NONITERATIVE APPLICATION OF EPANET FOR PRESSURE DEPENDENT MODELLING OF WATER DISTRIBUTION SYSTEMS

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Noniterative Application of EPANET for Pressure Dependent Modelling of Water Distribution Systems

Abstract

EPANET 2 has been used previously to simulate pressure-deficient operating conditions in water distribution systems by: (a) executing the algorithm repetitively until convergence is achieved; (b) modifying the source code to cater for pressure-dependent outflows; or (c) incorporating artificial elements e.g. reservoirs in the data input file. This paper describes a modelling approach that enables operating conditions with insufficient pressure to be simulated in a single execution of EPANET 2 without modifying the source code. This is achieved by connecting a check valve, a flow control valve and an emitter to the demand nodes. Thus the modelling approach proposed enhances an earlier formulation by obviating the need for an artificial reservoir at the nodes with insufficient pressure. Consequently the connecting pipe for the artificial reservoir (for which additional data must be provided) is not required. Also, we removed a previous limitation in the modelling of pressure-dependent nodal flows to better reflect the performance of the nodes with insufficient flow and pressure. This yields improved estimates of the available nodal flow and is achieved by simulating pressure-deficient nodal flows with emitters. The emitter discharge equation enables the nodal head-flow relationship to be varied to reflect the characteristics of any network. The procedure lends itself to extended period simulation, especially when carried out with the EPANET toolkit. The merits of the methodology are illustrated on several networks from the literature one of which has 2465 pipes. The results suggest the procedure is robust, reliable and fast enough for regular use.

Keywords: water supply; pressure deficient water distribution system; dynamic hydraulic simulation algorithm; extended period simulation; flow control valve; pressure dependent nodal flow functions

1.0 INTRODUCTION

Traditional methods known as demand driven analysis of water distribution networks (Cross 1936; Martin and Peters 1963; Wood and Charles 1972; Isaacs and Mills 1980; Todini and Pilati 1987; Rossman 2000) assume that nodal flows are equal to the nodal demands. Any node pressures that are less than the required amount show the network's inability to supply the specified demands. Under such pressure-deficient conditions, the amount of water a network realistically can supply at different nodes is a key performance indicator. The actual amount of water that is available at a demand node under subnormal pressure conditions depends on the available pressure. Hence, a relationship exists between the flow and pressure at a demand node and is termed node head-flow relationship (NHFR). During simulation, NHFR at different nodes must be satisfied along with the equations for the conservation of mass and energy for the network as a whole. Accordingly, analysis based on NHFRs evaluates the performance of water distribution systems more realistically and has been used in tackling a variety of problems on water distribution systems such as: assessing reliability (Gupta and Bhave 1994; Kalungi and Tanyimboh 2003; Ozger and Mays 2003; Islam *et al.* 2014; Liserra *et al.* 2014), reliability-based design (Gupta and Bhave 1996a; Agrawal *et al.* 2007; Tanyimboh and Setiadi 2008), parameter calibration (Tabesh et al. 2011), vulnerability analysis (Li and Kao 2008), placement of isolation valves (Giustolisi and Savic 2010; Creaco *et al.* 2012), water quality (Gupta *et al.* 2012; Seyoum and Tanyimboh 2014), leakage management (Tabesh *et al.* 2009) and multi-objective evolutionary optimization (Siew and Tanyimboh 2012; Siew *et al.* 2013).

Pressure deficient network analysis can be carried out either by embedding a nodal head-flow relationship in the governing system of equations as described in Section 2. The user interface of the benchmark software for modelling water distribution systems EPANET 2 (Rossman 2000) currently does not include a ready-made procedure for incorporating nodal head-outflow relationships seamlessly. Consequently multiple runs of EPANET 2 are executed while adjusting the operational data for the water distribution system in successive runs of the demand driven analysis algorithm until an acceptable level of convergence is achieved. While this method may work well in practice for small water distribution systems, it is often time consuming and cumbersome especially for large systems (Jinesh Babu and Mohan 2012). More importantly, it is not practicable in situations requiring large numbers of hydraulic simulations, for example, dynamic simulations over an extended period of operation such as water quality modelling (e.g. Rossman 2000) or design optimization procedures based on evolutionary algorithms (e.g. Milan 2010).

One of the approaches that involve the iterative execution of EPANET 2 was proposed by Ang and Jowitt (2006) and Suribabu and Neelakantan (2011), with artificial reservoirs introduced at any demand nodes with insufficient pressure. Jinesh Babu and Mohan (2012) modified the algorithm that Ang and Jowitt (2006) proposed, in order to carry out pressure-deficient network modelling in a single execution of the unmodified EPANET 2 algorithm. They retained the above-mentioned artificial reservoirs and added a flow control valve, a check valve and a pipe of negligible resistance (for which additional data must be provided). However, the Jinesh Babu and Mohan (2012) algorithm does not model the transition between zero and full flow at a demand node satisfactorily and its convergence properties would appear to be poor (Gorev and Kodzhespirova 2013). Gorev and Kodzhespirova (2013) addressed the above-mentioned weaknesses by accounting properly for the transition between zero and full flow at a demand node, with suitable resistance properties assigned to the artificial pipes. This paper develops the approach further. We replace the artificial pipe and reservoir with an emitter and remove a restriction in the nodal head-outflow relationship to make it more generic. Our approach has the extra benefit that the additional data for the artificial pipe are consequently not required. The Gorev and

Kodzhespirova (2013) model assumed that an identical nodal head-outflow relationship applies to all networks. However, it is well-known that the nodal head-outflow relationship depends heavily on the characteristics of each network. Therefore the relationship cannot be represented accurately by a single curve for all networks. Rossman (2007) suggested that pressure-dependent analysis of water distribution systems could be accomplished using emitters. The coefficient of discharge for an emitter is a simple function of the nodal demand as shown here. Thus our approach accommodates diurnal variations in nodal demands. By taking advantage of the toolkit facility in EPANET 2, extended period simulation is considered here also. Results for some water distribution networks from the literature are included for demonstration purposes.

2.0 LITERATURE REVIEW

Bhave (1981) was the first to propose a NHFR as shown in Figure 1a, based on one hydraulic gradient level (HGL). In obtaining the performance of a network in which every outlet is considered, this HGL was taken as the outlet level and referred to as $H^{\min}$ (Figure 1a). Since velocity heads were neglected (as usual in demand-driven analysis also), HGL at a node more than $H^{\min}$ provided adequate flow (available flow q^{avl} = required flow q^{req}). HGL value less than $H^{\min}$ provided no flow ($q^{avl} = 0$); and HGL value equal to $H^{\min}$ provided partial flow ranging between no flow and adequate flow ($0 < q^{avl} < q^{req}$).

