NETWORK OPTIMIZATION FOR STEADY FLOW AND WATER HAMMER USING GENETIC ALGORITHMS

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ABSTRACT

The paper presents the water network optimization by selecting the optimal pipe diameters for steady state flow and water hammer. 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 Newton-Raphson 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 caused by the pump power failure. The application of the computer program to a case study 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.

Key Words: Water Hammer, Fluid Transients, Genetic Algorithm, Pipe Networks

INTRODUCTION

The selection of distribution system facilities is usually based on the results of hydraulic analyses. The outputs of the analyses such as the node pressures and flow velocities are compared to the minimum pressure at the nodes and the maximum allowable velocity requirements. Adjustments to design variables (such as pipe sizes), are made until all requirements are satisfied. Genetic Algorithms (GAs) automate the adjustment process, performing several hydraulic analyses in a directed search for the lowest cost combination of design variables that satisfy the requirements. The application of GAs for pipe network optimization has been applied to the steady state and recently to the transient conditions.

It is well known that water hammer in pipelines may shake the line system or damage the equipment, even causing serious accident, and it is a headache problem. Water hammer is usually resulted from abnormally shutting down pump, quickly opening or closing valve, wrong operation, etc. The strong surge will propagate along the pipeline and endanger running safety [1]. To solve the problem, on one hand, there are numerous techniques for controlling transients in water distribution systems. These include design considerations (pipe strengthening, larger diameter pipes utilization, and pipe material changing), operational considerations (valve opening and closing times adjustment, increasing pump inertia by adding a flywheel) and employing surge control devices (relief valves, check valves, bypass devices, surge tanks and air chambers).

On the other hand, the water hammer damages can be reduced or prevented when the initial water system design is examined numerically under the transient conditions. This can be achieved by integrating water hammer analysis with the design of an optimum pipe size for a pumping associated system.

The optimization of pipe networks under steady flow conditions has been studied and various researchers have proposed the use of mathematical programming techniques in order to identify the optimal solution for water distribution systems. Traditionally, the design of water distribution networks has focused on mathematical approaches including linear, nonlinear, and dynamic programming (Kessler and Shamir [2], Sonak and Bhave [3], Eiger et al. [4], Samani and Naeeni [5], Sârbu and Borza [6], and Sârbu and Kalmár [7]). However, these deterministic methods can not guarantee a global optimal solution. Also, they require that the functions satisfy certain restrictive conditions (e.g., continuity, differentiability to the second order, etc.) that cannot be generally guaranteed for a water distribution system.

Recently, the Genetic Algorithm (GA) has been a popular optimization choice for solving problems that are difficult for traditional deterministic optimization methods (Goldberg [8], Simpson et al. [9], and Dandy et al. [10]). The main advantage of GA is its ability to find the global optimum by using function values only.

The concept 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, for example, for a simple pipeline (Laine and Karney [11]), for sprinkler irrigation systems (Kaya and Güney [12]), selection of hydraulic devices for water hammer control (Jung and Karney [13], [14]) and pipeline optimization with sudden valve opening (Jung and Karney [15]).

The objective of the present paper is to obtain an optimal design of a water distribution system considering both steady and transient states. As an optimization method, GA is introduced. A formulation using GA is developed for the optimal design of water distribution system and applied to a case study by demonstrating the effect of water hammer in a pipe network. The network is equipped with pump, which its power failure will cause the transients.

OPTIMIZATION OF PIPELINE SYSTEMS

Water distribution network design is formulated as a least-cost optimization problem with the selection of pipe diameters as the decision variables. In addition to the general constraints, transient analysis is included in order to protect the system from negative or positive transient pressures. The pipe layout, nodal demand, and minimum head requirement are assumed known. The objective function is the total cost of the given network. The total cost C_T is calculated as:

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

where N is the total number of pipes, $c_i(D_i)$ the cost of pipe i with diameter D_i per unit length and L_i is the length of pipe i.

