

Article

Metaheuristic Optimization of Surge Protection Device in an Urban Water Distribution Network

Minsung Kim, Dongwon Ko , Jeongseob Lee , Dahong Kim , Yeun Choi, Bongseog Jung , Hyunjun Kim and Sanghyun Kim * 

Department of Environmental Engineering, Pusan National University, Busan 46241, Republic of Korea; mansu823@pusan.ac.kr (M.K.); tmxk2520@pusan.ac.kr (D.K.); dufwjd5786@pusan.ac.kr (J.L.); xodon89@pusan.ac.kr (D.K.); dolong03@pusan.ac.kr (Y.C.); jungbongseog@pusan.ac.kr (B.J.); hyunjun.kim@floworklab.com (H.K.)

* Correspondence: kimsangh@pusan.ac.kr; Tel.: +82-51-510-2479

Abstract

This study investigates the optimal placement of a surge tank to mitigate pressure fluctuations induced by water hammer in a complex, real-world water distribution network (WDN). A transient-flow numerical model was developed using the Method of Characteristics (MOC) integrated with surge tank theory, applied to a simplified and skeletonized representation of the target network. To determine the most effective installation site, Particle Swarm Optimization (PSO) was employed across 32 candidate nodes. Transient events were simulated through valve closure and reopening operations at the terminal nodes of the network. The results indicate that Node 41 is the optimal location for minimizing head fluctuations. Specifically, the maximum head fluctuation was reduced from 64.32 m in the unprotected system to 51.79 m with the surge tank at Node 41, representing a 19.48% improvement in hydraulic stability. These findings emphasize the critical role of strategic surge tank positioning and provide a robust technical framework for the design and operation of surge protection systems in complex WDNs.

Keywords: method of characteristics; particle swarm optimization; surge tank; transient analysis; water hammer impact assessment

1. Introduction

Water distribution networks (WDNs) are essential infrastructure systems that demand high operational stability and reliability to ensure a consistent water supply. However, within building plumbing systems and terminal service lines, rapid valve closures or the sudden cessation of water-use appliances can trigger abrupt fluctuations in flow rates and pressure. These rapid hydraulic shifts induce water hammer—a transient pressure phenomenon. Frequent water hammer events pose a significant threat to network integrity, potentially leading to pipe fatigue, leakage, service interruptions, and escalated maintenance costs [1].

To enhance the operational stability of WDNs, extensive research has been dedicated to the design and implementation of water hammer protection devices. However, the majority of existing literature has focused on surge mitigation within single pipelines or long-distance transmission mains [2–7]. In contrast, real-world WDNs feature intricate topological structures with numerous interconnected pipes and nodes, which limits the direct transferability of findings from linear, single-pipe studies. In such complex systems,



Academic Editor: Isam Shahrour

Received: 27 April 2026

Revised: 23 June 2026

Accepted: 25 June 2026

Published: 30 June 2026

Copyright: © 2026 by the authors.

Licensee MDPI, Basel, Switzerland.

This article is an open access article distributed under the terms and conditions of the [Creative Commons Attribution \(CC BY\)](https://creativecommons.org/licenses/by/4.0/) license.

the propagation, reflection, and interference of pressure waves are highly sensitive to branch configurations and diverse boundary conditions. Consequently, the efficacy of protection devices—determined by their type, spatial allocation, and design parameters—can vary significantly even under identical transient events. A systematic investigation into these factors is therefore imperative for ensuring robust water hammer mitigation in complex network environments.

In complex WDNs, the vast number of nodes and intricate connectivity render manual comparative analyses of surge tank efficacy across multiple candidate sites both computationally prohibitive and labor-intensive. To address this challenge, this study implements a modeling framework integrated with Particle Swarm Optimization (PSO). This approach enables the efficient evaluation of head fluctuation attenuation at each candidate node to identify the optimal location for a single surge tank. Due to its streamlined algorithmic structure and robust search performance, PSO has become a well-established heuristic for solving non-linear optimization problems. Its efficacy is further supported by its successful application in previous studies concerning the design of WDNs and the optimization of transient protection devices [8–12].

This study performs a transient numerical analysis on a real-world complex water distribution network (WDN) in Y District, D Metropolitan City, to identify the optimal installation location for a surge tank. Utilizing a skeletonized representation of the target network, the numerical model incorporates a surge tank and employs PSO for a quantitative evaluation of head fluctuation mitigation. To simulate realistic transient events—such as abnormal operations or sudden demand fluctuations—transient conditions were generated by converting terminal reservoir boundary conditions into valve-operation nodes. Surge tanks were selected as the primary protection device due to their proven efficacy in regulating water levels and preventing negative pressure through targeted water supply [3,13]. By constructing random valve-operation scenarios with varying sequences and closure times, this study comprehensively evaluates the mitigation performance at each candidate node. The identified optimal location aims to enhance the hydraulic stability of the WDN, providing a methodological foundation for the future implementation of combined surge protection systems.

2. Materials and Methods

2.1. Transient Analysis

Transient flow analysis within the WDN is performed based on the continuity and momentum equations, which describe the propagation of unsteady pressure waves. The governing equations, expressed in terms of the pressure head h and the discharge Q , are defined as follows [14]:

$$L_1 = \frac{\partial h}{\partial t} + \frac{c^2}{gA} \frac{\partial Q}{\partial x} = 0 \quad (1)$$

$$L_2 = \frac{\partial Q}{\partial t} + gA \frac{\partial h}{\partial x} + \frac{f|Q|Q}{2DA} = 0 \quad (2)$$

where t and x are time and distance. In Equations (1) and (2), c denotes the wave speed, g the gravitational acceleration, D the pipe diameter, A the pipe cross-sectional area, and f the Darcy–Weisbach friction factor. Since the governing equations for transient flow are partial differential equations in time and space, a direct analytical solution is difficult to obtain. The Method of Characteristics (MOC) was employed in this study to transform them into ordinary differential equations for numerical analysis. Although the method

provides a systematic framework for transient analysis, it generally requires the use of a uniform time step.

$$L = \lambda L_1 + L_2 = \left(\frac{\partial Q}{\partial t} + \frac{c^2 \lambda}{gA} \frac{\partial Q}{\partial x} \right) + \lambda \left(\frac{\partial h}{\partial t} + \frac{gA}{\lambda} \frac{\partial h}{\partial x} \right) + R|Q|Q = 0 \quad (3)$$

$$\frac{c^2 \lambda}{gA} = \frac{dx}{dt} = \frac{gA}{\lambda} \quad (4)$$

Equation (3) represents the characteristic form obtained by combining the partial differential equations in Equations (1) and (2). In this equation, R is defined as $f/(2DA)$, where f , D and A denote the Darcy–Weisbach friction factor, pipe diameter, and pipe cross-sectional area, respectively. In addition, λ is given by $\pm gA/c$ according to Equation (4). Each term in Equation (3) can be expressed as the total differentials of $Q(x, t)$ and $H(x, t)$, allowing the governing partial differential equations to be transformed into ordinary differential equations along the characteristic lines.

$$\frac{dQ}{dt} \pm \lambda \frac{dh}{dt} + R|Q|Q = 0 \quad (5)$$

Equation (5) is the ordinary differential form of Equation (3) obtained using the MOC. Here, $\lambda = +gA/c$ denotes the characteristic line in the downstream direction, whereas $\lambda = -gA/c$ denotes the characteristic line in the upstream direction. This equation is used to calculate the head and discharge at each node on the time–space grid and serves as the fundamental equation for numerically analyzing unsteady pressure waves in the network under transient conditions. It also forms the basis for evaluating the effects of installing protection devices such as surge tanks.