Gupta and Bhave (1996b) showed that for primary networks, in which demands at several outlets are lumped at a node, two HGL values are important in defining a NHFR. At some minimum HGL, $H^{\min}$, supply to the lowest outlet on secondary network would begin; and at some desirable HGL, H^{des}, all the outlets on secondary network would have adequate flows. However, Bhave's NHFR can be used also for obtaining performance of primary networks by suitably changing the value of $H^{\min}$ (Gupta and Bhave 1994; Ozger and Mays

2003; Ang and Jowitt 2006). Gupta and Bhave (1994) used desirable head (H^{des}) values at various nodes as $H^{\mathrm{min.}}$ and thus provided a lower bound on available partial flows. Ozger and Mays (2003) suggested considering H^{min} as maximum outlet level in the locality served by a node.

Ang and Jowitt (2006) mentioned that the relationship between the heads at the source nodes and the outflow at each demand node is a bi-product of the analysis and the elevation of demand node itself taken as H^{min}. The available flow at demand node j may be characterized as follows (Bhave 1981).

$$q_j^{\mathrm{avl}} = q_j^{\mathrm{req}} \text{ (adequate flow), if } H_j^{\mathrm{avl}} > H_j^{\mathrm{min}} \tag{1a}$$

$$0 \le q_j^{\mathrm{avl}} \le q_j^{\mathrm{req}} \text{ (no flow, partial flow or adequate flow), if } H_j^{\mathrm{avl}} = H_j^{\mathrm{min}} \tag{1b}$$

$$q_j^{\mathrm{avl}} = 0 \text{ (no flow), if } H_j^{\mathrm{avl}} < H_j^{\mathrm{min}} \tag{1c}$$

in which H_j^{avl} is the head at demand node j. Germanopoulos (1985) suggested zero flow for HGL values less than H^{min} and an exponential increase in the available flow for HGL values beyond H^{min} as shown in Figure 1b.

$$q_j^{\mathrm{avl}} = q_j^{\mathrm{req}} \left[1 - 10^{-c_j \left(\frac{H_j^{\mathrm{avl}} - H_j^{\mathrm{min}}}{H_j^{\mathrm{des}} - H_j^{\mathrm{min}}} \right)} \right] \text{ (partial flow), if } H_j^{\mathrm{avl}} > H_j^{\mathrm{min}} \tag{2a}$$

$$q_j^{\mathrm{avl}} = 0 \text{ (no flow), if } H_j^{\mathrm{avl}} \le H_j^{\mathrm{min}} \tag{2b}$$

where c_j is a coefficient. It can be observed from Figure 1(b) that available flows are less than required flows even at HGL value more than H^{des} and the curve given by Eq. (2a) is asymptotic to the q^{req} line. For higher values of c_j, the curve will approach the q^{req} line more rapidly. Wagner et al. (1988) and Chandapillai (1991) suggested a parabolic relationship for HGL values between H^{min} and H^{des} as shown in Figure 1(c).

$$q_j^{\mathrm{avl}} = q_j^{\mathrm{req}}, \text{ if } H_j^{\mathrm{avl}} \ge H_j^{\mathrm{des}} \tag{3a}$$

$$q_j^{\text{avl}} = q_j^{\text{req}} \left(\frac{H_j^{\text{avl}} - H_j^{\min}}{H_j^{\text{des}} - H_j^{\min}} \right)^{\frac{1}{n_j}}, \text{ if } H_j^{\min} < H_j^{\text{avl}} < H_j^{\text{des}} \tag{3b}$$

$$q_j^{\text{avl}} = 0, \text{ if } H_j^{\text{avl}} \leq H_j^{\min} \tag{3c}$$

where n_j is a coefficient; an approximate value close to 2.0 is frequently assumed. Fujiwara and Ganesharajah (1993) suggested a differentiable function as shown in Figure 1(d), for which Fujiwara and Li (1998) suggested an approximation. However, these relationships lack a good hydraulic justification.

$$q_j^{\text{avl}} = q_j^{\text{req}}, \text{if } H_j^{\text{avl}} \geq H_j^{\text{des}} \tag{4a}$$

$$q_j^{\text{avl}} = q_j^{\text{req}} \frac{\int_{H_j^{\min}}^{H_j^{\text{avl}}} \left(z - H_j^{\min}\right)\left(H_j^{\text{des}} - z\right) \mathrm{d}z}{\int_{H_j^{\min}}^{H_j^{\text{des}}} \left(z - H_j^{\min}\right)\left(H_j^{\text{des}} - z\right) \mathrm{d}z}, \text{ if } H_j^{\min} < H_j^{\text{avl}} < H_j^{\text{des}} \tag{4b}$$

or

$$q_j^{\text{avl}} = q_j^{\text{req}} \frac{\left(H_j^{\text{avl}} - H_j^{\min}\right)^2 \left(3H_j^{\text{des}} - 2H_j^{\text{avl}} - H_j^{\min}\right)}{\left(H_j^{\text{des}} - H_j^{\min}\right)^3}, \text{ if } H_j^{\min} < H_j^{\text{avl}} < H_j^{\text{des}} \tag{4c}$$

$$q_j^{\text{avl}} = 0, \text{ if } H_j^{\text{avl}} \leq H_j^{\min} \tag{4d}$$

Kalungi and Tanyimboh (2003) suggested a multi-step approach as shown in Figure 1(e). The number of steps and their sizes depends on the number of sets of critical nodes determined in the algorithm. The NHFR may be represented generically as

$$q_j^{\text{avl}} = q_j^{\text{req}}, \text{if } H_j^{\text{avl}} > H_j^{des} \tag{5a}$$

$$0 \leq q_j^{\text{avl}} \leq q_j^{\text{req}}, \text{ if } H_j^{\min} \leq H_j^{\text{avl}} \leq H_j^{\text{des}} \tag{5b}$$

$$q_j^{\text{avl}} = 0, \text{if } H_j^{\text{avl}} < H_j^{\min} \tag{5c}$$

Finally, Tanyimboh and Templeman (2010) suggested

$$q_j^{\text{avl}} = q_j^{\text{req}} \frac{\exp(\alpha_j + \beta_j * H_j^{\text{avl}})}{1 + \exp(\alpha_j + \beta_j * H_j^{\text{avl}})} \tag{6a}$$

where α_j and β_j are calibrated using filed data. Possible default values were given as

$$\alpha_j = \frac{-4.595H_j^{\text{req}} - 6.907H_j^{\min}}{H_j^{\text{des}} - H_j^{\min}} \tag{6b}$$

$$\beta_j = \frac{11.502}{H_j^{\text{des}} - H_j^{\min}} \tag{6c}$$

Kovalenko *et al.* (2014) compared Eq. 6a (Tanyimboh and Templeman 2010) and Eq. 3 (Wagner *et al.* 1988) and concluded that Eq. 6a has superior convergence properties in the computational solution of the system of equations. Ciapioni *et al.* (2015) used Monte Carlo simulation to study the nodal head-flow relationship in two urban areas with different topographical characteristics. The results showed that Eq. 6a performed better than the other nodal head-flow relationships considered in the study. Vairagade *et al.* (2015) also reached a similar conclusion based on a study of a skeletonized network.

