Traditional water distribution models solve the network problem by considering instantaneous demands lumped as nodal outflows. Such demand-driven analysis assumes that demands are independent of pressures and can be met under all operating conditions. Under transient conditions, however, the resulting positive- or negative-pressure surges can drastically alter the local pressures and affect the demand magnitude that can be extracted. A pressure-sensitive demand representation is needed to assess the effect of pressure changes and produce more accurate transient results. A comparative study of demand formulations for surge analysis showed that the pressure-insensitive demand assumption is intrinsically inaccurate and tends to overdesign surge protection devices. This overdesign results in unnecessary additional costs but does not necessarily ensure greater safety. The authors conclude that a pressure-sensitive demand formulation should be used for surge analysis to adequately evaluate both system performance and the ultimate cost of system protection.

Effect of pressure-sensitive demand on surge analysis

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Meeting the goals of the Safe Drinking Water Act demands a multi-barrier approach that ensures adequate protection of source water from contamination, effective treatment of raw water, and safe distribution of this treated water to consumers' taps. The latter objective requires, among other precautionary measures, protecting the distribution system from intrusion of contaminants as a result of objectionable pressure transients. Given that all pipeline systems leak and hydraulic transients occur continuously in most distribution systems, low-pressure transients can represent a significant risk of drawing untreated and possibly hazardous water into a pipeline system (Karney, 2003).

BACKGROUND

Transient pressures and their effects. Intrusion refers to the flow of nonpotable water into drinking water mains through leaks and other openings resulting from low transient or negative pressures (Boulos et al, 2006; NRC, 2006). Depending on the size of the leaks, the volume of intrusion can range from a few gallons to hundreds of gallons (LeChevallier et al, 2002; Funk et al, 1999; LeChevallier, 1999). Transient regimes are inevitable. Any disturbance in the water caused during a change in hydraulic state (typically from one steady or equilibrium condition to another) can initiate a sequence of transient pressures in the water distribution system. At some point in time, all systems will be started up, switched off, or undergo rapid flow changes and will likely experience the effects of human errors, equipment breakdowns, earthquakes, or other risky disturbances. Transient pressure can cause breaches in distribution system integrity that have significant implications for water quality and public health. Transients can generate high intensities of fluid shear and may cause resuspension of settled particles as well as biofilm detachment. Moreover, a low-pressure transient (such as one arising from a power failure or a broken water main) has the potential to cause the intrusion of contaminated groundwater into a pipe at a leaky joint or break. This risk is especially significant for systems with pipes below the water table. Dissolved air (gas) can also be released from the water whenever the local pressure drops considerably, and this may promote the corrosion of steel and iron sections with subsequent rust formation and pipe damage.

Transient pressure can cause breaches in distribution system integrity that have significant implications for water quality and public health.

If not properly designed and maintained, even some common strategies for protection from transients (such as relief valves or air chambers) may permit pathogens or other contaminants to find a "back door" route into the potable water distribution system. In the event of a significant intrusion of pathogens (e.g., as a result of a broken water main), the level of chlorine residual normally sustained in drinking water distribution systems may be insufficient to disinfect the contaminated water, which can lead to damaging health effects. A recent case study in Kenya showed that in the event of a 0.1% raw sewage contamination, the available residual chlorine within the distribution network would not be sufficient to safeguard the water (Ndambuki, 2006). Although not all intrusions are caused by pressure transients, excellent reviews of the effects of pressure transients on distribution system water quality degradation are available in the literature (Boulos et al, 2006; NRC, 2006; Lansey & Boulos, 2005; Wood et al, 2005a, 2005b; Gullick et al, 2004; McInnis, 2004; Karim et al, 2003; LeChevallier et al, 2003; Kirmeyer et al, 2001; Funk et al, 1999; LeChevallier, 1999).

Transient analysis. Hydraulic transient analysis provides the most effective and viable means of identifying weak spots, predicting potentially negative effects of hydraulic transients under various worst-case scenarios, and evaluating how transients may be eliminated or controlled. Transient flow in pipes is described by nonlinear hyperbolic partial differential equations of continuity and momentum principles. A direct solution of these equations is not possible because of the presence of nonlinearity and the complexity associated with the pipe network configuration and associated boundary conditions. Various numerical methods have been developed for analyzing transient flow in pressurized conduits, including a linear analyzing solution scheme (an approximate analytical solution by linearizing the friction term in the governing transient flow equations), implicit method (finite difference procedure), finite element method, and graphical water-hammer approach (Wylie & Streeter, 1993; Chaudhry, 1987). However, application of these methods has been limited because of their insufficient accuracy and inherent difficulty in solving complex multiloop networks.

In general, the most popular and viable methods for solving hydraulic transient problems are the method of characteristics and the wave characteristic method (Boulos et al, 2006; Wood et al, 2005a, 2005b). The method of characteristics transforms the governing transient flow equations into ordinary equations. These ordinary differential equations are then integrated to obtain a finite difference representation of the pressure head and flow. The wave characteristic method solves the transient flow problem in an event-oriented system simulation environment. In this environment, the pressure wave propagation process is driven by the distribution system activities. Other researchers have made a detailed comparison of the accuracy and computational time requirements for the two methods (Wood et al, 2005a; 2005b). They found that both methods produced virtually identical results but that the wave characteristic method was more efficient in terms of both time and memory for analyzing large water distribution systems. Ghidaoui and colleagues provided a general history and introduction to water-hammer phenomena, a compendium of key developments and literature reference, and an updated view of the current state of the art, with respect to both theoretical advances of the last decade and modeling practices (Ghidaoui et al, 2005).

Strategies for controlling transients. Several techniques are available for suppressing and controlling hydraulic transients in water distribution systems (Boulos et al, 2006, 2005; Wood et al, 2005a). These strategies range from system modification and operational considerations to installation of surge-control devices. Operational considerations focus on the root causes of flow changes, such as adjusting valve or pump operations. They may include prolonging valve opening and closing times, using a two-stage valve closure and opening, or increasing pump inertia by adding a flywheel to prolong pump stoppage and startup times. System modifications can include reinforcing pipes (i.e., increasing the pressure rating), rerouting and changing the network topology, using larger-diameter pipes, or installing different pipe material (Jung & Karney, 2004). The final surge-protection strategy, most commonly considered in pipeline systems, involves adding surge-control devices. The general principles of these devices are to store water or otherwise delay the change of flow or to discharge water from the piping system so that rapid or extreme fluctuations in the flow regime are minimized. Devices such as pressure-relief valves, surge-anticipation valves, surge vessels, surge tanks, and pump bypass lines are commonly used to control maximum pressures. Minimum pressures can be controlled by adding surge vessels, surge tanks, air-release/vacuum valves, or pump-bypass lines. Overviews of the various common surge-protection devices and their functions can be found elsewhere (Boulos et al, 2006, 2005; Wood et al, 2005a; Thorley, 1991).