2.2. Surge Tank Theory Based on Transient Flow Analysis

Under transient conditions, the surge tank model was incorporated into the existing transient flow model. The surge tank analysis was based on the conceptual diagram shown in Figure 1 [14].

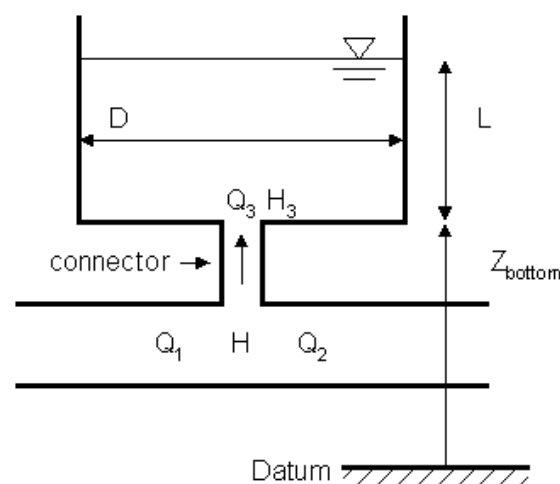


Figure 1. Schematic diagram of the surge tank. Arrows indicate the flow directions and geometric reference dimensions used in the surge-tank formulation.

As shown in Figure 1, the flow rate in the main pipeline, Q_1 , is divided into the downstream flow rate, Q_2 , and the flow rate entering the surge tank, Q_3 . In this formulation, Q_3 is defined as positive when the flow enters the surge tank from the main pipeline

through the connector pipe. Therefore, the mass conservation equation can be written as Equation (6).

$$Q_1 = Q_2 + Q_3 \quad (6)$$

For the connector pipe, a lumped inertia model incorporating inertial effects was adopted. The relationship between the pressure head in the main pipeline, H , and the head at the surge tank junction, H_3 , can then be expressed as Equation (7).

$$H - H_3 = C_1 + C_2 Q_3 \quad (7)$$

Here, C_1 and C_2 are defined by Equations (8) and (9), respectively.

$$C_1 = H_3' - H' - \frac{2L_c}{gA_c\Delta t} Q_3' \quad (8)$$

$$C_2 = \frac{2L_c}{gA_c\Delta t} + \frac{f_c L_c}{gD_c A_c^2} |Q_3'| \quad (9)$$

In Equation (8), C_1 represents a correction term associated with the head difference and flow variation from the previous time step, whereas C_2 in Equation (9) denotes a coefficient including the inertia and friction loss terms. Here, L_c , D_c , and A_c represent the length, diameter, and cross-sectional area of the connector pipe, respectively, and f_c denotes the friction factor of the connector pipe. The head at the surge tank junction is then calculated using Equation (10).

$$H_3 = H_3' + B_0(Q_3 + Q_3') \quad (10)$$

In addition, the water level inside the surge tank, L , is defined as Equation (11).

$$L = H_3 - Z_{bottom} \quad (11)$$

Here, B_0 is defined as $\Delta t / (2A_t)$ and is determined by the time step Δt and the cross-sectional area of the surge tank, A_t . Overall, by coupling the surge tank equations with the existing transient flow analysis framework, the six unknowns Q_1 , Q_2 , Q_3 , L , H , and H_3 can be solved simultaneously, thereby enabling analysis of surge tank behavior under transient conditions.

2.3. Particle Swarm Optimization (PSO)

PSO is an optimization method originally proposed for continuous nonlinear functions and derived from simulations of a simplified social behavior model [6]. Each particle searches the solution space by iteratively updating its velocity and position using information on its own best position and the global best position of the swarm. In this study, PSO was applied to a numerical framework coupling the transient flow model and the surge tank model to determine the optimal installation location of the surge tank. In PSO, the velocity and position of each particle are updated at each iteration as follows.

$$v_i^{t+1} = \left(0.5 + \frac{r_0}{2}\right) v_i^t + 2r_1(p_i^t - x_i^t) + 2r_2(g^t - x_i^t) \quad (12)$$

$$x_i^{t+1} = x_i^t + v_i^{t+1} \quad (13)$$

In Equation (12), v_i^t and x_i^t represent the velocity and position of each particle i at iteration t , respectively, while p_i^t and g^t denote the personal best position of each particle i and the global best position of the swarm. The inertia coefficient, $(0.5 + r_0/2)$, is determined by a uniformly distributed random number $r_0 \in [0, 1]$, such that the inertia term varies probabilistically within the range $[0.5, 1.0]$. This formulation allows particles to retain their

previous direction to some extent while introducing diversity into the search process at each iteration. The random numbers r_1 and r_2 are independently generated from a uniform distribution over $[0, 1]$, and both the cognitive and social acceleration coefficients were fixed at 2. Equation (13) updates the position of each particle by adding the updated velocity to the current position.

In this study, the search space was defined as one-dimensional, and the design variable was defined as the index of the candidate node for surge tank installation. A total of 32 candidate nodes were considered, and the number of particles and the maximum number of iterations were set to 20 and 20, respectively. To ensure reproducibility of the stochastic components in the optimization process, the random seed was fixed at -1000 . At each iteration, all particles performed MOC-based transient analysis under the assumption that a surge tank was installed at the candidate node corresponding to the current particle position, which was rounded to the nearest candidate node index. The objective function was defined based on the maximum head fluctuation over the simulation period for all evaluation nodes. For each candidate surge-tank installation node s , the head fluctuation at evaluation node n was defined as follows:

$$\Delta h_n(s) = h_n^{\max}(s) - h_n^{\min}(s) \quad (14)$$

In Equation (14), $h_n^{\max}(s)$ and $h_n^{\min}(s)$ are the maximum and minimum heads at evaluation node n , respectively, when the surge tank is installed at candidate node s . The fitness value for candidate node s was then defined as the largest head fluctuation among all evaluation nodes:

$$F(s) = \max_{n \in N} [\Delta h_n(s)] \quad (15)$$

In Equation (15), N is the set of evaluation nodes. Finally, the optimal surge-tank installation node was determined as follows:

$$s^* = \arg \min_{s \in C} F(s) \quad (16)$$

In Equation (16), s^* denotes the optimal surge-tank installation node, and C is the set of candidate surge-tank installation nodes.

