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Fielding Ditch Pipeline Computer Simulation Study

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FIELDING DITCH PIPELINE COMPUTER SIMULATION STUDY

Prepared for the

Soil Conservation Service

by

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ABSTRACT

The Fielding Ditch Company Pipeline is almost 3 miles long and supplies irrigation water under low pressure to adjacent fields through 33 turnouts along its length. The 24-inch nonreinforced concrete pipeline began to experience repeated structural failures soon after it was placed in operation. This study was done for the Soil Conservation Service by the Utah Water Research Laboratory to gather field data on the pipeline operating characteristics, to analyze the hydraulic transients in the pipeline with the help of a computer simulation model, and to suggest modifications to protect the pipeline from future failures caused by transient pressures.

Following a description of the pipeline system, the concepts and principles of unsteady flow in pipelines are summarized. Then the general equations for transient flow are presented followed by a summary of their solution using numerical methods. Under the field verification data collection program, instruments and recorders were set up at four locations along the pipeline. Pressure and flow measurements during both steady and unsteady flows were recorded to obtain data on the operating characteristics of the pipeline. These field data as well as preliminary analysis indicate that moderate closure times of valves could generate pressure waves which could overstress the nonreinforced concrete pipe. The field data also provided a way to verify that the computer simulation model could truly represent the behavior of the actual pipeline system. The field data also showed the pressure wave speed to be about 1170 feet per second rather than the 3640 feet per second predicted by the wave speed equations. This significant change in wave speed was attributed to the effect of free air trapped in pipe joints and high spots in the pipeline.

Seven increasingly complex computer models were developed to represent the pipeline. The first was a simple basic water hammer program for a pipe with a reservoir upstream and a valve at the downstream end which could close instantly. Later programs added the effects of air pockets along the pipeline, damping or dissipation at the air pockets, gradual closure of the downstream valve, gradual closure of a valve at an interior point, simultaneous closure of two valves and provision for protective standpipes at any or all interior valve locations. Comparison of the final programs with field data showed the system to be adequately represented.

The computer programs were then used to compare the effectiveness of various proposed protective modifications to the pipeline. Modifications considered but not recommended included requiring a longer valve closure time (not fail safe), installation of pressure relief valves (not reliable), and installation of air chambers at each valve (not economical). The recommended pipeline modification was to install eleven 18-inch pressure relief standpipes at selected interior turnout locations and one 36-inch standpipe at the downstream end. The study showed that this would provide the required pressure surge protection while limiting the spillage at the standpipes to an acceptable amount. Smaller 2- or 3-inch diameter standpipes should also be installed at all other turnouts to release trapped air and to serve as indicators (to nearby valve operators) of too rapid closure of the valves.

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CHAPTER 1

INTRODUCTION

Almost immediately after being put into operation, the Fielding Ditch Company Pipeline began to experience repeated structural failures. The 24-inch nonreinforced concrete pipeline is almost 3 miles long and supplies irrigation water under low pressure to a number of farmers through turnouts along its length. The supply water is taken from a canal and enters the pipeline through a head box. At each turnout, the water flows upward through a riser from the buried pipe into a distribution box. A butterfly valve in the riser, as it enters the distribution box, is used to control the flow. Two inline valves are located in the pipeline, one at its downstream end and one near its midpoint.

Because of the repeated failures of this pipeline, the Soil Conservation Service, which had constructed the pipeline and were responsible to make sure that it was structurally sound before turning it over to the local farmers, entered into a cooperative agreement with Utah Water Research Laboratory to analyze the hydraulic transients that might arise in the normal operation of this pipeline. The results are given in this report.

The study began with a program of measurements to determine the characteristics of hydraulic transients in the pipeline. These field data provide the basis for the analysis and a means of verifying the computer model. This report continues with a description of the pipeline, a discussion of the principles and concepts of unsteady flow, the methods of solving the equations of flow, the field verfication data collection program, the various computer models that were developed during the study, and the suggested modifications to protect the pipeline from further damage.

CHAPTER 2

DESCRIPTION OF THE FIELDING DITCH PIPELINE SYSTEM

The Fielding Ditch Company Pipeline is mainly a 24-inch inside diameter non-reinforced concrete pipe that delivers irrigation water through 33 turnouts. A diversion box directs the water from the canal through a Parshall measuring flume into the pipeline. Each turnout consists of a tee connection with a vertical riser, a 90° elbow, and a butterfly valve, and each discharges into a distribution box. Two of the turnout valves are 8-inch, 4 are 12-inch, and the 27 others are 18-inch as shown in Table 2.1.

The butterfly valves are opened and closed by means of a hand wheel that operates the valve through a gear box. The gear boxes require 12 1/2 turns of the hand wheel to turn the valve from fully closed to fully open. Two butterfly valves are located in the line--an 18-inch valve at the downstream end (station 154+53.8) and a 24-inch valve at about the midpoint of the pipeline (station 73+25.9). Combination air/vacuum relief valves are located at station 19+03, on each side of the inline valve at station 73+25.9, and on each side of the end inline valve at station 154+53.8. At the downstream end of the pipeline an 18-inch drain line goes from the inline valve to a nearby channel where the excess water is released. In late summer 1980, some small diameter pressure relief valves, pressure gages, and 1/4-inch air bleed stopcocks were installed at some of the turnouts, especially near the lower end to remove from the risers any accumulated air.

During the program of field data collection for verification of the transients model, pressure recording devices were installed at the four stations shown in Table 2.1. A pressure transducer measured the pressure in the pipeline, and the measurements were recorded on a paper chart at each recorder station.

A plan view of the system with its essential features appears as Figure 2.1 and shows the pipe bends as well as the turnouts already mentioned and the proposed standpipes. A profile of the pipeline is shown in Figure 2.2. A typical riser and box are shown in Figure 2.3.

Table 2.1. Fielding pipeline turnout locations.

Node	Station (Feet)	Comment	Turnout Size (Inches)
1	1+34.0	Pipeline Start	
2	19+03.0		18
3	23+63.0		12
4	28+01.5		12
5	32+18.0		18
6	35+63.0	Recorder 1	18
7	45+54.4		18
8	50+18.4		18
9	54+74.4		8
10	58+73.9		8
11	65+04.9		18
12	66+80.9		12
13	66+88.9		18
14	69+48.9		18
15	72+27.9		12
16	73+15.9	Recorder 2	18
	73+25.9	Inline Valve	24
17	77+79.4		18
18	81+86.9		18
19	85+62.3		18
20	90+67.0		18
21	93+07.0		18
22	99+26.5		18
23	101+62.9		18
24	104+66.9		18
25	107+45.9		18
26	110+66.7		18
27	114+10.2	Recorder 3	18
28	120+89.7		18
29	128+15.2		18
30	133+89.8		18
31	142+92.8		18
32	147+40.5		18
33	149+88.8		18
34	154+44.5	Recorder 4	18
	154+53.8	Inline Valve	18
		Drain Line	18

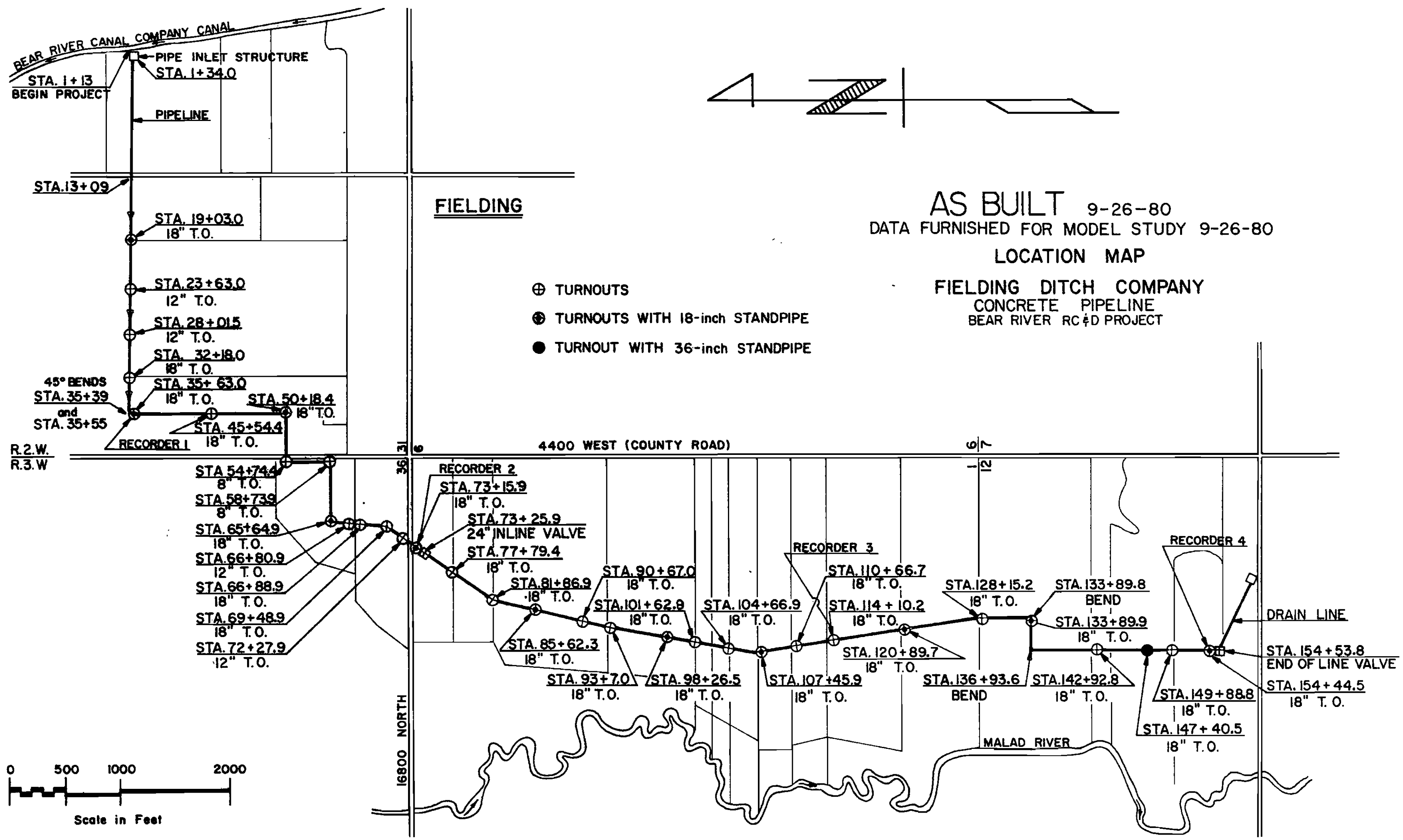

Figure 2.1. Location map.

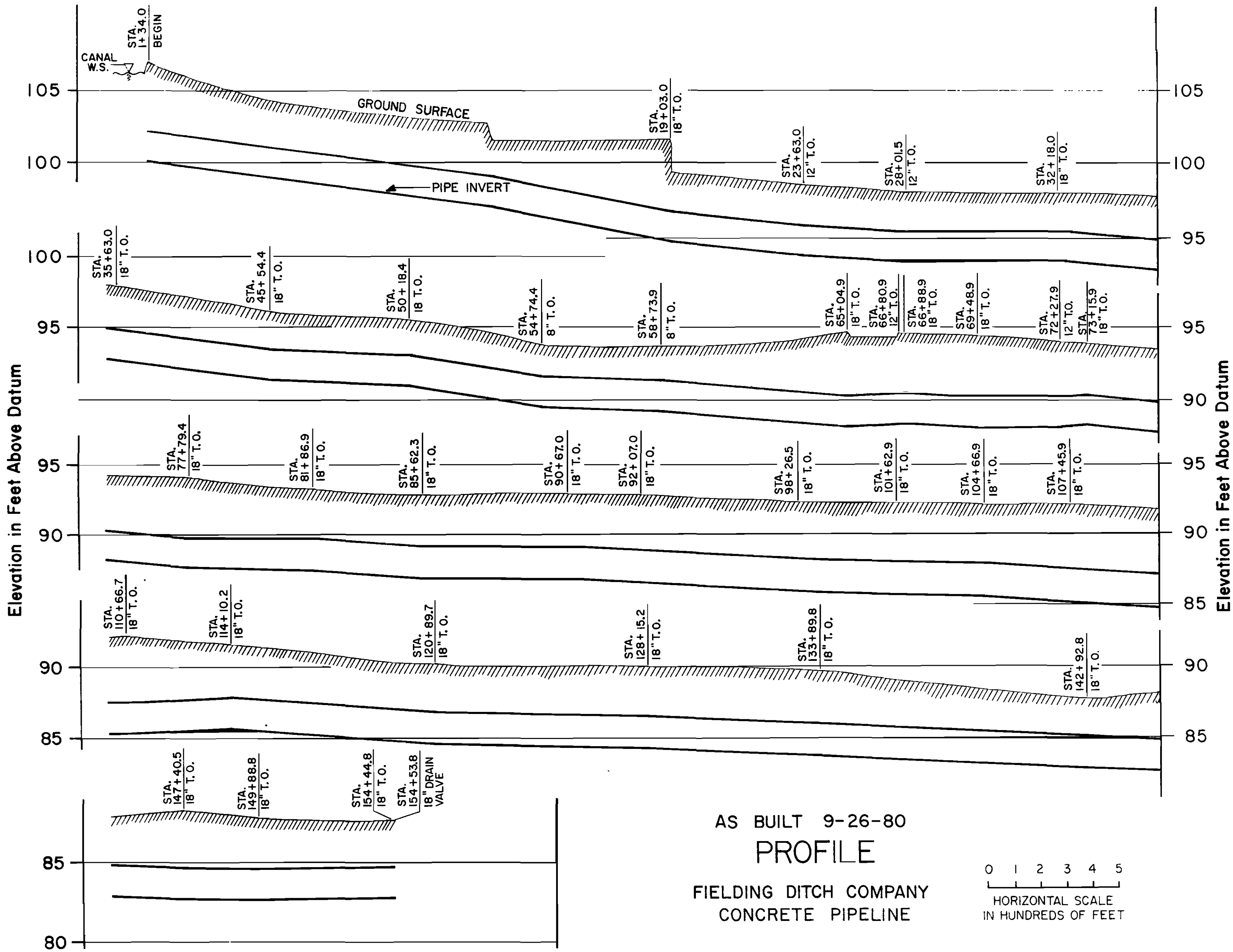

Figure 2.2. Profile of Fielding Ditch Company Pipeline.

