The most advantageous way to formulate the design of a water distribution system under transient conditions is as a multi-objective optimization problem. The first objective in such a design is a least-cost optimization problem using pipe diameters as decision variables. The second objective is to minimize the likelihood of a damaging transient event. In this article, a new surge damage measure called surge damage potential factor is introduced. Evolutionary algorithms are applied to produce a set of Pareto-optimal solutions in the search space of pipe cost and surge damage potential. The model was tested on the New York City tunnel system. Comparison of the proposed method with that of a conventional approach shows that the modifications of pipe size in the design process can result in an effective and inexpensive surge control strategy.

Optimal transient network design: A multi-objective approach

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A water distribution system (WDS) is designed and operated to consistently deliver water from source to consumer in sufficient quantity, of acceptable quality, at appropriate pressure, and as economically as possible. After choosing the critical loading conditions for a distribution system design (i.e., the greater of peak-hour demand or peak-day demand and a chosen fire flow), optimization methods are applied to select the most economical set of pipe sizes that will produce the desired range of pressures in the network. The rationale behind an economical design is that by selecting the smallest possible diameter pipe set to minimize overall cost, pressures are marginally higher than an acceptable level for the specified design loading conditions. However, because the design problem is posed as a static one (i.e., the design loads are not treated as dynamic variables), a conventional design could well be suboptimal or even seriously inadequate for handling hydraulic transient events.

Transient events in a WDS are inevitable and usually occur because of actions at pump stations and control valves. Boulos et al (2006) described typical events that require transient considerations: pump startup or shutdown, valve opening or closing (i.e., any variation in a cross-sectional flow area), changes in boundary pressures (e.g., adjustments in the water level at reservoirs, pressure changes in tanks), rapid changes in demand condi-

tions (e.g., hydrant flushing), changes in transmission conditions (e.g., pipe break, line freezing), or pipe filling or draining. When transient events occur too quickly, they induce a rapid change in flow rate within the system and cause potentially objectionable pressure surges—also called water hammers—that could lead to unacceptable operating conditions. These conditions can cause breaches in the hydraulic and physical integrity of the distribution system. High-pressure transients (upsurges) can lead to system failure and excess leakage, whereas low-pressure transients (downsurges) can create vacuum conditions and pipeline collapse as well as opportunities for contaminated groundwater to intrude into the distribution system at a leaky joint or break, which can affect public health. The volume of the intrusion can range from a few gallons to hundreds of gallons (Boyd et al, 2004). Because all pipeline systems eventually leak and hydraulic transients occur continuously in WDSs, it is not surprising that low-pressure transient events offer considerable potential to draw untreated and possibly hazardous water into the piping system. Water quality studies have emphasized the need for transient analysis of large pipe networks to properly assess the potential level of intrusion associated with negative pressure events and the resulting consequences on disinfectant residual effectiveness (Boulos et al, 2006; Friedman et al, 2004; LeChevallier et al, 2003).

When transient events occur too quickly, they induce a rapid change in flow rate within the system and cause potentially objectionable pressure surges.

Pressure surges can create serious consequences for pipeline systems if not properly recognized and addressed. Many hydraulic transient approaches have been developed to identify system weak points, to predict the potentially damaging effects of hydraulic transients under various worst-case scenarios, and to evaluate how they may be eliminated or controlled (Jung et al, 2007; Boulos et al, 2005; Wood et al, 2005; Wylie & Streeter, 1993). In particular, Boulos et al (2005) provided a detailed transient analysis flow chart for the selection of components for surge control and suppression in WDSs and concluded that a transient analysis should always be performed to determine the effect of each proposed strategy on the resulting system performance. Jung et al (2007) further argued that only a systematic transient analysis can be expected to resolve complex transient characterizations and adequately protect WDSs.

Optimization methods have been widely applied to many problems associated with WDS design, management, and operation. Alperovits and Shamir (1977) applied linear programming, and Lansey and Mays (1989) suggested using nonlinear programming to optimize component sizing and the operational decisions arising in WDSs. Simpson et al (1994) and Dandy et al (1996) compared a genetic algorithm (GA) approach with both complete enumeration and nonlinear programming in the context of pipeline optimization. Although much of the pipeline optimization research has been concerned with systems under steady or near-steady flow conditions, few optimization approaches have dealt with the operating conditions pertaining to system integrity, safety, and performance. Laine and Karney (1997) applied optimization to a simple pipeline connecting a pump and a storage reservoir. A complete enumeration scheme and a probabilistic selection procedure were incorporated with both transient and steady-state analysis. Lingireddy et al (2000) described a surge tank design model based on a bilevel genetic optimization framework that produces optimal tank sizes while satisfying a specified set of pressure constraints. Jung and Karney (2004) considered the effect of transients on the choice of optimal diameter in a network considering both steady and transient criteria. More recently, Jung and Karney (2006) presented an optimum selection approach of hydraulic devices for water-hammer control in a WDS. GA and particle swarm optimization were used to optimize the preliminary selection, sizing, and placement of surge protection devices. Previous approaches, however, have focused separately on the optimal component size in the steady and transient levels; few approaches have considered the interaction of both analysis and the effect of selecting component size on surge control. In addition, these approaches have been limited to single-objectives, so their results make it difficult for the decision-maker to fully appreciate the significant amount of interaction taking place between steady and transient hydraulic analyses.

