CHAPTER 12

NETWORK TRANSIENTS

12.1 INTRODUCTION

Both rigid column theory and elastic or water hammer analysis will be applied in this chapter to the solution of unsteady flows in pipe networks. Simplified solutions that ignore both inertial and elastic effects in pipe networks were covered in Chapter 6 under the name "extended time simulations."

What conditions require the full consideration of inertial effects, and what situations will make the elastic properties of the liquid and pipe so important that a full water hammer network analysis is necessary? When is an extended time simulation sufficient? There are no precise answers for these questions. The next few paragraphs mention some relevant factors in making such a decision, but in the end professional judgment and personal experience are also factors.

An elastic analysis is required whenever the changes in velocity are sufficiently rapid to cause substantial changes in the flow variables over time intervals that are less than several times the value of L/a for the pipe(s) under investigation. Examples are the rapid closure of a valve, the filling of a pipeline with liquid that moves at high velocity and forces air from the lines, an abrupt change in the operation of pumps, and in general any event that is sufficiently rapid to prevent the fluid throughout the network from gradually accommodating the change. However, the occurrence of a rapid change in a single pipe does not necessarily mean that an elastic analysis of the entire network is warranted. When demands are changing throughout a large distribution system, large pressure changes will alter the system demands so the pressure wave is rapidly absorbed. In this case the need for an elastic analysis may be restricted to that pipe, or possibly to it and a few nearby pipes.

Rigid column theory can be applied to situations in which the demands on an elastic pipe network change rather rapidly but not instantaneously, causing the inertial effects in accelerating the liquid to have a significant effect on the pressure. Examples are found during the morning hours in a large city when additional pumps must accommodate relatively rapid increases in demand, or whenever a major user may shut down rapidly. These changes in demand are not so rapid that elastic effects become significant, yet the effect of accelerating the fluid, owing to the long pipelines that exist between the supply sources and the demand sites, can cause the pressures far downstream in a distribution system to be significantly different than would be the case if only fluid friction were considered.

If both inertial and elastic effects can be ignored, then a quasi-static or extended time simulation would be valid for much of the operation of a water distribution system.

12.2 RIGID-COLUMN UNSTEADY FLOW IN NETWORKS

12.2.1. THE GOVERNING EQUATIONS

In the latter portion of Chapter 7 some unsteady flows in single pipes were studied. That theory assumed the liquid to be incompressible and the pipes to be rigid, thus ignoring the elastic properties of the liquid and the pipe. Here this same rigid column theory will be expanded to multiple-pipe systems. Here we ignore the convective acceleration term $V\partial V/\partial s$ for reasons discussed in Section 8.5.2. In the analysis of steady flows in networks it is also common practice to ignore the difference between the hydraulic grade line and the energy line by assuming they are coincident. This simplification is consistent with the deletion of the convective acceleration term, and it is standard practice in the application of rigid column theory, so long as velocities are low.

Equation 8.59, the equation of motion, can be written as

$$\frac{dV}{dt} = -\,g\frac{\partial H}{\partial s} - \frac{f}{2D}V|V| \tag{12.1}$$

Since $\partial H/\partial s$ is constant along a pipe, it can be expressed as $(H_j - H_i)/L_k$. The subscripts indicate that pipe k has an upstream node i and a downstream node j. Substituting one expression for the other in Eq. 12.1 gives

$$\frac{dV_k}{dt} = g\frac{H_i - H_j}{L_k} - \frac{f_k V_k|V_k|}{2D_k} \tag{12.2}$$

in which the subscript k has been added to the velocity V, the diameter D, and the friction factor f, to show the equation applies to pipe k in the system. Usually it is more convenient to use the discharge $Q = VA$ as a dependent variable in place of V; then Eq. 12.2 can be written as

$$\frac{dQ_k}{dt} = gA_k\frac{H_i - H_j}{L_k} - \frac{f_k Q_k|Q_k|}{2D_k A_k} \tag{12.3}$$

For unsteady flows Eq. 12.3 relates the time-varying discharge in pipe k, the frictional loss, and the instantaneous heads at the end nodes of the pipe. If $dQ_k/dt = 0$ so the flow is steady, we recover from Eq. 12.3 the Darcy-Weisbach equation itself. Thus Eq. 12.3 is the unsteady-flow analog of the Darcy-Weisbach equation, or an empirical equation such as the Hazen-Williams formula, for the relation between the frictional head loss and the discharge.

The junction continuity equations must also be satisfied for unsteady flows. Therefore, in addition to the equations that can be written by applying Eq. 12.3 to a network, NJ (or NJ - 1 if all external flows are specified) junction continuity equations must be written, one for each node, in the form

$$\sum Q_k - QJ_i = 0 \tag{12.4}$$

Here the summation includes all pipes that join at junction i, and QJ_i is the demand at this junction. In Eq. 12.4 the discharge is positive if it flows into junction i and negative if it flows from the junction.

12.2.2. THREE-PIPE PROBLEM

We begin by describing how Eqs. 12.3 and 12.4 can be used to model the unsteady flow in a small network. For this example we select the three-pipe network in Fig. 12.1. Since all external flows are specified, there are NJ - 1, or 2, junction continuity equations for this network. These continuity equations are

$$\begin{aligned} F_1 &= Q_1 - Q_2 - QJ_2 = 0 \\ F_2 &= Q_2 + Q_3 - QJ_3 = 0 \end{aligned} \tag{12.5}$$

These two equations require the negative demand QJ_1 at node 1 to equal the sum of the

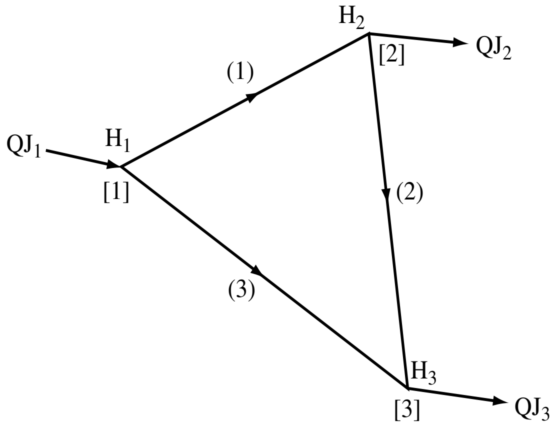


Figure 12.1 Three-pipe network.

positive demands QJ_2 at node 2 and QJ_3 at node 3. In addition, the following three ordinary differential equations apply, one for each pipe:

$$\begin{aligned}
\frac{dQ_1}{dt} &= gA_1 \frac{H_1 - H_2}{L_1} - \frac{f_1 Q_1 |Q_1|}{2D_1 A_1} \\
\frac{dQ_2}{dt} &= gA_2 \frac{H_2 - H_3}{L_2} - \frac{f_2 Q_2 |Q_2|}{2D_2 A_2} \\
\frac{dQ_3}{dt} &= gA_3 \frac{H_1 - H_3}{L_3} - \frac{f_3 Q_3 |Q_3|}{2D_3 A_3}
\end{aligned} \qquad (12.6)$$

In this network we assume that the head H_1 is constant since the fluid is supplied from a reservoir. Therefore, if the demands QJ_2 and QJ_3 are specified for all time, then the five variables Q_1, Q_2, Q_3, H_2, and H_3 are the unknown variables in this network.

To determine five unknown variables, we must have five independent equations. In this problem Eqs. 12.5 and 12.6 satisfy this requirement. But how can this system of equations be solved when some of the equations are ordinary differential equations (ODEs) rather than algebraic equations? If a solution can be found, it must then be applied repeatedly as time advances. Thus such a solution is far more than a single steady-flow solution. Instead the solution process must be repeated over and over until the results are known over a sufficiently long time period to satisfy our need for knowledge of the behavior of this system. To simulate the performance of a water main network over a twenty-four-hour period, say at 10 second intervals, would require 8640 incremental solutions. Obviously this is a task for a fast computer if the network is very large. But usually such unsteady-flow solutions are only required for much shorter time intervals, over which rapid changes occur.

With the correct approach, the Newton method will allow us to solve a set of algebraic and ordinary differential equations simultaneously. As a first step in this method, the equations are each equated to zero, as shown:

$$F_1 = Q_1 - Q_2 - QJ_2 = 0$$
$$F_2 = Q_2 + Q_3 - QJ_3 = 0$$
$$F_3 = Q_1 - Q_{ODE1} = 0 \tag{12.7}$$
$$F_4 = Q_2 - Q_{ODE2} = 0$$
$$F_5 = Q_3 - Q_{ODE3} = 0$$

In Eqs. 12.7 each of the variables is in general a function of time. At time $t = 0$ we assume all variables are known. These initial values are usually obtained by solving the steady-state network problem. The notation in the last three of Eqs. 12.7 has the following meaning: the Q with a subscript 1, 2, or 3 is the discharge in each pipe. Each Q with the ODE subscript is the discharge that is found from the solution of the first, second or third ordinary differential equation at the designated time. When these latter discharges equal the respective pipe flows, then the last three equations are clearly satisfied. However, Q_{ODE1}, Q_{ODE2}, and Q_{ODE3} are obtained by solving the corresponding ODEs numerically over the latest time increment from the last known solution to the new time instant. Thus the last three equations are expressions or functions, just as algebraic equations are expressions. The only difference is that much more algebra is required to evaluate each expression, since an ODE is solved for this purpose. The Newton method is a systematic and effective means of directing the solution process so convergence to the correct solution is obtained in relatively few iterations.

Often we cannot compute formally for every function all of the partial derivatives that are required in the process of solving the ODEs; then the evaluation of the elements of the Jacobian matrix is most conveniently done by using numerical approximations. This numerical evaluation can be accomplished in the same way as it was for algebraic equations, namely by evaluating the equation twice and dividing the difference of these two values by the increment of the unknown Δx_j for which the derivative is sought, or

$$\frac{\partial F_i}{\partial x_j} = \frac{F_i\left(x_1, x_2, \ldots x_j + \Delta x_j, \ldots x_n\right) - F_i\left(x_1, x_2, \ldots x_j, \ldots x_n\right)}{\Delta x_j} \tag{12.8}$$

To see the solution process in operation, we now solve the three-pipe network over 8 seconds in 2 second intervals. At $t = 0$ the flows are steady, and for simplicity let (1) the friction factors f for the three pipes retain their steady-state values, and (2) the demand at node 2 be constant in time, so only QJ_3 changes with time. The head $H_1 = 100$ ft at node 1 is also constant. The values in the following tables define the problem further:

Values at time = 0:

Pipe	***D* in.**	***L* ft.**	***e* in.**	**Node**	**QJ, ft^3/s**
1	8	2000	0.005	1	- 3.0
2	8	2400	0.005	2	1.5
3	8	3000	0.005	3	1.5

Variation of QJ_3 with time:

Time, sec	0	2	4	6	8
QJ$_3$, ft^3/s	1.5	2.0	2.5	3.0	3.5

First the network solution to the steady problem is obtained:

Pipe	Q ft^3/s	h_L ft	f	Node	HGL ft
1	1.652	20.10	0.0193	1	100.00
2	0.152	0.29	0.0270	2	79.90
3	1.348	28.16	0.0196	3	79.61

The program THREPIP.FOR for the solution of this problem can be found on the CD; the reader is encouraged to obtain a listing before reading further. The solution for this unsteady problem is summarized in Table 12.1:

Table 12.1 Unsteady Flow Solution

Step	Time sec	QJ_3 ft^3/s	Q_1 ft^3/s	Q_2 ft^3/s	Q_3 ft^3/s	H_2 ft	H_3 ft
0	0.0	1.500	1.652	0.152	1.348	79.90	79.61
1	2.0	2.000	1.860	0.360	1.640	58.63	35.60
2	4.0	2.500	2.076	0.576	1.924	51.98	26.09
3	6.0	3.000	2.299	0.799	2.201	44.57	14.80
4	8.0	3.500	2.527	1.027	2.473	36.49	1.72

Problem 12.1 (end of chapter) seeks the solution of four analogous steady-flow problems that can be used to study the differences between steady and unsteady network-flow behavior. That comparison will show that inertial effects lower the head at node 3 in much the same way as do steady-state frictional losses. For networks with rapidly-changing nodal demands we conclude that inertial effects must be included in an analysis.

Some study of the listing of THREPIP.FOR to understand the program structure can be valuable. This program calls two standard programs, DVERK from IMSL (see Appendix A) to obtain the solution to the ODEs, and SOLVEQ to solve the linear equation system that is the result of implementing the Newton method. The main program first sets up the problem by obtaining input data from the user, and then it implements the Newton method by defining the Jacobian matrix and the equation vector for each iteration. Then it subtracts the latest linear solution from the previous vector of unknowns to improve the solution. A solution is sought for each time step within the `DO 60` loop, which prompts the user to supply new values for QJ_3. The main program calls subroutine DEFFUN to evaluate the five functions in Eqs. 12.7; the first two functions are the continuity equations, and the other three functions are obtained by calling DVERK. The actual derivatives are computed in subroutine SLOPE, which DVERK calls. After DVERK solves the three ODEs over the latest time increment, subroutine DEFFUN defines functions F(3), F(4), and F(5).

The program assumes constant friction factors. This assumption is questionable in the example because the small discharge in pipe 2 caused the friction factor $f_2 = 0.027$ to be larger than the others. Some program changes would allow the friction factors to be computed from the Colebrook-White Equation. Instead of assigning these factors as input data, the Colebrook-White equation could be solved in subroutine SLOPE to provide a friction factor for each initial discharge.

In the example only the demand at node 3 was a function of time primarily because the effects of inertia on the nodal heads could then be more easily understood. Changes in input data would allow both QJ_2 and QJ_3 to vary with time. The heads H_2 and H_3 might alternatively have been specified as functions of time with the demands QJ_2 and QJ_3 being considered as unknown variables, along with Q_1, Q_2, and Q_3. In this case the set of five governing equations would be unchanged, but the computer program to solve the problem must then to be modified to indicate correctly which variables are known and which are unknown.

When a steady network problem is solved by using the Q-equations or the ΔQ-equations, the nodal heads were then computed later as secondary dependent variables, as discussed in earlier chapters. This sequential process no longer is successful for unsteady network analyses, for the discharges and heads are now coupled. The computations are more extensive because ODEs have replaced algebraic equations, and discharges and heads are wanted at numerous time increments.

12.3 A GENERAL METHOD FOR RIGID-COLUMN UNSTEADY FLOW IN PIPE NETWORKS

12.3.1. THE METHOD

The solution methodology that was applied in the unsteady-flow analysis of the three-pipe network will now be described in general terms so it can be used to analyze any network having NP pipes and NJ nodes. We assume that the network has at least two supply sources. For such networks the number of independent simultaneous equations consists of NJ junction continuity equations and NP ODEs that govern the rigid-column unsteady flow in the pipes. These equations are

NJ junction continuity equations $$\sum Q_k - QJ_i = 0 \tag{12.9}$$

NP ODEs $$\frac{dQ_k}{dt} = gA_k \frac{H_i - H_j}{L_k} - \frac{f_k Q_k |Q_k|}{2D_k A_k} \tag{12.10}$$

in which one head is the water surface elevation of a reservoir if this pipe connects the reservoir to the network. This equation system will allow NP + NJ unknowns to be determined. For the analysis problem these unknowns are NP discharges in NP pipes and the heads at NJ nodes. This set of unknowns assumes that the time-dependent nodal demands are specified. But one might alternatively specify heads as functions of time at the nodes, and then the demands at the nodes would replace the heads as the unknowns. In fact it is possible to mix the specification of demands and heads as functions of time. For any node where QJ is specified as a function of time, the head must be unknown at that node, and at any node where the head is specified as a function of time, the demand QJ there must be an unknown.

The continuity equations are linear and are identical to those that would be written for any steady-state analysis. The equations of motion for the fluid in the pipes, i.e. the ODEs, must be appropriately solved over the time increment for which new information is wanted, and these solutions must also relate the discharges and heads to each other at the nodes of the network. To solve this system of algebraic and ordinary differential equations, any iterative method could in principle be used, but we prefer to use Newton's method. When applying the Newton method to the ODEs, functions are created that are simply the difference between the current discharge value in the pipe and the value that is found by solving the ODE for this pipe over the present time increment. In short, we create functions of the form

$$F_i = Q_k - Q_{ODE} = 0 \tag{12.11}$$

This set of equations must be solved for each new time increment.

Thus the process of obtaining an unsteady solution to a problem in which demands or heads are specified functions of time consists of seven tasks:

1. The time span, over which the unsteady solution is to be obtained, is divided into NT time increments or steps.
2. The discharges in all pipes and the heads at all nodes are assigned initial values that are chosen from a steady state solution that has the same demands, etc. as the unsteady solution has at time zero. (In place of a steady state solution, this initialization may be obtained from the last time-step solution from a prior unsteady-flow solution.)
3. All demands over each time increment must be specified.
4. Over each new time increment define and evaluate the functions (identify the equations to be solved, and substitute the current value of each variable into them if they are algebraic, or solve the ODE using the current values of all variables) and the Jacobian 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 heads for this time increment, and then repeat steps 3 through 7 until the unsteady solution spans the entire time period.

