

Many surge analysis and design rules have evolved over time to help utilities cope with the complexity of transient phenomena. These rules have been widely applied to simplify analysis by restricting both the number and difficulty of the transient cases that need to be evaluated. On further reflection, however, the implicit assumption that elementary and conservative rules are a valid basis for design has often been shown to be questionable and sometimes dangerous. Indeed, many published guidelines are so misleading and so frequently false that they should only be used with extreme caution, if at all. This article specifically reviews a number of guidelines or suggestions found in various AWWA publications for water hammer analysis and provides a set of warnings about the misunderstandings and dangers that can arise from such simplifications. The authors conclude that only systematic and informed water hammer analysis can be expected to resolve complex transient characterizations and adequately protect distribution systems from the vagaries and challenges of rapid transients.

# The need for comprehensive transient analysis of distribution systems

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**A** distribution system is not a single entity but rather comprises a complex network of pipes, pumps, valves, reservoirs, and storage tanks that transports water from its source or sources to various consumers. It is designed and operated to consistently and economically deliver water in sufficient quantity, of acceptable quality, and at appropriate pressure. Huge amounts of capital will continue to be spent on the design of new distribution systems and the rehabilitation of existing systems in both developing and developed countries. The magnitude of the needs is a challenge even to visualize: in the United States alone, some 880,000 mi of unlined cast-iron and steel pipes are estimated to be in poor condition, representing an approximate replacement value of \$348 billion (Clark & Grayman, 1998).

## BACKGROUND

**Transients.** Of the many challenges that face water utilities, one critical but too-often-forgotten issue is protecting the system from excessive transient or water hammer conditions. Surge analysis is essential to estimate the worst-case scenarios in the distribution system (Boulos et al, 2005). In essence, transients occur whenever flow conditions are altered, for they are the physics of change, bringing “news” of any adjustment throughout the network. However, transients are most severe when rapid changes occur, such as those resulting from power failure, emergency valve operations, or firefighting. These

changes are generally characterized by fluctuating pressures and velocities and are critical precisely because pressure variations can be of high magnitude, possibly large enough to break or damage pipes or other equipment or to greatly disrupt delivery conditions.

Transient regimes in a distribution system are inevitable and will normally occur as a result of action at pump stations and control valves. Regions that are particularly susceptible to transients are high elevation areas, locations with either low or high static pressures, and regions far removed from overhead storage (Friedman, 2003).

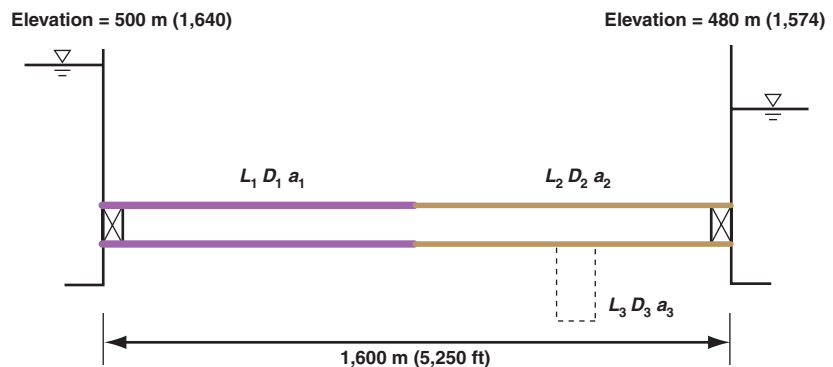
**Firefighting demands.** Distribution systems sometimes are called on to deliver large flow demands at adequate pressures for firefighting. Although these fire demands occur infrequently, they may constitute a highly constraining factor in pipeline design. Design procedures therefore should evaluate the ability of the system to meet firefighting demands at all relevant hydrant locations. Even though the occurrence of simultaneous fires at all possible locations is not realistic, a variety of firefighting demand patterns must still be considered. Under transient conditions, the designer must anticipate both the establishment of firefighting flows and their ultimate curtailment, a process that often unfolds rapidly in time and can create significant transient pressures, particularly if fire crews receive little specific training or instruction.

**Water quality considerations.** A more recently highlighted motivation for conducting a surge analysis arises from water quality considerations. One of the challenges in managing distribution system water quality is that contaminants can intrude into pipes through leaks from reduced- or negative-pressure transients. In reality, all pipeline systems leak, and hydraulic transients occur more or less continuously in distribution systems, so it is not surprising that low-pressure transients introduce a considerable risk of drawing untreated and possibly hazardous water into a pipeline system (Fernandes & Karney, 2004; McInnis, 2004; Kar-

ney, 2003). In fact, soil and water samples were recently collected adjacent to drinking water pipelines and then tested for occurrence of total and fecal coliforms, *Clostridium perfringens*, *Bacillus subtilis*, coliphage, and enteric viruses (Karim et al, 2003). The study found that indicator microorganisms and enteric viruses were detected in more than 50% of the samples examined.

These and other results suggest that during negative- or low-pressure situations, microorganisms can enter the

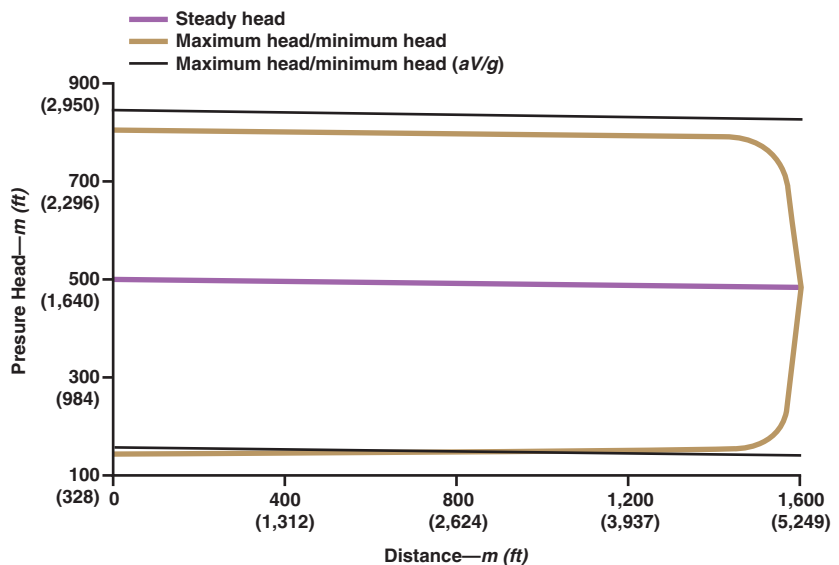
**FIGURE 1** Case study of single-pipeline system



a—wave speed, D—diameter, L—length

Three systems are considered: a pipeline with uniform properties, a pipeline with reflection points created by changes in properties, and a pipeline with an attached dead-end pipe segment.