2.4. Random Valve-Operation Transient Scenarios

To account for the diverse abnormal operating conditions inherent in real-world WDNs, stochastic transient scenarios were developed based on randomized valve operations. Rather than imposing a predefined initial event, all valve-closing sequences were determined by a consistent stochastic rule. For each event, a uniformly distributed random number $[0, 1]$ was generated and divided into 13 equal-probability intervals to select the target valve for closure. To prevent the disproportionate activation of specific nodes, each valve was restricted to a maximum of two closures per scenario. Following each closure, the valve was programmed to reopen after a fixed duration of 1.337 s. This duration was not intended to represent an instantaneous or abrupt valve closure, but was adopted as a representative average valve-operation duration for the transient simulations. To ensure the reproducibility of these simulations, the random seed was fixed, allowing for a rigorous comparative analysis of surge tank performance under identical transient conditions.

2.5. Study Area and Simulation Conditions

The target WDNs are illustrated in Figure 2. Figure 2a displays the QGIS-based spatial data of the actual network in Y District, D Metropolitan City, which comprises 8605 pipes

and 8405 nodes. To facilitate transient analysis, this extensive network was simplified and skeletonized, as shown in Figure 2b.



Figure 2. QGIS-based maps of the water distribution network in Y District, D Metropolitan City: (a) original network; (b) skeletonized network. Colored dots and number labels indicate network nodes and node IDs, respectively.

The reconstructed model consists of 47 pipes and 46 nodes, focusing on major transmission pipelines with diameters of 150 mm or greater. During the skeletonization process, the actual pipe lengths and primary flow paths were strictly preserved, while terminal service lines and secondary branch pipes (diameters < 150 mm) were removed to enhance computational efficiency without compromising hydraulic integrity. Furthermore, reservoir boundary conditions were initially assigned to the terminal nodes to define the network's hydraulic limits; these were subsequently reconfigured into valve boundary conditions to induce the transient events required for the study.

The nodes within the simplified network were categorized based on their functional characteristics. Specifically, all 13 nodes initially designated as reservoir boundary conditions were reconfigured into valve boundary conditions. This reconfiguration was strategically implemented to simulate potential water hammer events occurring at various terminal points throughout the actual network. Consequently, these nodes served as the primary locations for valve-induced transients, allowing for a comprehensive evaluation of the system's hydraulic response to operational fluctuations.

For temporal discretization of the transient flow model, the Courant number was defined as $a\Delta t/\Delta x$. A wave speed a of 1175 m/s and a time step Δt of 0.00426 s were adopted, and the pipelines were discretized at spatial intervals of 8–10 m to satisfy a Courant number of 0.5. Surge tanks were installed at 32 of the 46 nodes, excluding reservoir and valve boundary nodes, and their performance was evaluated.

In this study, the surge tank was integrated into the system via a connector pipe. To isolate and focus specifically on the impact of the installation location, the design parameters of the connector pipe and the surge tank were maintained as constant values rather than optimization variables. The surge tank diameter was fixed at 0.2 m based on a preliminary diameter-sensitivity analysis conducted for the same skeletonized network. In the preliminary analysis, several surge tank diameters were compared under selected transient scenarios, and 0.2 m showed an effective reduction in head fluctuation. In addition, preliminary tests showed that changes in the connecting pipe diameter and length had only a limited influence on the head fluctuation and did not significantly affect the optimal installation location. Therefore, in the present study, the surge tank diameter and connecting pipe parameters were fixed to focus on installation-location optimization. Based on the

minimum pipe diameter within the target network, the connector pipe's diameter and length were set to 0.075 m and 0.5 m, respectively. As illustrated in Figure 1, the initial water level in the surge tank, z , was assigned to match the steady-state hydraulic head at the respective installation node. Additionally, the friction factor for the connector pipe was defined as 0.0362 to account for minor losses during transient transitions. No additional orifice or damping device was considered in the present surge tank model.

3. Results and Discussion

3.1. Transient Analysis in a Complex Water Distribution Network

Prior to evaluating the mitigation efficacy of the surge tank, the baseline water hammer characteristics under the no-surge-tank condition were analyzed. To establish this baseline, the reservoir boundary nodes were reconfigured into valve nodes. Transient events were then simulated by independently applying scenarios in which each valve was closed and subsequently reopened using a fixed valve-operation duration of 1.337 s. This duration was applied consistently to all valve-operation events according to the predefined simulation protocol. The pressure wave speed (a) within the network was defined as 1175 m/s. This value represents the arithmetic mean of the theoretical wave speeds (ranging from 1054 to 1326 m/s) calculated based on the elasticity of water and the pipe material, as well as the specific diameter and wall thickness of the pipelines.

Table 1 summarizes the reservoir nodes designated for transient generation, including their respective flow rates in the pipe and hydraulic head values. These parameters represent the steady-state initial conditions derived from a prolonged simulation (200 s) under static boundary conditions to ensure hydraulic stability prior to the water hammer analysis. All subsequent transient scenarios were initialized based on these stabilized values to ensure consistency and numerical convergence.

Table 1. Transient-generation nodes.

Transient in Reservoir	Flow in the Pipe (m ³ /s)	Head (m)
Transient in reservoir 11	0.00331	78.09
Transient in reservoir 18	0.00087	80.62
Transient in reservoir 20	0.00378	80.61
Transient in reservoir 22	0.00064	80.60
Transient in reservoir 24	0.00063	80.60
Transient in reservoir 25	0.00056	80.60
Transient in reservoir 28	0.01325	78.60
Transient in reservoir 30	0.01038	78.42
Transient in reservoir 32	0.00578	78.44
Transient in reservoir 38	0.00828	76.23
Transient in reservoir 42	0.00562	76.22
Transient in reservoir 44	0.00168	78.12
Transient in reservoir 46	0.00392	78.11

Figure 3 illustrates the hydraulic response following the conversion of the 13 reservoir boundary conditions into valve nodes, specifically highlighting the maximum, minimum, and steady-state heads at each node. Figure 3a,b display the maximum and minimum head distributions, respectively, with the red line denoting the steady-state head at each location. The analysis reveals that the most significant head fluctuations occurred during transient events at Nodes 38 and 42, with the most pronounced difference between maximum and minimum heads observed at Node 43. When a transient was induced at Node 38, the steady-state head at Node 43 (76.5 m) escalated to a maximum of 113.48 m and dropped to a minimum of 49.16 m, resulting in a total fluctuation range of 64.32 m. Similarly, a

transient event at Node 42 caused the head at Node 43 to fluctuate between 110.79 m and 60.22 m, representing a range of 50.57 m. Table 2 summarizes the six nodes experiencing the highest magnitudes of head fluctuations for each simulated scenario.