12.3.2. AN EXAMPLE

Figure 12.2 depicts a network with 19 pipes and 12 nodes. The nodal demands sum to 10.3 ft^3/s, and this discharge must come from the two reservoirs. The steady state solution to this network is listed in Tables 12.2. As in the tables, all pipe diameters are in inches and lengths in feet. The largest head loss, 24.3 ft, occurs in pipe 1 that supplies

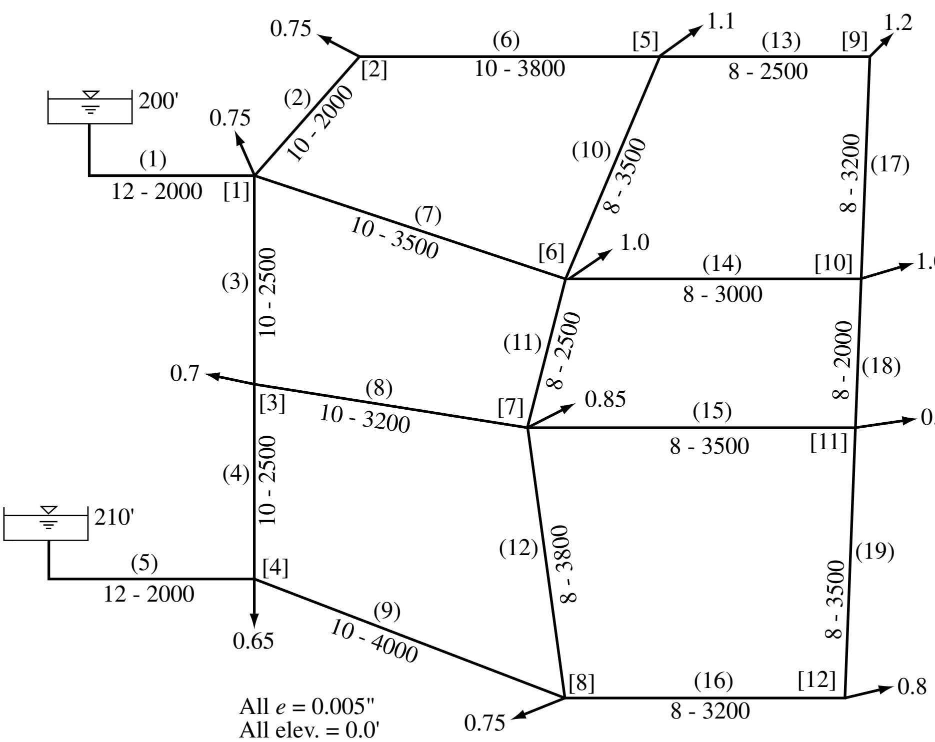


Figure 12.2 Typical pipe network.

the network from a reservoir, and the third largest head loss, 21.7 ft, occurs in the other reservoir supply line, pipe 5. The steady problem was solved by using seven ΔQ-equations around loops. Six of these loops are real loops, and one is a pseudo loop that connects the two reservoirs through a sequence of connected pipes.

Tables 12.2 Steady-flow solution, 19-pipe network.

PIPE DATA

PIPE NO.	NODES FROM	TO	L ft.	DIA. in	e x10^3 in	Q ft^3/s	VEL. ft/s	HEAD LOSS ft.	HLOSS /1000
1	0	1	2000.	12.0	5.0	5.30	6.75	24.26	12.13
2	1	2	2000.	10.0	5.0	2.17	3.98	10.96	5.48
3	1	3	2500.	10.0	5.0	0.41	0.74	0.60	0.24
4	4	3	2500.	10.0	5.0	2.13	3.90	13.19	5.27
5	0	4	2000.	12.0	5.0	5.00	6.37	21.67	10.84
6	2	5	3800.	10.0	5.0	1.42	2.61	9.29	2.44
7	1	6	3500.	10.0	5.0	1.97	3.62	15.95	4.56
8	3	7	3200.	10.0	5.0	1.83	3.36	12.70	3.97
9	4	8	4000.	10.0	5.0	2.22	4.07	22.89	5.72
10	6	5	3500.	8.0	5.0	0.55	1.56	4.30	1.23
11	7	6	2500.	8.0	5.0	0.50	1.44	2.65	1.06
12	8	7	3800.	8.0	5.0	0.43	1.23	3.00	0.79
13	5	9	2500.	8.0	5.0	0.87	2.48	7.34	2.94
14	6	10	3000.	8.0	5.0	0.93	2.66	10.06	3.35
15	7	11	3500.	8.0	5.0	0.91	2.61	11.29	3.23
16	8	12	3200.	8.0	5.0	1.04	2.99	13.32	4.16
17	10	9	3200.	8.0	5.0	0.33	0.95	1.59	0.50
18	11	10	2000.	8.0	5.0	0.40	1.16	1.41	0.70
19	12	11	3500.	8.0	5.0	0.24	0.69	0.97	0.28

AVE. VEL. = 2.80 ft/s, AVE. HL/1000 = 3.625, MAX. VEL. = 6.75 ft/s, MIN. VEL. = 0.69 ft/s

NODE DATA

NODE	DEMAND ft^3/s	DEMAND gal/min	ELEV. ft.	HEAD ft.	PRESSURE lb/in^2	HGL ELEV. ft.
1	0.75	336.6	0.0	175.74	76.15	175.74
2	0.75	336.6	0.0	164.78	71.40	164.78
3	0.70	314.2	0.0	175.14	75.89	175.14
4	0.65	291.7	0.0	188.33	81.61	188.33
5	1.10	493.7	0.0	155.49	67.38	155.49
6	1.00	448.8	0.0	159.79	69.24	159.79
7	0.85	381.5	0.0	162.44	70.39	162.44
8	0.75	336.6	0.0	165.44	71.69	165.44
9	1.20	538.6	0.0	148.15	64.20	148.15
10	1.00	448.8	0.0	149.74	64.89	149.74
11	0.75	336.6	0.0	151.15	65.50	151.15
12	0.80	359.1	0.0	152.11	65.92	152.11

AVE. HEAD = 162.4 ft., AVE. HGL = 162.4 ft.,
MAX. HEAD = 188.3 ft., MIN. HEAD = 148.2 ft.

The number of simultaneous equations to model this unsteady flow problem is 31, based on rigid column theory. Twelve of these are algebraic junction continuity equations, and the other 19 are ODEs that define the relation between the rates at which the discharges change and the slopes of the HGL in the pipes.

We will seek the solution of the unsteady problem in which all demands but one remain constant; at node 9 the demand gradually increases from 1.2 ft^3/s to 2.5 ft^3/s over 20 seconds, as outlined in the table:

Time, sec	0	5	10	15	20
Demand at node 9, ft^3/s	1.20	1.50	1.75	2.00	2.50

This increase in demand at node 9 of 1.3 ft^3/s over 20 seconds is approximately a doubling of the original demand and is typical of demand increases that might be expected in a network that has an average demand of 10.3 ft^3/s. An increased demand of 1.3 ft^3/s in 20 seconds could occur when a single major water user begins operation at the beginning of the work day. Thus this problem, while having relatively few pipes, can provide a basis for an evaluation of the importance of inertial effects during periods when one or more demands vary rapidly. The input data for the solution of the problem can be found in file EPB12F_2.IN on the CD.

Results for the unsteady-flow solution are tabulated in Tables 12.3a-d after each of four 5-sec time increments. Tables 12.3a-b list the discharge in each pipe after each of the 5-sec intervals. In these tables the results from this solution are also compared with results from four steady-state solutions; in each steady solution the demand at node 9 matches the unsteady-flow demand at this node at the end of the five-second interval. The first tables compare the discharges in the pipes, and Tables 12.3c-d compare the heads at the nodes. We find the steady and unsteady discharges differ only by small amounts.

Table 12.3a Discharge Comparison, ft^3/s.

Pipe	$QJ_9 = 1.50\ ft^3/s$ Time = 5 sec			$QJ_9 = 1.75\ ft^3/s$ Time = 10 sec		
	Unsteady	Steady	Difference	Unsteady	Steady	Difference
1	5.49	5.48	0.01	5.65	5.63	0.02
2	2.28	2.26	0.02	2.36	2.34	0.02
3	0.41	0.42	- 0.01	0.41	0.42	- 0.01
4	2.18	2.18	0.00	2.23	2.23	0.00
5	5.11	5.12	- 0.01	5.20	5.22	- 0.02
6	1.53	1.51	0.02	1.61	1.59	0.02
7	2.06	2.05	0.01	2.13	2.12	0.01
8	1.89	1.90	- 0.01	1.94	1.95	- 0.01
9	2.27	2.29	- 0.02	2.32	2.34	- 0.02
10	0.61	0.59	0.02	0.65	0.64	0.01
11	0.54	0.54	0.00	0.57	0.57	0.00
12	0.45	0.45	0.00	0.47	0.47	0.00
13	1.03	1.01	0.02	1.16	1.13	0.03
14	0.99	1.00	- 0.01	1.05	1.05	0.00
15	0.95	0.96	- 0.01	0.99	1.00	- 0.01
16	1.07	1.09	- 0.02	1.10	1.12	- 0.02
17	0.47	0.49	- 0.02	0.59	0.62	- 0.03
18	0.47	0.50	- 0.03	0.54	0.57	- 0.03
19	0.27	0.29	- 0.02	0.30	0.32	- 0.02
Absolute Averages			0.013			0.015

Table 12.3b Discharge Comparison, ft^3/s.

Pipe	$QJ_9 = 2.00\ ft^3/s$ Time = 15 sec			$QJ_9 = 2.50\ ft^3/s$ Time = 20 sec		
	Unsteady	Steady	Difference	Unsteady	Steady	Difference
1	5.80	5.78	0.02	6.12	6.09	0.03
2	2.44	2.42	0.02	2.61	2.57	0.04
3	0.41	0.43	- 0.02	0.42	0.44	- 0.02
4	2.28	2.28	0.00	2.37	2.37	0.00
5	5.30	5.32	- 0.03	5.48	5.51	- 0.03
6	1.69	1.67	0.02	1.86	1.82	0.04
7	2.20	2.19	0.01	2.34	2.32	0.02
8	1.99	2.01	- 0.02	2.09	2.11	- 0.02
9	2.37	2.39	- 0.02	2.46	2.49	- 0.03
10	0.69	0.68	0.01	0.79	0.77	0.02
11	0.60	0.60	0.00	0.66	0.67	- 0.01
12	0.48	0.48	0.00	0.52	0.52	0.00
13	1.28	1.25	0.03	1.54	1.50	0.04
14	1.10	1.11	- 0.01	1.21	1.22	- 0.01
15	1.02	1.04	0.02	1.10	1.11	- 0.01
16	1.14	1.16	- 0.02	1.20	1.22	- 0.02
17	0.71	0.75	- 0.04	0.96	1.00	- 0.04
18	0.61	0.64	- 0.03	0.74	0.78	- 0.04
19	0.34	0.36	- 0.02	0.40	0.42	- 0.02
Absolute Averages			0.019			0.024

Table 12.3c Head Comparison, ft.

Node	$QJ_9 = 1.50\ ft^3/s$ Time = 5 sec			$QJ_9 = 1.75\ ft^3/s$ Time = 10 sec		
	Unsteady	Steady	Difference	Unsteady	Steady	Difference
1	171.70	174.10	- 2.40	170.71	172.69	- 1.98
2	157.76	162.25	- 4.49	156.36	160.06	- 3.70
3	171.16	173.47	- 2.31	170.04	172.05	- 2.01
4	186.18	187.32	- 1.14	185.51	186.46	- 0.95
5	143.17	151.82	- 8.65	141.53	148.60	- 7.07
6	151.41	156.88	- 5.47	149.95	154.37	- 4.42
7	155.91	159.89	- 3.98	154.44	157.73	- 3.29
8	160.35	163.15	- 2.80	158.94	161.22	- 2.28
9	127.13	142.09	- 14.96	124.26	136.55	- 12.29
10	137.25	145.37	- 8.12	134.95	141.61	- 6.66
11	141.56	147.43	- 5.87	139.48	144.29	- 4.81
12	144.74	148.75	- 4.01	142.71	145.91	- 3.20
Averages			- 5.35			- 4.39

The largest difference in discharges occurs at 20 sec when the average absolute difference in discharges is 0.024 ft^3/s, with individual differences no larger than 0.04 ft^3/s. In Tables 12.3c-d where head solutions are compared, the difference should probably be examined in comparison with the frictional losses. From the steady solutions for this network we find the head losses in pipes 13 and 17 that supply node 9 are 7.34 and 1.59 ft, respectively. The largest difference in heads between the unsteady and steady solutions is 23.47 ft, which is approximately five times the average frictional head loss in the pipes that supply node 9. It is therefore obvious that an extended time simulation would be quite inadequate in determining the transient pressure distribution, especially near the location (node 9) of a rapid change in demand over a short time period (20 sec).

Table 12.3d Head Comparison, ft.

Node	$QJ_9 = 2.00\ ft^3/s$ Time = 15 sec			$QJ_9 = 2.50\ ft^3/s$ Time = 20 sec		
	Unsteady	Steady	Difference	Unsteady	Steady	Difference
1	169.28	171.25	- 1.97	164.38	168.25	- 3.87
2	154.12	157.80	- 3.68	145.85	153.06	- 7.21
3	168.58	170.59	- 2.01	163.62	167.57	- 3.95
4	184.64	185.59	- 0.95	181.91	183.79	- 1.88
5	138.24	145.25	- 7.01	124.37	138.14	- 13.77
6	147.39	151.80	- 4.41	137.76	146.44	- 8.68
7	152.24	155.52	- 3.28	144.46	150.96	- 6.50
8	157.00	159.26	- 2.26	150.69	155.24	- 4.55
9	118.43	130.57	- 12.14	93.84	117.31	- 23.47
10	131.09	137.75	- 6.66	116.50	129.65	- 13.14
11	136.28	141.09	- 4.81	124.90	134.51	- 9.61
12	139.88	143.04	- 3.16	130.71	137.19	- 6.48
Averages			- 4.36			- 8.59

12.4 SEVERAL PUMPS SUPPLYING A PIPE LINE

A common network problem concerns several pumps that deliver discharges to a single pipeline. As an example, Fig. 12.3 shows two pumps. In general the number of pipes containing pumps is NPUMPS, and the number of pipes is NP = NPUMPS + 1 in the system. Often the supply source water surface elevations are identical for all pumps, but

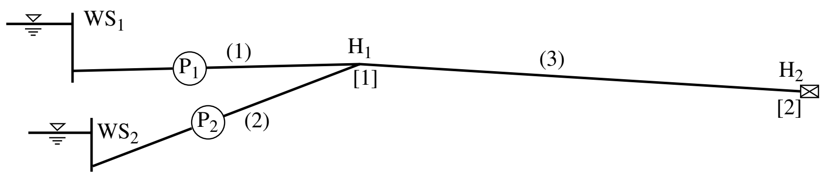


Figure 12.3 Multiple Pumps.

for generality each supply water surface elevation is denoted by WS_i, $i = 1, 2 \ldots$ NPUMPS, and each pipe containing a pump is also indexed by i. There will always be two nodes in such systems, node 1 where the pipes containing the pumps join with the downstream pipe, and node 2 at the other end of this pipe. The downstream pipe will be numbered NP; at its downstream end there will be a valve to control the discharge, or in its place there may be some other type of boundary condition. To allow several possibilities, we assume at node 2 that either the discharge or the head is specified as a function of time.

The equations to model the network behavior are therefore (1) one junction continuity equation at node 1 that indicates that the sum of the discharges in the pipes containing the pumps must equal the discharge in pipe NP, and (2) an ordinary differential equation for each of NP pipes. Thus the number of unknowns that can be found is NE = NP + 1. In the example we have 4 equations and 4 unknowns. If the discharge in pipe 3 is specified, then these unknowns will be Q_1, Q_2, H_1, and H_2. But if H_2 is specified, then the unknowns are Q_1, Q_2, Q_3, and H_1. The equations can be written as

$$F_1 = \sum Q_i - Q_{NP} = 0 \qquad i = 1,\ldots, NPUMPS$$
$$F_{i+1} = Q_i - Q_{ODEi} = 0 \qquad i = 1,\ldots, NP \tag{12.12}$$

in which Q_{ODEi} is obtained by solving the unsteady ODE with the discharge as the dependent variable. For the pipes containing the pumps this ODE will be

$$\frac{dQ_i}{dt} = gA_i \frac{WS_i + h_{pi} - H_1}{L_i} - \frac{\left(f_i L_i / D_i + K_e\right) Q_i |Q_i|}{2 D_i A_i} \tag{12.13}$$

in which the pump head can be given by the usual second-order polynomial equation. If this representation is used for h_p, then

$$h_{pi} = (A_i Q_i + B_i) Q_i + C_i \tag{12.14}$$

For the last pipe, numbered NP, the ODE is

$$\frac{dQ_{NP}}{dt} = gA_{NP} \frac{H_1 - H_2}{L_{NP}} - \frac{\left(f_{NP} L_{NP} / D_{NP}\right) Q_{NP} |Q_{NP}|}{2 D_{NP} A_{NP}} \tag{12.15}$$

In our 2-pump, 3-pipe system the equation system becomes

$$F_1 = Q_1 + Q_2 - Q_3 = 0 \tag{12.16a}$$

$$F_2 = Q_1 - Q_{ODE1} = 0, \qquad \frac{dQ_1}{dt} = gA_1 \frac{WS_1 + h_{p1} - H_1}{L_1} - \frac{\left(f_1 L_1 / D_1 + K_e\right) Q_1 |Q_1|}{2 D_1 A_1} \tag{12.16b}$$

$$F_3 = Q_2 - Q_{ODE2} = 0, \qquad \frac{dQ_2}{dt} = gA_2 \frac{WS_2 + h_{p2} - H_1}{L_2} - \frac{\left(f_2 L_2 / D_2 + K_e\right) Q_2 |Q_2|}{2 D_2 A_2} \tag{12.16c}$$

$$F_4 = Q_3 - Q_{ODE3} = 0, \qquad \frac{dQ_3}{dt} = gA_3 \frac{H_1 - H_2}{L_3} - \frac{\left(f_3 L_3 / D_3\right) Q_3 |Q_3|}{2 D_3 A_3} \tag{12.16d}$$

The program PUMPPAR on the CD solves problems of this type. As the listing shows, this program consists of the main program, a subroutine FUNCT which inserts equation values into array F when it is called, and a subroutine DQT that evaluates the derivative *dQ/dt* when it is called by the ODE solver. The program permits either the discharge or the head at the downstream node to be given as a function of time.