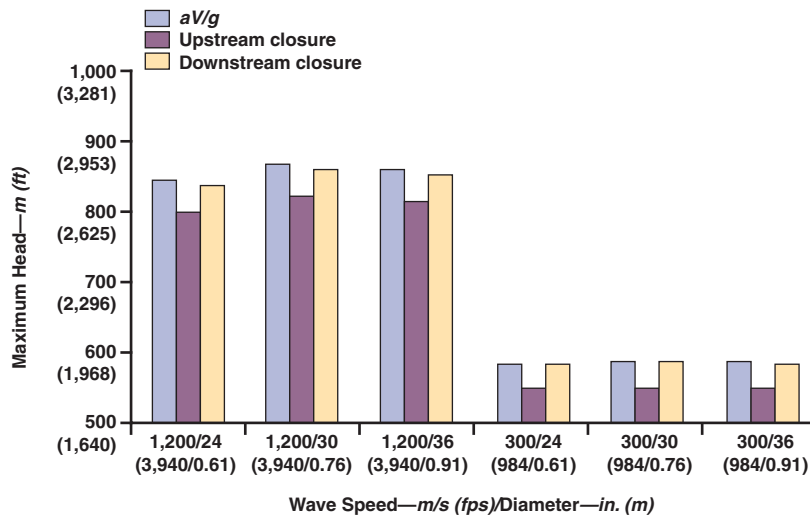
**FIGURE 2** Transient response for upstream valve closure



a—wave speed in m/s, aV/g—Joukowski surge head, D—diameter in in.

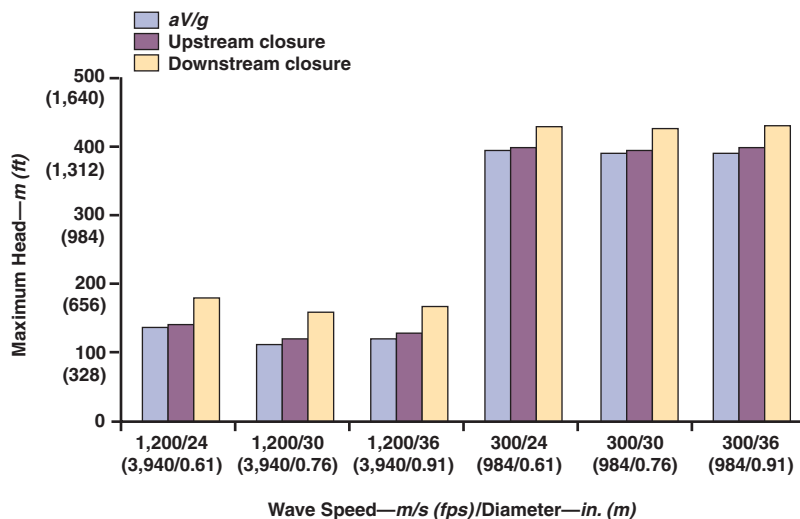
Properties are a/D (1,200/24 in.).

**FIGURE 3** Maximum head of uniform pipeline



aV/g—Joukowski surge head

**FIGURE 4** Minimum head of uniform pipeline



aV/g—Joukowski surge head

distribution system directly through pipeline leaks. For these reasons, the designer should not overlook the effects of water hammer or pressure surges in the design and operation of the distribution system or the evaluation of either system performance or ultimate system cost.

**Design obstacles.** In spite of the importance of water hammer, the remaining and obviously troublesome problem is the relative complexity of the required computer modeling and engineering analysis. The governing equations describing the transient flow represent a set of nonlinear partial differential equations with sometimes sophis-

ticated boundary conditions. In addition, the hydraulic devices are complex, performance data are difficult to obtain and sometimes poorly understood, and pipeline systems themselves are subject to a variety of operating conditions and requirements. To make matters worse, the physical characteristic of the pulse wave propagation is frequently hard to visualize or interpret, even for the analyst accustomed to transient phenomena (Karney & McInnis, 1990).

This complexity of both transient phenomena and analysis has at times induced engineers to use simplified design procedures. Many simplified guidelines have been published in the past and can be found in various AWWA literature, e.g., Manual M11, *Steel Water Pipe—A Guide for Design and Installation* (AWWA, 2004); Manual M23, *PVC [polyvinyl chloride] Pipe—Design and Installation* (AWWA, 2002); and C403-00, *Selection of Asbestos-Cement Transmission Pipe, Sizes 18 in. Through 42 in. (450 mm Through 1,050 mm; AWWA, 2000)*. However, any limited approach should carefully consider a fundamental question: “Are the simplifications both conservative and reasonable?” Unfortunately, the a priori assumption of design that some rudimentary and conservative system can be found is questionable. This article identifies several of these misconceptions or limitations of simplified rules and describes the general weakness and danger of the simplified designs for water hammer. Case studies illustrate the

potential for erroneous application by comparing a comprehensive analysis with a simplified one.

## REVIEW OF RULES

**Guideline examples.** To set the stage for more detailed discussion, it is useful to briefly summarize a few specific guidelines found in the AWWA literature. The AWWA literature has not been singled out for its particularly extreme views; rather, it is readily at hand and is typical of a large body of relatively accessible and widely dispersed literature. Seven examples from the guidelines are given,

followed by a discussion of the basis—and sometimes the danger—of each articulated position.

**Surge pressure.** The pressure rise for instantaneous closure is directly proportional to the fluid velocity at cutoff and to the velocity of the predicted surge wave but is independent of the length of the conduit (AWWA, 2004; 2000). Thus, the relation used for analysis is simply the well-known Joukowski expression for sudden closures in frictionless pipes:

$$h = \frac{aV}{g} \quad (1)$$

in which  $h$  is the surge pressure,  $V$  is the velocity of water in the pipeline,  $a$  is the wave speed, and  $g$  is the gravitational acceleration.

**Wave speed.** Pressure waves are established that move through the pipeline system at rates of 2,500–4,500 fps (760–1,370 m/s), with the exact rate depending primarily on the pipe wall material. The velocity of the wave is the same as the velocity of sound in water, modified by physical characteristics of the pipeline and is estimated by the following equation (AWWA, 2000):

$$a = \frac{V_s}{\sqrt{1 + \frac{kd}{Ee}}} \quad (2)$$

in which  $k$  is the modulus of compression of water,  $d$  is the internal pipe diameter,  $E$  is the modulus of elasticity of the pipe,  $e$  is the pipe wall thickness, and  $V_s$  is the velocity of sound in water.