In general, a combination of these devices may prove to be the most desirable and most economical strategy. Other research provided a detailed transient flow chart to the selection of components for surge control and suppression in water distribution systems and concluded that a transient analysis should always be carried out to determine the effect of each proposed strategy on the resulting system performance (Boulos et al, 2005). Such an approach is a fundamental part of rational network design. A more recent study showed that only systematic and informed transient analysis can be expected to resolve complex transient characterizations and adequately protect a distribution system from the vagaries and challenges of rapid transients (Jung et al, 2007a).

WATER DEMAND MODELS

In conventional water distribution transient models, it is presumed that the nodal demand is independent of pressure (demand-driven analysis) and is always satisfied under all operating conditions (including zero or negative pressure). In the actual system, the demand would not be met (the demand becomes zero when the pressure drops to zero or less than zero). Various techniques for modeling nodal demand as a function of nodal pressures (generally termed head-driven analyses) have been proposed (Ang & Jowitt, 2006; Gupta & Bhave, 1996; Jowitt & Xu, 1993). Of the several methods reviewed by Gupta and Bhave (1996), the method using the parabolic head-discharge relationship (no flow at minimum head to required flow at desired head) yielded the best prediction of network performance. These researchers also detailed the basic differences between the various demand modeling approaches, including the parabolic head-discharge and the standard orifice-based methods (Gupta & Bhave, 1996). However, the applications of these methods were restricted to steady network analysis under normal conditions, and only a few methods have been implemented in transient analysis. McInnis and Karney (1995) introduced a distributed pipe flux demand model and compared it with both constant (pressure-insensitive) and orifice-based (pressure-sensitive) demand models. These three demand models were also compared with field test data using a network transient model. More recently, Karney and Filion (2003) assessed the primary energy-dissipation mechanisms, including orifice-type leaks, commonly found in pipeline systems under water-hammer conditions.

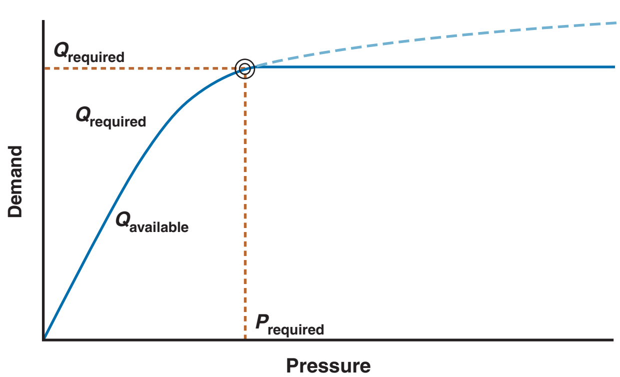
Pressure-sensitive demand can be simulated with emitters that discharge the flow through a nozzle or orifice to the atmosphere. The flow rate through the emitter varies as a function of the pressure available at the junction node and can be expressed as

$$Q = C_d AH^\gamma = C'_d Ap^\gamma = C_{\text{emit}} p^\gamma \tag{1}$$

in which Q is the flow rate from the emitter; H and p are the total head and pressure at that emitter, respectively; C_d and C'_d are discharge coefficients for the total head and pressure, respectively; C_{emit} is the emitter coefficient; and γ is a pressure exponent, usually taken as 0.5. The value of the emitter coefficient is calculated as the flow through the device at a 1-psi pressure drop with units of gpm/psi$^{0.5}$. Using Eq 1, the emitter is used to simulate the effect of the pressure-sensitive demand. A typical pressure-sensitive demand curve for an extended period simulation is shown in Figure 1. Starting with no flow at zero pressure, the nodal demand increases with increasing pressure and stabilizes to the required flow when the desirable pressure is reached, typically 20 psi.

The constant demand formulation could benefit the steady-state model (especially at large demand points) and could provide a conservative measure in transient pipeline systems. However, pressure transients can drastically alter the local pressures, which in turn can sig-

FIGURE 1 Design of pressure-sensitive demand in the extended period simulation



P—*pressure*, Q—*flow rate*

nificantly affect the magnitude of nodal demands that can be extracted. The fixed demand ignores the pressure-sensitive characteristic implicit in actual transient pipeline systems, which can lead to poor design, unreliable protection, and/or unsafe operation of these systems. Therefore a pressure-sensitive demand representation is needed to assess the effect of pressure changes

Surge modeling is important to safeguard against breaches in distribution system integrity.

on the supplied flows and produce more accurate transient modeling results. This article describes a comparative study of demand formulation for surge analysis. The study encompassed both pressure-sensitive and pressure-insensitive demands. Both demand analysis methods are demonstrated using two pipeline system, examples. The first example is for a small pipeline system, and the second is for an actual water distribution network system. Results showed that the assumption of pressure-insensitive demand exaggerates a surge wave in the distribution system, which could lead to overdesign of selected surge-protection devices and added costs. In light of these findings, the authors concluded that a pressure-sensitive demand formulation should always be used for surge analysis in order to adequately evaluate both system performance and the ultimate cost of system strengthening.

WATER DEMAND: ASSUMPTION AND APPROXIMATION

Water demand classifications. Water demand is the main driving force behind water distribution system dynamics. Municipal water demands are commonly classified according to the nature of the user. The general classifications are customer consumption, nonrevenue water, and fire flow demand. Customer consumption is computed on the basis of various customer types or categories, which include single and multifamily residential (domestic and irrigation), industrial (e.g., manufacturing plants), commercial (e.g., restaurants, shopping malls, and office buildings), government (e.g., city and federal buildings), recreational (e.g., parks and golf courses), institutional (e.g., schools and universities), and agricultural (e.g., livestock). Nonrevenue water represents the flow lost through leakage, pipe breaks, unmetered services, unauthorized use, hydrant flushing activities, as well as losses from accounting and meter errors. It is normally computed as the difference between the total volume of water produced (annual water production) and the total amount of water billed (annual metered consumption). In the network model, nonrevenue water is normally distributed uniformly across the nodes, unless measurements for specific zones are available. Nonrevenue water is system-specific and can be significant in systems operating under high pressures as leakage increases with pressure.

Fundamental questions related to network demand. Water demand levels vary considerably over time and among users. In general, water consumption is driven by population and economic activity as well as weather patterns. In order for a hydraulic simulation to properly reflect system performance, accurate demand estimates must be developed and incorporated into the model, but determining peak design demands is not as straightforward as it is sometimes assumed to be. Estimating network demands is related to three fundamental questions.

- How much water will be used?
- How will actual water demand be represented in a network model?
- How will water use change as a function of time?