Example Problem 12.1

The flows from two pumps (with operating characteristics defined by tabular data which follow) are combined into a single pipe line, as diagrammed in Fig. 12.3. The supply water surface elevations are $WS_1 = 80$ m and $WS_2 = 70$ m; assume also $\nu = 1.31\times10^{-6}$ m^2/s and $e = 0.5$ mm. Additional pipe data are listed in the following table:

Pipe	1	2	3
L, m	500	400	2000
D, mm	400	400	500

Determine the unsteady discharges in the pipes if (a) the discharge in pipe 3 is given by

Time sec	0.5	1.0	2.0	3.0	4.0	5.0	7.5	10	15	20	25	30	35
Q_3 m^3/s	0.65	0.60	0.55	0.50	0.45	0.40	0.35	0.30	0.25	0.20	0.15	0.10	0.05

and (b) if the head at node 2 is given by

Time sec	0.5	1.0	2.0	3.0	4.0	5.0	7.5	10	15	20	25	30	35
H_2 m	70	80	90	100	100	100	90	80	70	60	60	60	60

Assume prior to time zero that the head at the downstream end of pipe 3 is 60 ft.

Pump 1		Pump 2	
Q, m^3/s	h_p, m	Q, m^3/s	h_p, m
0.25	25	0.25	30
0.35	23	0.35	28
0.45	20	0.45	25

We must first solve the steady-flow problem to provide the initial condition for the unsteady problem. This solution can be obtained from NETWK, which is $Q_1 = 0.334$ m^3/s, $Q_2 = 0.357$ m^3/s, $H_1 = 91.5$ m, and $H_2 = 60$ m. Fitting second-order polynomials to the pump data yields $h_{p1} = 25.625 + 10Q_1 - 50Q_1^2$, and $h_{p2} = 30.625 + 10Q_2 - 50Q_2^2$. The input data to solve part (a) of this problem using program PUMPPAR follows:

```
3 2 0.000001 0.0001 0.0005 9.81 1.31E-6
0.4 500
0.4 400
0.5 2000
- 50 10 25.625 80 0.334
- 50 10 30.625 70 0.357
91.5 60
1   (to indicate Q3 will be specified)
13 .5 .65 1 .6 2 .55 3 .5 4 .45 5 .4 7.5 .35 10 .3 15 .25 20 .2 25 .15 30 .1 35 .05
```

The solution for part (a) can be found in file EPB12_1.OU1 on the CD; we encourage the reader to run the program and compare results with this file.

For part (b) the input file is unchanged, except for the last two lines: a 2 will be given to indicate that H_2 is specified. Then the last two lines of the input file are

```
2
13 .5 70 1 80 2 90 3 100 4 100 5 100 7.5 90 10 80 15 70 20 60 25 60 30 60 35 60
```

The solution for part (b) can be found in file EPB12_1.OU2 on the CD.

* * *

12.5 AIR CHAMBERS, SURGE TANKS AND STANDPIPES

Air chambers, surge tanks and simple standpipes are all devices, basically tanks of various kinds, that are used in piping systems to protect the lines from extreme pressure surges which may be caused when a velocity quickly changes. When any appurtenances are a part of a network, the equations to describe them must be added to the set of continuity equations and unsteady equations of motion, and this enlarged equation system must then be solved. Two example problems which follow will illustrate the inclusion of these devices into the equation systems for the networks.

If the upper portion of the tank contains air under pressure, the tank is called an air chamber; if the top is open to the atmosphere, it is called a surge tank. As the water surface level rises in the closed air chamber, the air above the water is compressed. Consequently the head at node i that is connected to the air chamber has two components, the elevation of the water surface plus an additional water pressure head to represent the air pressure in excess of atmospheric pressure, or $H_i = \mathrm{WS}_i + \Delta p_{air}/\gamma_w$; here Δp_{air} is the air pressure above atmospheric pressure. (See also Section 13.2.6) In a tank of constant volume from which no air escapes, the air mass in the chamber is constant. This air undergoes a process that is likely to be somewhere between isothermal (constant temperature) and adiabatic (no energy transfer), and if a very small time interval is involved in the compression process, then the process will be essentially adiabatic so p/ρ^k = constant. Since the pressure in this equation is the absolute pressure, we compute Δp_{air} as $(\Delta p_{air} + p_{atm})/\rho^k = p_o/\rho_o{}^k$ = Constant (assuming the initial conditions p_o and ρ_o can be used to compute the constant for the adiabatic process), or

$$\Delta p_{air} = p_o\left(\frac{\rho}{\rho_o}\right)^k - p_{atm} \tag{12.17}$$

Since the density ρ is the ratio of mass M to volume $\forall$ we have $\rho = (M/\forall)_{air}$ with the air volume_

$$\forall_{air} = (\forall_{air})_o - \int Q\,dt \tag{12.18}$$

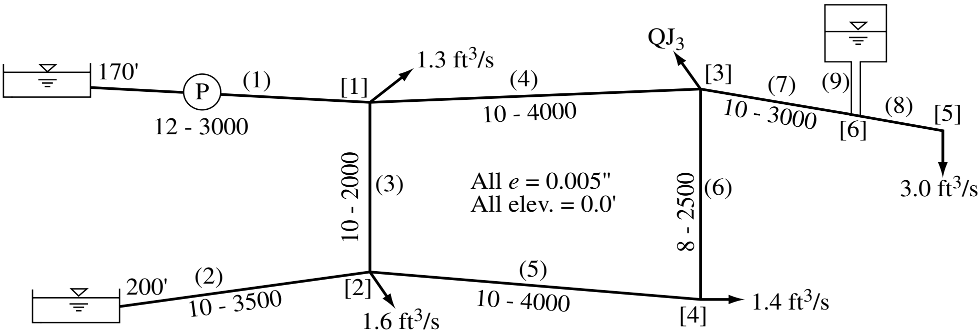
The air volume at any instant is the original air volume minus the increase in water volume $\int Qdt$ as the water flows into the air chamber. The magnitude of the initial air mass is found by first determining the pressure of the air; then its density is computed from the perfect gas law and other initial conditions as $\rho_o = p_o/(RT_o)$, followed by multiplying this density by the initial air volume in the tank, or

$$M = \rho_o(\forall_{air})_o = \rho_o(\forall_{total} - \forall_{water})_o \tag{12.19}$$

Many types of surge tanks exist, and they vary considerably in complexity. Some of the simplest tanks are basically a vertical standpipe, and these pipes may or may not contain an orifice constriction at the base where the device is connected to the network. We turn to two examples to see how air chambers and tanks are incorporated into a network model.

Example Problem 12.2

This network has an air chamber at the midpoint of pipe 7. To accommodate the air chamber, the original pipe 7 is divided into pipes 7 and 8, and the pipe from the connection point to the air chamber is called pipe 9. The tank volume is 100 ft^3 with a cross-sectional area of 10 ft^2, and initially half of the tank is filled with water so the initial water level x_o = 5 ft. The pipe connecting the surge tank to the network is 100 ft long and has a 6-in diameter. Data which describe the pump characteristic curve are presented in the table below the figure. At node 5 is a butterfly valve with discharge coefficient $c_D = c_o \exp(cD)$, in which D is the degree of opening (0^o is completely



Q, ft³/s	H, ft
5	50
6	45
7	39

closed and 90^o is fully open). Valve tests indicate that the ratio of c_D between $D = 80^o$ and $D = 10^o$ is 53.3. From an initial valve setting of 40^o at time $t = 0$, the valve is closed linearly at 7^o/sec for 5 sec and thereafter remains at the 5^o position. The demand (discharge) at node 3 is a linear function of the pressure head at this node; thus $QJ_3 = 0$ when $H_3 = 0$, and $QJ_3 = 1.5$ ft^3/s when H_3 is at its steady state value. We seek to simulate the unsteady flow in the network for 10 sec, according to rigid-column theory, to find the discharge in each pipe and the head at each node as a function of time.

The solution process begins with the determination of the initial flow state; NETWK can be used to generate these steady-flow data. There are 17 unknowns in this problem; they are the 9 discharges in the pipes, the nodal heads at the original 5 nodes, the head at node 6 where the air-chamber connector pipe is linked to pipe 7, the head H_7 in the air chamber itself, and finally the unknown demand QJ_3 at node 3. The program SURGNET on the CD will solve this type of problem. Examine its listing. In this program the array of unknowns X( ) is indexed in the following sequence: 1 = Q_1, 2 = Q_2 . . . 9 = Q_9, 10 = H_1, 11 = H_2, 12 = H_3, 13 = H_4, 14 = H_5, 15 = H_6, 16 = H_7, 17 = QJ_3. The set of 17 equations to be solved follows.

$$F_i = \sum Q_j - QJ_i = 0 \quad i = 1, 2, 3, 4, 6$$

The sums in these junction continuity equations range over the pipes that are connected to node i. The demand vs. head relation at node 3 must express the nodal demand there as being linearly proportional to the pressure head, as the problem specifies. Thus we write

$$F_5 = QJ_3 - c_3(H_3 - Elev_3) = X(17) - 0.0123 H_3 = 0$$

in which the constant $c_3 = 1.5/121.76 = 0.0123$; at steady state the demand is 1.5 ft^3/s, and the associated steady state head, from the input data list which follows, is 121.76 ft. In a similar way the continuity equation for node 5 with the butterfly valve is written

$$F_7 = Q_8 - c_o e^{cD}(H_5 - Elev_5)^{1/2} = Q_8 - 0.0324 e^{0.0568 D} H_5^{1/2}$$

In these two equations $Elev_3 = Elev_5 = 0.0$, as the diagram indicates. The next 9 equations are the ODEs that describe the rigid-column pipe transients, equation numbers $i = 8, 9, \ldots, 15, 16$; pipe numbers $k = 1, 2, \ldots, 9$:

$$F_i = Q_{ODE\,k} - Q_k = 0 \qquad \frac{dQ_k}{dt} = gA_k \frac{H_u - H_d}{L_k} - \frac{f_k Q_k |Q_k|}{2D_k A_k}$$

In these equations H_u is the upstream head, including any additional head from a pump, and H_d is the downstream head. The final equation models the air chamber:

$$F_{17} = H_7 - (x - x_o) - \left[p_o \left(\frac{M}{\rho_o \left\{ (\forall_{air})_o - \int Q_7 dt \right\}} \right)^k - p_{atm} \right] / \gamma_w = 0$$

The input data to SURGNET consists of

```
2 3 0.0000001 0.001 1 10 0.000417 32.2 1.217e-5 5 60 10 100 50
```

which is entered from the keyboard, and the input file SURGNET.DAT that can be found on the CD. The remaining data are

```
3000 1.0   5.9
3500 0.833 2.9
2000 0.833 1.3
4000 0.833 3.3
4000 0.833 2.6
2500 0.667 1.2
1500 0.833 3.0
1500 0.833 3.0
100  0.5   0.0
1.5 170.71
1.6 166.55
1.5 121.76
1.4 135.47
3.0  91.15
170 200 -0.5 0.5 60
```

The resulting solution can be found on the CD in file EPB12_2.OUT. That file contains a sequential listing of the 17 unknown variables at intervals of one second.

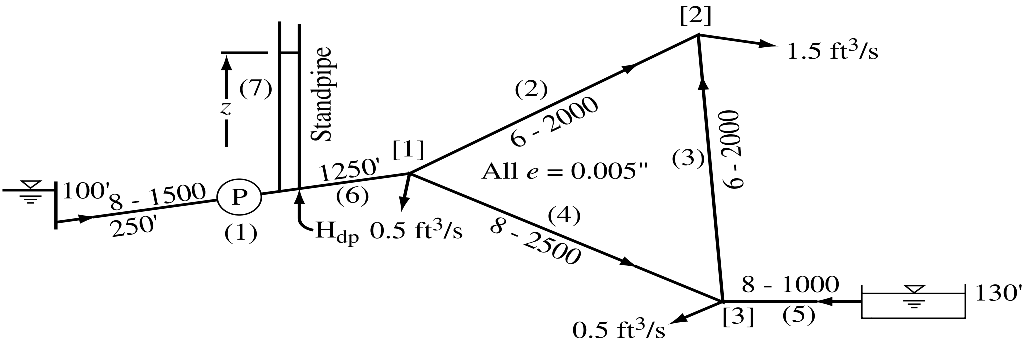
* * *

Example Problem 12.3

A five-pipe network, shown atop the next page, contains a standpipe downstream from the pump in pipe 1. The orifice diameter at the base of this standpipe is 2 in (assuming a contraction coefficient of unity), and the standpipe has a diameter of 1 ft. Initially the system is in steady-state operation, when the demand at node 2 is reduced to zero in 7 seconds according to the following schedule:

Time, sec	1.0	2.0	3.0	4.0	5.0	6.0	7.0
QJ_2, ft^3/s	1.25	1.0	0.8	0.6	0.4	0.2	0.0

Applying rigid-column theory, develop the equation system that describes this network, and then simulate the performance of the network over a 10-sec time period.



Pump data

Q, ft^3/s	1.5	2.0	2.5
h_p, ft	50	48	45

Without the standpipe there are three interior nodes in the network, a total of five pipe discharges, and a need for eight equations to describe the network operation. To model the addition of the standpipe we must divide pipe 1 at the standpipe location into pipes 1 and 6, as the figure shows. The standpipe itself becomes pipe 7. To describe this modified system we now need 4 junction continuity equations and 7 rigid-column unsteady flow ODEs for the 7 pipes. In addition, the water surface elevation z in the standpipe becomes another variable. If the nodal demands are all regarded as known, then the list of unknowns becomes Q_1, Q_2, Q_3, Q_4, Q_5, Q_6, Q_7, z, H_1, H_2, H_3, and H_{dp}. This last variable is the head just downstream from pump at the new standpipe node. The full equation set takes the following form:

$$F_1 = Q_6 - Q_2 - Q_4 - QJ_1 = 0$$

$$F_2 = Q_2 + Q_3 - QJ_2 = 0$$

$$F_3 = Q_5 + Q_4 - Q_3 - QJ_3 = 0$$

$$F_4 = Q_1 - Q_6 - Q_7 = 0$$

$$F_5 = Q_1 - Q_{ODE1} = 0, \qquad \frac{dQ_1}{dt} = gA_1 \frac{WS_1 + h_p - H_{dp}}{L_1} - \frac{f_1|Q_1|Q_1}{2D_1A_1}$$

$$F_6 = Q_2 - Q_{ODE2} = 0, \qquad \frac{dQ_2}{dt} = gA_2 \frac{H_1 - H_2}{L_2} - \frac{f_2|Q_2|Q_2}{2D_2A_2}$$

$$F_7 = Q_3 - Q_{ODE3} = 0, \qquad \frac{dQ_3}{dt} = gA_3 \frac{H_3 - H_2}{L_3} - \frac{f_3|Q_3|Q_3}{2D_3A_3}$$

$$F_8 = Q_4 - Q_{ODE4} = 0, \qquad \frac{dQ_4}{dt} = gA_4 \frac{H_1 - H_3}{L_4} - \frac{f_4 |Q_4| Q_4}{2D_4 A_4}$$

$$F_9 = Q_5 - Q_{ODE5} = 0, \qquad \frac{dQ_5}{dt} = gA_5 \frac{WS_2 - H_3}{L_5} - \frac{f_5 |Q_5| Q_5}{2D_5 A_5}$$

$$F_{10} = Q_6 - Q_{ODE6} = 0, \qquad \frac{dQ_6}{dt} = gA_6 \frac{H_{dp} - H_1}{L_6} - \frac{f_6 |Q_6| Q_6}{2D_6 A_6}$$

$$F_{11} = Q_7 - Q_{ODE7} = 0, \qquad \frac{dQ_7}{dt} = gA_7 \frac{H_{dp} - z}{z} - \left[\frac{f_7}{2D_7 A_7} + \left(\frac{A_7}{A_o} \right)^2 \right] |Q_7| Q_7$$

$$F_{12} = z - z_{ODE} = 0, \qquad \frac{dz}{dt} = \frac{Q_7}{A_7}$$

In equation F_{11} $A_7 = \pi D_7^2/4$ is the cross-sectional area of the standpipe, and A_o is the area of the orifice at the base of the standpipe. In the equation called F_5 the substitution for pump head $h_p = AQ_1^2 + BQ_1 + C$ must be made; otherwise the equation $F_{13} = h_p - AQ_1^2 - BQ_1 - C = 0$ must be added to the equation set, and h_p must be added to the list of unknowns.