It should be noted that in Eqs. (2a)-(2b), (3a)-(3c), or (4a)-(4d) the available flows can be obtained directly for any HGL value. Similar is the case with Eqs. (1a), (1c), (5a) and (5c). However, in Eqs. (1b) and (5b), the available flow cannot be obtained directly and is therefore calculated either through optimization (see e.g. Ackley *et al.* 2001) or through repeated analysis as described later. Eqs. (6a)-(6c) have the advantage of a smooth transition from zero to partial flow and also from partial to full demand satisfaction as shown in Figure 1(f).

The nodal head-flow relationship proposed by Wagner *et al.* (1988) is well established and was recommended by Gupta and Bhave (1996b); see also Tanyimboh *et al.* (1997). Kovalenko *et al.* (2014) investigated its convergence properties recently. The head-flow relationship under partial flow conditions for a secondary network may be written as

$$H_j^{avl} = H_j^{\min} + R_j(q_j^{avl})^{n_j} \tag{7}$$

where

$$R_j = \frac{H_j^{des} - H_j^{\min}}{\left(q_j^{req}\right)^{n_j}} \qquad (8)$$

Gupta and Bhave (1996b) showed that the values of R_j and n_j can be obtained by detailed flow analysis of secondary networks. The value of n_j is shown to lie between 1 and 2 depending on the location of consumers on the secondary network and the head loss in the pipes of secondary network. An average value of n_j of 1.5 was recommended, in the absence of a detailed analysis of the secondary network.

Regarding the computational solution of the resulting systems of equations, Bhave (1981) suggested an iterative methodology based on Eqs. (1a)-(1c). Gupta and Bhave (1996a) used the Hardy-Cross head correction method on the system of equations based on the nodal heads, where the available flows are corrected in each iteration using Eqs. (3a)-(3c). Tanyimboh and Templeman (2010) used the Newton-Raphson method to develop a globally convergent solution procedure. Giustolisi et al. (2008) and Wu et al. (2009) extended the global gradient algorithm that Todini and Pilati (1987) developed. Giustolisi and Laucelli (2011) proposed an enhanced global gradient algorithm.

However, developing the requisite software to make these methods work reliably for the simulation of real-life networks is extremely challenging. Consequently several alternative methods have been developed including those that are based on the most widely used demand-driven hydraulic solver EPANET 2. Siew and Tanyimboh (2012) modified the source code of EPANET to incorporate Eqs. (6a)-(6c) and termed this version EPANET-PDX. Jun and Gouping (2013) suggested iterative execution of EPANET. Ozger and Mays (2003), Ang and Jowitt (2006), and Suribabu and Neelakantan (2011) suggested iterative analysis in EPANET based on Eqs. (1a)-(1c). Their methodology puts artificial reservoirs at the pressure-deficient nodes. Jinesh Babu and Mohan (2012) used artificial reservoirs with flow control valves to ensure the flows to the reservoirs do not exceed the respective nodal demands. However, all these methods are based on Eqs. (1a)-(1c) with potentially poor convergence properties (see e.g. Gorev and Kodzhespirova 2013). Gorev and Kodzhespirova (2013) considered an artificial string that has a flow control valve, a pipe, a check valve and a reservoir. We improved the pressure-dependent analysis procedure by replacing both the artificial pipe and reservoir with an emitter and improved the accuracy of the hydraulic simulations by introducing a more generic nodal head-outflow relationship.

3.0 MODEL FOR PRESSURE-DEFICIENT NETWORKS

Emitters are used for modelling sprinklers, where outflow is uncontrolled and depends on available pressure. Given the relationship between the flow and pressure at an emitter node, Rossman (2007) suggested that pressure dependent analysis of water distribution systems could be accomplished using emitters. The generalized equation for the flow at an emitter is (Rossman 2000)

$$q_j^{\mathrm{avl}} = C_d (H_j^{\mathrm{avl}} - H_j^{\min})^{\gamma}; \quad H_j^{avl} \geq H_j^{\min} \tag{9}$$

in which C_d is the discharge coefficient and γ is an empirical exponent. However, Eq. (9) is identical to Eq. (3b), if C_d and γ are taken as

$$C_d = \frac{q_j^{\mathrm{req}}}{\left(H_j^{\mathrm{des}} - H_j^{\min}\right)^{\frac{1}{n_j}}} \tag{10}$$

and

$$\gamma = \frac{1}{n_j} \tag{11}$$

It is therefore proposed to consider an artificial string of a flow control valve (FCV), an emitter and a check valve (CV) as shown in Figure 2(a). The FCV will restrict the flow to the desired maximum, the emitter will simulate partial flow conditions, and the CV at the demand node will prevent reverse flows. Thus the method involves modifying the data for the demand nodes. This may be done using the graphical user interface in EPANET. Alternatively a computer program can be created in C to modify the data input file of EPANET using EPANET's toolkit functions.

The proposed algorithm using the graphical user interface involves the following steps; for simplicity, all nodes except for source nodes are considered as demand nodes, including those with zero demand.

1. Add two nodes near to each demand nodes. Add a CV pipe with negligible resistance (i.e. length of pipe can be given a very small value such as 0.001) between the original and the first added node. Add an FCV between first and second added nodes.
2. Make the base demand at all original demand nodes as zero.
3. Set the elevation at both the added nodes same as that of respective demand node.
4. Set the valve settings for each FCV to the demand at the respective demand node.
5. The second added node is provided with emitter coefficient C_d for respective demand node.
6. Set the emitter exponent γ to desired value.
7. Carry out the analysis by executing EPANET. Having introduced an emitter node as shown in Figure 2(a), the demand node may be visualized as a dead end. Therefore, for each demand node, the flow is available at the emitter node and the residual head at the demand node.

