

Mainlines deliver water from the water source to irrigation zones or from point A to point B. Zones are activated by solenoid valves that supply water to the submains, which then supply water to the laterals (Fig. 8.1). Buried mainlines are generally constructed with PVC pipe, and surface mainlines are generally constructed with aluminum or polyethylene pipe. As with laterals, mainline pipe friction loss is generally minimized; however, instead of the goal of high uniformity, the goal is to save energy. Pipe friction loss is calculated with the Hazen-Williams or Darcy-Weisbach equation and with equations for minor losses in fittings and valves. Pipe selection is based on pipe cost, energy cost (friction loss), and pipe pressure rating. Because mainlines are generally blocked at the end by valves, pipe transients such as water hammer are of concern. Sudden changes in velocity can cause pressure surges that might burst the pipe. Pressure relief valves dissipate pressure surges, and air vents allow air to escape as the pipe is filled. Proper installation techniques and adequate pipe structural strength prevent pipe structural failure.

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### Pipe Size Classifications

There are three PVC pipe dimensional classification systems: Class (Standard dimension ratios), Schedule (based on steel pipe dimensions), and PIP (Plastic Irrigation Pipe). Class and Schedule pipes are manufactured with outside diameters corresponding to the IPS (Iron Pipe Size) system while PIP pipe has different outside diameters; thus, PIP pipe and IPS pipe are not mixed within the same irrigation system because PVC fittings glue to the outside of pipes and PIP and IPS fittings have different inside diameters. Within each PVC pipe classification system, pipes are classified based on wall thickness. While all pipes of a given size have the same outside diameter, they have different wall thicknesses and inside diameters. The Class system refers to the pressure rating of PVC pipe

(maximum allowable pressure for the pipe). Pressure ratings are calculated based on the ratio of wall thickness to pipe diameter (dimension ratio). The Class system standard dimension ratios (SDR) are: 41, 32.5, 26, and 21 for Class 100, 125, 160, and 200, respectively (Class 200 pipe has a pressure rating of 200 psi). Dimension ratios are ratio of wall thickness to pipe diameter, Schedule 40 and Schedule 80, which are based on iron pipe size specifications. Pipe class and schedule sizes in the United States and metric units for pipe sizes less than 12" (300 mm) are listed in Tables 8.1 and 8.2, respectively.

Large diameter pipe is generally sold as PIP pipe (gasketed pipe) or Schedule 40 pipe (glued fittings). Large diameter Schedule 40 pipe diameters and pressure ratings are listed in Table 8.3.

**In-class Exercise 8.1** Determine whether Schedule 40 25 mm (1 in.) pipe has a thicker wall than Class 200 25 mm (1 in.) pipe.

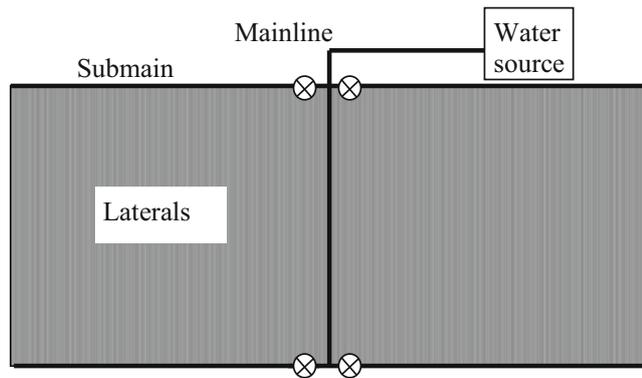
Determine the inside diameter (m) of Class 200 25 mm (1 in.) pipe.

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### Pipeline Energy Calculations

Pressure and elevation change (energy gradients) propel water through pipelines while water molecules moving past each other cause friction, which dissipates energy and resists flow. In turbulent flow, water molecules move past each other in three scales of eddies; in laminar flow, water molecules in the layered flow profile move past each other in concentric layers. Although friction energy loss is dissipated as heat in irrigation pipelines, the heat does not increase the pipe temperature to the point where it is noticeable to human touch.

In irrigation pipelines, the energy of water includes elevation potential energy, kinetic energy and pressure energy, and is calculated with the Bernoulli equation.



**Fig. 8.1** Irrigation pipe network

$$H = \frac{v^2}{2g} + \frac{P}{\rho g} + z \quad (8.1)$$

where

$z$  = elevation of pipeline above datum, m

$P$  = pressure, Pa

$v$  = velocity, m/sec

$H$  = head, m.

Energy of water in pipelines is typically divided into hydraulic head (elevation + pressure) and velocity head: total energy of the water is the sum of hydraulic head and velocity head. Hydraulic head is the elevation that water rises in a vertical pipe (manometer) attached to an irrigation pipeline and is also called piezometric head. Velocity head is generally negligible in comparison to hydraulic head in sprinkler and drip irrigation pipelines but may be significant in low pressure (surface irrigation) pipelines or channels.

**In-class Exercise 8.2** Calculate the hydraulic head and total energy of water flowing in a pipe that is 5 m above the datum. Pressure is 35 kPa (~3.5 m) and water velocity is 1.5 m/sec. Draw the pipe with a vertical manometer extending upward and show the pipe elevation, piezometric head elevation (water level in manometer) and total energy elevation.

For friction loss calculations, irrigation pipelines are assumed to operate at steady state; thus, there is no change in internal energy within the pipeline over time. The law of conservation of energy states that energy out (point 2 downstream) = energy in (point 1 upstream) – energy lost + energy added.

$$H_2 = H_1 - h_f - h_m + H_p$$

$$\frac{v_2^2}{2g} + \frac{P_2}{\rho g} + z_2 = \frac{v_1^2}{2g} + \frac{P_1}{\rho g} + z_1 - h_f - h_m + H_p \quad (8.2)$$

where

$h_f$  = energy lost to pipe friction loss between points 1 and 2, m

$h_m$  = minor losses (energy lost in fittings and valves), m

$H_p$  = energy added by pump, m.

Pipe friction loss in straight pipes,  $h_f$ , can be calculated with either of two equations, the Hazen-Williams equation or the Darcy-Weisbach equation. The Hazen-Williams equation is simpler than the Darcy-Weisbach equation; however, the Hazen-Williams does not differentiate between laminar and turbulent flow or account for change in viscosity due to temperature. The Hazen-Williams equation is valid within the normal ranges of flow velocity and temperature found in irrigation pipelines. Because of its simplicity, the Hazen-Williams equation is generally used to calculate friction loss in larger pipelines.

$$h_f = k_1 L \frac{\left(\frac{Q}{C}\right)^{1.85}}{D^{4.87}} \quad (8.3)$$

where

$k_1$  = conversion factor (Table 8.4)

$L$  = length of the pipe, m of ft

$Q$  = pipe flow rate, various units

$C$  = roughness coefficient

$D$  = diameter, mm.

The Hazen-Williams  $C$  value assumes viscosity =  $1 \times 10^{-6}$  m<sup>2</sup>/s and so is only valid between 40 and 75°F (4 °C and 24 °C). The design Hazen-Williams  $C$  value ranges from 140 to 150 for smooth pipes constructed from polyvinyl chloride (PVC), polyethylene (drip tubing), and epoxy coated steel. However,  $C$  is lower for rough pipes because surface roughness increases turbulence. Iron pipe degrades over time, and the  $C$  value decreases from 130 for new iron pipe to 80 for 40-year-old iron pipe. The  $C$  value is approximately 130 for new galvanized steel pipe and new concrete pipe. Aluminum pipe has a range of reported  $C$  values depending on the type of couplings between pipes (from 130 to 90) and the application. Conventional 3" (75 mm) aluminum hand lines with 30 ft pipe lengths joined by aluminum couplers have a  $C$  value of 130 (*NRCS NEH, Sec. 15, Ch. 11*).

The Darcy-Weisbach equation has different forms for laminar and turbulent flow.

$$h_f = f \frac{L v^2}{D 2g} \quad (8.4a)$$



**Table 8.2** Nominal diameters, wall thickness, and inside diameters for PVC pipe (metric: mm); Schedule 40 dimensions are Schedule 40 steel pipe dimensions

	Spec	18	25	31	37	50	62	75	100	150	200	250	300
kPa rating	OD (mm)	26.7	33.4	42.2	48.3	60.3	73.0	88.9	114.3	168.3	219.1	273.1	323.9
689	Wall (mm)							2.16	2.79	4.11	5.33	6.65	7.90
862				1.30	1.49	1.86	2.25	2.74	3.51	5.18	6.73	8.41	9.96
1103				1.63	1.85	2.31	2.79	3.43	4.39	6.48	8.43	10.49	12.45
1379		1.52	1.60	2.01	2.29	2.87	3.48	4.24	5.44	8.03	10.41	12.98	15.39
SCH40		2.87	3.38	3.56	3.68	3.91	5.16	5.49	6.02	7.11	8.18	9.27	10.31
689	ID (mm)							84.6	108.7	160.0	208.4	259.7	308.1
862				39.6	45.3	56.6	68.5	83.4	107.3	157.9	205.6	256.2	303.9
1103				38.9	44.6	55.7	67.4	82.0	105.5	155.3	202.2	252.1	299.0
1379		23.6	30.2	38.2	43.7	54.6	66.1	80.4	103.4	152.2	198.2	247.1	293.1
SCH40		20.9	26.6	35.1	40.9	52.5	62.7	77.9	102.3	154.1	202.7	254.5	303.2
SCH40	kPa	3360	3150	2590	2310	1960	2100	1820	1540	1260	1120	980	900

**Table 8.3** Larger diameter PVC and steel Schedule 40 pipe dimensions

United States units						Metric units				
Nominal diameter (in)	OD (in)	Wall thickness (in)	ID (in)	Rating (PSI)	DR	Nominal diameter (mm)	OD (mm)	Wall thickness (mm)	ID (mm)	rating (kPa)
14	14	0.437	13.126	130	32	350	356	11.1	333	900
15	15	0.469	14.062	130	32	375	381	11.9	357	900
16	16	0.5	15	130	32	400	406	12.7	381	900
18	18	0.562	16.876	130	32	450	457	14.3	429	900
20	20	0.593	18.814	120	34	500	508	15.1	478	830
21	21	0.617	19.766	120	34	525	533	15.7	502	830
24	24	0.687	22.626	120	35	600	610	17.4	575	830

**Table 8.4** Conversion constants for the Hazen-Williams equation given different combinations of units (From Cuenca 1989)

$h_f$	length	Q	D	$k_1$
m	m	L/sec	mm	$1.22 * 10^{10}$
m	m	L/hr	mm	3163
m	m	$m^3/day$	mm	$3.162 * 10^6$
ft	ft	$ft^3/sec$	ft	4.73
ft	ft	GPM	in	10.46

**Table 8.5** Conversion constants for the Darcy-Weisbach equation given different combinations of units (From Cuenca 1989)

$h_f$	Length	Q	D	$k_2$
m	m	L/sec	mm	$8.26 * 10^7$
m	m	L/hr	mm	6.376
m	m	$m^3/day$	mm	$1.107 * 10^4$
ft	ft	$ft^3/sec$	ft	0.0252
ft	ft	gpm	in	0.0311

where

- L = pipe length, m
- D = lateral inside diameter, m
- f = Darcy-Weisbach friction factor
- $h_f$  = head loss, m
- v = velocity, m/sec.

