CHAPTER 6

EXTENDED TIME SIMULATIONS AND ECONOMICAL DESIGN

6.1 INTRODUCTION

This chapter looks primarily at two topics that are important to the design of looped networks, which includes networks for water distribution to numbers of "on demand" users, as occur in large cities. These systems do not operate under steady-state conditions. First we introduce and describe "extended time simulations" to simulate the performance of these systems as they respond to demands which vary with time, and which may have pumps turned on or off, depending upon those demands. The chapter will also describe some useful elements of engineering economic analysis. Both of these topics will then be applied to the design of large looped networks. Thereafter, subsequent chapters explore methods to analyze unsteady flows, including inertial and/or elastic effects. For networks of pipes the analysis of unsteady flow requires the simultaneous solution of combined systems of ordinary differential equations and algebraic equations. The reader will be introduced in Chapter 7 to such analyses. In Chapters 8-11 progressively more comprehensive systems will be studied. In Chapter 12 true transients in looped networks will be examined.

Extended time simulations consist of a series of steady-state solutions based on changing demands and reservoir water levels, the number of operating pumps etc. This type of time-dependent solution is obtained by solving a system of simultaneous nonlinear algebraic equations, as was done in Chapters 4 and 5. Another term for this type of unsteady flow problem is "quasi-steady," since inertia is ignored and the equation of motion is a steady-state form, even though individual terms in the equation do change with time. Section 7.2 will offer additional perspective on this class of flows.

Time-dependent analyses that account for inertia require the simultaneous solution of a combined system of ordinary differential and algebraic equations. Transient analyses that also account for elastic effects must use partial differential equations in place of ordinary differential equations since the pressure and velocity now vary not only with time in each pipe but also with the position along the pipe. Not only does this mean that the computational effort increases dramatically for a solution, but also that the amount of information (numbers) that is required to describe the network behavior increases correspondingly, since it is necessary to provide pressures and velocities at a number of positions along each pipe in the network to describe the hydraulic transient after each successive time increment. For example, if the flow in a network is described by the HGL and pressure (two unknowns) at each node and the discharge and velocity (two unknowns) in each pipe in a 100-pipe, 80-node network, then the description consists of 360 numbers at an instant in time. A typical extended time simulation would use a one hour time increment, and therefore the analyst must examine these 360 values for each of 24 time steps if the simulation were for a 24-hour period. If inertia were included, then the time increment must be on the order of seconds (or less if the pipes are relatively short, as in a fire-fighting sprinkler system in a building). One would prefer not to have to conduct such an investigation over a full day. But if a solution that accounted for inertia were to be performed for only 100 time increments, then the solution consists of 36,000 values. For a transient analysis with elastic effects, instead of just two values (discharge and velocity) for each pipe, there will be two values for each pipe increment (these space increments must be compatible with the time increment), so if 20 increments are used for each pipe, the number of values in the network description jumps to 416,000. As the comprehensiveness of the network description increases, the amount of data that is needed to describe the solution adequately expands rapidly, and it becomes clear that compromises are needed.

When will an extended time simulation that ignores elastic effects be adequate? The answer is obviously subjective. For the operation of most municipal water systems the changes in demands are normally slow enough to cause the effects of inertia to be relatively minor, and certainly the elastic effects can be ignored. Furthermore, in a large network the effect of a very rapid change in flow in a single pipe, which may have a valve at one end closed rapidly, will soon be dissipated in the network of pipes. Thus it is sufficient to recognize that a high-pressure transient wave may propagate though this pipe and possibly affect a few pipes near it. There may be a few times in the operation of many water distribution systems, and other liquid distribution systems, when the neglect of inertia will cause a simulation to produce results that are notably different than those that actually occur. Such conditions may occur when major flows are changed in seconds, or perhaps minutes. For shorter pipes these changes may be more rapid without creating a significant change in pressures and discharges that is attributable to inertial effects.

6.2 EXTENDED TIME SIMULATIONS

This section describes a type of time-dependent solution that has become known as an "extended time simulation." These solutions are for pipe networks rather than single pipes. Since this type of solution ignores both elastic and inertial effects, the solutions are actually a series of steady-state solutions in which a past solution is updated over a time increment in response to changes in time-dependent parameters to the new solution for the new instant in time. Thus these time-dependent solutions are quasi-steady solutions. The following six items commonly change in extended time simulations:

1. Demands at nodes. The nodal demands will change in almost all extended time simulations, and a typical means of specifying these changes is to provide peaking factors as functions of time for selected groups of nodes. Such changes in demand patterns over time might be thought of as demand schedules.
2. Storage versus elevation relations for reservoirs. Some reservoirs may have constant water surface elevations, but most are storage tanks with a water surface elevation that varies with time as water is withdrawn from, or added to, the tank. Typically a storage versus water surface elevation function is constructed to describe changing reservoir water surface elevations. When this function is described by data pairs for water surface elevation and volume in storage, then the bottom water surface elevation will be the lowest operating level of the tank, and the largest water surface elevation will be the top of the tank.
3. Pump schedules. A pump schedule states how many pumps must operate in parallel or in series at a given station at any time. In other words, a schedule specifies the number of pumps that are turned on for each time step. An alternative is to specify the rotational speed of a pump as a function of time.
4. Pump rules. A pump rule relates the number of operating pumps to either the magnitude of the pressure (or HGL) at a selected node, or the water level in a reservoir. Rules are distinguished from schedules by a condition that dictates the number of pumps in operation rather than having pumps start or stop at a specified time. Instead of specifying the number of operating pumps, the rule might give the rotational speed of a pump.
5. Flow rules. The difference between flow rules and demand functions (schedules) is the same as between pump rules and pump schedules. That is, the demand at selected nodes is determined by the pressure at some node or by the water surface

elevation in a reservoir. Flow rules would typically be given for negative demands, which are external flows coming into the network.

6. Discharge rules. Specify the discharge that must exist in selected internal pipes in the network. Internal pipes are distinguished from dead end pipes and pipes that connect supply sources to the network.

There are many additional items that might be a part of the specifications that describe the time-dependent solution, such as the following:

7. Schedules for valves. These schedules may specify the valve setting (percent open) as a function of time, which may in turn employ a relation between valve position and head loss to determine how the valve restricts the flow, or specify the valve loss coefficient as a function of time.
8. Rules for valves. The rules can either prescribe the valve setting (percent open) or give its loss coefficient as a function of the pressure at a node. In place of pressure, the rule may be based on the water surface elevation in a reservoir.
9. Differential head devices. These devices may specify the amount of differential head (positive or negative) in selected pipes as a function of time, i.e., a schedule of head losses in pipes, or the amount of the differential head may be computed so that a specified HGL (or pressure) is achieved at a selected node, and the HGL may vary with time.
10. Tank level or pressure control algorithms. Such algorithms simulate controllers that may activate valves etc. to maintain the water levels in reservoirs at or between specified limits, or to maintain a pressure at a designated value, or between specified limits, by changing the flow into the network or adjusting a valve setting.

It is common to implement these items, which prescribe changes in network behavior over the next time step, and which are rules based on pressure or water surface elevation, in terms of values that are taken from the solution for the current time instant. In other words, the implementation of the rule lags the solution itself by one time step. To do otherwise would require an iterative approach.

We will not describe any implementation details for these rules. However, as they act to change the network behavior over each time step, it is generally not necessary to redefine the equations that govern the mathematical problem as if a new network problem were being solved. Instead the existing equations are simply modified to reflect the conditions that apply to the new time step. For example, to change nodal demands we simply change the values of those demands. But when a pump is turned off or a pressure reduction valve opens fully, the type and/or number of equations that describes the system must be altered.

Example Problem 6.1

Obtain an extended time simulation for the 30-pipe, 16-node network described in Chapter 5 and shown in Fig. 5.15, using the diameters (for all pipes $e = 0.004$ in) found there by using DESIGN=1 that are listed in the pipe data table. The following specifications control this simulation: (1) The storage tank attached to the network by pipe 30 is circular with a diameter of 115 ft, and its bottom is at elevation 590 ft; at the beginning of the simulation its water surface elevation is 605 ft. (2) Two different demand functions are described on the graph which follows; the first applies to the north portion of the network at nodes 1, 2, 5, 6, 9, 10, 13, and 14, and the second applies to nodes 3, 4, 7, 8, 11, 12, 15, and 16. (3) Initially three pumps are in parallel at each pump station, and the tables give pump characteristics that apply to all three operating pumps. The number of operating pumps is given by the pump schedule.

PUMP SCHEDULE

	Pump Station 1					Pump Station 2				
Time, hr.	0	8	10	15	17	0	5	8	15	20
Number operating	3	2	1	2	3	3	2	1	2	3

NODE DATA

No.	Demand ft^3/s	Elevation ft
1	1.2	500
2	1.2	490
3	0.8	485
4	1.6	480
5	1.4	495
6	1.2	494
7	1.0	490
8	0.8	483

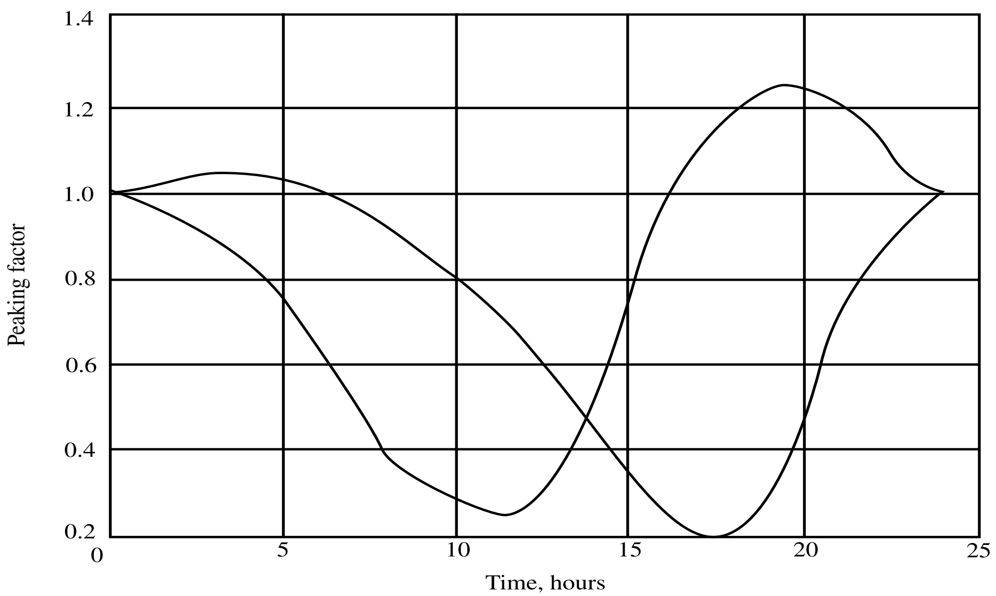
No.	Demand ft^3/s	Elevation ft
9	2.0	493
10	2.0	492
11	3.6	488
12	2.8	484
13	4.0	480
14	2.0	478
15	1.8	475
16	2.0	470

PIPE DATA

No.	Length ft	Diameter in
1	500	18
2	500	15
3	800	12
4	800	6
5	800	12
6	1800	12
7	1800	12
8	1800	12
9	1800	10
10	800	6

No.	Length ft	Diameter in
11	800	6
12	800	6
13	1600	12
14	1600	12
15	1600	12
16	1600	12
17	800	6
18	800	6
19	800	6
20	1600	12

No.	Length ft	Diameter in
21	1600	8
22	1600	10
23	1600	6
24	800	6
25	800	6
26	800	6
27	2500	6
28	2500	6
29	2500	6
30	1000	10



We begin the solution of this problem by reading key demand function data from the two plots of peaking factor at points where the curves break; this step allows us to digitize the demand functions (each function can have a separate set of times), as listed in this table:

Hour:	0	2	3.5	6.5	10	11.5	14	17	19	20	22	24
DF(1)	1.0	0.95	0.875	0.625	0.30	0.25	0.50	1.10	1.23	1.25	1.10	1.0
DF(2)	1.0	1.05	1.06	1.00	0.80	0.68	0.50	0.20	0.30	0.50	0.85	1.0

A solution from NETWK can be obtained by first adding the option ISIML=1 to the $SPECIF list of options and supplying input that would describe the network appropriately, and then adding the additional lines that describe the extended time simulation that is desired. The input file follows:

```
Extended time simulation for example 30 pipe network.
/*
$SPECIF ISIML=1,NODESP=0 $END
PIPES
1 0 2 500 18 .004
2 0 3 500 15
3 2 1 800 12
4 2 3 800 6
5 3 4 800 12
6 1 5 1800 12
7 2 6 1800 12
8 3 7 1800 12
9 4 8 1800 10
10 6 5 800 6
11 6 7 800 6
12 7 8 800 6
13 5 9 1600 12
14 6 10 1600 12
15 7 11 1600 12
16 8 12 1600 12
17 10 9 800 6
18 10 11 800 6
19 11 12 800 6
20 9 13 1600 12
21 10 14 1600 8
22 11 15 1600 10
23 12 16 1600 6
24 14 13 800 6
25 14 15 800 6
26 15 16 800 6
27 2 5 2500 6
28 2 7 2500 6
29 3 8 2500 6
30 0 14 1000 10
RESER
30 605
PUMPS
1 2.67 157 5 152 7.33 144 500
2 2 152 4 147 6 139 500
PARALLEL
1 3
2 3
NODES
1 1.2 500
2 1.2 490
3 0.8 485
4 1.6 480
5 1.4 495
6 1.2 494
7 1.0 490
8 0.8 483
9 2.0 493
10 2.0 492
11 3.6 488
12 2.8 484
13 4.0 480
14 2.0 478
15 1.8 475
16 2.0 470
RUN


$TDATA ALTV=0,HTIME=24,INCHR=1,ISUNIT=0,LINEAR=1,NPUNOD=2,PRINTT=3 $END
PIPE TABLE
ALL
NODE TABLE
ALL
RESER. TABLE
ALL
END TABLES
STORAGE FUNCTION
1 590 0 600 103870 605 155805/
30/
```

```
DEMAND FUNCTION
1 0 1 2 .95 3.5 .875 6.5 .625 10 .3 11.5 .25 14 .5 17 1.1 19 1.23 20 1.25 22 1.1 24 1./
1 2 5 6 9 10 13 14/
2 0 1 2 1.05 3.5 1.06 6.5 1 10 .8 11.5 .68 14 .5 17 .2 19 .3 20 .5 22 .85 24 1/
3 4 7 8 11 12 15 16/
PUMP SCHEDULES
1 2 0 3 8 2 10 1 15 2 17 3/
2 2 0 3 5 2 8 1 15 2 20 3/
END SIMULATION
END
```

The input after the RUN command provides specifications for the time-dependent solution. A brief explanation of this part of the file (see the NETWK manual for more detail) follows:

1. The $TDATA line sets options associated with the extended time simulation: (a) ALTV=0 tells NETWK to extrapolate the volume-elevation data that is provided for the storage tank beyond the given limiting values; if ALTV=1, then the tank will no long supply water when the elevation falls to the smallest elevation in the data, nor will it fill further if the water surface elevation reaches the largest elevation in the data; (b) HTIME=24 indicates the simulation is to cover 24 hours, the default; (c) INCHR=1 indicates one-hour increments and is also the default; (d) ISUNIT=0 indicates that storage volumes will be given in ft^3; (e) LINEAR=1 specifies a linear interpolation (or extrapolation if necessary) of given data; (f) NPUNOD=2 indicates that source pumps and reservoirs will be referenced by pipe number; (g) NPRINTT=3 tells NETWK to write special tables, with time in the first column, for pressure at designated nodes and discharges in designated pipes.
2. The ALL after PIPE TABLE and NODE TABLE indicates all pipes and nodes are to be in these special tables; similarly, all reservoirs are to have their water surface elevations reported in the tables.
3. The individual demand functions are described next under the command DEMAND FUNCTION. Each separate demand function consists of two lines; the first value on the first line is a number the user chooses to assign to this demand function as an identifier, which is followed by time and peaking factor data pairs. The second line indicates the nodes at which this demand function applies.
4. After the PUMP SCHEDULES command the second value on each line, a 2 after the number of the pump station has been given, indicates parallel pump operation, and the times and numbers of operating pumps are given thereafter as pairs.

The special tables follow; the varying discharges in pipes 1, 2, 4, 18, 25, and 30 are plotted in a figure, and the pressures at nodes 1, 2, and 16, plus the water surface elevation in the storage tank, are plotted in the other figure. From this simulation we note that the storage tank initially has a water surface elevation of 605 ft and ends the 24-hour period with a water surface elevation of 603 ft. In other words the tank will not be full at the beginning of the next day; hence the capacity of either one pump or both pumps should be increased, or the lengths of time intervals when pumps are in operation should be increased. The discharge reverses direction in several pipes over the 24-hour period, including pipe 30 connecting the storage tank to the network. For the first 7 hours the storage tank supplies water to the network; then it fills until 18 hours, and thereafter it again supplies water. If the middle point used to define the pump curves is the normal capacity, then the discharge at maximum efficiency for station 1 is 15 ft^3/s at the start of pump operation, and for station 2 the discharge is 12 ft^3/s. When the number of operating pumps is reduced from 3 to 2 and then to 1 during the period of lower demand, the pumps are then producing flows that are considerably above their normal capacities, as seen in the plot of discharges in pipes 1 and 2 in relation to the normal

Pressure (lb/in^2) at Nodes as a Function of Time

Hours	Node Number								
	1	2	3	4	5	6	7	8	9
1	57.67	67.26	66.95	65.71	52.12	53.53	54.89	55.53	46.51
2	57.79	67.29	66.81	65.47	52.31	53.53	54.62	55.09	46.79
3	57.91	67.33	66.67	65.23	52.49	53.55	54.34	54.63	47.06
4	58.36	67.54	66.74	65.29	53.21	54.08	54.54	54.75	48.00
5	58.98	67.85	66.93	65.56	54.17	54.90	55.05	55.25	49.22
6	59.40	67.97	65.09	63.99	54.85	55.39	54.41	54.56	50.11
7	60.16	68.37	65.47	64.50	56.02	56.47	55.20	55.40	51.60
8	61.04	68.84	66.03	65.27	57.40	57.82	56.34	56.66	53.41
9	59.10	66.38	59.94	59.75	56.19	56.41	52.59	52.92	52.95
10	60.49	67.32	61.46	61.53	58.21	58.34	54.67	55.25	55.40
11	55.80	61.76	60.70	60.94	55.04	55.27	53.84	54.97	53.46
12	57.17	62.97	62.36	62.88	56.65	56.90	56.05	57.46	55.23
13	57.32	63.16	63.24	63.95	56.79	57.16	57.17	58.85	55.32
14	56.44	62.49	63.48	64.35	55.71	56.20	57.40	59.47	53.96
15	55.70	61.99	63.64	64.65	54.75	55.68	57.46	59.97	52.66
16	60.64	68.04	69.64	70.75	58.12	60.18	62.97	65.93	54.38
17	59.26	67.45	69.76	71.06	55.91	58.91	63.47	66.61	51.55
18	59.30	68.68	70.70	72.23	54.48	58.75	64.99	68.40	48.84
19	58.46	68.27	70.23	71.62	53.12	57.74	63.98	67.31	47.06
20	57.54	67.82	69.72	70.94	51.61	56.61	62.86	66.13	45.05
21	56.84	67.40	69.36	69.90	50.48	55.22	60.18	63.20	43.64
22	57.06	67.29	68.45	68.38	51.03	54.66	57.83	60.34	44.59
23	57.19	67.17	67.60	66.92	51.29	53.86	56.03	57.71	45.12
24	57.37	67.18	67.24	66.28	51.61	53.59	55.35	56.55	45.65

