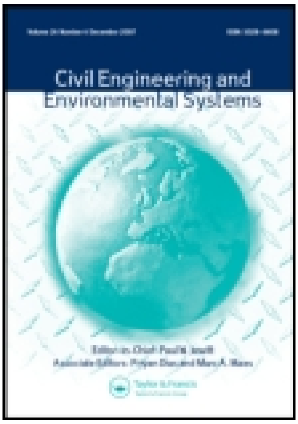

Civil Engineering and Environmental Systems
Publication details, including instructions for authors and subscription information:
http://www.tandfonline.com/loi/gcee20

Comparative analysis of approaches to modelling water distribution networks

Stephen Nyende-Byakika [a] , Gaddi Ngirane-Katashaya [b] & Julius M. Ndambuki [a]

[a] Department of Civil Engineering, Tshwane University of Technology, Pretoria, South Africa

[b] Department of Civil and Environmental Engineering, Makerere University, Kampala, Uganda
Version of record first published: 23 Feb 2012.

To cite this article: Stephen Nyende-Byakika , Gaddi Ngirane-Katashaya & Julius M. Ndambuki (2012): Comparative analysis of approaches to modelling water distribution networks, Civil Engineering and Environmental Systems, 29:1, 79-89

To link to this article: http://dx.doi.org/10.1080/10286608.2012.663358

Civil Engineering and Environmental Systems
Vol. 29, No. 1, March 2012, 79–89

Comparative analysis of approaches to modelling water distribution networks

Stephen Nyende-Byakika[a]*, Gaddi Ngirane-Katashaya[b] and Julius M. Ndambuki[a]

[a]*Department of Civil Engineering, Tshwane University of Technology, Pretoria, South Africa;*
[b]*Department of Civil and Environmental Engineering, Makerere University, Kampala, Uganda*

(*Received 1 June 2011*)

Models for water distribution networks have become indispensable tools for understanding system behaviour by simulating pressures and flows at different locations and times in the networks. This helps inform actions that should be taken in order to improve network performance. The rationale behind traditional modelling of water networks uses a simplifying assumption that pressures in the network are a function of demand, so that it is demand that drives network performance. This demand-driven approach yields good results when pressures in the network are generally sufficient, but exhibits profound weaknesses as it produces unreliable results when network pressures are low. It therefore appears that if the network to be modelled has low pressures, then rather than employing the aforementioned demand-driven approach, the pressure-/head-driven approach should be used. In demand-driven analysis, the higher the outflow the lower the pressure while in head-driven analysis, the higher the pressure, the higher the outflow. The key difference and advantage of the head-driven approach is observed when a more realistic network model is analysed. This paper uses a real-world network to clarify the difference between the demand-driven and head-driven approach, and highlights the superiority of the latter during low-pressure conditions.

Keywords: demand-driven analysis; head-driven analysis; water distribution networks; modelling; simulation; pressures; flows

1. Introduction

Most cities rely on networks of pressurised water supply systems for conveyance of potable water. Thus, networks are designed to operate under a full-time pressurised flow regime. Since pressures are assumed constantly available to satisfy demands defined at nodes throughout the network, models for analysing water distribution networks consider nodal demand as a primary model input while pressure is regarded as a primary output. This is the so-called demand-driven analysis (DDA) of water distribution networks which represents the traditional approach to water distribution network modelling. This approach is well developed and valid for scenarios in which pressures in a system are adequate for delivering required nodal demands. It gives realistic results when network pressures are high enough to meet demand, however, should the pressures fall substantially, unrealistic and meaningless results are obtained.

*Corresponding author. Email: stenbyak@gmail.com

ISSN 1028-6608 print/ISSN 1029-0249 online

http://dx.doi.org/10.1080/10286608.2012.663358
http://www.tandfonline.com

From the solution of continuity and energy equations that underlie the operation of models in order to determine nodal hydraulic heads and pipe flows, demands are always fixed but pressures vary (see, e.g. Rossman 2000). Subsequently, when demand is excessive, in order to satisfy fixed demand, the models compute heads below the minimum required for outflow to occur physically at some or all of the nodes (Chandapillai 1991, Ang and Jowitt 2006, Wu *et al*. 2006, 2009) and this represents a fundamental weakness of water distribution models. This has led to development of analysis that incorporates a relationship between demand and pressure, also called pressure-/head-driven analysis (PDA/HDA), which resulted in desirable solutions by showing compensation among nodal demands and available pressures (Cheung *et al*. 2005).