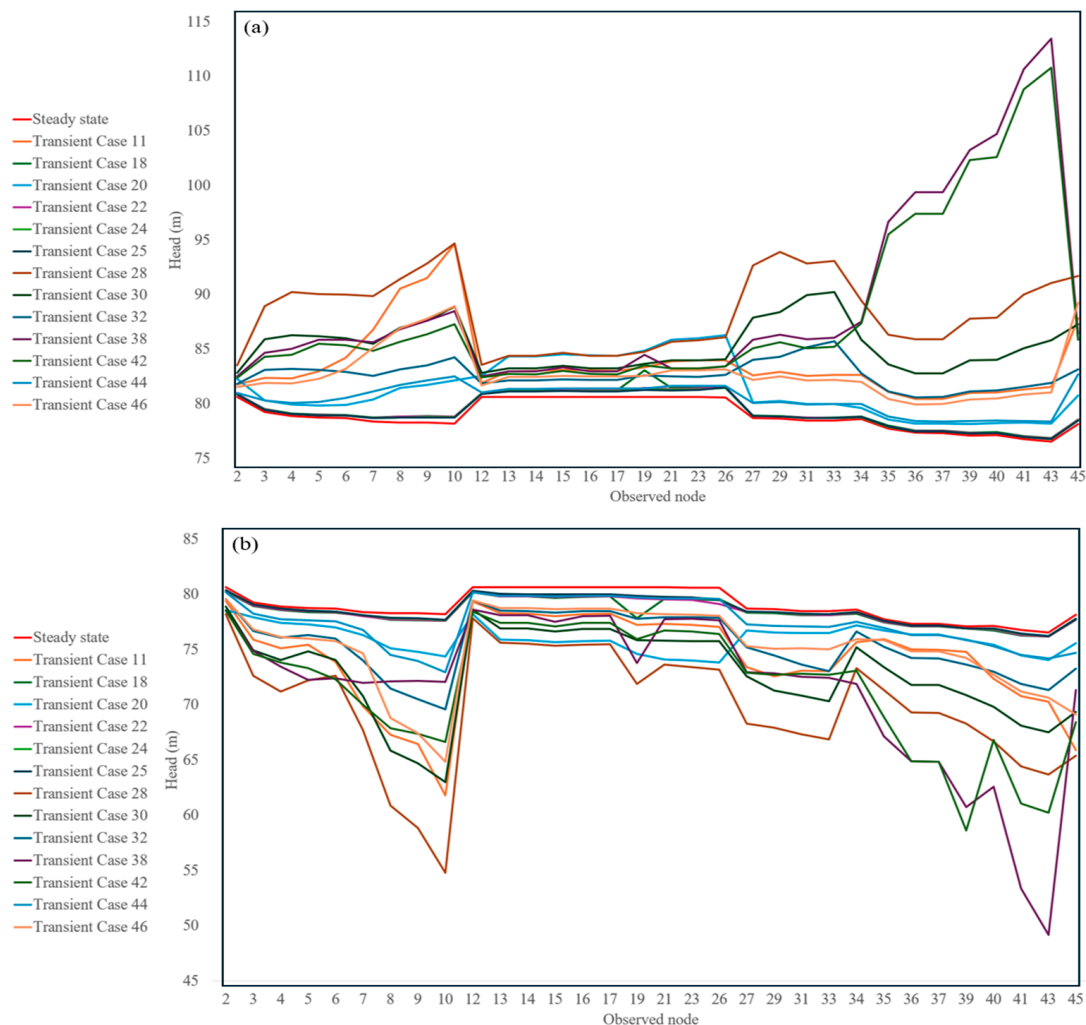


Figure 3. Node-wise maximum, minimum, and steady-state heads under individual valve operation transient scenarios: (a) maximum head; (b) minimum head.

Table 2. Top six nodes with the largest head fluctuations under individual valve operation transient scenarios.

Observed Node	Steady (m)	Max Head (m)	Min Head (m)	Difference (m)
Node 10	78.20	94.64	61.76	32.87
Node 10	78.20	94.68	54.80	39.88
Node 33	78.47	90.23	70.29	19.94
Node 43	76.56	113.48	49.16	64.32
Node 43	76.56	110.79	60.22	50.57
Node 45	78.11	89.28	69.13	20.15

As summarized in Table 2, the six transient scenarios exhibiting the most significant head fluctuations were identified from the 13 initial cases. In conventional pipeline systems, the maximum pressure rise typically occurs at or near the valve location immediately following a valve-closure event [15]. However, the results from this complex network demonstrate a different behavior: the propagation, reflection, and superposition of pres-

sure waves are highly influenced by the network's intricate topology, including multiple branches, reservoirs, and junctions.

Consequently, as evidenced by the simulation results, maximum head fluctuations frequently manifest at nodes distant from the transient-generation source rather than at adjacent points. The abrupt and severe head fluctuations detailed in Table 2 simultaneously subject the system to extremely high-pressure loads and precarious low-pressure conditions. Such hydraulic instability poses significant risks to infrastructure integrity, potentially leading to pipe bursts, joint leakage, structural damage to supports, and intensified levels of vibration and noise [16–18].

Furthermore, severe low-pressure or negative-pressure transients pose a significant risk of drawing external contaminants into the pipeline via cracks or deteriorated joints, thereby severely compromising water quality and overall system reliability [19]. Conversely, the hydraulic response across the network proved highly heterogeneous; while certain regions experienced drastic pressure variations, others exhibited only marginal fluctuations. For instance, an induced transient at Node 24 resulted in a minimal fluctuation at Node 26—shifting from a steady-state head of 80.6 m to a maximum of 81.44 m and a minimum of 79.56 m, representing a range of just 1.88 m. Although such low-amplitude fluctuations might appear benign compared to extreme surges, the cyclic loading from repeated minor transients accelerates fatigue damage. Over time, this cyclic stress exacerbates existing micro-defects or corrosion pits, ultimately leading to unexpected leaks. Moreover, due to complex wave reflection and superposition, even small-amplitude pressure waves can unpredictably amplify localized stress, compounding the risk of structural failure [20–23]. Consequently, mitigating head fluctuations of all magnitudes is imperative for preserving the long-term structural integrity of the network, robustly validating the necessity of strategically placed surge protection devices.

3.2. Comparative Evaluation of Candidate Surge Tank Locations

Randomized transient scenarios were generated based on the predefined valve operation conditions, followed by a PSO-based search to identify the optimal spatial allocation of the surge tank. The total simulation duration was established at 780 s, a timeframe selected to accommodate the sequential closure of 13 valves at an average interval of 30 s, with each valve constrained to a maximum of two operations.

