COST OPTIMIZATION OF WATER DISTRIBUTION SYSTEMS SUBJECTED TO WATER HAMMER

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ABSTRACT

The paper presents the water distribution systems optimization by selecting the optimal pipe diameters for water hammer transients. The optimization method used is the Genetic Algorithm (GA). The GA's have been used in solving the water network optimization for steady state conditions. The GA is integrated with the steady state hydraulic analysis program and a transient analysis program to improve the search for the optimal diameters under certain constraints. These include the minimum allowable pressure head constraints at the nodes for the steady state flow, and the minimum and maximum allowable pressure heads constraints for the water hammer. Three cases are studied including the following causes of water distribution transients: changes in water demands at the nodes, sudden valve closure and pump power failure. The application of the computer program to the studied cases shows the suitability of the method to find the least cost in a favorable number of function evaluations. This technique can be used in the first stages of the design of water distribution networks to protect it from the water hammer damages. The technique is very economical as the network design can be achieved without using hydraulic devices for water hammer control.

Keywords: Water Hammer, Fluid Transients, Genetic Algorithm, Pipe Networks

1. INTRODUCTION

The cost optimization of pipe networks under steady flow conditions was the subject of various researches. The optimization techniques were used to identify the optimal solution for water distribution systems. These techniques are classified into deterministic (linear, non-linear and dynamic programming) and stochastic techniques (simulated annealing and genetic algorithm). The deterministic methods can not guarantee a global optimal solution and require that the objective functions satisfy certain restrictive conditions (e.g., continuity, differentiability to the second order, etc.) that cannot be generally guaranteed for a water distribution system. Stochastic techniques and especially the genetic algorithm became a popular technique for optimization.

The optimization of piping network designs have been addressed by a range of researchers without considering the occurrence of water hammer event, they addressed the same in different ways during the past decades, Kessler and Shamir (1989), Djebedjian et al. (2000) and Sârbu and Kalmár (2002).

The perception of network optimization in steady state analysis linked to the consequences of water hammer is recently examined. Few water network optimization approaches have been achieved.

Laine and Karney (1997) studied the event of water hammer in a simple pipeline. Zhang (1999) studied the fluid transients and pipeline optimization using Genetic Algorithms. Kaya and Güney (2000) studied the same for sprinkler irrigation systems.

Jung and Karney (2003) studied the optimum selection of hydraulic devices for water hammer control in the pipeline systems using Genetic Algorithm.

Jung and Karney (2004a) studied the optimal selection of pipe diameters in a network considering steady state and transient analysis in water distribution systems by using Genetic Algorithm (GA) and Particle Swarm Optimization (PSO).

Jung and Karney (2004b) studied the pipeline optimization by selecting, sizing and placement for hydraulic devices in pipeline systems considering the occurrence of water hammer event.

Djebedjian et al. (2005a) studied the water distribution systems in both steady and transient (water hammer) states. They developed a numerical technique to analyze the network in the steady and transient states and select the optimum solution to overcome the different water hammer events using genetic algorithms. They linked between the hydraulic network solver (Newton-Raphson), transient analyzer and genetic algorithm as an optimization tool. The model was applied successfully on a network under water hammer event caused by pump station power failure. The model was based on selecting the proper (optimum) pipes sizes from a range of the available pipe sizes, which satisfied the network requirements such as pressure heads, demands and pressure limits for water hammer. This approach provided the opportunity for potential savings in costs.

Djebedjian (2006) studied the reliability-based optimization of potable water networks by selecting the optimal pipe diameters for water hammer under hydraulic reliability. Genetic Algorithm (GA) as an optimization tool has been linked with the Monte Carlo Simulation for estimating network capacity reliability and node capacity reliability.

The previous literature review demonstrates the suitability of the Genetic Algorithms as an optimization technique to handle small and large-scale water distribution systems and to minimize the cost in the steady state. From the previously mentioned literature review, it appears that there were no studies carried out considering the following extremes altogether, water distribution systems design, optimization using genetic algorithms and water hammer as a major risk, all networks should be designed to eliminate it.

2. OPTIMIZATION OF PIPELINE SYSTEMS

Water distribution system (WDS) design problem is formulated and solved here as a single-objective optimization problem with the selection of pipe diameters as the decision variables. The main parameter is subject to minimization which is the cost of the network design and construction. The optimization problem is solved using a single-objective genetic algorithm (GA). The proposed robust design method is applied to a case study with three causes of water hammer.

The objective of the optimum design model presented here is to minimize total design costs under the constraint of minimum head requirements in steady state condition and minimum and maximum heads requirements in transient condition (water hammer). The later is included in order to protect the system from negative or positive transient pressures. More specifically, the optimization problem is to minimize the objective function *Z*. It is the summation of the network cost and penalty cost in both cases: steady state and water hammer:

$$Z = C_T + C_{P-SS} + C_{P-WH} \tag{1}$$

Network pipe cost is described as follows:

$$C_T = \sum_{i=1}^{i=N} c_i \left(D_i\right).L_i \tag{2}$$

Penalty cost in case of steady state is described as follows:

$$C_{P-SS} = \begin{cases} 0 & \text{if } H_{\min,ST} - H_j \le 0 \\ \left[\dfrac{C_T}{M}\sum_{j=1}^{M}\left(H_{\min,ST} - H_j\right)\right] & \text{if } H_{\min,ST} - H_j > 0 \end{cases} \tag{3}$$

Equation (3) was proposed by Djebedjian et al. (2005b, 2006). The total penalty cost in case of water hammer is described as follows:

$$C_{P\text{-}WH} = C_{P-WH-MAX} + C_{P-WH-MIN} \tag{4}$$

$$C_{P\text{-}WH\text{-}MAX} = \begin{cases} 0 & \text{if } H_{j,\max} - H_{\max,Tr} \le 0 \\ \left[C_T \sum_{j=1}^{M} \left(H_{j,\max} - H_{\max,Tr} \right) \right] & \text{if } H_{j,\max} - H_{\max,Tr} > 0 \end{cases} \tag{5}$$

$$C_{P-WH\text{-}MIN} = \begin{cases} 0 & \text{if } H_{\min,Tr} - H_{j,\min} \le 0 \\ \left[C_T \sum_{j=1}^{M} \left(H_{\min,Tr} - H_{j,\min} \right) \right] & \text{if } H_{\min,Tr} - H_{j,\min} > 0 \end{cases} \tag{6}$$

where the parameters are as follows:

- $c_i(D_i)$: Cost of pipe i per unit length
- $C_{P\text{-}SS}$: Penalty cost in case of steady state
- $C_{P\text{-}WH}$: Penalty cost in case of water hammer
- $C_{P\text{-}WH\text{-}MAX}$: Penalty cost in case of water hammer when the pressure head exceeds the maximum allowable pressure head limit
- $C_{P\text{-}WH\text{-}MIN}$: Penalty cost in case of water hammer when the pressure head decreases below the minimum allowable pressure head limit
- C_T : Network total cost
- D_i : Diameter of pipe i
- H_j : Pressure head at node j
- $H_{j,\max}$: Maximum pressure head at node j under water hammer
- $H_{j,\min}$: Minimum pressure head at node j under water hammer
- $H_{\max,TR}$: Maximum allowable pressure head for water hammer
- $H_{\min,ST}$: Minimum allowable pressure head for steady state
- $H_{\min,TR}$: Minimum allowable pressure head for water hammer
- L_i : Length of pipe i
- M : Total number of nodes
- N : Total number of pipes
- Z : Total cost of the network (design and penalty)

Generally, the penalty cost is a function of minimum allowable pressure head at each node, pressure at each note and number of nodes violating the criteria.