In this article, the optimal design of a WDS under transient conditions is formulated as a multi-objective optimization problem. Unlike most optimization models in which demands are set to their end-of-life levels, this approach assumes that the demand loadings vary throughout the design life of the system. The first objective is formulated as a least-cost optimization problem with pipe diameters as the decision variable. The second objective is to minimize the likelihood of damaging transient events. In this study, the authors use a new surge damage measure called the surge damage potential factor (SDPF). For any transient event, the SDPF is defined as the integration of the transient pressures that are lower than the minimum required level (e.g., datum) or higher than the maximum allowable transient pressure level (e.g., pipe ratings). Evolutionary algorithms are applied to produce a set of Pareto-optimal solutions in the search space of pipe cost and SDPF. The model is tested (by

simulation) on the New York City tunnel system, and relevant conclusions are presented.

PRESSURE SURGE CONTROL IN WDS

Because pressure surges can introduce contaminants into the WDS, break pipes, or damage hydraulic equipment such as pumps or valves, it is not surprising that many protective strategies have been proposed. These include solutions ranging from system modification and operational considerations to the addition of one or more dedicated surge protection devices (Boulos et al, 2006, 2005; Walski et al, 2003). The most common surge protection strategies use various protection devices to store water or otherwise delay the change in flow rate or to discharge water from a given pipe. For example, air-release/vacuum-breaking valves are installed at high points in a pipeline to prevent negative pressure and cavitation by emitting air into the pipe when the line pressure drops below atmospheric conditions. A pressure-relief valve prevents excessive high pressure subsequent to an upsurge by ejecting water out of a side orifice. No two systems are hydraulically identical; therefore, the ultimate choice and combination of surge protection devices will usually differ. Other surge protection strategies are to influence the root transient causes of flow changes, such as adjusting valve or pump operations. Too-rapid valve closure/opening or pump shutdown/startup may lead to water-column separation or excessively high surge pressures. The transient effect might be eased or avoided through comprehensive operator training or locking out a quick operation mechanism in the system.

System modifications can also be considered—such as pipe reinforcement (i.e., increasing a pipe's pressure rating), rerouting conduits, using larger diameter pipes, changing the pipe material, or making strategic changes in system topology. Consider the example of a single pipeline with a 1-m^3/s flow and a 1,000-m/s wave speed. When the rate of flow is changed rapidly, the kinetic energy associated with the flowing water can be converted rapidly into strain energy in the fluid and the pipe wall, thereby causing either abnormally high surge pressure or stresses. This relationship can be represented by a Joukowski equation showing the change in pressure, called a potential surge or Joukowski pressure change, ΔH, and is directly proportional to the change in the flow velocity, ΔV

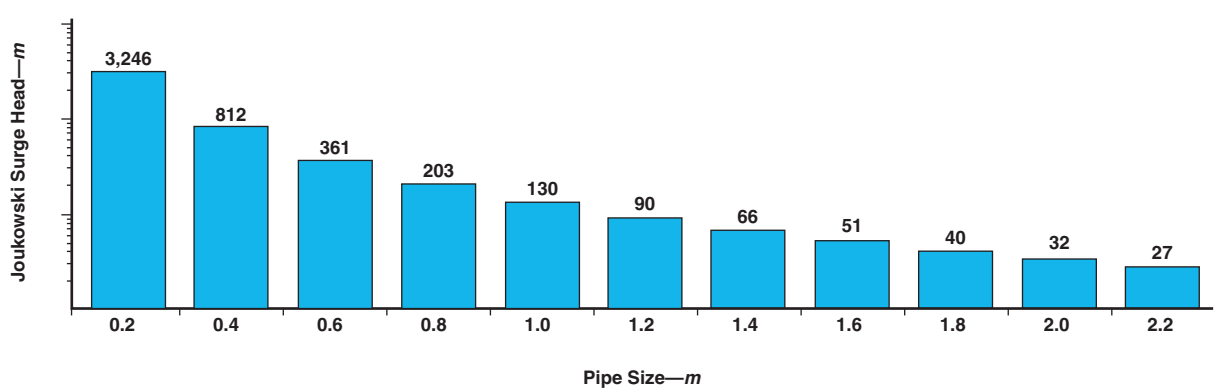
$$\Delta H = \frac{c}{g}\,\Delta V = \frac{c}{gA}\,\Delta Q \qquad (1)$$

in which c is the wave speed, g is the gravitational acceleration, and A is the cross-sectional area of pipe. Figure 1 shows the effect of pipe size on a Joukowski surge head when the 1-m^3/s flow rapidly decreases to zero. As pipe size increases, the resulting reduction in velocity decreases the magnitude of the surge pressure. For example, the increase of pipe size from 1 to 2 m results in a 75% decrease of surge pressure (from 130 to 32 m). If pipe size modifications are implemented after a system is in place, they are often expensive, but if they are examined early in the design process, they may form part of an effective and inexpensive surge approach. As a result, this approach incorporates the effects (of varying pipe sizes) on surge control to minimize the likelihood of a damaging transient event. This is in conjunction with the traditional least-cost optimization problem with the selection of pipe diameters.