Darcy-Weisbach friction factor can be modified in order to make it more convenient for use in pipe flow calculations. (Table 8.5)

$$h_f = k_2 f L \frac{Q^2}{D^5} \tag{8.4b}$$

where

- $k_2$  = conversion factor (Table 8.8)
- L = lateral length, m
- Q = pipe flow rate, L/h or other flow units
- D = lateral inside diameter, mm or other flow units

- f = Darcy-Weisbach friction factor
- $h_f$  = head loss, m.

The Blasius friction factor is appropriate for smooth pipe (such as PVC) with turbulent flow ( $Re > 2,000$ ).

$$f = \frac{0.316}{Re^{1/4}} \tag{8.5}$$

$$Re = \frac{vD}{\nu} \tag{8.6}$$

where

- v = velocity in the pipeline, m/sec
- D = pipe inside diameter, m
- $\nu$  = kinematic viscosity of water,  $1 * 10^{-6} m^2/sec$  at standard temperature
- Re = Reynold's number, dimensionless.

For laminar flow (Reynolds number  $< 2,000$ ) with any pipe roughness, the friction factor is.

$$f = 64/Re \tag{8.7}$$

**In-class Exercise 8.3** What ratio does the Reynold's number represent, and why are higher Reynold's numbers associated with turbulent flow?

In the transitional flow regime between  $2,000 < Re < 4,000$ , there is a discontinuity between Eqs. 8.5 and 8.7. In general, the choice of equations in this range is not critical because friction loss is insignificant at low velocities.

Water velocity is the flow rate over the pipe cross-sectional area, A

$$v = Q/A = Q/(\pi D^2/4) \tag{8.8}$$

where

- Q = flow rate,  $m^3/sec$
- A = pipe cross-sectional area,  $m^2$ .

**Table 8.6** Equivalent lengths of valves and steel fittings

Type of fitting	Equiv. length/diameter
Globe valves, fully open (solenoid valve)	340
Gate valves, fully open	13
Gate valves, ¾ open	35
Gate valves, ½ open	160
Swing check valves, fully open	135
In-line check valves, fully open	150
90° standard elbow	20–30
45° standard elbow	16
90° street elbow (larger radius)	50
Standard tee, flow through run	20
Standard tee, flow through branch	60

Minor losses in fittings, valves, and transitions are a function of velocity squared.

$$h_m = K_m \frac{v^2}{2g} \quad (8.9)$$

where

$h_m$  = minor energy losses in fittings, valves, and transitions, m

$v$  = velocity, m/sec

$K_m$  = minor loss coefficient.

Minor loss coefficients for most valves and fittings can be found online by typing “minor loss coefficients” into Google. Many valves used in irrigation systems, such as solenoid valves, backflow prevention valves, and pressure regulators, have geometries that are specific to each manufacturer. Manufacturer’s catalogs should be consulted to find minor loss coefficients or graphs of pressure loss vs. flow rate for these valves.

In some cases, it is convenient to set up Eq. 8.2 with minor losses written in terms of the equivalent length of pipe rather than as a function of velocity squared. The equivalent length is a function of the pipe diameter. Table 8.6 lists losses in fittings in terms of equivalent length.

Minor loss coefficients for long-radius butt-welded or flanged 90° elbows are listed in Table 8.7, which are a function of the radius of the angle divided by the inside diameter of the pipe.  $L/D$  represents the equivalent length to pipe diameter ratio.

Minor loss coefficients for mitered bends (cut and welded pipes) are listed in Table 8.8.

**Example 8.1** Calculate equivalent length and length to diameter ratio ( $L/D$ ) of a square edged inlet and pipe discharge to a pond if water velocity in the pipe is 1.0 m/sec and

pipe diameter is 0.1 m. Assume that the Darcy-Weisbach friction factor is calculated with the Blasius equation for smooth pipe. Use the minor loss coefficient from *Chapter 8 minor loss coefficients worksheet*:  $K_m = 0.5$ . Note: the purpose of this example is to show that there is a relationship between equivalent lengths and minor loss coefficients (one would never actually do this in real life).

The equivalent length is found by setting the minor loss equation equal to the friction loss in the straight pipe equation and solving for  $L/D$ .

$$\begin{aligned} \frac{fL}{D} \frac{v^2}{2g} &= K_{ent} \frac{v^2}{2g} \rightarrow fL = DK_{ent} \rightarrow L = \frac{K_{ent}D}{f} = \frac{0.5D}{f} \\ f &= \frac{0.316}{Re^{1/4}} = \frac{0.316}{(vD)^{0.25}} = \frac{0.316 \left( (1 \cdot 10^{-6})^{0.25} \right)}{(vD)^{0.25}} \\ &= 0.01(vD)^{-0.25} \end{aligned}$$

$$L = \frac{0.5D}{\left( \frac{0.01}{(vD)^{0.25}} \right)} = 50v^{0.25}D^{1.25} = (50)(1^{0.25})(0.1^{1.25}) = 2.8 \text{ m}$$

$$\frac{L}{D} = \frac{0.5}{\left( \frac{0.01}{(vD)^{0.25}} \right)} = 50(vD)^{0.25} = 50(1.0 \cdot 0.1)^{0.25} = 28$$

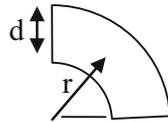
PVC fittings have a particular geometry, and the minor loss coefficients are listed in Table 8.9.

For gravity flow systems, all terms in the Bernoulli equation may be relevant. The goal in setting up these problems is to set up the Bernoulli equation (Eq. 8.2) so that as many terms as possible are known. This is generally accomplished by selecting the open water surface or open discharge at either end of the pipeline as points 1 and 2. The reason is that velocity and pressure at the pond surfaces are zero, thus eliminating those terms.

**Example 8.2** There is a 6 m elevation difference between two reservoir surfaces (Fig. 8.2). They are connected by 2" Class 125 pipe that is 200 m long. The inlet is square-edged and flush with the wall. There is no screen or valve at the inlet. The inlet and outlet pipes are 1.0 m below the water surfaces. Calculate the flow rate and plot the elevation, hydraulic head, and total energy lines over the length of the pipeline and at the inlet and outlet. Set the datum at the elevation of the lower reservoir water surface. Let  $C = 150$ .

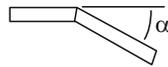
Set the control volume limits at the water surfaces as shown in Fig. 8.3. The minor loss coefficient for a square-edged inlet is 0.5, and for discharge to a reservoir is 1.0. The inside diameter of 2" Class 125 pipe is 56.6 mm (Table 8.5). Substitute terms into the Bernoulli Eq. (8.2).

**Table 8.7** Minor loss coefficients for long-radius butt-welded or flanged 90° elbows



ID (in) (mm)	L/D	½ 12	¾ 18	1 25	1¼ 31	1½ 37	2 50	3 75	4 100	6 150	8 200	12 300	18 450
r/d = 1	20	0.54	0.5	0.46	0.44	0.42	0.38	0.36	0.34	0.3	0.28	0.26	0.24
r/d = 2	12	0.32	0.3	0.28	0.26	0.25	0.23	0.22	0.2	0.18	0.17	0.16	0.14
r/d = 3	12	0.32	0.3	0.28	0.26	0.25	0.23	0.22	0.2	0.18	0.17	0.16	0.14
r/d = 4	14	0.38	0.35	0.32	0.31	0.29	0.27	0.25	0.24	0.21	0.2	0.18	0.17
r/d = 6	17	0.46	0.43	0.39	0.37	0.36	0.32	0.31	0.29	0.26	0.24	0.22	0.2
r/d = 8	24	0.65	0.6	0.55	0.53	0.5	0.46	0.43	0.41	0.36	0.34	0.31	0.29
r/d = 10	30	0.81	0.75	0.69	0.66	0.63	0.57	0.54	0.51	0.45	0.42	0.39	0.36
r/d = 12	34	0.92	0.85	0.78	0.75	0.71	0.65	0.61	0.58	0.51	0.48	0.44	0.41
r/d = 14	38	1.03	0.95	0.87	0.84	0.8	0.72	0.68	0.65	0.57	0.53	0.49	0.46
r/d = 16	42	1.13	1.05	0.97	0.92	0.88	0.8	0.76	0.71	0.63	0.59	0.55	0.5
r/d = 18	45	1.24	1.15	1.06	1.01	0.97	0.87	0.83	0.78	0.69	0.64	0.6	0.55

**Table 8.8** Minor loss coefficients for mitered bends



ID (in) (mm)	L/D	½ 12	¾ 18	1 25	1¼ 31	1½ 37	2 50	3 75	4 100	6 150	8 200	12 300	18 450
α = 0°	2	0.05	0.05	0.05	0.04	0.04	0.04	0.04	0.03	0.03	0.03	0.03	0.02
α = 15°	4	0.11	0.1	0.09	0.09	0.08	0.08	0.07	0.07	0.06	0.06	0.05	0.05
α = 30°	8	0.22	0.2	0.18	0.18	0.17	0.15	0.14	0.14	0.12	0.11	0.1	0.1
α = 45°	15	0.41	0.38	0.35	0.33	0.32	0.29	0.27	0.26	0.23	0.21	0.2	0.18
α = 60°	25	0.68	0.63	0.58	0.55	0.53	0.48	0.45	0.43	0.38	0.35	0.33	0.3
α = 75°	40	1.09	1	0.92	0.88	0.84	0.76	0.72	0.68	0.6	0.56	0.52	0.48
α = 90°	60	1.62	1.5	1.38	1.32	1.26	1.14	1.08	1.02	0.9	0.84	0.78	0.72

**Table 8.9** Equivalent lengths of PVC fittings (From Harvel Plastics PVC and CPVC design manual. Courtesy of Spears Manufacturing)

Equivalent length of pipe (ft)												
Size (in.)	½"	¾"	1"	1¼"	1½"	2"	2½"	3"	4"	6"	8"	
Tee Run	1	1.4	1.7	2.3	2.7	4	4.9	6.1	7.9	12.3	14	
Tee Branch	3.8	4.9	6	7.3	8.4	12	14.7	16.4	22	32.7	49	
90° Ell	1.5	2	2.5	3.8	4	5.7	6.9	7.9	11.4	16.7	21	
45° Ell	0.8	1.1	1.4	1.8	2.1	2.6	3.1	4	5.1	8	10.6	
Equivalent length of pipe (m)												
Size (mm)	12	18	25	32	28	50	62	75	100	150	200	
Tee Run	0.3	0.4	0.5	0.7	0.8	1.2	1.5	1.9	2.4	3.7	4.3	
Tee Branch	1.2	1.5	1.8	2.2	2.6	3.7	4.5	5.0	6.7	10.0	14.9	
90° Ell	0.5	0.6	0.8	1.2	1.2	1.7	2.1	2.4	3.5	5.1	6.4	
45° Ell	0.2	0.3	0.4	0.5	0.6	0.8	0.9	1.2	1.6	2.4	3.2	