Pressure (lb/in^2) at Nodes as a Function of Time (cont'd)

Hours	Node Number							Reservoir Water Surface Elev., ft
	10	11	12	13	14	15	16	
1	49.07	48.63	50.56	48.83	52.34	50.69	46.73	605.00
2	49.06	48.14	50.00	49.17	52.19	50.12	45.80	604.18
3	49.07	47.62	49.43	49.49	52.05	49.52	44.84	603.40
4	49.70	47.79	49.57	50.53	52.30	49.67	44.88	602.65
5	50.66	48.42	50.17	51.77	52.77	50.31	45.61	602.00
6	51.33	48.19	49.89	52.62	53.15	50.36	45.84	601.53
7	52.56	49.18	50.88	54.16	53.39	51.30	47.09	601.24
8	54.09	50.67	52.39	56.16	53.61	52.68	49.06	601.19
9	53.38	48.18	49.51	56.30	53.64	51.17	47.83	601.41
10	55.65	50.70	52.12	59.01	54.45	53.60	51.07	601.59
11	53.69	50.47	52.06	57.75	54.44	53.75	51.83	602.05
12	55.43	53.13	54.84	59.57	55.41	56.26	55.32	602.45
13	55.71	54.58	56.39	59.58	56.01	57.40	57.34	603.04
14	54.63	55.08	57.16	57.92	55.85	57.90	58.55	603.71
15	53.99	55.45	57.85	56.34	55.65	58.49	59.69	604.28
16	57.65	60.61	63.90	56.84	56.49	63.40	65.37	604.75
17	56.11	61.46	64.93	53.95	55.48	64.67	66.83	605.36
18	55.41	63.44	67.04	50.65	55.30	67.04	69.39	605.61
19	54.25	62.07	65.76	48.71	55.09	65.51	67.74	605.60
20	52.93	60.56	64.36	46.47	54.59	63.81	65.98	605.39
21	51.22	56.41	60.45	44.98	53.65	58.86	60.54	605.02
22	50.49	53.10	56.71	46.24	53.07	55.19	55.58	604.44
23	49.25	50.74	53.37	47.03	52.11	52.93	51.14	603.80
24	48.93	49.62	51.88	47.72	51.79	51.70	48.88	603.03

Discharges (ft^3/s) in Pipes as a Function of Time

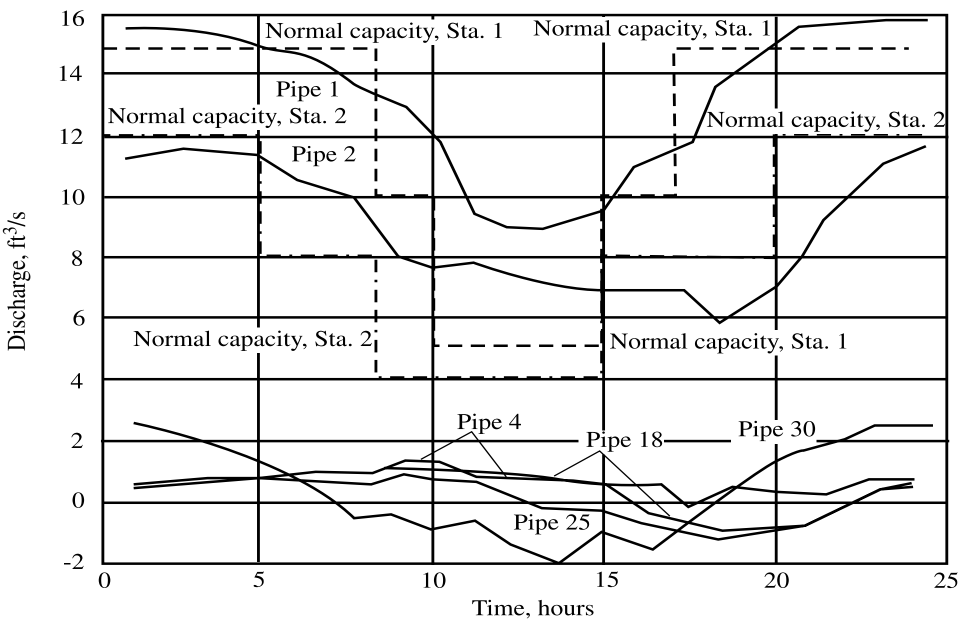
Hours	Pipe Number									
	1	2	3	4	5	6	7	8	9	10
1	15.71	11.33	6.05	0.66	4.83	4.85	6.10	5.52	3.23	0.40
2	15.66	11.46	6.00	0.68	4.91	4.83	6.11	5.56	3.27	0.36
3	15.62	11.59	5.95	0.70	4.99	4.81	6.11	5.60	3.31	0.31
4	15.33	11.53	5.80	0.72	4.99	4.72	6.03	5.56	3.30	0.26
5	14.90	11.35	5.61	0.74	4.93	4.61	5.89	5.47	3.25	0.21
6	14.73	10.52	5.42	0.95	4.73	4.52	5.79	5.11	3.09	0.12
7	14.15	10.26	5.17	0.96	4.64	4.37	5.60	4.98	3.02	0.04
8	13.43	9.86	4.88	0.95	4.47	4.19	5.34	4.79	2.92	- 0.04
9	12.91	8.07	4.49	1.26	3.99	3.90	5.02	3.95	2.53	- 0.18
10	11.98	7.58	4.12	1.22	3.76	3.65	4.70	3.72	2.39	- 0.21
11	9.33	7.83	3.29	0.75	3.59	2.93	3.77	3.74	2.31	- 0.17
12	8.82	7.27	3.12	0.70	3.31	2.80	3.60	3.51	2.16	- 0.16
13	8.74	6.96	3.16	0.60	3.10	2.80	3.57	3.40	2.07	- 0.09
14	9.03	6.86	3.39	0.44	2.93	2.91	3.69	3.41	2.01	0.09
15	9.24	6.81	3.63	0.29	2.75	3.03	3.70	3.45	1.95	0.28
16	11.21	6.82	4.58	0.30	2.63	3.74	4.31	3.67	1.99	0.52
17	11.84	6.70	5.16	- 0.14	2.36	4.08	4.55	3.50	1.88	0.67
18	13.68	5.65	5.93	0.14	2.00	4.61	5.01	3.23	1.68	0.83
19	14.29	6.20	6.18	0.17	2.23	4.79	5.20	3.49	1.83	0.86
20	14.94	6.75	6.45	0.20	2.47	4.98	5.40	3.74	1.99	0.91
21	15.52	8.77	6.61	0.17	3.29	5.11	5.68	4.62	2.49	0.88
22	15.67	9.82	6.42	0.41	3.88	5.01	5.80	5.09	2.80	0.75
23	15.82	10.70	6.29	0.54	4.40	4.97	5.99	5.38	3.04	0.61
24	15.81	11.05	6.18	0.60	4.63	4.92	6.06	5.47	3.15	0.51

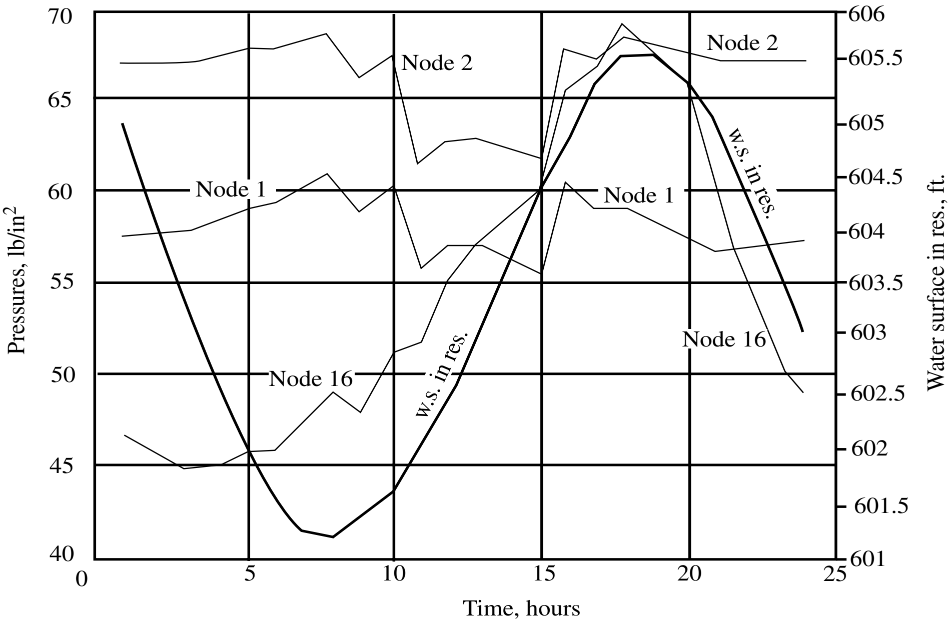
Discharges (ft^3/s) in Pipes as a Function of Time (cont'd)

Hours	Pipe Number									
	11	12	13	14	15	16	17	18	19	20
1	0.24	0.64	4.72	4.26	4.95	3.91	0.61	0.61	- 0.17	3.32
2	0.32	0.67	4.68	4.26	5.03	3.97	0.56	0.68	- 0.13	3.29
3	0.39	0.69	4.65	4.26	5.11	4.02	0.52	0.75	- 0.10	3.26
4	0.46	0.70	4.56	4.23	5.12	4.01	0.46	0.80	- 0.07	3.22
5	0.52	0.70	4.46	4.16	5.08	3.97	0.41	0.84	- 0.04	3.20
6	0.69	0.71	4.37	4.09	4.94	3.78	0.35	0.94	0.06	3.22
7	0.73	0.70	4.24	4.03	4.86	3.71	0.29	0.96	0.06	3.19
8	0.75	0.69	4.06	3.94	4.73	3.59	0.19	0.96	0.05	3.09
9	1.00	0.69	3.72	3.61	4.24	3.14	- 0.01	1.13	0.25	2.73
10	0.99	0.65	3.51	3.45	4.05	2.98	- 0.16	1.10	0.21	2.56
11	0.75	0.57	2.83	2.84	3.78	2.85	- 0.17	0.95	0.14	2.06
12	0.67	0.52	2.74	2.77	3.56	2.67	- 0.18	0.85	0.05	2.02
13	0.54	0.48	2.77	2.76	3.40	2.57	- 0.08	0.71	- 0.10	2.09
14	0.29	0.40	2.94	2.83	3.26	2.46	0.18	0.46	- 0.23	2.32
15	- 0.08	0.29	3.13	2.90	3.08	2.33	0.38	0.20	- 0.33	2.51
16	- 0.42	0.10	3.95	3.36	3.28	2.26	0.70	- 0.45	- 0.51	3.25
17	- 0.70	- 0.12	4.22	3.50	3.08	1.98	0.86	- 0.80	- 0.54	3.28
18	- 0.90	- 0.24	4.72	3.76	2.81	1.69	1.06	- 1.07	- 0.56	3.58
19	- 0.90	- 0.21	4.88	3.83	3.03	1.87	1.11	- 1.05	- 0.58	3.66
20	- 0.90	- 0.18	5.06	3.92	3.25	2.06	1.17	- 1.03	- 0.60	3.77
21	- 0.76	0.04	5.16	4.06	3.96	2.76	1.14	- 0.78	- 0.63	3.80
22	- 0.49	0.29	5.02	4.13	4.37	3.26	0.99	- 0.38	- 0.57	3.66
23	- 0.26	0.48	4.92	4.32	4.59	3.62	0.81	0.19	- 0.38	3.53
24	- 0.05	0.56	4.84	4.34	4.76	3.78	0.71	0.42	- 0.29	3.45

Discharges (ft^3/s) in Pipes as a Function of Time (cont'd)

Hours	Pipe Number									
	21	22	23	24	25	26	27	28	29	30
1	1.04	2.14	0.94	0.68	0.72	1.06	0.86	0.84	0.84	2.36
2	1.07	2.16	0.96	0.61	0.77	1.09	0.86	0.85	0.85	2.26
3	1.10	2.18	0.98	0.54	0.83	1.12	0.85	0.86	0.86	2.16
4	1.16	2.19	0.99	0.38	0.84	1.13	0.83	0.86	0.86	1.86
5	1.25	2.19	0.98	0.13	0.82	1.12	0.81	0.86	0.85	1.37
6	1.30	2.10	0.96	- 0.22	0.85	1.10	0.79	0.88	0.80	0.83
7	1.45	2.12	0.94	- 0.53	0.77	1.08	0.76	0.87	0.79	0.13
8	1.63	2.15	0.92	- 0.78	0.62	1.02	0.72	0.84	0.76	- 0.63
9	1.53	1.82	0.83	- 0.79	0.82	1.00	0.67	0.89	0.66	- 0.53
10	1.72	1.85	0.79	- 0.99	0.61	0.92	0.62	0.85	0.63	- 1.32
11	1.46	1.71	0.75	- 0.86	0.58	0.85	0.50	0.66	0.60	- 1.14
12	1.57	1.77	0.70	- 0.95	0.26	0.74	0.47	0.62	0.56	- 1.72
13	1.52	1.89	0.67	- 0.89	- 0.11	0.62	0.47	0.57	0.53	- 1.92
14	1.39	1.88	0.64	- 0.72	- 0.35	0.51	0.50	0.53	0.51	- 1.66
15	1.32	1.81	0.60	- 0.51	- 0.51	0.40	0.53	0.49	0.49	- 1.34
16	1.71	1.90	0.63	- 0.45	- 1.01	0.17	0.66	0.52	0.50	- 1.77
17	1.65	1.74	0.60	0.32	- 1.20	0.00	0.73	0.46	0.46	- 0.72
18	1.58	1.59	0.56	0.82	- 1.39	- 0.16	0.83	0.44	0.41	0.05
19	1.45	1.65	0.59	1.00	- 1.30	- 0.09	0.86	0.48	0.45	0.59
20	1.32	1.73	0.62	1.15	- 1.21	- 0.02	0.90	0.52	0.49	1.08
21	1.20	2.01	0.72	1.20	- 0.83	0.28	0.92	0.63	0.62	1.67
22	1.17	2.13	0.80	1.04	- 0.36	0.55	0.90	0.73	0.71	1.86
23	1.12	2.10	0.86	0.87	0.27	0.84	0.89	0.80	0.78	2.22
24	1.12	2.13	0.90	0.75	0.48	0.95	0.88	0.82	0.81	2.21



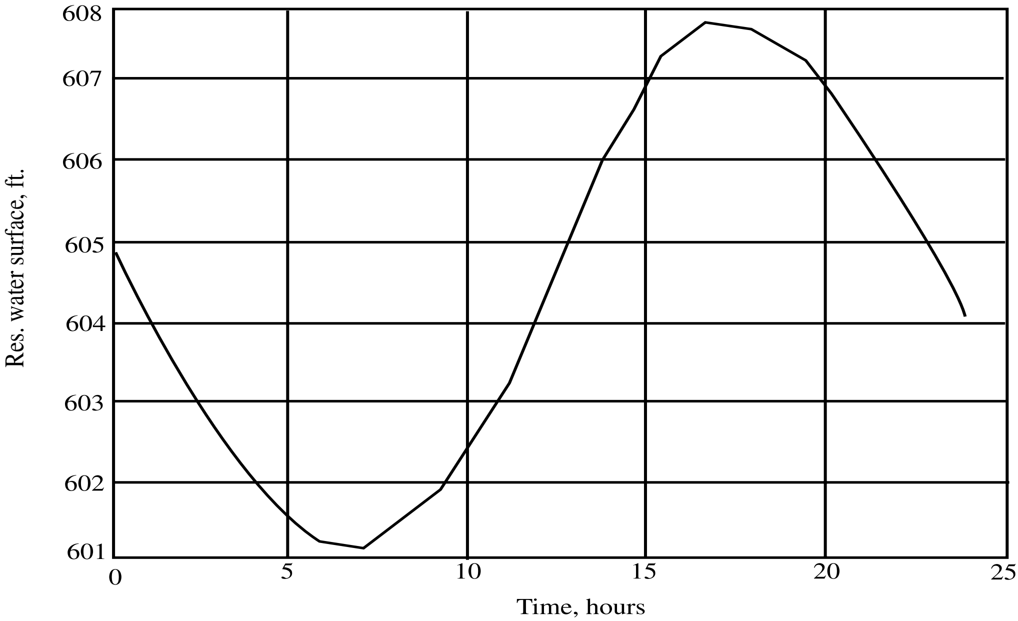


capacity lines on the figure, with accompanying reductions in efficiency. In fact, it would appear that the pump schedule should never reduce the number of pumps in operation at either station to one; then the tank would be full at the end of the 24-hour period.

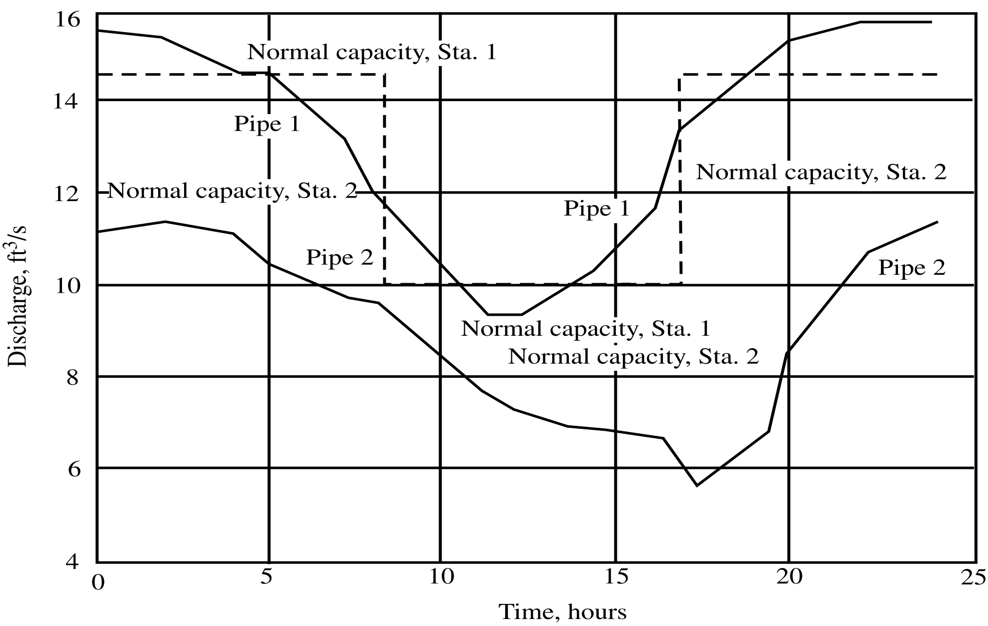
The simulation can be run again with the following changes to the input data:

PUMP SCHEDULES
1 2 0 3 8 2 17 3/
2 2 0 3 5 2 20 3/

The solution then shows that the tank ends the simulation period with a water surface at elevation 604.03 ft, and the discharges in pipes 1 and 2 are more nearly at their normal capacities, as the graphs below show. Can the reader develop an operating scenario that would cause the tank to end the period with a water surface elevation of 605 ft?