**Assumption of uniform properties.** When the flow rate is changed in a time greater than zero but less than or equal to  $2L/a$  s in which  $L$  is the pipeline length,  $a$  is the wave speed, and  $s$  is the time in seconds, the magnitude of the pressure rise is the same as with instantaneous closure, but the duration of the maximum value decreases as the time of closure approaches  $2L/a$  s (AWWA, 2004). The thinking here is that the time it takes for a water hammer wave to travel the length of the system and back (i.e.,  $2L/a$ ) is the minimum time needed for the possibly mitigating effect of boundary conditions at the far end of the system to be experienced.

**Maximum pressure.** The maximum pressure at the control valve exists along the full length of the

line with instantaneous closure and for slower rates moves up the pipe a distance equal to  $L - (Ta/2)$  in which  $T$  is the closing time and then decreases uniformly (AWWA, 2004).

**Assumption of surge pressure independence from pipeline profile.** The surge pressure distribution along the conduit is independent of the profile or ground contour of the line as long as the total pressure remains above the vapor pressure of the fluid (AWWA, 2004).

**Valve closing and maximum pressure rise.** For valve closing times greater than  $2L/a$  s, the maximum pressure rise is a function of the maximum rate of change in flow with respect to time,  $dV/dt$  (AWWA, 2004).

**Pipe design and selection for pressure surges.** To design or select a pipe for occasional pressure surges, the following approach is sometimes recommended:

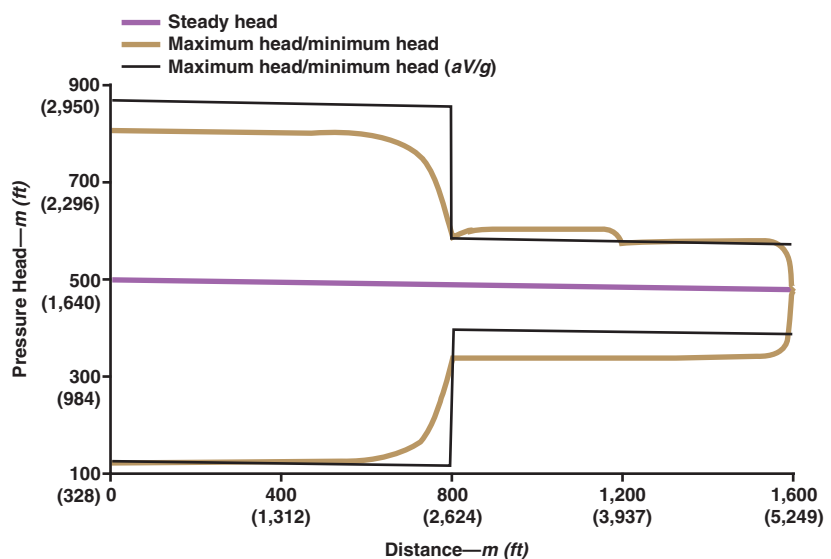
$$WPR = STR - (V \times P'_s) \quad (3)$$

in which  $WPR$  is the working pressure rating in psi,  $STR$  is the short-term rating of the pipe in psi,  $V$  is the actual system velocity in fps, and  $P'_s$  is 1-fps surge pressure in psi. The  $STR$  is calculated by applying a factor of safety ( $SF' = 2.5$ ) to the short-term strength ( $STS$ ) of PVC pipe as in Eq 4:

$$STR = STS/SF' \quad (4)$$

These levels should be considered to be the design surge capacity limits for PVC pressure pipe manufactured to AWWA standards for a transmission main application (AWWA, 2002).

**FIGURE 5** Transient response for upstream valve closure



$a$ —wave speed in m/s,  $aV/g$ —Joukowski surge head,  $D$ —diameter in in.

Properties are  $a_1/D_1$  (1,200/30) and  $a_2/D_2$  (300/30).

**Where simple rules can break down.** Of course, these examples from AWWA transient guidelines do have considerable basis in fact. For example, the origin of the first rule is the famous fundamental equation of water hammer, which is also called the Joukowski relation. The origins of this relationship are somewhat complex, as Tijsseling and Anderson (2004) have pointed out. This relation equates the change in head in a pipe to the associated change in fluid velocity. However, such a relation is applicable only under restricted circumstances. When the required conditions are met, the simple relationships are often as powerful and accurate as they are easy to determine. However, most published

guidelines, such as those found in various AWWA literature, are primarily applicable to simple changes such as a sudden flow stoppage in a single pipeline. Although the initiating trigger is often assumed to be conservative in that sudden stoppage is a severe event, the degree or lack of conservatism is never evaluated. Thus the overall effect of the approach may not be conservative.

To demonstrate some of the difficulties for many of the AWWA rules summarized in the previous sections, the following discussion is intended to raise a number of warnings about where these guidelines might be misleading and thus lead to a poor basis of design. After these general concepts, specific systems are described to demonstrate some of these warnings more precisely.

**Surge pressure: how the rule breaks down.** The guidelines suggest that the pressure rise for instantaneous closure is directly proportional to the fluid velocity and is independent of conduit length; the associated initial upsurge ( $aV/g$ ) is often referred to as the potential surge. In general, it might be reasonable to use this potential surge concept in a short pipeline fed from a reservoir and controlled by a valve. However, in a long pipeline, the total

drop in hydraulic grade line over the pipe length for the initial flow may be greater than the potential surge (Wylie & Streeter, 1993). Because only part of the flow is stopped by the first compression wave and then the flow is stopped totally at the valve, an increase in stored mass continues—a phenomenon known as line packing. Thus, the pressure continues to rise, the pipe wall expands, and the

Huge amounts of capital will continue to be spent on the design of new distribution systems and the rehabilitation of existing systems in both developing and developed countries.

liquid continues to be compressed after the initial flow stoppage. More generally, the guideline relation attributes no significant role either to friction loss (which can either dissipate or accentuate the surge pressure) or the system's profile. In fact, it is often important to remember that most pumping systems move water uphill, so that the "natural" flow direction is negative; thus, after a power failure, the flow tends to reverse if a check valve is not installed to prevent this, and the potential change in velocity is often much greater than the initial velocity. This and many other circumstances can create an actual surge much larger than the so-called potential surge.

**Wave speed: how the rule breaks down.** Wave speed is a function of many fluid and pipe properties (e.g., pipe diameter, thickness, and material; pipe restraint conditions; water density, elasticity, temperature, air, and solids content). Some of these conditions can be accurately assessed, but others can be difficult or uncertain and depend on a complex set of interacting operating conditions. For this reason, an analysis sensitive to uncertainty in the value of the wave speed is an essential component of surge analysis and design work, and variations in the wave speed should be expected and accounted for. The largest estimate of wave speed may not correspond to the greatest actual surge conditions, even though it is clearly associated with the largest potential surge.