The above procedure yields the instantaneous response of the water distribution system. To address temporal variations, extended period simulation is carried out. This involves consideration of any changes in the network including demands and water levels in the tanks over time. To carry out extended period simulation in EPANET under pressure-deficient conditions, the settings of the emitters and flow control valves should track any changes in the demands. Extended period simulation is accomplished through the toolkit functions in EPANET. Thus the discharge coefficients of the emitters and the settings of the flow control valves are updated at the beginning of each hydraulic time step using time varying demands in Eqs. (9-10). A flow chart of the proposed algorithm, using EPANET's toolkit functions, for extended period simulation (EPS) is shown in Figure 2(b). This differs from Gorev and Kodzhespirova (2013) who executed EPANET repeatedly by re-starting the program for each successive hydraulic time steps. Example 3 in the next section is concerned with extended period simulation.

4.0 RESULTS AND DISCUSSION

The results provided here to demonstrate the proposed algorithm were obtained on a computer with specifications as follows: Intel Core 2 Duo, CPU T6600 @ 2.20GHz; RAM of 4.00 GB; and 32-bit Windows 7. The default convergence tolerance in EPANET 2 is 0.001, which is the ratio of the sum of the absolute values of the changes in the pipe flow rates to the sum of the pipe flow rates. We used the default values of other EPANET 2 parameters i.e. CHECKFREQ = 2, MAXCHECK = 10 and DAMPLIMIT = 0. This allows frequent checking of the status of flow control valves, pumps, check valves and pipes connected to tanks and tends to produce solutions in the least number of iterations (Rossman 2000). All CPU times (s) reported in the examples that follow have been rounded up.

4.1 Example 1: Small looped network

Figure 3(a) shows the layout of the network (Ozger and Mays 2003). The head at both supply nodes RES_1 and RES_2 is 60.96 m. The network has 13 demand nodes and 21 pipes. The required residual head at demand nodes is 15 m and n_j is 1.5 (Gupta and Bhave 1996b). Other relevant data are available in Ozger and Mays (2003). Typical results for the proposed approach, with pipe 3 closed, are summarized in Table 1 along with Jinesh Babu and Mohan (2012) and Gorev and Kodzhespirova (2013). The number of nodes with partial supply is seven in the proposed approach, with a total flow of 2709.36 m^3/hour. The corresponding values for Gorev and Kodzhespirova (2013) are seven and 2749.64 m^3/hour, respectively. For Jinesh Babu and Mohan (2012) the values are four and 2390.96 m^3/hour, respectively. These differences arise because Jinesh Babu and Mohan (2012) use only H_j^{min} the head above which flow at a node begins and, consequently, do not account properly for the required residual head. On the other hand, Gorev and Kodzhespirova (2013) impose a nodal flow exponent parameter n_j value of 2.0; modelling errors are thus introduced in cases in which the value of n_j is not 2.0 (as in the present example). The approach proposed here has the advantages that it addresses both issues and, furthermore, obviates the artificial pipe and reservoir. The number of iterations required by Jinesh Babu and Mohan (2012) was 14. Gorev and Kodzhespirova (2013) and the present approach achieved the solution in only 6 iterations; as might be expected, the simulations take less than a second in EPANET. We also obtained solutions for various other pipe closures as summarized in Table 2 which shows that the proposed model predicts higher nodal flows than Jinesh Babu and Mohan (2012) for the network as a whole.

4.2 Example 2: Large water distribution system

The EXNET network in Farmani et al. (2005b) [Figure 3(b)] resembles a large real life reinforcement problem in a water distribution system with a single loading. The network serves a population of approximately 400,000. It has 1891 nodes of which five are source nodes and 283 have no demand. Two of the source nodes have constant heads. There are 2465 pipes and five valves. The required residual head at all demand nodes is 20 m and n_j the nodal flow exponent parameter is 1.5. The existing network is pressure-deficient and this example aims to identify nodes with supply shortfalls using the proposed algorithm.

The pressure deficient analysis by the proposed algorithm yields an available demand fraction value of 0.926 for the network as a whole. The available demand fraction is the ratio of the flow that is available to the flow that is required and is also known as the demand satisfaction ratio (Ackley *et al.* 2001). Only 511 demand nodes are affected because of low pressure as compared to the 819 demand nodes identified by demand driven analysis. The performance of the network can be predicted realistically with respect to the failure of any components e.g. pipes and valves, or during excessive withdrawal due to fire at any node. Such results are not discussed herein for brevity. The simulation of EXNET using the proposed method required 7 iterations of the global gradient algorithm in EPANET, with a CPU time of less than a second. To check the accuracy of the simulation results, demand-driven analysis was carried out by changing the demands at pressure deficient nodes to the respective outflows obtained by pressure dependent modelling. It was observed that the pressure head values at all the nodes were the same in both cases. This confirms the accuracy and hydraulic feasibility of the results (Ackley *et al.* 2001).

4.3 Example 3: Extended period simulation

The network shown in Figure 3(c) has one source, two tanks, eight demand nodes and 15 pipes (Gupta and Bhave 1996a). Tanks 1 and 2 have initial total heads of 101 m and 100 m, and constant cross sectional area of 1500 and 1000 m^2 respectively. Tank 1 is filled from an external source from 00:00 to 04:00 and 12:00 to 16:00 hours at a constant rate of 23.5 m^3/minute. Tank 2 is a balancing tank and both tanks are floating on the system. Source 3 is a sump node with a constant water level of 70 m. The head-discharge relationship of the pump is $h_p = 40 + 0.01Q - 0.025Q^2$ where Q is the supplied flow in m^3/minute and h_p is the supplied head in metres. The pump operates from 04:00 to 12:00 and 16:00 to 24:00 hours. The required residual head for all demand nodes is 10 m. The nodal flow exponent parameter

n_j is 1.5. Additional details are available in Bhave and Gupta (2006).