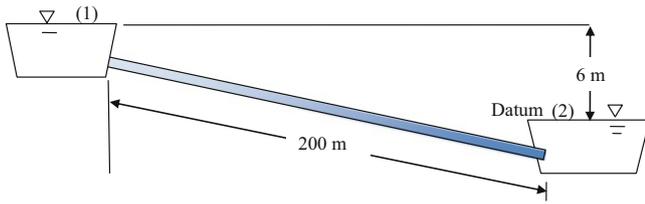


Fig. 8.2 Flow between two reservoirs

$$\begin{aligned} \frac{v_2^2}{2g} + \frac{P_2}{\rho g} + z_2 &= \frac{v_1^2}{2g} + \frac{P_1}{\rho g} + z_1 - h_f - h_m + H_p \frac{0^2}{2g} + \frac{0}{\rho g} + 0 \\ &= \frac{0^2}{2g} + \frac{0}{\rho g} + 6 - h_f - h_m + 0 \rightarrow h_f + h_m = 6 \\ k_1 L \frac{\left(\frac{Q}{C}\right)^{1.85}}{D^{4.87}} + K_{entrance} \frac{v^2}{2g} + K_{exit} \frac{v^2}{2g} &= 6 \end{aligned}$$

Calculate the Hazen-Williams friction loss in terms of velocity because the minor loss term also uses velocity. The entrance minor loss coefficient for a square-edged entrance is 0.5, and the discharge coefficient is 1.0. Convert to appropriate units as follows.

$$\begin{aligned} (1.22 \cdot 10^{10})(200) \frac{\left(\frac{1000 \text{ vA}}{C}\right)^{1.85}}{D^{4.87}} + (0.5 + 1) \frac{v^2}{2g} &= 6 \text{ m} \\ (1.22 \cdot 10^{10})(200) \frac{\left(\frac{1000 \text{ v} \left(\frac{\pi \cdot 0.0566^2}{4}\right)\right)^{1.85}}{56.6^{4.87}} + (1.5) \frac{v^2}{(2)(9.81)} &= 6 \text{ m} \end{aligned}$$

The solution of the equation is  $v = 1.286$  m/sec. Calculate the flow rate  $Q$ , L/hr.

$$\begin{aligned} Q &= (1,000 \text{ L/m}^3)(3,600 \text{ sec/hr})(1.286 \text{ m/sec}) \\ &\left(\frac{0.0566^2 \pi}{4} \text{ m}^2\right) = 11,650 \text{ L/hr} \end{aligned}$$

The *Pipe flow calc* worksheet in the *Chapter 8 Pipes* program performs the calculations in this example (Fig. 8.3). The sum of minor loss coefficients is input in cell B2. The elevation difference between the two reservoir surfaces is input in cell B3. The kinematic viscosity is input in cell B4. The Hazen-Williams C value is input in cell B5 and the surface roughness used to calculate the Darcy-Weisbach  $f$  is input in cell F5: the roughness is specified as zero in this example since the pipe is smooth. The pipe length and inside diameter are input in cells B6:B7. An initial guess for velocity is in cell B10. The program then calculates a velocity in cell B12. The initial solution in cell B12 is generally the correct solution, but the user might want to input this value

in B10 to see if the program converges to a different value. The Darcy-Weisbach solution in column F uses the Hazen-Williams solution as the first guess. The velocity calculated with the Hazen-Williams equation, 1.286 m/sec, is approximately the same as that calculated with the Darcy-Weisbach equation, 1.30 m/sec (cell F14 in Fig. 8.3).

The lower section of the *Pipe flow calc* worksheet (Fig. 8.4) illustrates the elevation, hydraulic and velocity head just outside the pipeline and just inside the pipeline.

The datum is the lower reservoir water surface. Thus, the elevation of the upper pipe inlet is 5 m since it is 1 m below the upper reservoir surface (cell B24). The pressure head at the inlet is 1 m since that is the depth of the pipe in the reservoir (cell B25). The hydraulic head, 6 m, is the sum of elevation and pressure head (cell B26). The velocity head in the upper reservoir is zero since the water is not moving in the reservoir (cell B27). The energy difference between the reservoir and the inside of the pipe is equal to the entrance loss.

$$K_{entrance} \frac{v^2}{2g} = 0.5 \frac{1.30^2}{(2)(9.81)} = 0.043 \text{ m}$$

Thus, the total energy inside the pipeline is 6 m – 0.043 m = 5.957 m (cell C28). The velocity head inside the pipeline is  $1.303^2/(2)(9.81) = 0.087$  m (cell C27). The hydraulic head is the total energy minus the velocity head: 5.957 – 0.087 = 5.87 m (cell C26). The pressure head inside the pipeline is the difference between the hydraulic head and elevation head, 0.87 m (cell C26). Energy is lost due to pipe friction between the upper and lower reservoir. The energy lines from the upper to the lower reservoir are shown in Fig. 8.5.

In Fig. 8.5, the total energy and the hydraulic head are almost indistinguishable; this illustrates the insignificance of velocity head in many irrigation calculations, and it is generally ignored. The energy lines from inside the discharge outlet to the lower reservoir are shown in Fig. 8.6. The energy loss from the pipeline to the lower reservoir is equal to the velocity head, which is dissipated as water enters the lower reservoir. The elevation is –1.0 because the pipe enters the reservoir 1.0 m below the datum (water surface of lower reservoir).

The following example also defines the control volume limits as the water surfaces, but uses equivalent lengths rather than minor losses to solve the problem.

**Example 8.3** A surface irrigation system requires a maximum flow rate of 20 L/sec and is supplied by a distribution box (36" stacked concrete rings). The distance from the

Hazen-Williams calculation of velocity			Darcy-Weisbach calculation of velocity (main pipeline)		
Minor + vel. losses Km	1.5		Minor + vel. losses Km	1.5	
Pot + Press. Energy diff	6 m		Pot + Press. Energy diff	6 m	
Pipe parameters			Pipe parameters		
C	150		Kinematic viscosity	1.00E-06 m <sup>2</sup> /sec	
Pipe length	200 m		Surface roughness K	0.00000 mm K/D = 0.0000000	
Inside pipe diameter	0.0566 m		Pipe length	200 m	
Cross-sectional area	0.0025 m <sup>2</sup>		Inside pipe diameter	0.0566 m	
Adjust cell C10 until cell C13 = cell C3			Cross-sectional area	0.0025 m <sup>2</sup>	
Initial main velocity guess	1.2858 m/sec		Laminar flow?	FALSE	
Initial energy diff	6.00 m		Velocity from Hazen-W	1.286 m/sec	
Adjusted velocity	1.2861 m/sec		Reynolds number	72793	
Calculated energy diff	6.000 m		f factor	0.0192	
Flow rate	3.24 LPS		Initial energy diff	5.8574 m	
Velocity head	0.0843 m		Adjusted velocity	1.303 m/sec	
Input data in white cells			Adjusted f factor	0.0192	
			Adjusted flow rate	3.28 LPS	
			Calculated energy diff	5.99 m	
			Velocity head	0.0865 m	

Fig. 8.3 Upper section of Pipe flow calc worksheet in Chapter 8 Pipes program

Calculation below for pipe between two reservoirs with free flow (graphs are below)						
Upper reservoir			Lower reservoir			
	Outside pipe	Inside pipe	Inside pipe	Outside pipe		
Distance	0.000	0.000	200.000	200.000		
Elevation head	5.000	5.000	-1.000	-1.000		
Pressure head (m)	1.000	0.870	1.000	1.000		
Hydraulic head	6.000	5.870	0.000	0.000		
Velocity head	0.000	0.087	0.087	0.000		
Total energy	6.000	5.957	0.087	0.000		
This section is for plotting						
	Distance	Elevation	Pressure head (m)	Hydraulic head	Velocity head	Total energy
Inside upper reservoir	0	5	1.000	6.000	0.000	6.000
Inside pipe	0	5	0.870	5.870	0.087	5.957
Inside pipe	200	-1	1.000	0.000	0.087	0.087
Inside lower reservoir	200	-1	1.000	0.000	0.000	0.000

The figure shows three graphs illustrating energy head components versus distance (m):

- Graph 1 (Left):** Shows Elevation (blue line), Hydraulic head (red line), and Total energy (green line) from 0 to 200 m. Elevation drops from 5 m to -1 m. Hydraulic head starts at 6.0 m and ends at 0.0 m. Total energy starts at 6.0 m and ends at 0.0 m.
- Graph 2 (Middle):** Focuses on the pipe section (0 to 200 m). Elevation is constant at 5 m. Hydraulic head drops from 5.870 m to 0.000 m. Total energy drops from 5.957 m to 0.000 m.
- Graph 3 (Right):** Focuses on the lower reservoir section (200 m). Elevation is constant at -1 m. Hydraulic head is 1.000 m. Total energy is 0.087 m.

Fig. 8.4 Lower section of Pipe flow calc worksheet in Chapter 8 Pipes program

reservoir to the distribution box is 300 m, and the working water surface elevation of the distribution box is 1.0 m lower than the water surface elevation of the reservoir (Fig. 8.7). Determine the required diameter of the PVC pipe. There is a sharp-edged entrance to the pipe that is flush with the wall in the reservoir (1). Solve the problem in terms of equivalent length rather than minor loss coefficients. Use Hazen-Williams equation and let  $C = 145$ . Use Class 100 pipe.

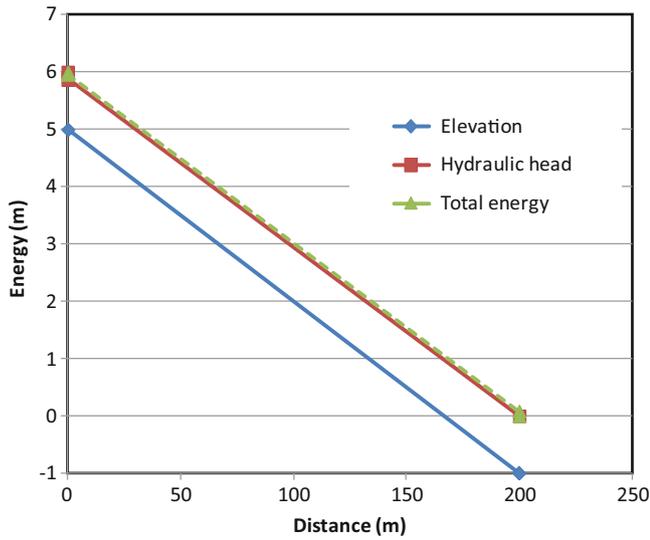
The  $L/D$  ratio for the entrance loss is 28. Because the exit loss  $K_m$  is twice the entrance  $K_m$ , assume that the exit  $L/D$  is 56. The pipe size is unknown, so make an initial guess of

100 mm. Thus the equivalent length of the entrance plus exit losses is  $2.8 m + 5.6 m = 8.4 m$ . Therefore, the length of pipe used in the Hazen-Williams equation is 308.4 m.