**Assumption of uniform properties: how the rule breaks down.** Simple relationships are available or applicable only for a single uniform pipeline experiencing simple events. Simple relationships do not consider wave reflections from different pipe properties, nor do they allow for the influence of friction. Thus another problem

**TABLE 1** System information for a uniform pipeline

| $a$<br>m/s (fps) | $D$<br>in. (m) | $V$<br>m/s (fps) | $aV/g$<br>m (ft) |
|------------------|----------------|------------------|------------------|
| 1,200 (3,940)    | 24 (0.61)      | 2.82 (9.25)      | 344.7 (1,131)    |
| 1,200 (3,940)    | 30 (0.762)     | 3.01 (9.88)      | 368.0 (1,207)    |
| 1,200 (3,940)    | 36 (0.914)     | 2.94 (9.65)      | 359.4 (1,179)    |
| 300 (984)        | 24 (0.61)      | 2.82 (9.25)      | 86.2 (283)       |
| 300 (984)        | 30 (0.762)     | 3.01 (9.88)      | 92.0 (302)       |
| 300 (984)        | 36 (0.914)     | 2.94 (9.65)      | 89.9 (295)       |

$a$ —wave speed,  $aV/g$ —Joukowski surge head,  $D$ —diameter,  $V$ —velocity

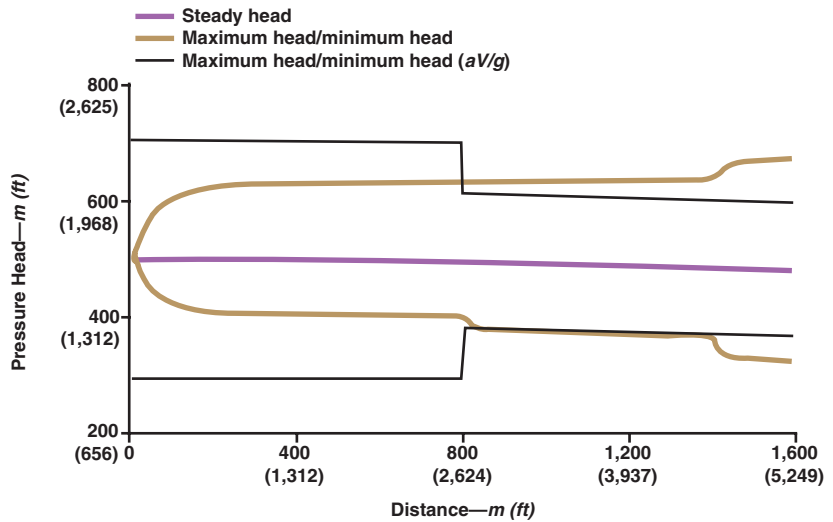
is the implicit assumption of uniform pipeline properties. If a single pipeline has different physical properties (i.e., diameter, pipe material, and wall thickness), a reflected wave will originate from each discontinuity point, producing a different, and sometimes more severe, transient response (Wylie & Streeter, 1993). If the system in question is a network of pipes, the pressure rise is strongly influenced by system topology. The pipes in a system can thus be a source of transient waves or a receiver-transmitter, and these different roles influence the nature of the assessment.

**Maximum pressure: how the rule breaks down.** For slower rates of change in simple systems, the relationships indicate the maximum pressure travels up the pipe a distance equal to  $L - (Ta/2)$  and then decreases uniformly. Only at the extreme end of the pipe are simple rules applicable. Granted, considering relatively sudden changes is an attempt to be conservative, but as is shown later, the rules are not conservative in many cases, nor are sudden changes particularly rare. Modern pumps typically have such small rotational inertia that power failures often generate essentially instantaneous changes; however, the overall system dynamics can create maximum pressures that are significantly greater than those predicted through the potential surge concept.

**Assumption of surge pressure independence from pipeline profile: how the rule breaks down.** The independence of the profile or ground contour of the line, as well as the characteristic of the hydraulic grade line, can directly influence the pressure heads that occur under surge conditions. Clearly, if the surge pressure is expressed as pressure head (as is appropriate to the stress conditions in the pipe wall), the surge pressure depends on the profile along the pipeline.

**Valve closing and maximum pressure rise: how the rule breaks down.** For more complex transients, simple rules cannot be applied even for a single uniform pipeline because the reflected waves modify the overall response.

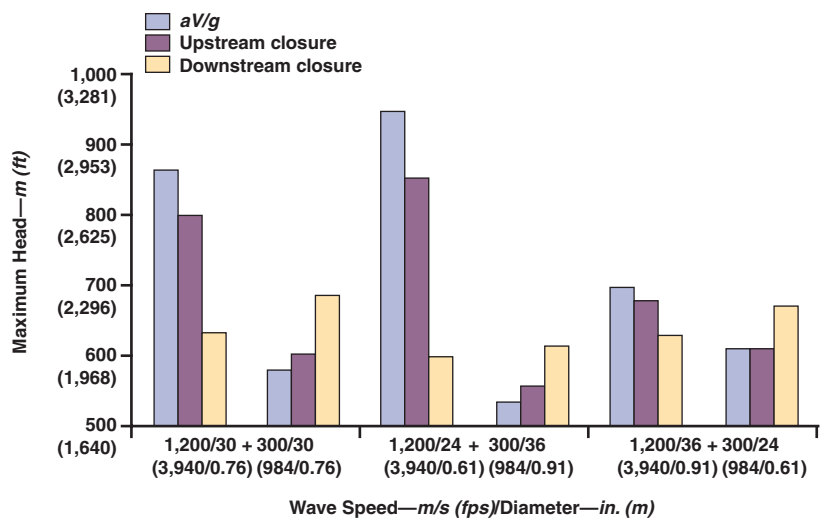
**FIGURE 6** Transient response for downstream valve closure



$a$ —wave speed in m/s,  $aV/g$ —Joukowski surge head,  $D$ —diameter in in.

Properties are  $a_1/D_1$  (1,200/36) and  $a_2/D_2$  (300/24).