The minimization of the objective function Z in Eq. (1) is subject to:

(a) Mass balance constraint:

$$\sum_{j=1}^{M} Q_j = 0 \tag{7}$$

where Q_j represents the discharges into or out of the node j (sign included).

(b) Energy balance constraint:

$$\sum h_f = E_p \tag{8}$$

The conservation of energy states that the total head loss around any loop must equal to zero or is equal to the energy delivered by a pump, E_p, if there is any. The head loss due to friction in a pipe h_f is expressed by the Hazen-William formula:

$$h_{f\,i} = \frac{4.727}{C_i^{1.852}} \frac{L_i Q_i^{1.852}}{D_i^{4.8704}} \tag{9}$$

where Q_i is the pipe flow (ft^3/s), D_i is pipe diameter (ft), L_i is pipe length (ft) and C_i is the Hazen-Williams coefficient.

(c) Decision variables constraint:
The design constraints (the pipe diameter bounds (maximum and minimum)) and the hydraulic constraints are given respectively as:

$$D_{min} \leq D_i \leq D_{\max} \qquad i = 1,...,N \tag{10}$$

where D_i is the discrete pipe diameters selected from the set of commercially available pipe sizes.

(d) The hydraulic constraints for steady state and water hammer are given as:

$$H_j \geq H_{\min ST} \qquad j = 1,...,M \tag{11}$$

where H_j is the pressure head at node *j*, $H_{j,\min ST}$ is the minimum allowable pressure head at node *j* for the steady state.

$$H_{\min,TR} \leq H_k \leq H_{\max,TR} \quad k = 1,...,M_p \tag{12}$$

where $H_{\min TR}$ and $H_{\max TR}$ are the minimum and maximum allowable pressure heads at node *k* for the transient conditions, and M_p is the number of parts into which the pipe is divided.

From the previous constraints, item (a) through (d), it is noted that only pipes are considered for the design. Minor losses are neglected with respect to friction losses due to pipe length. Therefore; pumps, valves, and other special hydraulic appurtenances are not included for purposes of discussion and simplicity of the model development.

Using GA to solve the optimization problem in Equation (1), constraints (a), (b), (c) and (d) can be satisfied by linking GA to the deterministic WDS solver such as Newton-Raphson method and transient analyzer.

The Newton-Raphson method is used to simulate hydraulically the given network for the steady state and the water hammer analysis is implemented by a method of characteristics, Wylie et al. (1993). Constraint (c) can also be automatically satisfied by using the appropriate GA coding.

Transients' Analysis in Piping Networks

Larock et al. (2000) stated that the process of obtaining an unsteady solution for a specific problem in which the demand or heads are specified functions of time consists of the following tasks:

1. The time span, over which the unsteady solution is to be obtained, is divided into NT time increments.
2. The discharges in all pipes and the heads at all nodes are assigned initial values that are chosen from steady state solution that has the same demands, and all other data. As the unsteady solution has at time zero.
3. All demands over each time increment must be specified.
4. Over each new time increment, define and evaluate the function and the Jacobean matrix of derivatives of these functions.
5. Solve the resulting linear equation system. The solution of this equation system is then subtracted from the set of unknown values, according to the Newton method.
6. Steps 4 and 5 are repeated iteratively, until the specified convergence criterion has been satisfied.
7. Write the solution for the discharges and the nodal for this time increment, and then repeat steps 3 through 7 until the unsteady solution spans the entire time period.

The steps from 1 through 7 are the general method for analyzing an unsteady piping system.

Equations Describing Unsteady Flow in Pipes

The analysis of the unsteady flow in piping networks is based on the characteristic method. A pair of equations can be developed to find the pressure head *H* and the velocity *V* in a pipe divided into *N* segments at the interior point *P* starting from point 2 to point *N* (point 1 is related to the boundary condition), Larock et al. (2000):

$$V_P = \frac{1}{2}\left[\left(V_{Le} + V_{Ri}\right) + \frac{g}{a}\left(H_{Le} - H_{Ri}\right) - \frac{f\,\Delta t}{2D}\left(V_{Le}\left|V_{Le}\right| + V_{Ri}\left|V_{Ri}\right|\right)\right] \tag{13}$$

$$H_P = \frac{1}{2}\left[\frac{a}{g}\left(V_{Le} - V_{Ri}\right) + \left(H_{Le} + H_{Ri}\right) - \frac{a}{g}\frac{f\,\Delta t}{2D}\left(V_{Le}\left|V_{Le}\right| - V_{Ri}\left|V_{Ri}\right|\right)\right] \tag{14}$$

Le and *Ri* are considered as the left and right points on the characteristic grid with respect to a certain point *P* and at the same distance from it.

3. IMPLEMENTATION OF GENETIC ALGORITHMS OVER PIPE NETWORK

Genetic algorithms (GAs) are adaptive methods which may be used to solve search and optimization problems. They are based on the genetic processes of biological organisms. Over many generations, natural populations evolve according to the principles of natural selection and "survival of the fittest. By mimicking this process, genetic algorithms are able to "evolve" solutions to real world problems, if they have been suitably encoded. The basic principles of GAs were first laid down rigorously by Holland (1975) and Glodberg (1989).

The flow chart in Fig. 1 shows the sequence of the basic operators used in genetic algorithms. We start out with a randomly selected first generation. Every string in this generation is evaluated according to its quality, and a fitness value is assigned. Next, a new generation is produced by applying the reproduction operator. Pairs of strings of the new generation are selected and crossover is performed. With a certain probability, genes are mutated before all solutions are evaluated again. This procedure is repeated until a maximum number of generations is reached. While doing this, the all time best solution is stored and returned at the end of the algorithm.

As one notices from the flow chart, the genetic algorithm serves as a framework which provides the outer cycle of the search or optimization process. An important part of the loop is the evaluation function which determines the fitness value of a specific string. Within this method, the string has to be mapped to a realistic solution, and the objective function has to be evaluated. For this, heuristic methods might be necessary.

The brief idea of GA is to select population of initial solution points scattered randomly in the optimized space, then converge to better solutions by applying in iterative manner the following three processes (reproduction/selection, crossover and mutation) until a desired criteria for stopping is achieved.