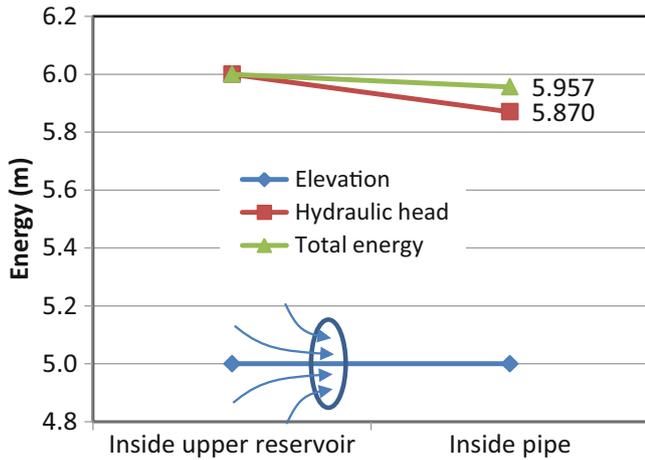
By choosing the water surfaces as the control volume boundaries and by using equivalent lengths, Bernoulli's equation reduces to

$$h_f = 1 m = kL \left(\frac{Q}{C}\right)^{1.85} / D^{4.87}$$

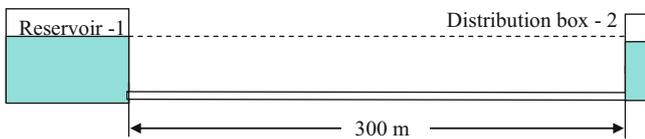
Rearrange the equation and solve for the required pipe diameter,  $D$ .



**Fig. 8.5** Energy lines between upper and lower reservoirs



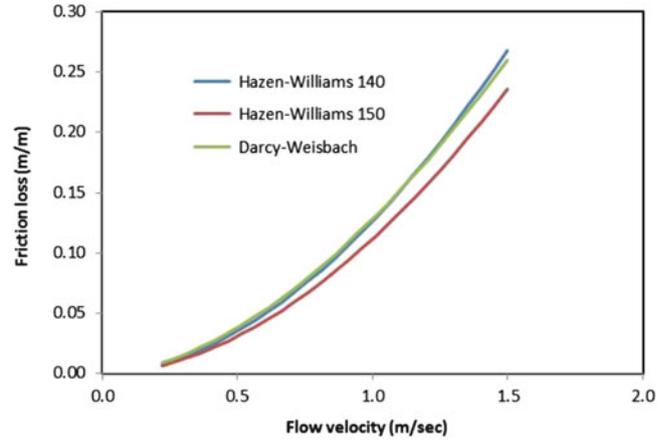
**Fig. 8.6** Energy change from inside pipe to lower reservoir



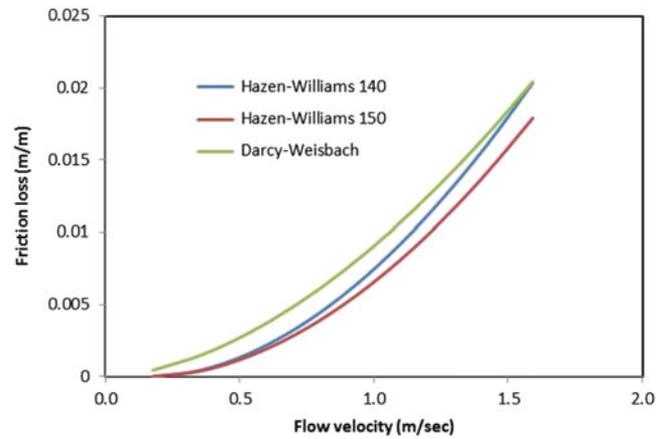
**Fig. 8.7** Irrigation distribution box supplied by reservoir

$$D = \left( kL \frac{Q^{1.85}}{h_f} \right)^{1/4.87}$$

$$= \left( (1.22 \cdot 10^{10}) (308.4) \frac{(20)^{1.85}}{1} \right)^{1/4.87} = 180 \text{ mm}$$



**Fig. 8.8** Comparison of friction loss equations (m/m) for 12 mm ID smooth pipe



**Fig. 8.9** Comparison of friction loss equations for 100 mm ID smooth pipe

The closest available manufactured PVC pipe inside diameters are 160 and 208 mm (Table 8.2). In problems like this, calculated pipe diameter must be rounded up to the next larger inside diameter, 208 mm (8 in). The calculation can then be refined based on the equivalent length for 208 mm ID pipe. The equivalent length for the 208 mm pipe is  $0.208(28 + 56) = 17.5$  m, which results in a total length of 317.5 m. The required pipe diameter with  $L = 317.5$  m is 181 mm instead of 180 mm as calculated previously, so there is no change in the selection of 208 mm ID pipe for the project.

There is often uncertainty over the correct C value to select for smooth pipe. Most commonly, the Hazen-Williams C for smooth PVC is assumed to be 140; however, 150 is also recommended in some cases. Hazen-Williams C = 140 generally results in a close agreement with the Darcy-Weisbach equation (Figs. 8.8 and 8.9). Figure 8.8 shows that the Darcy-Weisbach equation and the Hazen-Williams equation with C = 140 have almost the same

**Table 8.10** Temperature-pressure derating for PVC fittings and pipes (Courtesy of Spears Manufacturing Company)

Temp (°F)	73	80	90	100	110	120	130	140	150
Temp (°C)	23	27	32	38	43	49	54	60	66
% PR	100 %	90 %	75 %	62 %	50 %	40 %	30 %	22 %	0 %

friction loss for 12 mm ID tubing, whereas the Hazen-Williams equation with  $C = 150$  has a significantly lower estimate of friction loss. The Hazen-Williams equation has reasonable agreement with the Darcy-Weisbach equation up to 1.5 m/sec velocity, which is generally the maximum allowable velocity for PVC pipe. Engineers often use  $C = 130$  or less in order to account for gradual degradation of pipe or crusting of interior pipe surfaces.

### Pressure Rating

The pressure rating is calculated based on the dimension ratio, which is the ratio of the wall thickness to the outside diameter.

$$DR = \frac{OD}{T} \tag{8.10}$$

where

OD = average outside diameter, mm

T = minimum wall thickness, mm.

The pressure rating of PVC pipe is calculated as follows:

$$PR = \frac{2S}{DR - 1} \tag{8.11}$$

where

PR = pressure rating of pipe, kPa

S = hydrostatic design stress, kPa.

The hydrostatic design stress is the maximum tensile stress due to internal hydrostatic pressure that can be applied continuously with a high degree of certainty that failure will not occur (Cuenca 1989). The hydrostatic design stresses for PVC 1120 and polyethylene (PE 3408) are 13.8 and 5.5 MPa, respectively.

PVC pipe manufacturers are required to pressure test pipe samples at 4 times the rated pressure for five minutes. They also perform a burst pressure test of PVC pipes at five times the rated pressure, and they perform semiannual tests of 24 hours at 3.5 times the rated pressure.

**Example 8.4** Calculate the pressure rating for 4" Class 200 PVC 1120 pipe.

The standard dimension ratio (SDR) for Class 200 pipe is 21.

$$PR = \frac{2S}{DR - 1} = \frac{(2)(13.8)}{21 - 1} = \frac{27.6}{20} = 1.38 \text{ Mpa}$$

$$= 1,380 \text{ kPa}$$

This same pressure rating (1,379 kPa) is listed in Table 8.2 for Class 200 pipe.

Pressure ratings for PVC pipe and fittings are calculated at 73 °F (23 °C). If water temperature is hotter, then plastic becomes weaker, and the pressure rating decreases. Pressure ratings are listed as a percentage of the maximum pressure rating in Table 8.10. For example, at 43 °C, 125 PSI pipe would have a pressure rating of 62 PSI.

The weakness in PVC pipe networks is at the fittings. While plastic pipe is symmetrical and pressure is directed outward, leading to uniform stress in the plane of the pipe wall, fittings are not symmetrical and stresses are directed at angles in the fitting. Thus, even though fittings may be thicker than the pipe, they are generally weaker. Keller/Bleisner Engineering prepared Table 8.11, which gives the pressure rating for glued and threaded fittings. Although manufacturing companies may report pressure ratings for PVC fittings, there is no national or professional standard by which these pressure ratings are calculated so they are cannot be relied on.

**In-class Exercise 8.4** Discuss the impact of fitting strength (Table 8.14) on design of pipelines and selection of fittings.

### Transient Flow, Surge Pressure, and Air

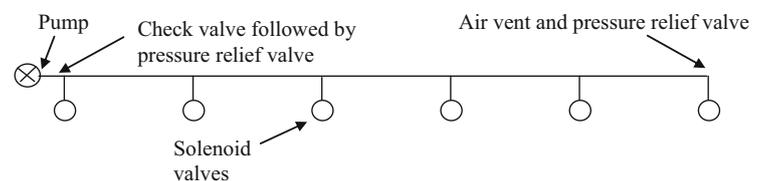
After installation of a long 250 mm diameter (10") irrigation pipeline with a closed valve at the end, a worker suddenly opened the inlet valve, and a short time later, the pipe blew a hundred feet out of the trench (story from an old irrigation engineer). If moving water suddenly stops, then Newton's second law states that the resultant force is equal to the rate of change of momentum,  $F = ma$ . When the worker opened the valve, the water rushed down the pipe at high velocity until it reached the end and suddenly stopped. This resulted in a huge force and resultant pressure surge (water hammer). Pressure waves traveled along the pipeline at extremely high velocity, and blew the entire pipeline out of the trench.

Irrigation system operators should slowly fill a pipe with water, and then, only when the pipe is full, completely open

**Table 8.11** Pressure ratings for pipe and fittings (at 73 °F) (Courtesy of Spears Manufacturing Company. Prepared for Spears by Keller/Bleisner Engineering). SR refers to Special Reinforced plastic thread

Nom. Size (in)	Schedule 40						Schedule 80					
	Pipe		Solvent cemented joint and Spears SR threaded joint		Standard threaded joint		Pipe		Solvent cemented joint and Spears SR threaded joint		Standard threaded joint	
	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa
¼	780	5.4	468	3.2	390	2.7	1130	7.8	678	4.7	565	3.9
½	600	4.1	360	2.5	300	2.1	850	5.9	510	3.5	425	2.9
¾	480	3.3	288	2.0	240	1.7	690	4.8	414	2.9	345	2.4
1	450	3.1	270	1.9	225	1.6	630	4.3	378	2.6	315	2.2
1 ¼	370	2.6	222	1.5	185	1.3	520	3.6	312	2.2	260	1.8
1 ½	330	2.3	198	1.4	165	1.1	470	3.2	282	1.9	235	1.6
2	280	1.9	168	1.2	140	1.0	400	2.8	240	1.7	200	1.4
2 ½	300	2.1	180	1.2	150	1.0	420	2.9	252	1.7	210	1.4
3	260	1.8	156	1.1	130	0.9	370	2.6	222	1.5	185	1.3
4	220	1.5	132	0.9	110	0.8	320	2.2	192	1.3	160	1.1
5	190	1.3	114	0.8	95	0.7	290	2.0	174	1.2	145	1.0
6	180	1.2	108	0.7	90	0.6	280	1.9	168	1.2	140	1.0
8	160	1.1	96	0.7	80	0.6	250	1.7	150	1.0	125	0.9
10	140	1.0	84	0.6	70	0.5	230	1.6	138	1.0	115	0.8
12	130	0.9	78	0.5	65	0.4	230	1.6	138	1.0	115	0.8

**Fig. 8.10** Irrigation main line with solenoids, air vents and pressure relief valves



the valve in order to prevent catastrophes like this. ASAE Standard 376.1 recommends a filling velocity of no more than 0.1 m/sec. This fill velocity would result in a 10 minute filling time for a 300 m length pipe (ASAE 376.1). This principle also applies to centrifugal pump start up. The pump should be started with the discharge valve closed; then the valve should be cracked open, and water should be allowed to slowly fill the pipe before the valve is fully opened. Pumps should not be allowed to run for an extended period against a completely closed valve because pumps are water cooled.