**FIGURE 7** Maximum head of nonuniform pipeline

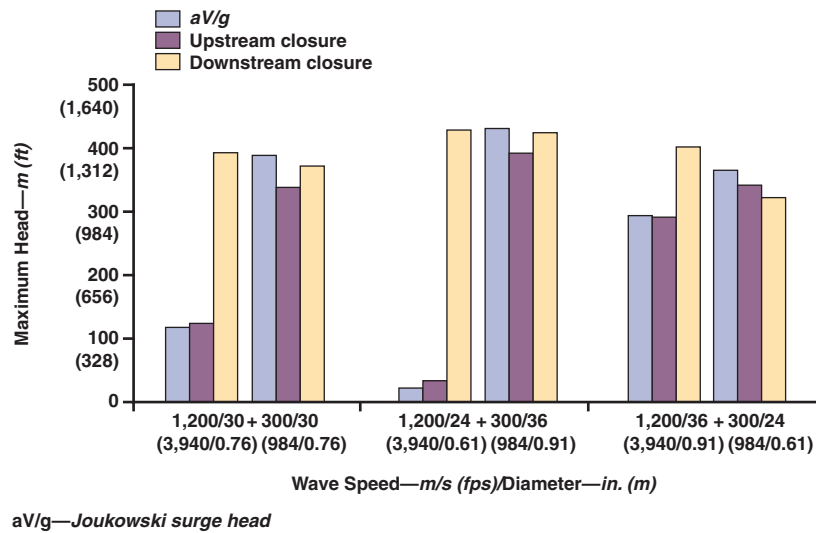


$aV/g$ —Joukowski surge head

Therefore, the calculation of the nature and influence of reflected waves should be included in the analysis and decision process.

**Pipe design and selection for pressure surges: how the rule breaks down.** Like many other simplified rules, the design for water hammer considers the rapid transient within a single uniform pipeline only. If the surge-induced operation time can be modified (i.e., to more than  $2L/a$ ), the diminished surge pressure may reduce the pipe cost.

**FIGURE 8** Minimum head of nonuniform pipeline



Because of the dangers of an imprecise water hammer analysis, a large safety factor, often set at 2.5 or higher, is sometimes used in an attempt to cover these contingencies. With the safety factor largely arbitrary, however, a high safety factor can create a twofold problem: (1) the strength might be unreasonably large, creating an unnecessarily expensive system or (2) should the factors of safety be insufficient, the pipe strength might be inadequate, leaving the system vulnerable to water hammer.

To summarize, simple rules such as those found in several AWWA publications ignore the complications of interaction of the different pipe properties in a distribution system. Actual pipes in distribution systems are necessarily connected, and water hammer waves are significantly affected by these connections. At pipe junctions and dead ends, wave reflections and refractions occur, which often magnify or attenuate the surge waves. Moreover, simplified rules cannot simulate a variety of loadings in the quest for the worst-case scenarios in a distribution

network. Any reflective practitioner must ask, “What’s at stake?” In fact, both overdesign and underdesign can put the system at risk. This risk can take the form of a risk to the pipeline and its associated hydraulic devices, a risk of water contamination, and even a risk to human life. The next section explores and illustrates these claims in more detail.

**CASE STUDIES**

The purpose of these case studies was to apply and compare comprehensive surge analyses and simplified analyses suggested by the rules published in the AWWA literature. In particular, the studies provide counter-examples to show how and when the simplified rules can break down. Comprehensive water hammer analysis is defined here as the transient analysis that can simulate

a head loss resulting from friction and wave reflection from any hydraulic devices or boundary conditions in the system. It can be produced numerically using either the method of characteristics (Wylie & Streeter, 1993) or the wave characteristic method (Boulos et al, 2006; Wood et al, 2005a; 2005b). Indeed, any of these results are reproducible using any number of commercial or in-house water hammer codes.

The case study shown in Figure 1 represents a single pipeline system. The system comprises a pipe connected to two reservoirs with a head difference of 20 m (65.6 ft). The length and Hazen-Williams roughness coefficient for the pipe are 1,600 m (5,250 ft) and 120, respectively. Three pipeline systems are considered: one with uniform properties, one with reflection points created by changes in properties, and one with an attached dead-end pipe segment. The terminal reservoirs, each having a valve with a discharge coefficient (e.g., the valve’s *C<sub>v</sub>* or *E<sub>s</sub>* value) of unity, which means that a head loss of 1 m (3.28

**TABLE 2** System information for nonuniform pipelines

| Pipe 1                |                     |                       |                       | Pipe 2                |                     |                       |                       |
|-----------------------|---------------------|-----------------------|-----------------------|-----------------------|---------------------|-----------------------|-----------------------|
| <i>a</i><br>m/s (fps) | <i>D</i><br>in. (m) | <i>V</i><br>m/s (fps) | <i>aV/g</i><br>m (ft) | <i>a</i><br>m/s (fps) | <i>D</i><br>in. (m) | <i>V</i><br>m/s (fps) | <i>aV/g</i><br>m (ft) |
| 1,200 (3,940)         | 30 (0.762)          | 3.01 (9.88)           | 368.0 (1,207)         | 300 (984)             | 30 (0.762)          | 3.01 (9.88)           | 92.0 (302)            |
| 1,200 (3,940)         | 24 (0.61)           | 3.71 (12.2)           | 453.3 (1,487)         | 300 (984)             | 36 (0.914)          | 1.65 (5.41)           | 50.5 (166)            |
| 1,200 (3,940)         | 36 (0.914)          | 1.65 (5.41)           | 201.9 (662.4)         | 300 (984)             | 24 (0.61)           | 3.71 (12.2)           | 113.3 (371.7)         |

*a*—wave speed, *aV/g*—Joukowski surge head, *D*—diameter, *V*—velocity

ft) occurs when the valve discharges 1 m<sup>3</sup>/s (35 cu ft/s; Karney & McInnis, 1992). To introduce transient conditions into this case study, a rapid valve closure (1 s) was chosen.

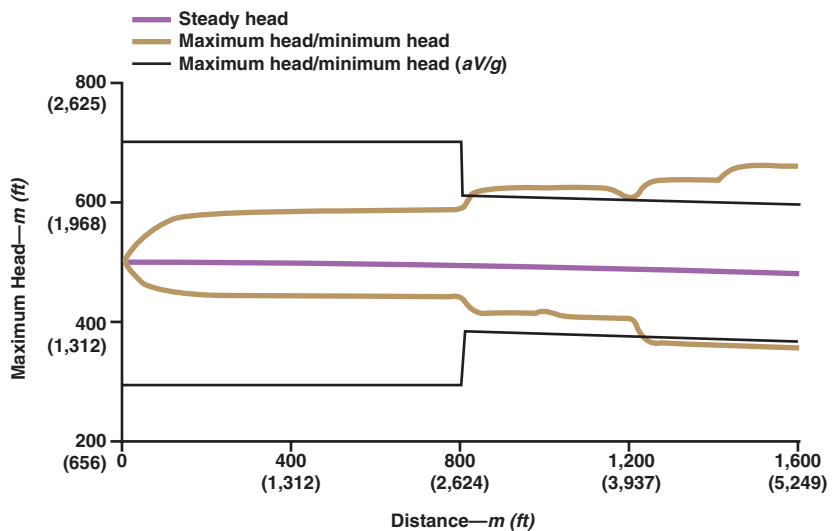
**Uniform pipeline.** A uniform single pipeline is first considered. Table 1 shows the wave speed (*a*), diameter (*D*), corresponding velocity (*V*), and Joukowski surge head (*aV/g*). Two values of wave speed—1,200 m/s (3,940 fps) and 300 m/s (984 fps)—were used to represent a rigid pipe (e.g., steel) and an elastic pipe (e.g., PVC). In addition, different pipe diameters—24 in. (0.61 m), 30 in. (0.762 m), and 36 in. (0.914 m)—were used and compared to set the stage for subsequent study into nonuniform pipeline systems.