The objective function is to be minimized under the constraints of the steady state and transient conditions. For the steady state, the conservation of mass:

$$\sum_{j=1}^{M} Q_j = 0 \tag{2}$$

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

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:

$$\sum h_f = E_p \tag{3}$$

where h_f is the head loss due to friction in a pipe and expressed by the Darcy-Weisbach formula:

$$h_f = \frac{8}{\pi^2 g} f_i \frac{L_i\ Q_i^2}{D_i^5} \tag{4}$$

where Q_i is the pipe flow and f_i is the Darcy-Weisbach friction factor.

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

$$D_{\min} \le D_i \le D_{\max} \qquad i = 1,\ldots,N \tag{5}$$

$$H_{j,\min ST} \le H_j \le H_{j,\max ST} \qquad j = 1,\ldots,M \tag{6}$$

where D_i are the discrete pipe diameters selected from the set of commercially available pipe sizes, H_j is the pressure head at node *j*, $H_{j,\min ST}$ and $H_{j,\max ST}$ are the minimum and maximum allowable pressure heads at node *j* for the steady state.

The water hammer computations in closed conduits are based on the continuity equation and the equation of motion, Wylie et al. [1]:

$$\frac{\partial H}{\partial t} + \frac{a^2}{g A}\frac{\partial Q}{\partial x} = 0 \tag{7}$$

$$\frac{1}{g A}\frac{\partial Q}{\partial t} + \frac{\partial H}{\partial x} + \frac{R}{\Delta x} Q\left|Q\right|^{n-1} = 0 \tag{8}$$

in which *Q* is the fluid discharge, *H* the piezometirc head above arbitrary datum, *D* the inside pipe diameter, *A* the cross-sectional area of the pipe, *g* gravitational acceleration, *a* wave speed, *x* distance along the pipe axis and *t* time. For the Darcy-Weisbach formula, the friction term *R* is represented as:

$$R = f\,\Delta x / 2\,g D A^2, \quad n = 2 \tag{9}$$

The hydraulic constraints is given as:

$$H_{k,\min TR} \le H_k \le H_{k,\max TR} \qquad k = 1,\ldots, M_p \tag{6}$$

where $H_{k,\min TR}$ and $H_{k,\max TR}$ 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.

The Newton-Raphson method, [16], is used to simulate hydraulically the given network for the steady state and the water hammer analysis is implemented by a method of characteristics solution of the governing partial differential equations (Wylie et al. [1]).

GENETIC ALGORITHM (GA)

Genetic algorithms are nature based stochastic computational techniques. The major advantages of these algorithms are their broad applicability, flexibility and their ability to find optimal or near optimal solutions with relatively modest computational requirements. GAs have proven useful in a variety of search and optimization problems in engineering, science and commerce (Goldberg [17]). The algorithms are based on the principle of the survival of the fittest, which tries to retain genetic information from generation to generation. The GA approach does not require certain restrictive conditions (e.g., continuity, differentiability to the second order, etc.); properties that can seldom be guaranteed for water distribution problems, particular under transient states.

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.

The micro-Genetic Algorithm (μGA), Krishnakumar [18], is a "small population" GA. In contrast to the Simple Genetic Algorithm, which requires a large number of individuals in each population (i.e., 30 - 200), the μGA uses a small population size.

The optimization program 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. A brief description of the steps in using GA for pipe network optimization, [19], 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.
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.*** 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), [20].
5. ***Transient analysis of each network.*** A transient analysis solver computes the transient pressure heads resulting from the pump power failure. 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.*** 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).
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.

These steps for the optimization of water network considering both steady state and transient conditions are illustrated in the flow chart of the program, Fig. 1. This program is an extension of the GANRnet computer program, Djebedjian et al. [20]. It has been developed to optimize pipe networks for steady state using the genetic algorithm approach.

The genetic algorithm in the GANRnet 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.