For the PSO algorithm, both the swarm size and the maximum number of iterations were set to 20, resulting in a total of 400 transient simulations. To examine the effect of computational budget, preliminary sensitivity tests were conducted by increasing the number of simulations beyond the 20×20 configuration. The results showed that the top-ranked candidate group remained stable and that the objective-function values showed only limited differences, whereas the computational cost increased. Therefore, the 20×20 configuration was adopted as a practical setting for identifying major candidate locations while maintaining computational efficiency.

Furthermore, to rigorously compare the mitigation performance across the top candidate locations under uniform hydraulic stress, a specific representative scenario was isolated. The scenario exhibiting the highest frequency of valve-closing events (26 discrete events) was selected. This scenario was selected as a conservative high-disturbance condition because it produced the maximum number of valve-closure events within the same simulation duration. If a randomly generated scenario includes only a small number of valve closures or mainly involves valves that induce weak transient responses, the resulting head fluctuations may remain relatively small and may not sufficiently represent the hydraulically vulnerable conditions of the network. The longer the simulation period, the greater the demand for computational cost in conjunction with the metaheuristic approach.

We assumed that two valve maneuvers for each transient location sufficiently address the worst possible transient generations during 780 s. Therefore, the 26-event scenario was used to compare candidate surge-tank locations under the most severe valve-operation condition considered in this study. The magnitude of head fluctuations at each candidate node was subsequently evaluated and compared under this maximized transient loading condition.

Table 3 summarizes the top-ranked optimization results derived from the most severe transient scenario, which involved the maximum frequency of valve-closing events (26 events). Specifically, Table 3a ranks the best computational solutions based on the minimized head fluctuation differences, while Table 3b presents the final hierarchy of candidate surge tank locations by excluding duplicate nodal outputs. An analysis of the 400 simulations revealed that installing a single surge tank at Node 41 was the most effective strategy for mitigating head fluctuations across the target network. The subsequent optimal locations were identified as Nodes 40, 39, and 9, respectively. To rigorously validate these findings, the mitigation performances of the top four candidate nodes were re-evaluated under the same transient-loading scenario, while varying only the surge-tank installation location. The transient-loading scenario used for this comparison corresponds to the selected 26-event valve-operation scenario under which Node 41 was identified as the most effective installation location. The detailed composition of this scenario is summarized in Table 4, including the valve node, operation sequence, closure time, and reopening time for each event.

Table 3. Top 4 surge tank locations under the selected transient scenario. (a) Top 4 solutions including duplicates. (b) Top 4 candidate surge tank locations excluding duplicates.

(a)		
Surge Node	Node of Maximum Head Difference	Max-Min Head Difference (m)
Node 41	Node 37	51.78635
Node 41	Node 37	51.79692
Node 41	Node 37	51.79885
Node 41	Node 37	51.80103
(b)		
Surge Node	Node of Maximum Head Difference	Max-Min Head Difference (m)
Node 41	Node 37	51.78635
Node 40	Node 37	54.29550
Node 39	Node 41	62.6857
Node 9	Node 41	63.7505

To validate the optimization results, Figure 4 evaluates the mitigation performance of the top four candidate locations (Nodes 41, 40, 39, and 9) under the representative severe transient scenario. As depicted in Figure 4a–d, solely varying the installation site confirms that the relative ranking of these nodes remains highly consistent with the preliminary findings in Table 3. The peak-to-peak head differences showed negligible deviation; for example, locating the surge tank at Node 40 yielded a head fluctuation of 54.28 m at Node 37, nearly identical to the 54.30 m reported in Table 3b. Crucially, Node 41 maintained its superior mitigation capacity with minimal performance variation, despite the shift to the extreme 26-event scenario. This robustly confirms Node 41 as the definitive optimal installation point. The detailed maximum head fluctuation ranges for each tested location under this representative scenario are quantitatively summarized in Table 4.

Table 4. Detailed composition of the selected 26-event valve-operation scenario.

Event No.	Valve Node	Closed Time (s)	Opened Time (s)	Event No.	Valve Node	Closing Time (s)	Opened Time (s)
1	42	1.337	2.674	14	28	350.844	352.181
2	30	4.733	6.070	15	32	364.976	366.313
3	42	16.283	17.620	16	25	417.826	419.163
4	46	46.223	47.560	17	32	420.872	422.209
5	38	75.260	76.597	18	24	440.618	441.955
6	20	88.140	89.477	19	46	485.404	486.741
7	28	114.416	115.753	20	20	533.720	535.057
8	38	158.420	159.757	21	11	576.887	578.224
9	44	207.465	208.802	22	11	620.021	621.358
10	18	217.924	219.261	23	25	648.841	650.178
11	44	239.168	240.505	24	30	664.650	665.987
12	18	298.383	299.720	25	22	674.620	675.957
13	24	332.562	333.899	26	22	728.053	729.390