Res. water surface, ft.
608
607
606
605
604
603
602
601
0
5
10
15
20
25
Time, hours



Discharge, ft³/s
16
14
12
10
8
6
4
0
5
10
15
20
25
Time, hours
Normal capacity, Sta. 1
Pipe 1
Normal capacity, Sta. 2
Pipe 2
Normal capacity, Sta. 2
Pipe 1
Pipe 2
Normal capacity, Sta. 1
Normal capacity, Sta. 2

* * *

6.3 ELEMENTS OF ENGINEERING ECONOMICS

As with most engineering endeavors, the design process for water distribution systems explores alternative solutions to a given situation, analyzes these alternatives and then relies on the designer's engineering experience and judgment to select the best alternative. An important element of this process is economics, wherein the total cost of the delivery of water at the pressures and in the quantities that are required is examined. A brief review of engineering economics is given here as a base for the consideration of economics in the selection of pipe and pump sizes. This review may repeat parts of the reader's first course in engineering economics or another economics course; this background is essential in understanding some of the material that follows.

In engineering economics we seek the least cost solution, that is, the one that calls for the smallest overall expenditure over its expected life, taking into account the time value of money. The costs can be divided into two major categories, those needed now to build the system and start its operation, and the recurring annual costs to keep the system in operation. The first category is commonly called the capital investment, and the second category contains the operating costs. An alternative way to view these two categories (but with yearly income) is to consider the present value of an annuity (a recurring payment) over some time interval as equivalent to the capital investment, and the amount of the annual annuity as the operating cost. If these two categories of costs are to be combined to provide the total cost, the two must be put on an equivalent basis, considering that there is interest (value) associated with the use of money. To develop a fair comparison, we work with the terms **present worth** and **series payment**. A capital investment cost adds directly to the present worth, but a recurring cost must be multiplied by a **present worth factor**, **pwf**, before it is added to the capital investment to compute the overall present worth. Similarly, recurring costs are added directly to give the series payment amount, but capital investment amounts must be multiplied by a **capital recovery factor**, **crf**, before being added together to obtain the total series payment amount. In other words, the two alternatives are either (1) to convert recurring or annual operating costs to the equivalent of a capital investment by multiplying by the pwf, or (2) converting capital investments to series costs by multiplying them by the crf. We assume that the individual payments are a constant amount so we are dealing with a uniform series payment.

The formula for the present worth factor pwf is

$$pwf = \frac{1}{i}\left[1 - \frac{1}{(1+i)^n}\right] = \frac{(1+i)^n - 1}{i(1+i)^n} \tag{6.1}$$

and the capital recovery factor crf is the reciprocal thereof, or

$$crf = \frac{1}{pwf} = \frac{i}{1 - \dfrac{1}{(1+i)^n}} = \frac{i(1+i)^n}{(1+i)^n - 1} \tag{6.2}$$

in which i is the interest rate per time period and n is the number of recurring payments or time periods. Usually the series payment is on an annual basis, i.e., once per year, and then n is the life of the project in years.

To illustrate the use of these factors, assume it will cost 1 million dollars to build a water distribution system and prepare it to begin operation. Thus a loan is taken (or a bond is issued) for $1,000,000 at a particular interest rate, and this loan is to be repaid by constant annual payments over the life of the project $n = 15$ years. Table 6.1 reports payment data for interest rates $i = 0.06$ and $i = 0.10$ for $n = 15$. The columns headed

"Payment" are obtained by multiplying the capital recovery factor by $1,000,000, i.e., for $i = 0.06$ the capital recovery factor is $crf = 0.06/[1 - 1/(1.06^{15})] = 0.102963$, and for $i = 0.10$ the factor is $crf = 0.1/[1 - 1/(1.10^{15})] = 0.131474$. The columns headed "Accum." accumulate or sum these annual payments. The column headed "Interest" is the amount of interest accrued during that year, found by multiplying the entry in the row above (in the next column) by the interest rate, and the column headed "Owing" gives the amount of the loan still outstanding. The values in this last column are obtained by subtracting the payment from, and adding the interest to, the previous entry, i.e., $957,037 = 1,000,000 - 102,963 + 60,000. In both halves of this table the payment at the end of the fifteenth year exactly equals the amount still owed plus the interest on that amount over the last year (within roundoff error), or $102,296 = 97,135 + 5828, and $131,474 = 119,521 + 11,952. Thus for an interest rate of 6% a constant annual payment of $102,963 is equivalent to a present worth of $1,000,000, and for an interest rate of 10% $131,474 paid at the end of each year for 15 years is equivalent to $1,000,000. In other words, multiplying the pwf by this annual payment will reproduce the principal amount, in this case $1,000,000.

Table 6.1
A $1,000,000 loan

	Interest Rate = 6%				Interest Rate = 10 %			
Yr	Payment	Accum.	Interest	Owing	Payment	Accum.	Interest	Owing
				1000000				1000000
1	102963	102962	60000	957037	131474	131473	100000	968526
2	102963	205925	57422	911497	131474	262947	96852	933905
3	102963	308888	54689	863224	131474	394421	93390	895822
4	102963	411851	51793	812054	131474	525895	89582	853930
5	102963	514813	48723	757815	131474	657368	85393	807849
6	102963	617776	45468	700321	131474	788842	80784	757161
7	102963	720739	42019	639378	131474	920316	75716	701403
8	102963	823702	38362	574777	131474	1051790	70140	640069
9	102963	926664	34486	506301	131474	1183264	64006	572603
10	102963	1029627	30378	433717	131474	1314737	57260	498389
11	102963	1132590	26022	356777	131474	1446211	49838	416754
12	102963	1235553	21406	275221	131474	1577685	41675	326956
13	102963	1338515	16513	188771	131474	1709159	32695	228178
14	102963	1441478	11326	97135	131474	1840632	22817	119521
15	102963	1544441	5828	0	131474	1972106	11952	0

To carry this illustration further, assume that a solution to the network problem indicates that the annual cost of electrical energy to operate the pumps is $120,000, and the maintenance department will require an average of $50,000 per year to operate and repair the system. The total annual costs are then obtained as 120,000 + 50,000 = $170,000. The two alternative approaches for comparing the capital investment and operating costs are the following: 1. Add pwf x (annual cost) to the capital investment; or 2. Add crf x (capital investment) to the annual cost. These two alternatives are listed in Table 6.2. As the interest rate increases, we see in the table that the total cost decreases if the present worth basis is used, but when a series payment is used to obtain the total cost we find that the total cost increases with interest rate. These differences occur because the pwf decreases with interest rate, but the crf increases with interest rate.

Table 6.2 Total Costs

As Present Worth		As a Series Payment	
For $i = 0.06$		For $i = 0.06$	
1. Capital investment	= 1,000,000	1. Cap. invest., crfx$1,000,000	= 102,963
2. Operating, pwfx$170,000	= 1,651,082	2. Operating	= 170,000
Total	$2,651,082		$272,963
For $i = 0.10$		For $i = 0.10$	
1. Capital investment	1,000,000	1. Cap. invest., crfx$1,000,000	= 131,474
2. Operating, pwfx$170,000	= 1,293,034	2. Operating	170,000
Total	$2,293,034		$301,474

Example Problem 6.2

Compare the cost of using a 6 in, 8 in, or 10 in pipe line (wall roughness e = 0.005 in) to pump 1.5 ft^3/s of water ($\nu = 1.217 \times 10^{-5}$ ft^2/s) for 200 days per year from a groundwater elevation of 1500 ft to an elevation of 1550 ft with a delivery pressure of 50 lb/in^2. The total pipe length is 4000 ft. Energy costs $0.11/kWh; the capital investment cost of the well and pumps is $50,000; the combined efficiency of the pump-motor is $\varepsilon = 0.70$, and the pipe costs are $30/ft for 6-in-pipe, $45/ft for 8-in-pipe, and $55/ft for 10-in-pipe. The interest rate is $i = 0.10$, and the project life is 30 years.

The solution is summarized in two tables. We begin the solution by computing the capital recovery and present worth factors as crf = $0.1/[1 - 1/1.1^{30}] = 0.10608$ and pwf = $1/0.10608 = 9.4268$. The pump must supply a head that is the sum of the frictional head loss, the elevation difference, and the delivery pressure head, or $h_p = h_f + 50 + 50(144)/62.4 = 165.38 + h_f$ (ft). The power is $P = Q\gamma h_p(0.746)/(550\varepsilon) = 0.18137 h_p$ kW. The Darcy-Weisbach and Colebrook-White equations can be solved for the frictional head loss that would occur in each of the three pipes. These head losses are listed in column (2) in the tables. Columns (3) and (4) in these tables list the computed pump heads that must be supplied and the power requirement of the pumps in kW. The annual operating cost, in $/yr in column (5), is found by multiplying this power requirement by 24 hr/day times 200 days times the $0.11/kWh unit cost of energy. In the first table the total cost is stated in terms of annual amounts; hence column (6), which contains the entire capital investment cost (the cost of the pipe plus $50,000 for the pumps and well), is multiplied by the crf to obtain an equivalent annual cost which is in column (7), and this amount is added to the annual energy cost to find the total annual cost in column (8). We note that the use of 8-in pipe leads to an annual cost of $ 43,411 over the 30-year life of the project, which is the lowest cost of the three alternatives. It is the higher energy costs that cause the total cost with 6-in pipe to be larger, and the higher cost with 10-in pipe is caused by the expense of the pipe itself. A present worth computation is presented in the second table. The total cost is the present worth; here the cost of energy is converted to a present value by use of the pwf before it is added to the capital cost.

Cost on an annual payment basis

Pipe in (1)	h_f ft (2)	h_p ft (3)	P kW (4)	\$/yr (5)	Cap. Cost \$ (6)	x crf \$ (7)	Cost \$ (8)
6	144.46	309.90	56.194	29,670	170,000	18,023	47,703
8	33.17	198.55	36.009	19,013	230,000	24,398	43,411
10	10.72	176.10	31.938	16,863	270,000	28,641	45,504

Cost on a present worth basis

Pipe in (1)	h_f ft (2)	h_p ft (3)	P kW (4)	\$/yr (5)	x pwf \$ (6)	Cap. Cost \$ (7)	Cost \$ (8)
6	144.46	309.89	56.194	29,670	279,695	170,000	449,695
8	33.17	198.55	36.009	19,013	179,233	230,000	409,233
10	10.72	176.10	31.938	16,863	158,965	270,000	428,965

* * *

6.3.1. ECONOMICS APPLIED TO WATER SYSTEMS

To apply economics to the evaluation of piping systems, it is productive to classify each system according to whether (1) the demands are constant or variable, (2) it is gravity feed or pump feed, and (3) branched or looped. When the demands are constant, the discharge in each pipe is fixed, and an optimal design can be achieved by minimizing the total cost over the life of the system. When the demands vary, the changes usually have one or more random components, so in real life a repeating pattern of change with time does not occur, and the identification of an optimal design becomes a matter of interpretation. Gravity-feed systems have no energy costs that can change with the head losses and the desired pressures; if the demands are also fixed, then the optimal design process will simply select the smallest possible set of pipes that will deliver the required pressures and discharges. If the system is branched, then the flow in each pipe is fixed regardless of its diameter, while in looped systems the discharges are dependent upon all of the pipe diameters as well as the demands.

Some irrigation systems fall into the classifications that are simplest to analyze, e.g., branched systems with constant demands. Municipal water distribution systems generally are among the most complex types. They are looped for redundancy to allow individual components to be taken out of operation; they must be able to respond to emergency flows, they have varying demands, and at least some of the water is pumped. Even when the supply enters the network from a reservoir (storage tank), this water often has been pumped into that tank at a previous time. For these complex systems the completion of a formal optimal design cannot be achieved by an application of mathematics and the use of a computer program. Sound engineering judgment based on experience and a thorough understanding of the system's vital components is needed to achieve even a "good," let alone a "near optimal," design. The issues that contribute to the complexity of such systems include the following:

1. Reliability considerations: standby power, manual versus automatic control, types of storage, and the extent and type of monitoring of operations.
2. Demand: spatial and temporal variations in use, types of users, costs for failure to deliver, fire flow requirements, future trends.
3. Storage requirements: groundwater storage with pumping versus elevated or ground-level tanks, tank volumes for reserves and/or peaking, location and variations of water levels.

4. Maximum and minimum pressure requirements: residential areas, high value business districts, industrial districts, future areas that will be served.
5. Population distribution and future trends.
6. Topographic changes: pressure controls (PRV's, BPV's, valves) versus separate systems.
7. Separate systems for irrigation and/or fire fighting and other uses versus a single system etc.

There is no attempt to handle all these issues in this Chapter. Optimization techniques won't even be used. But basic cost considerations will be applied to insure that a system is cost effective.

6.3.2. LEAST COST

In each design we want to select the pipe size which will produce the lowest overall cost, considering both capital recovery and annual operating costs. Of course these two costs must be put on a common basis before they are summed. One way to do this is to solve the problem for different pipe sizes, compute the cost for each and plot the costs on a graph as a function of pipe size. If the water is pumped, a graph of these costs might look like Fig. 6.1, in which the annual capital recovery cost for the pipe and other facilities will increase with the pipe diameter, but the costs associated with pumping will decrease with pipe size. These two opposite trends cause the total cost to decrease with pipe diameter to a minimum and then increase. The nearest standard pipe diameter to this minimum is the diameter to select.

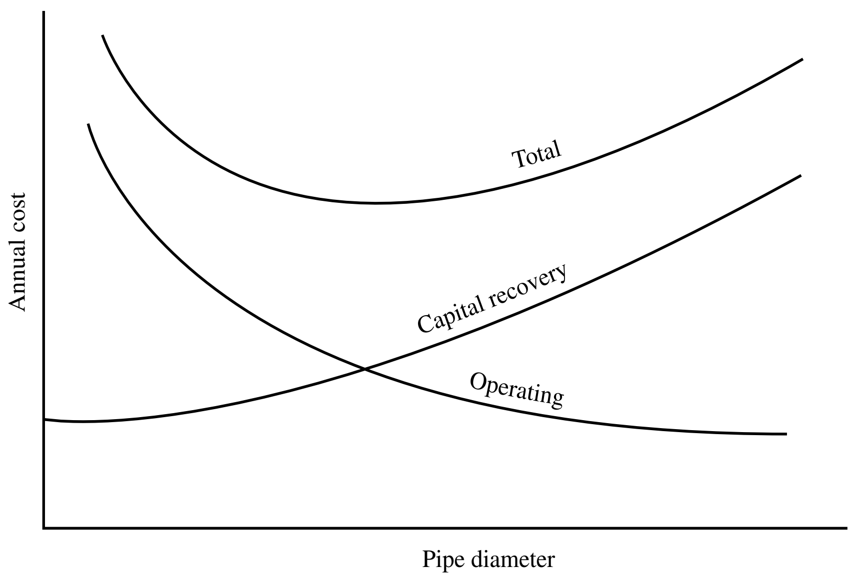


Figure 6.1 System cost as a function of pipe diameter.

For a network of pipes the discharges will usually differ in each pipe, and it would be nonsense to design a network with all pipes of the same diameter, compute the cost and make a graph like Fig. 6.1 to select a pipe size that produces the least total cost. An alternative that does make sense, however, is to replace the pipe diameter, which is the independent variable, by the slope of the HGL. Since the slope of the HGL is the head loss divided by the pipe length over which this loss occurs, it also makes sense to rescale the ordinate to be the cost per unit length of pipe. Solving the Darcy-Weisbach equation for the diameter gives

$$D = \{ fQ^2/(2gS(\pi/4)^2 \}^{0.2} \qquad (6.3)$$

in which S is the slope of the HGL, and f is given by the Colebrook-White equation. The cost is then computed in the following way: (1) for a given S, solve the Darcy-Weisbach and Colebrook-White equations for the pipe diameter; (2) with this pipe diameter determine the cost of pipe per unit length C_p from a table of costs for different pipe diameters (this cost might be obtained by interpolation or by using the standard diameter that is closest to the computed value); (3) compute the energy cost for pumping, per unit length of pipe, from

$$C_e = \gamma QRt(S + \Delta H/L)/(eC) \qquad (6.4)$$

in which R is the unit cost of energy ($/kWh), ΔH is difference in total head (elevation plus pressure) between the ends of the pipe, t is the time that the pump operates, e is the combined motor-pump efficiency, and C is a unit constant. It is important for C and the energy costs to be on the same time basis. For example, if t is in days, then we have $C = (550/0.746)/24 = 30.72$ for ES units, and $C = 1000/24 = 41.7$ for SI units.

Example Problem 6.3

Prepare a graph of total cost as a function of the slope of the HGL for several different discharges, and then determine the least cost pipe diameter to convey this water in 2000 ft of horizontal pipe with a delivery pressure of 40 lb/in^2. The water is pumped from a reservoir with a water surface elevation that is one foot below the pipe elevation. A table provides the cost per unit length for installing different pipe sizes. Other economic data are as follows: energy costs $0.10/kWh; project life = 30 years; the operating period is 365 days per year; the pipe wall roughness is $e = 0.005$ ft.