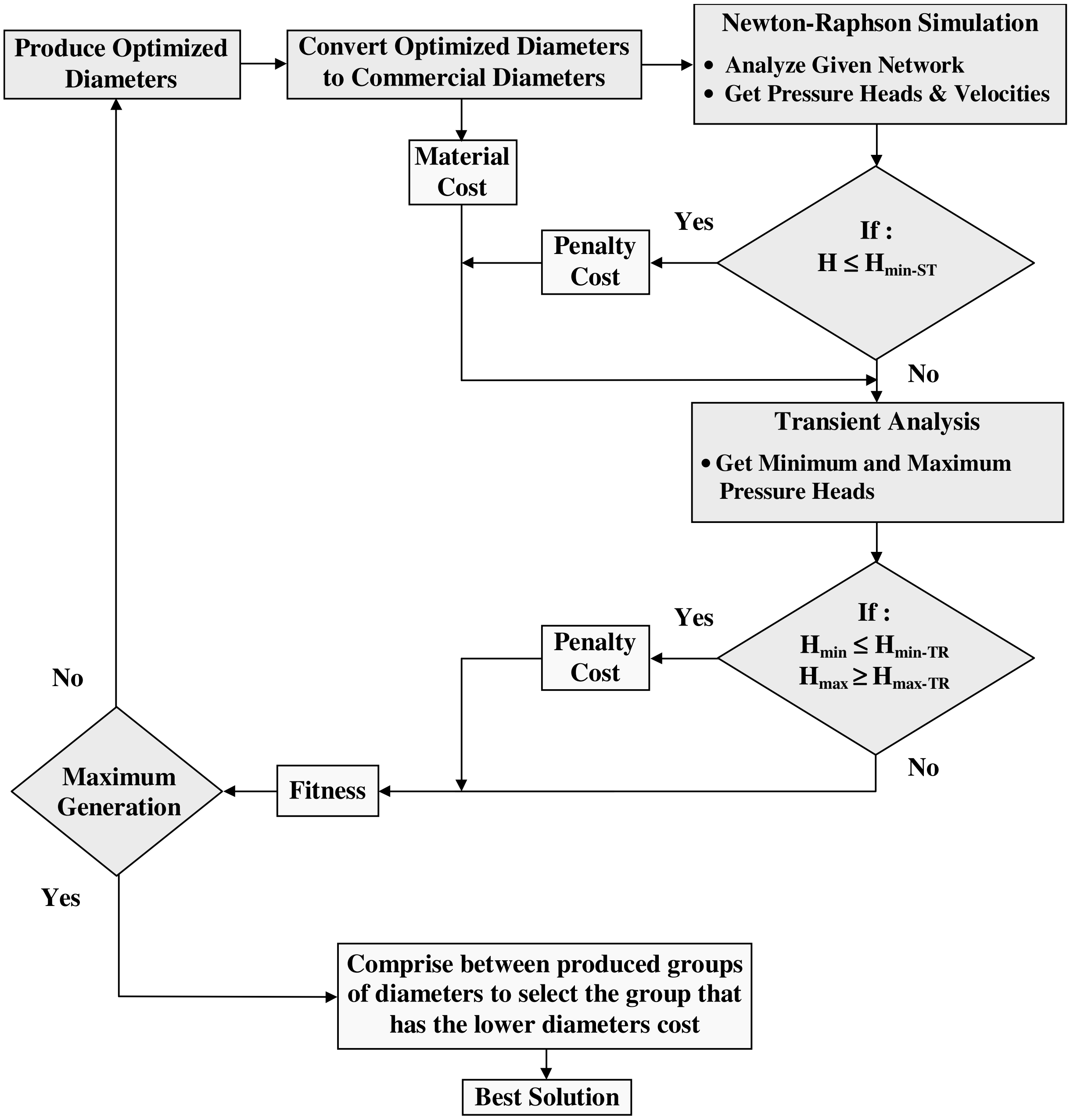


Fig. 1. Flow chart of the program

CASE STUDY: A Typical Piping Network

The case study is based on a pre-defined water supply piping network, Fig. 2, Larock et al. [21]. The system comprises two reservoirs at nodes 1 and 6, nine nodes and eleven pipes. The demands at nodes (3, 4, 5, 8 and 9) are (3, 2, 4, 1 and 2 ft^3/s), respectively. The lengths of the pipes are given in Table 1. Darcy-Weisbach friction factor and wave speed is 0.02 and 3300 ft/sec, respectively. In order to introduce transient conditions into the case study, a variety of possible causes could be selected. For convenience, a pump power failure is chosen to characterize the transient performance of the system.