The solution relies on the program PIPSTAND on the CD, which is written specifically to solve this problem. It calls on DVERK to solve eight ODEs that are part of the combined system of algebraic and differential equations that describe the behavior of the 12 unknowns in this system. The subroutine DEFFUN returns values for the 12 equations F_1, F_2, ... F_{12}, when it is called, and the main program then obtains the solution by applying the Newton method. The array X contains the unknowns listed in the sequence given in the earlier paragraph. The input to PIPSTAND to solve this problem consists of

```
2  3  0.000001  0.001  1  10  0.000417  32.2  1.217e-5  0.1667
```

from the keyboard, and the following data from an input file (PIPSTAND.IN on the CD):

```
250 0.667 1.81
2000 0.5 0.78
2000 0.5 0.72
2500 0.667 0.53
1000 0.667 0.69
1250 0.667 1.81
1.0 1.0 0.0
0.5 130.97
1.5 110.49
0.5 128.08
100.0 130.0 -2.0 3.0 50.0 145.88
```

The discharges in the pipes, given as the last item on the first seven lines of this file, were obtained with the nodal heads from the steady-state solution of the original network before pipes 6 and 7 were added. (The length of the standpipe, pipe 7, is given as 1.0 since in the program GA(7) = G*AA/L(7), and the varying length z is the length of the fluid in this pipe.) The nodal heads are the second values on the next three lines for the three original nodes. The last line contains the two water surface elevations, the three coefficients that describe the polynomial pump curve, and finally the head (145.88) at the standpipe location. This value is obtained from the steady-state solution by apportioning the head loss in pipe 1 along its length.

The solution, with the output re-organized into two tables for easier review (or as given by program PIPSTAND), follows.

Pipe Discharges

Time sec	QJ_2 ft^3/s	Q_1 ft^3/s	Q_2 ft^3/s	Q_3 ft^3/s	Q_4 ft^3/s	Q_5 ft^3/s	Q_6 ft^3/s	Q_7 ft^3/s
1.0	1.25	1.719	0.659	0.591	0.542	0.549	1.701	0.018
2.0	1.0	1.617	0.540	0.460	0.557	0.403	1.597	0.021
3.0	0.8	1.539	0.446	0.354	0.572	0.282	1.518	0.021
4.0	0.6	1.464	0.354	0.246	0.588	0.158	1.442	0.022
5.0	0.4	1.393	0.263	0.137	0.607	0.030	1.370	0.023
6.0	0.2	1.323	0.173	0.027	0.626	- 0.100	1.300	0.024
7.0	0.0	1.257	0.086	- 0.086	0.647	- 0.233	1.233	0.024
8.0	0.0	1.273	0.095	- 0.095	0.659	- 0.254	1.254	0.019
9.0	0.0	1.293	0.105	- 0.105	0.670	- 0.275	1.275	0.018
10.0	0.0	1.311	0.114	- 0.114	0.680	- 0.294	1.294	0.018

Heads

Time sec	QJ_2 ft^3/s	z ft	H_1 ft	H_2 ft	H_3 ft	H_{dp} ft
1.0	1.25	145.90	146.48	167.38	140.99	148.29
2.0	1.0	145.93	148.41	173.89	142.03	149.18
3.0	0.8	145.96	146.88	168.08	140.30	149.23
4.0	0.6	145.98	147.96	171.53	140.83	149.60
5.0	0.4	146.01	148.93	174.25	141.27	149.93
6.0	0.2	146.04	149.76	176.22	141.59	150.22
7.0	0.0	146.07	150.53	177.56	141.95	150.49
8.0	0.0	146.10	139.13	135.67	132.21	148.74
9.0	0.0	146.12	138.91	135.52	132.12	148.59
10.0	0.0	146.14	138.77	135.41	132.05	148.53

*   *   *

12.6 A FULLY TRANSIENT NETWORK ANALYSIS

12.6.1. THE INITIAL STEADY STATE SOLUTION

A steady-state solution for a network must be available before a transient (water hammer) analysis of the network can be conducted. Herein these steady-state solutions will be obtained from NETWK, since it can produce a file with the information that is needed by the transient network analysis program TRANSNET. The use of TRANSNET will be described and illustrated in subsequent pages. By setting the option NETPLT=4 (as is done when NETWK is asked to write an input file for UNSTPIP), a file will be written that will enable TRANSNET to know the physical configuration of the network and a suitable set of initial conditions. The next subsection describes the use of program TRANSNET. In this section the use of NETWK to obtain the steady-state solution is described.

To ensure that the file written by NETWK for TRANSNET will contain the correct information, it is necessary (in addition to setting the option NETPLT=4) to set two additional options, NODESP=1 and PCHAR3=0; the first of these assigns node numbers to reservoirs and source pumps, and the second option allows more than three pairs of (Q, h_p) data to be used to define pump characteristics. Program TRANSNET defines pump curves by piecewise linear segments, as illustrated in Section 11.1, and requires six pairs of points. Therefore each pump curve must be defined by supplying six pairs of (Q, h_p) values; the first pair should contain $Q = 0$, and the last pair should contain $h_p = 0$ so the entire pump curve is defined. Booster pump stations, pressure reduction valves, back pressure valves, check valves and some similar appurtenances are not currently accommodated by program TRANSNET; when used to write a file for TRANSNET, the commands to NETWK should be limited to PIPES, NODES, PIPE-, RESER, PUMPS, RUN and END. Furthermore, with current dimensions a maximum of four pipes may join at a node. Under the NODES command the listing of nodes for source pumps should follow real nodes, and nodes assigned to reservoirs should follow source pump nodes.

Since the file written for TRANSNET is different from a file written for UNSTPIP, NETWK will ask which unsteady program will use the file when the option NETPLT=4 is set. Upon selecting 2 for TRANSNET and providing a file name, the user will be asked for the following additional information that is needed for a transient network analysis:

(a) the wave speed in the pipes of the network, and

(b) the following two lines of information for each pump station:

On line one, (1) the number of stages (pumps in series), (2) the number of pumps in parallel, (3) the rotational speed in rev/min for which the pump curve is defined, and (4) the rotational moment of inertia in units of force times length squared (lb-ft^2 for ES units, kN-m^2 for SI units) for each pump stage.

On line two, a list of six values for power (horsepower for ES units, kW for SI units), corresponding to the six (Q, h_p) data pairs that define that pump's characteristic curve.

This additional information that is needed by TRANSNET can be provided in any of three ways: (1) after the RUN (or END) command in the same file provided to NETWK to obtain the steady-state solution, (2) in a separate file, or (3) from the keyboard during execution of NETWK. NETWK will prompt the user to learn which option is to be used to enter the additional information. For example, if 3 is specified, meaning that the keyboard will be used, then a prompt will tell the user what is expected next.

If this information is to be in a file, then the data must be sequenced:

First: One line that contains the wave speeds, formatted as pipe numbers and their wave speeds in pairs, or a range of pipe numbers and the wave speed for this range. For example,

1 3000 2-8 2800 9 2500

assigns a wave speed of 3000 to pipe 1, a wave speed of 2800 to pipes 2 through 8, and a wave speed of 2500 to pipe 9. The pipes with numbers that are not included in the list will be given the wave speed that was assigned to the previous pipe in the list; if there are 12 pipes in the example, then pipes 10, 11 and 12 will be assigned a wave speed of 2500. NETWK contains the default wave speed of 3000 ft/s; if pipes 1 through 5, for example, are not assigned a wave speed, then their wave speed will default to 3000. There is considerable flexibility in providing this wave speed data, since any pipes without assigned wave speeds are given the wave speed of the previous pipe. A comma can be used in place of a blank in any of the lists, and blanks may follow commas. However, both end values in a range of pipes must be specified.

Second: Two lines for each pump, in the same order as the pumps are listed in the input data file, following the instructions for item (b) above.

The program NETWK allows composite pump curves for pumps in series or pumps in parallel to be given with the data pairs rather than using the SERIES and/or PARALLEL commands, and the number of pumps in series or parallel need not be integer values, thus allowing non-identical pumps to be lumped at a pumping station. The user can apply this same treatment of pumps when preparing a file for TRANSNET. However, if data are presented for a composite curve in place of a single pump stage, then one must place a minus sign before the number of stages and/or the number of pumps in parallel; then the discharges and heads will be adjusted before the file for TRANSNET is written. In other words, the final input file for TRANSNET must contain the discharge, head and rotational moment of inertia for one stage of a single pump.

This input is illustrated in the next section as three example network problems are described, analyzed by using NETWK to obtain steady-state solutions, and then transient analyses are performed by using the file written by NETWK to drive TRANSNET.

12.6.2. TRANSNET

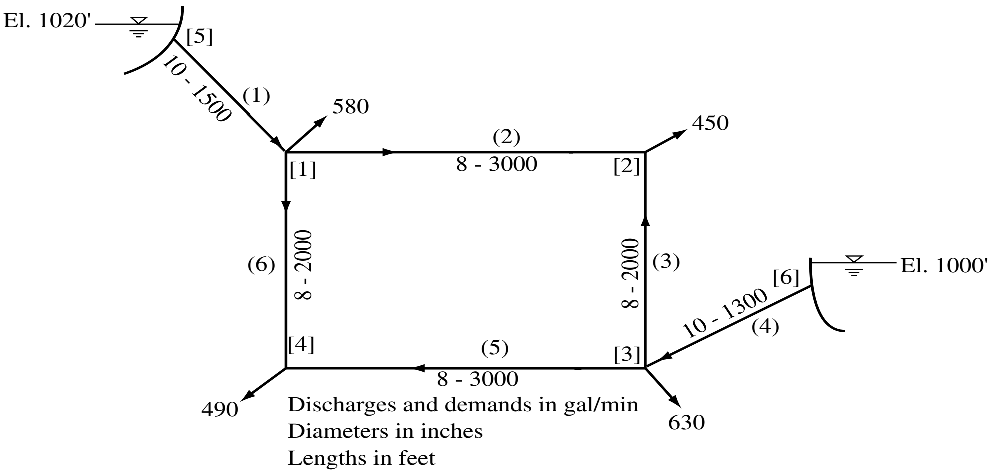
The transient program TRANSNET permits the user to investigate the effects of several events that can cause rapid, possibly severe, transients. The equations that are solved for each pipe of the network are the equations that have been developed in Chapters 7-9 and applied to individual pipes and smaller systems in Chapters 8-11. Power failure at any number of pump stations can be simulated. The effects of sudden valve closure at either end of any number of pipes can be investigated. Staged valve closure at the downstream end of any pipe can be specified. The consequences of sudden demand changes at any number of junctions can also be studied.

Most of the data that are required to initiate the study of a transient is stored in the data file which is created during the execution of NETWK with the option NETPLT=4, as described in the previous section. The only additional data is the specification of the transient-causing event. If the assumed flow direction in any pipe is found to be incorrect, NETWK corrects this direction before writing the file for TRANSNET; the investigator should also change these flow directions on a diagram of the network. The example problems will illustrate these procedures.

A description of the input data parameters which determine the transient behavior is included at the beginning of the source listing of TRANSNET; before reading further, a listing of TRANSNET should be obtained from the CD so it can be studied.

Example Problem 12.4

This network is supplied by gravity flow from two elevated reservoirs. Pipe and node numbers are shown in the diagram with the diameters and lengths of the pipes. Nodes 1 through 4 have ground elevations of 860 ft, while nodes 5 and 6 have ground elevations of 980 ft and 960 ft, respectively. The equivalent sand roughness of all pipes is $e = 0.002$ in. The demand is increased from 450 gal/min to 900 gal/min at node 2 to meet a sudden need for more water for fire suppression. Determine the effect of this increase in demand on the heads at nodes 1, 2, and 4.



In this analysis the first step is to determine the steady-state solution which will define the initial conditions for the ensuing transient. Two alternative input data files have been prepared for NETWK to use in obtaining this steady-state solution. The option OUTPU1=4 tells NETWK to provide the values of friction factors, rather than *e*'s, in the PIPE DATA table. One input file is listed below, and both input files are available on the CD under the names EPB12_4.IN and EPB12_4.IN1.

```
Example Problem 12.4
/*
$SPECIF NETPLT=4,NFLOW=1,OUTPU1=4
 COEFRO=0.002,NODESP=1 $END
PIPE-
1 10.0 1500. 5 980. 1 580. 860.
2  8.0 3000. 1 2 450. 860.
3  8.0 2000. 3 2
4 10.0 1300. 6 960. 3 630. 860.
5  8.0 3000. 3 4 490. 860.
6  8.0 2000. 1 4
RESER
5 1020
6 1000
RUN
1 3000/  Assigns a wave speed of 3000 to all pipes.
```

Since additional data are provided after the RUN command, NETWK will use a prompt to learn where the wave speed information is to be found; the user should select 1, meaning "in the same file after the other data." The steady-state solution found by NETWK is listed in the following two tables:

PIPE DATA

PIPE NO.	NODES FROM	TO	L ft.	DIA. in	*f*	Q gal/min	VEL. ft/s	HEAD LOSS ft.	HLOSS/ 1000
1	5	1	1500	10.0	0.01598	1447	5.91	15.61	10.41
2	1	2	3000	8.0	0.01877	389	2.48	8.08	2.69
3	3	2	2000	8.0	0.02693	61.2	0.39	0.19	0.10
4	6	3	1300	10.0	0.01750	703	2.87	3.49	2.69
5	3	4	3000	8.0	0.04171	11.7	0.07	0.02	0.01
6	1	4	2000	8.0	0.01820	478	3.05	7.90	3.95

NODE DATA

NODE	DEMAND ft^3/s	DEMAND gal/min	ELEV. ft.	HEAD ft.	PRESSURE lb/in^2	HGL ELEV. ft.
1	1.29	580	860.	144.39	62.57	1004.4
2	1.00	450	860.	136.31	59.07	996.3
3	1.40	630	860.	136.51	59.15	996.5
4	1.09	490	860.	136.49	59.15	996.5
5	- 3.22	- 1447	980.	40.00	17.33	1020.0
6	- 1.57	- 703	960.	40.00	17.33	1000.0

Now the execution of the program TRANSNET can proceed. Two files are needed as input to TRANSNET, the file written by NETWK and a file that describes the transient analysis to be done. The file written by NETWK is currently unformatted, so it can not easily be examined. The reader should have the experience of having NETWK produce this file, but the CD also contains this unformatted information in file EPB12_4.OU1. The following file contains the transient analysis data (on the CD under the name EPT12_4.DAT):

```
DEMONSTRATION OF PROGRAM TRANSNET - INPUT DATA FILE "EPB12_4.DAT"
DEMAND AT JUNCTION 2 IS INCREASED FROM 450 TO 900 GAL/MIN
&SPECS NPARTS=4,IOUT=1000,NQNEW=1,HATM=28.,TMAX=60.,GRAPH=T,
          NODEQ(1)=2,QNEW(1)=900./
&GRAF NSAVE=3,IOUTSA=2,PIPE=1,2,6,0,NODE=999,999,999,0/
```

The output file written by TRANSNET (also on the CD in file EPB12_4.OUT with the plot file EPB12_4.PLT) follows:

```
                    *****************************
                    * NETWORK TRANSIENT ANALYSIS *
                    *****************************

   DEMONSTRATION OF PROGRAM TRANSNET -  INPUT DATA FILE "EPB12_4.DAT"
   DEMAND AT JUNCTION 2 IS INCREASED FROM 450 TO 900 GAL/MIN

          IOUT   = 1000
        NPARTS =       4
         NPIPES =       6

         HATM  = 28.0 FT

         TMAX  =  60.0 SEC
         DELT   =  0.10 SEC
```

TRANSIENT CONDITIONS IMPOSED

DEMAND DISCHARGES SUDDENLY CHANGED AT NODE 2 TO 900.0 GAL/MIN

PIPE INPUT DATA

PIPE	DIAMETER in	LENGTH ft	WAVE SPEED ft/s	PIPEZ ft	f	VELOCITY ft/s
1	10.00	1500.	3000.	980.	0.0160	5.89
2	8.00	3000.	3000.	860.	0.0188	2.47
3	8.00	2000.	3000.	860.	0.0267	0.41
4	10.00	1200.	3000.	980.	0.0175	2.90
5	8.00	3000.	3000.	860.	0.0389	0.10
6	8.00	2000.	3000.	860.	0.0182	3.03

PIPE	DELTA sec	PARTS	SINE	L/A sec	INTERPOLATION
1	0.124	4	- 0.0800	0.50	0.202
2	0.249	10	0.0000	1.00	0.003
3	0.167	6	0.0000	0.67	0.103
4	0.100	4	- 0.1000	0.40	0.003
5	0.250	10	0.0000	1.00	0.003
6	0.166	6	0.0000	0.67	0.103

NODE INPUT DATA

NODE	HGL ft	GRND EL ft	PIPES 1	2	3	4	DEMAND ft³/s
1	1004.51	860.0	- 1	2	6	0	1.29
2	996.52	860.0	- 2	- 3	0	0	2.00
3	996.73	860.0	3	- 4	5	0	1.40
4	996.70	860.0	- 5	- 6	0	0	1.09
5	1020.00	980.0	1	0	0	0	- 3.21
6	1000.00	980.0	4	0	0	0	- 1.58