Dia., in.	4	6	8	10	12	15	18
$/ft	3.67	5.33	7.67	10.67	16.67	24.00	43.33

Dia., in.	20	24	30	36	42	48	54
$/ft	56.67	80.00	100.00	120.00	145.00	170.00	200.00

We begin the solution by computing the difference in total head between the reservoir water surface and the end of the pipe as $\Delta H = 40(144)/62.4 + 1.0 = 93.3$ ft; upon dividing this result by the pipe length, we obtain $\Delta H/L = 0.0466$. The program MCOST will generate the cost data as a function of S. The input consists of the following: RATE = energy cost per kWh, LIFE = project life in years, Q = discharge, DZ = $\Delta H/L$, DAYS = number of days per year of system operation, G = acceleration of gravity, E = pipe wall roughness (inches for ES units, m for SI units), EFF = combined pump-motor efficiency, N = the number of pairs of (pipe diameter, cost per unit length) to follow. The next input line lists these data pairs; each pipe diameter is in inches for ES units or in m for SI units; the cost is the installation cost per unit length for that diameter. The `DO 80` loop within the program repeats the computations for the 13 values of S in the data statement. Within this loop the Darcy-Weisbach and Colebrook-White equations are solved simultaneously for D and SF = $1/f^{1/2}$. After the diameter is obtained, a second-order polynomial interpolation, using Lagrange's formula, obtains the capital cost per unit length associated with this diameter from the table of data pairs (D, $/L). This value is multiplied by the crf, the energy cost of pumping is computed, and these costs, and their sum, are printed to an output file.

```
C.     PROGRAM MCOST.FOR
       REAL D(20),CP(20),S(13)
       DATA S/.0001,.00025,.0005,.001,.002,.003,
      &.004,.005,.006,.007,.008,.009,.01/
       READ(2,*) RATE,LIFE,Q,DZ,DAYS,G,E,EFF,N
       READ(2,*)(D(I),CP(I),I=1,N)
       IF(G.GT.20.) THEN
       POF=2.031290182
       VIS=1.217E-5
       DO 20 I=1,N
    20 D(I)=D(I)/12.
       E=E/12.
       ELSE
       POF=235.344
       VIS=1.31E-6
       ENDIF
       CRF=RATE*(1.+ RATE)**LIFE/((1.+ RATE)**LIFE - 1.)
       SF=8.
       G2=1.23370055*G
       DD=.8
       K2=2
       DO 80 I=1,13
    30 DD1=DD
    40 SF1=SF
       SF=1.14-2.*ALOG10(E/DD1+7.343472826*VIS*
      &DD1*SF/Q)
       IF(ABS(SF-SF1).GT. 1.E-6) GO TO 40
       DD=((Q/SF)**2/(S(I)*G2))**.2
       IF(ABS(DD1-DD) .GT. 1.E-5) GO TO 30
    50 IF(DD.LT.D(K2+1).OR. K2.GT.N-2) GO TO 60
       K2=K2+1
       GO TO 50
    60 IF(DD.GE.D(K2) .OR. K2.EQ.2) GO TO 70
       K2=K2-1
       GO TO 60
    70 K1=K2-1
       K3=K2+1
       C1=CP(K1)/((D(K1)-D(K2))*(D(K1)-D(K3)))
       C2=CP(K2)/((D(K2)-D(K1))*(D(K2)-D(K3)))
       C3=CP(K3)/((D(K3)-D(K1))*(D(K3)-D(K2)))
       AC=C1+C2+C3
       BC=-C1*(D(K2)+D(K3))-C2*(D(K1)+D(K3))-C3*(D(K1)+D(K2))
       CC=C1*D(K2)*D(K3)+C2*D(K1)*D(K3)+C3*D(K1)*D(K2)
       COST=(AC*DD+BC)*DD+CC
       CPIP=CRF*COST
       CENE=POF*DAYS*(S(I)+DZ)*Q*RATE/EFF
    80 WRITE(3,100) S(I),DD,COST,CPIP,CENE,CPIP+CENE
   100 FORMAT(F10.7,5F10.4)
       END
```

An example of the input data file follows:

.1 30 1 .0466 365 32.2 .005 .7 14
4 3.67 6 5.33 8 7.67 10 10.67 12 16.67 15 24 18 43.33 20 56.67 24 80 30 100
36 120 42 145 48 170 54 200

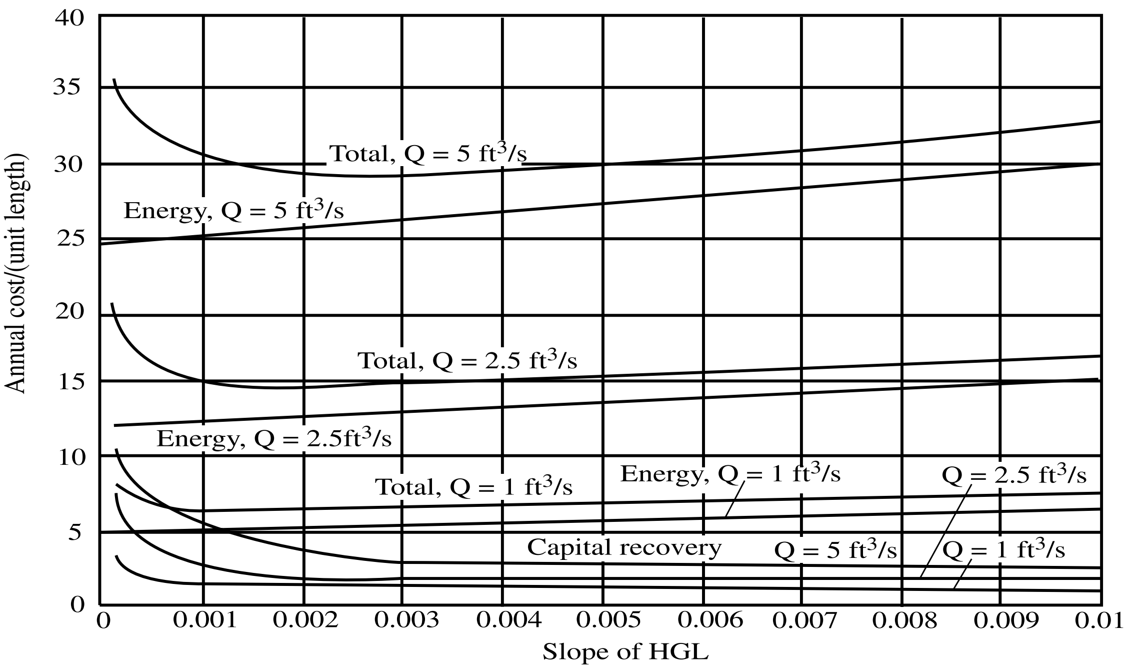
For $Q = 1.0$ ft^3/s the following output table is created:

S	***D*, ft**	C_p	$C_p$xcrf	C_e	Total Cost	
0.0001	1.3869	33.0954	3.5107	4.9463	8.4571	
0.0002	1.1506	21.3257	2.2622	4.9622	7.2244	
0.0005	0.9999	16.6665	1.7680	4.9887	6.7567	
0.0010	0.8697	11.7235	1.2436	5.0417	6.2853	
0.0020	0.7571	9.2161	0.9776	5.1476	6.1252	
0.0030	0.6984	8.1907	0.8689	5.2535	6.1224	*Min. Cost
0.0040	0.6597	7.5582	0.8018	5.3594	6.1612	
0.0050	0.6312	7.1147	0.7547	5.4653	6.2201	
0.0060	0.6088	6.7811	0.7193	5.5712	6.2906	
0.0070	0.5906	6.5178	0.6914	5.6772	6.3686	
0.0080	0.5753	6.3026	0.6686	5.7831	6.4517	
0.0090	0.5621	6.1223	0.6494	5.8890	6.5384	
0.0100	0.5506	5.9681	0.6331	5.9949	6.6280	

Results from these cost calculations for the discharges Q = 1.0 ft^3/s, 2.5 ft^3/s and 5.0 ft^3/s are plotted in the graph on the next page. The HGL slope that led to the lowest total cost, the costs themselves, and the associated pipe diameters are given in the next table:

Minimum Total Cost

Q, ft^3/s	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0
Cost	6.12	8.97	11.88	14.84	17.69	20.47	23.31	29.02
S	0.003	0.003	0.003	0.003	0.002	0.002	0.002	0.003
D, ft	0.698	0.814	0.908	0.988	1.147	1.216	1.279	1.285
D, in	8.38	9.77	10.89	11.85	13.76	14.59	15.35	15.42



* * *

As the discharge increases, the pipe size with the least total cost increases, as one would expect. However, the slope S is nearly constant; for discharges of 3 to 4 ft^3/s the slope $S = 0.002$ led to the smallest total cost, but for the other discharges $S = 0.003$ gave the smallest total cost.

Example Problem 6.4

Augment the program MCOST so it (1) determines the slope S of the HGL that results in minimum total cost, and (2) writes this total cost per unit pipe length and the corresponding pipe diameter to an output file.

In this project there a several approaches that could be taken. As it is, MCOST generates a table that lists in the last column the total cost per unit length as a function of the HGL-slope S (listed in column 1). As this table is being generated, we monitor the last column to see if the total cost has increased from the previous line. If so, we then pass a second-order polynomial through the last three data pairs of $(S, \$)$ with the Lagrangian interpolation formula. The polynomial is of the form $\$ = aS^2 + bS + c$. The minimum cost can be found by taking the derivative of this equation with respect to S, setting it to zero, and solving for S as $S_{min} = -b/(2a)$. Upon substituting S_{min} into the polynomial equation, we find $\$_{min}$. In a similar way the corresponding pipe diameter D_{min} can be obtained with a similar polynomial $D = a_1 S^2 + b_1 S + c_1$, with S_{min} substituted for S. This procedure is implemented by replacing `80 WRITE ...` with the statements given below. The arrays CT(3), SS(3), DS(3) have also been added to the REAL declaration, and I1 = 1 and CT(1) = 1.E20 initialize these variables.

```
      IF(CTO.LT.CT(I1).
     &OR.I.LT.3) THEN
      IF(START) THEN
      CT(1)=CTO
      SS(1)=S(I)
      DS(1)=DD
      START=.FALSE.
      ELSE
      IF(NFIRST) THEN
      CT(1)=CT(2)
      SS(1)=SS(2)
      DS(1)=DS(2)
      ENDIF
      CT(2)=CTO
      SS(2)=S(I)
      DS(2)=DD
      I1=2
      NFIRST=.TRUE.
      ENDIF
      ELSE
      CT(3)=CTO
      SS(3)=S(I)
      DS(3)=DD
      AA=0.
      DA=0.
      BB=0.
      DB=0.
      CC=0.
      DC=0.
      DO 76 J=1,3
      CR=CT(J)
      DR=DS(J)
      DO 74 K=1,3
      IF(K.EQ.J) GO TO 74
      CR=CR/(SS(J)-SS(K))
      DR=DR/(SS(J)-SS(K))
   74 CONTINUE
      AA=AA+CR
      DA=DA+DR
      SUM=0.
      PRO=1.
      DO 75 K=1,3
      IF(K.EQ.J) GO TO 75
      SUM=SUM+SS(K)
      PRO=PRO*SS(K)
   75 CONTINUE
      BB=BB-CR*SUM
      DB=DB-DR*SUM
      DC=DC+DR*PRO
   76 CC=CC+CR*PRO
      SHMIN=-0.5*BB/AA
      CTMIN=(AA*SHMIN+BB)*SHMIN+CC
      DMIN=(DA*SHMIN+DB)*SHMIN+DC
      GO TO 90
      ENDIF
   80 CONTINUE
   90 WRITE(6,110) Q,SHMIN,
     &CTMIN,DMIN
  110 FORMAT(' Q =',F8.2,', HGL-S
     &=',F9.6,', $/L =',F8.2,
     &', Dmin =',F8.3)
      END
```

*   *   *

6.4 ECONOMIC NETWORK DESIGN

6.4.1. ONE PRINCIPAL SUPPLY SOURCE

The method in Section 6.3 can be used to select a single pipe that will yield the least cost. If a network of pipes exists, then the method must be modified. In a real water distribution system the capital costs associated with installing pipes might vary widely from location to location for the same pipe size. In a new subdivision the only costs in addition to the purchase of the pipe may be the costs associated with the operation of a trenching machine or backhoe. In a highly developed business district there will be costs associated with replacement of roads, replacement and/or relocation of other utilities, rerouting traffic, acquisition of right-of-way, etc., and these costs may be enormous in comparison with the pipe costs, so there is then no significant difference in cost with pipe size. Annual costs might also vary considerably, depending upon whether the pipe is fed by gravity and whether the water comes directly from a pump or from a storage facility that receives its water via pumps. For the latter there is the capital recovery cost associated with the storage facility in addition to the energy cost of pumping. Obviously the variability of costs is dependent upon the city and/or location of the water system, and to describe procedures to follow in the design of a "near least-cost" system we will use only the simplest case, in which no variability of either capital recovery or operating costs occurs. The same principles apply, however, for the more complex real water system, but they must be applied on an individual basis.

Let us assume that the layout, or a proposed layout, of the system is given. It includes the supply sources as well as the lengths and locations of pipes, some of which may later be eliminated. For design purposes the demands are known and fixed. Such design demands may be the maximum demands that are expected, without emergency flows for fire fighting etc. From Example Problem 6.3 we have seen that the slope of the HGL that is related to the least total cost does not vary greatly with the discharge. Considering the uncertainty of costs over the life of the network and the variability of discharges, one HGL slope for all pipes in the network might be used, rather than finding the least cost HGL slope for each pipe. A practical approach to the design of an economical pipe network might in general follow these eight steps:

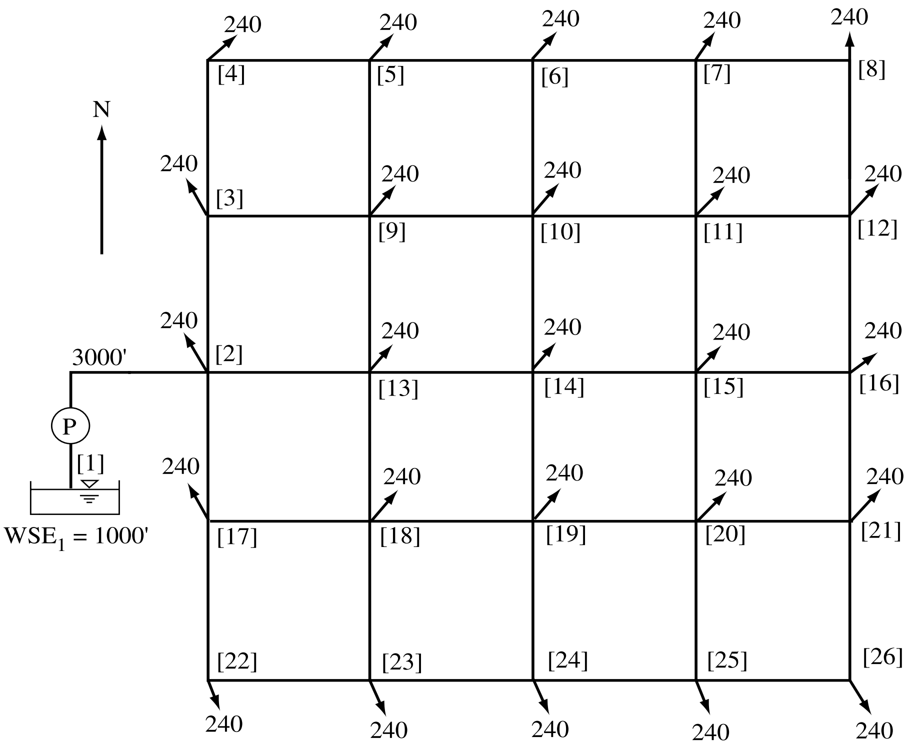
1. Remove pipes that will carry the smaller discharges so a branched network is created.
2. Determine the demands for which the network is to be designed. These design demands normally are the peak hourly demands, but they do not include fire, or other emergency, flows.
3. Use the demands from step two to determine the discharge in every pipe.
4. Compute the pipe diameters, based on a selected HGL slope that is consistent with the satisfaction of the desired operating pressures for the system. After finding the pipe diameters, replace them with the nearest (or with the next larger, depending on one's judgment) standard pipe size and obtain by analysis a solution for this system.
5. Compute the costs associated with the pipe sizes that were found in step four, as well as the pumping cost and other operating costs.
6. Repeat steps four and five with a set of different HGL slopes, until the minimum cost can be identified.
7. Each pipe that was removed in step one can now be re-installed as a pipe having a minimum diameter or, as judgment dictates, a larger size, especially if the pipe is located so its flow will be important for fire fighting, or if it will be needed when another key component of the system is out of operation. We now analyze this piping system for several demand levels and/or fire flow requirements and attempt to identify any deficiencies in the network's performance. If deficiencies are found, we must exercise judgment in deciding how they might best be corrected.
8. Based on a knowledge of network performance that was obtained in step seven, formulate logical schedules and/or rules for the operation of the network. Select the

elevation, sizes and heights of storage tanks that will produce a "good" daily operation that will fully meet the anticipated varying demands, maintain pressures above the minimum and not create any excessive pressures. Test these choices by obtaining an extended time simulation of the proposed network and its operation. From an examination of results from this time-dependent solution, make appropriate changes to the network and/or its operation, and repeat the extended time simulations until the designer concludes that no further significant cost reductions can be made, especially in view of the uncertainty of the demand data and some of the other data that are the foundation of the computation.

Example Problem 6.5

A water distribution system is to be designed to serve a new community development. The land is level where the development is located and has an elevation of 1100 ft. As a consulting hydraulic engineer you are responsible for the design of the water system. The following requirements exist:

1. The entire water supply must come from a well that has recently been drilled. All indications, including pumping tests, suggest the well will provide enough water for the community for the coming 20 years.
2. The skeletonized pipe layout for design should coincide with the proposed network of primary streets with locations on a square grid that is one-quarter mile on each side, so all pipes, with the exception of the supply line from the well, are 1320 ft long. (Secondary streets and individual service connections to buildings will exist in the interior of indi-vidual blocks, but they are omitted in the skeletonization.) All pipes have e = 0.005 in.



All nodal demands in gal/min

3. A survey of future water consumption indicates that the peak hourly demand will be 240 gal/min at each node at the street intersections.
4. For this region of the country it is anticipated that the peak hourly demand is 2.3 times the average daily demand.
5. For a town of this size and the anticipated industrialization, the National Bureau of Fire Underwriters recommends that the system be able to supply an emergency fire flow of 2000 gal/min at any node at a pressure of at least 20 lb/in^2.

In estimating costs for the project, the following information is provided to you by the firm's economist: interest rate = 9 percent; project life = 20 years; electricity costs \$0.090/kWh; overall cost for the well, buying the pump and installing it is \$180,000; the cost of construction C for a water storage tank is $C = \$15000V^{0.5}$, in which V is the tank capacity in thousands of cubic feet; the cost of the pipe is divided into (a) the purchase price of the pipe and (b) the costs associated with its installation. These costs are given in the following table:

Pipe Costs per Foot of Length

Diameter of Pipe, in	6	8	10	12	15	18	24	36
Purchase price, \$	4.00	6.00	12.00	18.00	25.00	45.00	60.00	100.00
Installation, \$	8.00	8.20	8.40	8.60	9.00	9.20	9.40	9.60

As the hydraulic engineer that is responsible for the design of the water system, you are to (1) specify the size of each pipe in the system, (2) specify the pump(s) to be installed (this includes the characteristics, i.e., the discharge and head that the pump should produce), (3) specify where storage tanks should be located, and give their elevations and capacities, (4) provide an engineering economic analysis of the proposed water system, and suggest what price should be charged for the water to recover the costs associated with the construction and maintenance of the system.

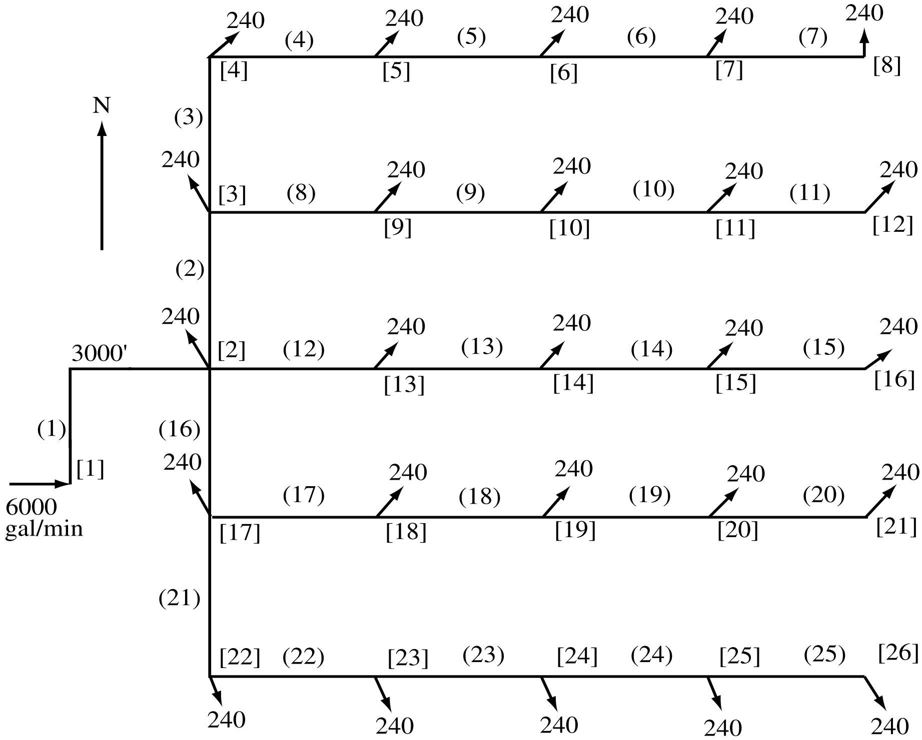
The solution will proceed according to the steps outlined above.