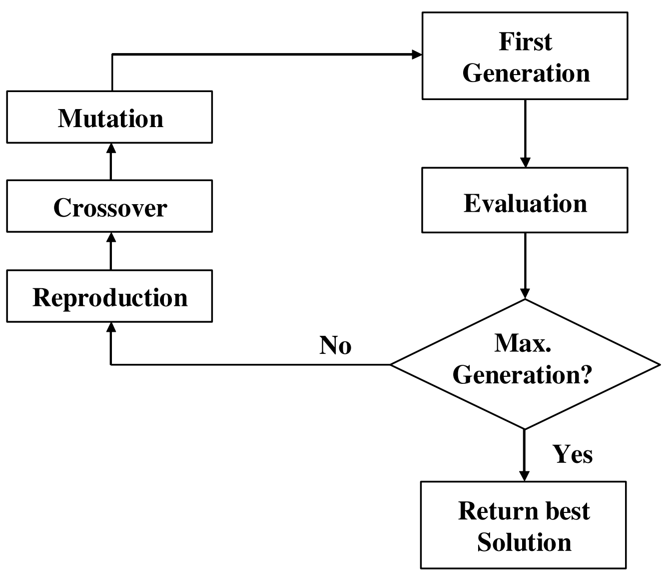

Fig. 1 Genetic algorithm flow chart

The optimization program GAST*net* (Genetic Algorithm Steady Transient network) is written in FORTRAN language and it links the GA, the Newton-Raphson simulation technique for the steady state hydraulic simulation and the transient analysis. The Newton-Raphson and transient analyzer are considered as subroutines in the main code genetic algorithms. A brief description of the steps in using GA for pipe network optimization, and including water hammer is as follows:

1. **Generation of initial population.** The GA randomly generates an initial population of coded strings representing pipe network solutions of population size *Npopsiz*. Each of the *Npopsiz* strings represents a possible combination of pipe sizes.
2. **Computation of network cost.** For each *Npopsiz* string in the population, the GA decodes each substring into the corresponding pipe size and computes the total material cost. The GA determines the costs of each trial pipe network design in the current population, as described in Equation (2).
3. **Hydraulic analysis of each network.** A steady state hydraulic network solver computes the heads and discharges under the specified demands for each of the network designs in the population. The actual nodal pressures are compared with the minimum allowable pressure heads, and any pressure deficits are noted. In this study, the Newton-Raphson technique is used.
4. **Computation of penalty cost for steady state.** The GA assigns a penalty cost for each demand if a pipe network design does not satisfy the minimum pressure constraints. The pressure violation at the node at which the pressure deficit is maximum, is used as the basis for computation of the penalty cost. The maximum pressure deficit is multiplied by a penalty factor (C_T / M) as described in Equation (3).
5. **Transient analysis of each network.** A transient analysis solver computes the transient pressure heads resulting from the pump power failure, sudden valve closure or sudden demand change. The minimum and maximum pressure heads are estimated in each pipe of the network and compared with the minimum and maximum allowable pressure heads, and any pressure deficits are noted.
6. **Computation of penalty cost for transient state.** The GA assigns a penalty cost if a pipe design does not satisfy the minimum and maximum allowable pressure heads constraints. The penalty cost is estimated as the pressure violation multiplied by a penalty factor equals to the cost of the specified pipe $c(D).L$.
7. **Computation of total network cost.** The total cost of each network in the current population is taken as the sum of the network cost (Step 2), the penalty cost (Step 4), plus the penalty cost (Step 6), this step is an expression to Eq. (1).
8. **Computation of the fitness.** The fitness of the coded string is taken as some function of the total network cost. For each proposed pipe network in the current population, it can be computed as the inverse or the negative value of the total network cost from Step 7.
9. **Generation of a new population using the selection operator.** The GA generates new members of the next generation by a selection scheme.

10. **The crossover operator.** Crossover occurs with some specified probability of crossover for each pair of parent strings selected in Step 9.
11. **The mutation operator.** Mutation occurs with some specified probability of mutation for each bit in the strings, which have undergone crossover.
12. **Production of successive generations.** The use of the three operators described above produces a new generation of pipe network designs using Steps 2 to 11. The GA repeats the process to generate successive generations. The last cost strings (e.g., the best 20) are stored and updated as cheaper cost alternatives are generated.