MATHEMATICAL FORMULATION

Whether designing a WDS using trial-and-error enumeration methods or with formal optimization tools, a broad range of concerns must be considered. Overall cost is likely to be the primary factor and includes costs for system construction, operation, and maintenance. The

FIGURE 1 Effect of pipe size on Joukowski surge head



initial capital investment for the system includes pipes, pumps, tanks, and valves. Energy consumption occurs over time as the system is operated. The main constraints are that the nodal demands are supplied at a minimum pressure and that network flows and pressure heads must satisfy the governing equilibrium laws of conservation of energy and mass.

In this article, the optimal design of a WDS under transient conditions is formulated as a two-objective optimization problem. The first objective is formulated in Eq 2 as a least-cost optimization problem with the selection of pipe diameters as the decision variables. The second objective is to minimize the likelihood of a damaging transient event, which is measured by the parameter SDPF, defined as the integration of the transient pressures that are lower than the minimum required level (e.g., datum) or higher than the maximum allowable transient pressure level (e.g., pipe ratings). As shown in Figure 2, the SDPF is the area of transient pressure below the minimum required pressure, H^*_{minimum} and above the maximum allowable pressure, H^*_{maximum}. An SDPF value of zero represents a condition of no damage with a given transient event; an SDPF value with a higher value has a chance of greater surge damage. Therefore, the second objective, given in Eq 3, is formulated to minimize the SDPF. The SDPF approach may be applied to the whole WDS, or to particular subsystems, including single pipes. In contrast to traditional optimization models in which demands are set to their end-of-life levels, this approach assumes that the demand loadings vary throughout the design life of the system. The pipe network layout, nodal demands, and minimum head requirements are assumed to be known. The optimal design of water distribution networks can be stated mathematically as:

$$\text{Minimize Pipe Cost} = \sum_{k \in N_{\text{pipe}}} C_k\,(D_k, L_k) \tag{2}$$

$$\text{Minimize SDPF} = \sum_{i \in N_{\text{node}}} \int |H_i\,(t)|\,dt, \quad \text{where } H < H^*_{\text{minimum}} \text{ or } H > H^*_{\text{maximum}} \tag{3}$$

Subject to the governing transient equations

$$\frac{1}{gA_p}\frac{\partial Q}{\partial t} + \frac{\partial H}{\partial x} + \frac{R}{\Delta x}\,Q\,|Q|^{n-1} = 0 \tag{4}$$

$$\frac{\partial H}{\partial t} + \frac{a^2}{gA_p}\frac{\partial Q}{\partial x} = 0 \tag{5}$$

and a set of algebraic constraints

$$H_i\,(t) = C_1,\ Q_i\,(t) = C_2, \text{ where } t = 0,\ \forall i \in N_{\text{node}} \tag{6}$$

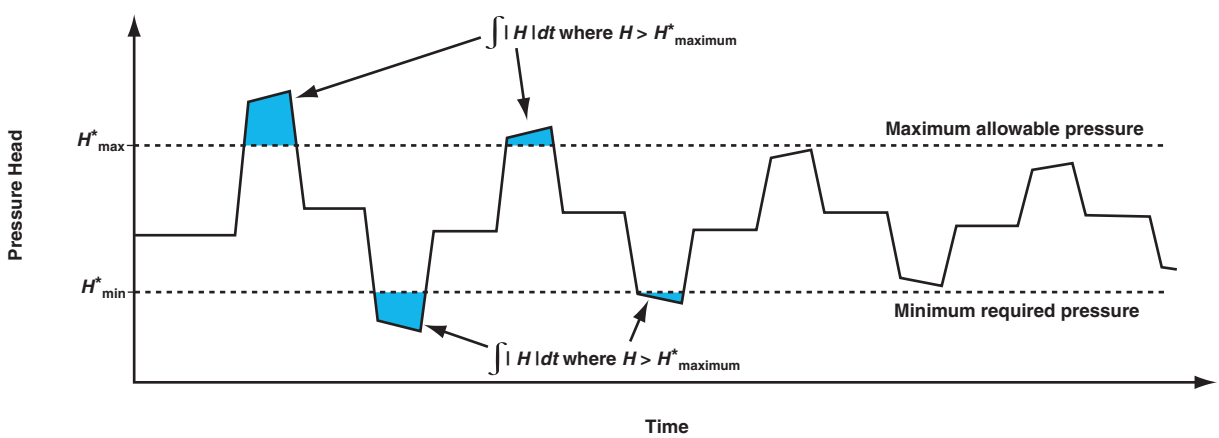
$$f\,[H_9\,(t), Q_i\,(t)] = C_3, \text{ where } t > 0,\ i = \text{boundary nodes} \tag{7}$$