The first question is linked to future demand projections, which are usually the focus of master planning or rehabilitation studies. Future demand projections ultimately are used for the design of new distribution facilities, such as booster stations, storage tanks, and pipes. They are also essential to determining the effects on the existing distribution system. Demand forecasting is neither simple nor certain, because the projections must take into account the average per capita level of demand, the estimated and projected changes in population, the variability of demand (particularly as it relates to peak use), and the physical attributes of the system (which will change with time). Natural caution usually leads to overestimation of demands, which then results in excessive additional costs (Babayan et al, 2005).

The second question is connected to demand allocation. The purpose of demand allocation is to load the hydraulic network model with demands at the nodes, given the available information. Water use occurring along the entire length of each pipe is spatially redistributed to the associated end nodes in the model. Demand allocation is often applied with the process of model skeletonization. Because demand allocation preserves the hydraulic performance and integrity of the larger original system, the resulting reduced network model produces the same steady-state results as the larger original model; however, this process must be more carefully assessed for surge analysis. Model skeletonization eventually changes the amount and location of demands, which in turn can affect the reflection and dissipation of transient pressure waves. The effect of water distribution model skeletonization for surge analysis has been described in other research (Jung et al, 2007b).

The final question is related to the temporal variation in demand. Temporal demand variations for municipal water systems generally follow quasi (stepwise) steady-state analysis for the simplified model and a transient analysis for a more accurate simulation. Demand patterns are used to represent the characteristics of diurnal curves for the various user types, typically set up in increments of 15, 30, or 60 min, depending on the data available. This formulation assumes that system pressures are adequate so that nodal demands are independent of nodal pressures. Under transient conditions, however, the nodal pressures may not be sufficient for supplying the imposed demands.

EFFECT OF PRESSURE-SENSITIVE DEMAND ON SURGE ANALYSIS

Models of real-world systems tend to be simplified representations of these systems. Which features of the actual system are incorporated into the model and which features are not depend in part on what the modeler considers important with respect to the issues under discussion, the problem at hand, or the questions being asked (Loucks, 1992). The time-varying demand model, despite its complexity, is generally simplified and represented as either a constant (pressure-insensitive) base demand with a diurnal curve or as a pressure-sensitive orifice. The diurnal curve is seldom used for surge models, however, and the rapid pressure fluctuation inherent in surge analysis may render the constant demand assumption less valid. A more realistic representation of demand fluctuation is required to provide more accurate transient modeling results, estimate worst-case scenarios, and more cost-effectively design adequate surge-control devices.

The constant demand can be replaced with the pressure-sensitive demand formulation using an "equivalent orifice" needed to pass the discrete constant demand at the steady system pressure. Eq 1 can be used to represent pressure-sensitive demand with the calculated emitter coefficient from the given initial constant demand Q_i, and initial pressure p_i, as

$$C_{\text{emit}} = Q_i / p^{\gamma}_i \tag{2}$$

To investigate the effects of a pressure-sensitive demand in transients, the orifice relations shown in Eqs 1 and 2 can be used to relate the difference in demands to the difference in pressures as

$$Q - Q_i = C_{\text{emit}}\, p^{\gamma} - C_{\text{emit}}\, p^{\gamma}_i = C_{\text{emit}}\, p^{\gamma}_i \left[\left(\frac{p}{p_i}\right)^{\gamma} - 1\right] \tag{3}$$

Because $C_{\text{emit}} p^{\gamma}{}_i = Q_i$, Eq 3 can be represented as

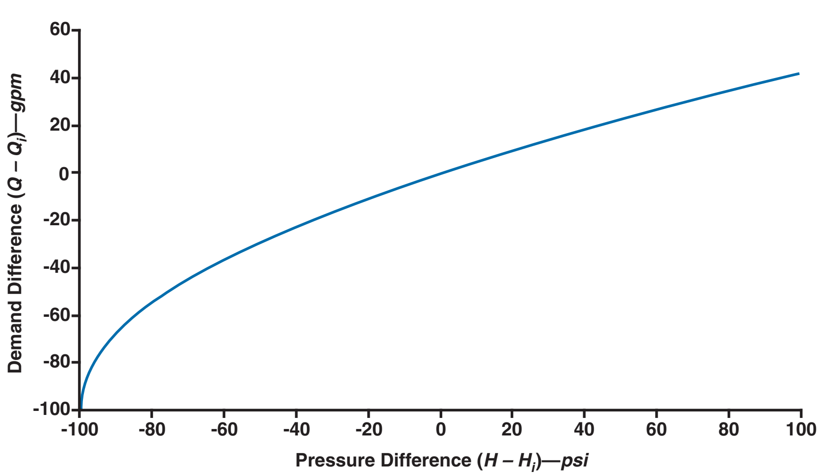
$$Q - Q_i = Q_i \left[\left(\frac{p}{p_i}\right)^{\gamma} - 1\right] \tag{4}$$

The difference between the demands calculated in Eq 4 is the type of error that arises in a hydraulic solution when demand is assumed to be independent of the junction pressure. More specifically, the assumption of constant demand causes an error whenever the junction pressure is different from its initial value. For example, if a constant demand of 100 gpm is assigned with an initial pressure of 100 psi, the resulting demand difference of the pressure-insensitive demand will vary as shown in Figure 2.

As Figure 2 illustrates, the demand difference is positively increased as the junction pressure is increased, which implies that the junction node produces a greater demand as it experiences a positive surge in the transient. Because the increased demand from a positive surge causes more energy to dissipate, the corresponding junction pressure of the pressure-sensitive demand is lower than that of the constant demand. Similarly, the junction node produces less demand as it experiences a negative surge in the transient; therefore, the corresponding junction pressure of the pressure-sensitive demand is higher during a negative surge than that of the constant demand. This characteristic of pressure-sensitive demand is important because models using the constant-demand assumption tend to overestimate or underestimate the transient pressure conditions and lead to the overdesign of surge-protection devices, resulting in unnecessary additional costs.

Another important motivation for conducting pressure-sensitive demand for surge analysis is water quality considerations. One of the challenging problems in the management of distribution system

FIGURE 2 Error of pressure-insensitive demand



H—*head,* Q—*flow rate*

water quality is that contaminants can intrude into pipes through leaks from reduced- or negative-pressure transients. As shown in Eq 1, a negative- or low-pressure transient (arising from a power failure or an intermittent or interrupted supply, for example) increases the risk of backflow and potential system contamination.

In summary, the assumption of constant demand in surge analysis ignores the pressure-sensitive characteristic inherent in actual pipeline systems. Demand and pressure at each junction node continuously vary, and the accurate representation of this fluctuation is essential in order to provide accurate transient modeling results for sufficient system protection. To more accurately evaluate both system performance and the ultimate cost of strengthening the system, the constant demand model can be replaced by a pressure-sensitive orifice model. The subsequent section explores the use of this model in two case studies and compares results with those for pressure-insensitive demand.