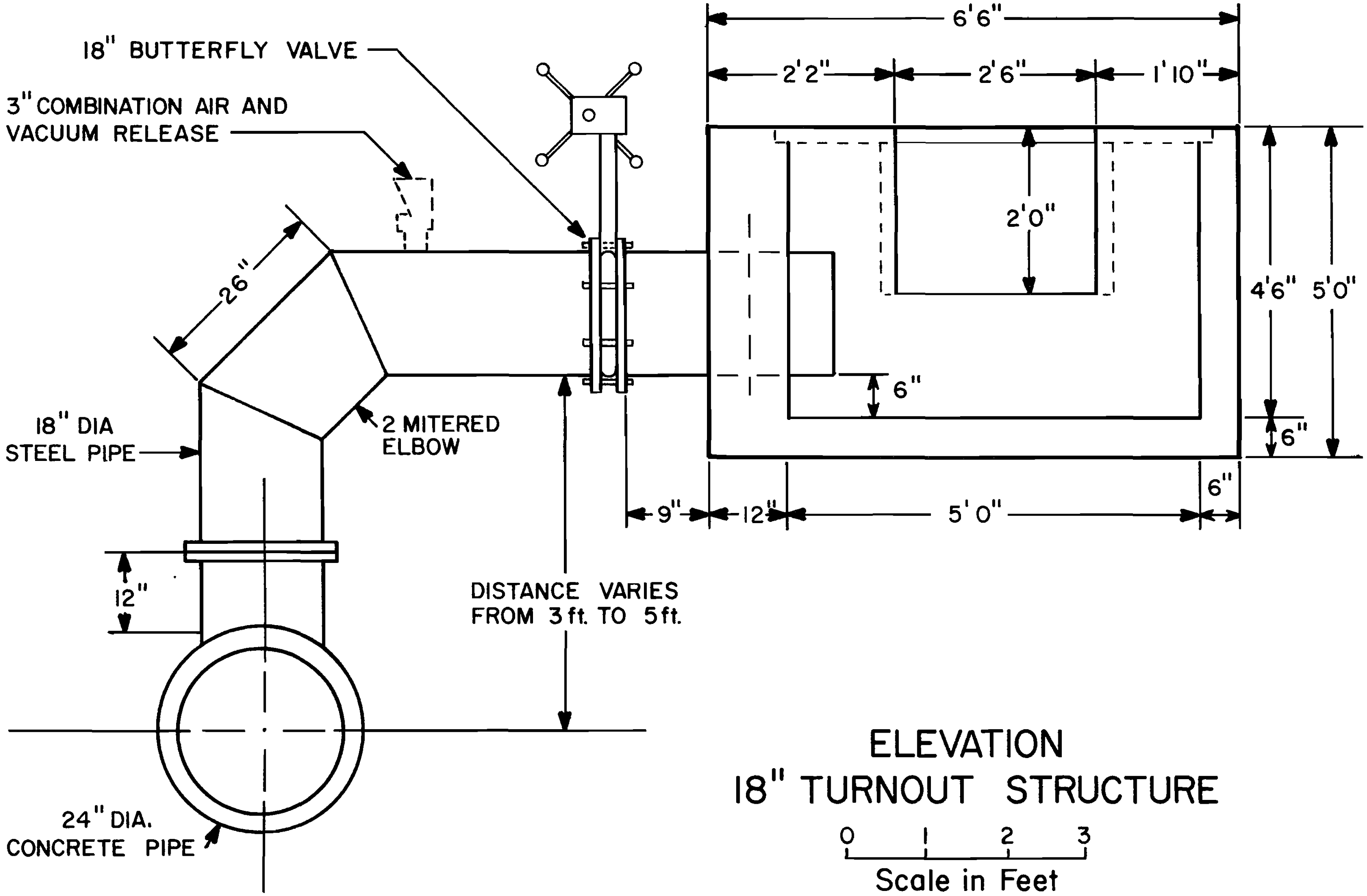

Figure 2.3. Typical turnout structure.

CHAPTER 3

CONCEPTS AND BASIC PRINCIPLES ASSOCIATED WITH

UNSTEADY FLOW IN PIPELINES

Concepts

Most books on hydraulics or fluid mechanics provide little information on unsteady flow in piping systems unless the book is specifically devoted to the subject of unsteady flow. As a result, in practice, steady flow hydraulics is often the only basis for design and little attention is given to the possible effects of unsteady flow. Yet truly steady state flow rarely exists. Why then are most hydraulic designs based on principles of steady flow rather than on principles of unsteady flow?

First, some pipeline systems operate under near steady-state conditions virtually all the time and, consequently, the unsteady occurrences are of little consequence since the magnitude of pressure changes, etc., due to time dependent parameters are minor. Secondly, the variety of unsteady occurrences are many and diverse and often not easily defined and, consequently, they are assumed to be covered in the design by safety factors for capacity and against failure. Thirdly, unsteady flow is complicated mathematically. It is defined by differential equations, rather than the simpler algebraic equations generally used for steady flow. Many design engineers do not feel as comfortable in using differential equations as they do in using algebraic equations. However, some pipeline systems develop adverse hydraulic conditions of pressures and/or flow rates as the result of unsteady occurrences. The hydraulic ram is an example of utilizing the increased pressure caused by unsteady flow to lift or increase the pressure of some of the flow without an external source of power. Whenever large flows are shut-off, restarted, or redirected repeatedly, especially if done rapidly and if the pipeline is long, the unsteady occurrences are very significant. The operation and conditions of the Fielding pipeline place it in the category of hydraulic systems governed by possible unsteady flow effects even though it is a low pressure pipeline.

Unsteady flow occurrences may be categorized according to whether or not elastic effects are important. Long pipelines in which the entire flow can be shut off quickly have the greatest elastic effects and are most likely to experience very high pressures. If standpipes are installed to prevent the development of high pressures, persistent repeating surges may occur because of the small velocities and the large mass of water that exists in a long pipeline which cause any change from equilibrium conditions to persist for some time. Theoretically a non-viscous fluid would oscillate forever in a U-tube if one column of the fluid were initially displaced from the equilibrium position and then released. With a real fluid, the frictional losses reduce the magnitude of the oscillation and eventually make it stop.

The high pressures related to the elastic effects are of concern in the Fielding pipeline because the resulting pressure can break the pipes. However, the tendency for any unsteady occurrence at any location to propagate throughout the pipeline and persist for some time needs to be considered. A brief analysis of such possible "sloshing" is included in Appendix D of this report.

Basic Principles of Hydraulic Transients Based on Elastic Theory

In this section a brief discussion of hydraulic transients and elastic theory is given as background information. To illustrate hydraulic transient theory, assume the downstream inline valve of the Fielding pipeline is instantly closed. Prior to this closure, consider the pipeline to be full, all turnouts closed and a steady velocity of 2 fps (Q = 6.3 cfs) to exist throughout the pipeline. The closure stops the flow at the downstream end of the pipe, but water continues to move for a brief time at 2 fps velocity in the upstream portion. In stopping this forward inertia, the water is compressed, the pressure increased, and the pipe diameter expanded. The increase in pressure head is given approximately by,

$$\frac{\Delta P}{\gamma} = \Delta H = \frac{a}{g} \Delta V \quad . \quad . \quad . \quad . \quad . \quad . \quad (3.1)$$

in which a is the wave velocity at which the elastic wave propagates, g is the acceleration of gravity ($g = 32.2$ fps^2) and ΔV is the change in velocity, i.e. $\Delta V = 2$ fps for the example. Figure 3.1(b) shows the situation at a time when the pressure wave has traveled one-fifth of the distance to the entrance. The wave speed, a, depends upon the compressibility of the fluid (i.e. its bulk modulus), the density of the fluid, the elasticity of the pipe, and the restraint against expansion of the pipe in its axial direction as it expands circumferentially. The equation giving this speed is

$$a = \left[\frac{K_m/\rho_m}{1 + \frac{K_m}{E}\frac{D}{e}C}\right]^{1/2} \quad . \quad . \quad . \quad . \quad . \quad (3.2)$$

in which K_m and ρ_m are the bulk modulus and density of the fluid, respectively, (m indicates a mixture of air and water; see Appendix A for discussion of the large effects of a small fraction of free air on K_m) E is the modulus of elasticity of the pipe wall, D is the pipe diameter, e is the wall thickness of the pipe, and C is a coefficient whose magnitude depends upon the type of axial restraint on the pipe. For thick-walled pipes, such as the Fielding pipeline, if the earth surrounding the pipe is assumed to prevent any movement in the axial direction, C is given by

$$C = \frac{1}{1+\frac{e}{D}}\left[(1-\mu^2) + 2\,\frac{e}{D}\,(1+\mu)\,\left(1+\frac{e}{D}\right)\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3.3)$$

in which μ is Poissons ratio = 0.3, e = 3 inches and D = 24 inches, making C = 1.13. If in addition K_m is taken equal to 300,000 psi = 4.32×10^7 lb/ft^2 for water without air and E for the concrete pipe is taken as 4.0×10^6 psi = 63.6×10^7 lb/ft^2, then Equation 3.2 gives a wave speed of 3640 fps. Measurements of wave speed in the Fielding pipeline were approximately 1200 fps. The fact that the measured values are slower than that given by Equation 3.2 is believed to be attributable to air trapped in the bells of the pipe joints, i.e., K_m for the mixture of water and air is considerably smaller than 300,000 psi. After long periods of continual service, this air might be dissolved and, therefore, the design for the magnitude of pressure wave should be based on the greater theoretical wave speed. If the velocity in the pipe is changed by ΔV = 2 fps, the potential increase in pressure head computed using Equation 3.1 equals 226 feet or a pressure increase of 78 psi.

Referring again to Figure 3.1, the pressure surge propagates upstream arriving at the entrance in a time t = L/a, which equals 4.2 seconds for a = 3640 fps or 12.8 seconds for a = 1200 fps in the 15,300 ft long Fielding pipeline. As the pressure surge is moving upstream, the pressure at the valve continues to rise gradually as the frictional head loss of the ever expanding region of no flow is recovered. The amount of this additional pressure for the Fielding pipeline may be as much as 17.5 ft. When the pressure surge arrives at the entrance, a small reversal of flow occurs. As water from the pipeline begins flowing back into the upstream head box, the increased pressure in the pipeline is relieved as the decompression wave moves back through the pipeline. Sketch (e) in Figure 3.1 shows this decompression wave after it has moved back one-fifth of the pipeline length. After a time of 2 L/a, which equals 8.4 seconds or 25.6 seconds, depending on whether a = 3640 fps or 1200 fps is used, the decompression wave arrives back at the closed valve (sketch (f) of Figure 3.1). Now the valve prevents the negative velocity from continuing, provided column separation does not occur, and the head at the valve reduces by twice the pressure head increment ΔH. This reduction in pressure can cause pressures below atmospheric to occur. If this negative pressure approaches the vapor pressure of the water, the negative velocity continues filling the pipe with a vapor cavity. If the pressure is well above vapor pressure as assumed in sketch (g) of Figure 3.1, the resulting negative pressure propagates up the pipeline again with a velocity, a, shrinking the size of the pipe and expanding the fluid elastically. After a time 3 L/a (sketch (i) of Figure 3.1) the shrinking effect reaches the entrance, and the velocity throughout the pipeline is again at zero. The pressure in the pipe is now less than that of the upstream head box and, therefore, water is again drawn into the pipe with a velocity nearly equal to the beginning velocity of 2 fps. This elastic wave now propagates down the pipe for a second time with a speed of a, and arrives at the valve at time t = 4 L/a (sketch (k) of Figure 3.1). These conditions repeat themselves. The pressure changes with time at the valve, at the midpoint, and at the entrance to the pipe are shown in Figure 3.2. Over time, the energy absorbed by the fluid and the pipe cause the magnitude of the pressure surge to diminish from cycle to cycle until it eventually dies out.

The above idealized treatment of a hydraulic transient must be modified to duplicate a real situation and this is shown in a subsequent chapter. A real valve cannot be closed instantaneously. However, if a valve is closed in a time shorter than the time required for the pressure wave, which starts when the valve first reduces the flow rate, to travel to the entrance (i.e. reservoir) and return (a time less than 2 L/a), the pressure at the valve will still be incremented by an amount ΔH as computed by Equation 3.1. If a wave speed of a = 1200 fps is used for the Fielding pipeline, this partial closure time is t = 2 (L/a) = 2(15,300)/1200 = 25.5 seconds. Valve closure times longer than 2(L/a) will result in pressure surges with magnitude smaller than ΔH. However, even very slow rates of

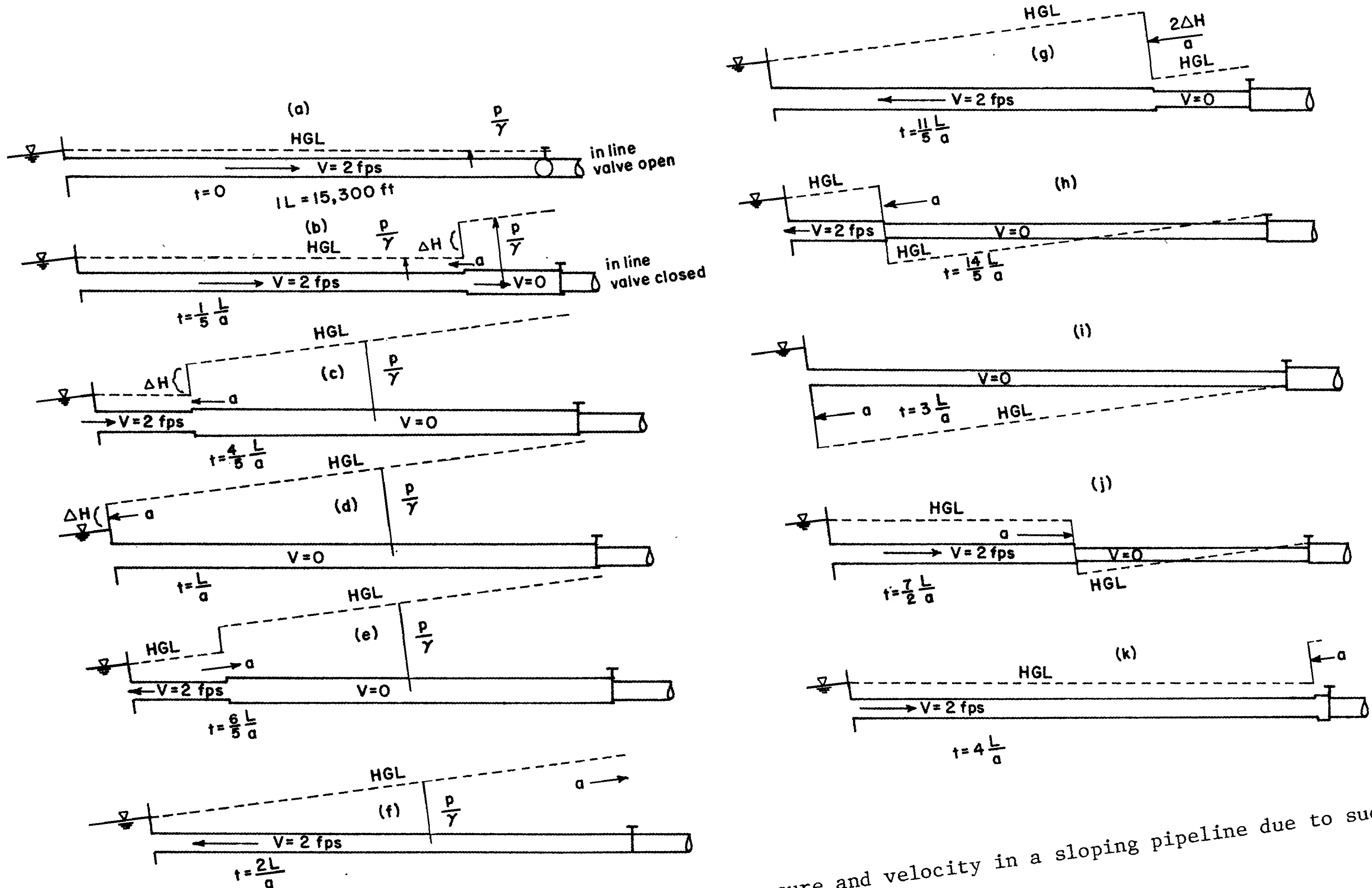

Figure 3.1. Sketches showing the time sequence of pressure and velocity in a sloping pipeline due to sudden valve closure.

valve closures must decelerate the fluid in order to eventually bring it to rest. This deceleration of the fluid is accompanied by a pressure rise. When this pressure rise is modest, such that little to no compression of the fluid occurs, it can be analyzed using the rigid water column theory discussed in Appendix D.