$$H_i\,(t) \geq H_{\text{minimum } i}, \text{ where } t = 0,\ \forall i \in N_{\text{node}} \tag{8}$$

$$D_k \in \{\text{D}\},\ \forall k \in N_{\text{pipe}} \tag{9}$$

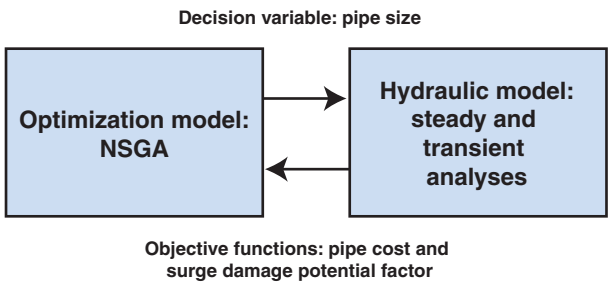
in which D_k = discrete pipe diameters selected from the set of commercially available pipe sizes $\{D\}$ (Eq 9); $C_k(D_k$ $L_k)$ = cost of pipe k with diameter D_k and length L_k; dt is the integration of transient pressures along the transient period (t); and H = piezometric head; H^*_{maximum} and

FIGURE 2 Measuring pipe surge damage potential factor



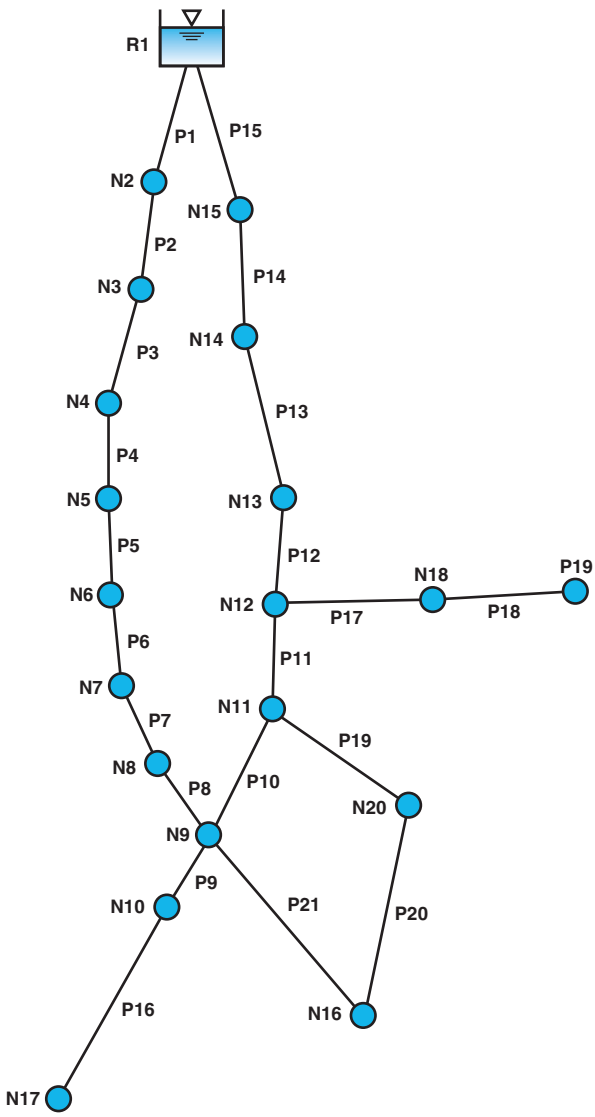
H—*piezometric head,* $H^*_{maximum}$—*maximum permissible head,* $H^*_{minimum}$—*minimum permissible head*

FIGURE 3 Flowchart of pipeline optimization



NSGA—nondominated sorting genetic algorithm

FIGURE 4 New York City tunnel system schematic



N—node, P—pipe size in millimetres; R—reservoir

H^*_{minimum} are the maximum and minimum permissible heads (e.g., representing pipe ratings, health concerns for negative pressures), respectively. Equations 4 and 5 represent the momentum equation and mass conservation for transient flow in closed conduits (Wylie & Streeter, 1993). Here x is distance along the centerline of the conduit; t is time; Q = volumetric flow rate; a = celerity of the shock wave; A_p = cross-sectional area of the pipe; and g = acceleration resulting from gravity. The friction term R in Eq 4 can be represented by