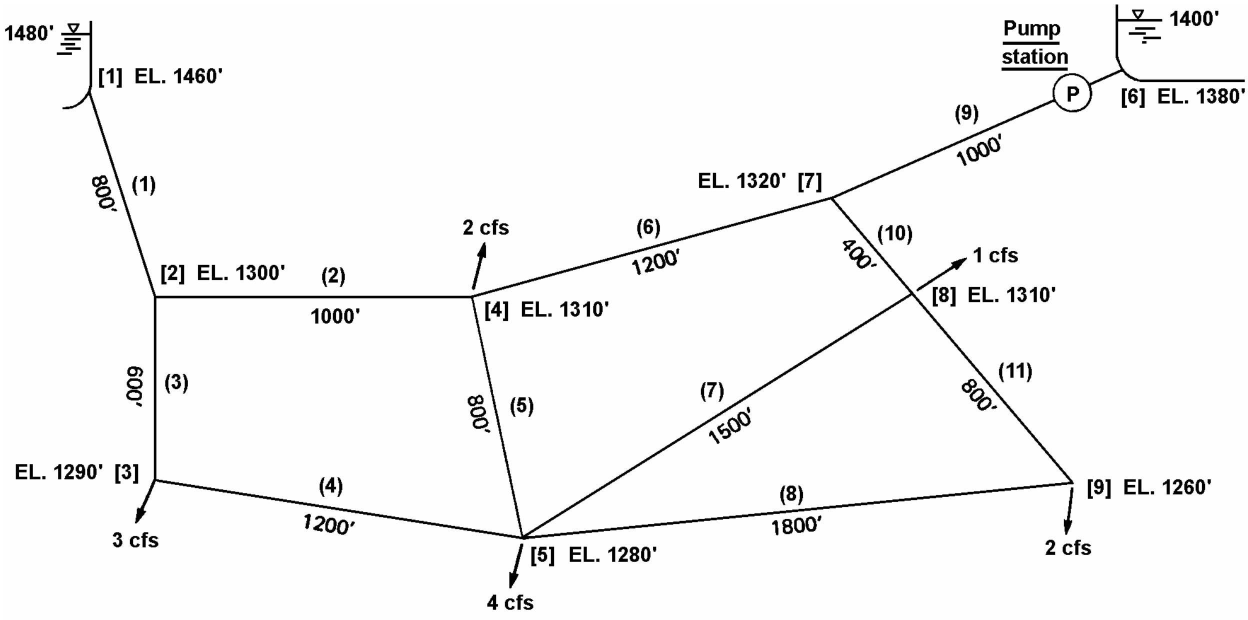


Fig. 2. Typical Piping Network

Table 1. Pipe data

Pipe Number	Start Node	End Node	Length (ft)
1	1	2	800
2	2	4	1000
3	2	3	600
4	3	5	1200
5	4	5	800
6	4	7	1200
7	5	8	1500
8	5	9	1800
9	6	7	1000
10	7	8	400
11	8	9	800

The set of commercially available pipe diameters (in inches) is (6, 8, 10, 12, and 15) and the corresponding cost per foot length is (15, 25, 35, 45, and 65), respectively. The Darcy-Weisbach friction factor is assumed to be 0.02 for all pipes. The wave speed for each pipe is 3300 ft/sec.

The pump performance curve for the pump in pipe 9 is defined by: $H_p = -0.5Q^2 - 0.3Q + 90$, with Q in ft^3/sec and H_p in ft, and the brake horsepower (in hp) is defined by: $Power = 1.2Q + 30$. The pump runs at 1750 rev/min, and the moment of inertia of pump and motor is 40 lb.ft^2.

The purpose of the optimization is to select the pipe diameters that for the steady state ensure the pressure head at the nodes is greater than the minimum allowable pressure head, and for the transient conditions, the pressures at all the parts into which the pipe is divided are in the allowable pressure head range.

The practical design of water networks recommends that the minimum pressure in the steady state conditions should not be less than 20 psi for proper operation. Transient pressure must be ranged from 20 to 100 psi, sufficient to overcome friction losses in piping. Pressures in excess of 100 pounds are not suitable.