****

* TABLE OF EXTREME VALUES *

****

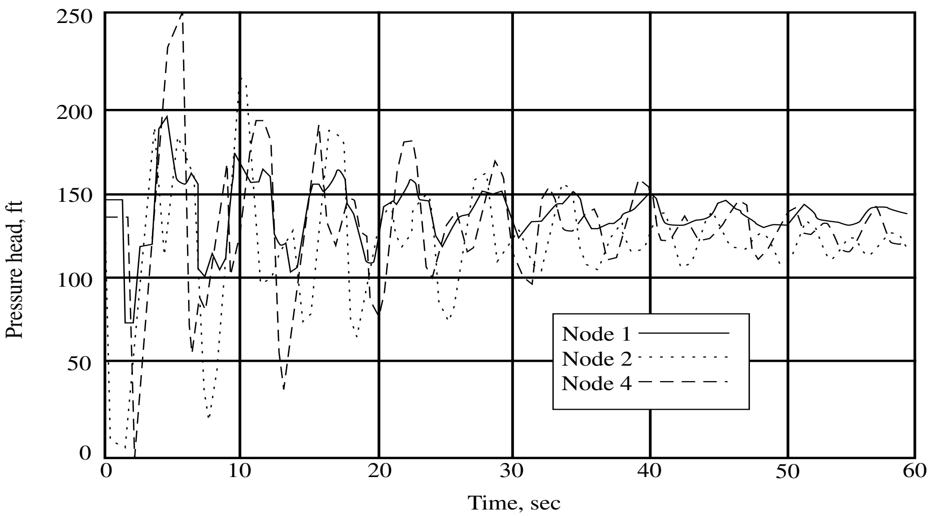
	X	MAX HEAD ft	TIME sec	MIN HEAD ft	TIME sec	MAX H ft	MIN H ft
PIPE 1							
	0.000	40.0	59.9	40.0	59.9	1020.	1020.
	1.000	200.0	3.9	71.0	1.8	1060.	931.
PIPE 2							
	0.000	200.0	3.9	71.0	1.8	1060.	931.
	1.000	233.9	9.7	- 1.8	1.2	1094.	858.
PIPE 3							
	0.000	190.8	4.2	61.5	1.4	1051.	921.
	1.000	233.9	9.7	- 1.8	1.2	1094.	858.
PIPE 4							
	0.000	20.0	59.9	20.0	59.9	1000.	1000.
	1.000	190.8	4.2	61.5	1.4	1051.	921.
PIPE 5							
	0.000	190.8	4.2	61.5	1.4	1051.	921.
	1.000	259.8	4.6	- 9.5	2.4	1120.	850.
PIPE 6							
	0.000	200.0	3.9	71.0	1.8	1060.	931.
	1.000	259.8	4.6	- 9.5	2.4	1120.	850.

MAXIMUM HEAD = 268.1 FT IN PIPE 5 AT X = 0.800 AT TIME = 4.59 SEC

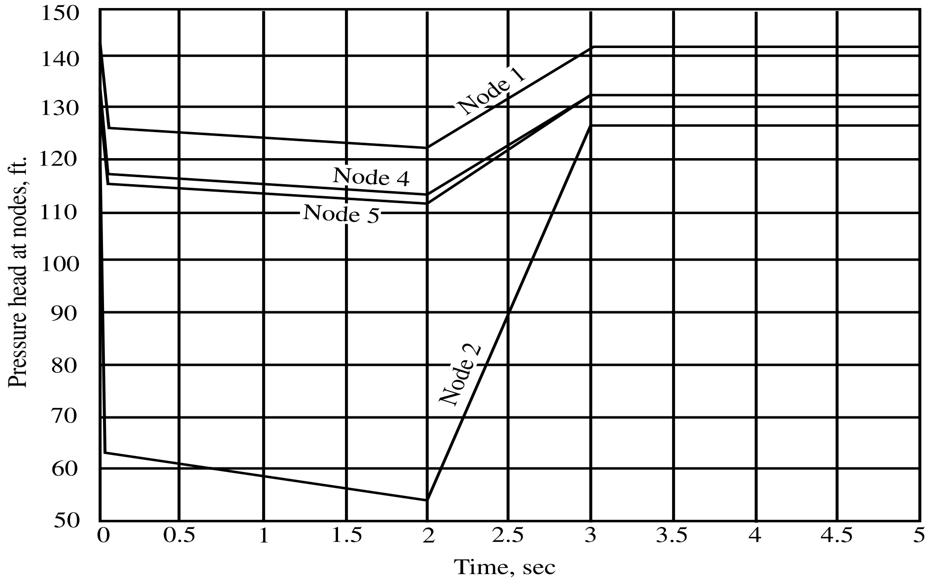
MINIMUM HEAD = - 12.5 FT IN PIPE 4 AT X = 0.250 AT TIME = 1.10 SEC

Since plot information was requested by setting GRAPH=T (for true), another output file is written by TRANSNET that contains data for the transient pressure heads at nodes 1, 2, and 4. The plot of these data, shown below, indicates how the pressure waves decay with time. From the extreme value table we note that neither the highest nor lowest pressures occur at the node where the demand changes; instead they occur near node 4. Can the reader explain why this occurs?

An unsteady solution that ignores elastic effects will now also be obtained. The input file is again provided to NETWK, but now select 1 when asked whether a file for 1. UNSTPIP or 2. TRANSNET should be written. Even when elastic effects are ignored, the demand clearly can not be increased instantly from 450 to 900 gal/min, so let us assume the increase occurs linearly over 2 sec. We encourage the reader to prepare the input for UNSTPIP and obtain this solution. The following graph presents some of the solution. We see that the pressure head at node 2 becomes smallest, at 53.7 ft, after 2 sec when the demand at this node has just become 900 gal/min and the increase in demand ceases. Note also from this solution, after the demand becomes constant, how rapidly the pressure heads approach the new steady-state conditions with a pressure head at node 2 of 127.8 ft, which is 8.5 ft less than the head for QJ_2 = 450 gal/min. One could solve again this problem (we encourage the reader to do so) with QJ_2 = 900 gal/min



and find after only 5 sec that the nodal pressure heads are essentially identical to these new values. If the demand were to increase more rapidly, then the pressure at node 2 would decrease still further. If one tries to double the demand in 1.5 or 1.0 sec, negative pressures will occur at node 2, which is obviously not physically possible. A more

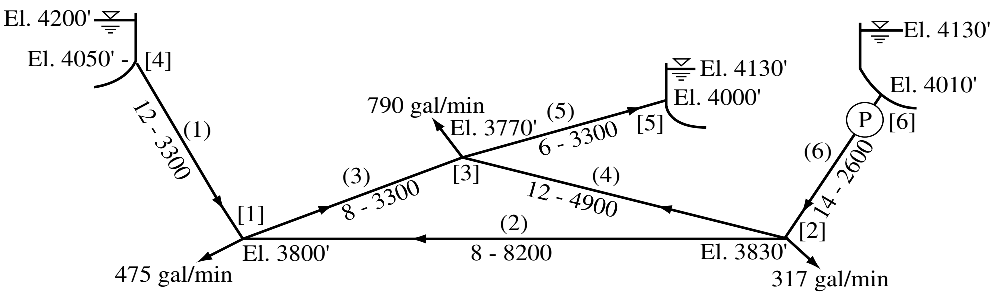


realistic problem description would very rapidly reduce the pressure head at node 2 to zero; then one could determine from the solution the time that is required to increase the demand at this node to 900 gal/min. Therefore rigid-column unsteady solutions should not be used to study rapid changes; that approach is more appropriate to the study of gradual changes that could occur continually throughout a network.

* * *

Example Problem 12.5

Here we see how a pump is incorporated into a network. A source pump (the operating characteristic data are listed in the input file for NETWK) is located at one of the three reservoirs. The Hazen-Williams roughness coefficient is $C_{HW} = 120$ for all pipes. This network experiences a transient that is caused by the sudden closure of a valve at the downstream end of pipe 5. Obtain a transient analysis of this network if the wave speed is 2850 ft/s for all pipes.



To begin the solution process, we have chosen to supply the following input data file (EPB12_5.IN or EPB12_5.IN1 on the CD) to NETWK to obtain the initial condition for the ensuing transient analysis:

```
Example Problem 12.5
/*
$SPECIF NETPLT=4, NFLOW=1, NPGPM=1, OUTPU1=4, NODESP=1, COEFRO=120
 PCHAR3=0 $END
PIPE-
1 12. 3300. 4 4050. 1 475. 3800.
2 8. 8200. 2 317. 3830. 1
3 8. 3300. 1 3 790. 3770.
4 12. 4900. 2 3
5 6. 3300. 3 5 4000.
6 14. 2600. 6 4010. 2
RESER
4 4200
5 4130
PUMPS
6 0 118 2000 92 3000 82 4000 67 4500 52 5300 0 4130/
RUN
1 2850
1 1 1180 50
57 68 77 80 76 60
```

In this input file the option PCHAR3=0 is set so that six (Q, h_p) pairs can be entered to define the pump curve. Additional data needed by TRANSNET are provided after the RUN command. The first line following RUN indicates that all pipes have a wave speed of 2850 ft/s; the second line indicates one pump stage at this station, one pump in parallel with a rotational speed of 1180 rev/min and a rotational moment of inertia of 50 lb-ft^2; the third line lists the six horsepower values corresponding to the six discharge-head pairs provided under the PUMPS command.

The steady-state solution from NETWK is described in the following two tables (and listed in file EPB12_5.OUT on the CD):

PIPE DATA

PIPE NO.	NODES FROM	TO	L ft.	DIA. in	C_{HW}	Q gal/min	VEL. ft/s	HEAD LOSS ft.	HLOSS /1000
1	4	1	3300	12.0	120	340.1	0.96	1.32	0.40
2	2	1	8200	8.0	120	273.0	1.74	15.70	1.91
3	1	3	3300	8.0	120	138.1	0.88	1.79	0.54
4	2	3	4900	12.0	120	1110.0	3.15	17.49	3.57
*5	3	5	3300	6.0	120	458.1	5.20	66.89	20.27
6	6	2	2600	14.0	120	1700.0	3.54	9.64	3.71

NODE DATA

NODE	DEMAND ft^3/s	DEMAND gal/min	ELEV. ft.	HEAD ft.	PRESSURE lb/in^2	HGL ELEV. ft.
1	1.06	475	3800.	398.68	172.76	4198.68
2	0.71	317	3830.	384.38	166.57	4214.38
3	1.76	790	3770.	426.89	184.99	4196.89
6	- 3.79	- 1700	4010.	214.03	92.74	4224.03
4	- 0.76	- 340	4050.	150.00	65.00	4200.00
5	1.02	458	4000.	130.00	56.33	4130.00

The supplemental input file (EPB12_5.DAT on the CD) for TRANSNET can take the following form:

```
DEMONSTRATION OF PROGRAM TRANSNET - INPUT DATA FILE "EPB12_5.DAT"
NETWORK EXAMPLE 12.5 -  SUDDENLY-CLOSED VALVE AT THE DS END OF PIPE  5
&SPECS NPARTS=4, IOUT=100, NSHUT=1, HATM=30., TMAX=20.0, ALLOUT=T,
          HVPRNT=T, ISHUT(1)=-5/
&GRAF  NSAVE=4,  IOUTSA=1,  PIPE=5,5,1,6,NODE=999,1,999,999/
```

Together with the file written by NETWK, this file can be executed by TRANSNET to produce the transient solution. Only part of the output file (see EPB12_5.OU1) is presented here:

```
                    ******************************
                    * NETWORK TRANSIENT ANALYSIS *
                    ******************************

DEMONSTRATION OF PROGRAM NO. 6 -INPUT DATA FILE "EPB12_5.DAT"
NETWORK EXAMPLE 12.5 - SUDDENLY-CLOSED VALVE AT THE DS END OF PIPE 5

      IOUT  =  100
    NPARTS =     4
     NPIPES =     6

     HATM  =  30.0  FT

     TMAX  =  20.00  SEC
      DELT  =   0.227 SEC

                    TRANSIENT CONDITIONS IMPOSED

SUDDENLY CLOSED VALVE AT DOWNSTREAM END OF PIPE    5
```

PIPE INPUT DATA

PIPE	DIAMETER in	LENGTH ft	WAVE SPEED ft/s	PIPEZ ft	C_{HW}	VELOCITY ft/s
1	12.00	3300.	2850.	4050.	120.	0.97
2	8.00	8200.	2850.	3830.	120.	1.74
3	8.00	3300.	2850.	3800.	120.	0.88
4	12.00	4900.	2850.	3830.	120.	3.15
5	6.00	3300.	2850.	3770.	120.	5.20
6	14.00	2600.	2850.	4010.	120.	3.54

PIPE	DELTA sec	PARTS	SINE	L/A sec	INTERPOLATION
1	0.289	5	- 0.07576	1.16	0.019
2	0.718	12	- 0.00366	2.88	0.052
3	0.289	5	- 0.00909	1.16	0.019
4	0.428	7	- 0.01224	1.72	0.075
5	0.288	5	0.06970	1.16	0.019
6	0.227	4	- 0.06923	0.91	0.004

NODE INPUT DATA

NODE	HGL ft	GRND EL ft	PIPES 1	PIPES 2	PIPES 3	PIPES 4	DEMAND ft^3/s
1	4198.68	3800.0	- 1	- 2	3	0	1.06
2	4214.38	3830.0	2	4	- 6	0	0.71
3	4196.89	3770.0	- 3	- 4	5	0	1.76
6	4224.03	4010.0	6	0	0	0	- 3.79
4	4200.00	4050.0	1	0	0	0	- 0.76
5	4130.00	4000.0	- 5	0	0	0	1.02

PUMP INFORMATION

			Q gal/min	HEAD/STAGE ft	HP/STAGE HP
LINE	=	6	0.0	118.0	57.0
PUMPS	=	1	2000.8	92.0	68.0
STAGES	=	1	3001.1	82.0	77.0
RPM	=	1180. RPM	4001.5	67.0	80.0
SUMP ELEV	=	4130. FT	4501.7	52.0	76.0
WRSQ	=	50. LB-FTSQ	5302.0	0.0	60.0

PRESSURE HEADS, HGL'S AND VELOCITIES AS FUNCTIONS OF TIME

TIME = 0.000 SEC

	X/L	HEAD ft	HGL ft	VEL ft/s	X/L	HEAD ft	HGL ft	VEL ft/s
PIPE 1	0.00	150.	4200.	0.97	0.20	200.	4200.	0.97
	0.40	249.	4199.	0.97	0.60	299.	4199.	0.97
	0.80	349.	4199.	0.97	1.00	399.	4199.	0.97
PIPE 2	0.00	384.	4214.	1.74	0.08	386.	4213.	1.74
	0.17	387.	4212.	1.74	0.25	388.	4210.	1.74
	0.33	389.	4209.	1.74	0.42	390.	4208.	1.74
	0.50	391.	4206.	1.74	0.58	393.	4205.	1.74
	0.67	394.	4204.	1.74	0.75	395.	4202.	1.74
	0.83	396.	4201.	1.74	0.92	397.	4200.	1.74
	1.00	398.	4198.	1.74				
PIPE 3	0.00	399.	4199.	0.88	0.20	404.	4198.	0.88
	0.40	410.	4198.	0.88	0.60	416.	4198.	0.88
	0.80	421.	4197.	0.88	1.00	427.	4197.	0.88
PIPE 4	0.00	384.	4214.	3.15	0.14	390.	4212.	3.15
	0.29	396.	4209.	3.15	0.43	403.	4207.	3.15
	0.57	409.	4204.	3.15	0.71	415.	4202.	3.15
	0.86	421.	4199.	3.15	1.00	427.	4197.	3.15
PIPE 5	0.00	427.	4197.	5.20	0.20	367.	4183.	5.20
	0.40	308.	4170.	5.20	0.60	248.	4156.	5.20
	0.80	189.	4143.	5.20	1.00	129.	4129.	5.20
PIPE 6	0.00	214.	4224.	3.54	0.25	257.	4222.	3.54
	0.50	299.	4219.	3.54	0.75	342.	4217.	3.54
	1.00	384.	4214.	3.54				