$$R = f_p\ \Delta x/2g\ D_p\ A_p^2,\ n = 2, \tag{10}$$

or

$$R = \Delta x/(0.278\ CD_p^{2.63})^{1/0.54},\ n = 1/0.54, \tag{11}$$

in which f_p = Darcy-Weisbach friction factor and C = Hazen-Williams roughness coefficient. The two hyperbolic partial differential calculations in Eqs 4 and 5 are subject to the initial conditions of Eq 6 and boundary conditions of Eq 7, in which C_1, C_2, and C_3 are constants. Initial conditions are typically taken as steady. Simple boundary conditions of constant reservoir level and fixed demand are assumed, but combined relationships between H and Q are typical for most boundaries. Equation 8 requires that the nodal pressure H for any node i (where total number of nodes is N_{node}) is equal to or greater than a specified minimum pressure H_{minimum} for a steady-state condition.

MULTI-OBJECTIVE OPTIMIZATION

The remaining but challenging question is how to apply an optimization method to the suggested problem of WDS optimization. Single-objective optimization algorithms have been applied to solve WDS problems. Gradient-based mathematical optimization methods (Boulos et al, 2006; Walski et al, 2003) have provided efficient computational procedures for achieving a lower-cost solution, but the methods suffered from some disadvantages: (1) being ineffective at reaching the global optimal solution because of zero-gradient optimality criteria, which easily trapped a search process at a local optimal solution; (2) the lack of flexibility in handling discrete design variables and optimizing a partial network that is often required for many practical engineering designs; (3) the complexity of implementing and using the techniques; and (4) the requirement to compute either first- or second-derivative information to generate improvements in the objective function. Several researchers have used GA optimization for solving such complex WDS optimizations (Dandy et al, 1996; Simpson et al, 1994). The ant colony optimization (Maier et al, 2003) and the shuffled frog leaping algorithm (Eusuff & Lansey, 2003) have also been applied for obtaining specific optimal designs of WDSs. These methods offer significant advantages over gradient-based optimization approaches in that they do

not require any gradient information and search for the optimal solution by continuing to evaluate multiple solution vectors simultaneously.

Multi-objective optimization algorithms have been introduced to solve WDS problems with multiple conflicting criteria or design objectives (Jeong & Abraham, 2006; Farmani et al, 2005; Prasas et al, 2004; Prasas & Park, 2004; Kapelan et al, 2003). As opposed to the single-objective optimization method for finding the best solution, which corresponds to the minimum or maximum value of the objective function, multi-objective optimization cannot produce a single optimal solution with conflicting objectives. The interaction among different objectives instead gives rise to a set of compromised solutions, known as Pareto-optimal solutions. Kapelan et al (2003) applied the multi-objective optimization approach to the sampling design for WDS with the objectives of maximizing the calibrated model accuracy by minimizing the relevant uncertainties and the total sampling design cost. Prasas et al (2004) investigated the booster disinfection facility location and injection scheduling problem in a WDS. They formulated the problem as a multi-objective optimization model to minimize the total disinfectant dose and to maximize the volumetric demand within specified residual limits. Farmani et al (2005) and Prasas and Park (2004) presented a multi-objective approach to a WDS design that minimized network cost and maximized network reliability by providing excess head greater than the minimum allowable head. Jeong and Abraham (2006) considered a physical attack scenario in a water infrastructure system and offered a model to generate a set of optimal operational strategies to minimize consequences of intentional physical attacks. They used a multiobjective GA to minimize the degree of the disruption of critical infrastructure services, economic loss, and the number of customers affected.

In this article, for the given dual-objective (pipe cost and SDPF) problem, nondominated sorting GA (NSGA), developed by Srinivas and Deb (1994), is used to circumvent subjective decision-making and to generate Pareto-optimal solutions for the multi-objective optimization problem. Figure 3 shows a flow chart of the framework for optimizing the pipeline system considering the dual-objective problem. First, an optimization program initializes the pipe sizes as decision variables and the pipe cost is calculated. The hydraulic model then analyzes the given system and uses the optimization program to check whether the solution satisfies the required constraints in Eqs 4–9 and then computes the second objective function in Eq 3. With the dual-objective function values, the optimization model then evaluates the system and creates a new set of system alternatives for the next iteration. The iterations continue until an optimal or an acceptable solution is reached.

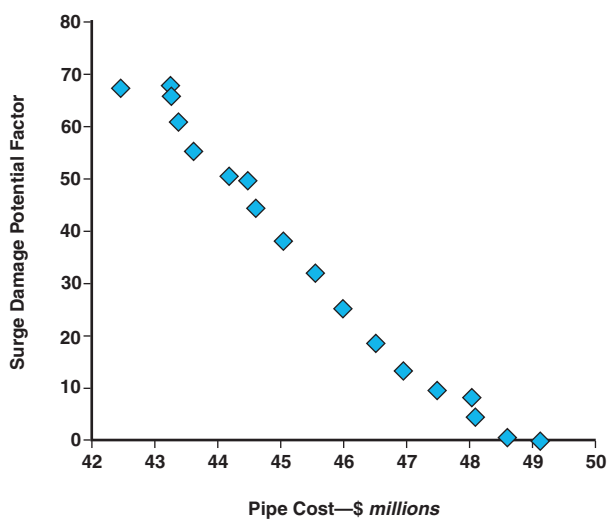
Water quality studies have emphasized the need for transient analysis of large pipe networks to properly assess the potential level of intrusion associated with negative pressure events.