Because valves should always be opened or closed slowly, gate valves or gear operated butterfly valves are preferable on large diameter pipelines because these valves require several turns to open, unlike a ball valve or lever operated butterfly valve. Large pipeline designs include calculation of the minimum allowable valve closure or opening time, and valves are generally designed so that it is impossible for a worker to open or close a valve too quickly.

Although the pipe blowout was caused by suddenly opening an inlet valve, most pressure surges in pipelines are caused by suddenly closing a valve at the end of the pipeline. These surges are accompanied by a banging sound called water hammer. The banging sound is caused by pressure waves

expanding and contracting the pipe as they move back and forth in the pipeline. Even if they do not burst the pipe, repeated incidences of pressure surge may eventually lead to pipe or joint failure. Irrigation systems typically have several electronic solenoid valves along the mainline that activate irrigation zones at a signal from of the irrigation controller (Fig. 8.10). These valves typically take a few seconds to close although they can suddenly finish closing during the last phase of valve closure and cause water hammer.

Pressure relief valves allow water to escape if the pipe pressure surges beyond a given threshold; they are typically placed just before the last valve on the mainline (Fig. 8.10), and just after a check valve at the pump. The check valve helps prevent water hammer the next time the system is turned on because it prevents the mainline from draining when the system is shut down. Pressure relief valves should be set to release water at no more than 5 PSI (35 kPa) greater pressure than the design operating pressure of the system (ASAE S376.1). Air vents are also placed on irrigation mainlines; they allow air to escape from the pipe as the pipe is filling, or they allow air to replace the water in the pipe when the pipe is draining. They have a small buoyant ball that is pressed against the upper vent hole when the pipe has water, but the ball drops down and allows air to escape when the pipe has air.

Although pressure relief valves and air vents should always be installed on large agricultural irrigation pipelines, the irrigation system should also be designed with the criterion that the operating pressure plus possible surge pressure (as if the valve were not there) should be no greater than 150 % of the rated pipe pressure (Harvel Plastics Engineering Guide). This number, 150 %, is based on an expected number of surges at which pipe failure will occur at a given surge pressure percentage over the rated pressure. The expected surge pressure in the event of rapid valve closure or other causes is a function of design water velocity. The rule of thumb is that water velocity in closed end PVC pipes should be kept below 1.5 m/sec (5 ft/sec) in order to prevent excessive surge forces. The 1.5 m/sec rule does not apply to pipes that have an open discharge.

The point in the irrigation system that has the highest flow velocity is most susceptible to water hammer. If water hammer is noticed (banging sound) in a landscape irrigation pipe system, then troubleshooting the water hammer problem should be conducted as follows (Stryker 2015): (1) check for under-designed pipe diameters (>1.5 m/sec velocity), (2) check for more than one zone turning on at a time resulting in double the design flow rate, (3) check for valves closing too quickly, which may be solved by installing valves that close slowly, (4) check for lack of pressure relief valves and air vents at the end of mainlines, (5) and check for pressure surges caused by abrupt changes in direction such as at 90° elbows..

In order to prevent water hammer and sudden changes in velocity, the maximum recommended pipe velocity in PVC pipe is 1.5 m/sec (5 ft/sec). The minimum acceptable pipe diameter based on the 1.5 m/sec rule is

$$D = \sqrt{\left(\frac{4Q}{v_{\max}\pi}\right)} \tag{8.12}$$

where

Q = flow rate, m<sup>3</sup>/sec

v<sub>max</sub> = maximum allowable velocity, 1.5 m/sec for PVC pipe

D = pipe diameter, m.

**Example 8.5** Find the minimum required PVC pipe inside diameter for flow rate 95.3 L/sec.

$$D = \sqrt{\left(\frac{4Q}{v_{\max}\pi}\right)} = \sqrt{\left(\frac{(4)(0.0953 \text{ m}^3/\text{sec})}{(1.5 \text{ m/sec})\pi}\right)} = 0.284 \text{ m}$$

$$= 28.4 \text{ cm}$$

The expected surge pressure, the increase in pressure over the operating pressure, is proportional to the product of the design water velocity and the pressure wave velocity.

$$\Delta H = \Delta v \frac{a}{g} \tag{8.13}$$

where

Δv = change in water velocity (design water velocity), m/sec

a = pressure wave velocity, m/sec

g = gravity, 9.8 m/sec<sup>2</sup>

ΔH = pressure surge, m.

The pressure wave velocity is a function of pipe material and the water properties.

$$a = \frac{\left[\frac{K}{\rho}\right]^{0.5}}{\left[1 + \left(\frac{K}{E}\right)\left(\frac{D}{t}\right)C_1\right]^{0.5}} \tag{8.14}$$

where

K = bulk modulus of elasticity of water, 2.2 \* 10<sup>9</sup> Pa at 25 °C

ρ = density of water, 1,000 kg/m<sup>3</sup>

D = inside pipe diameter, m

t = pipe wall thickness, m

C<sub>1</sub> = pipe support coefficient

E = pipe modulus of elasticity (Table 8.15), Pa.

The pipe support coefficient, C<sub>1</sub>, is a function of how the pipe is anchored (Cuenca 1989) by thrust blocks or other constraints.

C<sub>1</sub> = 1.25μ, pipes anchored at one end,

C<sub>1</sub> = 1μ<sup>2</sup>, pipes anchored at both ends,

C<sub>1</sub> = 1.0, pipes with expansion joints.

where

μ = Poisson ratio (Table 8.12).

Typically, irrigation pipelines are buried and constrained at both ends so C<sub>1</sub> = 1 – μ<sup>2</sup>. However, gasketed – bell

**Table 8.12** Bulk modulus of elasticity and Poisson ratio for pipe materials (After Cuenca 1989)

Material	Modulus of elasticity (E) (MPa)	Poisson's ratio (μ)
Asbestos-cement	2.07 * 10 <sup>4</sup>	0.2
Cast iron	1.03 * 10 <sup>5</sup>	0.29
Ductile iron	1.65 * 10 <sup>5</sup>	0.29
Polyvinyl chloride	2.76 * 10 <sup>3</sup>	0.46
Polyethylene	6.89 * 10 <sup>2</sup>	0.40
Steel	2.06 * 10 <sup>5</sup>	0.30

ended pipe is not glued but slipped together with lubricated gaskets. The gasketed joints serve as expansion joints so  $C_1$  is 1.0 for gasketed bell-ended pipe.

**Example 8.6** Calculate the surge pressure in a 21.9 cm OD (8.625") 6 gage steel pipe buried in the ground and carrying a design flow velocity of 2.3 m/sec. The pipe is anchored at both ends.

$$C_1 = 1\mu^2 = 10.3^2 = 0.91$$

Calculate the velocity of pressure wave: 6 gage (0.2" thick walls) tubing has an inside diameter of 8.625 - (0.2) (2) = 8.225".

$$a = \frac{\left[\frac{K}{\rho}\right]^{0.5}}{\left[1 + \left(\frac{K}{E}\right)\left(\frac{D}{t}\right)C_1\right]^{0.5}} = \frac{\left[\frac{2.2 \times 10^9}{1,000}\right]^{0.5}}{\left[1 + \left(\frac{2.2 \times 10^9}{2.06 \times 10^{11}}\right)\left(\frac{8.225}{0.2}\right)0.91\right]^{0.5}}$$

$$= 1,253 \text{ m/sec}$$

Calculate the magnitude of pressure wave.

$$\Delta H = \Delta v \frac{a}{g} = (2.3) \left(\frac{1,253}{9.8}\right) = 294 \text{ m}$$

Because the water bulk modulus of elasticity is a constant (with slight changes due to temperature), Eqs. 8.13 and 8.14 can be simplified. For each standard dimension ratio and pipe material, the surge pressure is directly proportional to water velocity. The surge pressure coefficients for PVC pipe at SDR 21, 26, 32.5, and 41 are 16.1-, 14.4-, 12.9-, and 11.4- (psi/(ft/sec)) or 364-, 326-, 292-, and 258- (kPa/(m/sec)).

$$\Delta H = C_{sp} v \quad (8.15)$$

where

$v$  = design velocity, m/sec

$\Delta H$  = surge pressure, m

$C_{sp}$  = surge pressure coefficient, sec.

If the total pressure in the pipe under surge (surge pressure plus design operating pressure) is greater than 1.5 \* rated pressure (left column of Table 8.5 or calculated with Equation 8.11), then the pipe diameter should be increased in order to increase flow velocity, or the wall thickness should be increased to increase the pressure rating.

**Example 8.7** A pipe must carry a flow rate of 286 L/sec at a design pressure of 530 kPa. Calculate the minimum acceptable pipe diameter according to the 1.5 m/sec rule. Use Schedule 40 PVC pipe. Fittings have a pressure rating that is 80 % of pipe pressure rating, and water temperature is

27 °C. The pipe support coefficient,  $C_1$ , is 1.0. Select a pipe size based on surge pressure.

As a first guess, find the minimum acceptable diameter based on the 1.5 m/sec rule.

$$D = \sqrt{\left(\frac{4Q}{v_{\max}\pi}\right)} = \sqrt{\left(\frac{(4)(0.286 \text{ m}^3/\text{sec})}{(1.5 \text{ m/sec})\pi}\right)} = 0.497 \text{ m}$$

$$= 497 \text{ mm}$$

The next larger pipe size is 21-inch (502 mm ID) SCH 40 pipe (Table 8.3).

$$v = \frac{Q}{A} = \frac{0.286 \text{ m}^3/\text{sec}}{(0.502 \text{ m}/2)^2\pi} = 1.44 \text{ m/sec}$$

The rated pressure of 21-inch pipe is 830 kPa; however, this pressure must be reduced because of fittings and high temperature. At 27 °C, the plastic has a pressure rating that is 90 % of the reported pressure rating. Fittings have a pressure rating that is 87 % of the pipe pressure rating.

$$\text{Maximum design pressure} = (830)(0.9)(0.87) = 650 \text{ kPa}$$

The maximum allowable design + surge pressure is equal to 150 % of the rated pressure.

$$\text{Maximum surge + design pressure} = 650(1.5) = 975 \text{ kPa}$$

The dimension ratio for 21-inch SCH 40 pipe is 34 (Table 8.3). The surge pressure coefficient can be found by interpolation between surge pressure coefficients for Class rated pipe.

$$C_{sp} = 258 + (292 - 258) \frac{41 - 34}{41 - 32.5} = 286 \text{ kPa}/(\text{m}/\text{sec})$$

$$\Delta H = C_{sp} v = (286 \text{ kPa}/(\text{m}/\text{sec}))(1.44 \text{ m/sec})$$

$$= 412 \text{ kPa}$$

The design pressure + surge pressure (530 kPa + 412 kPa = 942 kPa) does not exceed the maximum allowable design + surge pressure of the pipe (975 kPa).