For the case study and according the given configuration, the required minimum pressure head at all nodes is given the value of 80 ft for the steady state and for the transient conditions, the minimum and maximum pressure heads are given the values of 80 ft and 180 ft, respectively.

The optimization program was applied to the network using the following values for μGA parameters: *Npopsiz* = 5, *Idum* = −5000, *Maxgen* = 400 and *Nposibl* = 16. The mutation and crossover rates were set to 0.2 and 0.5, respectively. For the steady state, the accuracy for the calculations was 0.001 ft^3/s. The time of the transient flow simulation was taken as 40 sec and the hydraulic time step Δt was 0.04 sec.

Figure 3 depicts the evolution of the solution as the program develops in a single run. A rapid decrease in the cost value for the first group of evaluation then quite slow changes in the later evaluations is observed.

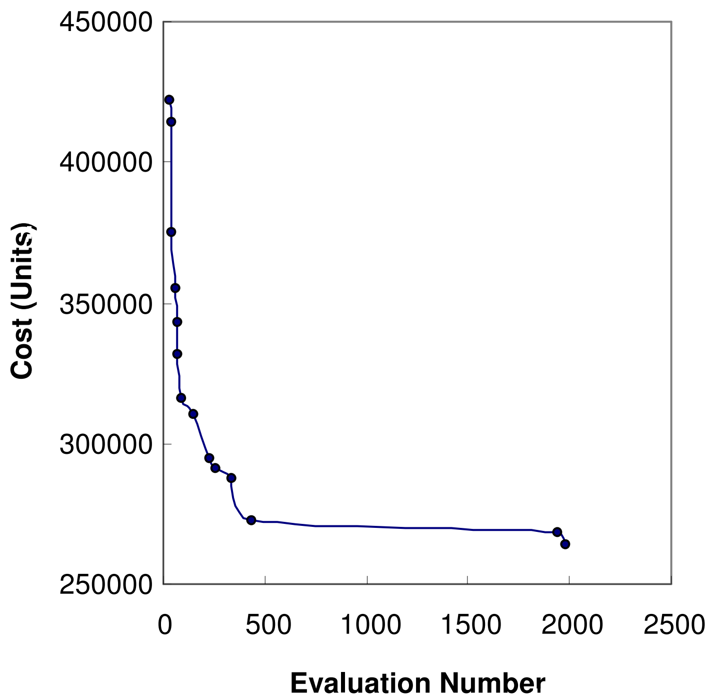


Fig. 3. Cost units versus evaluation number

The network containing 11 pipes and with 5 available commercial pipe sizes has a total solution space of $5^{11} = 4.88 \times 10^7$ different network designs. Using the GA optimization techniques, the number of function evaluations was 1976 to reach the optimal solution and this is only a very small fraction of the total search space (0.004%).

Table 2 shows the optimal diameters for the network. The least cost is 264,500 units. The run time was 13 minutes and produced on a computer with Pentium 3 (550 MHz) processor.

At the optimal diameters, the hydraulic analysis for the steady state gives the flows through the pipes. The total demands at nodes 3, 4, 5, 8 and 9 are 12 ft^3/sec. These are provided by the pump and the reservoir with an elevation of 1480 ft. The pump discharge is 1.61 ft^3/sec while the flow rate in pipe 1 connected to the reservoir is 10.39 ft^3/sec. The corresponding head that added by the pump at this discharge is 87.5 ft.

Table 2. Optimal diameters (in) and cost (units)

Pipe Number	**Diameter (in.)**
1	15
2	12
3	10
4	6
5	10
6	6
7	6
8	6
9	6
10	6
11	6
Cost	**264,500**
Run Time	**13 min**

Table 3 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. The two reservoir nodes 1 and 6 have heads of 20 ft and 107.5 ft, respectively. For node 1, this is the reservoir water level and for node 6, the pressure head is the sum of the pump head and the water level in the reservoir.