CASE STUDY

The proposed method is illustrated using the New York City tunnel system (Schaake & Lai, 1969). The network (Figure 4) has been extensively studied for steady-state conditions. It comprises 22 nodes (20 demand nodes), 21 pipes, and one source node. The system is gravity-driven and draws water from the Hillview reservoir to the downstream network. The objective of the optimization problem is to add new pipes parallel to the existing ones. The new pipe diameters need to be selected from 15 available sizes. A single demand pattern (57,130 L/s) was considered and a minimum allowable hydraulic grade was specified for each node. The network and cost data are shown in Dandy et al (1996).

Since the system was first examined in 1969 by Schaake and Lai, many researchers have used it to test the numerical effectiveness, efficacy, and performance of their respective techniques (Eusuff & Lansey, 2003; Maier et al, 2003; Wu et al, 2001; Savic & Walters, 1997; Dandy et al, 1996). However, all of these approaches were based only on steady-state optimization. In this study, the optimization process includes the effect of a surge event as well as the existing steady-state considerations. By doing so, different

FIGURE 5 Pareto-optimal solutions of pipe cost and surge damage potential factor



design decisions would be required, and the restrictive search based on limited operating conditions is likely suboptimal for a broader range of demand loadings.

To introduce transient conditions into this case study, a variety of possible causes could be selected. For convenience, a valve opening that increases the demand at node 10 from 28 L/s to 4,814 L/s for 1 s was chosen to characterize the transient performance of the system. This increased demand may be the result of a fire flow, a burst pipe, an operator error, or a temporary increase in water consumption. The maximum permissible heads H^*_{maximum} and H^*_{minimum} in Eq 3 are assumed to be 304.8 m and 54.9 m, respectively, for the whole system. Because of the rapid demand increase at node 10, a reduced pressure wave moves through the system. This wave is reflected from the upstream reservoir and then propagates back and forth in the system while being tracked numerically using the method of characteristics (Wylie & Streeter, 1993) or the wave characteristic method (Ramalingam et al, 2009; Boulos et al, 2006).

Because pressure surges can introduce contaminants, break pipes, or damage hydraulic equipment, it is not surprising that many protective strategies have been proposed.

The multi-objective method, NSGA, was applied to satisfy Eqs 2–9. For this problem, the probability of mutation was set to 0.025, the probability of (single-point) crossover was set to 0.9, the population size was set to 400, the length of each chromosome was set to 84, and the simulations run for 100 generations. For this problem, 16 decision variables including the "do-nothing" option made up a solution space of 16^{21} or 1.93×10^{25} possible pipe combinations. The NSGA initialized the population of pipe diameters, calculated the cost of pipelines and the SDPF of transient network design model to satisfy the given constraints and then created a new population for the next generation. The SDPF approach was applied to the whole WDS into which a single transient event was introduced.

TABLE 1 The pipe size of Pareto-optimal solutions

Multi-Objective Solutions	Pipe Cost *$ millions*	SDPF*	Pipe Size—*mm*								
			1–3, 5–8, 10–14, 20	4	9	15	16	17	18	19	21
Dandy†	38.8	99.86	‡			3,000	2,100	2,400	2,100	1,800	1,800
M1	42.5	67.5				3,000	2,100	3,000	1,800	3,000	1,500
M2	43.2	67.4		900		3,000	2,100	3,000	1,800	3,000	,1500
M3	43.3	65.6				3,300	2,100	3,000	1,800	3,000	1,500
M4	43.4	60.8			900	3,000	2,100	3,000	1,800	3,000	1,500
M5	43.6	55.4				3,000	2,100	3,000	2,100	3,000	1500
M6	44.2	50.6			1,500	3,000	2,100	3,000	1,800	3,000	1,500
M7	44.5	49.7			900	3,000	2,100	3,000	2,100	3,000	1,500
M8	44.6	44.5			1,800	3,000	2,100	3,000	1,800	3,000	1,500
M9	45.0	38.2			2,100	3,000	2,100	3,000	1,800	3,000	1,500
M10	45.5	31.7			2,400	3,000	2,100	3,000	1,800	3,000	1,500
M11	46.0	25.2			2,700	3,000	2,100	3,000	1,800	3,000	15,00
M12	46.5	19.1			3,000	3,000	2,100	3,000	1,800	3,000	1,500
M13	47.0	13.8			3,300	3,000	,2100	3,000	1,800	3,000	1,500
M14	47.5	9.7			3,600	3,000	2,100	3,000	1,800	3,000	1,500
M15	48.0	8.7			3,900	3,000	2,100	3,000	1,800	3,000	1,500
M16	48.1	5.0			3,300	3,000	2,100	3,000	2,100	3,000	1,500
M17	48.6	0.9			3,600	3,000	2,100	3,000	2,100	3,000	1,500
M18	49.1	0.0			3,900	3,000	2,100	3,000	2,100	3,000	1,500

M—multi-objective solution, SDPF—surge damage potential factor

*SDPF shown in Eq 2
†Optimal results from Dandy et al (1996)
‡Blank spaces indicate no pipe required.