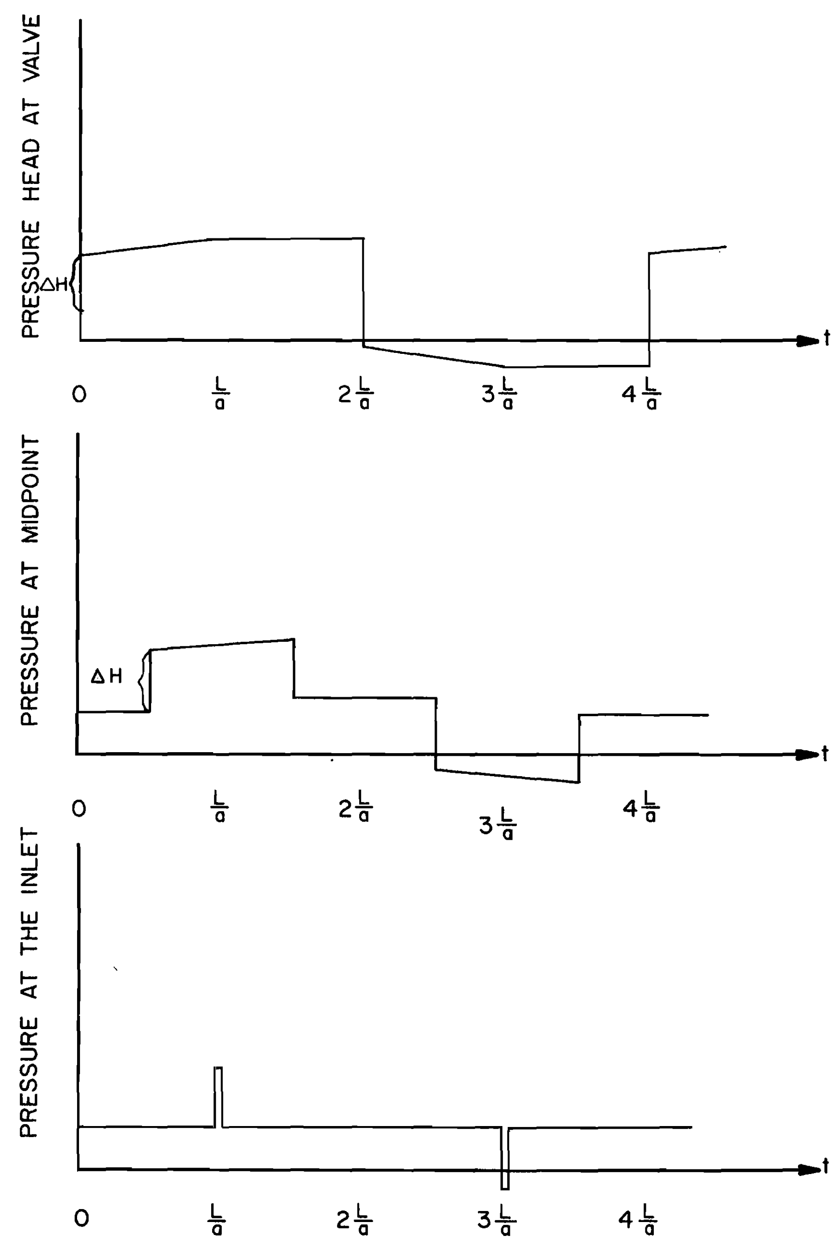

Figure 3.2. Time dependent pressure changes in the pipeline.

CHAPTER 4

SOLUTION OF THE GENERAL CASE OF TRANSIENT FLOW IN PIPELINES

To describe the transients and to predict the pressure increases at any selected point in a pipeline at any given time requires partial differential equations. Wylie and Streeter[1] present the derivation of the equations of motion and continuity shown below,

$$\frac{\partial V}{\partial t} + g\frac{\partial H}{\partial x} + \frac{fV|V|}{2D} = 0 \quad . \quad . \quad . \quad . \quad . \quad (4.1)$$

$$\frac{a^2}{g}\frac{\partial V}{\partial x} + \frac{\partial H}{\partial t} = 0 \quad . \quad . \quad . \quad . \quad . \quad (4.2)$$

where X = distance measured along the pipeline and other symbols have the meanings given earlier.

The maximum pressure one can get by solving these equations is the "water hammer" transient produced by instantaneous and complete valve closure as already discussed. The maximum pressure rise under such conditions was predicted to be 226 ft in the Fielding Ditch Company Pipeline.

Many methods for solving these equations have been proposed. At present the most general and exact method is called the method of characteristics. This method is amenable to numerical solution on a digital computer and for this reason the method of characteristics was chosen for application to the Fielding Ditch Company Pipeline investigation. Wylie and Streeter[1] discuss the technique in great detail, but a brief summary is given here.

If the assumption is made that the velocity of the pressure wave is constant and known, and the velocity of the water in the pipe is small compared to this wave velocity, then Equations 4.1 and 4.2 can be combined to give

$$\left.\begin{aligned} &\frac{g}{a}\frac{dH}{dt} + \frac{dV}{dt} + f\frac{V|V|}{D} = 0 \\ &\text{and} \quad \frac{dx}{dt} = a \end{aligned}\right\} \text{along } C^+ \text{ characteristic} \quad . \quad (4.3)$$

$$\left.\begin{aligned} &-\frac{g}{a}\frac{dH}{dt} + \frac{dV}{dt} + f\frac{V|V|}{2D} = 0 \\ &\text{and} \quad \frac{dx}{dt} = -a \end{aligned}\right\} \text{along } C^- \text{ characteristic} \quad . \quad (4.4)$$

in which dependent variable H is the piezometric head and dependent variable V is the average velocity of flow and both are functions of position x and time t. The auxiliary equations $dx/dt = \pm a$, define the characteristics along which Equations 4.3 and 4.4 apply respectively. Replacing the derivatives in Equations 4.3 and 4.4 by finite difference approximations and combining the results gives the equations that follow for advancing the flow velocity and hydraulic head respectively through one time increment.

The solution to a problem in liquid transients usually begins with steady-state conditions at time zero, so that H and Q are known initial values at each computing section. At any interior grid point, section i, the two compatibility equations (4.3 and 4.4) are solved simultaneously for the unknowns Q_{P_i} and H_{P_i}. In finite difference form Equations 4.3 and 4.4 may be written in a simple form, namely

$$C^+ : H_{P_i} = C_P - BQ_{P_i} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.5)$$

$$C^- : H_{P_i} = C_M + BQ_{P_i} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.6)$$

in which C_P and C_M are always known constants when the equations are applied:

$$C_P = H_{i-1} + BQ_{i-1} - RQ_{i-1}\,|Q_{i-1}| \quad . \quad . \quad . \quad (4.7)$$

$$C_M = H_{i+1} - BQ_{i+1} + RQ_{i+1}\,|Q_{i+1}| \quad . \quad . \quad . \quad (4.8)$$

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[1]Wylie, E. Benjamin, and Victor L. Streeter. 1978. "Fluid Transients." McGraw-Hill Inc.

$$B = a/gA \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.9)$$

$$R = f\Delta X/(2gDA^2) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.10)$$

By first eliminating Q_{P_i} in Equations 4.5 and 4.6

$$H_{P_i} = (C_P + C_M)/2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.11)$$

Then Q_{P_i} may be found directly from either Equations 4.5 or 4.6. It may be noted that section i refers to any grid intersection point in the x direction. Subscripted values of H and Q at each section are always available for the preceding time step, either as given initial conditions or as the results of a previous stage of the calculations. The new heads and flows at the current time during the transient have the letter P appended to the variables.

At the upstream and downstream ends of the pipeline, at surge tanks, at pressure relief valves, and at all other such devices, additional equations are needed for each device to make the mathematical solution duplicate the behavior of the actual system. To illustrate how such conditions are imposed, consider the programmed closure of a butterfly valve at the end of the pipe discharging into a small tank.

The head loss and discharge coefficients for a valve are defined as:

$$K = 2g\ \Delta H/V^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.12)$$

and

$$C_d = V/\sqrt{2g\ \Delta H} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.13)$$

These equations are related by:

$$K = 1/C_d^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.14)$$

With the valve at the end of the pipe there are three unknowns: V, H and C_d. The C+ characteristic Equation 4.5 provides one equation in H_P and Q_P. Equations 4.12 or 4.13 provide a second equation and the specified manner of closing the valve and the valve characteristics provide K or C_d at any time. With C_d known at any time, the following equations are sufficient to solve for the head (H_P) and discharge (Q_P) at the valve at any time.

$$H_P = C_P - BQ_P \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.15)$$

$$H_P = H_D + K\ Q_P^2/2g\ A^2 \quad . \quad . \quad . \quad . \quad . \quad (4.16)$$

where H_D is the piezometric head in the discharge tank. Simultaneous solution of these two equations produces H_P and Q_P.

The final step in solving the transient flow problem with a digital computer is to write a program which carries out the operations shown in Equations 4.5 to 4.11 to compute the velocity and pressure head conditions at all the desired locations and times for the pipeline. While general purpose programs have been written, the special conditions met in the Fielding pipeline were so numerous and complicated as to require special modifications to the basic program as each new condition was investigated. For example, the presence of air in the pipeline (in joints, high spots, risers, and entrained as bubbles) affects both the wave speed and the pressure intensity. These effects, as described later, were handled by modifications to the program to introduce the appropriate conditions at each computing section. A detailed discussion of the effect of free air on water hammer wave speeds is included in Appendix A.

CHAPTER 5

FIELD VERIFICATION DATA COLLECTION PROGRAM

An important part of any computer simulation study is verification of the model to be sure that it represents the behavior of the actual system. Validation of the Fielding Ditch Company Pipeline hydraulic transient simulation required that pressure and flow rate measurements be made on the actual pipeline while in operation to gather data for comparison with the computer simulation. A summary of the field tests to make these measurements is given in this chapter, and a more complete description appears in Appendix B.

Preparation for Tests

Preparation for the field tests included the following:

1. Installation of head gates and V-notch weirs in the outlet boxes at stations 73+15.9 and 154+44.5 so that outflow from the turnouts could be measured.

2. Installation of threaded connectors to the riser pipes at selected turnouts so that pressure gages and pressure transducers could be readily attached.

3. Installation of markers on the butterfly valve operators to indicate with greater precision the position of the valve leaf (open or closed).

4. Installation of chart recorders at four stations to record pipeline pressure as a function of time.

5. Installation of motor-generator sets for each recorder station.

6. Establishment of radio communication among the recorder stations.

Once the preparations were completed and the system had been filled with water, any accumulated air at the turnouts was removed by bleeding air from the small valves installed at many stations for this purpose or by bleeding through the turnout butterfly valves. The following field tests were then performed.

Steady State Tests

Two steady state tests were conducted for equipment calibration and to establish background conditions.

No Flow. With no flow in the pipeline (all valves closed) the static piezometric surface elevation was determined along the pipeline.

Steady Flows. With constant flow rates of approximately 3 cfs, 5.4 cfs, and 6.5 cfs the piezometric elevations were observed along the pipeline.

The steady flow tests are summarized in Table B.1, Appendix B.

Transient Tests

A number of transient condition tests were conducted to evaluate the pressure fluctuations in the pipeline. Some of the tests were for verification of the model and some were to establish operating and test procedures.

Suddenly Closing of Turnout Valve. A small flow rate was established through one turnout near the end of the pipeline and then the butterfly valve was suddenly closed to produce a pressure wave. Chart recorders were operated continuously during both the opening and closing of the valve. Pipeline flow velocities of 0.05, 0.10, 0.15, 0.20, and 0.39 fps were successively established in the tests to slowly approach the pressure limit of the pipeline. Turnouts at the middle and the end of the pipeline were used in turn while pressures were recorded at four stations.

Inline Valve Operation. A velocity of 0.20 fps was established in the pipeline by opening the turnout at the lower end. The midpoint inline valve (24-inch) was suddenly closed and the pressure fluctuations in the pipeline were recorded.

Air Generated Transients. A turnout riser was filled with air, and the air was released by suddenly opening the valve. The transient so generated was recorded.

Cushioned Transient. With a turnout riser completely filled with air, the adjacent next downstream turnout was opened to develop a 0.20 fps flow velocity in the pipeline. The valve was then closed and the pressure wave recorded.

Typical Irrigation Operation. All the recorders were in operation during a "typical" irrigation operation. The turnout at the end of the line was opened slowly until a flow of about 6 cfs was delivered with a pipeline velocity of 1.91 fps. After the opening transient died out, the valve was slowly closed in 2 minutes and the resulting transient was recorded.

The data from the tests were analyzed to determine wave speed, magnitude of pressure waves, etc., and the results are summarized in Tables B.2 and B.3, Appendix B.

CHAPTER 6

COMPUTER MODELS

Computer Program 1

The first computer program developed to analyze transients caused by valve closure consisted of a simple basic waterhammer program with the reservoir upstream and a valve at the downstream end which could close instantly. This program is based on the theory outlined in Chapter 3 and a listing of the program is found in Appendix E-1. The program starts with a steady state flow rate in the pipeline and assumes the flow is instantly stopped at the downstream end, thus generating a transient. The flow conditions for field test number 13 (0.2 fps flow velocity with instant valve closure) were used as the input conditions for this program. The computer output for these conditions is shown in Figure 6.1. The first wave speed used in the program was the computed wave speed for the pipeline, based on no free air in the system, of 3640 feet per second. The result is the taller square wave shown in Figure 6.1. When this is compared to the results of field test 13 (Appendix B-13) it is obvious that there is no similarity between the two. The computer program produces too high an amplitude, has a square wave instead of a sinusoidal wave, and has a period about one-third of that measured in the field.

The field test indicated that the measured wave speed in the pipeline was approximately 1170 feet per second rather than the calculated 3640 feet per second. The difference was attributed to the presence of free air entrapped in the pipeline. The air is trapped in the bells connecting the pipe sections and at high points in the pipeline. Program number 1 was rerun using the measured wave speed of 1170 feet per second. The results are also shown in Figure 6.1 as the curve identified by the triangle for run number 13. Comparison of this curve with the field test data indicates the magnitude and the period are approximately correct, but again a square wave is produced by the computer program while the measured wave form was more sinusoidal.

It became apparent that the free air in the pipeline has considerably more influence on the transient than is reflected by the reduction in wave speed. It was therefore necessary to develop a means of handling the influence of the air on the transient.

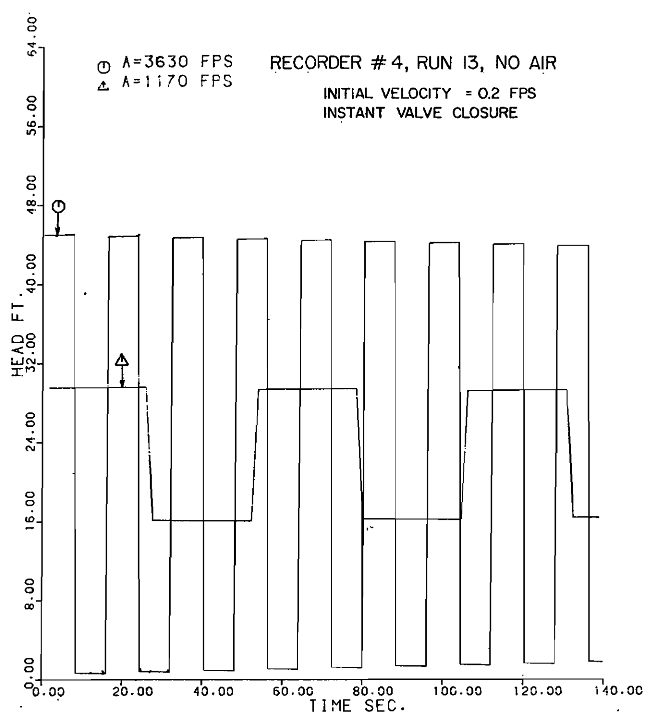

Figure 6.1. Output of program 1 showing basic water hammer waves.