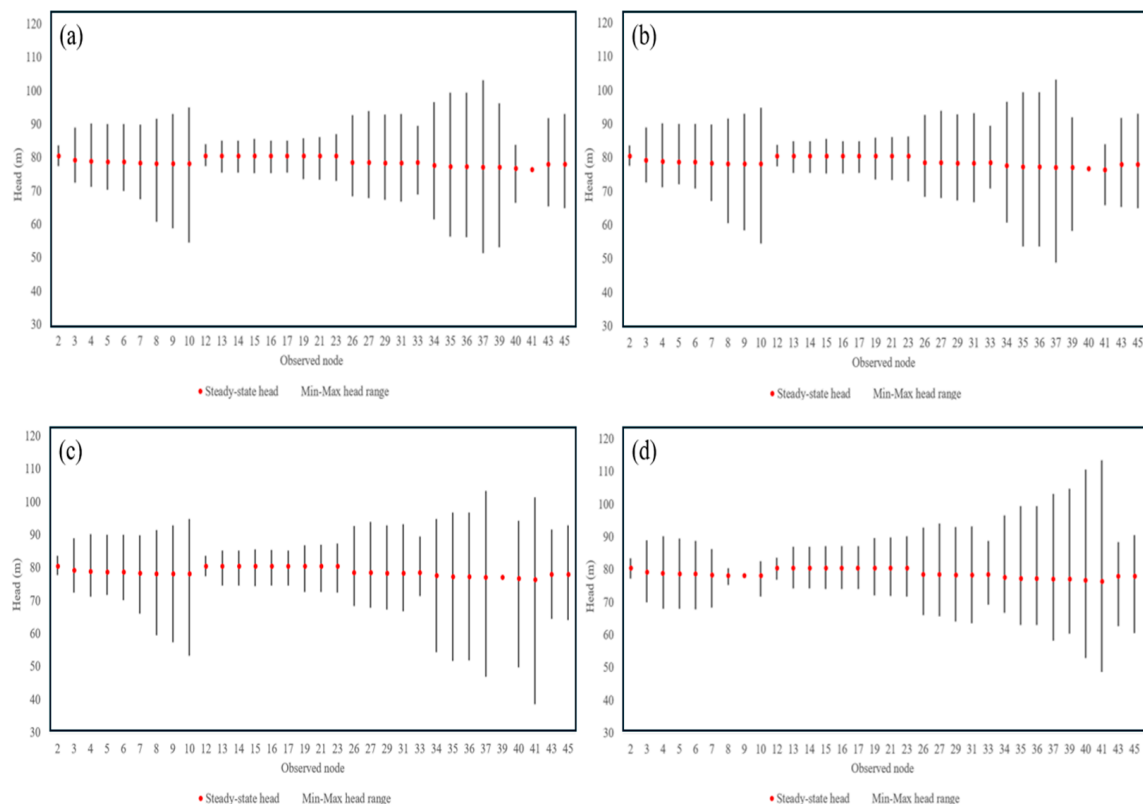


Figure 4. Comparison of steady-state heads and min–max head ranges at the top 4 surge tank locations under the representative transient scenario: (a) Node 41, (b) Node 40, (c) Node 39, and (d) Node 9.

Table 5 quantifies the peak-to-peak head variations for the top-tier candidate locations evaluated under the representative severe transient scenario. The results establish Node 41 as the definitive optimal site for a single surge tank, consistently providing superior and stable mitigation regardless of the specific valve-closure sequence. Node 40 ranks as a strong secondary alternative, exhibiting a mitigation profile nearly identical to Node 41, with only a 2.5 m variance in the peak-to-peak head difference.

Table 5. Top 4 surge tank locations under the representative transient scenario.

Rank	Surge Node	Node of Maximum Head Difference	Max-Min Head Difference (m)
1	Node 41	Node 37	51.79
2	Node 40	Node 37	54.28
3	Node 39	Node 41	62.68
4	Node 9	Node 41	64.75

In contrast, positioning the surge tank at Nodes 39 or 9 provided markedly inferior protection. Under these configurations, critical head fluctuations still manifested; for example, relative to a 76.4 m steady-state baseline, the head fluctuated between 38.64 m and 101.32 m for the Node 39 setup, and between 48.72 m and 110.67 m for the Node 9 setup. Based on these findings, Nodes 39 and 9 are relegated to low-priority status due to their constrained hydraulic dampening capabilities. Therefore, for securing network stability with a single surge tank, Node 41 is conclusively identified as the optimal installation point, closely followed by Node 40.

3.3. Mitigation Effect of the Surge Tank at the Optimal Location

Having established Node 41 as the optimal installation site for a single surge tank, the overall mitigation efficacy was quantitatively evaluated. This was achieved by directly comparing the hydraulic response of the unprotected baseline system (no surge tank) against the optimized configuration (surge tank installed at Node 41). Employing the identical representative valve-closure scenario utilized in the preceding analyses, the resultant head fluctuations for both conditions are comparatively illustrated in Figure 5.

Figure 5 illustrates the comparative head fluctuations between the unprotected baseline and the optimized configuration (surge tank at Node 41) under the identical multi-valve closure scenario. Figure 5a depicts the baseline condition, while Figure 5b shows the hydraulic response with the surge tank in operation; red markers in both panels denote the steady-state heads.

As shown in Figure 5a, without surge protection, the most severe head fluctuation manifested at Node 41. Deviating from a steady-state head of 76.74 m, the pressure surged to a maximum of 113.48 m and plummeted to a minimum of 49.16 m, yielding a peak-to-peak differential of 64.32 m. Notably, while Node 43 experienced the largest fluctuation during individual valve closures (as previously shown in Figure 3), the critical node shifted to Node 41 under the repeated, multi-valve operations of this complex scenario. This spatial shift highlights the intricate propagation, reflection, and superposition of pressure waves across the network topology, which significantly alter transient behaviors compared to isolated events [24–26].

Conversely, Figure 5b demonstrates that installing the surge tank at Node 41 substantially reduced the head fluctuations at the major vulnerable nodes located near the installation point. In particular, the head fluctuation at Node 41 decreased from 64.32 m to 0.49 m, indicating a strong local mitigation effect of the surge tank. However, this mitigation effect was not uniformly distributed throughout the network. At Node 37, the head fluctuation was not reduced but increased from 45.22 m without the surge tank to 51.78 m after surge tank installation. This quantitative comparison indicates that the surge tank effectively attenuated the transient response near the installation node, especially at Nodes 41 and 40, whereas its mitigation effect was relatively limited at Node 37. Therefore, the shift in the maximum-fluctuation node from Node 41 to Node 37 can be reasonably attributed to the localized attenuation effect of the surge tank and the remaining transient response at Node 37, resulting from the redistribution of pressure-wave transmission,

reflection, and superposition patterns within the network. Consequently, the maximum system-wide fluctuation after surge tank installation occurred at Node 37, where the head varied between 103.21 m and 51.43 m against a steady-state head of 77.10 m, corresponding to a fluctuation range of 51.78 m. Finally, Table 6 provides a comprehensive comparison of the hydraulic heads at major vulnerable nodes before and after the surge tank installation.