The resulting solutions, obtained after 100 generations, are shown in Figure 5. The *x* axis plots pipe cost in Eq 2 and the *y* axis plots the SDPF in Eq 3. The interaction among different objectives gives rise to a set of compromised Pareto-optimal solutions. Each solution on the Pareto-optimal curve of Figure 5 is not dominated by any other solution. In going from one solution to another, it is not possible to improve on one objective without making the other objective worse; for example, improving the first objective of minimizing the pipe cost worsens the second objective of minimizing the SDPF. This leads to a tradeoff relationship between pipe cost and SDPF in which a decision-maker can choose a preferred solution.

Table 1 shows the pipe cost, SDPF, and pipe diameters chosen in the Pareto-optimal solutions that correspond to the 18 distinct results. The tradeoff relationship provides useful data on the cost-effectiveness of adding pipe capacity to reduce surge damage. The results indicate that when the pipe cost is increased from $42.5 million to $45 million (multiobjective solutions [M] M1–M9), the 6% additional investment in pipe can achieve a 43% reduction in surge damage (67.5–38.2%). Similarly, when the pipe cost is increased from $45 million to $49.1 million (M9–M18), the 9% additional investment in pipe can achieve the condition of no damage with the given transient event. A decision-maker can use the information in Figure 5 and Table 1 to evaluate the marginal rate of tradeoff between pipe capacity and SDPF. The Dandy et al (1996) results are shown in Table 1 to compare the optimal result of steady-state analysis alone with the authors' multiobjective approaches, including both steady-state analysis and transient analysis for surge protection. Dandy's pipe cost is lower than those of the Pareto-optimal solutions, but its SDPF is much higher, because it did not include any consideration of surge protection. Table 1 also shows the size of pipe P9 increased from 0 to 3,900 mm as the pipe cost increases. This occurs because pipe P9 is next to the surge-creating node 10 and the proper sizing of pipe P9 is most crucial for controlling the surge pressure effectively.

Table 2 shows the violating maximum and minimum pressure heads and their locations. In this study, the minimum required head, 54.9 m, is the dominant factor for controlling surge pressure. Therefore, no solutions in the Pareto-optimal curve or in the Dandy et al (1996) results violate the maximum allowable pressure. Figure 6 shows the transient head profiles obtained using Dandy's result and three multiobjective results (M1, M9, and M18) at node 17. Because pipe cost is increased with a larger diameter, the resulting reduction in velocity decreases the magnitude of the pressure wave, increasing the minimum pressure. The minimum pressures of Dandy, M1, M9, and M18 are 41.2, 41.2, 46.9, and 55.8 m, respectively. Similarly, Figure 7 shows the comparison of the transient results obtained using Dandy's result and three results of the multi-objective solutions (M1, M9, and M18) at node 19. The minimum pressures of Dandy, M1, M9, and M18 are 50.1, 53.1, 52.9, and 55.4 m, respectively. The results indicate that the previous approaches considering the steady-state design alone are, not surprisingly, inadequate for coping with a waterhammer event. The results also suggest that the proper sizing of pipe diameters is crucial to prevent water hammer. As a result, if the modifications of pipe size are considered in the design process, they can form a reliable and cost-effective surge control strategy.

CONCLUSION

Transient analysis, despite its significant concern for WDS design, is a complicated problem, and so is the optimization of a transient control strategy for WDSs. The purpose of this article was to obtain an optimal pipe network design by considering simultaneously steady and transient conditions. The objectives were to minimize both pipe cost and the likelihood of a damaging transient event. A parameter called SDPF is defined as the integration of the transient pressures that are lower than the minimum required level or higher than