PIPE 6 PUMP SPEED = 1180.0 RPM PUMP DISCHARGE = 1700.6 GAL/MIN EACH
PUMP HEAD = 94.0 FT

COLUMN SEPARATION HAS OCCURRED AT 7.73 SEC IN PIPE 5 AT LOCATION 1.00

	X/L	HEAD ft	HGL ft	VEL ft/s	X/L	HEAD ft	HGL ft	VEL ft/s
TIME = 7.726 SEC								
PIPE 1	0.00	150.	4200.	- 0.25	0.20	201.	4201.	- 0.25
	0.40	268.	4218.	- 0.40	0.60	332.	4232.	- 0.47
	0.80	375.	4225.	- 0.25	1.00	407.	4207.	0.04
PIPE 2	0.00	379.	4209.	1.63	0.08	335.	4163.	1.48
	0.17	299.	4124.	1.17	0.25	272.	4094.	1.10
	0.33	240.	4060.	1.12	0.42	228.	4046.	1.34
	0.50	223.	4038.	1.49	0.58	223.	4035.	1.59
	0.67	228.	4038.	1.69	0.75	267.	4074.	1.58
	0.83	316.	4121.	1.71	0.92	367.	4169.	2.15
	1.00	407.	4207.	2.61				
PIPE 3	0.00	407.	4207.	- 0.33	0.20	396.	4190.	- 0.22
	0.40	396.	4184.	- 0.23	0.60	406.	4188.	- 0.23
	0.80	415.	4191.	- 0.22	1.00	422.	4192.	- 0.24
PIPE 4	0.00	379.	4209.	2.60	0.14	397.	4218.	2.66
	0.29	402.	4215.	2.69	0.43	392.	4197.	2.46
	0.57	390.	4186.	2.19	0.71	404.	4191.	2.13
	0.86	414.	4193.	2.10	1.00	422.	4192.	2.07
PIPE 5	0.00	422.	4192.	- 1.11	0.20	373.	4189.	- 1.14
	0.40	254.	4116.	- 0.56	0.60	146.	4054.	- 0.35
	0.80	41.	3995.	- 0.28	1.00	- 39.	3961.	0.00
PIPE 6	0.00	219.	4229.	3.07	0.25	260.	4225.	3.07
	0.50	301.	4221.	3.07	0.75	340.	4215.	3.09
	1.00	379.	4209.	3.10				

PIPE 6 PUMP SPEED = 1180.0 RPM PUMP DISCHARGE = 1470.6 GAL/MIN EACH
PUMP HEAD = 98.9 FT

****
* TABLE OF EXTREME VALUES *
****

	X/L	MAX HEAD ft	TIME sec	MIN HEAD ft	TIME sec	MAX HGL ft	MIN HGL ft
PIPE 1							
	0.00	150.0	7.7	150.0	7.7	4200.	4200.
	0.20	256.1	3.6	176.8	5.9	4256.	4177.
	0.40	308.9	3.9	224.3	6.1	4259.	4174.
	0.60	359.7	4.1	275.5	5.9	4260.	4175.
	0.80	412.0	4.3	327.6	5.7	4262.	4178.
	1.00	463.1	4.5	378.6	5.5	4263.	4179.
PIPE 2							
	0.00	476.5	4.5	337.4	6.8	4307.	4167.
	0.08	476.8	4.8	317.7	7.5	4304.	4145.
	0.17	522.0	5.0	283.6	7.3	4347.	4109.
	0.25	532.8	5.2	267.0	7.5	4355.	4090.
	0.33	535.1	5.5	240.2	7.7	4355.	4060.
	0.42	537.1	5.7	228.2	7.7	4355.	4046.
	0.50	539.0	5.9	223.3	7.7	4354.	4038.
	0.58	538.2	5.7	222.7	7.7	4351.	4035.
	0.67	535.3	5.5	227.6	7.7	4345.	4038.
	0.75	512.4	5.2	266.5	7.7	4320.	4074.
	0.83	460.0	5.0	316.2	7.7	4265.	4121.
	0.92	461.6	4.8	367.0	7.7	4264.	4169.
	1.00	463.1	4.5	378.6	5.5	4263.	4179.
PIPE 3							
	0.00	463.1	4.5	378.6	5.5	4263.	4179.
	0.20	527.4	2.5	360.4	5.7	4321.	4154.
	0.40	537.3	2.7	353.8	5.9	4325.	4142.
	0.60	545.4	3.0	357.6	6.1	4327.	4140.
	0.80	555.1	3.2	367.4	5.9	4331.	4143.
	1.00	563.4	3.4	380.8	6.6	4333.	4151.
PIPE 4							
	0.00	476.5	4.5	337.4	6.8	4307.	4167.
	0.14	498.4	3.0	334.9	5.5	4320.	4156.
	0.29	521.5	3.2	329.6	5.5	4334.	4142.
	0.43	531.0	3.4	333.1	5.7	4335.	4137.
	0.57	540.3	3.6	338.6	5.9	4336.	4134.
	0.71	549.4	3.9	345.7	6.1	4337.	4133.
	0.86	556.4	3.6	374.3	6.6	4335.	4153.
	1.00	563.4	3.4	380.8	6.6	4333.	4151.
PIPE 5							
	0.00	563.4	3.4	380.8	6.6	4333.	4151.
	0.20	799.4	1.4	258.8	3.6	4615.	4075.
	0.40	761.6	1.6	200.1	3.9	4624.	4062.
	0.60	722.2	1.8	146.3	7.7	4630.	4054.
	0.80	682.8	2.0	40.5	7.7	4637.	3995.
	1.00	643.4	2.3	- 39.1	7.7	4643.	3961.
PIPE 6							
	0.00	227.4	5.9	214.0	0.0	4237.	4224.
	0.25	329.6	3.9	225.0	6.1	4295.	4190.
	0.50	386.3	4.1	256.1	6.4	4306.	4176.
	0.75	431.6	4.3	296.1	6.6	4307.	4171.
	1.00	476.5	4.5	337.4	6.8	4307.	4167.

MAXIMUM HEAD = 799.4 FT IN PIPE 5 AT X = 0.20 AT TIME = 1.36 SEC
MINIMUM HEAD = - 39.1 FT IN PIPE 5 AT X = 1.00 AT TIME = 7.73 SEC

The network experiences column separation in pipe 5 after 7.7 sec. The pump is still producing flow, although at a reduced discharge.

* * *

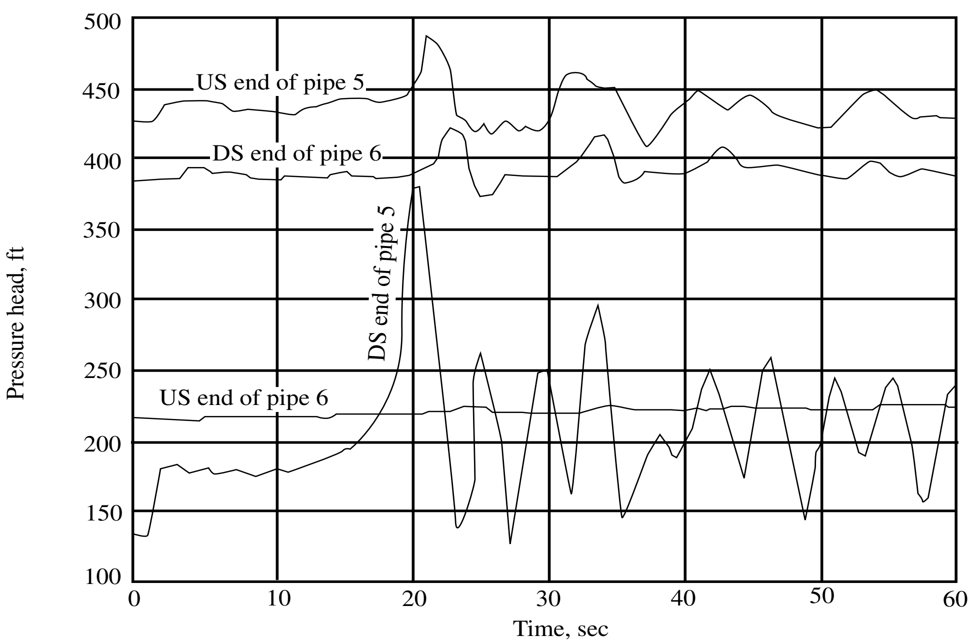
Example Problem 12.6

Reconsider Example Problem 12.5, replacing the sudden valve closure with a gate valve that closes in 20 sec at two different rates. The first stage is to 95 % closure in 1 sec, with the remainder of the closure in 19 sec. Use the gate valve loss coefficient data listed in Table 10.2 in Section 10.4.4.

In this problem the same initial condition applies as in the previous problem; however, a different file is needed to tell TRANSNET what to do. The revised file consists of the following instructions:

```
DEMONSTRATION OF PROGRAM TRANSNET -INPUT DATA FILE "EPB12_6.DAT"
NETWORK OF EXAMPLE 12.6 - GRADUALLY-CLOSED VALVE AT THE DS END OF PIPE 5
&SPECS NPARTS=4, IOUT=1000, IVALVE=5, HATM=30., TMAX=60., GRAPH=T,
            PC1=5., TC1=1., TC2=20., KLI=0.,0.0167,0.0313,0.0556,0.100,0.179,
           0.333,0.625,1.25,2.50,5.27/
&GRAF NSAVE=4, IOUTSA=2, PIPE=5,5,6,6, NODE=999,1,999,1/
```

The output from TRANSNET will be found in file EPB12_6.PLT on the CD. The output plot, followed by the information for the plot file, is presented next:



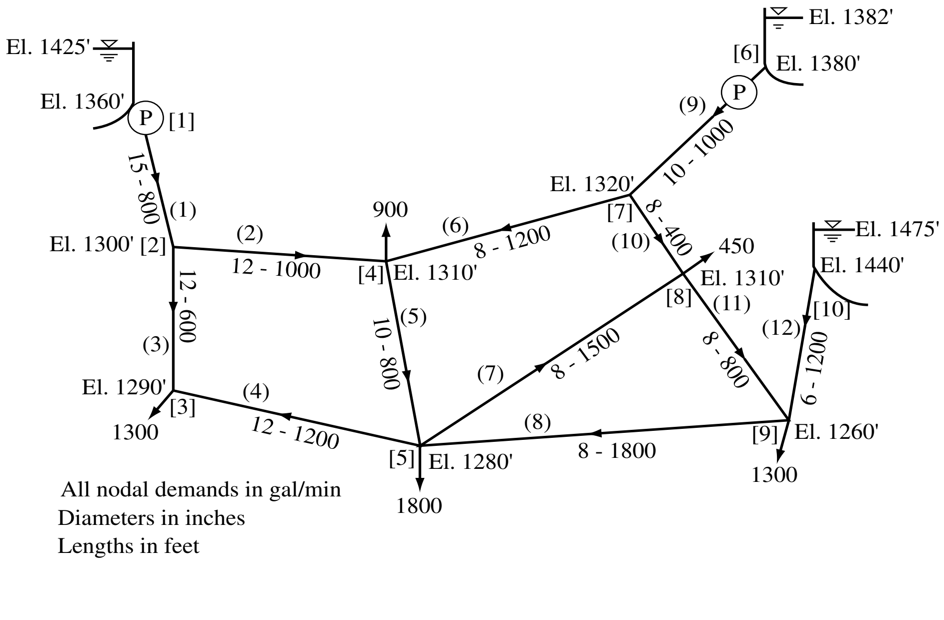
```
                    *************************
                    * PLOT FILE INFORMATION *
                    *************************

   PLOT DATA IS SAVED ON FILE:  prb12_6.plt
   TMAX =     59.99 SEC
   NUMBER OF PRESSURE HEAD VALUES IN FILE = 133
   PHMAX =  487.3 FT
   PHMIN   =  124.4 FT

   PIPE  5   NODE  6
   PIPE  5   NODE  1
   PIPE  6   NODE  5
   PIPE  6   NODE  1
```

*   *   *

Example Problem 12.7

This example examines a larger network which receives water by gravity from one reservoir and additional water that is pumped from two other reservoirs. The demands at all nodes are shown on the diagram, and the roughness is $e = 0.01$ in for all pipes.

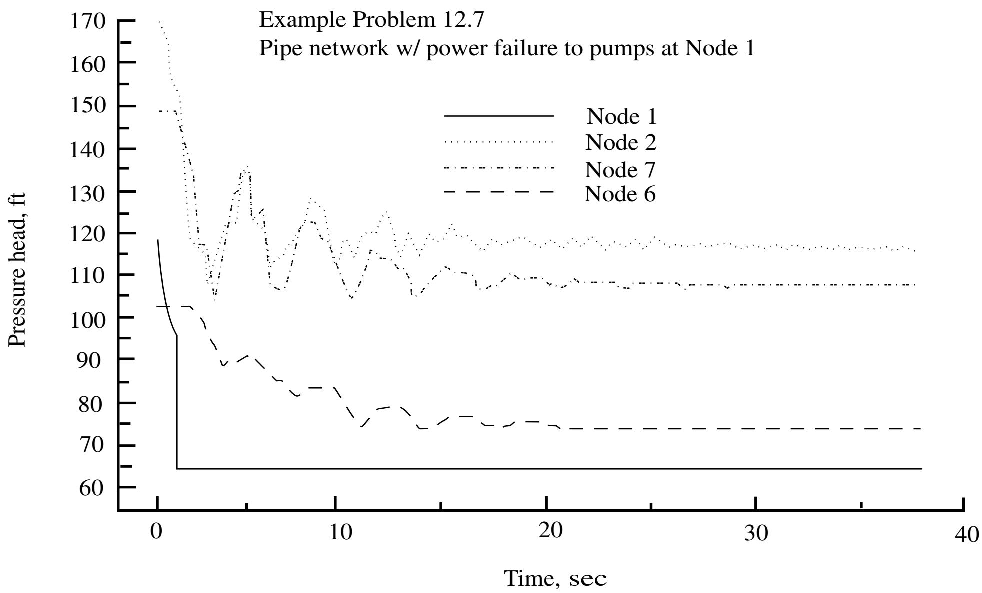
Investigate the effects on the network of power failure at the pump station in line 1. The wave speed is $a = 3000$ ft/s for all pipes. Data for the pump characteristics will be found in the input data file EPB12_7.IN.



The solution of this example problem follows the now-familiar routine. The input data to create the steady-state solution of the problem, to serve as the initial condition for the transient problem, can be found in file EPB12_7.IN on the CD; the steady-state solution is stored in file EPB12_7.OU1. In the input file we will see that the pipes and nodes have been entered in random order. Pipes and nodes need not be numbered sequentially. In addition, composite curves for the two parallel pumps, each with two stages in pipe 9 at node 6, have been developed and entered in this file; to communicate this information to

NETWK, a minus sign must precede the 2 after the RUN statement, thus indicating that there are two stages and two parallel pumps at this station. Moreover, the discharges from the second pump have been multiplied by 6, and the - 6 in the data file indicates that there are actually six parallel pumps at station 2, but that this fact is accounted for in the pump data that have been prepared.

The data for TRANSNET to solve this problem are provided in file EPB12_7.DAT on the CD, the file written by NETWK is EPB12_7.OU2, and the transient output will be found on the CD in file EPB12_7.OU3. A plot of the output data for four nodes is presented in the figure which follows.



*   *   *

12.7 PROBLEMS

12.1 Obtain steady-state solutions for the three-pipe network in Section 12.2.2. for demands QJ_3 of 2.0, 2.5, 3.0, and 3.5 ft^3/s, and compare the resulting heads at nodes 2 and 3 from each solution with the corresponding heads from the unsteady solution. The differences in these heads can be interpreted as the amount of head that is required to accelerate the fluid columns. Also compare the other discharges.

12.2 For the 6-pipe, 4-node network described below:

(a) Write the 10 equations that will allow the pipe discharges and nodal heads to be determined for any number of time increments with the specification of either the demands or heads at selected nodes. Assume $WS_1 = 100$ ft, $WS_2 = 95$ ft.

(b) Obtain the steady-flow solution for this network for the demands listed in the demand table, which will serve as the initial condition ($t = 0$) for the unsteady solution.

(c) Obtain the unsteady solution, according to rigid-column theory, over an 8-sec time period using 2-sec increments. Assume the demand at node 2 varies in the way listed in the table for QJ_2.

(d) Prepare a set of four steady-flow solutions as in (b), but replace QJ_2 for $t = 0$ with QJ_2 for $t = 2$ sec, $t = 4$ sec, $t = 6$ sec and $t = 8$ sec, respectively.

(e) Compare the pipe discharges and nodal heads from the steady-state solutions, part (d), with those from the unsteady solution, part (c).

Pipe Properties

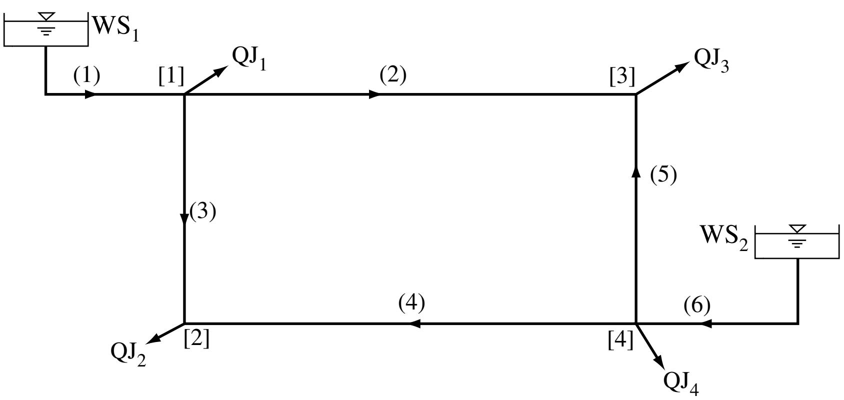
Pipe	Dia. in	Length ft
1	10	2000
2	8	2000
3	8	1500
4	8	2000
5	8	1500
6	10	2000

Demands

Node i	QJ_i ft^3/s
1	1.5
2	1.0
3	1.5
4	2.0

QJ_2 Schedule

Time sec	QJ_2 ft^3/s
0	1.0
2	1.5
4	2.0
6	2.5
8	3.0



12.3 Repeat parts (b) through (e) of problem 12.2 with this change: the Demand Schedule for QJ_2 begins at 3.0 ft^3/s at time $t = 0$, and it then decreases in 0.5 ft^3/s increments to 1.0 ft^3/s after 8 sec.