In practice, there can be periods with shortfalls in pressure and this affects the quantity of water that can be supplied. In addition, in the face of inadequate water supply, in order to provide equitable distribution of available water from constrained systems, intermittent water supplies with reduced system pressures are often introduced. When this happens, the quantity of water that can be supplied depends on the available pressure, thus, pressure becomes the driver of network performance. In recent years, the pressure dependency of withdrawals from water distribution systems under conditions of abnormal stress has been a subject of intensive research. Many researchers have attempted to predict behaviour of water distribution systems under pressure-deficient conditions (Chandapillai 1991, Ang and Jowitt 2006, Wu *et al*. 2006, 2009, Hayuti *et al*. 2007, Martínez-Solano *et al*. 2008) and have demonstrated that under conditions of insufficient pressures, the amount of water that can be supplied at a node directly depends on the pressure available at the node.

In this paper, the authors carry out a comparative analysis of performance of the demand-driven and head-driven approaches to water distribution modelling using a case study of the Rubaga subsystem of the Kampala Water Supply Network (KWSN), Uganda. In KWSN, there are sections which experience severe pressure shortfalls brought about by inadequate production, inadequate distribution and unplanned and excessive network expansions, leading to several cases of very low or no flow at all and this directly limits supply (Nyende-Byakika *et al*. 2010, Nyende-Byakika 2012). In such situations, traditional methods of analysis have limitations; demands in this case are not only a function of time, but of pressure as well and consequently, DDA alone fails when abnormal conditions prevail (Hayuti *et al*. 2007). Using PDA, an iterative method that uses a pressure–flow relationship is employed, that adjusts to both demand and pressure in order to yield an optimum solution that represents equilibrium of a water distribution network and certainly, the reality on the ground.

2. Methods

Methods that were employed in the study included characterisation of the Rubaga subsystem, development of a model of the Rubaga subsystem and analysis of response of the network under various scenarios. Data collected on the KWSN included water produced and supplied, pipe layout, pipe sizes and elevations, pipe lengths and materials, valves, reservoirs, pumps, consumption patterns (estimation of nodal demands) and pressures, heads and flows at strategic sections, well supplied zones and poorly supplied zones.

The second step entailed building a model of the Rubaga network in the EPANET2 hydraulic solver using the network data obtained under the demand-driven approach. The modelling process involved network schematisation, model building, testing and problem analysis. Principal hydraulic input parameters for pipes are start and end nodes, diameters, lengths and roughness coefficients for determining head loss. Computed outputs for pipes included flow rate, velocity and head loss. The hydraulic head lost by water flowing in a pipe due to friction with the pipe walls was computed using the Darcy–Weisbach formula (Equation (1)).

$$h_f = \lambda \frac{l}{d} \frac{v^2}{2g}, \tag{1}$$

where h_f is the head loss (m), λ the Darcy–Weisbach friction coefficient, l the length of duct or pipe (m), d the pipe diameter (m) and v the velocity (m/s).

The Rubaga subsystem was modelled to comprise of 22 pipes and one 4000 m^3 reservoir. A model of the schematised water distribution network of the Rubaga subsystem, showing node and link identification numbers (IDs) is shown in Figure 1 while node elevations and base demands are shown in Table 1. Pipe lengths and diameters are shown in Table 2.

Testing and calibration of the model of the existing system to field observed values was done for a range of operating conditions in order to evaluate the model's ability to represent actual