Step 1. To create a branched system from the proposed layout, the pipes along north-south streets have been removed, except the one that is nearest to the pump, to create the network shown atop the next page. There are many alternative branched networks that could be used. This one assumes that the north-south pipes closest to the pump will be the main transmission lines. An alternative which would assume that the primary north-south transmission line is nearer the center of the system would remove pipes 2, 3, 16, and 21, and put in their place pipes connecting node pairs 14 and 10, 10 and 6, 14 and 19, and 19 and 24.

Step 2. The problem statement provides the demands to be used for the design, i.e., 240 gal/min = 0.535 ft^3/s at each node of the network. In practice, obtaining these design demands requires studies of current water demands in the area and projections of future trends over the life of the network, all tempered with a sound interpretation of these data and judgment.

Steps 3, 4, 5, and 6. These steps will be accomplished together. In fact, these steps will be completed via two alternative pathways. The first alternative will determine the most economical HGL-slope for each pipe by using MCOST1, the modified computer program from Example Problem 6.4. The second alternative is to use special input allowed by NETWK to define a branched network.

In the first alternative the program SOLBRAN, which was described in Chapter 5, will be modified by replacing subroutine DIAPIP, which finds f and D when the head loss for the pipe is given, by MCOST1 as a subroutine that determines the least cost HGL-slope, diameter, and cost per unit length of pipe, based on economic data on pipe cost and the energy cost of pumping. The program is called MCBRAN. The first portion of MCBRAN, almost to the `DO 90` loop, is the same as SOLBRAN with a COMMON



All nodal demands in gal/min

/COSTB/ statement and arrays H (nodal heads), QJJ (nodal demands), NOP (pipe numbers), L1 (upstream pipe node) and L2 (downstream pipe node), added. Aftert the DO 85 loop an added READ statement enters, as triples, the pipe number NOP and its upstream and downstream nodes, L1 and L2. The sequence of pipes in this input establishes the order in which the least cost pipes are to be found with subroutine MCOSTS. This sequence must start at the end of one branch and proceed to the point where another branch departs from a node and then downstream along the next branch, so an HGL-elevation is known at one end of each new pipe that is processed. In this example the order that is selected is 7, 6, 5, 4, 3, 8, and so on. The upstream and downstream node numbers are used to compute the elevations of the HGL at each node.

The next read statement contains (1) the node number for which a beginning HGL-elevation will be given, e.g. the downstream node of the first pipe in the previous list, (2) this HGL-elevation, and (3) the elevation slopes DZ that MCOSTS should use. The information in group (3) contains first the number of pairs that will be given, and then as pairs the pipe number and the DZ that should be used until the next pipe in the list is given. Since the slope of the HGL is the sum of the frictional slope and DZ for our example problem, we note the elevation difference of 100 ft between node 8 and the source and the length of the pipes between these two points is 6x1320 + 3000 = 10920 ft; hence DZ = 100/10920 = 0.0091575. This elevation gradient is constant, so the input consists of only 3 values: 1 7 0.0091575. Since this DZ will add a constant to the energy cost for all pipes, it will not change the least cost slope of the HGL.

The last input set contains the economic data. The first line consists of the following: RATE = cost of electricity in $/kWh; LIFE = life expectancy of the project in years; DAYS = the number of days per year that the pump will operate; EFF = the combined pump-motor efficiency; N = the number of data pairs to follow. This last data, consisting of pairs of pipe diameter and its cost per unit length, is read in subroutine MCOSTS when it is first called. The input for our network problem follows the program listings.

In determining the least-cost HGL-slope for each pipe in a network, the energy lost through the demands at the nodes must be included in the overall energy cost. To account for this energy, the program must determine the head at each node. The power loss is the product of the demand QJ, the specific weight γ and the difference between the nodal head H and the nodal elevation. Since the head at the upstream pipe node equals the head at the downstream node plus the product of HGL-slope and pipe length, the program passes the demand QJN and the difference between the known head and the nodal elevation, divided by the length of pipe, to subroutine MCOSTS. A positive slope S(I) is used, independent of direction, since the pressure at the end will be computed by the process when we move in the downstream direction, rather than being given. Program MCBRAN follows:

```
*************************************************************************
*   PROGRAM NO. 6.1, MCBRAN, FORTRAN
*   THIS PROGRAM HAS BEEN INCLUDED FOR THE CONVENIENCE OF THE READER.
*   THE AUTHOR ACCEPTS NO RESPONSIBILITY FOR ITS CORRECTNESS.
*   USERS OF THIS PROGRAM DO SO AT THEIR OWN RISK.
*************************************************************************
*   FINDS THE LEAST-COST HGL-SLOPE FOR ALL PIPES IN A BRANCHED NETWORK
*
      PARAMETER (N4=4)
      REAL L[ALLOCATABLE](:),E[ALLOCATABLE](:),Q[ALLOCATABLE](:),
     &QJ[ALLOCATABLE](:),H[ALLOCATABLE](:),ELEV[ALLOCATABLE](:),
     &QJJ[ALLOCATABLE](:),DHS(10)
      INTEGER*2 IPP(11),NUM[ALLOCATABLE](:),JN[ALLOCATABLE](:,:),
     &NOP[ALLOCATABLE](:),L1[ALLOCATABLE](:),L2[ALLOCATABLE](:)
      LOGICAL*2 LNODE[ALLOCATABLE](:)
      CHARACTER*38  F110
     &/'(I5,F8.1,F8.6,F10.3,F8.3,3F10.3,F10.6)'/
      COMMON CI4,G2,DI4,S,SF,CFDIA
      COMMON /COSTB/CRF,DZ,G,VISC,FK2,POF,N
      WRITE(*,*)' GIVE (1) NO. OF PIPES, (2) INPUT UNIT,'
     &,' (3) OUTPUT UNIT '
      READ(*,*) NP,INPUT,IOUT
      NJ=NP+1
      CONV=12.
      ALLOCATE (L(NP),E(NP),Q(NP),QJ(NJ),NUM(NJ),JN(NJ,N4),
     &LNODE(NJ),H(NJ),ELEV(NJ),QJJ(NJ),NOP(NP),L1(NP),L2(NP))
      READ(INPUT,*) G,VISC
      FK2=1.3757
      CFDIA=(5.01137E-4/0.003)**0.2053
      IF(G.LT.30.) THEN
      CONV=1.
      FK2=1.62613
      CFDIA=(1.13437E-3/0.003)**.2053
      ENDIF
      G2=2.*G
      SF=8.
      CI4=7.3434712828*VISC
      DI4=0.785398163
      DO 8 I=1,NP
    8 L(I)=0.
      READ(INPUT,*) L
      DO 9 I=1,NP
      IF(L(I) .LT. 0.001) L(I)=L(I-1)
    9 CONTINUE
      DO 10 I=1,NP
```

```
10 E(I)=0.
   READ(INPUT,*) E
   DO 20 I=1,NP
   IF(E(I).GT.0.) THEN
   IF(E(I).LT.10.) E(I)=E(I)/CONV
   ELSE
   E(I)=E(I-1)
   ENDIF
20 CONTINUE
   DO 22 I=1,NJ
22 ELEV(I)=-10001.
   READ(INPUT,*) ELEV
   DO 25 I=2,NJ
   IF(ELEV(I).LT.-10000.) ELEV(I)=ELEV(I-1)
25 CONTINUE
   SUM=0.
   DO 40 I=1,NJ
   DO 30 J=1,N4
30 JN(I,J)=0
   READ(INPUT,*) QJ(I),(JN(I,J),J=1,N4)
   SUM=SUM+QJ(I)
   DO 35 J=N4,1,-1
   IF(JN(I,J).EQ.0) GO TO 35
   NUM(I)=J
   IF(J.EQ.1) THEN
   IF(FLOAT(JN(I,1))*QJ(I).LE. 0.) GO TO 40
   WRITE(*,*)' DIRECTION OF FLOW IN PIPE',JN(I,1),
  &' IS NOT CONSISTENT WITH DEMAND NODE',I,QJ(I)
   GO TO 99
   ENDIF
   GO TO 40
35 CONTINUE
40 LNODE(I)=.FALSE.
   DO 41 I=1,NJ
41 QJJ(I)=QJ(I)
   IF(ABS(SUM).GT. 1.E-4) THEN
   WRITE(*,*)' DEMANDS DO NOT SUM TO ZERO, SUM =', SUM
   GO TO 99
   ENDIF
   NNJ=NJ
45 DO 65 I=1,NNJ
   IF(NUM(I).GT.1) GO TO 65
   LNODE(I)=.TRUE.
   IJ1=IABS(JN(I,1))
   Q(IJ1)= - FLOAT(JN(I,1)/IJ1)*QJ(I)
   DO 60 J=1,NNJ
   IF(J.EQ.I .OR. NUM(J).LT.2) GO TO 60
   K=1
48 IF(IABS(JN(J,K)).EQ.IJ1) THEN
   QJ(J)=QJ(J)+Q(IJ1)*FLOAT(JN(J,K)/IJ1)
   DO 50 KK=K+1,NUM(J)
50 JN(J,KK-1)=JN(J,KK)
   NUM(J)=NUM(J)-1
   GO TO 60
   ENDIF
   K=K+1
   IF(K.LE.NUM(J)) GO TO 48
60 CONTINUE
65 CONTINUE
```

```
      JJ=0
      DO 80 I=1,NNJ
      IF(LNODE(I)) GO TO 80
      JJ=JJ+1
      QJ(JJ)=QJ(I)
      LNODE(JJ)=.FALSE.
      DO 70 J=1,NUM(I)
   70 JN(JJ,J)=JN(I,J)
      NUM(JJ)=NUM(I)
   80 CONTINUE
      IF(JJ.GE.NNJ) THEN
      WRITE(*,*)' NOT A BRANCHED NETWORK.  NO ADDITIONAL'
     &,' DEAD END PIPES'
      GO TO 99
      ENDIF
      NNJ=JJ
      IF(NNJ.GT.0) GO TO 45
      IF(G.LT.30.) THEN
      D=SQRT(.85*Q(1))/.8
      ELSE
      D=SQRT(.25*Q(1))/.8
      ENDIF
      WRITE(IOUT,100)
  100 FORMAT(' PIPE  LENGTH ROUGHNESS    DIA.    AREA',
     &' DISCHARGE  VELOCITY  HEAD LOSS HGL-SLOPE'
     &,/,1X,78('-'))
      DO 85 J=1,NJ
   85 H(J)=-1001.
      READ(INPUT,*)(NOP(I),L1(NOP(I)),L2(NOP(I)),I=1,NP)
      READ(INPUT,*) K,H(K),NSTART,(IPP(I),DHS(I),I=1,NSTART)
      READ(INPUT,*) RATE,LIFE,DAYS,EFF,N
      IF(G.GT.20.) THEN
      POF=2.03*DAYS*RATE/EFF
      ELSE
      POF=235.344*DAYS*RATE/EFF
      ENDIF
      CRF=RATE*(1.+RATE)**LIFE/((1.+RATE)**LIFE-1.)
      JJ=2
      DZ=DHS(1)
      IPP(NSTART+1)=10000
      COSTE=POF*QJJ(K)*(H(K)-ELEV(K))
      COSTP=0.
      DO 90 J=1,NP
      I=NOP(J)
      IF(I.EQ.IPP(JJ)) THEN
      DZ=DHS(JJ)
      JJ=JJ+1
      ENDIF
      IF(H(L1(I)).LT.-1000.) THEN
      IF(H(L2(I)).LT.-1000.) THEN
      WRITE(*,*)' CANNOT CONTINUE SINCE NO H FOR PIPE',I,
     &' IS KNOWN'
      GO TO 99
      ELSE
      QJN=QJJ(L1(I))
      HDL=(H(L2(I))-ELEV(L2(I)))/L(I)
      ENDIF
      ELSE
      QJN=QJJ(L2(I))
```

```
      HDL=(H(L1(I))-ELEV(L1(I)))/L(I)
      ENDIF
      IF(QJN.LT.0.) QJN=0.
      CALL MCOSTS(Q(I),E(I),QJN,HDL,S,D,CL,CEL,CPL)
      IF(H(L1(I)).LT.-1000.) THEN
      H(L1(I))=H(L2(I))+S*L(I)
      ELSE
      H(L2(I))=H(L1(I))-S*L(I)
      ENDIF
      COSTE=COSTE+L(I)*CEL
      COSTP=COSTP+L(I)*CPL
      D12=CONV*D
      COEF=CONV*E(I)
      A=0.7853982*D*D
   90 WRITE(IOUT,F110) I,L(I),COEF,D12,A,Q(I),Q(I)/A,S*L(I),S
      WRITE(IOUT,130) COSTE,COSTP,COSTE+COSTP
  130 FORMAT(/' COST OF ENERGY    =  $',F10.2,/
     &' COST OF PIPE       =   ',F10.2,/
     &' TOTAL COST/YEAR    =   ',F10.2,/)
      WRITE(IOUT,*)' HGL-ELEVATIONS AT NODES, IN FT'
      WRITE(IOUT,120)(J,H(J),J=1,NJ)
  120 FORMAT(6(I4,F9.2))
   99 DEALLOCATE(L,E,Q,QJ,NUM,JN,LNODE,H,NOP,L1,L2)
      STOP
      END
      SUBROUTINE MCOSTS(Q,EE,QJN,HDL,SHMIN,DMIN,CTMIN,CEMIN,CPMIN)
      REAL D(20),CP(20),S(13),CT(3),SS(3),DS(3)
      LOGICAL*2 NFIRST/.FALSE./,START/.TRUE./,SREAD/.TRUE./
      COMMON /COSTB/CRF,DZ,G,VISC,FK2,POF,N
      DATA S/.0001,.00025,.0005,.001,.002,.003,.004,.005,
     &.006,.007,.008,.009,.010/
      IF(SREAD) THEN
      READ(2,*)(D(I),CP(I),I=1,N)
      IF(G .GT. 20.) THEN
      DO 1 I=1,N
    1 D(I)=D(I)/12.
      ENDIF
      SREAD=.FALSE.
      ENDIF
      I1=1
      CT(1)=1.E20
      SF=8.
      G2=1.23370055*G
      DIA=DMIN
      K2=2
      DO 80 I=1,13
      IF(EE.GT.10.) THEN
      DIA=FK2*(Q/(EE*S(I)**.54))**.380228
      ELSE
   30 DIA1=DIA
   40 SF1=SF
      SF=1.14-2.*ALOG10(EE/DIA1+7.343472826*VISC*DIA1*SF/Q)
      IF(ABS(SF-SF1).GT. 1.E-6) GO TO 40
      DIA=((Q/SF)**2/(S(I)*G2))**.2
      IF(ABS(DIA1-DIA) .GT. 1.E-5) GO TO 30
      ENDIF
   50 IF(DIA.LT.D(K2+1).OR. K2.EQ.N-1) GO TO 60
      K2=K2+1
      GO TO 50
```

```
60 IF(DIA.GE.D(K2) .OR. K2.EQ.2) GO TO 70
   K2=K2-1
   GO TO 60
70 K1=K2-1
   K3=K2+1
   C1=CP(K1)/((D(K1)-D(K2))*(D(K1)-D(K3)))
   C2=CP(K2)/((D(K2)-D(K1))*(D(K2)-D(K3)))
   C3=CP(K3)/((D(K3)-D(K1))*(D(K3)-D(K2)))
   AC=C1+C2+C3
   BC=-C1*(D(K2)+D(K3))-C2*(D(K1)+D(K3))-C3*(D(K1)+D(K2))
   CC=C1*D(K2)*D(K3)+C2*D(K1)*D(K3)+C3*D(K1)*D(K2)
   COST=(AC*DIA+BC)*DIA+CC
   CPIP=CRF*COST
   CENE=POF*((S(I)+DZ)*Q+QJN*(HDL+S(I)))
   CTO=CPIP+CENE
   IF(CTO.LT.CT(I1).OR.I.LT.3) THEN
   IF(START) THEN
   CT(1)=CTO
   SS(1)=S(I)
   DS(1)=DIA
   START=.FALSE.
   ELSE
   IF(NFIRST) THEN
   CT(1)=CT(2)
   SS(1)=SS(2)
   DS(1)=DS(2)
   ENDIF
   CT(2)=CTO
   SS(2)=S(I)
   DS(2)=DIA
   I1=2
   NFIRST=.TRUE.
   ENDIF
   ELSE
   CT(3)=CTO
   SS(3)=S(I)
   DS(3)=DIA
   AA=0.
   DA=0.
   BB=0.
   DB=0.
   CC=0.
   DC=0.
   DO 76 J=1,3
   CR=CT(J)
   DR=DS(J)
   DO 74 K=1,3
   IF(K.EQ.J) GO TO 74
   CR=CR/(SS(J)-SS(K))
   DR=DR/(SS(J)-SS(K))
74 CONTINUE
   AA=AA+CR
   DA=DA+DR
   SUM=0.
   PRO=1.
   DO 75 K=1,3
   IF(K.EQ.J) GO TO 75
   SUM=SUM+SS(K)
   PRO=PRO*SS(K)
```

```
 75 CONTINUE
    BB=BB-CR*SUM
    DB=DB-DR*SUM
    DC=DC+DR*PRO
 76 CC=CC+CR*PRO
    SHMIN=-0.5*BB/AA
    CTMIN=(AA*SHMIN+BB)*SHMIN+CC
    DMIN=(DA*SHMIN+DB)*SHMIN+DC
    CEMIN=CTMIN*CENE/CTO
    CPMIN=CTMIN*CPIP/CTO
    RETURN
    ENDIF
 80 CONTINUE
    END
```

The input data file (DESIGMU1.DAT) for MCBRAN for this problem follows:

```
32.2 1.41E-5
3000 1320/
0.005/
1100./
-13.36809 1/
.5347236 -1 2 12 16/
.5347236 -2 3 8/
.5347236 -3 4/
.5347236 -4 5/
.5347236 -5 6/
.5347236 -6 7/
.5347236 -7/
.5347236 -8 9/
.5347236 -9 10/
.5347236 -10 11/
.5347236 -11/
.5347236 -12 13/
.5347236 -13 14/
.5347236 -14 15/
.5347236 -15/
.5347236 -16 17 21/
.5347236 -17 18/
.5347236 -18 19/
.5347236 -19 20/
.5347236 -20/
.5347236 -21 22/
.5347236 -22 23/
.5347236 -23 24/
.5347236 -24 25/
.5347236 -25/
7 7 8 6 6 7 5 5 6 4 4 5 3 3 4 8 3 9 9 9 10 10 10 11 11 11 12
2 2 3 12 2 13 13 13 14 14 14 15 15 15 16 16 2 17 17 17 18 18 18 19
19 19 20 20 20 21 21 17 22 22 22 23 23 23 24 24 24 25 25 25 26 1 1 2
8 1192.3 1 7 .0091575
.09 20 365 .7 8
6 12 8 14.2 10 20.4 12 26.6 15 31 18 54.2 24 69.4 36 109.6
```