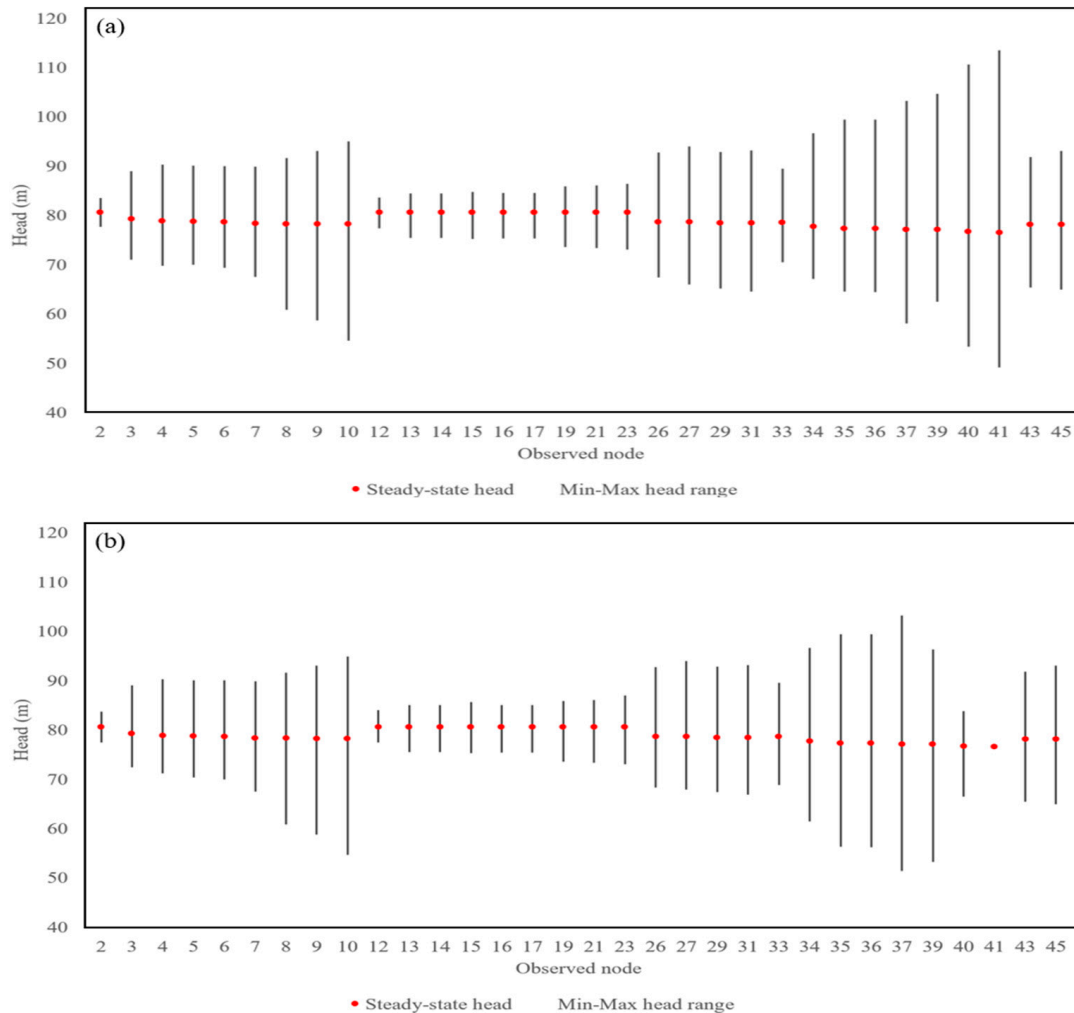


Figure 5. Comparison of steady-state heads and max-min head ranges under the selected valve-closing transient scenario: (a) no surge tank condition and (b) surge tank installed at Node 41.

Table 6. Comparison of the top 4 most vulnerable nodes before and after surge tank installation.

Without Surge Tank		With Surge Tank at Node 41	
Observed Node	Max–Min Head Difference (m)	Observed Node	Max–Min Head Difference (m)
Node 41	64.32	Node 41	0.49
Node 40	57.32	Node 40	17.43
Node 37	45.22	Node 37	51.79
Node 39	42.17	Node 39	43.13

Table 6 summarizes the shifts in peak-to-peak head variations for the four most vulnerable nodes identified in the unprotected baseline condition, comparing their responses before and after the surge tank installation at Node 41. Detailed examination reveals highly localized mitigation effects. At Node 41, the installation yielded the most profound

attenuation, with the head differential plummeting from 64.32 m to a mere 0.49 m. Node 40 similarly experienced a substantial reduction, dropping from 57.32 m to 17.43 m.

Conversely, the installation induced localized amplification at other locations. Most notably, the head differential at Node 37 increased from 45.22 m to 51.79 m, superseding Node 41 as the new critical point of maximum head fluctuation. Node 39 also exhibited a marginal increase from 42.17 m to 43.13 m. This amplification can be attributed to the altered transient boundary conditions and the dynamic mass oscillation between the surge tank and the main pipeline. The introduction of the surge tank fundamentally modifies the propagation, reflection, and superposition patterns of the pressure waves, thereby intensifying wave interactions at specific locations such as Node 37 [25–27]. Ultimately, these findings emphasize that a single surge tank does not uniformly dampen hydraulic transients across a complex network; rather, it redistributes the nodal response patterns by reshaping the system's overall wave dynamics [25–27].

4. Conclusions

This study proposed a comprehensive numerical analysis framework integrating transient-flow simulation and PSO to determine the optimal installation location for a single surge tank within an actual complex WDN (Y District, D Metropolitan City). Diverging from conventional water hammer studies—which predominantly focus on isolated pump stations or single transmission pipelines—this research investigated transient phenomena induced at multiple locations across a complex network topology. Specifically, the optimization rigorously accounted for the intricate propagation, reflection, and superposition characteristics of pressure waves inherent to such structurally complex systems.

By systematically evaluating the mitigation performance across 32 candidate nodes, the results conclusively established Node 41 as the optimal installation site, providing the most effective attenuation of head fluctuations system-wide. Node 40 emerged as a highly viable secondary candidate, exhibiting a comparable mitigation capacity with only marginal differences in hydraulic response. Conversely, while Nodes 39 and 9 were initially identified as potential sites, their relatively limited dampening effects relegated them to lower-priority options.

Crucially, the comparative analysis revealed that installing the surge tank at Node 41 substantially diminished overall head fluctuations but shifted the critical point of maximum fluctuation from Node 41 to Node 37. This phenomenon demonstrates that the introduction of a surge protection device does not uniformly suppress hydraulic transients across the entire network; rather, it fundamentally redistributes the nodal response patterns by altering the transmission and interaction behaviors of the pressure waves. Ultimately, these findings underscore the critical necessity of strategically optimizing surge tank locations for effective system-wide protection. This study provides significant practical value by establishing a quantitative and robust methodology for securing actual complex WDNs against unpredictable hydraulic transients. Future studies should explore the optimal sizing of multiple surge tanks simultaneously or incorporate varying demand conditions to further enhance the resilience of water distribution systems.

Author Contributions: Conceptualization, M.K. and S.K.; methodology, B.J.; validation, B.J. and S.K.; formal analysis, M.K. and S.K.; investigation, M.K.; data curation, D.K. (Dongwon Ko), J.L. and H.K.; writing—original draft preparation, M.K.; writing—review and editing, S.K.; visualization, D.K. (Dongwon Ko), J.L., D.K. (Dahong Kim) and Y.C.; supervision, S.K. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Data Availability Statement: No new data were created or analyzed in this study. Data sharing is not applicable to this article.

Acknowledgments: The authors would like to express their appreciation to the reviewers and editors.