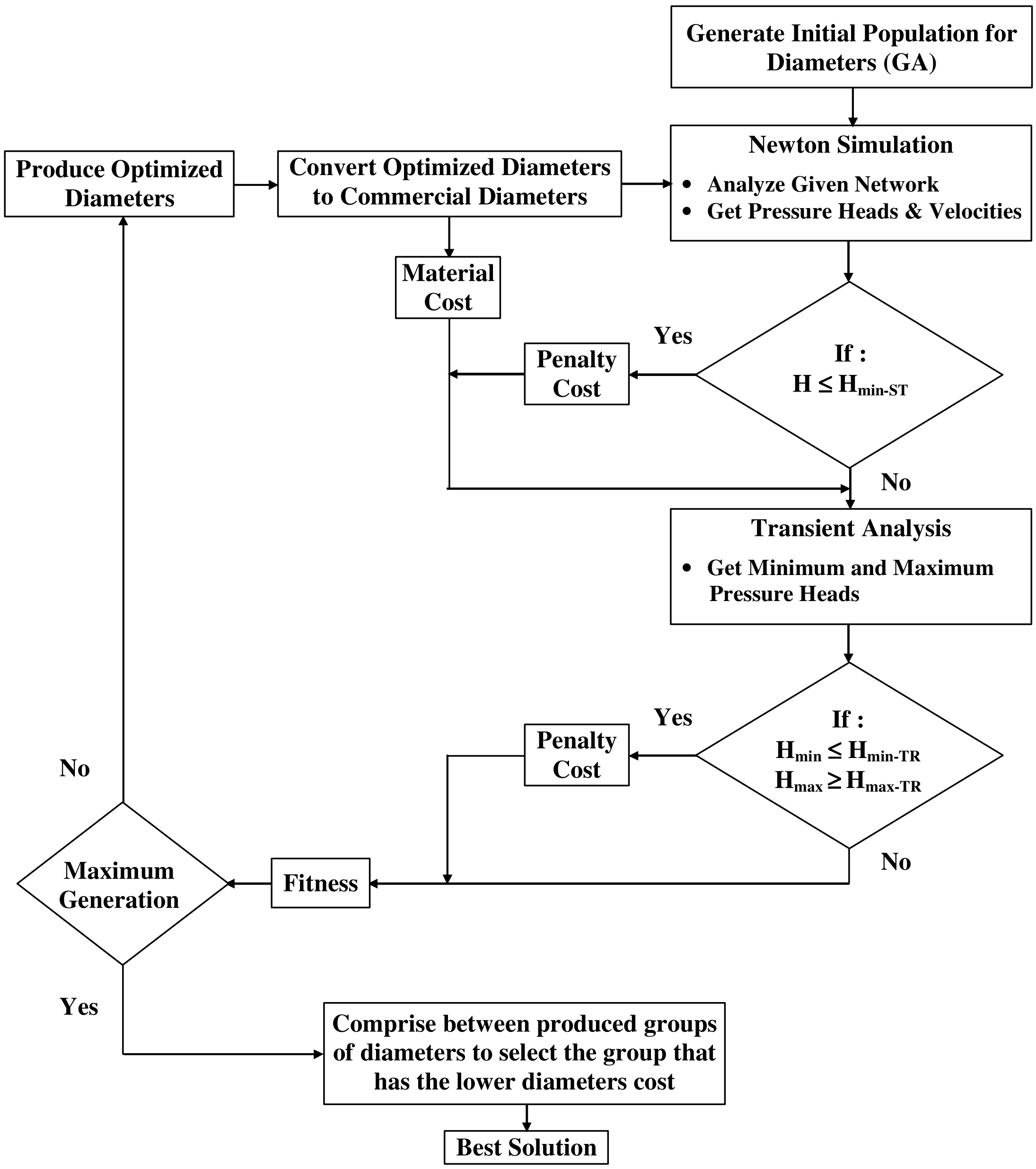

Fig. 2 Flow chart of the GAST*net* program

These steps for the optimization of water network considering both steady state and transient conditions are illustrated in the flow chart of the GAST*net* program, Fig. 2. This program is an extension of the GANR*net* computer program, Djebedjian et al. (2005b). It has been developed to optimize pipe networks for steady state using the genetic algorithm approach.

The genetic algorithm in the GAST*net* program has several parameters that enable moving to different search regions to approach the global solution; these parameters are: *Npopsiz*: the population size of a GA run, *Idum*: the initial random number seed for the GA run, and it must equal a negative integer, *Maxgen*: the maximum number of generations to run by the GA, and *Nposibl*: the array of integer number of possibilities per parameter.

Case Study

As illustrated in Fig. 3, a pre-defined water supply piping network, Larock et al. (2000), consists of three reservoirs at nodes 1, 6 and 10, ten nodes, two pump stations and twelve pipes. The demands at nodes (3, 4, 5, 8 and 9) are (1300, 900, 1800, 450 and 1300 gpm), respectively. The lengths, diameters of pipes and Hazen-Williams roughness coefficients are given in Table 1. The pumps data are given in Table 2.

The case study used in this study is subjected to the following causes of water hammer: pump power failure, valve sudden closure and sudden demand change. The set of commercially available pipe diameters are (6, 8, 10, 12, and 15) inches and the corresponding cost per foot length is (15, 25, 35, 45, and 65) units, Table 3.

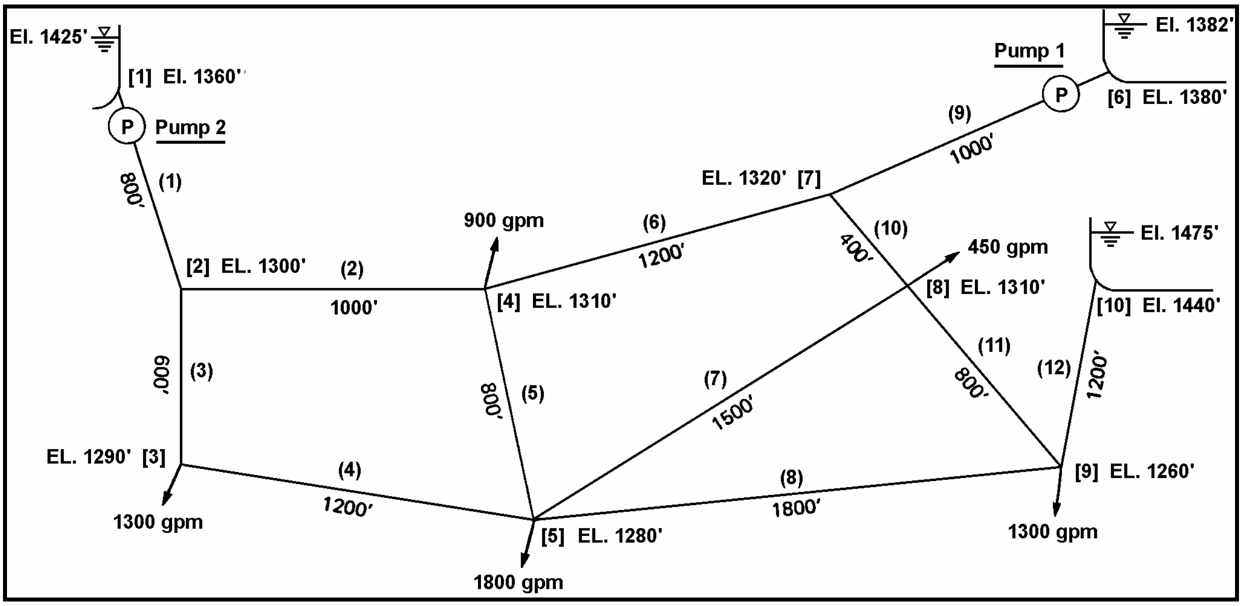

Fig. 3 Typical piping network with two pump stations

Table 1 Pipes data for the network with two pump stations

Pipe ID	Start Node	End Node	L (ft)	D (in.)	C	Wave Speed (ft/s)
1	1	2	800	15	100	3300
2	2	4	1000	12	100	3300
3	2	3	600	12	100	3300
4	3	5	1200	12	100	3300
5	4	5	800	10	100	3300
6	4	7	1200	8	100	3300
7	5	8	1500	8	100	3300
8	5	9	1800	8	100	3300
9	6	7	1000	10	100	3300
10	7	8	400	8	100	3300
11	8	9	800	8	100	3300
12	10	9	1200	12	100	3300

Table 2 Pumps data for the network with two pump stations

	Pump Station 1	Pump Station 2
No. of Parallel Pumps	2	6
No. of Stages	2	1
N (r.p.m.)	1175	1175
Rotational Moment of Inertia	45	45

Q (gpm)	0	400	600	800	1000	1280
H (ft)	57.5	56.0	54.5	50.0	40.5	0.0
Power (hp)	5.0	8.5	10.5	12.0	13.5	12.0

Table 3 Network pipes unit cost

Diameter (in.)	Cost (Units)
6	15
8	25
10	35
12	45
15	65

All calculations were produced on a computer with Pentium 4 (3.00 GHz) processor and 512 MB of RAM.

Theoretically, the required minimum pressure head at all nodes was assumed to be 80 ft for the steady state and for the transient conditions, the minimum and maximum pressure heads were considered as 80 ft and 180 ft, respectively.

The GA parameters used in GAST*net* optimization program in this case were: *Npopsiz* = 5, *Idum* = −5000, *Maxgen* = 1000 and *Nposibl* = 16. Mutation and crossover rates were set to 0.2 and 0.5, respectively. For the steady state calculations, the accuracy was 0.0001 ft^3/s. The time of the transient flow simulation was taken as 40 s and the hydraulic time step Δt was 0.04 s.