CASE STUDIES

In the following case studies, surge analysis results of the pressure-sensitive demand model were compared with those of pressure-insensitive (constant) demand formulations. In particular, these studies highlighted the limitations of pressure-insensitive demand for surge analysis. For all examples, the orifices were placed at the elevation of their respective nodes. All of the transient modeling results presented here can be obtained using the method of characteristics (Wylie & Streeter, 1993) or the wave characteristic method (Boulos et al, 2006; Wood et al, 2005a) and can be reproduced using available commercial or in-house water-hammer codes.

Case study 1: A small pipeline system. The first case study used the small water pipeline system shown in Figure 3. This system consisted of a 100-m (328.1-ft) head reservoir feeding a network of five pipe sections and five junctions. The diameter, length, Hazen-Williams roughness coefficient, and wave speed for each pipe were 1 m (3.3 ft), 1,000 m (3,280.8 ft), 100, and 1,000 m/s (3,280.8 fps), respectively. The elevation of each junction was assumed to be 0 m. Each junction node had an external demand of 0.2 m^3/s (7.06 cfs) so the total discharge from the upstream reservoir was 1 m^3/s (35.3 cfs). A rapid demand decrease over a 1-s time period at the ter-

FIGURE 3 A small pipeline system

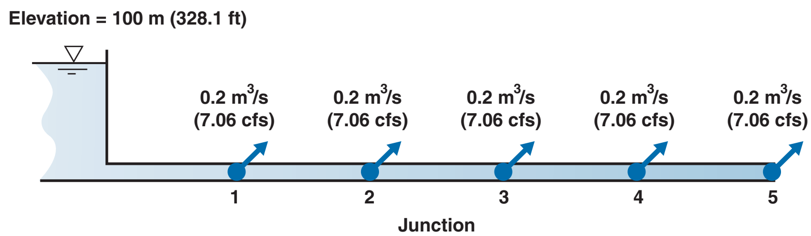


FIGURE 4 Pressure head profiles using pressure-insensitive demand

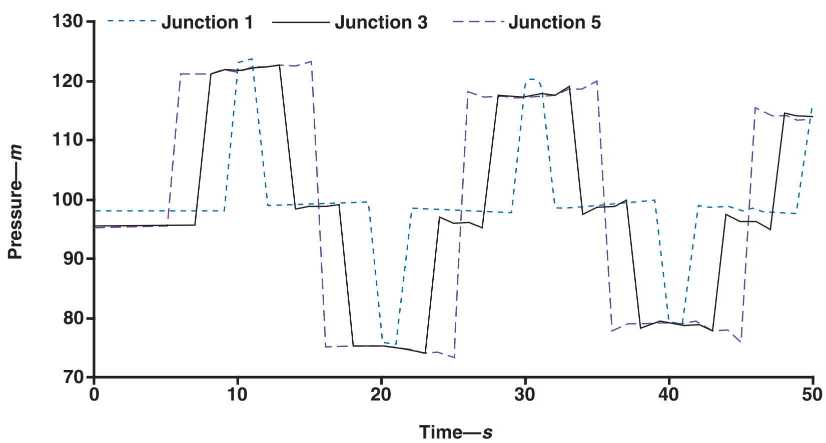
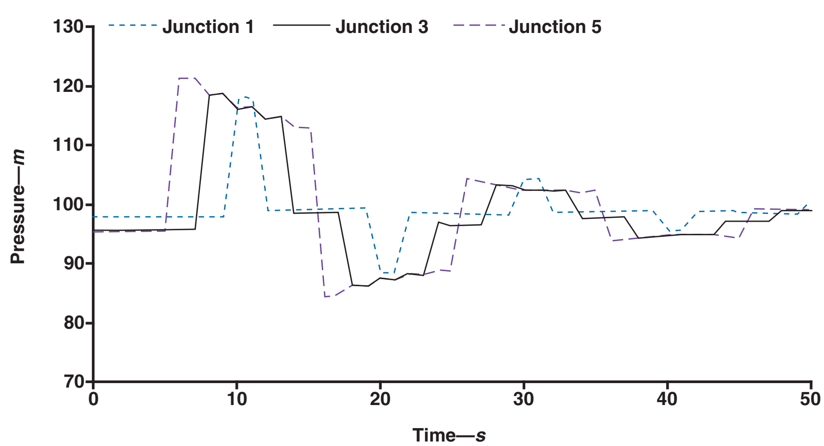


FIGURE 5 Pressure head profiles using pressure-sensitive demand



minal junction 5 was initiated at 5 s to introduce a transient condition.

Figures 4 and 5 show the transient head profiles at junctions 1, 3, and 5 using pressure-insensitive and pressure-sensitive demands, respectively. As shown in the figures, the initial steady pressures at junctions 1, 3, and 5 were identical because the emitter coefficient of Eq 1was calculated from the initial steady head and flow, and the head profiles of both Figures 4 and 5 were steady for the first 5 s. In addition, the initial positive surges in both figures for junction 5 were identical at 121.15 m (397.49 ft). After creating the initial surge, the surge wave of the pressure-insensitive demand was propagated without any disturbance except that the friction loss along the pipeline caused a slight dissipation (Figure 4). In contrast, the surge wave of the pressure-sensitive demand experienced dramatic pressure dissipation when it passed the junctions (Figure 5). The rationale for this difference is that the positive surge pressure caused a greater discharge (higher than the constant demand) through the demand junction, and then the demand increase caused a negative surge that was propagated in both upstream and downstream directions. Therefore, the negative surge created from the pressure-sensitive demand interacted with the initial positive surge, causing some pressure dissipation. When the initial positive surge reached the upstream reservoir, the behavior of pressure-sensitive demand was the opposite of the initial positive surge because the positive surge was converted into a negative surge. The negative surge caused a lower discharge (lower than the constant demand) through the demand junction, and then the resulting demand decrease caused a positive surge, which dissipated the reflected negative surge from the reservoir.

The pressure-insensitive constant demand model can be easily replaced with a pressure-sensitive orifice model.

Table 1 shows the maximum positive surge (obtained by subtracting the maximum transient pressure from the initial steady pressure), the maximum negative surge (obtained by subtracting the minimum transient pressure from the initial steady pressure), and their differences for all five junctions and for both pressure-insensitive demand and pressure-sensitive demand analyses. Because the pressure-insensitive demand model cannot accurately represent the pressure-sensitive characteristics of a nodal demand, the difference between the two transient demand models increased as the distance from the location causing the transient (junction 5) increased. For example, the difference in the maximum pressure at junction 5 was negligible (1.9/26.1 = 7%) whereas the maximum pressure difference for junction 1 was significant (5.7/20 = 29%). In addition, the reflected surge wave shown in Table 1 provided the worst results, with more than 100% difference for all junctions. Certainly, in this particular case with the pressure-insensitive demand, estimating the occurrence of local vacuum conditions and cavitation at specific locations or contaminant intrusion during negative transients would be exaggerated. The differences shown in Table 1 were within the first cycle of a surge wave normally used to estimate the maximum and minimum pressures in the system. Therefore, the corresponding surge-protection device, especially under the negative transient, could well be overdesigned.