Table 3. Pressure heads at nodes for the steady state

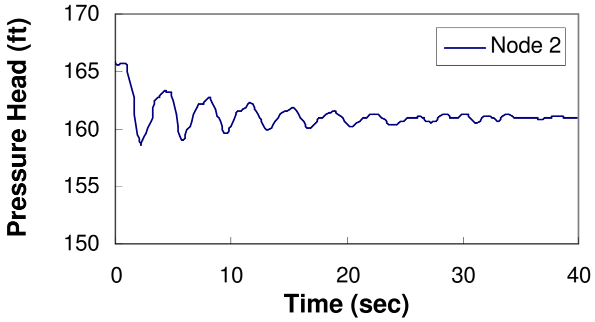
Node	Pressure Head (ft)
1	20
2	165.76
3	163.02
4	135.95
5	148.95
6	107.50
7	125.49
8	115.34
9	148.17

Figure 5 shows the pressure head versus time response at all nodes excluding the reservoir nodes. The heads at zero second corresponds to the steady state condition, Table 3, before the pump power failure. Examining Fig. 5, it can be observed that the convergence to steady state associated with the pump power failure is rapid. Also, these heads are within the specified range of heads given by the minimum and maximum pressure constraints of 80 ft and 180 ft. It is obvious that the pressure heads at the nodes near the pump (7, 8 and 9) are more quantitatively affected by the pump power failure than the other nodes. The choice of the time of the transient flow simulation as 40 sec was sufficient to obtain nearly steady state condition at the end of this time. Increasing the time makes the computational effort for optimization more pronouncing.

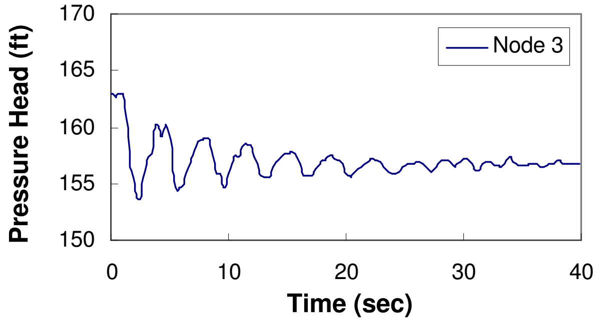
This study of pipeline optimization for steady state and transient conditions generated by the pump power failure shows the capability of the linkage between the optimization tool, the hydraulic analysis solver and the transient analysis solver to select the optimal diameters satisfying the given pressure constraints.

CONCLUSIONS

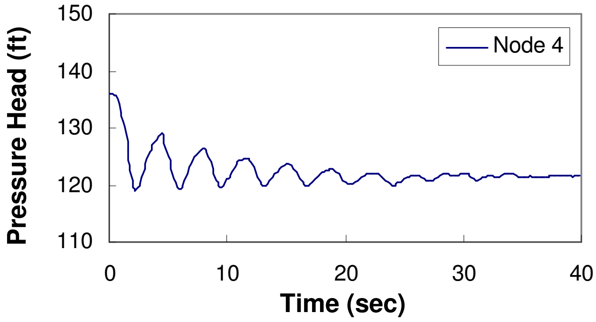
The optimization of a transient flow for water distribution systems is investigated recently. The previous studies were concerned with the optimization of networks under steady state conditions in spite of the fundamental importance of transients. In this study, the genetic algorithm GA is used as an optimization method to obtain the optimal diameters in a water distribution system for steady state and transient conditions. A linkage between the optimization tool, the hydraulic analysis solver and the transient analysis was achieved. The approach was applied on a case study of a network consisted of two reservoirs and a pump. The transient flow is introduced to the water system by the pump power failure.