The GAST*net* program has the capability to run under the following modes:

1- Steady State-Simulation Mode: It uses the hydraulic analysis of network to obtain the flows in pipes and the heads t nodes under steady state conditions.
2- Transient-Simulation Mode: After the application of the steady state conditions, the water hammer cause is applied and the transient simulation is carried out giving the pressure head against time variation at nodes.
3- Transient-Optimization Mode: In this mode, the GA chooses a set of diameters and the previous two simulations are applied and checked by the pressure head requirements. This procedure is repeated to a maximum number of generations, *Maxgen*, and the best set of diameters giving the least cost is selected.

The subject network is predefined one, meaning that network has its own characteristics (pipe diameters, layout…). The GAST*net* program was applied on the predefined network in a transient-simulation mode, to delineate the efficiency of the GAST*net* program before optimization, then the GAST*net* program was applied in the transient-optimization mode. All the program outputs (pressure against time) were plotted on one diagram for each node before and after optimization.

1. Two Pumps Power Failure

The network of Figure 3 is used to demonstrate the effect of water hammer event by pump station power failure in the two stations. The results of the water hammer event initiated separately by shutting down each one can be found in AbdelBary (2008).

The GAST*net* optimization program was applied and the network with the new optimal diameters was found. Figure 4 depicts the evolution of the solution as the program develops in a single run. A quite slow decrease in the cost value for the first group of evaluation then fast changes in the later evaluations is observed.

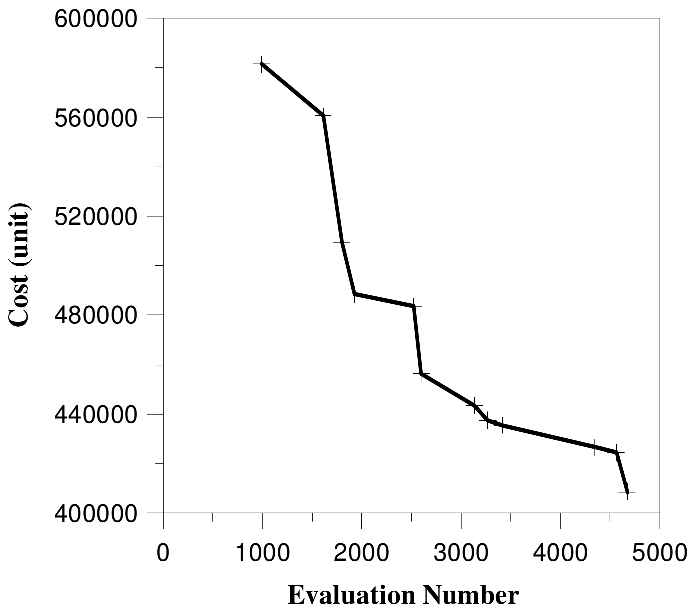

Fig. 4 Cost units versus evaluation number for the two pump power failure

The network contains 12 pipes and with 5 available commercial pipe sizes, the available number of solutions is $5^{12} = 24.41 \times 10^7$. Applying the GAST*net* program, it is found that the number of function evaluations was 4669 to reach the optimal solution. It is very minor value compared to the total solution space (0.0019%).

Table 4 shows the optimal diameters for the network against the original ones. The least cost is 408,500.00 units after optimization against 437,500.00 units, which indicates 0.934 of the original pipe network cost.

Table 5 displays the subsequent nodal pressure heads for the steady state. These values are fulfilling the minimum pressure constraint of 80 ft at all nodes except the reservoirs nodes. The three reservoirs at nodes 1, 6 and 10 have heads of 65, 2 and 35 ft, respectively.

Table 4 Optimal against original diameters (in.) and associated cost for the two pumps power failure

Pipe Number	Original Diameter (in.)	Optimal Diameter (in.)
1	15	8
2	12	8
3	12	8
4	12	6
5	10	8
6	8	15
7	8	6
8	8	10
9	10	6
10	8	15
11	8	15
12	12	12
Cost (units)	**437,500.00**	**408,500.00**
Run Time (min)	7	30

Table 5 Pressure heads at nodes for the steady using the optimal diameters

Node	Pressure Head (ft)
1	**65**
2	126.48
3	113.67
4	111.28
5	128.35
6	**2**
7	103.73
8	113.87
9	165.43
10	**35**

Pump 1 in Pipe 9:
Discharge = 744.721 gpm,
Head = 111.549 ft
Pump 2 in Pipe 1:
Discharge = 1533.499 gpm
Head = 53.908 ft

The application of the GAST*net* program in transient-simulation mode using the original network pipes diameters, Table 1, which subjected to the same water hammer cause reveals the dashed curves in Fig. 5. It is apparent that the pressure heads at nodes are quantitatively affected by the two pump stations power failure. The pressure fluctuations exceed the maximum pressure (180 ft) at nodes 8 and 9 and decrease below the lower limit (80 ft) in node 6.

The effect of the two pump stations power failure in the pipe network with the optimal diameters is realized after the application of the GAST*net* program in transient-optimization mode. The continuous curves in Fig. 5 show the pressure head versus time response at all nodes including the reservoir nodes. After optimization, the pressure fluctuations at nodes 8 and 9 became within the acceptable range (80 – 180 ft).