Furthermore, the maximum positive surge of pressure-insensitive demand at junction 5 of 28.0 m (91.9 ft)

TABLE 1 Maximum positive and negative surges

	Maximum Positive Surge—*m (ft)*			Maximum Negative Surge—*m (ft)*		
Junction	Pressure-insensitive Demand	Pressure-sensitive Demand	Difference	Pressure-insensitive Demand	Pressure-sensitive Demand	Difference
1	25.7 (84.2)	20.0 (65.5)	5.7 (18.7)	–22.6 (–74.2)	–9.8 (–32.1)	12.8 (42.1)
2	26.4 (86.5)	21.4 (70.3)	4.9 (16.2)	–22.0 (–72.2)	–9.6 (–31.4)	12.4 (40.8)
3	27.0 (88.6)	23.0 (75.3)	4.1 (13.3)	–21.8 (–71.4)	–9.7 (–31.9)	12.0 (39.5)
4	27.6 (90.4)	24.5 (80.4)	3.0 (10.0)	–21.9 (–71.8)	–10.2 (–33.6)	11.7 (38.3)
5	28.0 (91.9)	26.1 (85.5)	1.9 (6.4)	–22.3 (–73.2)	–11.1 (–36.4)	11.2 (36.8)

was higher than the potential surge of 26.0 m (85.3 ft), which is defined as aV/g in which a is the wave speed of 1,000 m/s (3,280 fps), V is the velocity of water in the pipeline at 0.255 m/s (0.837 fps), and g is the acceleration of gravity at 9.81 m/s^2 (32.2 ft/s^2). This increased pressure that was greater than the potential surge after the initial flow stoppage is termed line packing and refers to the increase in the storage capacity of a pipeline. Line packing occurs because only part of the flow is stopped by the first compression wave, and then the flow is stopped completely at the valve; thus, the pressure continues to rise, the pipe wall expands, and the liquid continues to be compressed. Packing varies with line diameter, wall thickness, and length. The potential surge aV/g is inaccurate in certain cases and thus is likely to lead to poor design. The inadequacies of this simplistic modeling approach for surge analysis have been described in detail (Jung et al, 2007a). Of interest is the fact that the maximum positive surge of the pressure-sensitive demand model at junction 5 (26.1 m [85.5 ft]) was almost the same as the potential surge. Because pressure-sensitive demands at the upstream of junction 5 caused negative surges, the negative surges were propagated to junction 5 after the head at junction 5 reached the potential surge (Figure 5).

Another major pitfall of pressure-insensitive demand is its incorrect independency on nodal elevation. Flow discharge through an emitter is dependent on both elevation and pressure. Tables 2 and 3 list the maximum positive surge, the maximum negative surge, and their differences for the system shown in Figure 3 but with different junction elevations of 20 m and –20 m, respectively. Because the system was pressurized and the demand fixed, the maximum positive and negative surges of the pressure-insensitive demand model for both elevations were identical to those shown in Table 1. As long as the total head is greater than the cavitation pressure, the surge wave of the pressure-insensitive demand model is independent of demand elevation. However, actual transient discharge through an emitter is dependent on the elevation. The higher elevation had the lower static pressure, causing the larger emitter coefficient in Eq 2 to maintain the same initial steady discharge. The increased emitter coefficient attributable to the higher elevation produced a greater discharge for a positive surge and a lower discharge for a negative surge. The increased discharge caused a negative surge that interacted with the initial positive surge, whereas the decreased discharge had the opposite effect. Therefore, higher elevation led to increased surge pressure dissipation. Figure 6 shows

TABLE 2 Maximum positive and negative surges with increased elevation (20 m)

Junction	Maximum Positive Surge—*m (ft)*			Maximum Negative Surge—*m (ft)*		
	Pressure-insensitive Demand	Pressure-sensitive Demand	Difference	Pressure-insensitive Demand	Pressure-sensitive Demand	Difference
1	25.7 (84.2)	18.8 (61.7)	6.9 (22.5)	–22.6 (–74.2)	–7.9 (–25.9)	14.7 (48.3)
2	26.4 (86.5)	20.5 (67.2)	5.9 (19.3)	–22.0 (–72.2)	–7.7 (–25.3)	14.3 (46.9)
3	27.0 (88.6)	22.3 (73.1)	4.7 (15.5)	–21.8 (–71.4)	–7.9 (–25.9)	13.9 (45.6)
4	27.6 (90.4)	24.2 (79.3)	3.4 (11.2)	–21.9 (–71.8)	–8.4 (–27.6)	13.5 (44.2)
5	28.0 (91.9)	26.1 (85.5)	1.9 (6.4)	–22.3 (–73.2)	–9.3 (–30.5)	13.0 (42.7)

TABLE 3 Maximum positive and negative surges with decreased elevation (–20 m)

Junction	Maximum Positive Surge—*m (ft)*			Maximum Negative Surge—*m (ft)*		
	Pressure-insensitive Demand	Pressure-sensitive Demand	Difference	Pressure-insensitive Demand	Pressure-sensitive Demand	Difference
1	25.7 (84.2)	20.8 (68.2)	4.9 (16.0)	–22.6 (–74.2)	–11.3 (–37.0)	11.3 (37.1)
2	26.4 (86.5)	22.1 (72.5)	4.3 (14.0)	–22.0 (–72.2)	–11.0 (–36.2)	11.0 (36.1)
3	27.0 (88.6)	23.4 (76.9)	3.6 (11.7)	–21.8 (–71.4)	–11.1 (–36.5)	10.7 (34.9)
4	27.6 (90.4)	24.8 (81.3)	2.8 (9.2)	–21.9 (–71.8)	–11.6 (–38.0)	10.3 (33.8)
5	28.0 (91.9)	26.1 (85.5)	1.9 (6.4)	–22.3 (–73.2)	–12.4 (–40.7)	9.9 (32.5)