Figure 2 shows the case of an upstream valve closure. The first downsurge initiated at the upstream valve is similar to the Joukowski downsurge except that the friction loss along the pipeline causes a slightly different slope along the pipe. However, the reflected upsurge from the downstream reservoir results in a significant head difference from the Joukowski upsurge even though the valve closure time (1 s) is fast enough to be classified as a rapid closure (i.e., the valve operation time is less than  $2L/a$ ). The reason for this difference is the dissipation of downsurge and upsurge in the downstream reservoir for 1 s. The downsurge from an upstream valve propagates to the downstream reservoir and then is converted into the corresponding upsurge. At the same time, the upsurge interacts with the remaining downsurge, causing some pressure dissipation, which is added to the frictional dissipation in the pipe.

Figures 3 and 4 depict the maximum and minimum pressure head for the uniform pipeline model shown in Table 1 for the different locations (upstream or downstream) of the valve closure. Not surprisingly, the higher wave speed systems have greater upsurge and downsurge pressures than those with lower wave

speeds. The two figures also show that the downsurge pressures attributable to the closure of the upstream valve, as already indicated in Figure 2, are similar to the Joukowski surge pressure (*aV/g*); however, the upsurge pressures consistently give a head difference of ~40 m (131 ft) regardless of wave speed and diameter. For the case of the downstream valve closure, the observed trends are exactly opposite those associated with the upstream

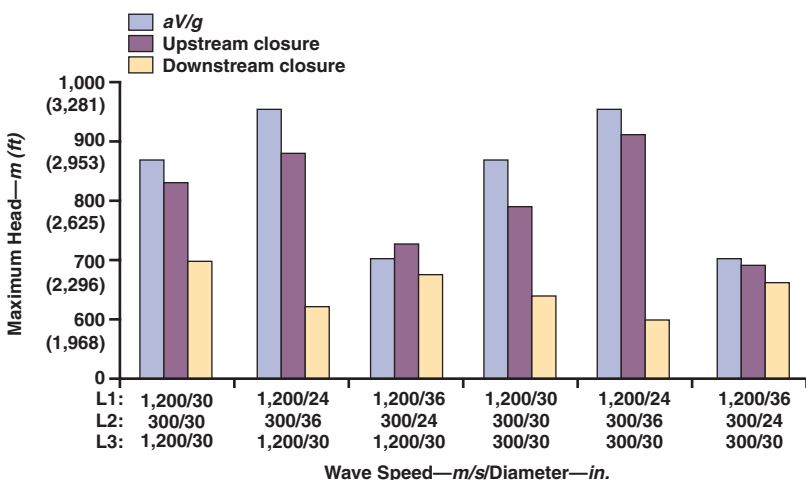
**FIGURE 9** Transient response for downstream valve closure



*a*—wave speed in m/s, *aV/g*—Joukowski surge head, *D*—diameter in in.

Properties are  $a_1/D_1$  (1,200/36),  $a_2/D_2$  (300/24), and  $a_3/D_3$  (300/30).

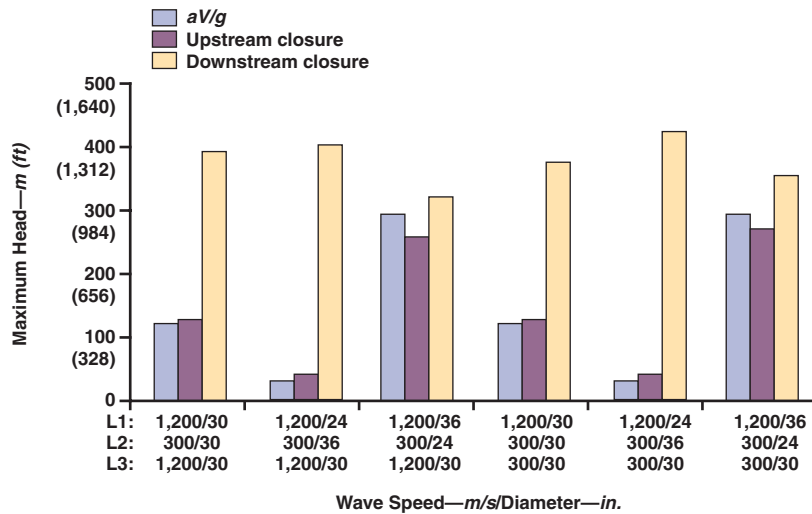
**FIGURE 10** Maximum head considering dead end



*aV/g*—Joukowski surge head



**FIGURE 11** Minimum head considering dead end



$aV/g$ —Joukowski surge head

closure. The upsurge attributable to the downstream valve closure is almost the same as the Joukowski surge pressure, but the minimum pressures are consistently ~40 m less. However, in this case overall agreement with the Joukowski relation is good. For a single pipeline subjected to a simple incident, the actual surge is well approximated by the potential surge.

**Nonuniform pipeline.** Another application encountered frequently in practice is a nonuniform series pipeline. In this study, the single pipeline consists of two pipes with the same length but with a stepwise change in diameter and/or wave speed. Table 2 shows wave speed, diameter, corresponding velocity, and Joukowski surge pressure in the two pipe sections. The first pipe has greater wave speeds, causing greater Joukowski surge pressure than anticipated in the second pipe; the smaller-diameter pipe also has the higher velocity, inducing a higher potential surge.

Figure 5 shows the transient response through a profile plot of the system for two pipes with the same diameter but different wave speeds; the first pipe has a wave speed of 1,200 m/s (3,940 fps), and the second pipe has a wave speed of 300 m/s (984 fps). The transient is initiated by closing the upstream valve. The first downsurge from the upstream valve is nearly the same as that predicted by the Joukowski relation. Yet when the wave arrives at the junction, a portion of the downsurge is transmitted downstream and some is

reflected upstream. The figure clearly shows that the minimum head in the second pipe is now much lower than its Joukowski value because of the transmitted pressure but higher than the Joukowski downsurge of the first pipe. Similarly, Figure 5 shows that the reflected positive pressure in the first pipe is much lower than the Joukowski upsurge, because of the reflection at the junction, the friction along the pipeline, and the energy dissipation at the downstream reservoir.