The table provides part of the solution, and additional information follows:

Pipe	Length ft	ex10^3 in	Dia. in	Area ft^2	Q ft^3/s	V ft/s	h_L ft	HGL-Slope
7	1320	5.0	7.02	0.268	0.535	1.992	2.966	0.00225
6	1320	5.0	8.45	0.389	1.069	2.746	4.363	0.00331
5	1320	5.0	9.57	0.499	1.604	3.214	5.076	0.00385
4	1320	5.0	11.16	0.680	2.139	3.147	4.028	0.00305
3	1320	5.0	13.77	1.034	2.674	2.586	1.979	0.00150
8	1320	5.0	11.16	0.680	2.139	3.147	4.028	0.00305
9	1320	5.0	9.57	0.499	1.604	3.214	5.076	0.00385
10	1320	5.0	8.45	0.389	1.069	2.746	4.363	0.00331
11	1320	5.0	7.02	0.268	0.535	1.992	2.966	0.00225
2	1320	5.0	15.43	1.298	5.347	4.120	4.523	0.00343
12	1320	5.0	11.16	0.680	2.139	3.147	4.028	0.00305
13	1320	5.0	9.57	0.499	1.604	3.214	5.076	0.00385
14	1320	5.0	8.45	0.389	1.069	2.746	4.363	0.00331
15	1320	5.0	7.02	0.268	0.535	1.992	2.966	0.00225
16	1320	5.0	15.43	1.298	5.347	4.120	4.523	0.00343
17	1320	5.0	11.16	0.680	2.139	3.147	4.028	0.00305
18	1320	5.0	9.57	0.499	1.604	3.214	5.076	0.00385
19	1320	5.0	8.45	0.389	1.069	2.746	4.363	0.00331
20	1320	5.0	7.02	0.268	0.535	1.992	2.966	0.00225
21	1320	5.0	13.77	1.034	2.674	2.586	1.979	0.00150
22	1320	5.0	11.16	0.680	2.139	3.147	4.028	0.00305
23	1320	5.0	9.57	0.499	1.604	3.214	5.076	0.00385
24	1320	5.0	8.45	0.389	1.069	2.746	4.363	0.00331
25	1320	5.0	7.02	0.268	0.535	1.992	2.966	0.00225
1	3000	5.0	25.49	3.544	13.37	3.772	4.386	0.00146

COST OF ENERGY = $ 248,987.70
COST OF PIPE = 87,081.89
TOTAL COST/YEAR = 336,069.60

The listed energy costs in this output do not include the capital recovery cost for the pump, which is crf(180,000) = 0.10955(180,000) = $19,719.

HGL-ELEVATIONS AT NODES, IN FT

1	1219.62	2	1215.23	3	1210.71	4	1208.73
5	1204.70	6	1199.63	7	1195.27	8	1192.30
9	1206.68	10	1201.61	11	1197.25	12	1194.28
13	1211.21	14	1206.13	15	1201.77	16	1198.80
17	1210.71	18	1206.68	19	1201.61	20	1197.25
21	1194.28	22	1208.73	23	1204.70	24	1199.63
25	1195.27	26	1192.30				

Next the nearest standard pipe diameters are selected to replace the computed diameters, this network is analyzed, and the costs are computed. The cost of the tank will be ignored for now, under the assumption that its size is independent of the pipe sizes and the amount of energy used by the pumps. The summary of these costs is given below.

Standard Pipe Diameters used in Analysis Solution

Pipe	1	2	3	4	5	6	7	8	9	10	11	12	13
D, in	24	15	15	15	10	8	8	12	10	8	8	12	10

Pipe	14	15	16	17	18	19	20	21	22	23	24	25
D, in	8	8	15	12	10	8	8	15	12	10	8	8

SUMMARY OF COSTS

ITEM	TYPE	PRESENT WORTH	SERIES AMOUNT
2	PIPE	832,560.00	75,951.85
8	PUMP	2,734,285.00	299,531.30
	TOTAL	3,566,845.00	375,483.10

From this solution the head at node 1 is 219.2 ft, which is the head that the pump(s) must supply.

A second alternative is to use special input that is employed by NETWK in defining branched networks; the analyst would indicate that the design solution should be followed by an analysis solution based on nearest standard pipe sizes and then obtain an economic analysis of this solution. The input data to request these analyses is given below, in which the HGL-slope has been specified as 0.002.

SPECIAL INPUT TO SOLVE MUNICIPAL DESIGN BRANCHED NETWORK:

```
/*
$SPECIF IHGL=-2,DESIGN=1,NFLOW=1,NPGPM=1,NOMSOL=1,ICOST=1 $END
1214.1 -6000 1000 240 .005
ELEV
1100
1 2 .002 3000/
2 8 .002 1320/
3 12/
2 16/
2 21/
17 26/
END
RUN
8 1192.3
INTEREST=.09
LIFE=20
PIPES
UNIT=8
6 12 8 14.2 10 20.4 12 26.6 15 24 18 54.2 24 69.4 36 109.6
EFFIC
.7
PUMPS
UNIT=.09
CAPI=180000.
END
```

In brief: (1) The option IHGL=-2 tells NETWK that special input will be given that defines a branched pipe system. The first line of this input contains (a) the HGL-elevation at the beginning node, (b) the demand here, (c) the elevation to use until it is changed, (d) the demand to apply at subsequent nodes until it is changed, (e) the pipe roughness to use until it is changed. Subsequent lines define the branched system by giving (a) the initial node, (b) the final node, (c) the HGL-slope, and (d) a list of pipe diameters, ending with / if the last given diameter is to be used for the remaining pipes along this branch. (2) The option NOMSOL=1 requests, after a solution to find precisely the pipe diameters (because DESIGN=1) that conform to the head differences between nodes (the heads will be established from the specified slopes of the HGL's), that a regular analysis be performed, in which the nearest standard pipe sizes are used. (3) The option ICOST=1 requests an engineering economic analysis. The HGL elevation at node 1 has been determined as 1192.3 (the HGL elevation at node 8) plus the sum of products of slope and pipe length between node 8 and node 1, or 1192.3 + 0.002(10920) = 1214.1 ft. NETWK must be given an initial HGL so it can compute HGL elevations, pressures, and heads. Since the above data does not contain any supply sources, a node and HGL at this node must be given immediately after the RUN command for the analysis solution.

To obtain solutions for different HGL-slopes, three values in the foregoing input data must be changed, the two HGL slopes (0.002) and the initial HGL elevation (1214.1 ft). The cost summary from the economic analysis for this solution is as follows:

SUMMARY OF COSTS

ITEM	TYPE	PRESENT WORTH	SERIES AMOUNT
2	PIPE	960,072.00	105,172.50
8	PUMP	2,686,039.00	294,246.10
	TOTAL	3,646,111.00	399,418.60

Similar solutions for HGL slopes of 0.0003, 0.0005, 0.00075, 0.001, 0.003, 0.004, and 0.005 provide the following total series costs: \$430,927 for S = 0.0003, \$405,135 for S = 0.0005, \$396,590 for S = 0.00075, \$393,539 for S = 0.001, \$399,418 for S = 0.002, \$397,424 for S = 0.003, \$410,013 for S = 0.004, and \$410,014 for S = 0.005. The least cost is found for S = 0.001 (but costs vary little between S = 0.00075 and 0.003, inclusive), and the nearest standard pipe sizes for this solution are listed in this table:

Pipe	1	2	3	4	5	6	7	8	9	10	11	12	13
D, in	30	20	15	15	12	10	8	15	12	10	8	15	12

Pipe	14	15	16	17	18	19	20	21	22	23	24	25
D, in	10	8	20	15	12	10	8	15	15	12	10	8

The head that the pump(s) must supply for this latter approach is 203.2 ft, or 16 ft less than the result from the first analysis. Several pipes are now one standard pipe size larger than was obtained when the best HGL slope was obtained for each individual pipe. For the second alternative the total annual cost is \$396,971. The other alternative led to an annual recovery cost of \$375,483, a reduction of \$21,489.

Step 7. Since pipe 1 is important in providing emergency flows, we will choose a 30-inch diameter for it, as indicated by the second analysis. A storage tank will be connected to the system by a 200-ft long 24-in-diameter pipe from node 16, and the pipes that were removed will be given 6-in diameters. For a preliminary choice of pumps we choose two parallel pumps, each with characteristics defined by the three (Q, h_p) pairs in the table:

Pump Operating Characteristics

Q, gal/min.	1500	3000	4500
h_p, ft	234.0	219.2	197.0

When the demands are 0.8 times 240 gal/min, we decide that we want the tank neither to receive nor supply any of the demand, and both pumps are operating then. The following input to NETWK will provide a solution that determines the tank's water surface elevation in this instance:

Analysis based on nearest standard diameters

```
/* 37 20 25
$SPECIF NODESP=1,PEAKF=.8,
  NFLOW=1,NPGPM=1 $END
PIPES
1 1 2 3000 24 .005
2 2 3 1320 15
3 3 4 1320 15
4 4 5 1320 12
5 5 6 1320 10
6 6 7 1320 8
7 7 8 1320 8
8 3 9 1320 12
9 9 10 1320 10
10 10 11 1320 8
11 11 12 1320 8
12 2 13 1320 12
13 13 14 1320 10
14 14 15 1320 8
15 15 16 1320 8
16 2 17 1320 15
17 17 18 1320 12
18 18 19 1320 10
19 19 20 1320 8
20 20 21 1320 8
21 17 22 1320 15
22 22 23 1320 12
23 23 24 1320 10
24 24 25 1320 8
25 25 26 1320 8
26 9 5 1320 6
27 13 9
28 13 18
29 18 23
30 10 6
31 14 10
32 14 19
33 19 24
34 11 7
35 15 11
36 15 20

38 12 8
39 16 12
40 16 21
41 21 26
NODES
1 0 1000
2 240 1100
3 240 1100
4 240 1100
5 240 1100
6 240 1100
7 240 1100
8 240 1100
9 240 1100
10 240 1100
11 240 1100
12 240 1100
13 240 1100
14 240 1100
15 240 1100
16 240 1100
17 240 1100
18 240 1100
19 240 1100
20 240 1100
21 240 1100
22 240 1100
23 240 1100
24 240 1100
25 240 1100
26 240 1100
PUMPS
1 1500 234 3000 219.2 4500 197 4000
PARALLEL
1 2/
RUN
```

The solution indicates that the pumps will supply 0.8(6000) = 4800 gal/min and provide a head of 274.2 ft (which appears to be more than needed), and the head at node 16 is 109.8 ft, so the middle level of the storage tank should be near this elevation. As a preliminary design we shall select the tank so its volume will supply the network for one day. The average total demand is 6000/2.3 = 2610 gal/min = 5.81 ft^3/s. Multiplying this demand by 24x3600 sec/day yields a volume of 502,000 ft^3. If the tank is 10 ft high and circular, then it should have a diameter of 253 ft. Let us specify the diameter as 250 ft (Volume = 490,874 ft^3) with a mid-level water surface elevation of 1200 ft. Then the bottom of the tank will be placed at elevation 1195 ft, and its top will be at 1205 ft.

Now several steady state solutions must be obtained to verify the adequacy of the network under a variety of possible operating conditions. In testing the network for fire flows, extra demands of 2000 gal/min will be located at nodes 6, 8, 9, and 20. The next table lists the smallest pressures and the flows from the pumps and the reservoirs for these analyses under the assumptions that both pumps were operating, that the water surface elevation in the storage tank is at 1200 ft, and that the peak daily demands are occurring at the time of the fires.

Fire Demand Consequences at a Node

Node	Min. Pressure lb/in^2	*Q*, Pumps gal/min	ft^3/s	*Q*, Reservoir gal/min	ft^3/s
6	31.4	6420	14.3	1580	3.52
8	2.5	6290	14.0	1710	3.82
9	39.3	6470	14.4	1530	3.41
20	29.5	6250	13.9	1750	3.91

These fire flow analyses indicate that the network can not supply an additional 2000 gal/min at node 8 (and obviously not at node 26 either) to combat a fire. This inadequate performance is caused by the (initially removed) pipes which run in the North-South direction and were assigned the minimum diameter of 6 inches; the problem can be ameliorated and possibly corrected fully by increasing the diameter of some of these pipes to 8 in. Let us try increasing the size of pipes 38, 39, 40, and 41 to 8 in. The same fire flow analyses now produce the following results, which shows that all pressures are now above the required minimum of 20 lb/in^2.

Fire Demand Consequences at a Node

Node	Min. Pressure lb/in^2	*Q*, Pumps gal/min	ft^3/s	*Q*, Reservoir gal/min	ft^3/s
6	35.0	6070	13.5	1930	4.29
8	22.0	5860	13.1	2140	4.76
9	42.0	6150	13.7	1850	4.13
20	34.5	5870	13.1	2160	4.74

If the water surface elevation in the tank is at its lowest level but still able to supply water when these fire flows occur, and if both pumps are then operating, another computation will produce the following pressures and discharges:

Fire Demand Consequences at a Node
Tank Water Surface at 4195 ft, Two Pumps Operating

Node	Min. Pressure lb/in^2	*Q*, Pumps gal/min	ft^3/s	*Q*, Reservoir gal/min	ft^3/s
6	31.5	6700	14.9	1300	2.89
8	17.9	6460	14.4	1540	3.43
9	38.5	6820	15.2	1180	2.64
20	30.5	6460	14.4	1540	3.42

The pressure was only slightly above the required minimum when the tank water surface was at mid-level, and now with the water surface at the base of the tank the pressure is only 18 lb/in^2 at node 8. The same would apply to node 26. If only one of the two parallel pumps were in operation with the tank nearly empty, then the following results would be found:

Fire Demand Consequences at a Node
Tank Water Surface at 4195 ft, One Pump Operating

Node	Min. Pressure lb/in^2	Q, Pumps gal/min	ft^3/s	Q, Reservoir gal/min	ft^3/s
6	22.8	5130	11.4	2870	6.40
8	11.2	5020	11.2	2990	6.65
9	29.4	5170	11.5	2840	6.32
20	23.5	5030	11.2	2970	6.63

As might have been anticipated, now the pressure at node 8 (and 26) is significantly deficient for a fire demand at these nodes. Some means of correcting the problem should be sought. From the solution for the fire demand at node 8, it is observed that the head losses in pipes 15, 34, and 35 are 15.1 ft, 15.1 ft, and 15.3 ft, respectively. Thus one possible solution might be the use of a 10-in diameter for pipe 15 and an 8-in diameter for pipes 34, 35, 36, and 37. With these additional pipes enlarged, the following pressures and discharges are obtained with the reservoir empty and only one pump in operation:

Fire Demand Consequences at a Node
Tank Water Surface at 4195 ft, One Pump Operating

Node	Min. Pressure lb/in^2	Q, Pumps gal/min	ft^3/s	Q, Reservoir gal/min	ft^3/s
6	25.9	4930	11.0	3070	6.84
8	15.5	4820	10.7	3180	7.09
9	31.5	4970	11.1	3030	6.76
20	28.8	4830	10.8	3170	7.06

Although the 15.5 lb/in^2 pressure at node 8 is deficient, it is notably better than in the previous case and not markedly less than 20 lb/in^2. Since the joint probability of finding the tank empty with only one pump in operation should be very low, we will tentatively accept these pipe sizes, move to step 8 and investigate this network further with an extended time simulation.

Step 8. A primary purpose of the use of an extended time simulation for this network is to examine the adequacy of the storage tank. Will it empty or overflow, and will the water depth be approximately unchanged after a 24-hour simulation? We begin this simulation at a moment when the average demands occur. We assume a demand function that is described by the following table, and that this function applies at all nodes.

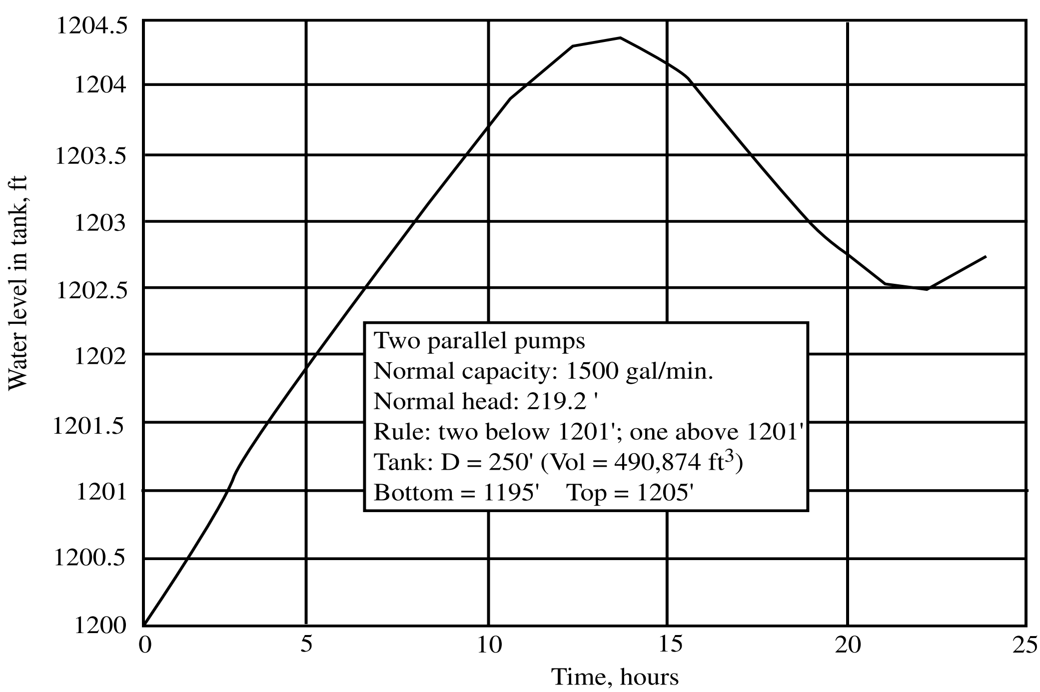
Demand Function

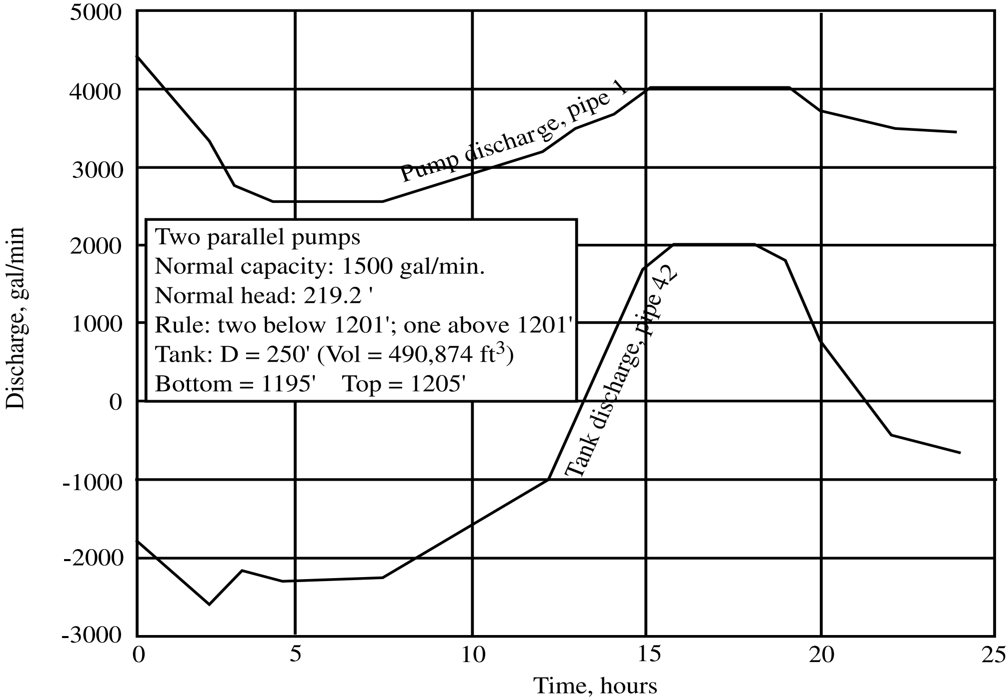
Hour	Peaking Factor
0	1.00
2	1.13
4	1.70
5	2.20
6	2.30
8	2.30
12	0.80
15	0.30
17	0.10
20	0.10
22	0.30
24	1.00