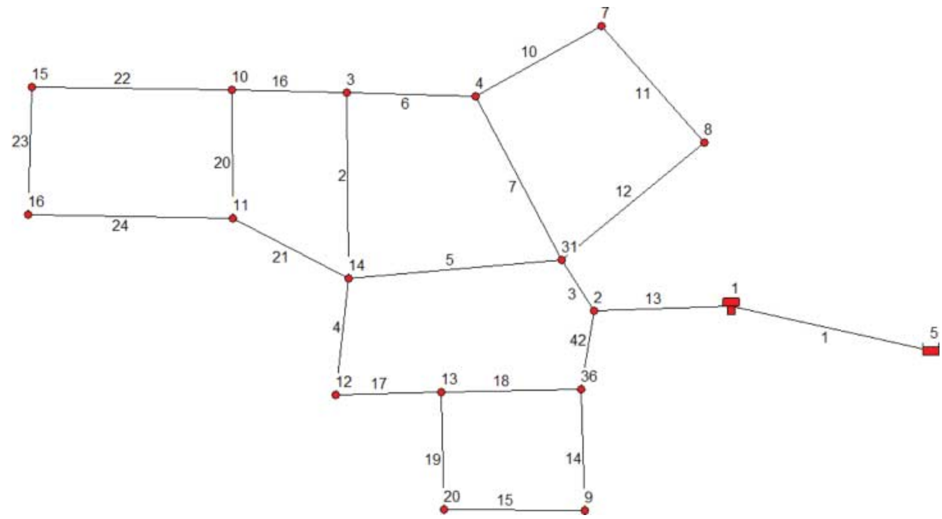

Figure 1. Model of Rubaga subsystem showing node and link IDs.

Table 1. Node elevations and base demands.

Node ID	Elevation (m)	Base demand (l/s)
2	1194	3.4
14	1184	3.5
20	1182	3.3
31	1190	3.4
36	1176	3.3
12	1167	3.5
13	1170	3.5
3	1172	3.5
4	1182	3.3
7	1180	3.2
8	1175	3.2
9	1178	3.3
10	1175	3.5
11	1180	3.5
15	1175	3.3
16	1174	3.5
Reservoir 5	1300	N/A
Tank 1	1200	N/A

Table 2. Pipe lengths and diameters.

Link ID	Length (m)	Diameter (mm)
42	150	400
13	413	500
1	5300	500
17	100	150
18	90	250
19	200	100
3	100	300
4	100	300
5	120	250
2	90	200
6	172	150
7	170	200
10	120	150
11	100	150
12	130	200
14	180	200
15	120	150
16	100	150
20	105	150
21	95	150
22	120	150
23	95	150
24	110	200

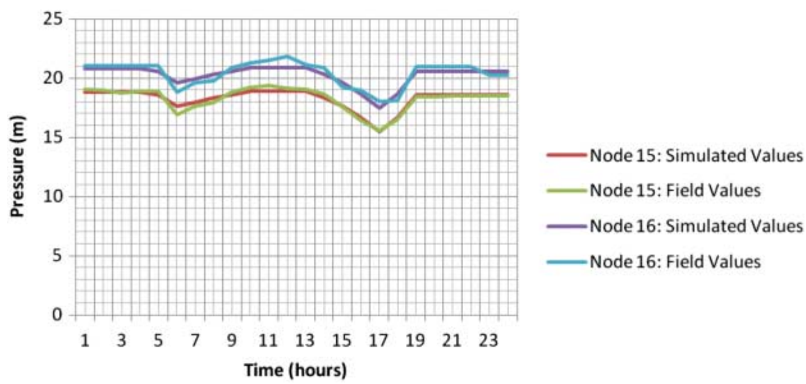

Figure 2. Comparison between field and model pressure outputs.

situations. After adjustment of demand loadings and model parameters in order to reflect the reality on the ground, a comparison between field pressure values and model pressure values for nodes 15 and 16 (Figure 1) is illustrated in Figure 2. It can be seen that there is good agreement between model values and field values.

In order to use the developed model to obtain the study objectives, two broad scenarios were considered as follows:

(1) Normal operating systems in which pressures are assumed sufficient to meet the demand throughout the network. This scenario involved monitoring the response and performance of the model during ideal flow conditions, i.e. periods and sections when pressures are sufficient, in order to find out system behaviour and pressures, heads and flows at various sections. This provided a control and benchmark to the subsequent scenarios.

(2) A constrained system that was created by imposing excessive demand loadings, insufficient supply and inadequate pipe sizes.

Having carried out DDA, nodes at which pressures were insufficient to fully supply their demands were identified. As already discussed, since demands are fixed under DDA while pressures vary, a nodal pressure value was considered insufficient if it was less than the pressure threshold, a situation that would result in less water supply than is required. The threshold value for each node can be approximated by the expected maximum outlet level in the locality served by that node represented by the height of the tallest building that can be agreeably supplied by the service provider without extra pumping (Tanyimboh 2000). A threshold value of 10 m was used in this study. When lower nodal pressures are obtained, only a fraction of the original demand is met. There is then the need to determine the available flows at the identified pressure-deficient nodes using the modifications summarised below (Ozger 2003, Mays 2004).