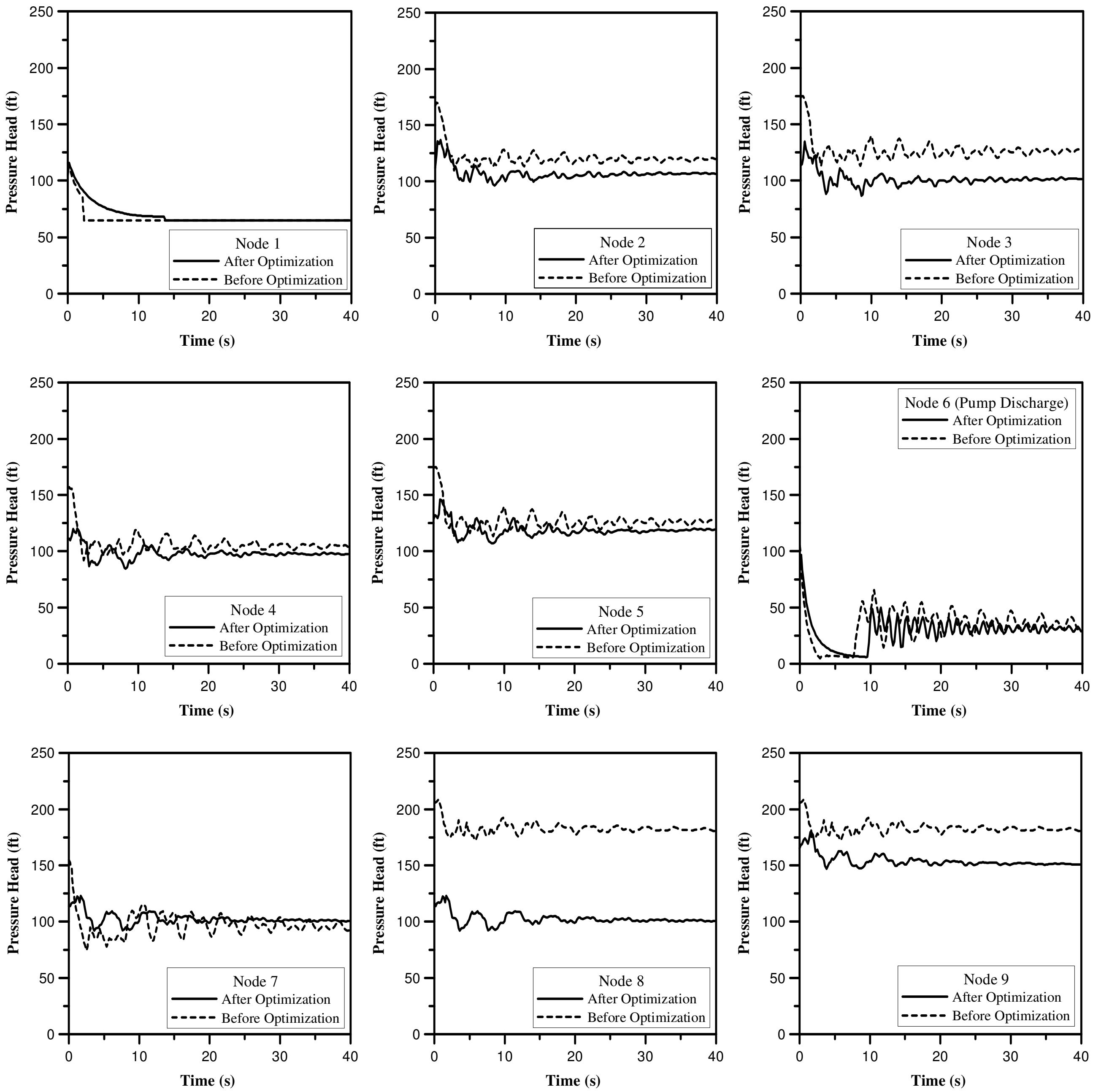

Fig. 5 Pressure head versus time for various nodes for the two pumps power failure

2. Sudden Valve Closure

One of the most important water hammer causes is the sudden valve closure. In this section, the effect of a sudden valve closure located at the downstream end of pipe 2 in the network in Fig. 3 is studied. The same values for steady state, transient conditions, and GA parameters were used. As mentioned above, the minor loss due to the valve is neglected.

Figure 6 depicts the evolution of the solution as the GAST*net* program develops in transient-optimization mode. The cost is decreased gradually over the final evaluation number till reaching to the least cost.

The total solution space is $5^{12} = 24.41 \times 10^{7}$ different network designs. Using the GA optimization techniques, the number of function evaluations was 10,800 to reach the optimal solution and this is only a very small fraction of the total search space (0.0044%).

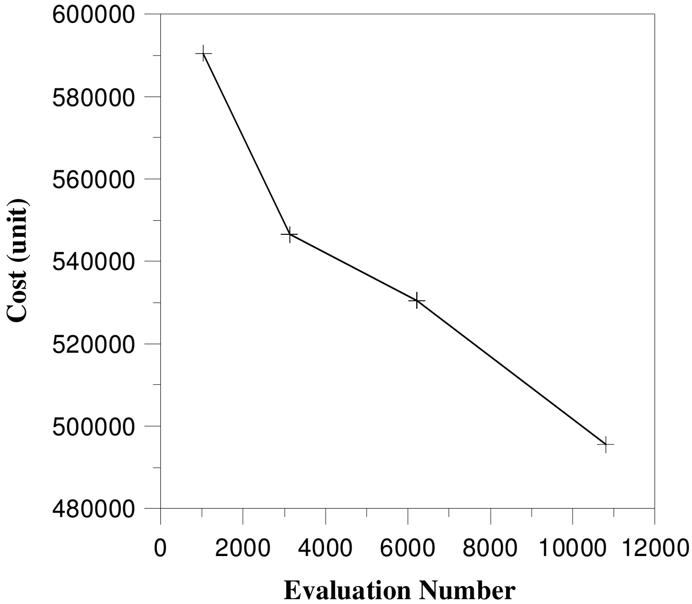

Fig. 6 Cost units versus evaluation number for the sudden valve closure case

Table 6 shows the optimal diameters for the network against the original ones. The least cost is 495,500.00 units after optimization against 437,500.00 units, which is equal 1.133 times the original cost. Here, the cost is not considered as a dominant factor as it is meaningless to design a cheap non-reliable network; the optimum cost exceeded the original cost design, that in order to overcome the water hammer event.

From Table 6, it can be noted that the diameter found by the genetic algorithm for pipe number 1 is 6 in. which is very small. If the pipes connected to the reservoirs at nodes 1, 6 and 10 were not included in the optimization process by giving their diameters a pre-specified value and by treating them as constraints; then these pipe diameters remain as specified and the pressure head can be kept at higher values. In this network optimization, all diameters were searched and the optimal pipe diameters for some pipes which are connected to the reservoirs were small.

Table 7 displays the corresponding nodal pressure heads for the steady state. These heads fulfill the network requirements.

Table 6 Optimal against original diameters (in.) and associated cost for the sudden valve closure

Pipe Number	Original Diameter (in.)	Optimal Diameter (in.)
1	15	6
2	12	12
3	12	8
4	12	8
5	10	12
6	8	15
7	8	12
8	8	6
9	10	15
10	8	15
11	8	15
12	12	10
Cost (units)	**437,500.00**	**495,500.00**
Run Time (min)	2	25

Table 7 Pressure heads at nodes for the steady state using the optimal diameters

Node	Pressure Head (ft)
1	**65**
2	109.26
3	106.99
4	99.26
5	126.23
6	**2**
7	94.50
8	104.22
9	155.14
10	**35**

Pump 1 in Pipe 9:
Discharge = 2553.746 gpm,
Head = 40.393 ft
Pump 2 in Pipe 1:
Discharge = 816.308 gpm
Head = 50.467 ft

The GAST*net* program was applied twice: in transient-simulation mode and transient-optimization mode to demonstrate the differences in simulation before and after optimization. For the case before optimization, the results of simulation were not plotted as dashed curves in Fig. 7. The GAST*net* program ceases the running operation at time $t = 0.04$ s due to the non stability of the network under the water hammer event caused by the sudden valve closure downstream pipe 2. The operation halt has occurred as an evidence of the instability and non-operability of the network with the original set of diameters that in case of sudden valve closure.

As depicted in Figure 7, the pressure head versus time response at all nodes are plotted. The convergence to steady state caused by sudden valve closure is rapid. The pressure heads at the nodes are quantitatively affected by the sudden valve closure; the more quantitatively affected node is node 2. The choice of the time of the transient flow simulation as 40 s was sufficient to obtain nearly steady state condition.