Assume the operation of the pumps will be determined by a level switch in the tank; when the water surface elevation reaches 1201 ft, a pump is turned off so only one pump is in operation, and when the level drops below elevation 1201, the second pump is turned on again. The additional input data for NETWK for the simulation, after the input that defines the network, is then as follows:

```
$TDATA ALTV=1,INCHR=1,ISUNIT=0,LINEAR=1,PRINTT=3,NPUNOD=0,NPNRES=1 $END
PIPE TABLE
ALL
NODE TABLE
ALL
RESER. TABLE
1/
END TABLES
DEMAND FUNCTION
1 0 1 2 1.13 4 1.7 5 2.2 6 2.3 8 2.3 9 2.15 12 .8 15 .3 17 .1 20 .1 22 .3
24 1/
2-26/
STORAGE FUNCTION
1 1195 0 1200 245437 1205 490874/
1/
PUMP RULES
1 2 1 1 1201 1 1199 2 1201 1/
END SIMULATION
```

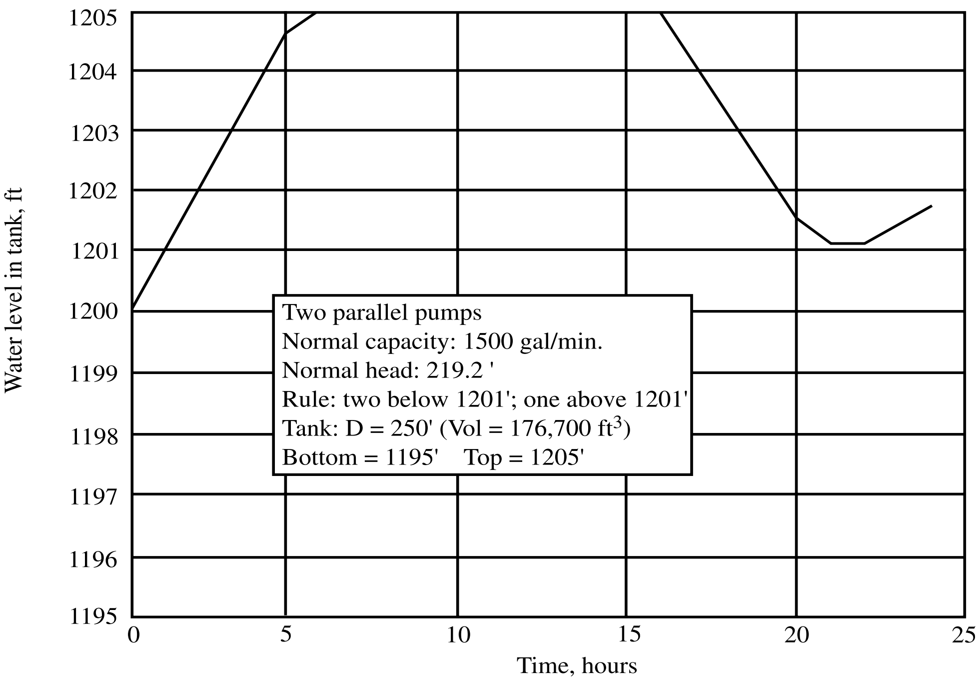
The results of this simulation are summarized in the following two plots. From the first plot we see that the tank's water level rises to within 0.7 ft of the top of the tank at 14 hours, and in satisfying the peak demands the water surface drops to elevation 1202.5, 2.5 ft above its initial mid-level elevation. The results indicate we have more pump capacity than is needed; if this operation were to continue for several days, the tank clearly

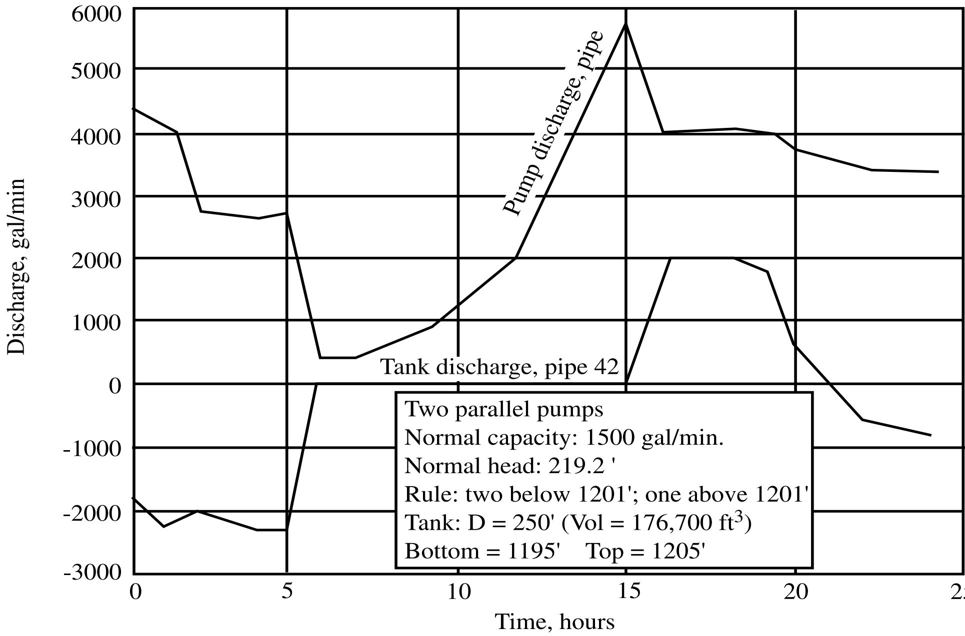




would overflow. We would not be using the tank storage effectively. We should consider reducing the diameter of the tank.

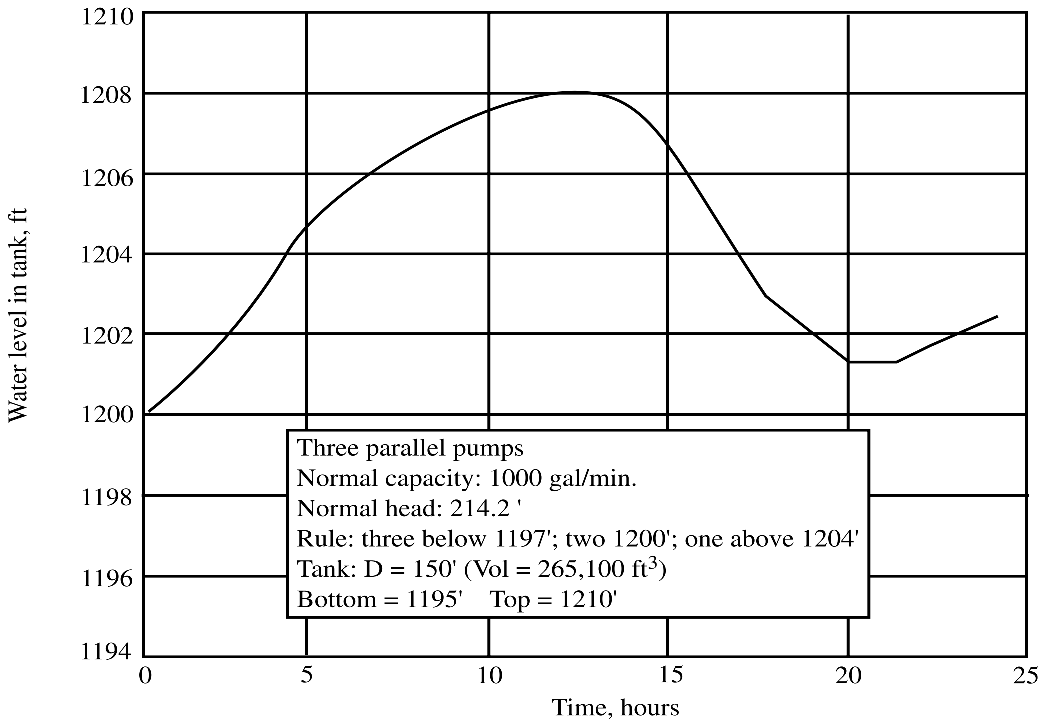
In the next simulation the tank diameter was reduced to 150 ft but the base remained at 1195 ft and the top at 1205 ft, so its new volume was 176,700 ft^3. The results of this simulation are shown in the following two graphs. Now the tank fills after 6 hours of



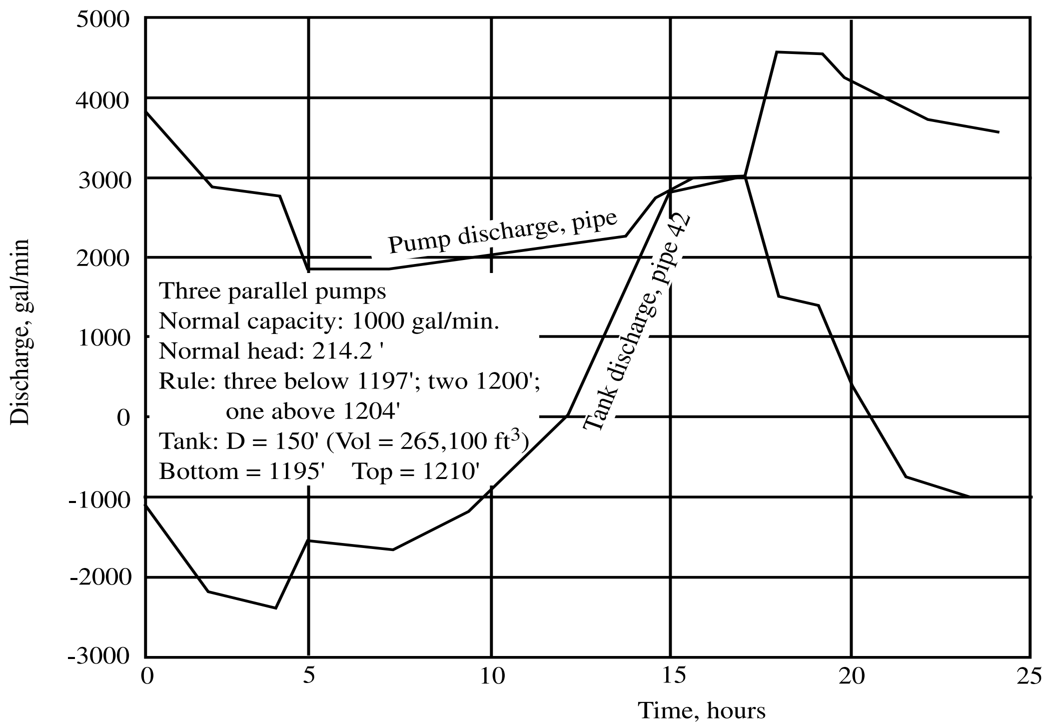


the simulation, and the demands must be met exactly by the pumps until the demand exceeds their capability to do so at 16 hours. Clearly this is not a good tank-pump configuration and operation. The simulation results thus force us to two conclusions: (1) the pumping capacity can be reduced so the water surface elevation in the tank returns nearly to mid-level at the end of the 24 hour simulation, and (2) the elevation of the top of the tank should be increased.

The third simulation will therefore employ three pumps in parallel, each with a normal capacity of 1000 gal/min and a normal head of 214.5 ft (5 ft less than the head of the previous two pumps). The tank will be given a diameter of 150 feet, but we shall raise the top of the tank by 5 ft to elevation 1210, so the tank is now 15 ft high rather than 10 ft, and it has a volume of 265,100 ft^3. The new pump rule is to start the simulation with two pumps operating but to utilize only one pump when the water surface in the tank equals or exceeds 1204 ft. When the water surface drops below 1197 ft, all three pumps will be placed in operation. The results from this simulation are shown below. At the end of the 24-hour simulation the water surface elevation in the tank is just over 2 feet above its initial level, and at 13 hours the water level is within 2 feet of the top. Considering the possibility of occurrence of an emergency demand, this operation is quite satisfactory. The input data to NETWK for this last simulation is given below. What improvements might you suggest? (The final cost analysis is left as an exercise.)



Water level in tank, ft
1210
1208
1206
1204
1202
1200
1198
1196
1194
0
5
10
15
20
25
Time, hours
Three parallel pumps
Normal capacity: 1000 gal/min.
Normal head: 214.2 '
Rule: three below 1197'; two 1200'; one above 1204'
Tank: D = 150' (Vol = 265,100 ft3)
Bottom = 1195' Top = 1210'



Discharge, gal/min
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
0
5
10
15
20
25
Time, hours
Pump discharge, pipe
Tank discharge, pipe 42
Three parallel pumps
Normal capacity: 1000 gal/min.
Normal head: 214.2 '
Rule: three below 1197'; two 1200'; one above 1204'
Tank: D = 150' (Vol = 265,100 ft3)
Bottom = 1195' Top = 1210'

Input to NETWK for the third extended time simulation:

```
Analysis solution based on
nearest standard diameters
/*
$SPECIF ISIML=1,PEAKF=.4347826
,NODESP=1,NFLOW=1,NPGPM=1 $END
PIPES
1 1 2 3000 24 .005
2 2 3 1320 15
3 3 4 1320 15
4 4 5 1320 12
5 5 6 1320 10
6 6 7 1320 8
7 7 8 1320 8
8 3 9 1320 12
9 9 10 1320 10
10 10 11 1320 8
11 11 12 1320 8
12 2 13 1320 12
13 13 14 1320 10
14 14 15 1320 8
15 15 16 1320 10
16 2 17 1320 15
17 17 18 1320 12
18 18 19 1320 10
19 19 20 1320 8
20 20 21 1320 8
21 17 22 1320 15
22 22 23 1320 12
23 23 24 1320 10
24 24 25 1320 8
25 25 26 1320 8
26 9 5 1320 6
27 13 9
28 13 18
29 18 23
30 10 6
31 14 10
32 14 19
33 19 24
34 11 7 1320 8
35 15 11
36 15 20
37 20 25
38 12 8 1320 8
39 16 12
40 16 21
41 21 26
42 27 16 200 24
NODES
1 0 1000
2 240 1100
3 240 1100
4 240 1100
5 240 1100
6 240 1100
7 240 1100
8 240 1100
9 240 1100
10 240 1100
11 240 1100
12 240 1100
13 240 1100
14 240 1100
15 240 1100
16 240 1100
17 240 1100
18 240 1100
19 240 1100
20 240 1100
21 240 1100
22 240 1100
23 240 1100
24 240 1100
25 240 1100
26 240 1100
27 0 1100
PUMPS
1 1000 229 2000
214.2 3000 192 4000
RESER
27 1200.
RUN


$TDATA ALTV=1,INCHR=1,ISUNIT=0,LINEAR=1,PRINTT=3,NPUNOD=0,NPNRES=1 $END
PIPE TABLE
ALL
NODE TABLE
ALL
RESER. TABLE
1/
END TABLES
DEMAND FUNCTION
1 0 1 2 .3 4 .1 7 .1 9 .3 12 .8 15 2.15 16 2.3 18 2.3 19 2.2 20 1.7 22 1.13
  24 1/
2-26/
STORAGE FUNCTION
1 1195 0 1200 88357 1205 176715 1210 265072/
1/
PUMP RULES
1 2 1 2 1197. 3 1200. 2 1204 1/
END SIMULATION
```

* * *

6.4.2. DESIGN GUIDELINES FOR COMPLEX NETWORKS

The design procedure, outlined in eight steps, works well if there is one major supply source so the branched network can be used to start the design process. If several major supply sources exist, the same basic procedure can be followed, with each supply at the head of separate branched systems that are later connected. However, this approach presupposes a knowledge of which portion of the network is supplied by each source. A more general procedure may use the following steps as guideline in the design:

1. Identify the two dominant supply sources for the system. A criterion for selecting these sources is to seek the sources with potentially the largest total heads. Connect these sources by the shortest path of pipes between them. In this methodology this path will be called the dominant path. In the branched system to be defined, all other paths will ultimately terminate at one of the nodes along this dominant path. If only one supply source exists, this dominant path is not defined.
2. Connect each other supply source to one of the nodes on the dominant path via the shortest available path of pipes. These additional paths will be called primary paths. In selecting the node of the dominant path at which a primary path terminates, preference is given to nodes closer to the dominant source with the largest total head. However, all primary paths can be sequenced in descending magnitude of the total head available at the primary path. If only two supply sources exist, they are the dominant sources and this step is omitted.
3. Connect the remaining nodes of the network, ones that are not included in the dominant path or any primary path, by the shortest path of pipes to one of the nodes of the dominant path. Whenever this path intersects a node in a previous path, it is terminated. These paths of pipes will be called secondary paths. The sequence in which these secondary paths are formed is first from nodes of degree one, i.e., dead end pipes, next from nodes of degree two, i.e., that have only two connecting pipes, and so on. The order in which nodes are selected within a given degree is by descending elevation.

Upon completing these three steps, a branched system of pipes has been formed. It includes all nodes of the network and presumably contains the pipes that will convey the majority of the flow from the supply sources to the various demand points throughout the network. The pipes that are not included in this branched network are called additional loop-forming pipes. Their diameters can be arbitrarily specified and, if not based on other criteria, will be the minimum diameter that is permitted.

4. Establish an appropriate head at each node of the network. We do this by working through the paths in the reverse order of their formation. By ignoring the carrying capacity of the additional loop-forming pipes, the discharge in each pipe of the branched system is determined. At the beginning node of each path, the total head is equated to the minimum allowable pressure head plus the elevation of the node. Proceeding from this node to succeeding nodes on the path, the total heads are established by utilizing the optimum S associated with this discharge, as established above. If the head at this node was previously assigned, then the currently computed head is compared with the previous head, and the larger of the two is retained. If any pressure head is computed to be less than the minimum acceptable pressure head, then all previously assigned HGL values along that path are raised. If the pressure head exceeds a maximum specified value that requires the inclusion of a booster pump, then consider putting a booster pipe in this pipe. The total head at nodes that are upstream from the pipe in which a booster pump is placed should then be reduced by the amount of head supplied by the pump.