FIGURE 6 Surge pressure profiles at junction 1 at different elevations

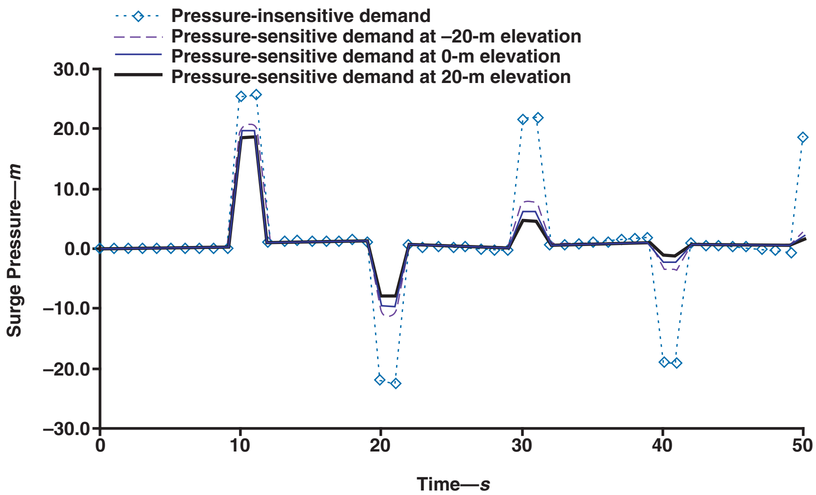


FIGURE 7 A network system

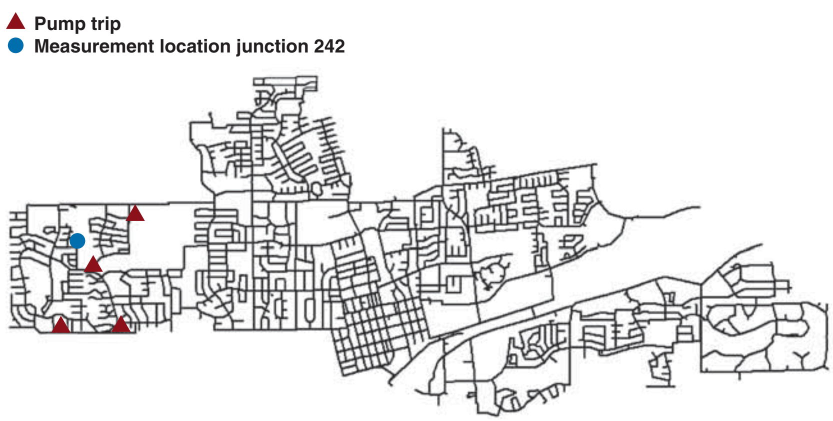
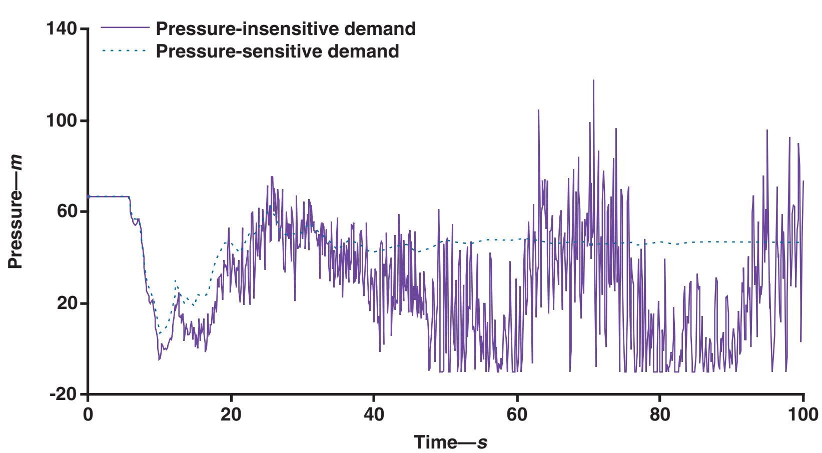


FIGURE 8 Pump trip transient results at junction 242



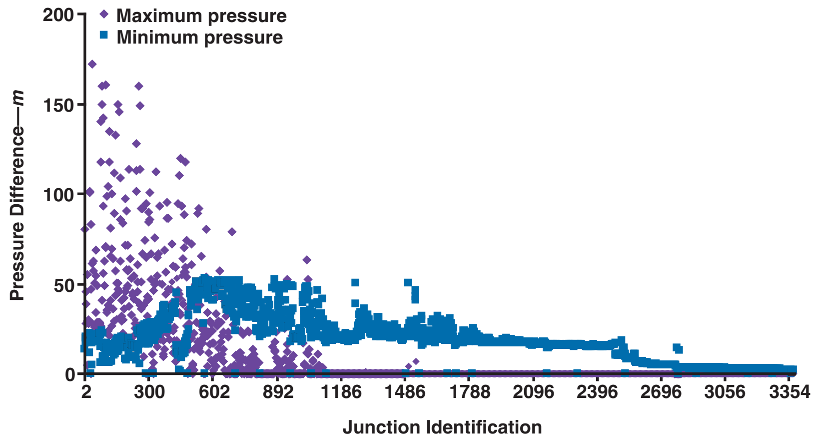
the surge pressure at junction 1 for pressure-insensitive demand and pressure-sensitive demand with three elevations (0, 20, and –20 m). As shown in the figure, the highest elevation case (20 m) of the pressure-sensitive demand caused the greatest surge pressure dissipation.

Case study 2: A pipe network system. To highlight the drawbacks of the pressure-insensitive demand model for surge analysis on a larger, more complex system, the pressure-insensitive and pressure-sensitive demand models were applied to an actual water distribution network (Figure 7). The system comprised 1,639 junctions, 2,088 pipes, 23 wells, 23 pumps, and 1 storage tank. (The identity of the corresponding water utility has been withheld because of security concerns.) For this example, the transient was initiated from pump trips at 5 s.

Figure 8 shows the transient head profiles at junction 242 using pressure-insensitive and pressure-sensitive demand analysis. As shown in the figure, both models produced the same steady hydraulic equilibrium condition (69.6 m [228.4 ft]), but the transient response for the pressure-sensitive demand model differed from that of the pressure-insensitive demand model. The maximum and minimum pressures of junction 242 for the pressure-insensitive demand model were 117.9 m (386.8 ft) and –10.1 m (–33.1 ft), whereas those of the pressure-sensitive demand model were significantly lower at 66.8 m (219.2 ft) and 4.6 m (15.1 ft). Figure 9 shows the difference in the maximum and minimum pressures for the two demand models. The greatest difference in the maximum pressures (233.9 m [767.5 ft] for pressure-insensitive demand and 61.9 m [203.2 ft] for pressure-sensitive demand) was 172.0 m (564.3 ft) at junction 30, indicating that the maximum pressure of the pressure-insensitive demand model was 278% higher than that of the pressure-sensitive demand model. Similarly, the maximum difference in the minimum pressures (–10.1 m [–33.2 ft] for pressure-insensitive demand and 42.8 m [140.5 ft] for pressure-sensitive demand) was 53.0 m (173.7 ft) at junction 30, indicating that the minimum pressure of the pressure-insensitive demand model was 124% lower than that of the pressure-sensitive demand model. These results clearly demonstrated that the pressure-insensitive demand model was unable to estimate the water-hammer phenomena correctly and often exaggerated surge waves in the distribution system. As a result, use of the pressure-insensitive demand model may lead to overdesign of surge-suppression and -protection devices. However, this overdesign does not necessarily convey a higher degree of safety unless all hydraulic transient conditions have been properly analyzed. An overdesigned system sometimes can be more detrimental than an underdesigned one because the overdesigned hydraulic devices themselves may deteriorate the system's surge response (Jung & Karney, 2006; Karney & McInnis, 1990).