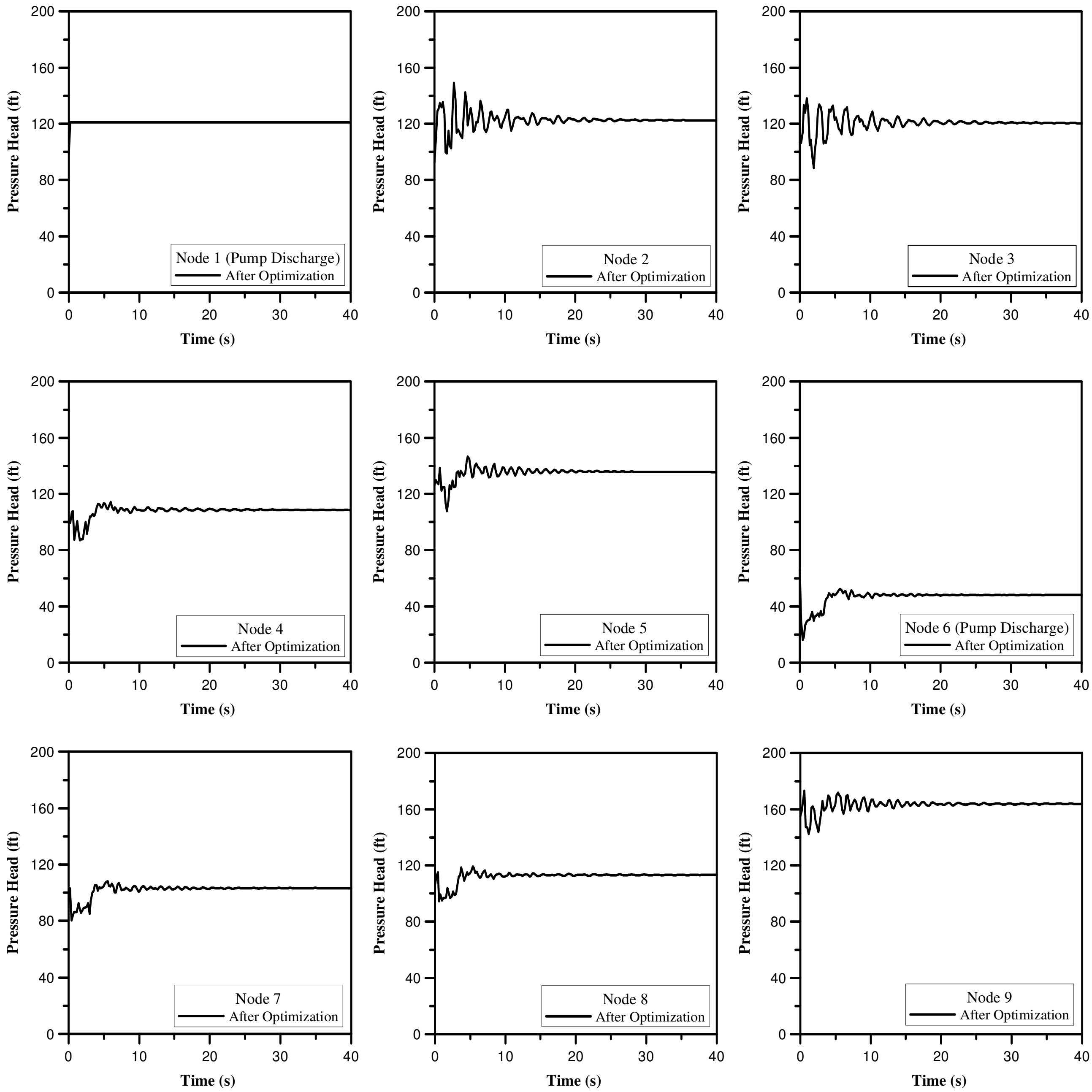

Fig. 7 Pressure head versus time for various nodes for the sudden valve closure

3. Sudden Demand Change

Sudden demand change event is one of the water hammer causes. It is introduced on the network of Fig. 3. The demand was changed at node 5 from 1800 GPM to 2000 GPM to meet a sudden need for more water for fire suppression.

For this example and as given in the previous examples for this network, the required minimum pressure head at all nodes was 80 ft for the steady state and for the transient conditions, the minimum and maximum pressure heads were 80 ft and 180 ft, respectively. The accuracy of the steady state calculations was 0.0001 ft^3/s. The time of the transient flow simulation was taken as 40 s and the hydraulic time step Δt was 0.04 s.

The evolution of the solution is depicted in Figure 8. A rapid decrease in the cost value for the first group of evaluation then slow changes in the later evaluations is observed.

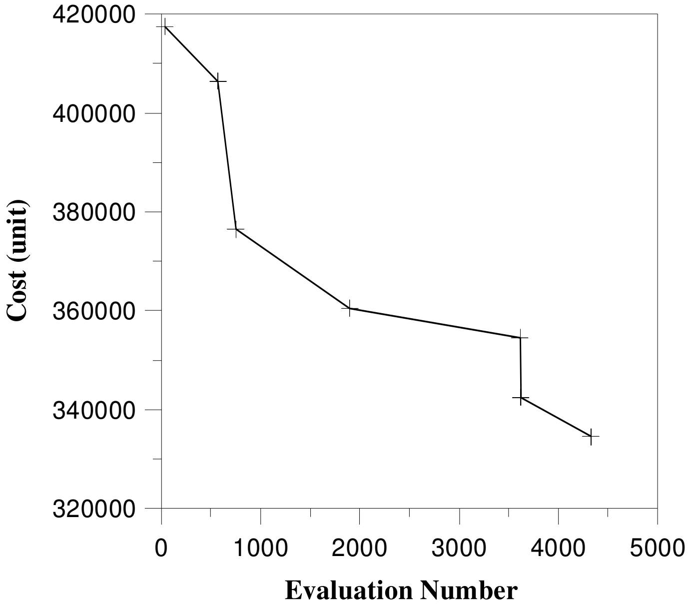

Fig. 8 Cost units versus evaluation number for the sudden demand change

The total solution space is $5^{12} = 24.41 \times 10^7$ different network designs. The number of function evaluations was 4327 to reach the optimal solution which is 0.00177% of the total search space.

Table 8 shows the optimal diameters for the network against the original ones. The least cost is 334,500.00 units after optimization against 437,500.00 units for the original network, which is equal 0.765 times the original cost.

Table 9 displays the corresponding nodal pressure heads for the steady state. These heads fulfill the minimum pressure constraint of 80 ft at all nodes except the reservoirs nodes.