(1) New node elevation = original node elevation + threshold pressure head
(2) Set demand to zero
(3) Connect an artificial reservoir to the node by an infinitesimally short pipe that allows flow only from the node to the reservoir
(4) Artificial tank elevation = new node elevation

3. Results: analysis of alternative modelling approaches

3.1. *DDA: pressure response to demand*

In this section, we looked at the response of nodal pressures to changing demand. Plots of variations of demand with pressure at node 16 were done at midnight (Figure 3) when demand was lowest and at 16:00 hours (Figure 4) at peak demand. Demands were shown to be met at different pressures in a relationship of inverse proportionality, i.e. the higher the demands the lower the pressures at which the demands are fulfilled. When water abstracted from the system is a determinant of system performance, then when more water is withdrawn from the system, there is more head losses in the pipes and this lowers the pressure in the system.

It can be seen that for demand variation at midnight, pressures were much higher and varied gently because the demand was very low (Figure 3). However at 16:00 hours, pressures were much lower and had a steeper slope. At 29 l/s, while the pressure value at node 16 was still high at midnight, negative pressures were exhibited at 16:00 hours. This showed that the model at this point was malfunctioning, since not all nodes could supply water.

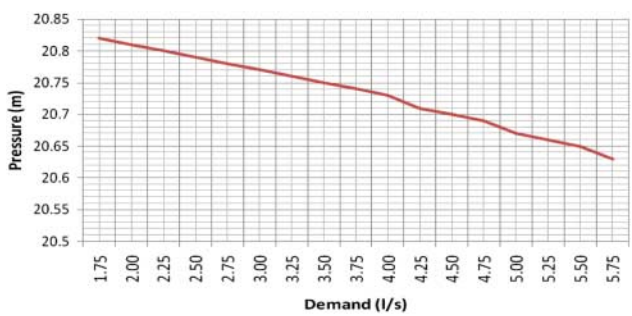

Figure 3. Plot of demand versus pressure at node 16 at 12:00 a.m.

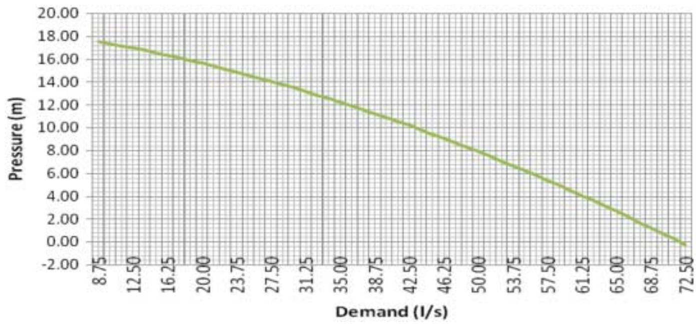

Figure 4. Plot of demand versus pressure at node 16 at 16:00 hours.

Table 3. Initial and subsequent pressure and demand values at 16:00 hours.

	Initial conditions		Subsequent conditions	
Node ID	Demand (l/s)	Pressure (m)	Demand (l/s)	Pressure (m)
2	8.5	10.12	8.5	8.5
14	8.75	19.25	17.5	13.51
20	8.25	21.66	17.5	18.51
31	8.5	13.63	17.5	10.09
36	8.25	28.06	8.25	26.25
12	8.75	36.26	17.5	30.54
13	8.75	33.93	17.5	31.61
3	8.75	31.06	17.5	24.19
4	8.25	21.31	17.5	16.82
7	8.00	23.30	8.0	18.99
8	8.00	28.44	8.0	24.64
9	8.25	25.8	17.5	23.13
10	8.75	27.27	17.5	13.07
11	8.75	22.27	17.5	8.09
15	8.25	27.01	37.5	8.25
16	8.75	18.01	37.5	−0.75
Reservoir 5	−584.97	0	−585.82	0
Tank 1	449.47	4.53	303.07	4.26

Table 3 shows the initial node demand and corresponding pressure values at 16:00 hours (peak period). The table also shows a different scenario when higher demand loadings were put on the network. Figure 5 shows initial pressures at 16:00 hours (peak demand hour) and Figure 6 shows pressures after higher demand loadings are made. It can be observed, as is expected of DDA, that lower pressures arise from higher demand loadings. It is particularly observed that negative pressures develop at junction 16, highlighted in Figure 6, which implies an inability to meet the demand at that node.