Another advantage of the pressure-sensitive demand model is its ability to more accurately estimate contaminant intrusion in water distribution systems. Contaminants can intrude into pipes through leaks during a negative-pressure transient; the surge model using pressure-sensitive demand can simulate the location, amount, and duration of these intrusions. The case study of Figure 7 was applied again with the assumption that 10% of water demand at all junctions was discharged (lost) as leakage. The emitter coefficient accounting for the leakage at the demand junction was calculated using Eq 2 on the basis of 10% of the demand. This emitter coefficient was applied for the intrusion

FIGURE 9 Pressure difference between pressure-insensitive and pressure-sensitive demands



Junction numbers are for identification purposes only and do not follow any specific order.

FIGURE 10 The locations of intrusion using the pressure-sensitive demand model

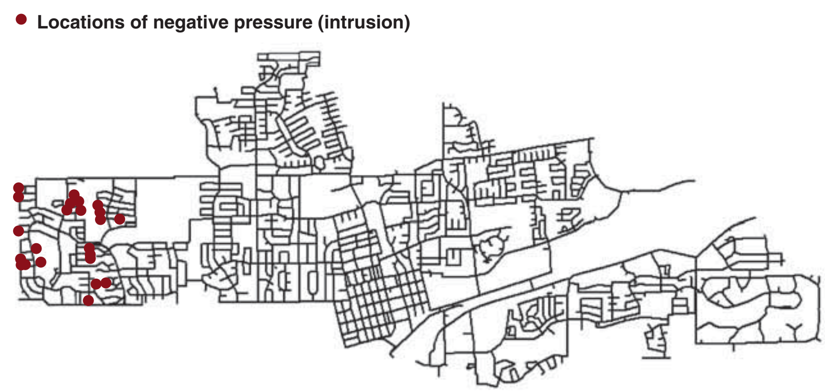
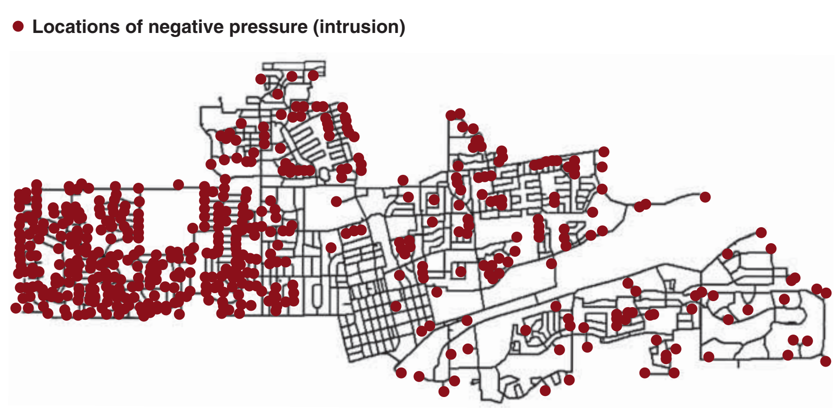


FIGURE 11 The locations of intrusion using the pressure-insensitive demand model



calculation during negative transients. Figure 10 shows the locations of negative pressure (intrusion) using the pressure-sensitive demand model. For the given transient condition, 23 nodes experienced contaminant intrusion, and the corresponding intruded volume was 0.064 L (0.017 gal). Similarly, the pressure-insensitive demand model was used to estimate the areas of possible contaminant intrusion for the system under identical transient conditions. Figure 11 shows that the pressure-insensitive demand model resulted in 379 nodes experiencing the negative transient pressures. These results demonstrated that the pressure-insensitive demand model may significantly overestimate the risk of contaminant intrusion and lead to an increased cost for surge-protection devices. However, even though the pressure-sensitive demand model may reduce the computed occurrence of intrusion in a system, the actual occurrence and amount of intrusion are also a function of other important factors, such as surrounding soil condition and standing water.

CONCLUSION

Managing and protecting water supply and distribution systems from contamination threats and emergency situations require implementation of best operational practices (e.g., maintaining a positive water pressure and an adequate level of disinfectant residual throughout the distribution system), more rigorous applications of existing engineering standards, and the use of surge modeling to predict and eliminate potential weak spots.

Surge modeling is important to safeguard against breaches in distribution systems' integrity. The assumption of pressure-insensitive demand (i.e., demand-driven analysis) has been widely applied to surge analysis, but this modeling approach may be problematic for several reasons.

- First, it ignores the implicit relationship between demand and pressure inherent in actual pipeline systems. Accurate transient modeling requires the accurate representation of fluctuating demands. A positive-pressure surge causes a higher discharge through the junctions, which dissipates the initial positive surge. Similarly, a negative-pressure surge yields a lower discharge through the junctions, causing a positive surge that dissipates the initial negative surge. Case studies showed that the surge wave of pressure-sensitive demand experienced dramatic pressure dissipation when it passed the demand junctions whereas the surge wave of pressure-insensitive demand was propagated without any disturbance except from the friction loss along the pipeline.
- Second, the transient results of the two demand models were quite different even within the first cycle of a surge wave normally used to estimate the maximum and minimum system pressures. The pressure-insensitive demand model tended to exaggerate a surge result, which could lead to overdesign of surge-protection devices. However, this overdesign does not readily imply a higher degree of safety unless all hydraulic transient conditions have been properly analyzed.
- Third, because of its fixed demand characteristic, pressure-insensitive demand is insensitive to nodal elevation as long as the total head is above the cavitation pressure. However, actual transient discharge through an emitter is dependent on the elevation. The higher elevation causes more surge dissipation because the low static pressure produces more demand from the positive surge and less demand for the negative surge.
- Finally, the pressure-sensitive demand model can more accurately assess the effect of transient-induced contaminant intrusions. The case study described in this article demonstrated that the pressure-insensitive demand model exaggerated a surge wave on the distribution system and significantly overestimated the risk of contaminant intrusion. The pressure-sensitive demand model also offers the capability of being extended to simulate the amount of intrusion from the given or assumed leakage amount.