Computer Program 2

The second computer program is based on the solution method of Chapter 4 and provides for instantaneous closure of a valve at the downstream end of the pipe. The program is listed in Appendix E-5. A major modification from computer program number 1 is that the air is handled as boundary conditions at the computing sections. With the method of characteristics, the pipeline is divided into a number of sections and the head and velocity are calculated at each of the sections. The air is handled by concentrating the air at the internal computing point and at the valve at the end of the pipe.

The equations used to express the relationship between the head, discharge, and air volume at each computing section are as follows:

1) The equation of state PV=MRT, V = air volume, M = mass of air.

2) The equation of continuity which accounts for the change in size of the air pocket.

$$\Delta \forall ol = 0.5 \; DT \; (Q_{in} - Q_{out}) \quad . \; . \; . \; . \; . \; . \; . \; (6.1)$$

where DT is the time interval between calculations, Q_{in} is the average flow upstream of the air bubble during the time step, and Q_{out} is the average flow downstream from the air bubble.

3) The C+ characteristic equation (see Equation 4.5).

4) The C- characteristic equation (see Equation 4.6).

These four equations must be solved simultaneously for the four unknowns. $\forall$ol, Q_{in}, Q_{out}, and Hp.

The equations for the boundary condition at the end of the pipe would be:

1) The equation of state, PV=MRT.

2) The continuity equation.

3) $Q_{out} = 0 \quad \Delta \forall ol = 0.5 \; DT \; Q_{in}$. . (6.2)

4) The C+ characteristic equation (Equation 4.5).

Equations 1, 2, and 4 are solved simultaneously for $\forall$ol, Q_{in}, and H_P.

The results of simulating experimental test 13 with computer program number 2 are displayed in Figure 6.2 which shows the head versus time relationships for the four test stations. Comparing Figure 6.2 with field test 13 (Appendix B-13 to B-16), one sees that the amplitude and the general shape of the curves are similar but that reflection from the air bubbles produces high frequency fluctuation on the main wave. Careful inspection also shows that there is not much attenuation of the wave with time. It was considered necessary to further modify the program to eliminate the high frequency variations in the curves and to increase the dissipation or attentuation of the pressure wave with time.

<u>Computer Program 3</u>

The third computer program (listed in Appendix E-5) also provides for an instant closure of the valve at the downstream end of the pipe with the air collected in air pockets. The major difference between programs 2 and 3 is that some extra dissipation of pressure at the air pockets is added to program 3. The dissipation was achieved by considering that the air is collected in an imaginary chamber installed on top of the pipe connected with a small hole in the pipe wall to allow water to flow into and out of the air chamber. Varying the size of the orifice opening in the computer program varies the amount of pressure dissipation at the air pocket. This modification adds two additional unknowns and therefore requires two additional equations to solve the boundary conditions. The equations used for the interior points are:

1) The equation of state, PV=MRT.

2) Equation of continuity which determines the volume of the air pocket (Equation 6.1).

3) The C+ characteristic Equation 4.5.

4) The C- characteristic Equation 4.6.

5) The equation for head loss across the orifice.

$$\Delta H_{orif} = \pm \; ZKA \; Q_T^2 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; (6.3)$$

where ZKA is the orifice coefficient and Q_T = flow rate into the air pocket.

6) Another continuity equation determining the flow rate in and out of the air pocket.

$$Q_U = Q_D + Q_T \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (6.4)$$

where Q_U and Q_D are the flow rates upstream and downstream from the air pocket.

Explicit solution of these equations was not possible. It was, therefore, necessary to use an iterative type solution to solve for the head and other unknowns at each computing section. The Newton-Raphson technique was used. This solution placed some limitations on the use of the program. It was no longer possible to simulate an instant valve closure for large initial flow rates because the rate of change of head with time caused the Newton-Raphson solution to fail to converge and give erroneous results. The method in program 3 can only handle instantaneous closures for very small initial flow rates where the change in head with time is kept small.

The equations for the boundary condition at the downstream end of the pipe are the same as those for the interior section except that Equation 4.6 for the C- characteristic is replaced with Q = 0.

Results of computer program number 3 for different values of dissipation at the orifices are shown in Figure 6.3 for the head at recorder 4 for run number 13. The data show that for a ZKA value (orifice loss coefficient) of 5 there are still a few ripples in the sinusoidal shape wave. As the

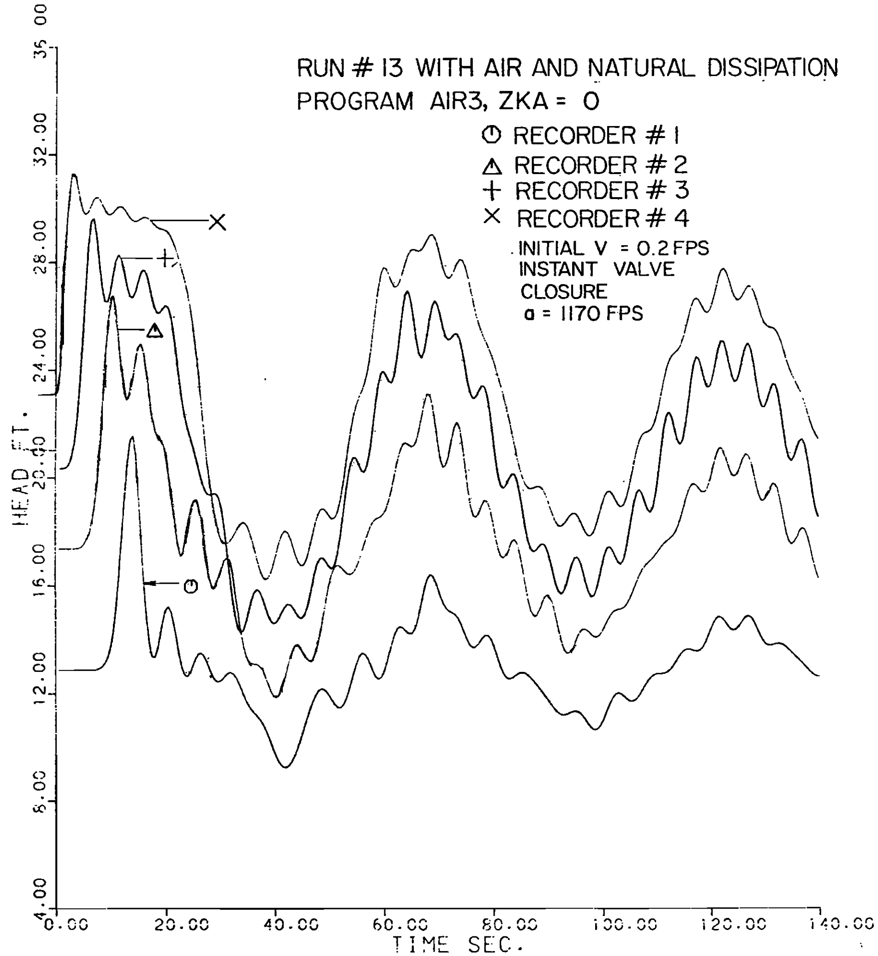

Figure 6.2. Output of program 2 showing transient pressure wave as modified by air in the pipeline.

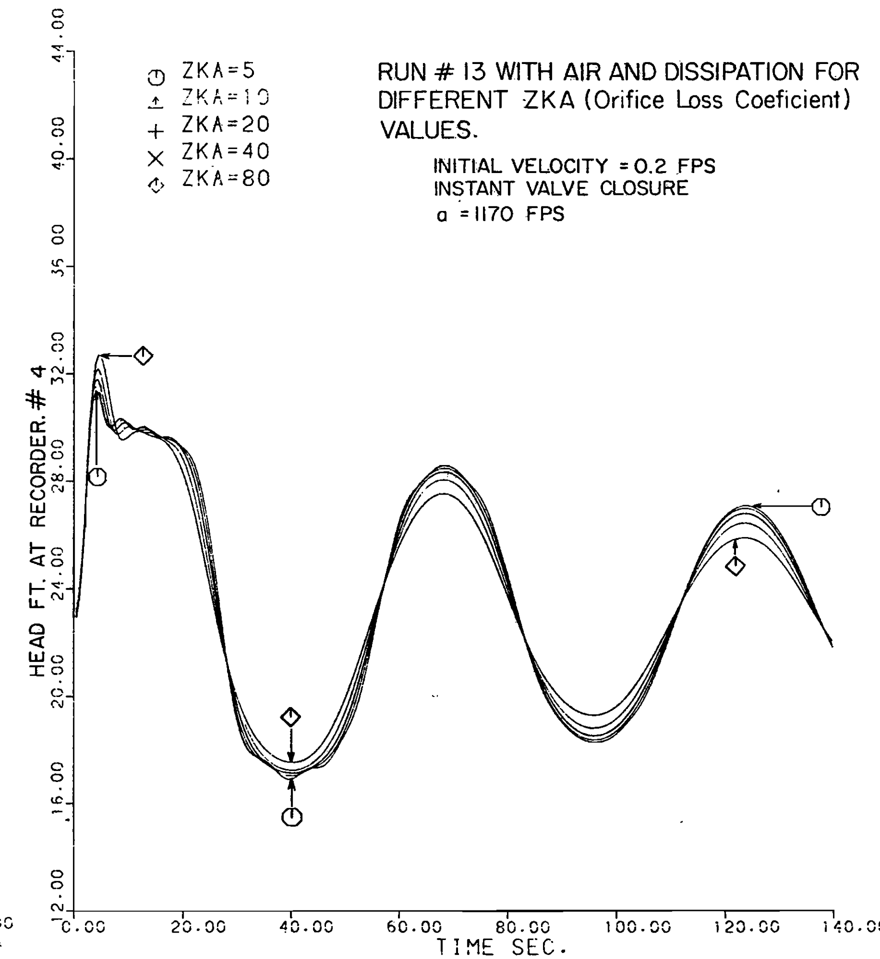

Figure 6.3. Output of program 3 showing the influence of pressure dissipation on the transient at recorder #4.

value of the loss coefficient is increased the dissipation is increased, the ripple becomes smaller and is eliminated. After comparing the output of the computer program with the field data for run number 13, it was decided to use a ZKA value of 80 for all subsequent calculations.

Figure 6.4 shows the head versus time data for all four stations for field test run number 13 and ZKA equal to 80. It is recalled that run 13 is an instant valve closure with an initial pipe velocity of approximately 0.2 feet per second. Comparison of Figure 6.4 with field test run number 13 (Appendix B-13 to B-16) shows good agreement between the computer model and the experimental data. This was the final revision of the program used to simulate instant valve closures at the end of the pipeline.

Computer Program 4

Computer program 4 was developed so that gradual valve closure at the downstream end of the pipe could be modeled. Program 4 is listed in Appendix E-5. The initial condition at the downstream end of the pipe consisted of a partially closed valve and an air pocket with no dissipation. Including dissipation at the first air pocket would have added to the complexity of the solution with little tangible benefits and was believed to be unnecessary. The four equations used to solve boundary condition at the valve are:

1) The equation of state PV=MRT.

2) The C+ characteristic equations (Equation 4.5).

3) The equation of continuity determining the size of the air pocket (Equation 6.1).

4) The equation for head loss across the valve (Equation 4.16).

Values for the discharge coefficient required in the equation for item 4 above were obtained from manufacturer's data for the butterfly valves used in the pipeline. Solution of the downstream boundary condition required use of the Newton-Raphson iterative technique as was required for the interior computing sections. Therefore, program number 4 may fail to converge to a proper solution if the transients are too severe. The way used to avoid the instabilities was to match the valve closure time with the number of computing sections selected. By increasing the number of computing sections, the pressure head rise between computations is reduced so the valve closure time can be reduced.

An example of the output of program number 4, simulating field test # 18, is shown in Figure 6.5. This is a 2-minute closure of the valve at recorder number 4 compared with the experimental data of field test run number 18 as shown in Appendix B-36 to B-39. The result of the computer program compilation as shown in Figure 6.5 showed higher initial transient pressures than the experimental data. The differences are likely due either to differences in the computed versus actual initial flow rate or to the difference between the computed rate of closure and the actual rate of closure in the field. The rate of closure of the valve for the last 10 or 20 degrees has a profound influence on the magnitude of the initial pressure surge. Variations of a few seconds closing time in that range could account for large differences in the computed versus the observed pressures. In any event, both the measured and the computed pressures for a 2-minute closure show excessive pressures in the pipeline.

Figure 6.6 shows the pressure at the four stations caused by closing the valve at recorder number 4 in 180 seconds. The computer maximum pressure at recorder number 4 was about 60 feet. This pressure wave is attenuated slowly as it travels up the pipe, and therefore the rapid closure subjects most of the pipe to a rather high pressure. The reason for the apparent decrease in pressure at recorders 3, 2, and 1 is due to the rise in elevation of the pipe with distance and hence the reduction in static head. The significance of this finding is that a transient generated at a valve can subject almost the entire pipeline to excessive pressures.

Additional information on the variation of pressure with closing time of the valve is shown in Figure 6.7. This figure shows the pressure at recorder 4 for four valve closing times between 120 and 300 seconds. Figure 6.8 shows the maximum head rise at each station in the pipe as a function of valve closing time at recorder 4. The shortest valve closure time for which the program was run was 30 seconds. This time was selected for two reasons: 1) the program becomes unstable for shorter closure times due to the instabilities of the Newton-Raphson solution, and 2) any closing time less than approximately 25 seconds is equivalent to an instantaneous valve closure and will produce approximately the same maximum head rise in the pipe. For closure times between 0 and 25 seconds, the maximum pressure head in the pipe is approximately 220 feet. As the closing time increases up to about 100 seconds, the maximum head rise decreases rapidly. Beyond 100 seconds, only slight decreases in the maximum head will occur as the closure time is increased. For a valve closure time in excess of 400 seconds, there is still a maximum head in the pipe greater than 40 feet.

Computer Program 5

Computer program 5 was developed by modifying program number 4 to calculate hydraulic transients should a valve be closed at an interior point. The program

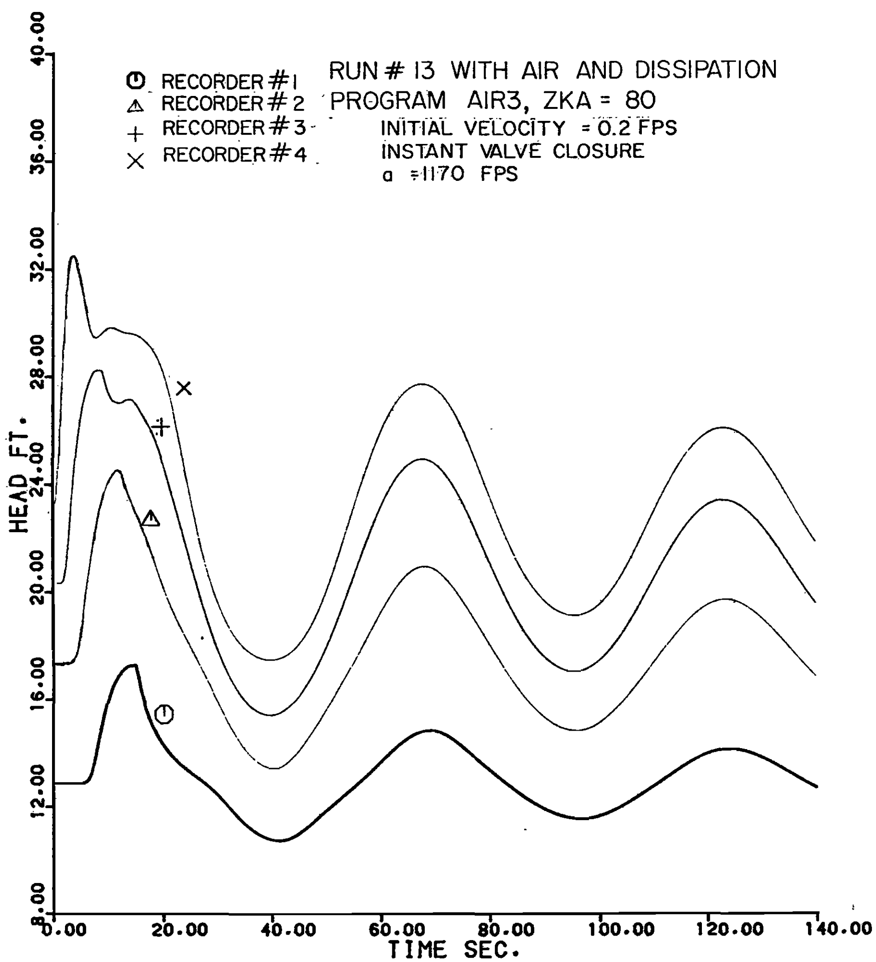

Figure 6.4. Output of program 3 for the conditions of field test #13.