When high demand loadings are put on the network, at junction 16, the model returns a negative pressure value in order to satisfy a demand of 37.5 l/s. This is logically interpreted to mean that at this demand value, no supply is possible at this node. This is computational result, however in reality, some water will come out of this node at a discharge less than 37.5 l/s, in proportion to the prevailing pressure at the node and this further underlies the chief weakness of DDA for water distribution networks. Using HDA however, it can be worked out that the demand that can be met at node 16 is 9.73 l/s at midnight and 0.4 l/s at the peak hour.

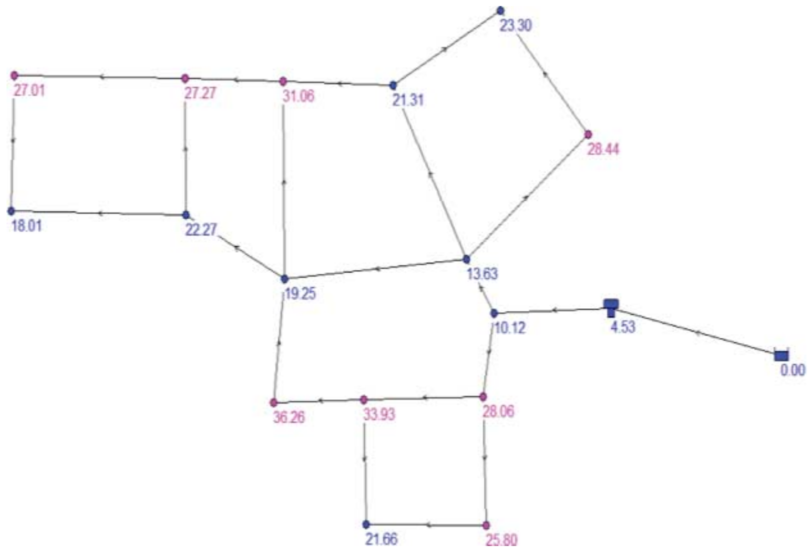

Figure 5. Initial pressures at 16:00 hours.

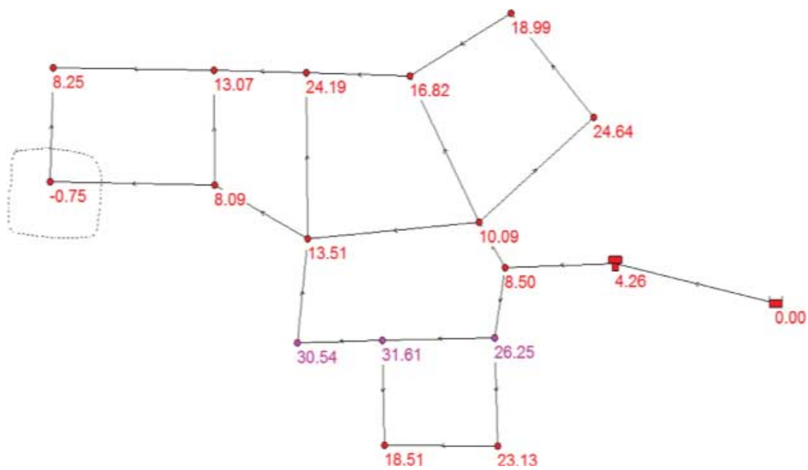

Figure 6. Pressures after higher demand loadings are made.

3.2. *HDA: demand response to pressure*

In this section, we looked at the response of water supplied to pressure when pressure is the driving factor/independent variable while demand is the dependent variable. Figure 7 shows variation of pressure with water supply at node 16 in the model at 16:00 hours. It can be seen that when pressure is the determining factor of system performance, then higher the pressure in the system the more the water supplied. It is observed that an increase in nodal elevation directly implies a reduction in nodal pressure of a similar magnitude. This can also be explained using Bernoulli's equation (Equation (2)) which can be interpreted as total head being a summation of elevation, pressure head and velocity head.