12.4 A water distribution system is shown below. In it the demands at nodes 2 and 5 change with time, but the demands at the other three nodes remain constant. For a long time there have been no changes in the demands, with the demands at nodes 2 and 5 being zero and 0.5 ft^3/s, respectively. Beginning at time $t = 0$, the demands at these two nodes change as shown in the demand table.

(a) The initial steady-flow condition must be determined before the unsteady problem can be solved. State the ΔQ-equations that can be solved to provide this initial condition.

(b) How many equations must be solved simultaneously over each time step of the unsteady problem using rigid column theory? How many of these are algebraic equations, and how many are differential equations? List the unknown variables.

(c) Write the equations that govern this unsteady problem, and describe how they are to be solved.

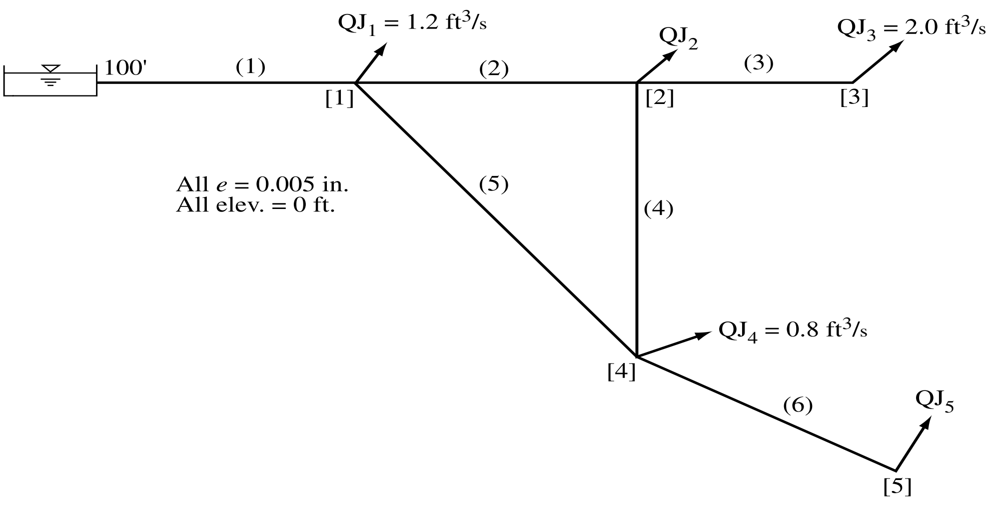
(d) Solve the unsteady problem with QJ_2 and QJ_3 varying as listed in the table.

Pipe Data

Pipe	Dia. in	Length ft
1	14	200
2	12	180
3	10	190
4	10	180
5	10	230
6	8	200

Demand Table

Time sec	QJ_2 ft^3/s	QJ_3 ft^3/s
0.0	0.0	0.5
2.0	0.5	1.0
5.0	0.7	1.2
7.5	0.8	1.0
10.0	0.8	0.8



12.5 A 6-pipe network supplied by a source pump and a reservoir is shown below; the diagram presents pipe data and the initial steady-flow nodal demands. Do the following:

(a) Prepare input data for NETWK to obtain a steady-state solution for this network.

(b) Assuming that the demands at the four nodes are to change in time, apply rigid-column theory to write the equation system that describes the unsteady-flow network problem. In performing this task, identify the unknowns and develop an appropriate number of independent equations to determine them.

(c) Prepare an input data file for the program UNSTPIPD to solve this problem if QJ_4 varies with time, as listed in the Demand Schedule.

Pump Curve

Q ft^3/s	h_p ft
2.0	35
3.0	32
4.0	28

Steady Discharges

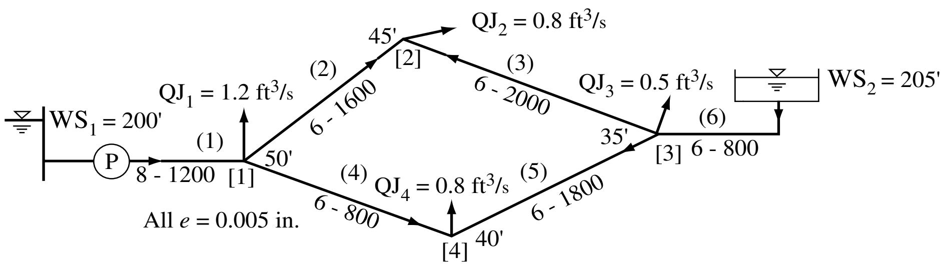
Pipe	Q ft^3/s
1	2.485
2	0.659
3	0.141
4	0.626
5	0.174
6	0.815

Steady Heads

Node	H ft
1	207.11
2	195.24
3	196.09
4	194.98

Demand Schedule

Time sec	QJ_4 ft^3/s
1.0	0.9
2.0	1.0
3.0	1.1
4.0	1.2
5.0	1.3
6.0	1.4



12.6 The network in the next diagram receives its water supply from a pump. The tank at the other end stores water during periods of low demand and supplies water during periods of larger demand. The pump characteristic curve is described by $h_p = -0.4Q^2 + Q + 75$. The demands have been constant for a long time. At time $t = 0$ the demands at nodes 3 and 4 begin to increase at a rate $dQ/dt = 0.05$ ft^3/s^2 for 20 sec and then become constant again.

(a) Describe how to obtain the initial condition for this unsteady problem, including the preparation of the input data file for NETWK to obtain a solution.

(b) Write the equations that must be solved simultaneously, using rigid-column theory, to obtain the unsteady solution at several subsequent times: 0.5, 1.0, 2.0, and 5.0 sec. For each of these times indicate which variables are unknown and which are known.

(c) For the changes shown in the Demand Schedule, solve this problem by modifying program UNSTPIP to include a pump and then applying the new program.

Pipe Data

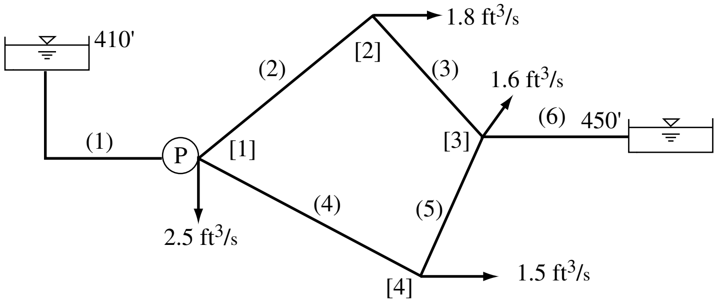
Pipe	Length ft	Dia. in
1	1000	14
2	1500	12
3	1600	12
4	1800	10
5	1400	8
6	1500	8

Nodal Data

Node	QJ ft^3/s	Elev. ft
1	2.50	350
2	1.80	360
3	1.60	340
4	1.50	335

Demand Schedule

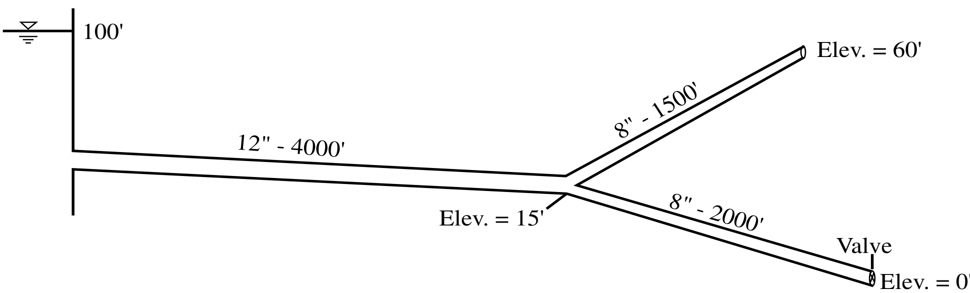
Time sec	QJ_2 ft^3/s	QJ_3 ft^3/s
0.0	1.60	1.50
0.5	1.70	1.60
1.0	1.80	1.75
2.0	1.80	2.00
5.0	1.80	2.30



12.7 A 12-in-diameter 4000 ft long pipe branches into two 8-in-diameter pipes. One is 1500 ft long which discharges freely into the air at an elevation of 60 ft. The other is 2000 ft long with a butterfly valve having a loss coefficient $K = 8000e^{8(x-1)}$ at the downstream end, in which $x = 0$ for a fully open valve, and $x = 1$ when the valve is 98% closed. The 12-in pipe is supplied by a reservoir with a water surface elevation that is 100 ft above the elevation of the valve. Do the following:

(a) Write the equation system whose solution will provide the initial condition for a transient if the valve is fully open at $t = 0$.

(b) Write the rigid-column equation system to describe the unsteady motion if the valve is 98% closed after 5 sec. Obtain this solution and provide a plot of discharge vs. time for each pipe and also a plot of head vs. time at the junction. Use Δt = 0.5 sec for about 30 time steps. Assume e = 0.004 in for all pipes, and $\nu = 1.41 \times 10^{-5}$ ft²/s.

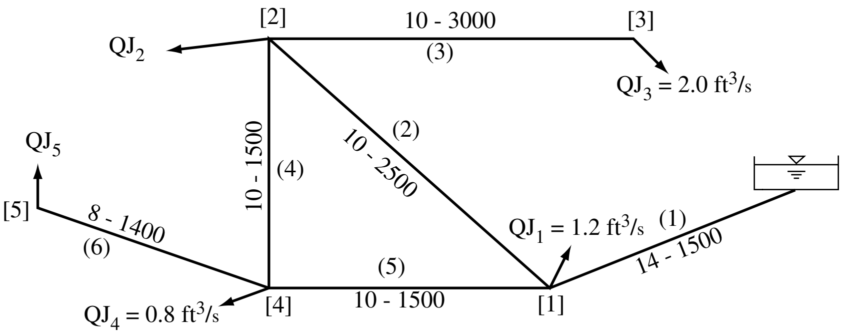


12.8 In the following network the constant demand at node 1 is QJ_1 = 1.5 ft³/s, but the demands vary at nodes 2 and 5, as are listed in the last portion of the file written by NETWK, shown below. Do the following:

(a) Indicate the appropriate unknown variables for an unsteady-flow solution based on rigid column theory.

(b) Write the system of equations that must be solved for these unknowns for each time step of the solution.

The pipe diameters and lengths, both in ft, are given in the file which follows. For all pipes e = 0.005 in = 0.000417 ft, and all nodal elevations are zero ft.



File written by NETWK with option NETPLT = 4:

```
      6    6    0    0   32.2  0.14100E-04    0.00100    3
      1    6 5555    1    1   1500.0  1.167  0.000417  6.30
      2    1    1    2    2   2500.0  0.833  0.000417  2.30
      3    2    2    3    3   3000.0  0.833  0.000417  2.00
      4    2    2    4    4   1500.0  0.833  0.000417 -1.00
      5    1    1    4    4   1500.0  0.833  0.000417  2.80
      6    4    4    5    5   1400.0  0.667  0.000417  1.00
          1.20        0.00        88.40
          1.30        0.00        73.11
          2.00        0.00        59.07
          0.80        0.00        75.00
          1.00        0.00        69.62
        - 6.30      100.00       100.00
   4 0
   2 2 2 1.5 5 1.2
   6 2 2 1.7 5 1.5
   10 2 2 1.5 5 2.0
   20 2 2 1. 5 2.5
```

12.9 Modify the program STANDPIP.FOR so it calls the ODE solver ODESOL rather than the subroutine DVERK.

12.10 Modify the program STANDPIP.FOR so the pipeline will lie at a slope S_o from the horizontal. Then obtain a solution to the following problem with this program:

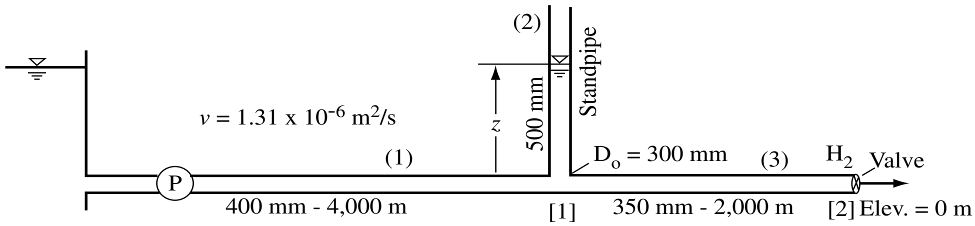
The pipeline has a 24-in diameter, a total length of 5000 ft, with a 36-in-diameter standpipe located 500 ft upstream from a butterfly valve. The diameter of the orifice opening at the base of the standpipe is 18 in. The pipeline is supplied by a constant-head reservoir at its upstream end with a water surface elevation of 40 ft. The delivery pipe slopes upward at 5 ft per 1000 ft and has a roughness $e = 0.003$ in. Assume $\nu = 1.41\text{x}10^{-5}$ ft^2/s. Laboratory tests of the butterfly valve indicate its flow coefficient C_v plots as a straight line on semi-log paper, with the valve opening in degrees (the linear scale), so that $C_v = 420$ at a 10^o opening and $C_v = 42{,}000$ at a 90^o opening, with C_v defined by Eq. C.1 in Appendix C. Initially the pipeline contains a steady discharge of 8 ft^3/s = 3590 gal/min. Starting at time $t = 0$, the valve angle changes linearly with time from its initial steady-flow position to a 3^o position in 10 sec.

12.11 Repeat problem 12.10, assuming there is an opening into the standpipe that creates a constant minor loss coefficient $K = 2$ in the expression $h_L = KV^2/(2g)$ for the head loss at the entrance to the standpipe, with V being the velocity in the standpipe. (In this problem one could also study how the maximum head at the valve and the water height in the standpipe are related to the magnitude of the loss coefficient K.)

12.12 A standpipe is located 4000 m downstream from a reservoir; the reservoir water surface is 30 m above the 400-mm-diameter horizontal supply pipe. The standpipe has a 500 mm diameter, and the orifice from the main pipe has a 300 mm diameter. Downstream 2000 m from the standpipe is a butterfly value, which is initially wide open but which can be almost entirely closed in 10 sec. The loss coefficient for this butterfly valve is $K = 10000e^{9(x-1)}$. Assume the pipe roughness is $e = 0.2$ mm, and use $\nu = 1.31 \times 10^{-6}$ m²/s for the kinematic viscosity of water. The pump in the upstream pipe has operating characteristics given by the data in the table below. Apply rigid-column theory to analyze the system for 35 sec of operation when the valve is closed over a ten-second interval (x varies linearly from 0 to 1 in 10 sec).

Pump Characteristic Data

Q, m³/s	0.20	0.25	0.30
h_p, m	31.0	29.6	29.0



12.13 The flow from three pumps is delivered to one pipe line, as shown below. All pipes have a roughness $e = 0.005$ in; assume $\nu = 1.41 \times 10^{-5}$ ft²/s for water.

(a) If the discharge for a long time is 7.5 ft³/s and the reservoir water surface elevations are $WS_1 = 50$ ft, $WS_2 = 45$ ft, and $WS_3 = 40$ ft, what are the steady discharges in pipes 1-3 and the heads at the two nodes? Assume the elevation of nodes 1 and 2 is 0.0 ft. The pump operating characteristics are given in the tables below.

(b) Determine the unsteady, rigid-column discharges and heads if the pressure at the downstream end decreases linearly from the steady value to 0 lb/in² in 5 sec, and then during the next 10 sec increases linearly to 40 lb/in². Follow the transient over 28 sec.

(c) If the discharge in the downstream pipe decreases linearly from 7.5 ft³/s to 0 in 5 sec, determine with rigid-column theory the velocities and nodal heads over the following 30 sec.

Pump 1

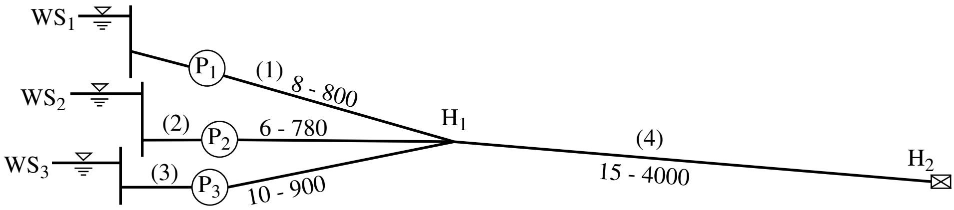
Q ft³/s	h_p ft
2.0	40
3.0	37
4.0	32

Pump 2

Q ft³/s	h_p ft
0.8	45
1.2	43
1.6	39

Pump 3

Q ft³/s	h_p ft
2.5	50
3.0	48
3.5	44

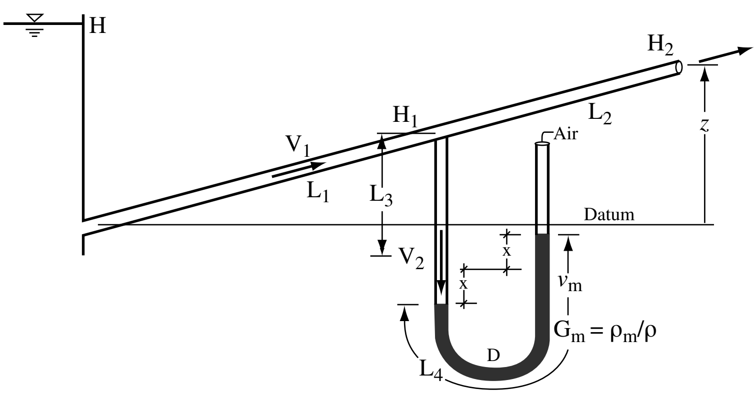


Diameters in inches
Lengths in feet

12.14 A U-tube (This is **not** a manometer that contains water on both sides of the manometer fluid and is used to measure pressure differences) taps a pipe at a distance L_1 downstream from a reservoir of constant head H, as shown in the sketch. At a distance L_2 further downstream is a valve that controls the discharge through the pipe. The discharge coefficient c_v in the relation $Q = c_v(H_2 - z)^{1/2}$ or $V_1 = c_v(H_2 - z)^{1/2}/A$ is known for the valve; in particular, it is known as a function of position during the valve closure process, and since the position of the valve is known as a function of time, c_v is known as a function of time.