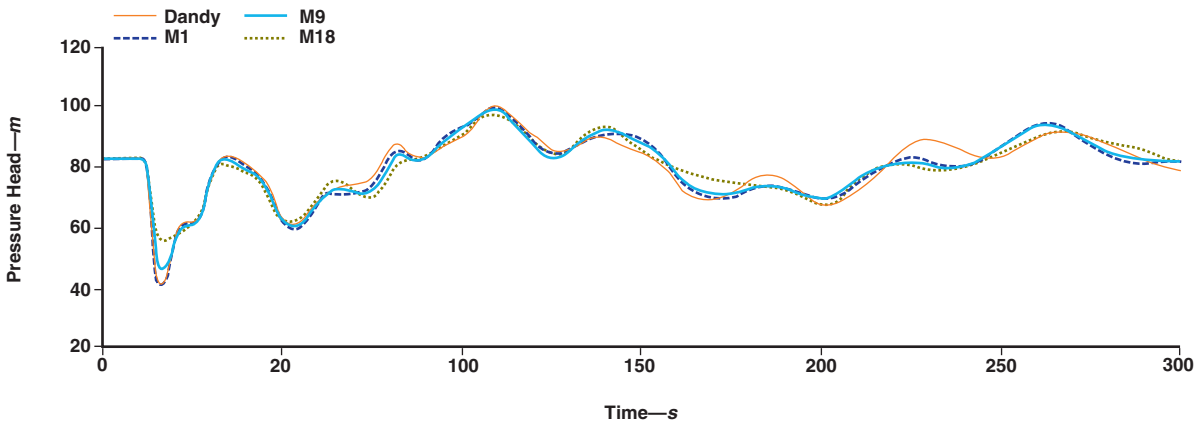
TABLE 2 Violating maximum and minimum pressure heads and their locations

Multi-Objective Solutions	Maximum		Minimum	
	Node	Head—*m*	Node	Head—*m*
Dandy*	†		17	41.2
M1			17	41.2
M2			17	41.2
M3			19	41.3
M4			19	42.4
M5			17	41.2
M6			19	44.3
M7			17	42.4
M8			19	45.6
M9			19	46.9
M10			19	48.4
M11			19	49.9
M12			19	51.4
M13			19	52.8
M14			19	52.8
M15			19	52.8
M16			19	52.9
M17			19	54.3
M18				

M—multi-objective solution

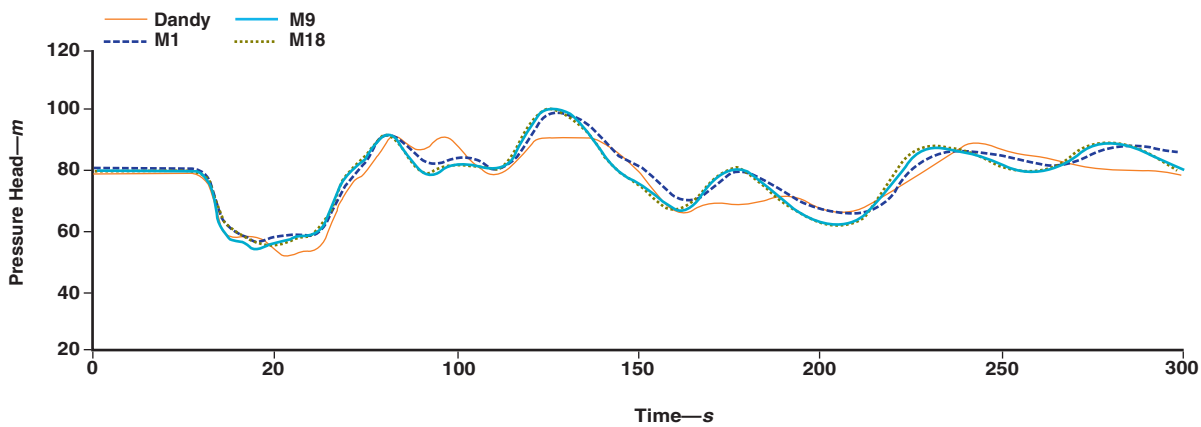
*Optimal results from Dandy et al (1996)
†Blank spaces indicate no violation of maximum permissible heads.

FIGURE 6 Transient head profile at N17



Dandy—optimal results from Dandy et al (1996), M—multi-objective solution, N—node

FIGURE 7 Transient head profile at N19



Dandy—optimal results from Dandy et al (1996), M—multi-objective solution, N—node

the maximum allowable transient pressure level. Evolutionary algorithms were applied to produce a set of Pareto-optimal solutions in the search space of pipe cost and SDPF. The case study using the New York City tunnel system indicated that the previous approaches, which considered steady-state design alone, are inadequate for coping with a water-hammer event. In addition, the study showed that pipe size is significant to controlling transient response; as a result, proper pipe selection and transient consideration can minimize the damage of water-hammer events and form an effective and inexpensive surge control strategy.

To provide a comprehensive analysis of a WDS design (or to fully evaluate the sensitivity to transients of an existing system), several transient events should be introduced into the system to collect useful data. The cumulative results would then be incorporated into determining Pareto-optimal solutions. For clarity of presentation in this study, only one transient event was shown.

Moreover, the authors considered the optimal selection of pipe diameters for a surge protection strategy. A more global approach will ultimately also be considered, in addition to pipe size, transient properties (e.g., operation speed), system characteristics (e.g., system topography, pipe material and thickness), and transient protection devices. This comprehensive design framework will offer a more complete range of systematic surge protection strategies and result in more reliable cost-optimization solutions.

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