$$\frac{P}{\rho g} + \frac{v^2}{2g} + Z = \text{constant}, \tag{2}$$

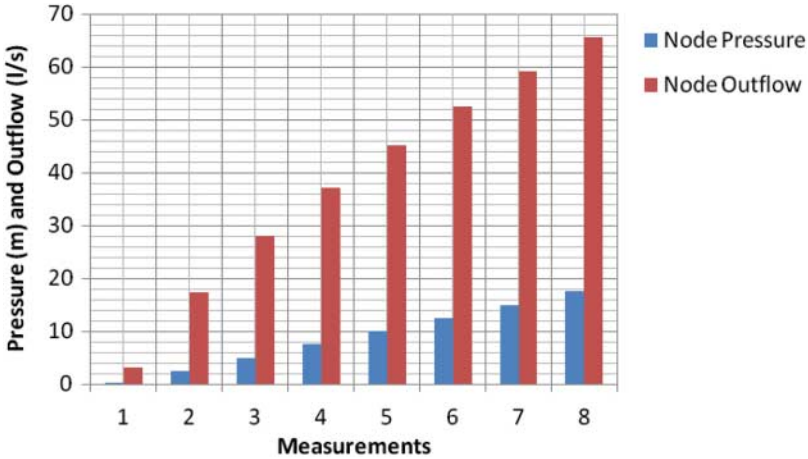

Figure 7. Response of available supply to changing pressures at 16:00 hours.

where $P/\rho g$ is the pressure head, ρ the water density, g the gravitational acceleration, $v^2/2g$ the velocity head, v the water velocity, and Z the static head or elevation. In order to maintain the same energy content of the water in accordance with the principle of conservation of energy, when elevation increases by a certain magnitude, pressure has to decrease by the same magnitude if water velocity is kept constant.

4. Comparison of demand-driven and head-driven results

In the demand-driven network model, before the network was subjected to additional loads, pressures were normal and consumers had their demands fully satisfied. Additional demand loads on the network led to drops in nodal pressures, especially at the peak hour and this demonstrates the fact that there is a demand limit on the network beyond which all demand will not be satisfied, contrary to common belief. In fact, DDA is valid when system pressures are high enough at all nodes so that demands are independent of pressure. If this is not the case, inaccurate outputs like demand fulfilment at negative pressures are obtained as is illustrated in Figure 6.

On the other hand, considering HDA, it is true that in reality, once the demand for a certain area has been fulfilled at a particular pressure, further increase in pressure may supply more water but the demand will not change. This implies that if a certain amount of water is required for a certain purpose, no more water is necessary even if it is available. However, below a certain threshold pressure, the amount of water that can be supplied begins to fall short of the required demand value. It is this relationship between pressure and demand that was studied in this paper. Whereas demands in current water distribution models are an input and nodal pressures are outputs (DDA), it is necessary to determine how much water different nodal pressures can supply, i.e. pressure becomes an input and supply becomes an output (HDA).

This study enabled the determination of the quantity of water which can be supplied at various pressures, an ability that is absent in water distribution models in current use. Figure 8 shows what proportion of the original demand can be met at various pressures in accordance with Figure 7. The higher the pressure at a node, the higher the outflow (supply) at the node.

Determination of pressures and flows in water distribution models involves solution of governing equations of energy and mass conservation and since demands are fixed model inputs under DDA, in the event of excessive demand loading, the feedback from the model is that input demands can be met at negative pressures. This is a purely mathematical result because practically

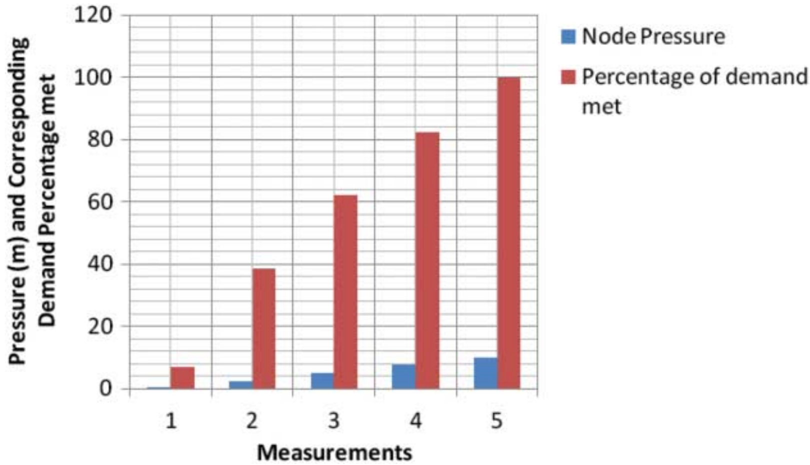

Figure 8. Proportion of original demand met at different pressures.

and physically we cannot obtain negative pressures within the domain of what is being studied. It is important to note that the negative pressure values returned in this model are different from the negative pressures that occur in pressurised flows resulting from cavitations.