Thus, 21-inch pipe is acceptable. Check the next smaller pipe diameter, 20-inch.

$$v = \frac{Q}{A} = \frac{0.286 \text{ m}^3/\text{sec}}{(0.478 \text{ m}/2)^2\pi} = 1.59 \text{ m/sec}$$

$$\Delta H = C_{sp} v = (286)(1.59) = 455 \text{ kPa}$$

The design pressure + surge pressure (455 kPa + 530 kPa = 985 kPa) **exceeds** the maximum allowable design + surge pressure of the pipe (975 kPa). Thus, 20-inch pipe is not acceptable for this application, and the 1.5 m/sec rule led to

the correct pipe selection in this case. The *Surge* worksheet in the *Chapter 8 Pipes* program performs the calculations shown in this example.

Minimum valve closure time can be calculated based on the length of the pipeline and the pressure wave velocity. The valve should not finish closing before the pressure wave reaches the other end of the pipeline and returns. Thus, minimum valve closure time is calculated as follows:

$$t_c = \frac{2L}{a} \quad (8.16)$$

where

$t_c$  = valve closure time, sec

$L$  = pipeline length, m

$a$  = pressure wave velocity, m/sec.

During operation, air tends to collect at high points in pipelines. Air pockets in pressurized pipelines can increase friction/pressure loss. Even worse, air pockets at high points in low-pressure gravity-flow pipelines (as in Example 8.4) can stop flow completely. For pipelines that travel over high and low points, air vents should be placed at high points in order to remove accumulated air.

Due to high tensile strength, PVC pipes are able to resist breakage under high positive pressure. However, low-pressure gravity flow pipes (“Class 63” and “100 ft head”) with thin walls cannot resist collapse under a negative pressure of one atmosphere (zero gauge pressure). If water drains from the lower end of a sloping pipe, and the upper end does not have a vent that allows air to enter the pipe, then a vacuum forms because nothing replaces the drained water. In order to prevent development of a vacuum, all high points on pipelines (closed ends or middle) should have a vacuum relief valve to prevent the formation of a vacuum when the pipe is drained. Vacuum relief valves have a vent at the top of the valve that opens when a vacuum forms. Because air vents and vacuum relief valves are both needed at high points, combination air vent/vacuum relief valves are manufactured.

ASAE Standard S376.1 specifies air vent dimensions (diameter of the threaded connection to the PVC pipe) for low-pressure and high-pressure pipelines (Table 8.13). Air vents and vacuum relief valves on gravity flow pipelines are larger because prevention of vacuum and air locks is more important so air must be allowed to rush into or out of the pipe more quickly.

The *Chapter 8 Pipes* program includes two worksheets that model the process of a suddenly filling pipe. This would be the case if someone suddenly opened a valve and allowed water to flow into an unfilled pipe (this should never be done except in a computer model). The first worksheet has no air vent at the end of the pipe so the pressure builds at the end of the pipe as the pipe fills and the air is compressed (Fig. 8.11).

**Table 8.13** Air vent and vacuum relief valve standards (After Cuenca 1989)

High pressure systems			
Pipe diameter		Valve threaded connection diameter	
≤102 mm	(≤4")	13 mm	(0.5")
125–200 mm	(5–8")	25 mm	(1")
250–500 mm	(10–20")	51 mm	(2")
≥525 mm	(≥21")	0.1 * pipe diameter	
Low pressure systems			
Pipe diameter		Valve threaded connection diameter	
≤150 mm	(≤6")	51 mm	(2")
200–250 mm	(8–10")	76 mm	(3")
≥300 mm	(≥12")	102 mm	(4")

The rate of pipe filling is based on the pressure (force) differential between the advancing front of the water and the pipe inlet, and the resistance to flow of pipe friction ( $h_f$ ). The model assumes that the pipe is initially filled to a length that is specified in cell B4. The unfilled length of the pipe is specified in cell B5. The rate that water increases or decreases in velocity in the pipe is based on the force differential divided by the mass of water in the pipe. Thus, the pressure and velocity fluctuate over time as the air pressure fluctuates in the closed end of the pipe. Note that this worksheet and the *air vent surge* worksheet are only for demonstration and are not calibrated for use in actual engineering designs.

The *air vent surge* worksheet (Fig. 8.12) assumes that the air is released as the pipe fills but that there is no pressure relief valve placed on the pipe. Thus, the only resistance to pipe filling is pipe friction ( $h_f$ ). The velocity initially increases to a maximum and then decreases as the length of water in the pipe increases. The surge pressure is then calculated at the velocity of the water when it reaches the end of the pipe with Eqs. 8.13 and 8.14.

## Installation

Irrigation system installation should include the following steps:

1. Laying out the locations of the trenches and possibly marking them with paint on the ground.
2. Verifying the locations of existing utilities and irrigation pipes.
3. Trenching.
4. Gluing the mainline pipe outside the trench.
5. Running water through the mainline in order to flush out dirt from the pipe.
6. Installing valves on the mainline.
7. Placing solenoid valve wires into the trench.
8. Dropping the mainline into the trench on top of the solenoid valve wires.
9. Gluing submains and laterals outside the trench.

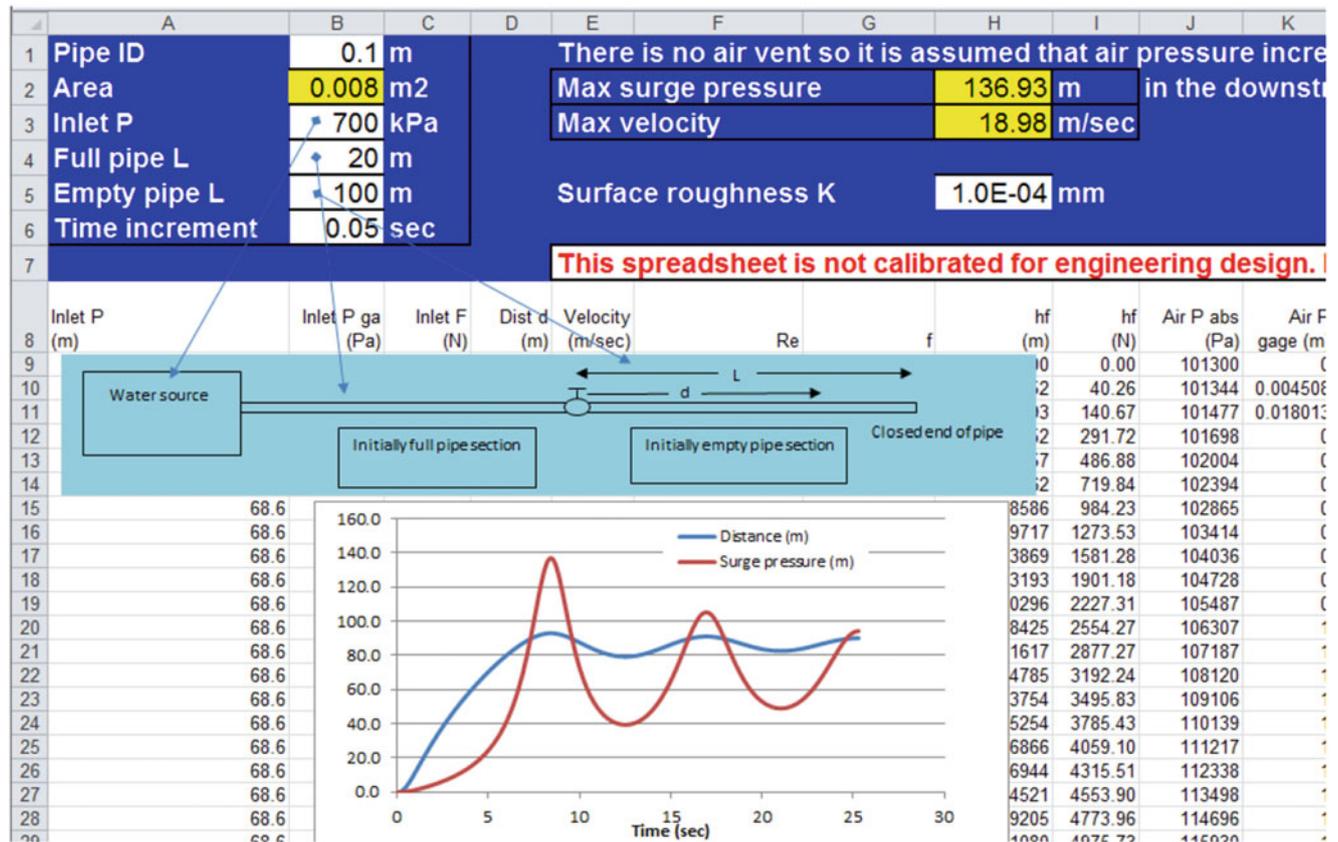


Fig. 8.11 Surge calculator for pipe filling in *No air vent* worksheet

- Attaching submains and/or laterals to the mainline solenoid valves.
- Dropping the submains and laterals into trenches.
- Running water through the submains and/or laterals in order to flush out dirt from the pipes.
- Attaching emitters, tubing, or sprinklers to the laterals.
- Pressure checking the entire system for leaks.
- Backfill the trench.

### Trenching, Installation, Pipe Strength, and Corrosion

Trench locations should be carefully laid out and verified with all stakeholders: it is very hard to move a trench. In some installations, it is helpful to mark the locations of trenches with spray paint in order to guide the trencher operator. Some trenchers are guided by lasers or GPS and do not require marked trench locations. Urban installations should be “Blue Staked:” utility companies mark the locations of all underground utilities. Neglecting this step can be hazardous and expensive. For example, trenching through 440 V electrical power lines will kill the tractor operator, and cutting through fiber optic communication lines can result in a fine of hundreds of thousands of dollars. Even marked utility

locations may not be correct, especially in the case of old utilities such as 440 V power lines. New installations have foil tape several cm above utility lines; thus, the location of the line can be found and exposed with a shovel before the trencher moves across the line. In agricultural fields, it is often necessary to find existing irrigation pipes in fields. Some specially endowed individuals can walk through a field with L-shaped metal rods in each hand pointing straight ahead; when the person crosses a buried pipe, the metal rods cross over each other. It is thought that lines of magnetic or gravitational force cause the rods to change direction. Others ascribe this talent to witchcraft and don’t want to have anything to do with it. Regardless of the reason, it does work.