When this procedure for establishing HGL elevations has progressed to the primary paths, it is necessary to know the discharges that the reservoirs supply, or receive, in order to determine the discharges in the pipes on these primary paths. Rules might be used to assign a fraction of the total demand (positive or negative) for each source to supply.

Aside from this discharge requirement, the total heads are computed at nodes along primary paths in the same manner as along secondary paths.

5. The total heads at nodes on the dominant path, which have not been assigned previously, are determined last by a process that is designed not only to allow an optimal, or near optimal, choice of the size of the pipes, but also to assist the designer in determining the minimum heads that the two dominant sources of supply should have.

To understand how these heads are determined, it will be helpful to assume that N_d nodes exist along the dominant path, excluding the two dominant supply sources themselves. The sketch in Fig. 6.2 has $N_d = 3$. A total of N_d different cases will be examined, which each assume the flow is directed to one of the N_d nodes from both sides.

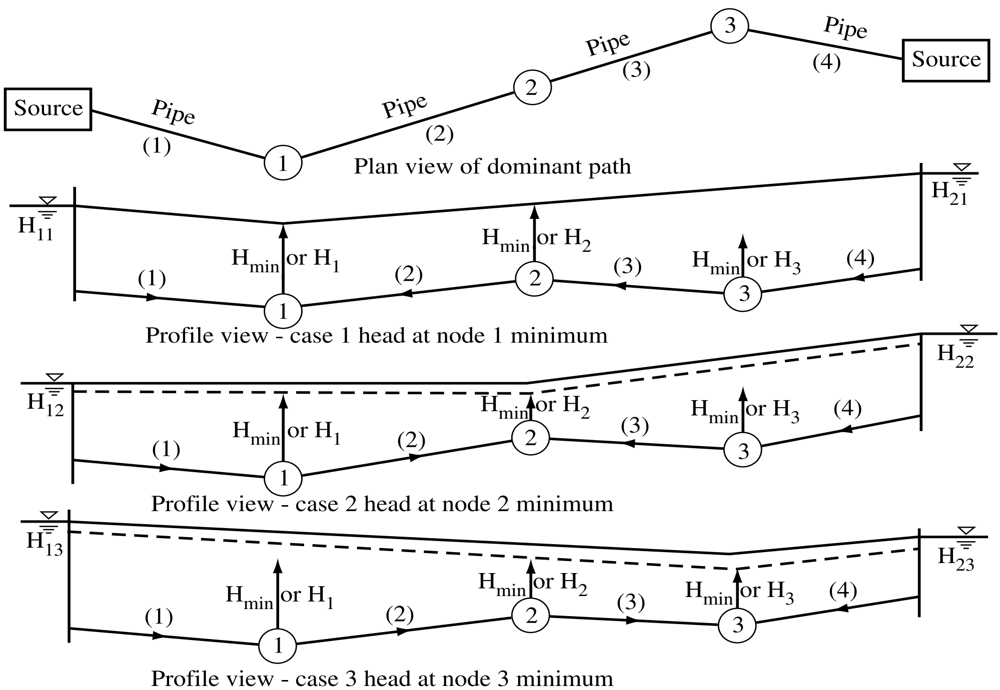


Figure 6.2 Dominant path cases.

For case 1 the HGL must slope from both directions toward node 1, which is nearest to one of the dominant sources; for case 2 the HGL slopes from both directions toward node 2, etc. until for the N_d-th case the HGL slopes toward the node which is nearest to the other dominant source. The elevation of the HGL for each case starts at the node at either the minimum head, H_{min}, above the elevation of the node, or at the head, H, that may have been established during step 4. The slope, or gradient, of the HGL is the optimum slope corresponding to the discharge carried by the pipe. These discharges can be determined since the discharges, or demands, leaving each of the nodes of the dominant path are known when step 4 has been completed. At the starting node for each case, the demand at this node is directed away from the dominant source, from which it receives its supply, and toward node $N_d + 1$. If the HGL should fall below the minimum head, H_{min}, or the total head, H, required from step 4, whichever is larger, then the entire HGL must be raised so it will not fall below any required head. The raising of the HGL is illustrated in cases 2 and 3 in Fig. 6.2. For each case, i, the required total heads, H_{1i} and H_{2i}, for the two dominant supply sources are computed. The case that produces the smallest sum of these source heads is selected, i.e. the case which produces the minimum value of $(H_{1i} + H_{2i})$ is used to establish the total head for all nodes along the dominant path unless judgment suggests that some other heads should be assigned to these supply sources. If the dominant source is a reservoir, then this sets the mid-level water surface elevation, or if this source is a pump, then this determines the head that the pump is to supply.

This procedure not only establishes the heads of the dominant sources, but it also determines the water surface elevations or heads at the other sources at the beginning of the primary paths.

6. With the total heads and discharges now known for each pipe in the branched system from steps 1 through 5, the diameters of all pipes can be computed. These diameters may be computed from the Darcy-Weisbach or Hazen-Williams equation, or even from some other equivalent equation.

6.5 PROBLEMS

6.1 Decide whether (a) an extended time simulation, (b) an unsteady solution that accounts for inertial but not elastic effects, or (c) a full transient analysis might be most appropriate for each of the following situations:

(1) The overall performance of a city water system is to be analyzed to evaluate its ability to accommodate an proposed new subdivision.
(2) A pipe supplies a lumber mill that uses a large jet of water to debark tree stumps, and the valves that control the jet are able to respond quickly so the jet can be shut down rapidly if needed.
(3) A network of pipes is used in a manufacturing plant to supply large amounts of water to numerous locations, and the usage at these locations varies rapidly.
(4) A pipe network in a large building is to be installed for fire protection.
(5) An automatic sprinkler system for a golf course has a timer that controls the irrigation of different portions of the course on a regular schedule.

6.2 Obtain an extended time simulation for the 30-pipe, 16-node network in Example Problem 6.1 if the nodes at which the two demand functions apply are interchanged, that is, the first demand function now applies to nodes 3, 4, 7, 8, 11, 12, 15, and 16, and the second demand function now applies to the other set of nodes.

6.3 Obtain an extended time simulation for the network of Example Problem 6.1 so you can examine the operational consequences of using "pump rules" that you create, based on the water level in the tank that is connected to the network by pipe 30.

6.4 Obtain an extended time simulation of the network in Example Problem 6.1. Start the simulation with two pumps on. Reverse in time the application of the two demand functions so, for example, the demand functions in this problem at 2 hours correspond to those at 23 hours in the original Example Problem 6.1, the demand functions at 3 hours correspond to those at 22 hours etc.

6.5 Obtain an extended time simulation for the network shown below if all of the demands change according to the peaking factor schedule in the table. In the diagram the ground elevation is listed at each node and below the base of the storage tank. The demands on the diagram are 1.5 times the average demands. The peaking factor is expressed as a multiple of the average demands.

Peaking factor PF as a function of time

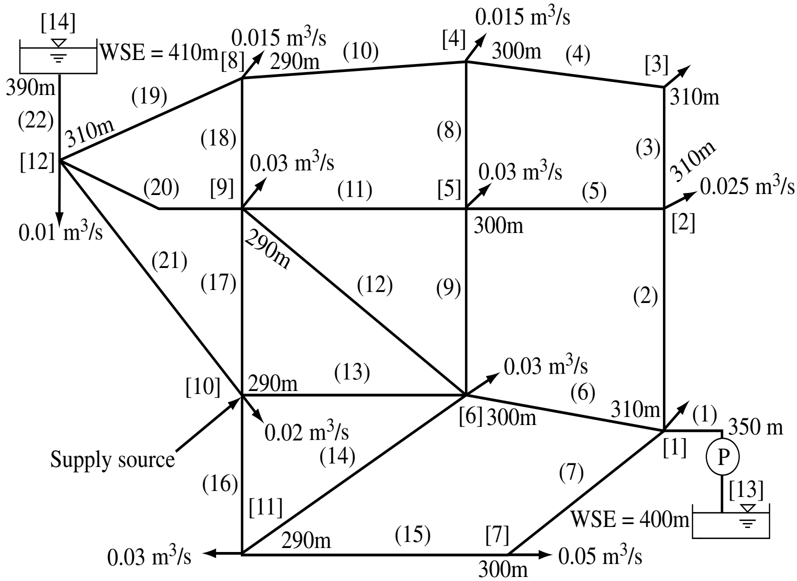
Time, hrs.	0	2	4	6	8	9	10	11	12
PF	1.0	1.2	1.5	2.1	2.5	2.5	2.0	1.4	1.0

Time, hrs.	14	16	18	20	21	22	23	24
PF	0.8	0.4	0.3	0.25	0.25	0.4	0.8	1.0

Pipe No.	Dia. mm	Length m	Pipe No.	Dia. mm	Length m	Pipe No.	Dia. mm	Length m
1	380	1000	9	205	1100	17	205	1000
2	305	1200	10	255	1200	18	150	800
3	305	800	11	205	1100	19	205	1100
4	255	1200	12	205	2000	20	205	1000
5	255	1000	13	255	1200	21	205	2200
6	255	1200	14	255	2000	22	305	800
7	305	1200	15	255	2000			
8	205	1000	16	150	800			

Pump Table, Two Parallel Pumps

Q, m^3/s	0.40	0.45	0.50
h_p, m	48	45	38



There are three parallel pumps that can be operated to increase the head in pipe 1, with the pairs of values given in the pump table actually representing two parallel pumps in operation. The pressure at node 9 is used to control the operation of these pumps as follows: if $p < 1000$ kPa, then 3 pumps are on; if $p = 1120$ kPa, then 2 pumps are on; if $p > 1200$ kPa, then 1 pump is on. The tank [14] at the end of pipe 22 has a diameter of 60 m; its bottom is at elevation 405 m, and it is 15 m high. At node 10 there is a supply source of water that can be purchased (at a relatively high price), and so it is only used when the water level in the storage tank [14] is below 403 m, and then the source will be turned on to supply 0.40 m^3/s.

6.6 From the extended time simulation of Problem 6.5 decide what components of the network should be altered to improve its performance. This may also include rules for the operation of the pumps and/or the purchase of water from the source at node 10.

6.7 Obtain an extended time simulation in hourly increments over a 24 hour period for the operation of the 20-pipe network shown below that is supplied by mountain reservoirs on pipes 1, 2 and 3, and has a storage reservoir on pipe 15. The water surface elevations of the reservoirs are shown on the sketch at time 0, and data in the table below provides the storage vs. water surface elevation relationship for the reservoirs. The demands for the 24-hour period begin with those shown on the diagram, and they then increase to 1.5 times these values in 3 hours and remain constant thereafter according to (t = 1 hr, PF = 1.1), (t =2 hr, PF = 1.2), (t = 3 hr, PF = 1.5). Two butterfly valves are used to control the discharges in pipes 14 and 16, with loss coefficients K given by the equation $K = 244e^{-0.0567x}$, in which x is the number of degrees of opening (0^o is closed and 90^o is open). The amount of opening of the valve in pipe 14 is controlled as a schedule:

Time, hr	0	1	2	3
x, degrees	8.57	15.71	56.29	90.00

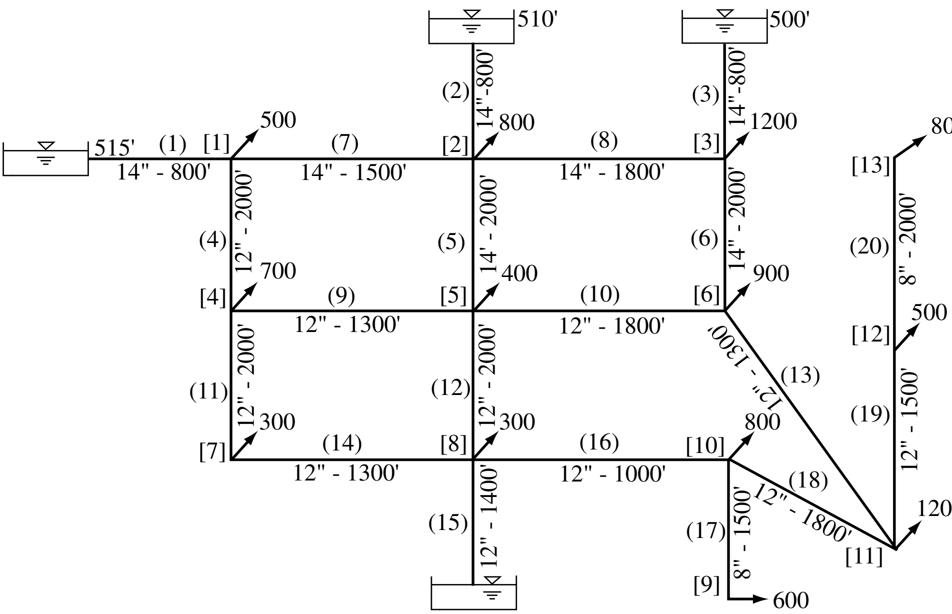
The opening of this valve at $t = 0$ produces a loss coefficient $K_{14} = 150$. The valve in pipe 16 operates on the following rule that depends on the pressure head at node 9:

Head at node 9	70	80	90	100
x, degrees	15.71	40.15	56.29	15.71

Its opening at t = 0 produces a loss coefficient $K_{16} = 24.6$.

Reservoir Storage Function:

For pipes 1, 2, and 3		For pipe 15	
Water surface ft	Volume acre feet	Water surface ft	Volume acre feet
490	0	420	0
510	40	440	15
530	80	460	30



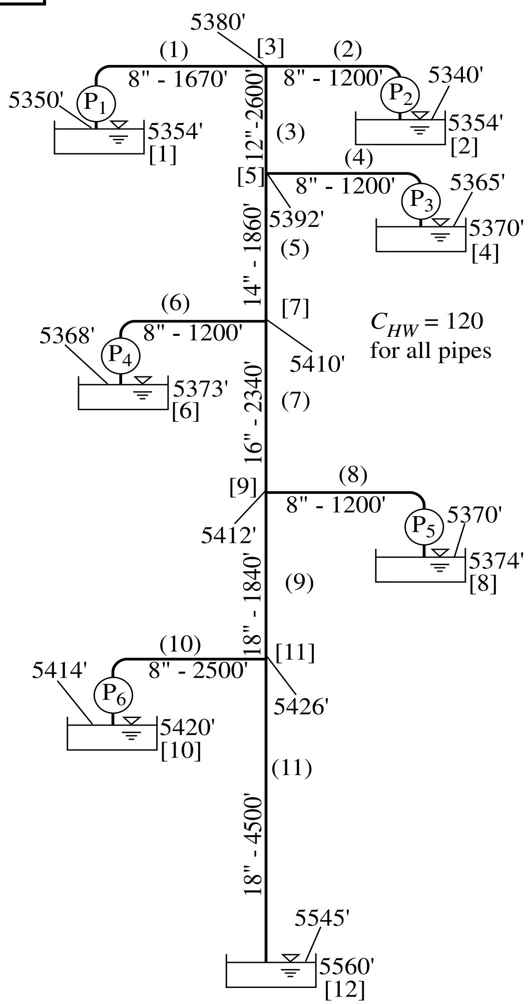
All demands in gal/min

6.8 Water at a rate $Q = 0.045$ m^3/s is to be pumped continuously from a well that has a water table that is 30 feet below the pump to a reservoir that has an average water surface elevation that is 100 ft higher than the pump. The capital investment for the pump and well is \$150,000; electrical energy costs \$0.09/kWh, and the life of the system is estimated to be 50 years. The length of pipe between the well and reservoir is 1200 m, and the cost of pipe for a selection of sizes is as follows: 150 mm pipe costs \$60/m; 205 mm pipe costs \$80/m; 255 mm pipe costs \$110/m; 305 mm pipe costs \$150/m; 375 mm pipe costs \$200/m. The interest rate is 10 percent. Select the most economical pipe size to use. Assume $e = 0.15$ mm.

6.9 The pumps which supply water to the reservoir of the Colorado Springs system are shown below during a period when no demands occur. For this operation determine the amount of energy dissipated by fluid friction, and how much energy is supplied by the pumps per hour of operation. If the motor-pump's combined efficiency, on average, is 70 percent, and energy costs $0.12/kWh, what is the daily electric bill for each of these units? What is the cost per acre foot of water supplied to the reservoir? If the life of the system is 35 years and the interest rate is 10 percent, what is the equivalent capital recovery cost of each of these items?

All pumps are identical with the following characteristics:

Q gal/min	h_p ft
400	351
600	285
700	234



6.10 A 13,100 ft long 12-inch diameter pipe line is anticipated to have a life of 60 years. If the pipe will cost $200 per foot to install, what annual benefit must the pipe produce to be economically viable? The interest rate for money is 10 percent.

6.11 Assume that the combined motor-pump efficiencies in Problem 6.9 vary with the discharge through the pumps according to the values in this table:

Q, gal/min	400	600	700
Efficiency	0.68	0.77	0.60

Recompute all the costs that were requested in Problem 6.9 by using linear interpolation and the data in this table.

6.12 Rework Problem 6.9 with (a) all pipes reduced in size by one inch, and (b) all pipes enlarged by one inch. Then compare these costs with the costs that were determined in Problem 6.9.

6.13 In Problem 6.9 all pipes were assumed to be steel with a Hazen-Williams coefficient $C_{HW} = 120$. If all pipes were made of PVC with an equivalent sand grain roughness $e = 0.000008$ in (with unchanged inside diameters that are the standard pipe sizes), again determine all of the quantities that were requested in Problem 6.9.

6.14 For the Colorado Springs network of Problem 6.9 assume there is a demand of 1000 gal/min at each of the tee intersections of the pipes, i.e. nodes 3, 5, 7, 9, and 11. Now what are the quantities requested in Problem 6.9?

6.15 A set of pump curves (not shown) describes the operating characteristics of a pump which is to supply a discharge Q (in gal/min) that varies in time according to

$$Q = 140 + 400\,sin(\pi t/24)$$

in which the time t is in hours. Accounting for the temporal variation in efficiency and head with discharge, determine the energy consumed by the pump during one day's operation. Either a cubic spline function or a second-order polynomial may be used to interpolate the variables. The tables below provide data which has been extracted from the original pump curves. If the efficiency of the electric motor that drives the pump is constant at 85 percent and electrical energy costs $0.10/kWh, what is the cost per year for electrical power to operate the pump, assuming it operates 365 days per year? The pump's life expectancy is 30 years, and interest is 8 percent. What is the equivalent capital recovery cost for this electrical energy?

Q, gal/min	140	152	188	224	260	280	308
Efficiency	0.58	0.60	0.65	0.70	0.73	0.75	0.78

Q, gal/min	352	400	428	456	480	505	534
Efficiency	0.80	0.78	0.75	0.73	0.70	0.65	0.60

Q, gal/min	140	260	340	420	540
h_p, ft	38.5	35.5	30.5	24.5	12.0