Figure 6 depicts the maximum and minimum pressures caused by the downstream valve closure; the wave speeds are the same as in the Figure 5 case, but the diameters of the two pipes are now 36 in. (0.91 m) and 24 in. (0.61 m), respectively. The pressure envelope for each pipe section is again significantly different from the Joukowski analysis. The initial upsurge from the downstream valve is the same as in the Joukowski analysis, but the reflected wave from the junction increases the maximum pressure. When the initial upsurge is transmitted through the junction, the wave speed increases from 300 m/s (984 fps) to 1,200 m/s (3,940 fps), causing the increase in pressure head. However, if the pipe diameter increases from 24 in. (0.61 m) to 36 in. (0.91 m), there is a resulting decrease in head. Overall, the wave reflections complicate a Joukowski-based analysis.

Figures 7 and 8 summarize the maximum and minimum pressures for a sequence of runs in the two-pipe model characterized in Table 2 with the different location

for the valve closure. Clearly the differences are much higher than those of Figures 3 and 4 because of the reflection at the junction. Another distinctly visible feature is that the location of valve closure affects the maximum and minimum pressure significantly, whereas the Joukowski analysis does not make this distinction. In the worst case, a head difference of ~400 m (1,310 ft) is shown in the minimum head (Figure 8). The overall conclusion drawn

A more recently highlighted motivation for conducting a surge analysis arises from water quality considerations.

for the valve closure. Clearly the differences are much higher than those of Figures 3 and 4 because of the reflection at the junction. Another distinctly visible feature is that the location of valve closure affects the maximum and minimum pressure significantly, whereas the Joukowski analysis does not make this distinction. In the worst case, a head difference of ~400 m (1,310 ft) is shown in the minimum head (Figure 8). The overall conclusion drawn

from this analysis is that the potential surge in complex systems is sometimes a conservative measure and at other times greatly underestimates the surge pressures.

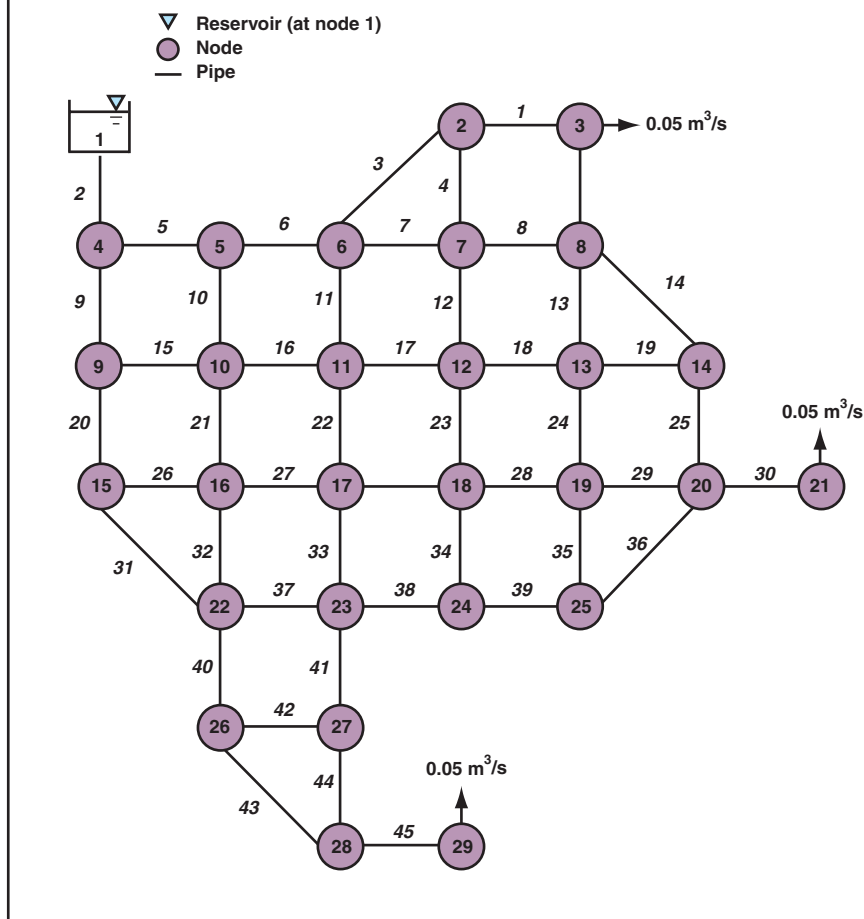
**Dead-end considerations.** An issue often ignored but sometimes crucially important is the influence of dead ends on surge pressure. Do dead ends make simple rules and published guidelines more conservative or more dangerous? On the basis of a potential surge analysis, the dead end itself would not even be expected to experience a water hammer problem; moreover, dead ends are routinely purged from steady-state simulations because they have no direct hydraulic effect on system behavior. Why then should they be of concern in a water hammer analysis?

To test this case, the hypothetical system is similar to the nonuniform case described in the second study but with a dead end attached at the middle of the second pipe (1,200 m [3,940 ft]). The length, diameter, and Hazen-Williams friction factor of the dead end are 200 m (656 ft), 30 in. (0.762 m), and 120, respectively. Two wave speeds—1,200 m/s (3,940 fps) and 300 m/s (984 fps)—are selected to consider the influence of different pipe properties. The properties of pipe 1 and pipe 2 are the same as shown in Table 2, but the dead-end pipe (pipe 3) is analyzed using different wave speeds.

Figure 9 shows the transient response of the case in Figure 6 including a dead end with a wave speed of 300 m/s (984 fps). The dead end is located at 1,200 m (3,940 ft), causing the wave reflection from that location and making the system response more complicated than that shown in Figure 6.

Figures 10 and 11 show the maximum and minimum head for the system with the different locations of valve closure. As a comparison of Figures 10 and 11 with Figures 7 and 8 shows, the dead end influences the water hammer response in dramatic and important ways. Furthermore, its different wave speeds alter the system response, especially for the case in which  $a_1/D_1 = 1,200/36$  and  $a_2/D_2 = 300/24$ . The reason the dead end can affect the maximum and minimum head significantly is that the surge pressure increase attributable to the wave speed increase conflicts with the surge pressure decrease attributable to the pipe diameter increase.