Conflicts of Interest: The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

References

1. Firouzi, A.; Yang, W.; Shi, W.; Li, C.-Q. Failure of Corrosion Affected Buried Cast Iron Pipes Subject to Water Hammer. *Eng. Fail. Anal.* **2021**, *120*, 104993. [CrossRef]
2. Stephenson, D. Simple Guide for Design of Air Vessels for Water Hammer Protection of Pumping Lines. *J. Hydraul. Eng.* **2002**, *128*, 792–797. [CrossRef]
3. Wan, W.; Zhang, B. Investigation of Water Hammer Protection in Water Supply Pipeline Systems Using an Intelligent Self-Controlled Surge Tank. *Energies* **2018**, *11*, 1450. [CrossRef]
4. Li, X.; Wang, T.; Zhang, Y.; Guo, P. Study on the Factors Influencing Air Valve Protection against Water Hammer with Column Separation and Rejoinder. *AQUA—Water Infrastruct. Ecosyst. Soc.* **2022**, *71*, 949–962. [CrossRef]
5. Shi, L.; Zhang, J.; Yu, X.-d.; Chen, S.; Zhao, W.-l.; Chen, X.-y. Water Hammer Protection for Diversion Systems in Front of Pumps in Long-Distance Water Supply Projects. *Water Sci. Eng.* **2023**, *16*, 211–218. [CrossRef]
6. Ahmed, H.M.; Imam, Y.E.; El-Ghandour, H.A.; Elansary, A.S. Identifying Worst Transient Cases and Optimizing Surge Protection for Existing Water Networks. *Water* **2025**, *17*, 1816. [CrossRef]
7. Zhang, X.; Wang, C.; Li, L.; Bian, S. Protection and Parameter Optimization of One-Way Surge Tower for Water Conveyance Pipelines under Complex Terrain Conditions. *Sci. Rep.* **2025**, *15*, 20498. [CrossRef] [PubMed]
8. Kennedy, J.; Eberhart, R. Max Swarm Optimization. In Proceedings of the IEEE International Conference on Neural Networks (ICNN'95), Perth, Australia, 27 November–1 December 1995; Volume 4, pp. 1942–1948.
9. Jung, B.S.; Karney, B.W. Hydraulic Optimization of Transient Protection Devices Using GA and PSO Approaches. *J. Water Resour. Plan. Manag.* **2006**, *132*, 44–52. [CrossRef]
10. Montalvo, I.; Izquierdo, J.; Pérez, R.; Tung, M.M. Particle Swarm Optimization Applied to the Design of Water Supply Systems. *Comput. Math. Appl.* **2008**, *56*, 769–776. [CrossRef]
11. Freitas, D.; Lopes, L.G.; Morgado-Dias, F. Particle Swarm Optimisation: A Historical Review up to the Current Developments. *Entropy* **2020**, *22*, 362. [CrossRef] [PubMed]
12. Li, H.; Zhang, Y.; Cui, J.; Zhang, F.; Yang, X.; Shi, W.; Mao, L. Optimization of Water Hammer Protection against Sudden Pump Stoppage Using Machine Learning Models and Intelligent Algorithms. *J. Hydraul. Res.* **2025**, *63*, 265–277. [CrossRef]
13. Zhang, X.; Guo, X.; Chen, Y.; Yang, C.; Liu, S.; Guo, L. Research on the Characteristics and Protection of Water Hammer in Long-Distance Dual-Pipe Water Supply Systems. *PLoS ONE* **2024**, *19*, e0314998. [PubMed]
14. Wylie, E.B.; Streeter, V.L. *Fluid Transients in Systems*; Prentice Hall: Englewood Cliffs, NJ, USA, 1993.
15. Karney, B.W.; Ruus, E. Charts for Water Hammer in Pipelines Resulting from Valve Closure from Full Opening Only. *Can. J. Civ. Eng.* **1985**, *12*, 241–264. [CrossRef]
16. Bergant, A.; van't Westende, J.M.; Koppel, T.; Gale, J.; Hou, Q.; Pandula, Z.; Tijsseling, A.S. Water Hammer and Column Separation due to Accidental Simultaneous Closure of Control Valves in a Large-Scale Two-Phase Flow Experimental Test Rig. In Proceedings of the ASME 2010 Pressure Vessels and Piping Division/K-PVP Conference, Bellevue, WA, USA, 18–22 July 2010; pp. 681–688.
17. Pozos-Estrada, O.; Sánchez-Huerta, A.; Breña-Naranjo, J.; Pedrozo-Acuña, A. Failure Analysis of a Water Supply Pumping Pipeline System. *Water* **2016**, *8*, 395. [CrossRef]
18. Kandil, M.; El-Sayed, T.A.; Kamal, A.M. Unveiling the Impact of Pipe Materials on Water Hammer in Pressure Pipelines: An Experimental and Numerical Study. *Sci. Rep.* **2024**, *14*, 30599. [CrossRef] [PubMed]
19. Fontanazza, C.M.; Notaro, V.; Puleo, V.; Nicolosi, P.; Freni, G. Contaminant Intrusion through Leaks in Water Distribution System: Experimental Analysis. *Procedia Eng.* **2015**, *119*, 426–433. [CrossRef]
20. Bohorquez, J.; Lambert, M.F.; Simpson, A.R. Identifying Head Accumulation Due to Transient Wave Superposition in Pipelines. *J. Hydraul. Eng.* **2020**, *146*, 04019044. [CrossRef]
21. Lee, J.-S.; Zeng, W.; Lambert, M.; Hilditch, T.; Gong, J. Fatigue Analysis of Metallic-Plastic-Metallic Pipeline Systems: A Numerical Study. *Results Eng.* **2023**, *17*, 100986. [CrossRef]
22. Jara-Arriagada, C.; Stoianov, I. Pressure-Induced Fatigue Failures in Cast Iron Water Supply Pipes. *Eng. Fail. Anal.* **2024**, *155*, 107731.
23. John, E.; Boxall, J.; Collins, R.; Bowman, E.; Susmel, L. Experimental Investigation of the Fatigue Strength and Leakage Failure Mode of Corroded Cast Iron Water Pipes. *Eng. Proc.* **2024**, *69*, 20. [CrossRef]

24. Simpson, A.; Wu, Z. Computer Modelling of Hydraulic Transients in Pipe Networks and the Associated Design Criteria. In Proceedings of the MODSIM 97, International Congress on Modelling and Simulation, Hobart, TAS, Australia, 8–11 December 1997; Volume 3, pp. 966–971.
25. Meniconi, S.; Maietta, F.; Alvisi, S.; Capponi, C.; Marsili, V.; Franchini, M.; Brunone, B. A Quick Survey of the Most Vulnerable Areas of a Water Distribution Network Due to Transients Generated in a Service Line: A Lagrangian Model Based on Laboratory Tests. *Water* **2022**, *14*, 2741. [[CrossRef](#)]
26. Urbanowicz, K.; Haluch, I.; Bergant, A.; Deptuła, A.; Śliwiński, P. Initial Investigation of Wave Interactions during Simultaneous Valve Closures in Hydraulic Piping Systems. *Water Resour. Manag.* **2023**, *37*, 5105–5125. [[CrossRef](#)]
27. Wan, W.; Zhang, B.; Chen, X.; Lian, J. Water Hammer Control Analysis of an Intelligent Surge Tank with Spring Self-Adaptive Auxiliary Control System. *Energies* **2019**, *12*, 2527. [[CrossRef](#)]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.