The return of negative pressures by the model indicates that the model has sought to fulfil excessive nodal demands. In this case, as a management tool, it is necessary to reduce the demands till the model produces positive pressure outputs under DDA. Other structural measures that can be taken include laying a bigger pipe which has the effect of increasing the rate of pressurisation in the pipe due to reduction of head losses. If link 24 connecting the pressure-deficient node is increased by 50 mm from 150 mm, the node will start to have positive pressures (Figure 9). Alternatively, one can carry out an investigation into available pressures at the affected nodes and how much water can be supplied at those pressures, i.e. what proportion of the demand can be met under HDA. HDA recognises the dependency of demand on pressure and this is supported by the fact that practically, during pressure-deficient conditions, the system supplies what is available in accordance with the prevailing pressures.

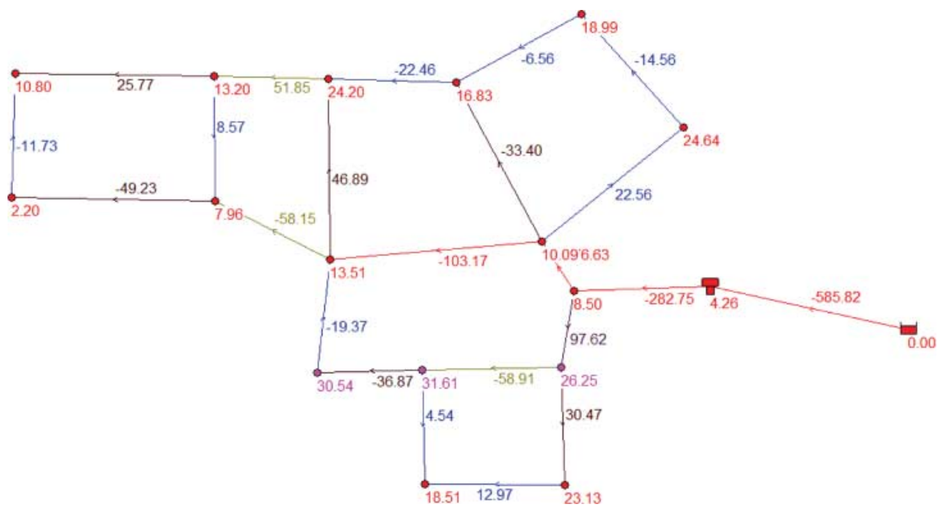

Figure 9. Node pressures and link flows.

When excessive demands are loaded onto the network as indicated in the second scenario, the model reveals that at junction 16, the pressure required to satisfy a demand of 37.5 l/s is negative, which is logically interpreted to mean that at this demand value no supply is possible at this node. This is computational result, however in reality some water will come out of this node at a discharge less than 37.5 l/s and in proportion to the prevailing pressure at the node. Our interest lies in determining how much water this node can supply and what other actions can be taken to alleviate the situation. If the threshold pressure head is predetermined at 10 m, then it can be worked out that the demand that can be met at node 16 is 9.73 l/s at midnight and 0.4 l/s at the peak hour. This provides much more accurate and meaningful information for real-time management.

The demand-driven approach works as long as the source pressure can supply minimum pressure head required at the demand nodes (Tanyimboh and Tabesh 1997, Tanyimboh 2000, Kalungi and Tanyimboh 2003, Cheung *et al*. 2005). Since DDA assumes that pressures are sufficient to fulfil any demands in both time and space, constitutive equations (conservation of mass and energy) are composed in order to solve both nodal heads and link flows. Of course, since demand at nodes is predetermined and therefore fixed, this parameter in the continuity equation is also fixed. Thus, nodal heads and link flows are calculated while the nodal demands are fixed. A major drawback of DDA is when it purports to meet the original (and fixed) water demand at very low or negative pressures. Clearly, when pressures are low, not all demands at nodes can be met. In such cases, if DDA is used, it may produce very unrealistic results.

It is important to realise that in financially viable economies characterised by satisfactory water supply system performance, all customers are expected to receive the amount of water they need and pressure is a key performance indicator whose non-fulfilment attracts penalties to the service provider. In low developed and water scarce regions, pressure is a luxury with flow becoming the more critical performance indicator. In these regions it is good enough that some water can run through faucets. In these areas, the notion of 'some for all rather than all for some' strongly holds. Focus then shifts from supplying water at pressures above a pre-established threshold figure to supplying some water to all, at whatever pressure. This means that while we cannot achieve the desired pressures, we need to maintain some flows to the populations and this makes the demand-driven approach along which traditional water distribution models are designed less helpful where supply pressures are deficient.