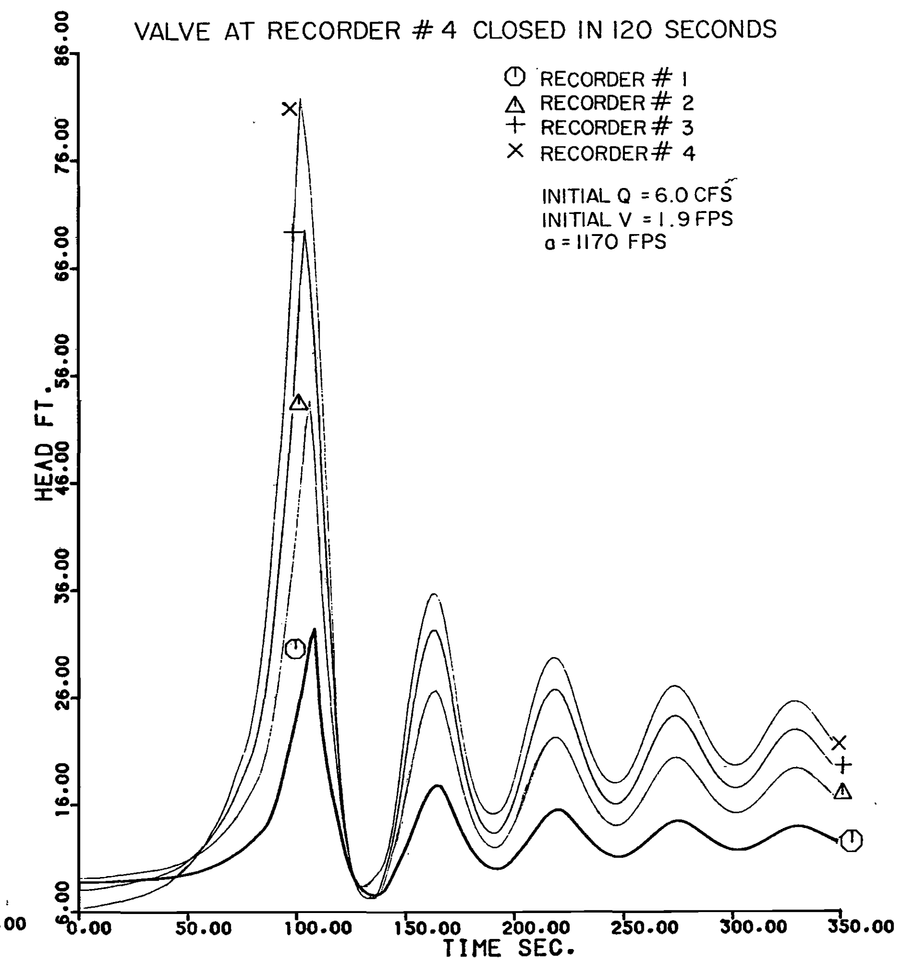

Figure 6.5. Transient produced by closing valve at recorder 4 in 120 seconds (program 4).

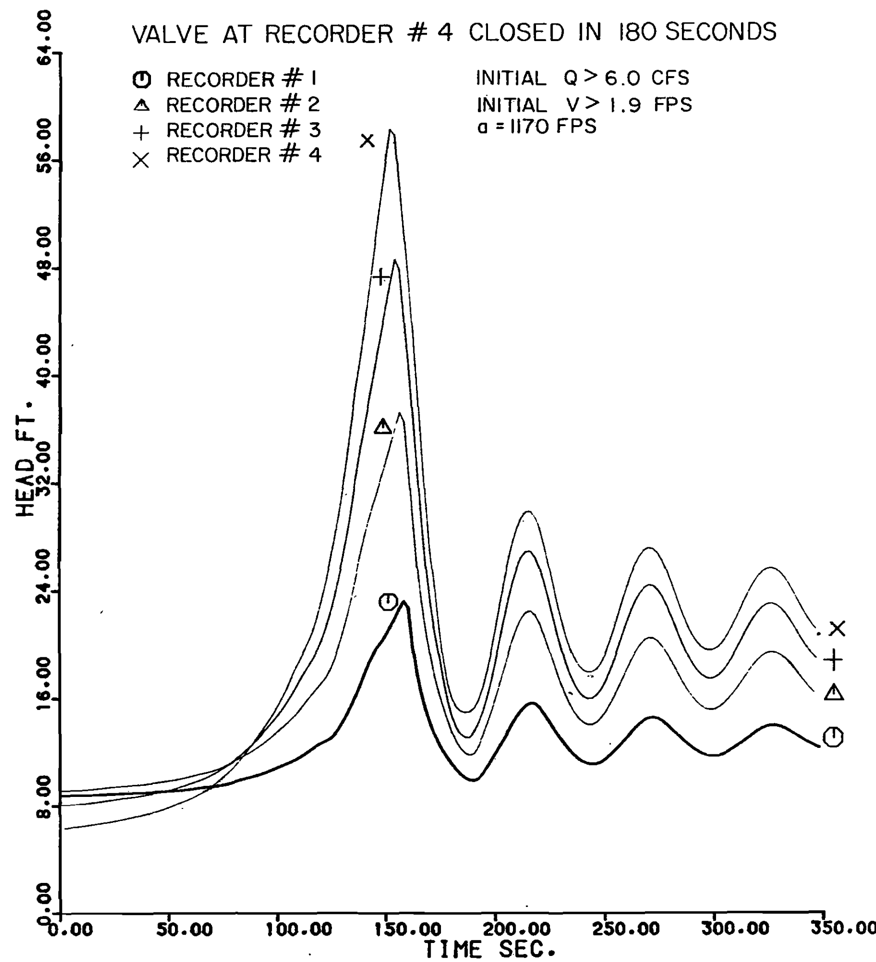

Figure 6.6. Transient produced by closing valve at recorder 4 in 180 seconds (program 4).

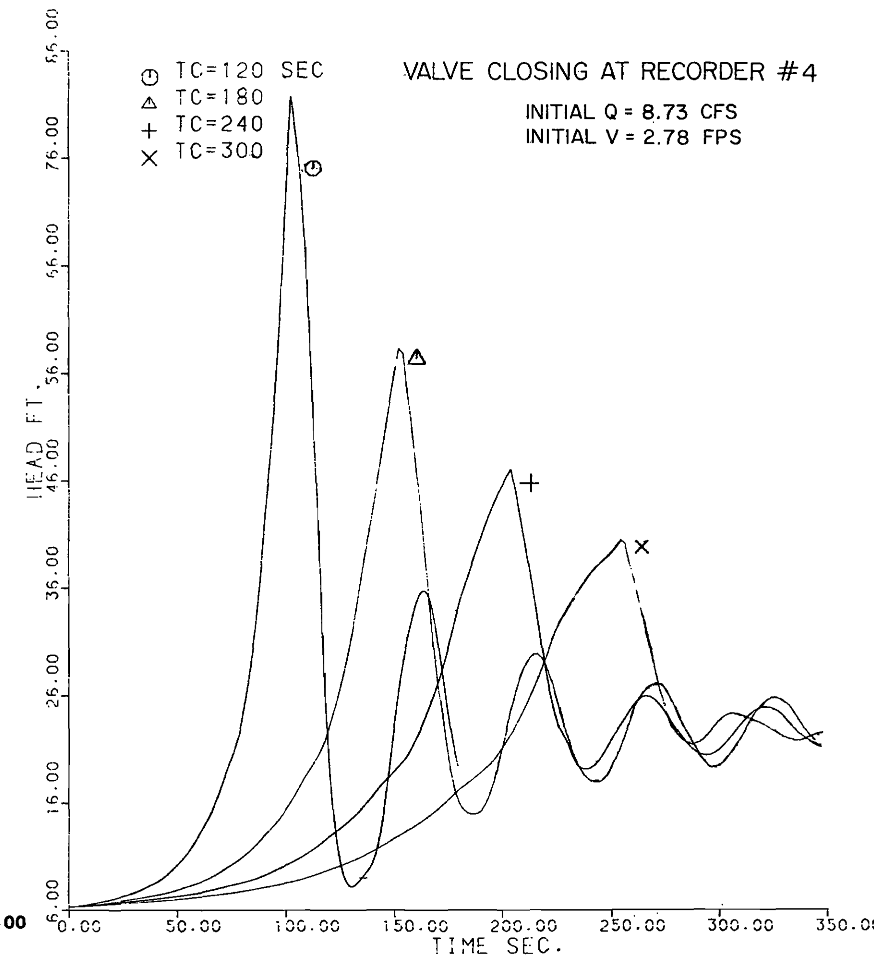

Figure 6.7. Transient produced by variable closing times of valve at recorder 4 (program 4).

listing is given in Appendix E-11. Figure 6.9 shows the calculated maximum head rise at recorder locations 1, 2, 3, and 4 for valve closure times at recorder 3 between 100 seconds and 420 seconds. The interesting point of Figure 6.9 is that even though the valve is closed at recorder 3, the maximum pressure occurs at recorder 4 at the lower end of the pipe. This is partially due to the greater static pressure at recorder 4, but the effect of the dead end pipe on a reflected wave is an added factor. A wave reflecting from a dead end doubles the pressure rise at the end.

This effect is also shown in the head rises for valve closure at recorder 2 in Figure 6.10. Once again it is seen that the maximum head occurs at recorder 4. The maximum head which occurs at recorder 4, however, is less when the valves at 2 or 3 are closed than it is when valve at recorder 4 is closed.

Computer Program 6

Programs 4 and 5 were then combined to calculate the head rise associated with simultaneous valve closure at an interior section and at the end of the pipe. The program listing is found in Appendix E-17. This program still maintained the air pocket and the pressure dissipation at the air pockets with ZKA = 80. Results of program number 6 for simulation of valves closing at recorder locations 2 and 4 are shown in Figure 6.11. The initial discharge for this case was 10.2 cfs. Figure 6.11 shows the head rise at the four locations for closing time between 120 and 420 seconds. Even for closing times of 420 seconds, the head rise at recorder 4 is still aproximately 40 feet.

Figures 6.12 and 6.13 show the head versus time curves for closing valves 2 and 4 simultaneously in 180 and 240 seconds respectively.

The preceding analysis shows that excessive pressure transients can be generated in the Fielding pipeline by closure of the valves. Even for valve closure times as long as 2 minutes, excessive pressures are generated throughout the pipeline. The analysis points out that it will be necessary to provide some pipeline protective features to eliminate the development of excessive pressure surges in the future.

Computer Program 7

This final version of the program was developed to incorporate protective standpipes into the system. The program is listed completely in Appendix E-24 and is also explained further in the appropriate section of the next chapter.

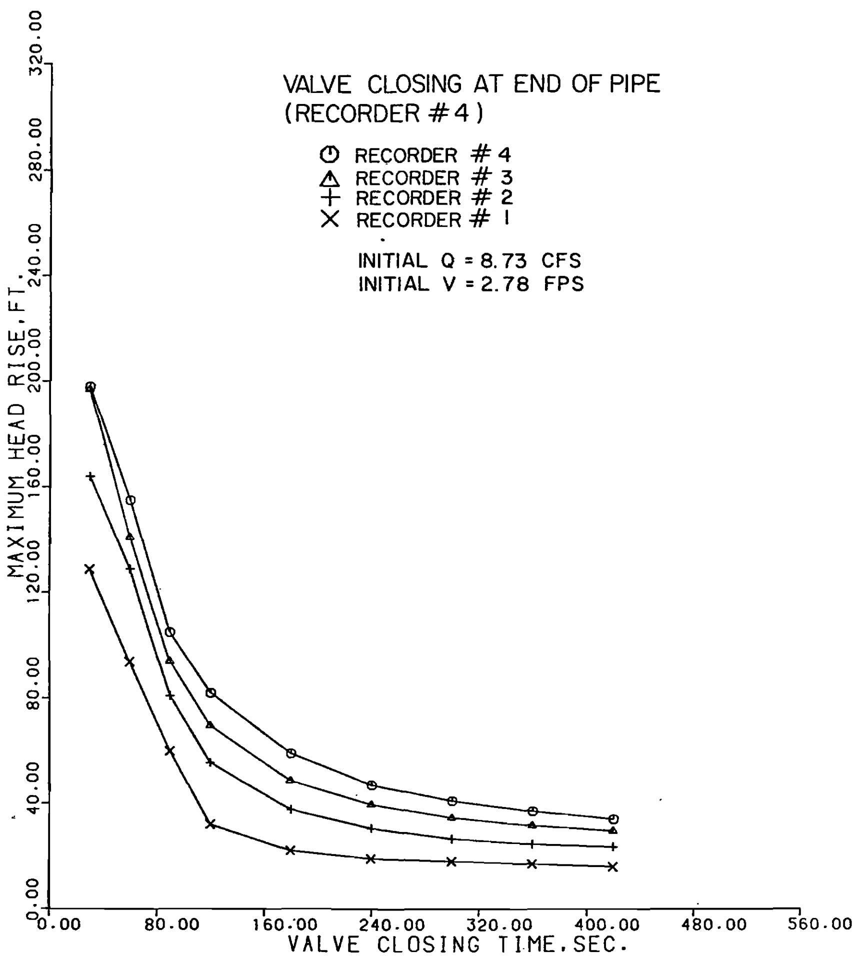

Figure 6.8. Maximum head in pipe for different closure times of valve at recorder #4 (program 4).

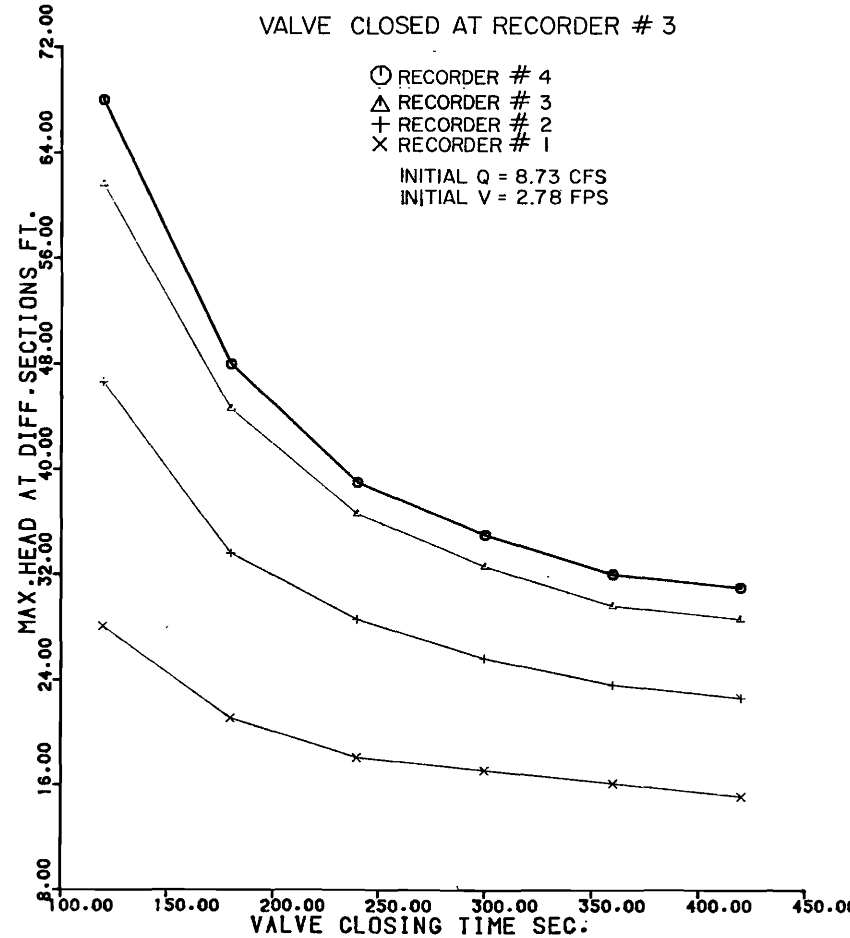

Figure 6.9. Maximum head in pipe for different closure times of valve at recorder #3 (program 5).