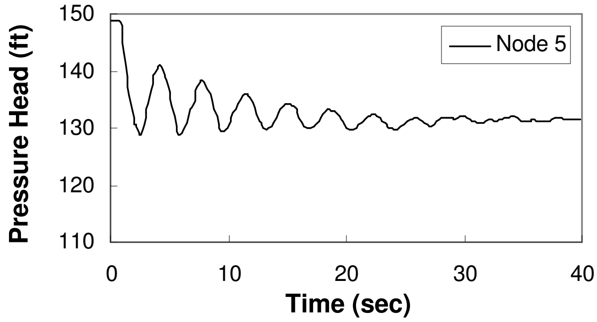
(a) Node 2



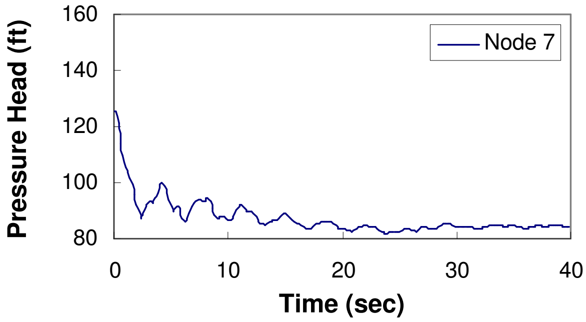
(b) Node 3



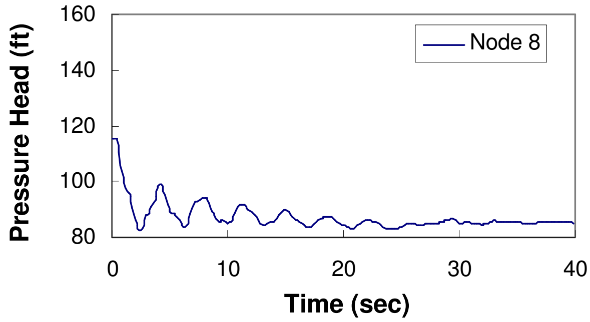
(c) Node 4



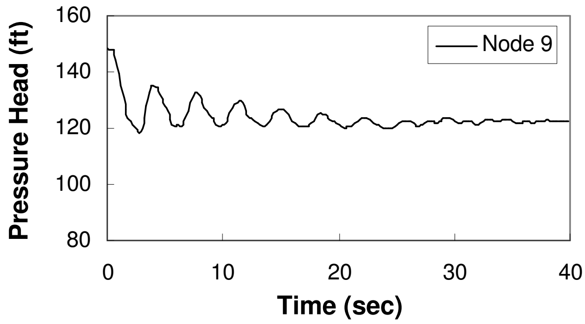
(d) Node 5



(e) Node 7



(f) Node 8



(g) Node 9

Fig. 4. Pressure head versus time for various nodes

The application of GA optimization tool to the case study 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 technique of the optimal pipe diameter selection is very economical as the network design can be achieved without using hydraulic devices for water hammer control. This technique is not only crucial to water networks design and performance, but also effective in minimizing costs.

NOMENCLATURE

A	cross-sectional area of the pipe (ft^2)
a	wave speed (ft/sec)
C_T	total cost
$c_i(D_i)$	cost of pipe *i* with diameter D_i per unit length
D_i	diameter of pipe *i*, (ft)
$D_{\max}$	maximum diameter, (ft)
$D_{\min}$	minimum diameter, (ft)
E_p	energy supplied by a pump, (ft)
f_i	Darcy-Weisbach friction factor of pipe *i*
g	gravitational acceleration, (ft/sec^2)
H_j	pressure head at node *j*, (ft)
$H_{j,\min ST}$	minimum allowable pressure head at node *j* for the steady state, (ft)
$H_{j,\max ST}$	maximum allowable pressure head at node *j* for the steady state, (ft)
$H_{k,\min TR}$	minimum allowable pressure head at node *k* for transient conditions, (ft)
$H_{k,\max TR}$	maximum allowable pressure head at node *k* for transient conditions, (ft)
h_f	head loss due to friction in a pipe, (ft)
Idum	initial random number seed for the GA run
L_i	length of pipe *i*, (ft)
M	total number of nodes in the network
M_p	number of parts into which the pipe is divided
Maxgen	maximum number of generations to run by the GA
N	total number of pipes
Npopsiz	population size of a GA run
Nposibl	array of integer number of possibilities per parameter
Q_i	fluid discharge in pipe *i*, (ft^3/sec)
Q_j	discharge into or out of the node *j*, (ft^3/sec)
R	friction term, Eq. (9)
t	time (sec)
x	distance along the pipe axis, (ft)

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