Table 8 Optimal against original diameters (in.) and associated cost for the sudden demand change

Pipe Number	Original Diameter (in.)	Optimal Diameter (in.)
1	15	10
2	12	15
3	12	8
4	12	6
5	10	15
6	8	10
7	8	6
8	8	6
9	10	8
10	8	8
11	8	6
12	12	6
Cost (units)	**437,500.00**	**334,500.00**
Run Time (min)	6	25

Table 9 Pressure heads at nodes for the steady state using the optimal diameters

Node	Pressure Head (ft)
1	**65**
2	105.97
3	95.73
4	89.63
5	115.14
6	**2**
7	86.81
8	87.47
9	124.89
10	**35**

Pump 1 in Pipe 9:
Discharge = 1635.658 gpm
Head = 98.658 ft
Pump 2 in Pipe 1:
Discharge = 3339.783 gpm
Head = 55.715 ft

The operability and reliability of the original network was checked using the GAST*net* optimization program in transient-simulation mode using the original network pipes diameters, Table 1. The program ceased its running operation after time $t = 0.04$ s therefore the results before optimization were not plotted in Fig. 9.

The GAST*net* optimization program results in transient-optimization mode are illustrated in Figure 9. The pressure head versus time response at all nodes are shown and it can be observed that the convergence to steady state caused by sudden demand change is rapid. It is obvious that the pressure heads at the nodes are affected by the sudden demand change. The simulation time was sufficient to obtain nearly steady state condition.

It is concluded that for the original pipe network, the piping network is not operational under sudden demand change which means that piping could not sustain such changes although the demand changed only from 1800 to 2000 gpm, i.e. by increasing about 11%. The application of the GA techniques converted the non-operational pipe network to operational one by the proper pipes diameters selection.

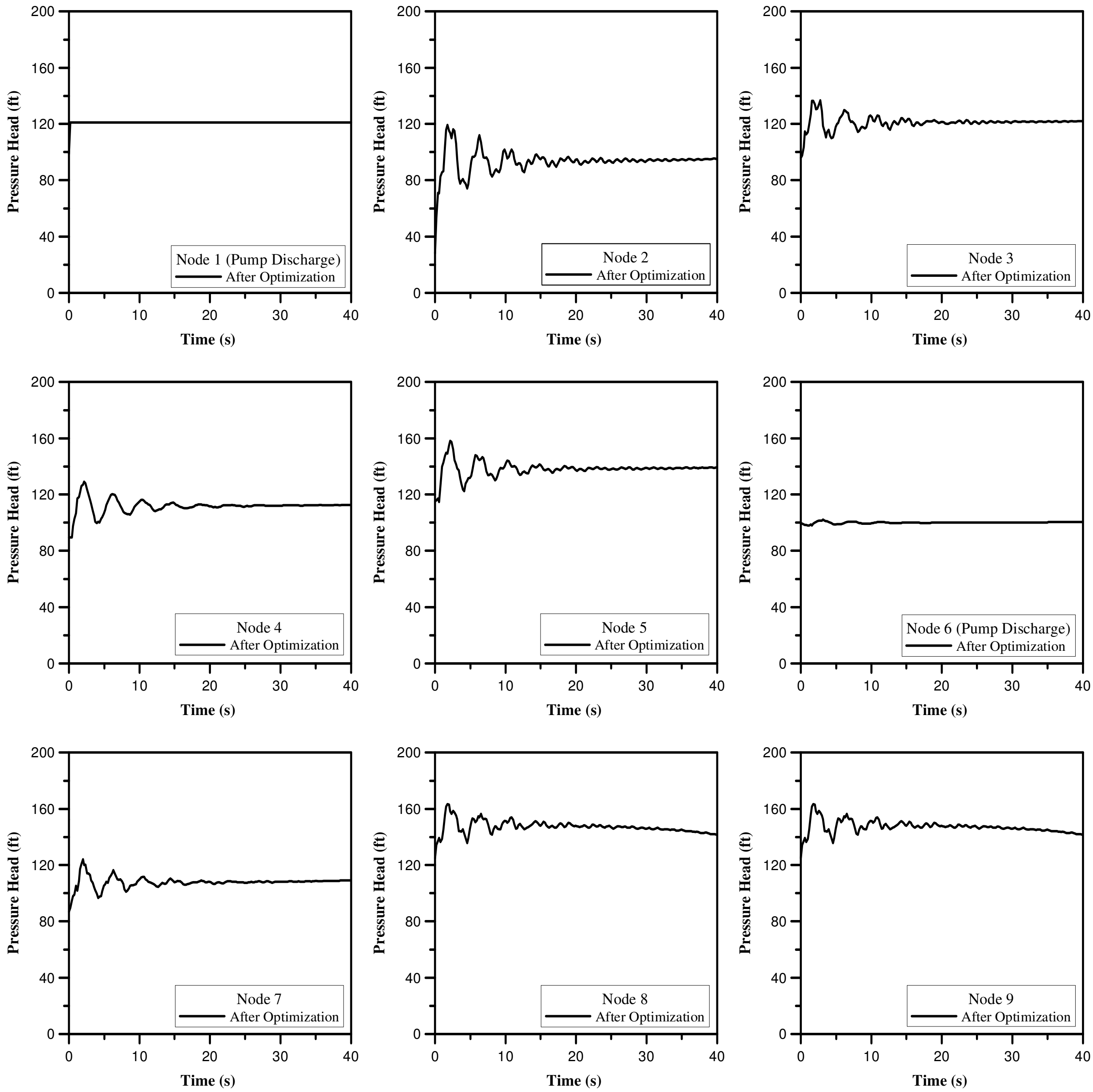

Fig. 9 Pressure head versus time for various nodes for the sudden demand change

4. CONCLUSIONS

The optimization of water distribution systems under transient state is a complex problem. In this paper the optimization is achieved by obtaining the optimal pipe diameter considering both steady and water hammer. The original piping networks are not operational or suffering from excessive pressure fluctuations under water hammer circumstances.

The genetic algorithm (GA) is utilized to find optimal pipe diameters in a case study with encountered water hammer causes. The case study shows that considering steady state design only is insufficient when there is a possibility for water hammer events. For that reason and depending on the encountered transient conditions, the appropriate sizing of pipe diameters can be selected for preventing water hammer. The selection of optimal pipe diameters is important to system performance under steady and transient states and decreasing costs.

The application of the GAST*net* optimization program reveals the following benefits:

- GAST*net* optimization program has been successfully applied on a case study with 3 causes of water hammer proving high performance with least cost avoiding using of water hammer arrestors.
- The application of GAST*net* program to the examples demonstrates the capability of the GA to find the optimal pipe diameters in a small fraction of the total search space and a reasonable run time in spite of the complicated behavior of fluid transients.
- The application of GA technique in GAST*net* optimization program finds the optimal diameters for the transient state and the nodal pressure fluctuations fallen between the water hammer predetermined limits and converges rapidly to the steady state case. This proves the validation of the integrated program water hammer analyzer and GA technique.
- The cost saving has been increased by 6.6, −13.3 and 23.5% in the theoretical Examples 1 through 3, respectively. Not necessarily the optimum cost to be the lowest, such as Example 2. The cost was slightly increased, but this is not a factor as the original network was not functional.

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