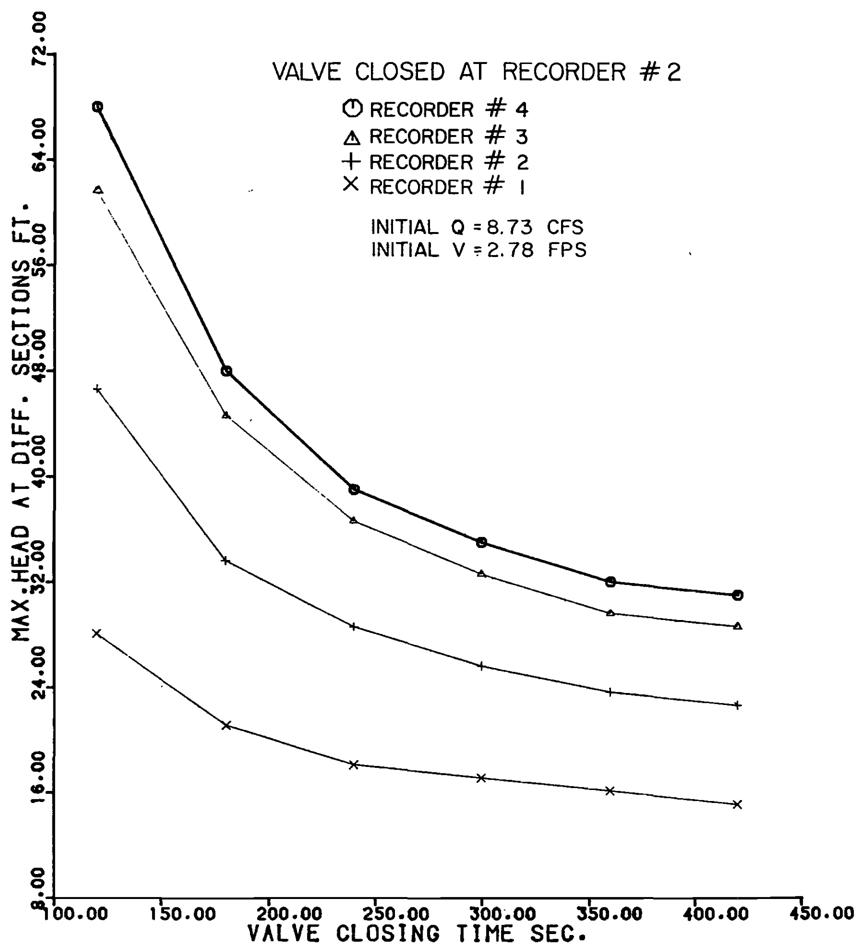

Figure 6.10. Maximum head in pipe for different closure times of valve at recorder #2 (program 5).

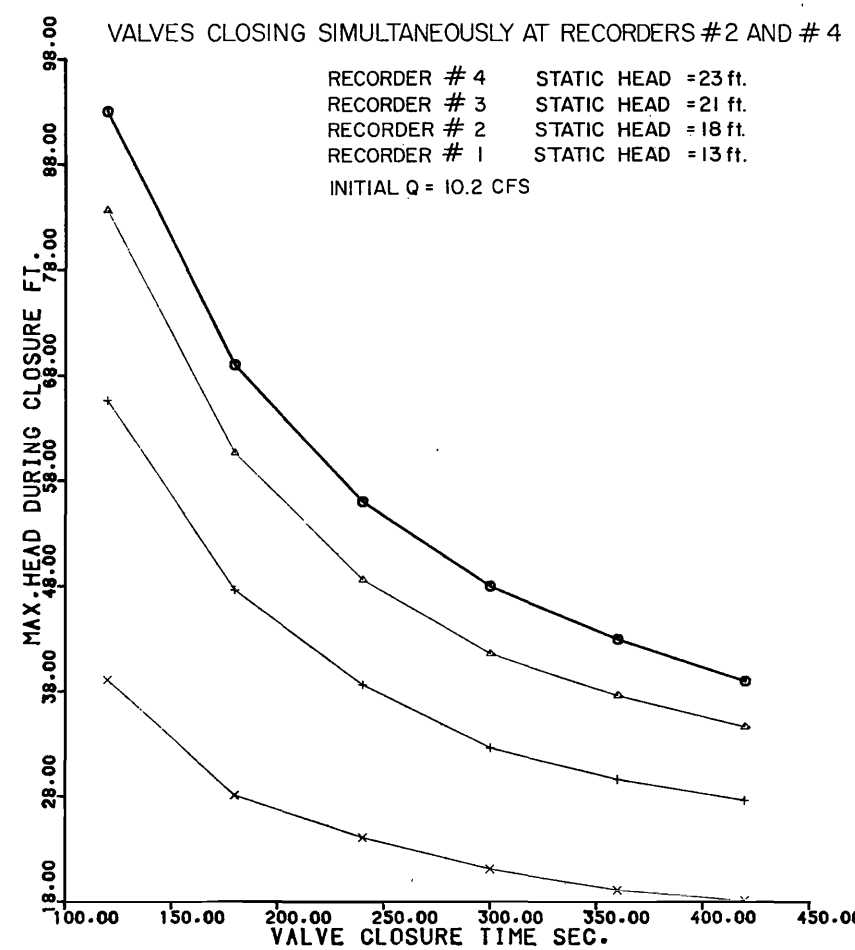

Figure 6.11. Maximum head in pipe for different closure times while closing valves 2 and 4 simultaneously (program 6).

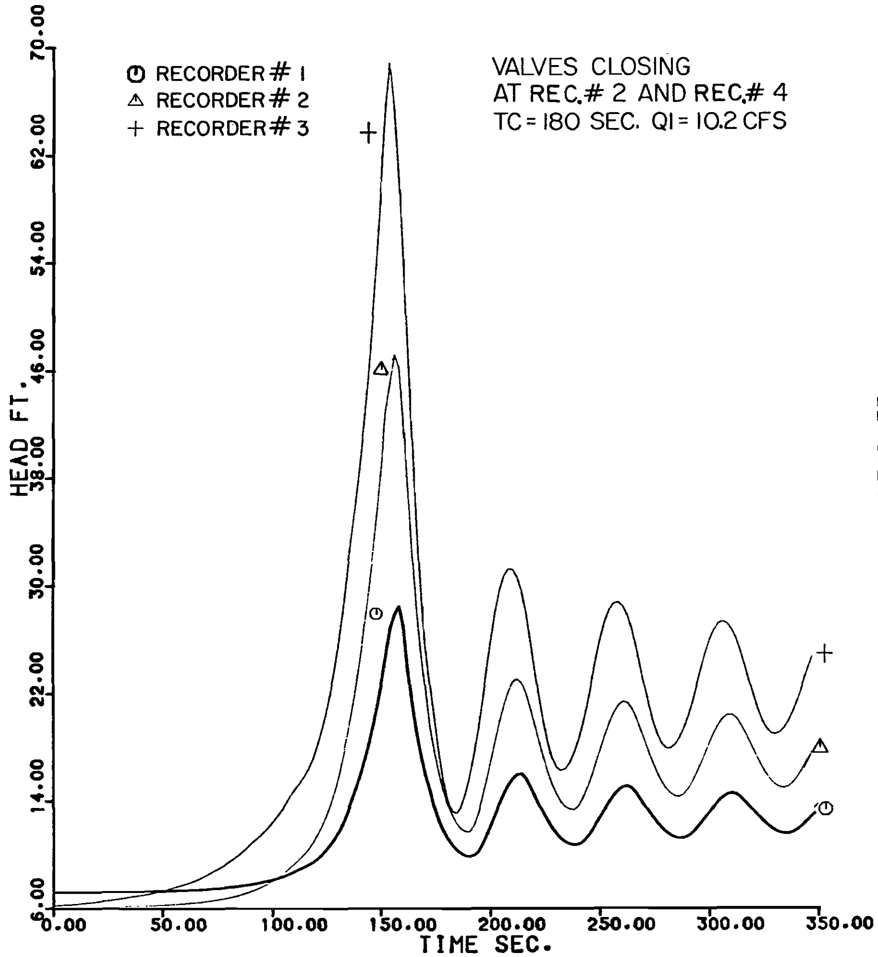

Figure 6.12. Transient pressure heads for simultaneous closure of valves at 2 and 4 in 180 seconds (program 6).

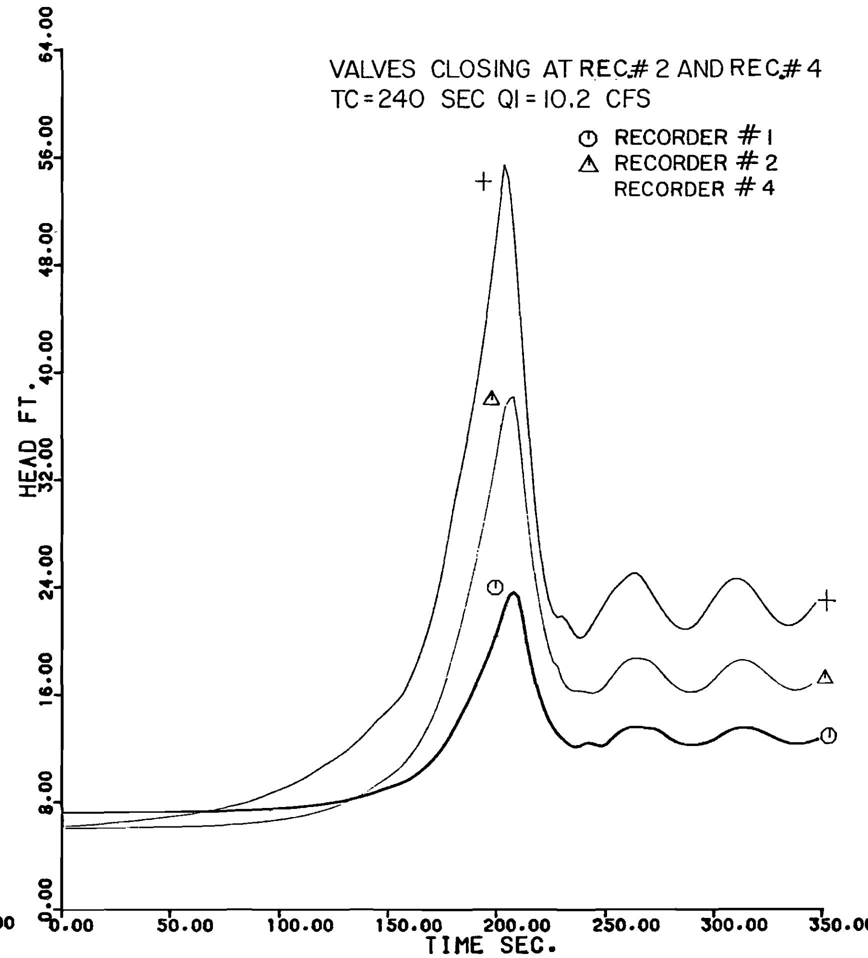

Figure 6.13. Transient pressure heads for simultaneous closure of valves at 2 and 4 in 240 seconds (program 6).

CHAPTER 7

POSSIBLE MODIFICATIONS TO THE PIPELINE TO REDUCE PRESSURE TRANSIENTS

Four different alternatives were considered for controlling the pressure transients generated by closing of the turnout valves in the Fielding Ditch Company Pipeline.

Long Closure Times

The lowest cost solution to the problem would be to specify an adequately long valve closure time such that the pressure surge would be controlled to a desirable level. This is not a practical solution because the preceding analysis indicates that in order to limit the pressure head in the pipe to less than 40 feet, valve closure times would have to be in excess of 6 or 7 minutes and there is no way to guarantee that the valves would not be closed in shorter periods of time. Sooner or later with changes of personnel in the field, a requirement for long valve closure time would be violated.

Pressure Relief Valves

There are two main limitations on the use of pressure relief valves to protect the pipeline. First, the mechanical nature of the operation of pressure relief valves means that they are not fool-proof and that there would always exist the possibility that the valves would not function properly. The second, and possibly more important reason for eliminating this alternative, is that once the pressure relief valves were opened, one would still have to be sure that they could not close in less than 5 or 6 minutes and much water would be spilled in the interim time. If the pressure relief valves were closed too quickly, they would also generate transients just as the sudden closing of any other valve.

Air Chambers

The possibility of adding air chambers either externally to the pipe or as an internal air-filled bladder inside the pipe was investigated. The design problem consisted of determining the required volume of internal air to limit the head rise to an acceptable level for a given rate of valve closure. Since it is possible that the valves can be closed instantly (any time less than 25 seconds), the analysis using air chambers was carried out for instantaneous valve closure. The results are summarized in Figure 7.1 which shows the maximum transient pressure head rise in the pipe as a function of volume of air in the air chambers.

It is immediately apparent that extremely large volumes of air are required to protect the pipe by this method. To limit the head rise to 40 feet requires a total air volume of 6,000 cubic feet. For comparison, the total internal volume of the pipeline is over 48,000 cubic feet. Even with an air volume of 10,000 cubic feet the head rise is greater than 30 feet. Because of the large volumes of air required, this method is considered impractical.

Standpipes At All Turnouts

Standpipes or small surge tanks were selected as the best and most reliable solution for protecting the pipeline. The number of standpipes required depends upon the desired degree of safety for the pipeline. Total protection of the pipeline, even for instantaneous valve closure, can be assured if a standpipe of adequate size is placed beside each valve. For the valves that are close together, fewer standpipes may be adequate. This alternative is discussed in a subsequent section.

Computer program number 7 was developed to analyze the system with a standpipe at each of the outlet valves except the valve that is operated or with fewer standpipes. The representation of the system did not exactly correspond to the field layout since the solution of the computer program requires that all valves be spaced at equal distances apart. Therefore, the valves in the program were at slightly different locations than they are in the field. This, however, would not have a major effect on the results of the analysis.