Formulate the unsteady flow problem in the pipe and U-tube using rigid-column theory, i.e., write the system of equations that govern the velocity $V_1(t)$ in the pipe, the deflection $x(t) = dV_2/dt$ of the manometer fluid in the U-tube, as well as $H_1(t)$ and $H_2(t)$ for a problem in which the following variables are known: H, the specific gravity of the manometer fluid G_m, the diameter d of the pipe, the diameter D of the tube, L_1, L_2, $L = L_1 + L_2$, the distance L_3 from the pipe to the manometer fluid when $x = 0$, the length L_4 of the manometer fluid in the U-tube, and $c_v(t)$.

Time, sec	1.0	2.0	3.0	4.0	5.0
Coefficient c_v	0.50	0.10	0.05	0.025	0.02



12.15 The sketch on the next page shows a 5-pipe network. During periods of low demand water is pumped into the upper reservoir through pipe 5, but during periods of larger demand the pump is turned off, and the valve in the bypass line around the pump is opened so the upper reservoir can supply part of the demand. The low demands are shown by the outward arrows at the nodes, and the larger demands are shown thereafter in parentheses. Assume the elevation of nodes 1, 2, and 3 is 300 ft, that the pipe roughness $e = 0.002$ in, and the kinematic viscosity of the water is $\nu = 1.41\text{x}10^{-5}$ ft^2/s.

Do the following:

(a) Prepare the input to NETWK to determine the steady flows during the period of low demand.

(b) List the changes that should be made to this input file to analyze the network performance in response to the larger demands.

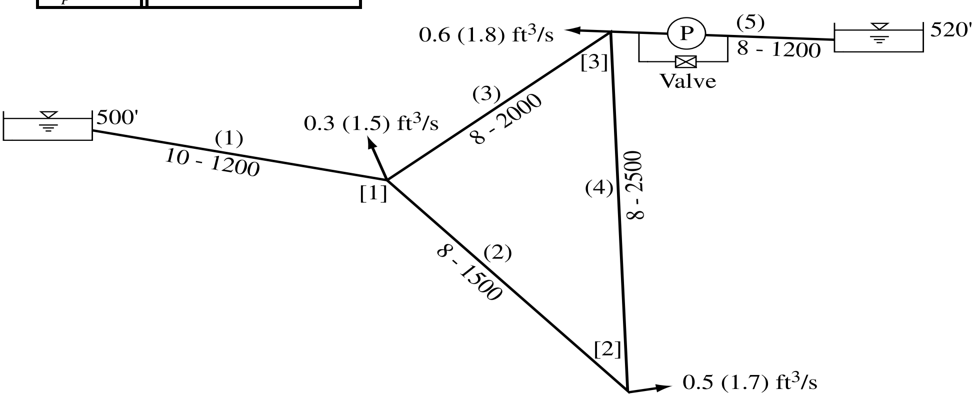
(c) Write the system of equations that would govern the solution of both steady state problems in parts (a) and (b). These should be general equations that would in principle allow pipe diameters or pump heads to be determined.

(d) Applying rigid-column theory, write the equations to be solved to obtain the unsteady discharges and nodal heads for this network if the demands change with time. What equation(s) will change, depending on whether the pump is operating?

(e) Obtain an unsteady solution with UNSTPIP when the pump is operating, the demands at all three nodes change linearly from the low values to the high values over 30 sec and thereafter remain constant over the next 70 sec.

Pump Characteristic Data

Q, ft^3/s	1.0	1.5	2.0
h_p, ft	50	48	45



12.16 An 8-in diameter ($e = 0.002$ in) horizontal pipeline obtains its water supply from a reservoir with a constant head of 50 ft above the pipeline. It has a closed cylindrical tank, with a total volume of 100 ft^3 and a cross-sectional area of 20 ft^2, connected to it at a pipe bend which is 6000 ft downstream from the reservoir. The pipe continues an additional 500 ft to a valve. The pipe that connects the pipeline to the closed tank has a 4-in diameter and is 100 ft long. The downstream valve has been open for a long time, and at $t = 0$ it is closed so the discharge through the valve is reduced linearly from the steady-state discharge to zero in 5 sec. The local loss coefficient for the valve is a function of the discharge through the valve in the form $K = 8000/e^{8Q}$. During steady flow the tank is half full of water and half full of air; when the water surface elevation in the tank is at this middle position, it has the same elevation as the centerline of the pipeline.

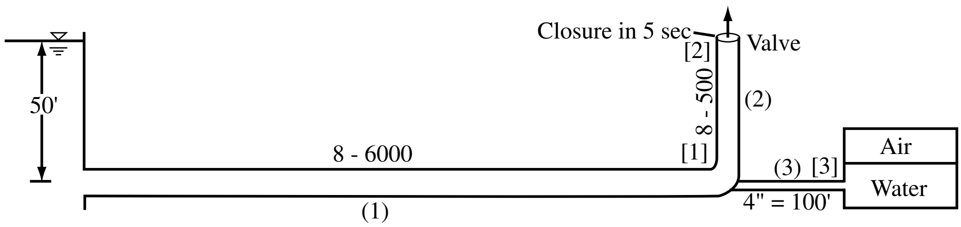
Complete the following tasks:

(a) Compute the steady-state head in the pipeline at the bend, which is node 1, when the valve is open. The steady discharge is $Q_o = 1.523$ ft^3/s.

(b) Determine the pressure and density of the air in the tank, assuming any changes are adiabatic from an initial temperature of 58 °F and pressure of 14.7 lb/in^2 absolute.

(c) Write a system of equations, using rigid-column theory, to describe the unsteady flow in the pipeline and surge tank that is caused by closing the valve in 5 sec (i.e. reducing the discharge at the end of the pipeline linearly from Q_o to 0 in 5 sec).

(d) Determine the time-dependent flow in the pipe lines and the surge tank.



12.17 Repeat problem 12.16, assuming that the total tank volume and cross-sectional area are reduced by one-half to 50 ft^3 and 10 ft^2, respectively.

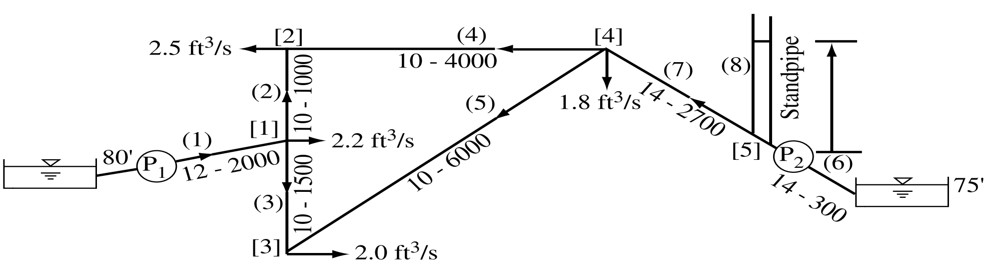
12.18 This network is supplied by two pumps. A 24-in-diameter standpipe exists downstream from the second pump, with a 6-in-diameter orifice at the entrance to the standpipe. Assume the nodal elevations are all zero feet and all pipes have roughnesses e = 0.005 in. The demand at node 4 is reduced by 0.1 ft^3/s per second until it becomes zero. Simulate with rigid-column theory the network performance over 10 sec, using 0.5 sec increments.

Pump 1

Q ft^3/s	h_p ft
1.5	60
2.5	52
3.5	41

Pump 2

Q ft^3/s	h_p ft
2.0	80
3.0	72
4.0	60



12.19 Determine the unsteady discharges and heads in the network in problem 12.18 when the demand at node 2 is reduced linearly in time to zero in 12.5 sec.

12.20 In problem 12.18 the demand at node 3 changes linearly in time from 2.0 ft^3/s to 2.5 ft^3/s and then to zero.

12.21 Program SURGNET, originally written to analyze the network in Example Problem 12.4, calls on DVERK to solve nine simultaneous first-order ODEs for the unsteady pipe discharges. Modify this program so that each call to DVERK requests the solution of only one first-order ODE. (In using DVERK in this way, note that the argument IND returns a value of 3 after a solution has been completed over the specified interval in anticipation that it will be called to continue the solution over additional increments of the independent variable.)

12.22 Modify program SURGNET, used in Example Problem 12.4, so ODESOL is used in place of DVERK to solve the system of ODEs.

12.23 Modify the program that was developed in problem 12.22 so that each call to ODESOL requests the solution of only one first-order ordinary differential equation.

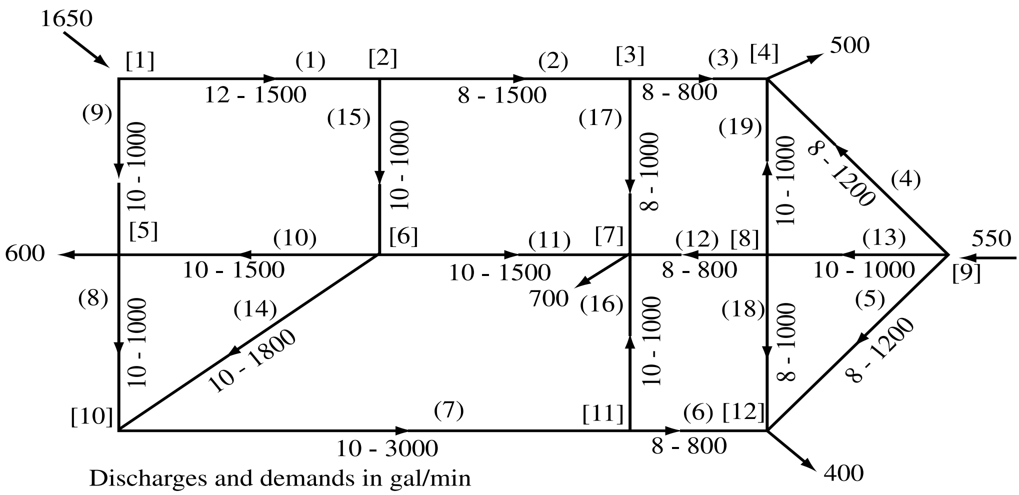
12.24 Modify the program that was developed in problem 12.23 so that the solution to the single first-order ODE is obtained by RUKUST rather than ODESOL or DVERK.

12.25 For the pipe network shown below the pressure at node 1 is 85 lb/in^2, and all pipes have a Hazen-Williams coefficient of 120. The network lies in a horizontal plane at an elevation of 1100 ft. Assume the wave speed in all pipes is 3000 ft/s.

(a) Obtain the steady state solution.

(b) Find the maximum and minimum pressures, their location and their time of occurrence if the demand at node 12 is instantly increased to 650 gal/min, and elastic effects are included in the analysis.

(c) Ignore elastic effects and increase the demand at node 12 linearly to 650 gal/min over 4 sec.



Discharges and demands in gal/min
Diameters in inches
Lengths in feet

12.26 For the network of problem 12.25, predict the consequences of a sudden stoppage of the 1650 gal/min input to the network at node 1.

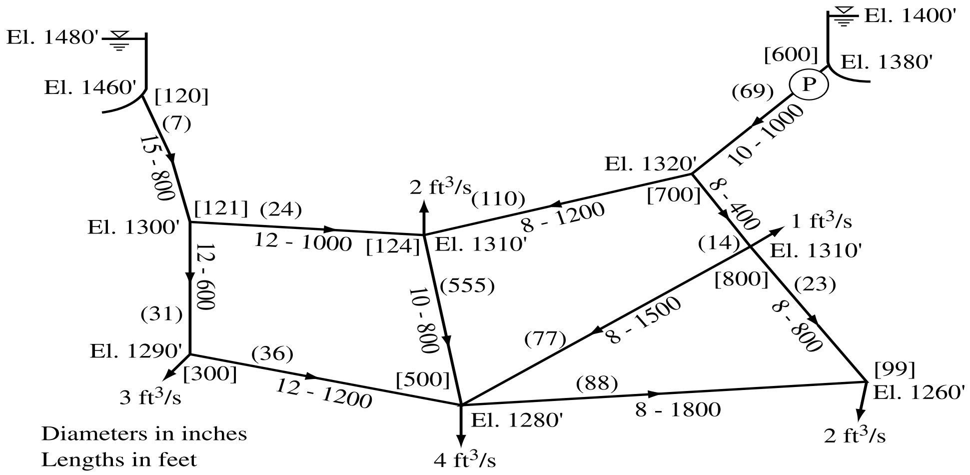
12.27 For the network of problem 12.25, find the maximum and minimum pressures, their location and time of occurrence if the valve at the downstream end of pipe 7 were suddenly closed.

12.28 For this network the pipe roughnesses are all $e = 0.02$ in, and the wave speed for each pipe is 3300 ft/s. The pump curve for the one pump in pipe 69 is defined by $h_p = -0.5Q^2 - 0.3Q + 90$, with Q in ft^3/s and h_p in ft. The pump runs at 1750 rev/min, and for this unit $Wr^2 = 40$ lb-ft^2.

(a) Obtain the steady state solution.

(b) Assuming the brake horsepower for the pump is constant at 40 hp, determine the consequences of pump power failure.

(c) Ignore elastic effects and determine the consequences if the head supplied by the pump dropped to zero in 4 sec.



12.29 For the network in problem 12.28, find the maximum and minimum pressures, their location and time of occurrence if the valve at the downstream end of pipe 7 is suddenly closed.

12.30 Solve problem 12.28 if pipe 23 is removed and the demand at node 99 is reduced to 1.0 ft^3/s.

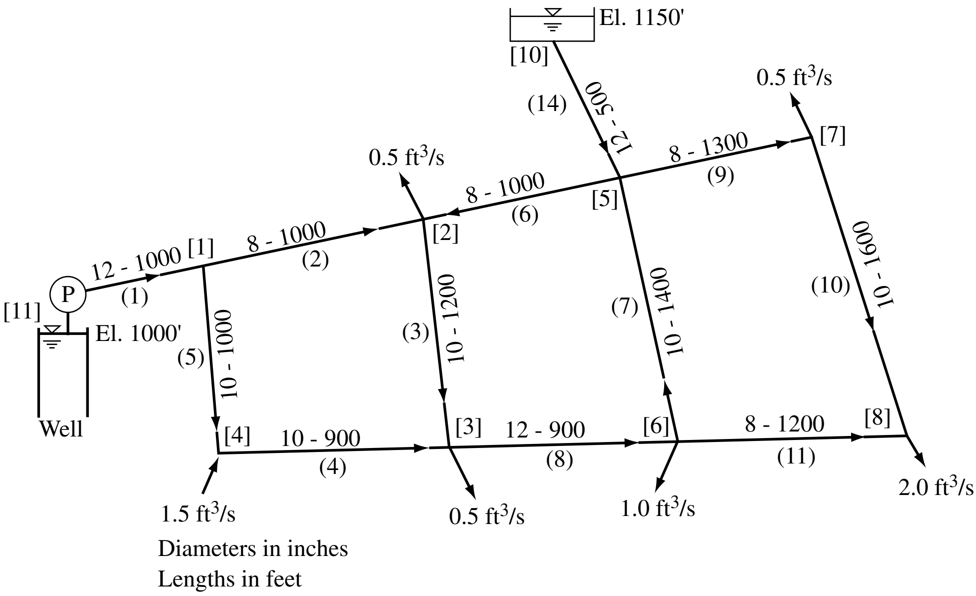
12.31 In problem 12.30 determine the consequences of sudden valve closure at the downstream end of pipe 88.

12.32 For the network shown atop the next page, all pipes have roughness $e = 0.008$ in and wave speeds of 2700 ft/s. The nodes are all at elevation 1050 ft. The pump curve for pipe 1 is $h_p = -1.5Q^2 - 1.5Q + 170$, with Q in ft^3/s and h_p in feet. The pump runs at 1180 rev/min, and for this unit $Wr^2 = 50$ lb-ft^2.

(a) Obtain the steady state solution.

(b) Assuming the brake horsepower is constant at 45 hp, determine the consequences of power failure.

(c) Let the head of the pump decrease to zero over 5 sec, ignore elastic effects, and determine how the pressures and discharges decrease throughout the network.



12.33 For the network in problem 12.32, find the maximum and minimum pressures, their location and time of occurrence if the valve at the downstream end of pipe 14 is suddenly closed.