The approach used in Bhave and Gupta (2006) accounts for the continuous variation in the demands and the total consumer demand for each hydraulic time step (Bhave 1988). However, in EPANET, demands are considered constant in each time step. Therefore, we used a small hydraulic time step of 1 second to make the results from the two algorithms comparable. Nodal demands were changed at the beginning of each time step using EPANET's toolkit functions. A 24-hour extended period simulation was carried out. Table 3 shows the results, which are essentially the same as Bhave and Gupta (2006). The CPU time required for the 24-hour extended period simulation with a *hydraulic time step of 1 second* is 18 seconds. The same 24-hour simulation was also carried out with hydraulic time steps of 2, 10, 30 and 60 seconds and the CPU times reduced to 9, 2, 1 and 1 second, respectively. The results for longer hydraulic time steps were, however, slightly inaccurate compared to Bhave and Gupta (2006) (as the continuous variations in the demands are treated differently as explained above). The corresponding EPANET 2 values for *demand driven analysis* are 4, 3, 2, 1 and 1 seconds respectively, for hydraulic time steps of 1, 2, 10, 30 and 60 seconds. The actual hydraulic time steps that would be used in practice would be much greater. These results are indicative of the computational efficiency of the proposed algorithm.

This network has spare capacity during periods of low demand; it can supply more water during periods of low demand at the nodes that have insufficient pressure during the peaks in demand. Extended period simulation models that consider local storage facilities at the demand nodes address any shortfall in supply that is carried forward due to local storage or unsatisfied demands (Agrawal et al. 2007; Giustolisi et al. 2014). In other words, the shortfall in supply in any time step is added to the normal demand in next time step to explore the possibility of extra withdrawal. Thus any shortfall in supply accumulates till it is met within a given day, subject to the overall capacity of the water distribution system. The procedure proposed here for pressure dependent modelling in EPANET was used to simulate the above situation. The results for two typical nodes 5 and 9 are shown in Figures 4(a) and (b), respectively.

It can be observed in Figure 4(a) that nodal demands are completely satisfied at node 5 up to around 07:00 hours. The shortfall in flow and pressure occurs from around 07:00 hours to 17:30 hours and the instantaneous demand keeps increasing in this period. The sudden rise and drop of available flow is due to pumps starting or stopping. The supply deficit starts reducing after about 17:30 hours and continues till 24:00 hours, when the accumulated supply deficit is completely met. At node 9 on the other hand, [Figure 4(b)] the supply deficit is not recovered in full by the end of the simulation period of 24:00 hours. The CPU time required for the 24 hour simulation with a hydraulic time step of 1 second is 18 seconds.

5.0 SUMMARY AND CONCLUSIONS

The model proposed here for pressure-deficient modelling of water distribution systems by executing EPANET 2 only *once* considers the head below which no flow is available and the head above which full flow is available at a demand node. Partial flows are estimated using a pressure dependent nodal head-flow function. The algorithm developed is demonstrated on the EXNET network that serves a population of about 400,000. Also, simulation of the dynamic behaviour of a water distribution network under pressure deficient conditions has been demonstrated. The results suggest the procedure is fast enough for regular use. Compared to Gorev and Kodzhespirova (2013) the proposed approach has the advantages that an artificial pipe and reservoir are not required and the modelling errors introduced by imposing a single nodal head-flow relationship that is applied in every situation are avoided. This leads to estimates of the nodal flows under pressure-deficient conditions that are more accurate. The EPANET 2 hydraulic simulator has excellent computational performance. Therefore, given that EPANET 2 can simulate large networks with thousands of elements i.e. links, nodes, etc. the main limitation of the procedure proposed here is the need to modify the original data input file. The proposed approach is even more practical when carried out with the EPANET programmers' toolkit. Future work may involve integrating a routine (i.e. a procedure in C) in the EPANET source code that would read the original network data input file of EPANET and create a new data file with the required changes, as an extra option for the user.

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FIGURE CAPTIONS

Figure 1. Nodal head-flow functions

Figure 2a The artificial links and nodes required at each demand node

Figure 2b Flow chart for the pressure dependent analysis algorithm using the EPANET toolkit functions

Figure 3. Topologies of the water distribution networks investigated

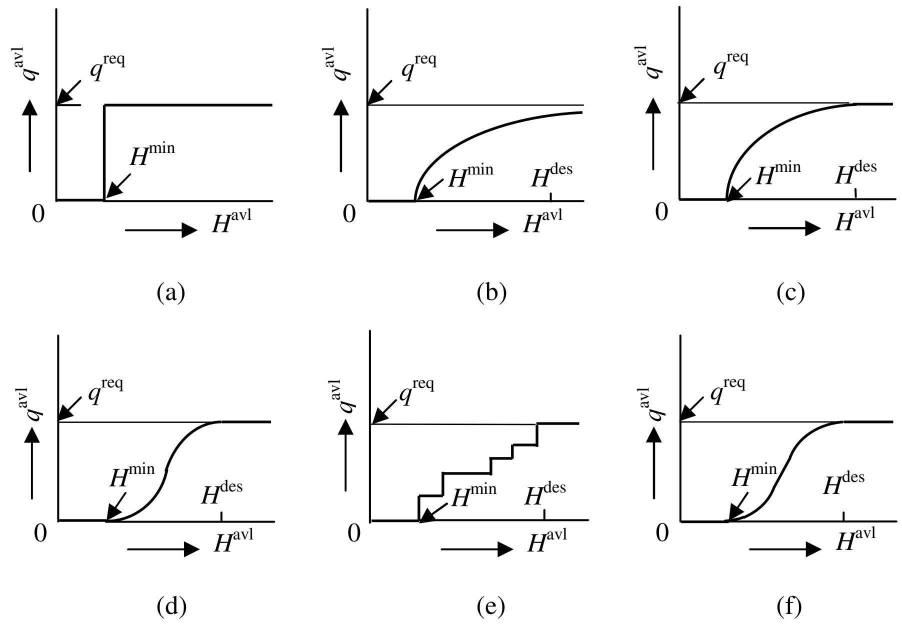
Figure 4. Flows at selected demand nodes of Example 3

TABLE CAPTIONS

Table 1. Results for the closure of pipe 3 in Example 1

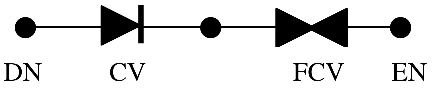
Table 2. Comparison of available demand fractions for single pipe closures in Example 1