The program was run using different diameters and heights of standpipes, with and without resistance of the flow into the standpipe. Such resistance would be created by having a small pipe or orifice between the main pipe and the standpipe. The resistance reduces the amount of spillage out of the standpipe and attenuates the surge but also results in higher pressure inside the pipeline. It was subsequently decided to

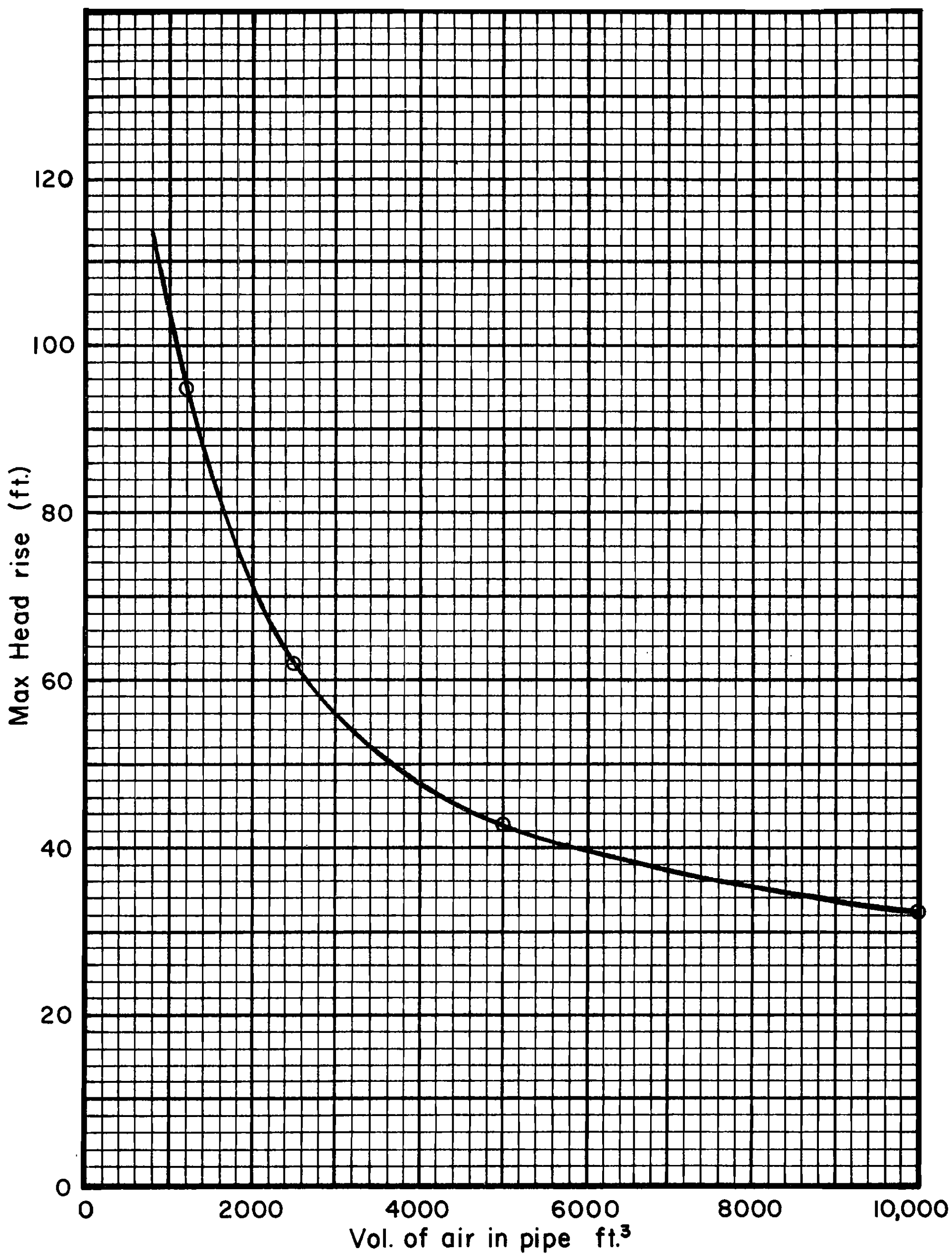

Figure 7.1. Effect of volume of air on the maximum head rise.

eliminate this resistance so as to minimize the pressure.

Standpipe diameters of 12, 24, 36, and 48 inches were studied with top elevations of all standpipes terminating at an elevation above datum of 115, 113, or 110 feet. Based on results of this preliminary part of the study, it was decided that the standpipe should be 18 inches in diameter and should only extend to an elevation of 113 feet. This limits maximum pressure inside the pipe to 30 feet.

With the number, size, and location of the standpipes fixed, the independent variables which occur in the operation of the system are the valve being closed, and the rate of valve closure. The spillage out of the standpipes is a dependent variable. The faster the valve is closed, the greater will be the spillage. Typical spillage from 18-inch standpipes as a function of valve closure time during the first 5 minutes after the valve starts to close is summarized in Figure 7.2. For an instant valve closure (T less than 25 seconds), the spillage in the first 5 minutes of operation will be approximately 240 cubic feet. The data in Figure 7.2 are for closure of valve number 33 at station 149+88.8 from fully opened with an initial discharge of 8.7 cfs. The actual total spillage will be greater than that shown in the figure because the oscillations of the water in the system will continue for longer than 5 minutes (most likely 20 to 30 minutes).

For valve closure of valve number 33 the largest amount of spillage occurs at the end of the pipeline. For instantaneous valve closure, 115 of the 240 cubic feet spill at the last standpipe. There is also spillage at almost all other standpipes throughout the system. As the closing time is increased, there is almost a linear decrease in the amount of spillage for+valve closure time up to 240 seconds. Beyond 240 seconds the

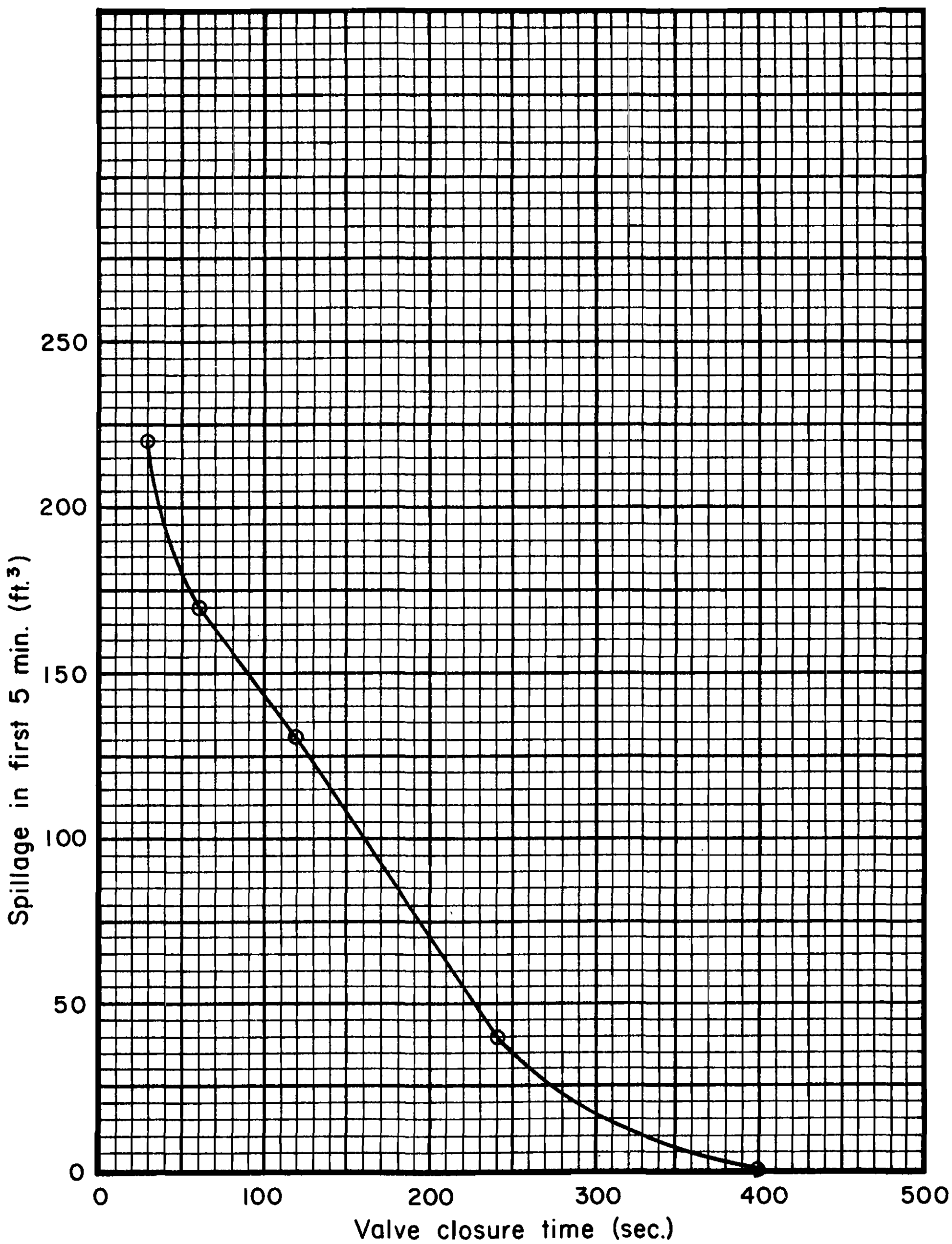

Figure 7.2. Effect of valve closure time on spillage from 18-inch standpipes.

rate of reduction of spillage decreases. Closing the valve at turnout number 33 without any spillage for an 18-inch standpipe terminating at elevation 115 requires approximately 400 seconds from fully opened to fully closed. The valve closure time can be reduced because most of the restriction of flow occurs during the last 30 to 40 degrees of valve closure. It would, therefore, be possible to rapidly close the valve from 90 degrees to about 40 degrees and then reduce the closing speed. By such a variable speed closing operation, it would be conceivable to reduce the closing time to about 200 seconds with little spilling.

The standpipe concept is shown in Figure 7.3. Some requirements of the design would be: 1) No significant amount of flow restriction between the riser pipe and the standpipe (a 12-inch pipe would be adequate); 2) flexible couplings to isolate structural movement of the standpipes and riser; and 3) structural support of the standpipe to withstand wind loading. It may also be desirable to make special provision for containing the spillage. This could be done by directing it into the distribution box at the riser or by forming a small catch basin at the base of the standpipe with an outlet to a ditch.

Since the recommended solution changes the pipeline from a closed to an open system, it will be subject to surging which will cause repeated cycles of spilling from the standpipes. Project resources and time did not allow for a thorough surge analysis of the system but the problem was considered. Transient program 7 properly simulates surges but, due to its small time step between calculations (0.1 second), it is not practical to simulate surges over periods of 30 to 60 minutes. The program was used only to estimate the spillage during the first 5 minutes after the valve started to close.

A simplified surge analyses of the system was carried out using rigid water column theory. The method and results are given in Appendix D. The method only considered one standpipe at the lower end of the pipeline. The results suggest that the period of the surge (time interval between spills) is about 140 seconds and that the surge can last considerably longer than 20 minutes. The number of spills and the time to dampen out the surge will be a function of which valve is closed, the initial flow rate, and the valve closure time.

The annoyance of periodic spillage from the standpipes can be reduced by incorporating additional features into the standpipes. One such possible feature is pop-up valves in the lower portion of the standpipes. These would not restrict the flow into the standpipe but would limit the rate at which the flow can leave the standpipe and reenter the main pipeline. By limiting the returning flow rate, additional losses are added into the flow system besides fluid

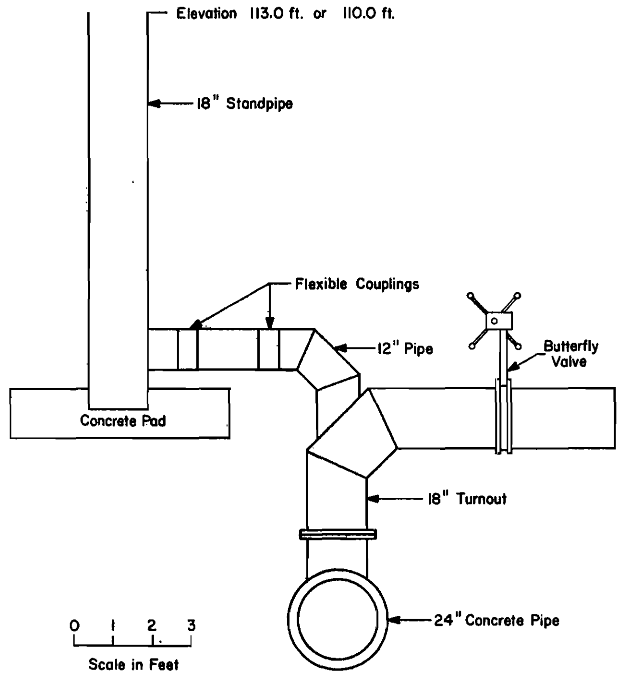

Figure 7.3. Standpipe for the protection of the pipeline.

friction to more rapidly damp out the oscillations. Pop-up valves, unlike orifices, would not increase the pressure in the pipeline since the water could move into the standpipe as readily as if the valves were not there. These pop-up valves could contain smaller orifices of 3-4-inch diameters to insure that water from the standpipes would drain into the main pipeline. The spillage problem is considered to be primarily a nuisance rather than posing any dangerous operating conditions on the pipeline.

Reducing the Required Number of Standpipes

Placing a standpipe at each valve would gives complete protection to the pipeline but is also costly. Such a choice would give protection even for instant closure of any valve. Since the valves can only be closed at a finite rate and not instantly, the number of standpipes can be reduced somewhat and still give sufficient protection. If fewer standpipes are used, the distance that any standpipe can be from a valve (which determines the total number required) must be selected so as to limit the head in the pipe to less than 30 ft.

The maximum distance that a standpipe can be from a valve and still provide the required protection was evaluated two ways. First, hand calculations were made assuming that the valve was being closed the last few degrees as fast as possible. Next the same rapid valve closure rate was used to estimate the shortest possible closure time (from fully opened to closed) and the head rise was calculated using the computer program.

Hand Calculations. The most important variable in this analysis is the maximum valve closure rate. The valve mechanism is geared such that it takes 12.5 revolutions of the handwheel to completely close the valve. Field tests by SCS personnel determined that the valve could be closed in about 25 seconds. This is an average closing rate of 3.6 deg/sec. Field tests by USU personnel indicated that about one second was required to rotate the valve handle one revolution. This is equivalent to a closing rate of 7.2 deg/sec. A still more conservative valve closure rate of 8.4 deg/sec was selected for analysis. This means that the valve is rotated 1 revolution in 0.86 seconds and complete closure would take 10.7 seconds. Any valve movements faster than this would seem highly unlikely. Using a valve closure speed of 8.4 deg/sec, the spacing of the standpipes is determined as follows:

1. Select a valve at the lower end of in the pipe where the static head is maximum.

2. Use full wave speed (3640 fps) to be still more conservative. (The actual measured wave speed due to air trapped in the pipe was about 1170 fps.)

3. Select a trial distance (such as 500 ft) from valve to standpipe.

4. Calculate the time for a pressure wave generated at the valve to travel to and from the standpipe ($T = 2L/a$ where $L = 500$ ft and $a = 3640$ fps for this example; so $T = 0.27$ sec).

5. Determine how far the valve can close in 2L/a seconds. (Valve closure = 8.4 deg/sec x 0.27 sec = 2.31 degrees for this example.) Note that even if the valve were moved more than 2.31 degrees, the head rise at the valve would not be increased significantly because of relief of the pressure due to reflections from the standpipe.

6. Since the worst pressure transients occur when the valve closes the last few degrees, find the flow rate when the valve is open 2.3 degrees. No water flows until the valve is open about 5 degrees so the actual valve opening will be 7.31 degrees to estimate the flow.

7. At 7.31 degrees the valve discharge coefficient is estimated to be 0.0045 which results in a pipe velocity of 0.08 fps.

8. Calculate the head rise ΔH caused by stopping the 0.08 fps (ΔV) in 0.27 seconds by $\Delta H = a\Delta V/g$ where $a = 3640$ fps, $g = 32.2$ fps^2 so $\Delta H = 9.04$ ft.

9. Since the valve is not at the end, the pressure surge can travel two directions in the pipe and, therefore, the transient headrise in the 24-inch pipe is 9.04/2 = 4.5 ft.