FIGURE 12 Pipe network

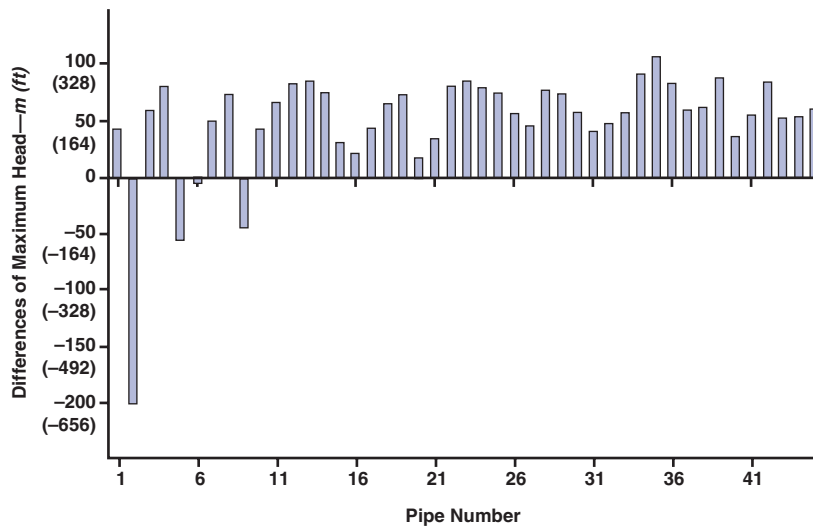


Therefore the dead end located at pipe 2 affects the maximum and minimum head more significantly than do the other system conditions. In addition, Figures 10 and 11 indicate that the dead end causes surges considerably different from Joukowski predictions, depending on the system characteristics. One of the significant conclusions of these studies is that the rules of skeletonization and simplification that often remove dead ends in steady-state analysis or replace a multidiameter pipe with an “equivalent” one having similar head loss do not apply to transient applications.

**Pipe network system.** In most distribution systems, loops are formed to ensure system reliability and flexibility. The intention of this case study was to demonstrate how the use of the simplified Joukowski analysis could lead to incorrect conclusions for the transient response in a comparatively complicated network (looped) system.

The example pipe network is shown in Figure 12. The system comprises one reservoir at node 1, 45 pipes, and 29 nodes. This is a gravity-flow system that draws water from the reservoir to supply the network. The elevation of the reservoir at node 1 is 50 m (164 ft), and all

**FIGURE 13** Difference between the maximum heads and Joukowski upsurge



other nodes have zero elevation. For simplicity, the length, diameter, wave speed, and Darcy-Weisbach friction factor of all pipes are 500 m (1,640 ft), 0.3 m (0.984 ft), 1,200 m/s (3,940 fps) and 0.015, respectively. At the end of the network, three 50-L/s (1.77-cu ft/s) demands at nodes 3, 21, and 29 are considered here. In order to introduce transient conditions into the case study, the valves at nodes 3, 21, and 29 are closed instantaneously to introduce a rapid transient into the system. Although clearly an arbitrary and somewhat dramatic transient load, the difficult question any analyst faces in practice is this: “What loading cases are appropriate and suitably severe?” Historically, little thought or reflection has been given to this important question.

Figure 13 shows the difference on the maximum heads between a detailed surge analysis and a simplified Joukowski one. Results clearly demonstrate that the Joukowski analysis results are not suitable to estimate the transient response in most pipes. In the worst case, the difference in surge pressure predictions for pipe 2 is more than 200 m (656 ft). This is because its steady-state velocity is higher than the other pipes, which causes the greater Joukowski upsurge. Another interesting and important feature of the results is that the system responses determined using the detailed surge analysis are worse than the Joukowski upsurge computed on the basis of initial pipe velocities for pipes in the middle of the network. The Joukowski upsurge and downsurge may be more severe

(and thus conservative) than results from more-detailed surge analysis in some systems; however, the opposite effect unfortunately is not rare in looped networks. In addition, the inability of the Joukowski rule to predict reasonable surge pressures is apparent if the wave speeds in pipes 1, 30, and 45 are changed to 300 m/s (984 fps) instead of 1,200 m/s (3,940 fps). These pipes are located next to the valves at nodes 3, 21, and 29, so the decreases of the wave speed eventually affect the system response in all the pipes. However, Joukowski analysis decreases the surge pressure prediction only in pipes 1, 30, and 45.

Another noticeable defect of the Joukowski rule is its inability to simulate a variety of loadings in the quest for the worst-case scenarios in a distribution network system. Logic using the Joukowski relation ignores wave reflections at the different pipe properties and the probability of conjunctive events in a distribution system, which can significantly magnify or attenuate the water hammer wave. Moreover, the Joukowski rule cannot consider liquid column separation in a pipeline. The pres-

Of the many challenges that face water utilities, one critical but too-often-forgotten issue is protecting the system from excessive transient or water hammer conditions.

sure below the vapor pressure of a liquid may produce vapor cavities in the flow, and the collapse of the cavities results in a large pressure rise, which may damage the pipeline system. Its occurrence may have a significant effect on subsequent transients in the system. Therefore, the simplified surge analysis cannot provide a reliable tool for estimating the risk of water hammer.

## CONCLUSIONS

Water hammer analysis is important, but its complexity (perhaps coupled with its mysterious nature and the need for specialized analysis tools) has led to a number of published guidelines promoting simplifications in

the analysis. AWWA literature in particular suggests simplified rules to estimate water hammer phenomena; this study showed these rules to be inaccurate in certain cases and thus likely to lead to poor designs.

Most simple expressions, such as the Joukowski relation, are applicable only under a set of highly restricted and often unrealistic circumstances. When the required conditions are met, the simple relationships are both powerful and accurate. In the case of the Joukowski relation, the two most important restrictions are that there should be only a small head loss resulting from friction and no wave reflections from any hydraulic devices or boundary conditions in the system. If these conditions are not met, the Joukowski expression is no longer valid and the conclusions based on this rule also may not be applicable. Moreover, the Joukowski relation does not consider liquid column separation. If a negative surge is below the vapor pressure, all gas within the water is gradually released, and the collapse of the cavities will result in a large pressure surge spike. Of course, the question might be raised as to whether system designers or water utilities can afford to complete a transient analysis. To this question, the authors pose another: Given the importance of this analysis and the magnitude of the errors that overly simplified rules can lead to, can utilities afford not to be comprehensive in their analyses of their distribution systems?

No simplified rules can provide a prediction of the worst-case performance under all transient conditions. The water hammer response in distribution systems is strongly sensitive to system-specific characteristics, and any careless generalization and simplification could eas-

ily lead to incorrect results and inadequate surge protection. Comprehensive water hammer analysis is not only needed, but this approach is both justified by its importance and practical, thanks to the rapid development of fast computers and both powerful and efficient numerical simulation models for water hammer analysis.

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