Table 3. 24-hour simulation results in Example 3



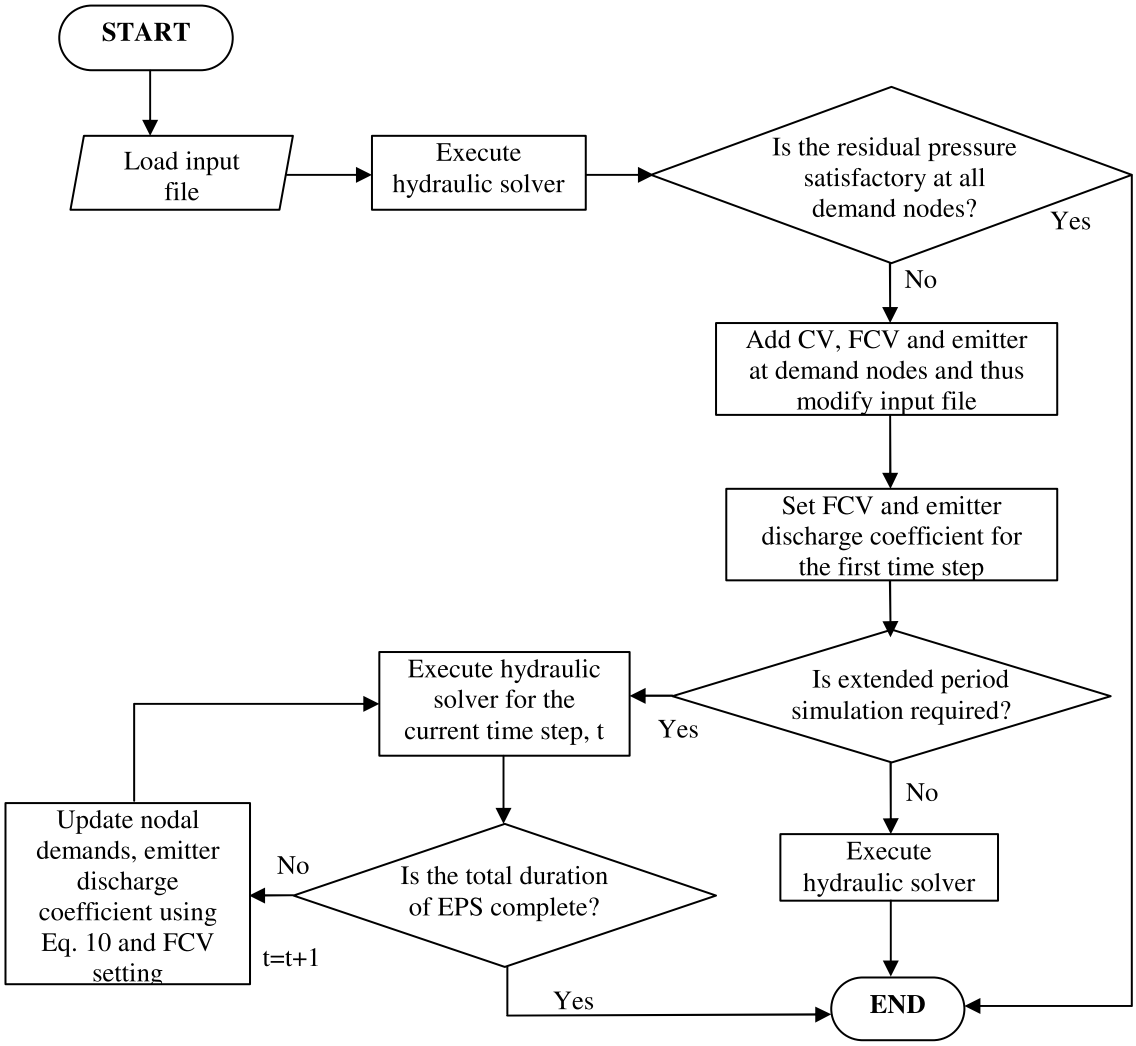
(a) Bhave (1981) (b) Germanopoulos (1985) (c) Wagner et al. (1988) (d) Fujiwara & Ganesharajah (1993) (e) Kalungi and Tanyimboh (2003) (f) Tanyimboh and Templeman (2010)

Figure 1. Nodal head-flow functions



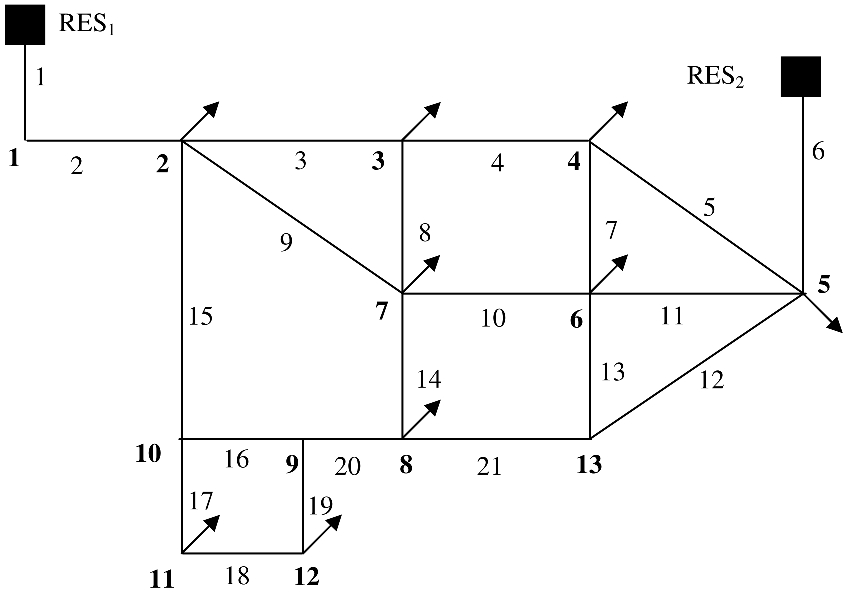
DN- Demand Node; CV-Check Valve; FCV-Flow Control Valve; EN-Emitter