While HDA is superior to DDA in that a modeller is able to determine nodes with insufficient supply and the respective magnitudes of the shortfalls, the HDA method is troublesome to apply due to the difficulty in establishing the pressure–flow relationship at every node which would require extensive field data collection and calibration. Moreover, models are schematised and some features are lost; secondary networks have variable head losses and outlet elevations which would require all the more data to collect (Gupta and Bhave 1996, Ozger 2003, Mays 2004). Worse still, resulting equations have no solution methodologies, which explain why there are no commercial hydraulic solvers, using HDA (Kalungi and Tanyimboh 2003).

5. Conclusions

In DDA, the higher the outflow, the lower the pressure while in HDA, the higher the pressure, the higher the outflow. Current water supply modelling philosophy assumes a demand-driven approach which may not be applicable in networks experiencing low-pressure situations. The models are thrown into chaos during pressure shortfalls when the required demands are shown to be satisfied in all circumstances, including periods of inadequate pressure, which is unrealistic. In reality, when pressures are low, water is supplied in accordance with the available pressures, that is, the lower the pressure the less the water supplied.

The paper has demonstrated the superiority of HDA during low network pressure conditions. If water distribution models are to play an important role in the simulation of low-pressure networks and provide realistic results that can help in the solution of low network pressure problems, they should take cognisance of the pressure dependence of withdrawals in order to aid determination of how much water can be supplied during low-pressure periods.

References

Ang, W.K. and Jowitt, P.W., 2006. Solution for water distribution systems under pressure-deficient conditions. *Journal of Water Resources Planning and Management*, 132 (3), 175–182.

Chandapillai, J., 1991. Realistic simulation of water distribution systems. *Journal of Transportation Engineering*, 117 (2), 258–263.

Cheung, P.B., Van Zyl, J.E., and Reis, L.F.R., 2005. *Extension of EPANET for pressure driven demand modelling in water distribution system*. Exeter, Devon: CCWI2005 Water Management for the 21st Century.

Gupta, R. and Bhave, P.R., 1996. Comparison of methods for predicting deficient network performance. *Journal of Water Resources Planning and Management*, 123 (6), 369–370.

Hayuti, M.H., Burrows, R., and Naga, D., 2007. Modelling water distribution systems with deficient pressure. *Water Management*, 160 (4), 215–224.

Kalungi, P. and Tanyimboh, T.T., 2003. Redundancy model for water distribution systems. *Reliability Engineering and System Safety*, 82 (3), 275–286.

Martínez-Solano, J., *et al*., 2008. Hydraulic analysis of peak demand in looped water distribution networks. *Journal of Water Resources Planning and Management*, 134 (6), 504–510.

Mays, L.W., 2004. *Water supply systems security*. New York, NY: McGraw-Hill.

Nyende-Byakika, S., 2012. *Water supply: analysis and decision support tools for intermittent flow*. Germany: LAP Lambert Academic Publishing.

Nyende-Byakika, S., Ngirane-Katashaya, G., and Ndambuki, J.M., 2010. Behaviour of stretched water supply networks. *Nile Water Science and Engineering Journal*, 3 (1), 51–60.

Ozger, S., 2003. *A semi-pressure driven approach to reliability assessment of water distribution networks*. Thesis (PhD). Arizona State University.

Rossman, A.L., 2000. *EPANET users' manual*. Cincinnati, OH: United States Environmental Protection Agency, National Risk Management Laboratory.

Tanyimboh, T.T., 2000. A quantified assessment of the relationship between the reliability and entropy of water distribution systems. *Engineering Optimisation*, 22 (2), 179–199.

Tanyimboh, T.T. and Tabesh, M., 1997. Discussion of comparison of methods for predicting deficient network performance. *Journal of Water Resources Planning and Management*, 123 (6), 369–370.

Wu, Z.Y., *et al*., 2006. Efficient pressure dependent demand model for large water distribution system analysis. *In*: *8th Annual symposium on water distribution system analysis*, 27–30 August, Ohio.

Wu, Z.Y., *et al*., 2009. Extended global-gradient algorithm for pressure-dependent water distribution analysis. *Journal of Water Resources Planning and Management*, 134 (1), 13–22.