There are many types of trenchers, backhoes, and pipe pullers. Efficient installation of irrigation or pipe systems requires an adequately sized machine. An inadequately sized trencher is dangerous and can result in the crew and trencher taking an extra week to install the job, which is not profitable. Making bids on excavation and trenching in potentially rocky or otherwise difficult soils should always be preceded by taking extensive soil cores at the site, or the bidders should assume the worst possible conditions (for example, rock) and bid a trencher accordingly. Recommended trench widths and depths are specified in ASAE standard S376.1. In general, the minimum trench width is 30 cm (12 in.) wider than the pipe

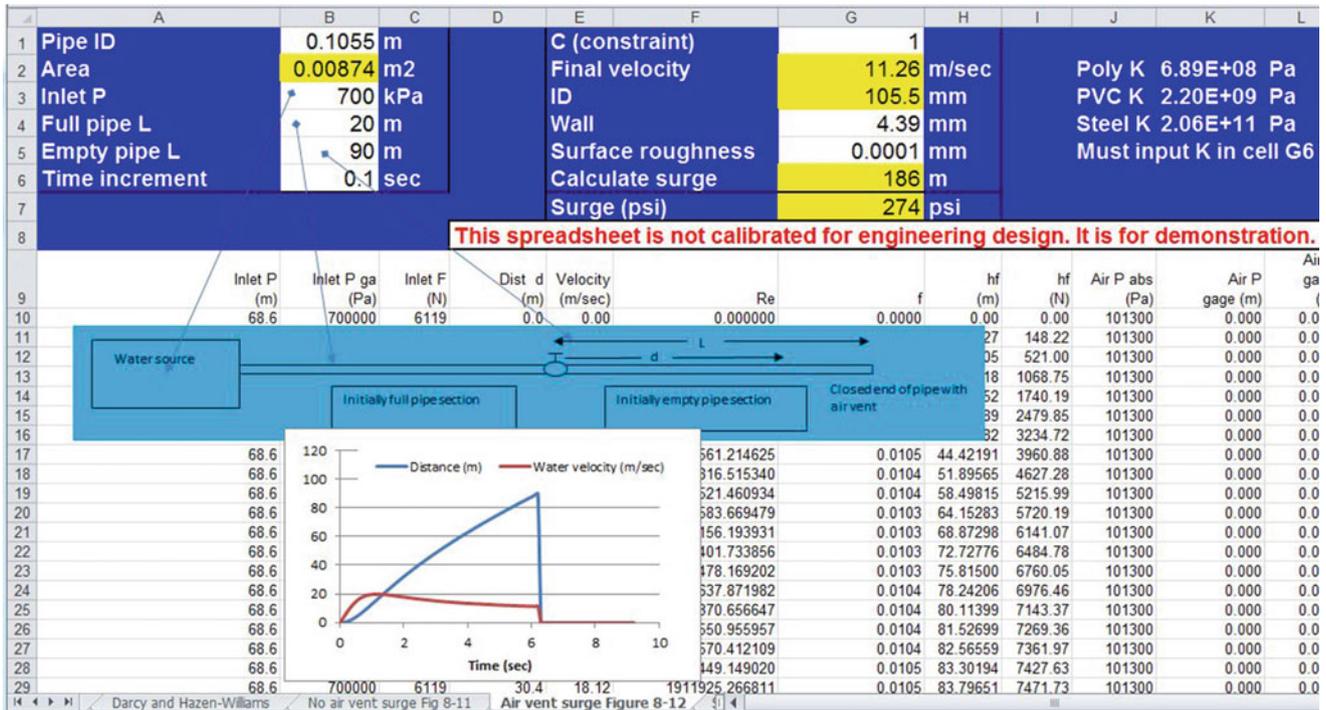


Fig. 8.12 Air vent surge worksheet

diameter, and the maximum trench width is 75 cm (30 in.) for pipes less than 37 cm (15 in.).

In the case of low-pressure, gravity-flow pipelines that are installed on nearly level ground, the grade of the pipe and the trench bottom should be carefully surveyed or the pipe should be installed with a laser-guided machine in order to avoid high points that would develop air pockets that stop the flow of water.

Pipe structural strength and stresses due to overload, sharp objects, and bends in pipe must be considered when selecting the correct backfill material. In general, small diameter, pressurized (Class 125 and heavier) irrigation pipes installed in shallow trenches with 18" (45 cm) of backfill over the pipe are structurally stable. The soil removed from the trench can be placed back over the pipe. However, large rocks should not be placed back in the trench. Larger pipes, lower pressure ratings (thinner wall), and deeper pipes are more susceptible to structural failure. Pipes buried deep in the ground should be "shaded" with sand backfill around the pipe. No rocks should be backfilled over the sand because the rocks will migrate downward in the trench over time.

Pipes can collapse due to an internal vacuum or external load. Pipe resistance to collapse is based on the ratio of pipe wall thickness to diameter. The maximum differentials between pipe internal pressure and external load (critical collapse pressure rating) for SDR 41 (Class 100), SDR 26 (Class 160), and SDR 21 (Class 200) are 17-, 74-, and 126-psi (120-, 510-, and 870-kPa) for all pipe diameters. Schedule 40 pipe, which does not have a constant SDR for

different diameters, has a maximum critical collapse pressure rating of 451 psi at 2.5" diameter and decreases to 25 psi at 24" diameter. Vehicular traffic or heavy soil loads can damage pipes. Tractors with large tires and uniform load distribution can be driven over lower SDR PVC pipes or pipe trenches without breaking the pipes. However, trucks with small tires, if they are oriented in the same direction can collapse large diameter pipes buried in trenches. The soil overburden pressure is added to the vehicle load in order to find the total pressure on the pipe.

**Example 8.8** A 6" Class 100 pipe is buried in a trench with 18" cover over the pipe. After a storm, a farm vehicle with small tires is driven over the wet field. A tire is oriented in the same direction as the trench and applies a load of 1,000 lbs over a 1 ft length of the empty pipe. The dry bulk density of the soil is 1.3 g/cm<sup>3</sup>, and the soil has a 40 % by volume water content. Determine whether the pipe will collapse.

$$\begin{aligned} &\text{Calculate the soil density under wet conditions. } 1.3 \text{ g/cm}^3 \\ &+ 0.4 \text{ g/cm}^3 \\ &= 1.7 \text{ g/cm}^3. \end{aligned}$$

Calculate the soil overburden pressure.

$$\begin{aligned} 1.7 \text{ g/cm}^3 &= 0.61 \text{ lb/in}^3 \\ (0.61 \text{ lb/in}^3)(18 \text{ in}) &= 1.1 \text{ psi} \end{aligned}$$

Calculate the vehicle load and total load.

$$1,000 \text{ lb}/((12 \text{ in})(6 \text{ in})) = 13.9 \text{ psi}$$

$$13.9 \text{ psi} + 1.1 \text{ psi} = 15 \text{ psi}$$

The critical collapse pressure is 17 psi so the pipe would probably not collapse under the load. However, if the load is suddenly placed on the pipe (+ momentum force), then it may collapse.

Because collapse is based on the difference between internal and external pressure, a filled pipe is more resistant to collapse than an empty pipe.

At points along the pipeline with changes in direction, momentum change can exert forces on the pipe that may cause the pipe to come apart. For this reason, concrete is poured into the trench around the pipe at these locations. Thrust blocks give pipe the strength to resist deformation or breakage. Required dimensions for thrust blocks are given in ASAE/ASABE Standard S376.1. Thrust blocks are especially important for gasketed pipe that does not have a fixed connection.

One of the new innovations in trenching is a machine that shanks in pipe or drip tubing without digging a trench. When these machines are used, the pipe is glued together prior to installation.

Steel pipes are subject to corrosion. In order to prevent corrosion, the pipes are often coated with cement or epoxy on the inside and outside of pipes, and they should have cathodic protection to prevent electrons from being transferred from the pipe to the soil. Electrical cells that transfer electrons from pipe to the soil (iron is oxidized) can be microscopic in size or up to several miles long if the pipe is laid in different types of soils. A small investment in cathodic protection can essentially preserve steel pipe indefinitely, whereas ignoring cathodic protection may cause the pipe to degrade within several years.

In cold regions, the required depth of cover over pipes is based on the depth of soil freezing in winters. In cold areas where it is difficult to make deep trenches through bedrock, pipes may be installed at a shallower depth, but they must be blown out with air before winter. One of the concerns with thermoplastic pipeline installation is expansion due to changes in temperature. The coefficient of thermal expansion for PVC pipe is  $2.9 \times 10^{-5} \text{ ft}/\text{ft}/^\circ\text{F}$  ( $5.2 \text{ m}/\text{m}/^\circ\text{C}$ ).

Standard PVC pipe is not resistant to UV degradation. Thus, if the pipe is left in the sun for an extended period (a few months) the pipe turns brown and is degraded because UV light breaks polymer bonds in the plastic. Painting the outside of pipe prevents UV degradation. Although it is standard practice to use steel pipe for all above ground pipe installations, some farmers successfully use PVC above ground in certain applications such as valve stations by painting the PVC pipe with latex or acrylic paint. Unlike

PVC pipe, polyethylene pipe is not sensitive to UV degradation and is often used for above ground applications.

## Pipe Connections

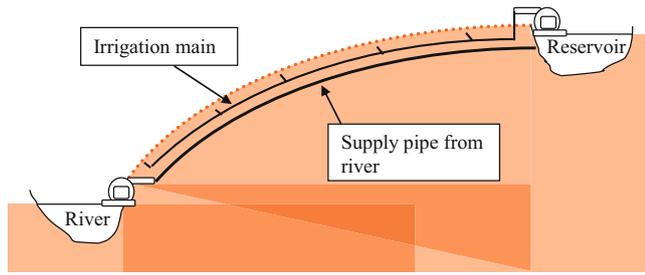
Connecting pipe sections correctly is a critical step in producing a high quality irrigation system because leaks and pipe failure generally occur at joints. Large diameter polyethylene pipe is fused together by placing the ends together and melting the pipes together. The joint is as strong as the pipe. PVC joint types include glued, threaded, flanged, and gasketed/bell-ended connections. Glued PVC fittings and pipes are connected by softening the outer pipe surfaces with a primer or cement and then fusing the surfaces with cement. Primer is generally used to soften large diameter PVC pipe prior to cement application while a more active cement (blue glue) can be used to both soften and cement small diameter pipes ( $<2''$ ). There are many solvent cement formulations. Higher viscosity and slower drying cements are used for large diameter pipe. Check the label in order to select the right cement for the given application. It is important to press the pipes completely together during gluing and to apply enough glue such that a small bead appears around the joint after pressing.

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## Pressure Regulation

On farms with major elevation changes, pressure regulators and other strategies may be needed in order to prevent pressure from increasing beyond the rated pressure of the pipe or irrigation components. For example, a sprinkler irrigated apple orchard in Washington State is constructed on a deep river canyon (700 m deep). The river water is first pumped by high pressure pumps through a steel pipeline to a reservoir at the upper end of the farm (Fig. 8.13). A much lower pressure pump removes the water from the reservoir and pressurizes the irrigation mainline. The irrigation mainline runs back down the hill and feeds each of the irrigation zones. The irrigation mainline can be designed so that pressure loss is equal to elevation gain; however, when irrigation is turned off the pressure greatly exceeds the rated pressure of the pipe. At this pressure, the pipes burst and can create huge holes in the orchard because of the high flow rate from mainlines. In order to prevent such catastrophes, pressure regulators are placed at strategic points along mainline in order to prevent pressure from building beyond the rated pressure of the pipe.