(a) The artificial links and nodes required at each demand node

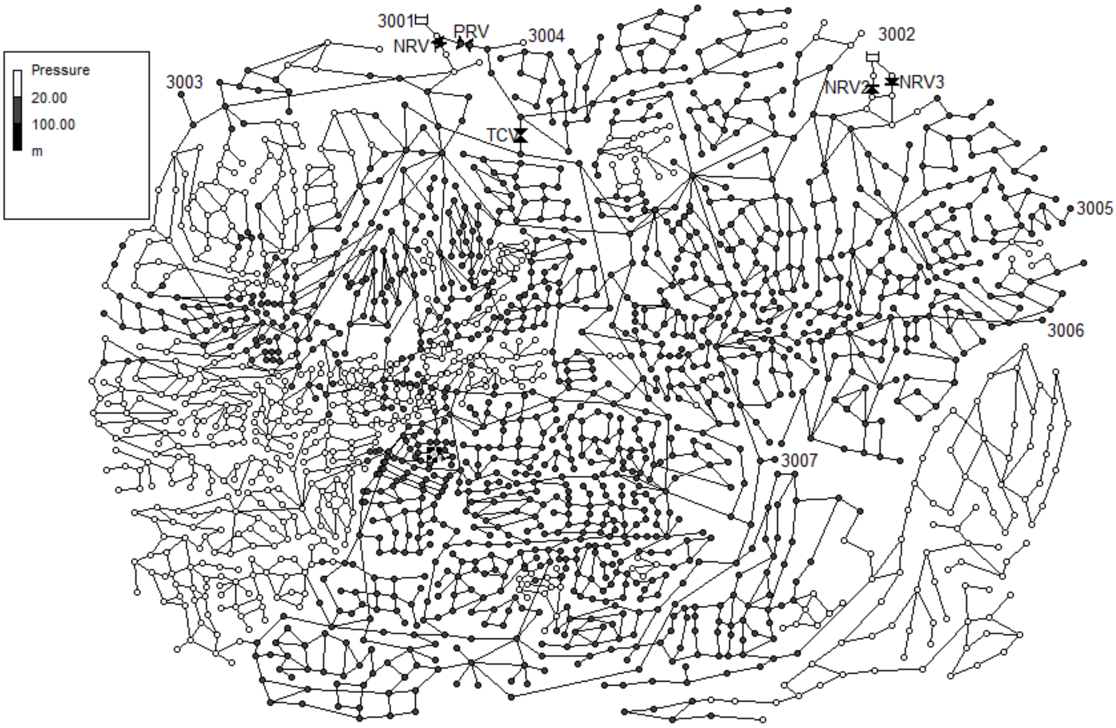


(b) Flow chart for the pressure dependent analysis algorithm using the EPANET toolkit functions

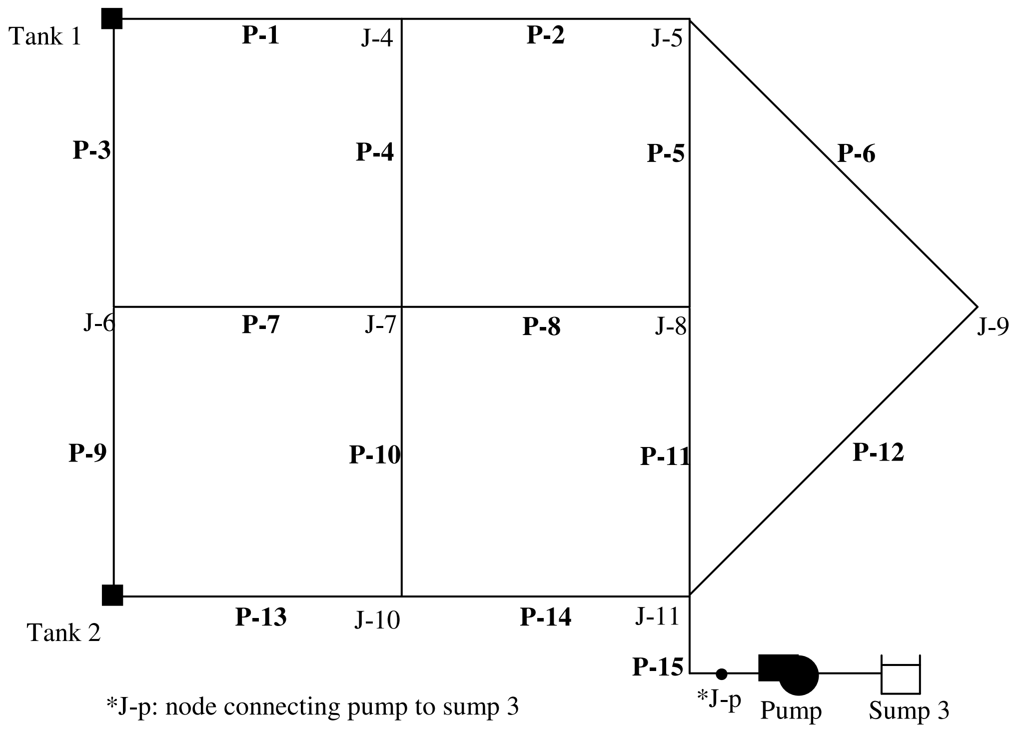
Figure 2.



(a) Network considered in Example 1

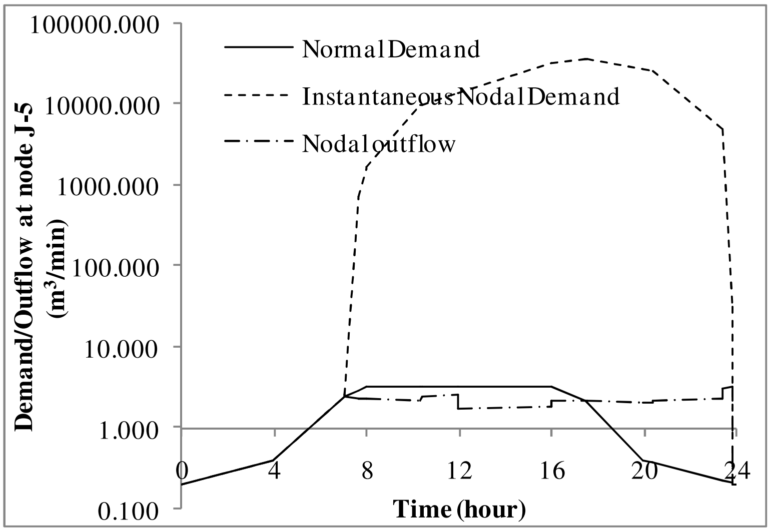


(b) Network considered in Example 2. The dark shaded nodes have sufficient pressure.