Every hydraulic modeling exercise requires that certain assumptions and approximations be made to simplify the problem and make it possible or easy to obtain a solution. However, the assumptions made while solving the problem should be reasonable and justifiable. Because water use or demand continuously varies rapidly with time and with local pressure (which strongly influences the surge response in the distribution system), the assumption of pressure-insensitive demand is not valid for surge analysis, and the results based on the assumption may be grossly incorrect. The pressure-insensitive constant demand model can be easily replaced with a pressure-sensitive orifice model. This approach is justified by its intrinsically more accurate estimation of water-hammer phenomena as well as its proper assessment and more cost-effective selection of surge-protection and -control strategies. As this research demonstrated, a pressure-sensitive demand model can provide more accurate information than that of a constant demand model for surge analysis. Independent field testing should be carried out to further validate this finding.

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REFERENCES

Ang, W.K. & Jowitt, P.W., 2006. Solution for Water Distribution Systems Under Pressure-Deficient Conditions. *Jour. Water Resource Planning & Mgmt.,* 132:3:175.

Babayan, A.; Kapelan, Z.; Savic, D.; & Walters, G., 2005. Least-Cost Design of Water Distribution Networks Under Demand Uncertainty. *Jour. Water Resource Planning & Mgmt.—ASCE,* 131:5:375.

Boulos, P.F.; Lansey, K.E.; & Karney, B.W., 2006. *Comprehensive Water Distribution Systems Analysis Handbook for Engineers and Planners* (2nd ed.). MWH Soft Inc. Publ., Broomfield, Colo.

Boulos, P.F.; Karney, B.W.; Wood, D.J.; & Lingireddy, S., 2005. Hydraulic Transient Guidelines for Protecting Water Distribution Systems. *Jour. AWWA,* 97:5:111.

Chaudhry, M.H., 1987. *Applied Hydraulic Transients* (2nd ed.). Van Nostrand Reinhold, New York.

Funk, J.E.; VanVuuren, S.J.; Wood, D.J.; LeChevallier, M.W.; & Friedman, M., 1999. Pathogen Intrusion Into Water Distribution Systems Due to Transients. Proc. ASME/JSME Joint Fluids Engrg. Conf., San Francisco.

Ghidaoui, M.S.; Zhao, M.; McInnis, D.A.; & Axworthy, D.H., 2005. A Review of Water-hammer Theory and Practice. *Appl. Mech. Rev.,* 58:49.

Gullick, R.W.; LeChevallier, M.W.; Svindland, R.C.; & Friedman, M.J., 2004. Occurrence of Transient Low and Negative Pressures in Distribution Systems. *Jour. AWWA,* 96:11:52.

Gupta, R. & Bhave, P.R., 1996. Comparison of Methods for Predicting Deficient Network Performance. *Jour. Water Resource Planning & Mgmt.—ASCE,* 122:3:214.

Jowitt, P.W. & Xu, C., 1993. Predicting Pipe Failure Effects in Water Distribution Networks. *Jour. Water Resource Planning & Mgmt.—ASCE,* 119:1:18.

Jung, B.S.; Karney, B.W.; Boulos, P.F.; & Wood, D.J., 2007a. The Need for Comprehensive Transient Analysis of Distribution Systems. *Jour. AWWA,* 99:1:112.

Jung, B.S.; Boulos, P.F.; & Wood, D.J., 2007b. Pitfalls of Water Distribution Model Skeletonization for Surge Analysis. *Jour. AWWA,* 99:12:87.

Jung, B.S. & Karney, B.W., 2006. Hydraulic Optimization of Transient Protection Devices Using GA and PSO Approaches. *Jour. Water Resource Planning & Mgmt.—ASCE,* 132:1:44.

Jung, B.S. & Karney, B.W., 2004. Fluid Transients and Pipeline Optimization Using GA and PSO: The Diameter Connection. *Urban Water Jour.,* 1:2:167.

Karim, M.R.; Abbaszadegan, M.; & LeChevallier, M.W., 2003. Potential for Pathogen Intrusion During Pressure Transient. *Jour. AWWA,* 95:5:134.

Karney, B.W. & Fillion, J.R., 2003. Energy Dissipation Mechanisms in Water Distribution Systems. Proc. Pumps, Electromechanical Devices, and Systems (PEDS) Conf., Valencia, Spain.

Karney, B.W. & McInnis, D., 1990. Transient Analysis of Water Distribution Systems. *Jour. AWWA,* 82:7:62.

Kirmeyer, G.J.; Friedman, M.; Martel, K.; Howie, D.; LeChevallier, M.W.; Abbaszadegan, M.; Karim, M.; Funk, J.; & Harbour, J., 2001. Pathogen Intrusion Into the Distribution System. AwwaRF, Denver.

Lansey, K.E. & Boulos, P.F., 2005. *Comprehensive Handbook on Water Quality Analysis in Distribution Systems.* MWH Soft Inc. Publ., Broomfield, Colo.

LeChevallier, M.W., 1999. The Case for Maintaining a Disinfectant Residual. *Jour. AWWA,* 91:1:86.

LeChevallier, M.W.; Gullick, R.W.; Karim, M.R.; Friedman, M; & Funk, J.E., 2003. Potential for Health Risks From Intrusion of Contaminants Into the Distribution System From Pressure Transients. *Jour. Water & Health,* 1:1:3.

LeChevallier, M.W.; Gullick, R.; & Karim, M., 2002. The Potential for Health Risks From Intrusion of Contaminants Into the Distribution System From Pressure Transients. EPA Distribution System White Paper, Washington.

Loucks, D.P., 1992. Water Resource Systems Models: Their Role in Planning. *Jour. Water Resource Planning & Mgmt.—ASCE,* 118:3:214.

McInnis, D.A., 2004. A Relative Risk Assessment Framework for Evaluating Pathogen Intrusion During Transient Events in Water Pipelines. *Urban Water Jour.,* 1:2:113.

McInnis, D.A. & Karney, B.W., 1995. Transients in Distribution Networks: Field Tests and Demand Models. *Jour. Hydraul. Engrg.,* 121:3:218.

Ndambuki, J.M., 2006. Water Quality Variation Within a Distribution System: A Case Study of Eldoret Municipality, Kenya. Proc. Environmentally Sound Technology in Water Resources Management Conf., Gaborone, Botswana.

NRC (National Research Council), 2006. *Drinking Water Distribution Systems: Assessing and Reducing Risks.* National Academies Press, Washington.

Thorley, A.R.D., 1991. *Fluid Transients in Pipeline Systems.* D&L George Ltd., Herts, England.

Wood, D.J.; Lingireddy, S.; & Boulos, P.F., 2005a. *Pressure Wave Analysis of Transient Flow in Pipe Distribution Systems.* MWH Soft Inc. Publ., Broomfield, Colo.

Wood, D.J.; Lingireddy, S.; Boulos, P.F.; Karney, B.W.; & McPherson, D.L., 2005b. Numerical Methods for Modeling Transient Flow in Distribution Systems. *Jour. AWWA,* 97:7:104.

Wylie, E.B. & Streeter, V.L., 1993. *Fluid Transients in Systems.* Prentice-Hall, Englewood Cliffs, N.J.