10. The maximum head at the valve is static head plus ΔH or about 24 + 4.5 = 28.5 ft.

11. Since the 28.5 ft is less than 30 ft a spacing of 500 ft is acceptable.

12. For comparison, use the lower measured wave speed (1100 fps) in the above method $2L/a = 0.91$ sec, $\Delta V = 0.22$ fps, and $\Delta H = 7.5$ ft. Thus the 500 ft spacing is acceptable under this assumption also.

Additional calculations are summarized in Table 7.1.

These results show that for valves near the downstream end where the static head is maximum (24 ft above the pipe), that no valve should be further than about 700 ft from a standpipe. For valves closer to the inlet, the spacing can be greater since the static head is less.

Computer solution. The hand calculations just discussed only apply to small valve closure times. The transients generated by valve closures lasting longer than 2L/a seconds require use of the computer program. Evaluation of the allowable spacing of the

Table 7.1. Allowable spacing of standpipes based on a valve closure rate of 8.4 deg/sec, and a = 3640 fps. Valve located where pipe elevation = 82 ft.

Distance to Standpipe ft	TC = 2L/a sec	Initial Valve Opening deg	C_d	Pipe Velocity fps	H ft	Maximum Head ft
500	0.27	7.31	0.0045	0.08	4.52	28.5
750	0.41	8.44	0.0066	0.12	6.78	30.8
1000	0.55	9.62	0.0089	0.15	8.48	32.5
1500	0.82	11.89	0.013	0.19	10.7	34.7

standpipes for complete valve closure was done for a 10 second total closure time. This is almost three times faster than is normally possible and hence the results will be very conservative. The other factor which also makes the results conservative is that the computer solution did not provide for restriction of flow at the pipeline inlet which normally keeps the maximum flow to less than 8 cfs. If the inlet stays submerged and the flow from the canal is not restricted, the pipe can pass almost 15 cfs.

The results of the computer solution for 18-inch standpipes terminating at an elevation of 113 ft are in the following table:

Table 7.2. Allowable spacing of standpipes based on a complete valve closure in 10 seconds, EL = 113, Dia = 18 in.

Distance to Standpipe	Valve being Closed	Initial Vlow cfs	Maximum Head at Valve (ft above pipe)
1275	6	14.9	35
	13	13.5	34
	18	10.7	30
	30	9.5	33
956	5	14.9	28
	15	12.7	28
	21	10.1	27
	30	9.1	31

The data in Table 7.2 suggest that the standpipe spacing at the downstream end of the pipe should be somewhat less than 956 ft to keep the maximum pressure below 30 ft if the standpipes are terminated at elevation 113. Later in the project it was decided to use standpipe tops at elevation 110. This drops the maximum head about 3 ft and would make a standpipe spacing of about 1000 ft acceptable. However, since the hand calculations for closure of the valve the last few degrees requires the maximum spacing to be about 700 ft, that is accepted as the controlling spacing.

Recommended standpipe locations. Based on the previous analyses the system will be adequately protected (that is no pressures greater than 30 ft will be created) if there are enough standpipes such that the distance from them to any valve being closed is less than 700 ft. Table 7.3 suggests five different standpipe layouts for 5, 9, 12, 16, 20, and 25 pipes. The most important data are those at the bottom of the table where the maximum distance to a valve is listed. For 5 and 8 standpipes the distance is greater than the 700 ft required. With 12 pipes the maximum distance is 664 ft. Increasing the number to 16 only reduces the distance to 631 ft. Twelve pipes, therefore, appear to be a reasonable choice.

Spillage. The variables which will control the spillage are the valve being closed, valve closure time, the standpipe diameter, and the height. Numerous combinations of these variables were checked with the computer simulation program to identify which combination of the variables would give the least spillage and still be an economical solution.

In analyzing the system it became apparent that the pipe has greater flow capacity than the intake. Depending on which valve or valves are opened, the pipe can handle over 15 cfs if the inlet stays full. Since the inlet restricts the flow, the final computer program was modified to limit the maximum flow to 8 cfs. Data for higher flows are included since such flow rates are possible for a very short time, until the intake draws down.

Results of the various runs are in Table 7.4. These data show the influence of valve closure time, standpipe elevation, and diameter. After studying the various possibilities, the following is recommended:

1. The valve closure time should be 2 minutes, or longer.

2. Terminate the standpipes at elevation 110.

3. The first 11 standpipes should be 18-inch diameter and the last one 36 inches in diameter.

4. Pipe the spillage from the end standpipe into the 24-inch drain line.

Air Vents

At valves where there is no standpipe, provision must be made to remove air that may get trapped in the riser. If the air is not removed before the turnout valve is opened, a transient may be generated which will subject the pipe to pressures greater than 30 ft. See field test number 15 in Appendix B-21 to B-23 for an example of such a transient.

Two solutions are possible: 1) installing an air release valve or 2) placing small standpipes or air vents to remove the air. The pipes would not require maintenance and would continually remove any trapped air. The water will rise and fall in these small pipes in the same period as it does in the large standpipes. No sudden rush of water will come out of these pipes unless they are releasing large volumes of air. It might be advisable to terminate them at 111 or 112 feet to prevent any spillage. However, to leave them at 110 feet would have a beneficial affect on operators of the nearby valve. The operator would then know at all times whether or not he was causing spillage from the pipeline by his operation of the valve.

Table 7.3 Options for standpipe locations.

			Turnout Locations					
Node	Station	Comment	5 T.O.	9 T.O.	12 T.O.[a]	16 T.O.	20 T.O.	25 T.O.
1	1+34.0	Pipeline Start						
2	19+03.0	18		X	X	X	X	X
3	23+63.0	12				X		X
4	28+01.5	12					X	X
5	32+18.0	18	X		X	X		
6	35+63.0	Recorder 1 18		X			X	X
7	45+54.4	18				X	X	X
8	50+18.4	18		X	X	X	X	X
9	54+74.4	8						X
10	58+73.9	8					X	X
11	65+04.9	18	X		X	X	X	X
12	66+80.9	12						
13	66+88.9	18						
14	69+48.9	18		X				X
15	72+27.9	12						
16	73+15.9	Recorder 2 18			X	X	X	X
17	77+79.4	18					X	X
18	81+86.9	18				X		X
19	85+62.3	18		X	X		X	X
20	90+67.0	18	X					
21	93+07.0	18				X	X	X
22	98+26.5	18			X			
23	101+62.9	18					X	X
24	104+66.9	18				X		
25	107+45.9	18		X	X		X	X
26	110+66.7	18						
27	114+10.2	Recorder 3 18				X	X	X
28	120+89.7	18	X		X	X	X	X
29	128+15.2	18		X			X	X
30	133+89.8	18			X	X	X	X
31	142+92.8	18		X		X	X	X
32	147+40.5	18			X	X	X	X
33	149+88.8	18						X
34	154+44.5	Recorder 4 18	X	X	X	X	X	X
Max. Dist. from Valve to Relief Riser			1400	919	664	631	460	345

[a]Recommended number of standpipes.

Table 7.4. Spillage from standpipes.

Standpipe Number and Size	Elevation of Standpipe ft	Valve Being Closed	Valve Closure Time Sec.	Initial Flow cfs	Total Spillage in 5 min. ft^3	Max. Spill ft^3 from Standpipe in 5 min.	Standpipe having Maximum Spillage	Maximum Spillage rate cfs	Maximum Head at Valve ft
6 @ 18"	113	8	60	12	48	17	11		25
+			120	12	33	21	12	.7	24
3 @ 24"			180	12	234	72	5		24
+	110	8	60	12	194	49	12	2.3	23
3 @ 36"			180	12	157	106	12	1.3	21
			240	12	110	54	12	1.0	21
	113	27	60	8.9	105	75	12	1.3	28
			120	8.9	84	44	12	.9	28
			180	8.9	67	38	12	.8	28
	110	27	60	8.9	259	114	11		25
			120	8.9	219	103	12	1.2	25
			240	8.9	163	75	12	1.4	25
9 @ 12"	110	8	180	12.1	339	63	12		23.1
+	110	8	240	12.1	454	92	7		22.0
3 @ 36"	110	27	240	8.9	378	74	5		25.5
(10,11,12)									
9 @ 18"	110	8	240	12.1	419	107	6		19.9
+									
3 @ 36"	110	27	240	8.9	175	88	12		24.8
10 @ 18"	110	25	120	8.0	102	60	12	1.1	23
+		27	120	8.0	120	68	12	1.1	25
2 @ 36"		29	120	8.0	134	74	12	1.1	26
(at 11&12)									
11 @ 18"	110	25	120	8.0	126	92	11	∿ 2	24
+		27	120	8.0	142	108	11	∿ 2	25
1 @ 36"		29	120	8.0	157	115	11	∿ 2	26
(at 11)									
11 @ 18"	110	25	120	8.0	130	112	12	2.1	24
+		27	120	8.0	145	119	12	2.1	25
1 @ 36"		29	120	8.0	158	134	12	2.1	26
(at 12)									
11 @ 18"	110	27	120	8.0	114	109	12	2.3	25
+									
1 @ 48"									
(at 12)									
12 @ 18"	110	1	120	8.0	0	0		0	13
		4	120	8.0	0	0		0	15
		6	120	8.0	1	1	4	.2	17
		8	120	8.0	2	1	1	.5	19
		10	120	8.0	0	0		0	21
		15	120	8.0	58	23	11&12	.7	21
		18	120	8.0	110	37	12	.8	22
		21	120	8.0	106	43	11	1.0	24
		25	120	8.0	145	59	11	1.0	24
		27	120	8.0	162	67	11	1.1	25
		29	120	8.0	187	77	11	1.3	26
12 @ 18"	113	10	30	12.0	435	98	12		27
			60	12.0	213	72	12		26
			120	12.0	149	42	11		27
			180	12.0	89	25	12		27
			240	12.0	23	6	7		26
			300	12.0					25
	110	27	30	12.0	444	170	12		26
			60	12.0	539	90	8		25
			120	12.0	261	63	11		25
			180	12.0	180	32	11		25
			240	12.0	126	41	11		24
			300	12.0	77	21	8&11		24

Table 7.4. Continued.

Standpipe Number and Size	Elevation of Standpipe ft	Valve Being Closed	Valve Closure Time Sec.	Initial Flow cfs	Total Spillage in 5 min. ft^3	Max. Spill ft^3 from Standpipe in 5 min.	Standpipe having Maximum Spillage	Maximum Spillage rate cfs	Maximum Head at Valve ft
	113	27	30	8.9	620	101	11		29
			60	8.9	571	115	11		28
			120	8.9	473	147	11		28
			180	8.9	351	88	11		28
			240	8.9	105	39	11		28
			300	8.9	215	82	11		28
	110	27	30	8.9	536	140	11		26
			60	8.9	452	129	11		25
			120	8.9	463	161	11		25
			180	8.9	406	148	11		25
			240	8.9	252	110	11		25
			300	8.9	125	54	10		25
	113	4	120	14	0	0			18
12 @ 24"	113	10	30	12	87	44	12		27.7
			60	12	128	27	12		26.4
			120	12	219	74	7		25.8
			180	12	56	28	12		24.8
			240	12	516	159	8		20.7
			300	12	144	36	6		23.2
	110	10	30	12	252	116	12		24.9
			60	12	254	143	12		24.5
			120	12	225	75	11		23.5
			180	12	204	56	11		23.7
			240	12	247	54	10		22.9
			300	12	137	33	6		22.5
	113	27	30	8.9	118	50	10		28.6
			60	8.9	117	28	12		28.1
			120	8.9	100	42	12		28.0
			180	8.9	75	39	11		27.9
			240	8.9	64	31	11		27.7
			300	8.9	19	11	11		27.7
	110	27	30	8.9	284	131	11		25.4
			60	8.9	275	99	11		25.1
			120	8.9	252	84	12		25.0
			180	8.9	223	92	12		24.9
			240	8.9	179	124	11		24.8
			300	8.9	93	54	11		24.9
12 @ 36"	113	10	30	12	0	0			24.7
			60	12	0	0			23.9
			120	12	0	0			23.6
			180	12	0	0			23.1
			240	12	0	0			21.3
			300	12	0	0			21.0
	110	10	30	8.9	38	13	7		24.2
			60	8.9	42	22	12		23.9
			120	8.9	17	7	9		23.6
			180	8.9	10	5	11&12		23.1
			240	8.9	2	1	12		21.3
			300	8.9	0	0			21.0
12 @ 36"	113	27	30	8.9	0	0			26.4
			60	8.9	0	0			25.8
			120	8.9	0	0			25.8
			180	8.9	0	0			25.6
			240	8.9	0	0			24.9
			300	8.9	0	0			24.8
	110	27	30	8.9	73	39	11		25.0
			60	8.9	73	29	12		24.8
			120	8.9	73	33	12		24.7
			180	8.9	45	21	10		24.7
			240	8.9	11	4	12		24.7
			300	8.9	0	0			22.8

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. Rapid closure of either turnout valves or the inline valves in the present system can generate severe pressure transients in the Fielding Ditch Company Pipeline which will overstress the nonreinforced concrete pipe.

2. The worst transient would correspond to a valve closure in a time equal to or less than 25 seconds and would generate a maximum head in the pipe of approximately 220 feet. This only could happen if most of the air were removed from the pipeline and full wave velocity of 3630 fps occurred.

3. Closing the turnout valve at the end of the pipeline in approximately 2 minutes results in a maximum head in the pipe of approximately 80 feet.

4. In order to limit the maximum head in the pipe to approximately 30 feet in the present system, a valve closure time in excess of 7 minutes is required.

5. The maximum pressure head due to transients generated by closing any valve in the system occurs at the downstream end of the pipe.

6. Closing two valves simultaneously from fully opened to fully closed creates a more severe transient than closing any one valve.

7. When a transient pressure is generated at any valve, the entire pipeline is subjected to almost the same pressure.

8. Air collected in the risers at the turnouts must be released slowly. If the air is suddenly released through the outlet valve, severe transients can be generated in the system.

9. Air distributed throughout the pipeline and collected in the joints, risers, or at high points in the pipeline is helpful in reducing the magnitude of the transients. Each of these pockets of air are individually small but their collective effect reduces the wave speeds and consequently the subsequent transient pressures. The existence of entrapped air was verified by field tests where the measured wave velocity was approximately 1170 feet per second compared to the theoretical computed velocity without air of 3640 feet per second.

10. Due to the potential for the generating serious transients under normal valve operation, modifications of the pipeline are necessary in order to limit the transient pressure increases to an acceptable level.

11. Modifications which were considered but not recommended included: 1) requiring a longer valve closure time, 2) installation of pressure relief valves, and 3) installation of air chambers.

Recommendations

1. It is recommended that one 36-inch diameter standpipe be installed at turnout 34 (Sta. 154+44.5) and eleven 18-inch standpipes at the locations shown in Table 7.3. The top elevations of all turnouts should be 110 feet. The standpipes should be connected to the riser with at least a 12 inch diameter pipe. These standpipes will give adequate protection against overpressurization of the pipeline but pose a problem of surging which can cause repeated cycles of spilling for a considerable length of time. Provision should be made to collect the spilled water at each standpipe and direct it into the diversion box or nearby ditch. Visible spillage at a standpipe will serve as an indicator that the valve is being closed too rapidly.

2. Small diameter (2 or 3-inch diameter) standpipes should be installed at all other turnouts that do not have large standpipes. These will release air and serve as indicators of too rapid valve closure.

3. Special care should be exercised in opening or closing the inline valve at Station 73+25.9 or the downstream drain valve at Station 154+53.8. Because of their size and location these pose an even greater hazard for creating damaging transients than do the turnout valves. No detailed analysis was made of these valves since they are so infrequently operated.

4. It is recommended that the Soil Conservation Service adopt as a design procedure for new pipelines that at least a limited transient analysis be performed on each pipeline to determine if any protective devices need to be incorporated into the pipeline design.

APPENDICES AVAILABLE FROM UTAH WATER RESEARCH LABORATORY

BY REQUEST