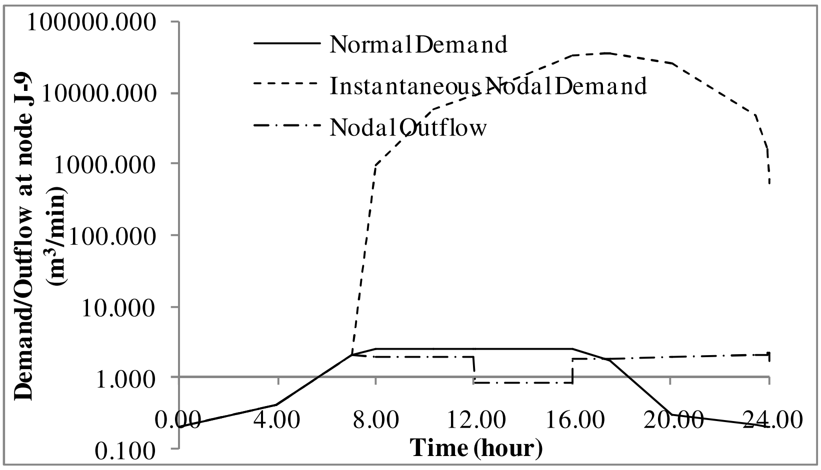


(c) Network considered in Example 3

Figure 3. Topologies of the water distribution networks investigated



(a) Flows at node J-5



(b) Flows at node J-9

Figure 4. Flows at selected demand nodes of Example 3

Table 1. Results for the closure of pipe 3 in Example 1

Node No.	Elevation	Demand	Jinesh Babu and Mohan (2012)		Gorev and Kodzhespirova (2013)		Proposed Approach	
			Available flow	Available HGL	Available flow	Available HGL	Available flow	Available HGL
	(m)	(m^3/h)	(m^3/h)	(m)	(m^3/h)	(m)	(m^3/h)	(m)
RES_1	60.96	0.00	-1168.55	60.960	-1315.32	60.960	-1298.65	60.960
RES_2	60.96	0.00	-1222.41	60.960	-1434.33	60.960	-1410.70	60.960
1	27.43	0.00	0.00	60.590	0.00	60.500	0.00	60.510
2	33.53	212.40	212.40	60.438	212.40	60.310	212.40	60.325
3	28.96	212.40	212.40	46.869	**193.06**	**41.353**	**193.27**	**41.980**
4	32.00	640.80	**165.11**	**47.000**	**506.92**	**41.387**	**489.54**	**42.016**
5	30.48	212.40	212.40	50.446	212.40	46.823	212.40	47.252
6	31.39	684.00	**499.05**	**46.390**	**558.58**	**41.393**	**543.71**	**42.020**
7	29.56	640.80	640.80	46.565	**588.00**	**42.190**	**587.73**	**42.736**
8	31.39	327.60	**274.56**	**46.390**	**277.52**	**42.154**	**271.54**	**42.710**
9	32.61	0.00	0.00	53.551	0.00	51.357	0.00	51.597
10	34.14	0.00	0.00	55.013	0.00	53.288	0.00	53.473
11	35.05	108.00	108.00	51.527	**103.87**	**48.925**	**103.80**	**49.183**
12	36.58	108.00	**66.24**	**51.580**	**96.89**	**48.654**	**94.97**	**48.950**
13	33.53	0.00	0.00	48.360	0.00	44.252	0.00	44.761
Total supply (m^3/h)		3146.4	2390.96		2749.64		2709.36	

Nodes with insufficient flow and pressure are shown in bold.

Table 2. Comparison of available demand fractions for single pipe closures in Example 1

	Jinesh Babu and Mohan (2012)		Proposed Approach		
Closed Pipe	Total available supply (m^3/h)	Network ADF	Total available flow (m^3/h)	Network ADF	Number of deficient nodes
NIL	3102.64	0.9861	3134.61	0.9963	2
1	1233.91	0.3922	1596.91	0.5075	9
2	1233.91	0.3922	1596.91	0.5075	9
3	2390.96	0.7599	2709.35	0.8611	7
4	2826.40	0.8983	2986.48	0.9492	5
5	3102.57	0.9861	3134.62	0.9963	2
6	2744.89	0.8724	2969.28	0.9437	5
7	3097.30	0.9844	3132.42	0.9956	2
8	3095.90	0.9839	3131.84	0.9954	2
9	2846.96	0.9048	2982.83	0.9480	5
10	3103.73	0.9864	3135.03	0.9964	2
11	3063.53	0.9737	3116.95	0.9906	3
12	3039.21	0.9659	3109.93	0.9884	3
13	3103.00	0.9862	3134.81	0.9963	2
14	3092.60	0.9829	3130.10	0.9948	2
15	2930.40	0.9314	3000.41	0.9536	3
16	3038.40	0.9657	3114.13	0.9897	2
17	3027.60	0.9622	3072.43	0.9765	2
18	3108.48	0.9879	3134.37	0.9962	1
19	3046.65	0.9683	3085.41	0.9806	2
20	3145.16	0.9996	3146.08	0.9999	1
21	3015.88	0.9585	3091.59	0.9826	3

Table 3. 24-hour simulation results in Example 3

Node Number	Time 0h	4h	8h	12h	16h	20h	24h
				Nodal flows (m^3/min)			
Sump-3	0.000	-4.727	-7.904	0.000	-9.103	-6.554	0.000
Tank-1	-1.909	-1.462	-10.196	-10.848	-10.147	-9.966	-2.066
Tank-2	0.009	2.289	0.347	-3.586	0.111	0.504	0.166
*J-p	0.000	0.000	0.000	0.000	0.000	0.000	0.000
J-4	0.200	0.500	3.000	2.500	2.800	3.500	0.200
J-5	0.200	0.400	**2.348**	**1.856**	**2.555**	0.400	0.200
J-6	0.300	0.800	3.100	1.500	1.700	**3.554**	0.300
J-7	0.200	0.300	2.500	**3.347**	3.800	3.500	0.200
J-8	0.300	0.500	**2.734**	**1.003**	**2.141**	**3.362**	0.300
J-9	0.200	0.400	**1.972**	**0.904**	**1.942**	0.300	0.200
J-10	0.200	0.400	1.000	**1.713**	2.000	0.800	0.200
J-11	0.300	0.600	1.100	**1.612**	2.200	0.600	0.300
				Nodal heads (m)			
Sump-3	70.000	70.000	70.000	70.000	70.000	70.000	70.000
Tank-1	101.000	104.269	103.253	101.702	103.653	102.005	101.213
Tank-2	100.000	100.041	100.363	100.326	99.496	99.561	99.867
*J-p	99.859	109.484	108.515	85.114	108.017	108.985	99.806
J-4	100.851	104.236	99.686	97.389	99.934	98.792	101.042
J-5	100.381	104.237	**89.785**	**87.919**	**90.635**	96.657	100.502
J-6	100.527	102.734	95.393	95.557	97.485	**92.375**	100.654
J-7	100.570	104.213	97.189	**91.266**	96.398	94.770	100.705
J-8	99.912	104.684	**88.899**	**85.157**	**90.599**	**90.415**	99.922
J-9	99.724	106.435	**87.003**	**82.174**	**86.848**	104.759	99.689
J-10	99.923	105.427	102.121	**87.170**	100.004	102.426	99.852
J-11	99.859	108.476	105.904	**85.114**	104.626	107.140	99.806

*J-p is a connecting node. Nodes with insufficient flow and pressure are shown in bold.