven demand modeling." In *In Water Distribution Systems Analysis 2010.* pp. 65-74.

Reyes, Trifunović, N., Sharma, S. and Kennedy, M. (2016). Data assessment for water demand and supply balance on the island of santa cruz (galápagos Islands). *Desalination and Water Treatment,* 57(45): 21335-49.

Reyes, Trifunović, N., Sharma, S. and Kennedy, M. D. (2017a). Assessment of domestic consumption in intermittent water supply networks: Case study of Puerto Ayora (Galápagos Islands). *Journal of Water Supply: Research and Technology-Aqua,* 55(8): 673-83.

Reyes, Trifunovic, N., d'Ozouville, N., Sharma, S. and Kennedy, M. (2017b). Quantification of urban water demand in the island of santa cruz (galápagos archipelago). *Desalination and Water Treatment,* 64(1): 1-11.

Reyes Pérez, M. F. (2017). *Water supply and demand management in the galápagos: A case study of.* CRC Press: Santa Cruz Island.

Rossman, L. A. (2000). EPANET 2: Users manual. Available: https://www.microimages.com/documentation/Tutorials/Epanet2UserManual.pdf

Seetharam, K. and Bridges, G., 2005. "Helping India achieve 24x7 water supply service by 2010." In *Roundtable discussion on private sector participation in water supply in India.* pp. 15-16.

Soares, A. K., Reis, L. F. R. and Carrijo, I. B., 2003. "Head-driven simulation model (HDSM) for water distribution system calibration. Advances in Water Supply Management." In *Proceedings of the ccwi '03 conference, london, 15-17 september 2003.* pp. 197-207.

Tanyimboh, T., Tabesh, M. and Burrows, R. (2001). Appraisal of source head methods for calculating reliability of water distribution networks. *Journal of Water Resources Planning and Management,* 127(4): 206-13.

Tanyimboh, T., Walters, G., Savic, D., Khoo, S. T. and King, R., 2004. "Availability of water in distribution systems. In Decision support in the water industry under conditions of uncertainty." In *Proceedings of the 2004 EPSRC ACTUI network international seminar.* pp. 159-64.

Trifunović, N. and Abu-Madi, M. O. R., 1999. "Demand modelling of networks with individual storage." In *In WRPMD'99: Preparing for the 21st Century*. pp. 1-10.

Vairavamoorthy, K., Gorantiwar, S. D. and Mohan, S. (2007). Intermittent water supply under water scarcity situations. *Water International*, 32(1): 121-32.

Vairavamoorthy, K., Gorantiwar, S. D. and Pathirana, A. (2008). Managing urban water supplies in developing countries–Climate change and water scarcity scenarios. *Physics and Chemistry of the Earth, Parts A/B/C,* 33(5): 330-39.