Large pressure regulators are expensive and tend to wear out after 3–4 years. Some farmers control pressure to low elevation zones with gate valves. The gate valves wear out after a few years, but a flanged gate valve is easy to replace and much cheaper than a large pressure regulator. The gate valve, however, would not work in the case of Fig. 8.13.



**Fig. 8.13** Apple orchard irrigation system on river canyon

**Table 8.14** Pipe prices for Example 8.9

Nominal diameter (in)	Nominal diameter (mm)	Wall thickness	Cost (\$/m)
3/4	18	Sch 40	0.39
1	25	Sch 40	0.56
1.5	37	CI 125	0.49
2	50	CI 125	0.75
3	75	CI 125	1.64
4	100	CI 125	2.72
6	150	CI 125	5.87
8	200	CI 125	9.71
10	250	CI 125	18.60
12	300	CI 125	24.54
15	375	SCH 40	26.51
18	450	SCH 40	34.51
21	525	SCH 40	50.00
24	600	SCH 40	70.00

**Economics of Pipe Sizing: Energy Cost Versus Capital Cost**

Pipe diameters are selected based on the capital cost of pipe and the present value of the annual cost of energy (pumping) over the life of the project. For the purpose of questions in this book, pipe prices are shown in Table 8.14; however, the price of PVC fluctuates with the price of oil. It also fluctuates during the year with supply and demand, the amount in inventory, and the date at which the inventory tax is calculated.

The determination of best economic diameter is made in the *Surge and Energy Cost worksheets* based on the price of plastic and the price of energy. However, in the next example, prices used in Table 8.14 are used.

**Example 8.9** An 800 m pipe supplies 3 center pivots. Each pivot flow rate is 95.3 L/sec. Based on Example 8.10, 502 mm (21 in. pipe is the minimum acceptable diameter to prevent surge pressure damage at 286 L/sec. Thus, select either 502 mm or 575 mm (24 in. pipe.

**Table 8.15** Hours of operation, friction loss, power, and annual cost of energy for 502 mm pipe

Flow rate (cms)	Hours of operation	Friction loss (m)	Power (kW)	Energy (kW-hr)	Cost (\$/year)
0.286	1,547	2.58	9.0	13,989	\$1,399
0.190	294	1.21	2.8	828	\$ 83
0.095	2,076	0.34	0.4	822	\$ 82
Total	3,917				\$1,564

**Table 8.16** Hours of operation, friction loss, power, and annual cost of energy for 575 mm pipe

Flow rate (cms)	Hours of operation @ diff. flow rates	Friction loss (m)	Power (kW)	Energy (kW-hr)	Cost (\$/year)
0.286	1,547	1.33	4.7	7211	\$ 721
0.190	294	0.62	1.4	424	\$ 42
0.095	2,076	0.17	0.2	411	\$ 41
Total	3,917				\$ 804

The financial parameters are 20 yr project life and 8 % ROR. Pumps are variable speed so that pressure and flow can be decreased as needed. It is expected that the system will have the following operation times during the year.

- 3 pivots operating at one time(286 L/ sec ) : 1,547 hr
- 2 pivots operating at one time(191 L/ sec ) : 294 hr
- 1 pivot operating alone(96 L/ sec ) : 2,076 hr

Costs are calculated in Tables 8.15 and 8.16. Example cost calculations are shown for the 502 mm pipe. First, evaluate capital + annual costs for 502 mm pipe. Calculate pipe friction loss in the 502 mm pipe.

$$h_f = (1.22 \times 10^{10})(800 \text{ m}) \left( \left( \frac{286}{140} \right)^{1.852} / 502^{4.87} \right) = 2.58 \text{ m}$$

Friction loss can be converted to a power requirement for 502 mm pipe follows:

$$\begin{aligned} \text{Power(kW)} &= (Q H) / (.102 \text{ Eff}). \\ &= (0.286 \text{ cms})(2.58 \text{ m}) / ((.102)(.80)) \\ &= 9.0 \text{ kW} \end{aligned}$$

Present value of energy costs for the 502 mm pipe is calculated based on 20 yr project life and 8 % ROR. Present value (\$1,564/year) is \$15,355 (PV function).

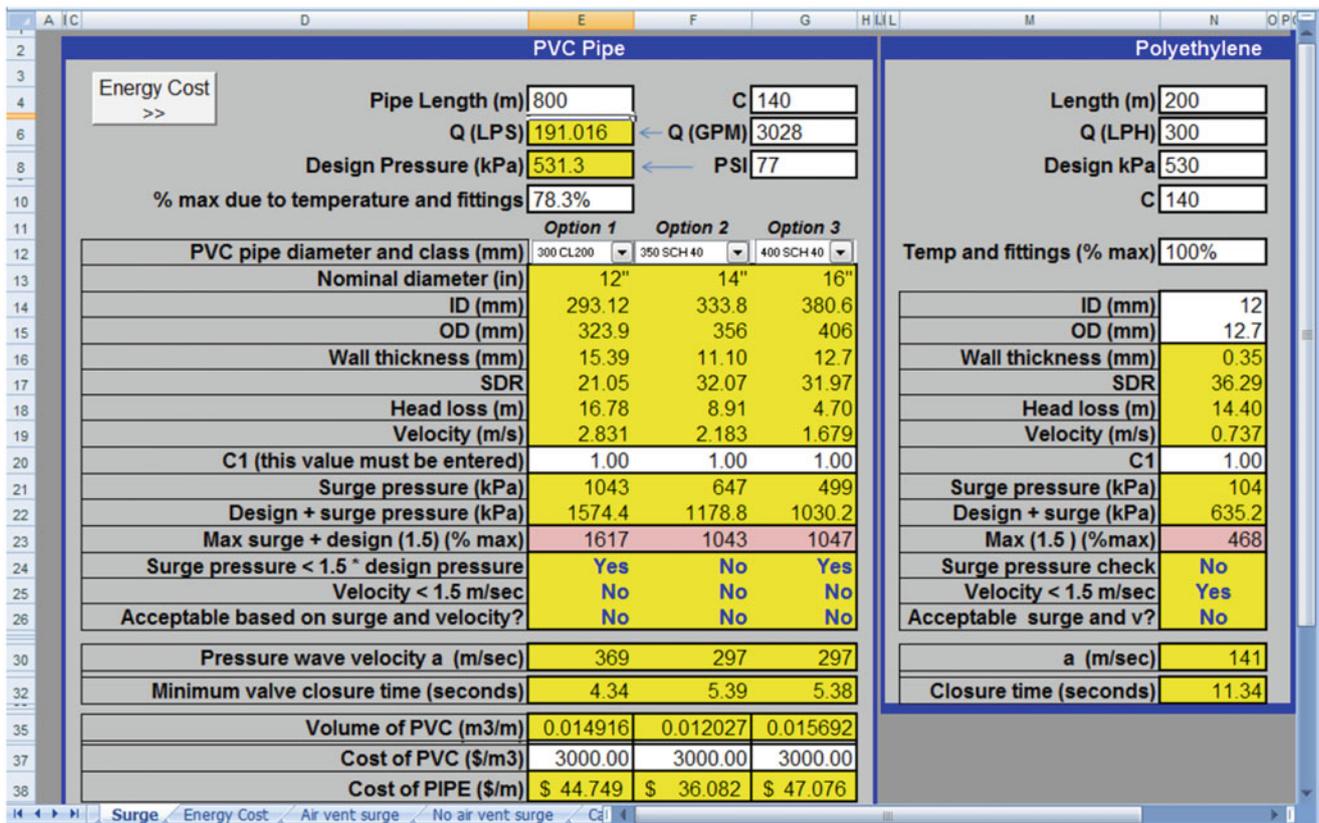


Fig. 8.14 Surge pressure analysis for Example 8.10

Capital cost of 502 mm pipe (Table 8.14)  
 = (\$50/m)(800 m) = \$40,000

Total cost of the 502 mm pipe is equal to capital + annual costs: \$40,000 + \$15,355 = \$55,355

Next, calculate the total cost (energy + capital) of 575 mm pipe (Table 8.15).

Present value(\$804/year)is \$7,894  
 Capital cost of 575 mm pipe = (\$70/m)(800 m) = \$56,000  
 Total cost of the 575 mm pipe is \$56,000 + \$7,894 = \$63,894

Because the total cost of 502 mm pipe, \$55,355, is less than \$63,894, select 502 mm pipe.

**Example 8.10** Change the number of pivots in Example 8.9 to two. Perform surge and economic analysis as in Examples 8.7 and 8.9, respectively. Calculate PVC cost based on volumetric cost of PVC: \$3,000/m<sup>3</sup>. Financial parameters are 20 years and 8 % rate of return. Cost of energy is \$0.1/kW-hr. Pump efficiency = 80 %. Design pressure is 530 kPa. Solve with the *Surge* and *Energy Cost* worksheets.

Assume that PVC only has 78 % of maximum strength as in Example 8.7.

- 2 pivots operating at one time(191 L/ sec) : 294 hr
- 1 pivot operating alone(96 L/ sec) : 2,076 hr

Surge pressure is evaluated in Fig. 8.14. Although none are acceptable based on velocity, two out of three are acceptable based on surge pressure is less than design pressure.

- 300 mm(12 in)Class 200 acceptable
- 350 mm(14 in)Schedule 40 not acceptable
- 400 mm(16 in)Schedule 40 acceptable

The next step is economic analysis. Click the *Energy Cost* button on the *Surge* worksheet (Fig. 8.15). First evaluate at the high flow rate for 294 hours.

The present value of energy costs at the low flow rate is shown in Fig. 8.16.

The sum of present value costs are shown in Table 8.17. The pipe diameter with the lowest present value cost for the sum of energy and capital is the 20" pipe.

**Fig. 8.15** Present value of capital and energy costs at high flow rate for Example 8.10

Based on PVC pipe dimensions from surge page			
Pump Efficiency (%)	80	Rate Return (%)	8
Cost energy (\$/kW)	0.1	# Years	20
Hours operation per Year	294		
	<i>Option #1</i>	<i>Option #2</i>	<i>Option #2</i>
Nominal Diam (in)	12"	14"	16"
Head Loss (m/m)	0.0210	0.0111	0.0059
Power (kW/m)	0.491069	0.260781	0.137648
Annual cost energy (\$/m)	\$ 14.437	\$ 7.667	\$ 4.047
Present value of energy (\$/m)	\$ 141.75	\$ 75.27	\$ 39.73
Based on price on surge page			
	<i>Option #1</i>	<i>Option #2</i>	<i>Option #2</i>
Pipe capital cost (\$/m)	\$ 44.75	\$ 36.08	\$ 47.08
<b>Total present Value (\$/m)</b>			
	<b>\$ 186.50</b>	<b>\$ 111.36</b>	<b>\$ 86.81</b>

**Fig. 8.16** Present value of capital and energy costs at low flow rate for Example 8.10

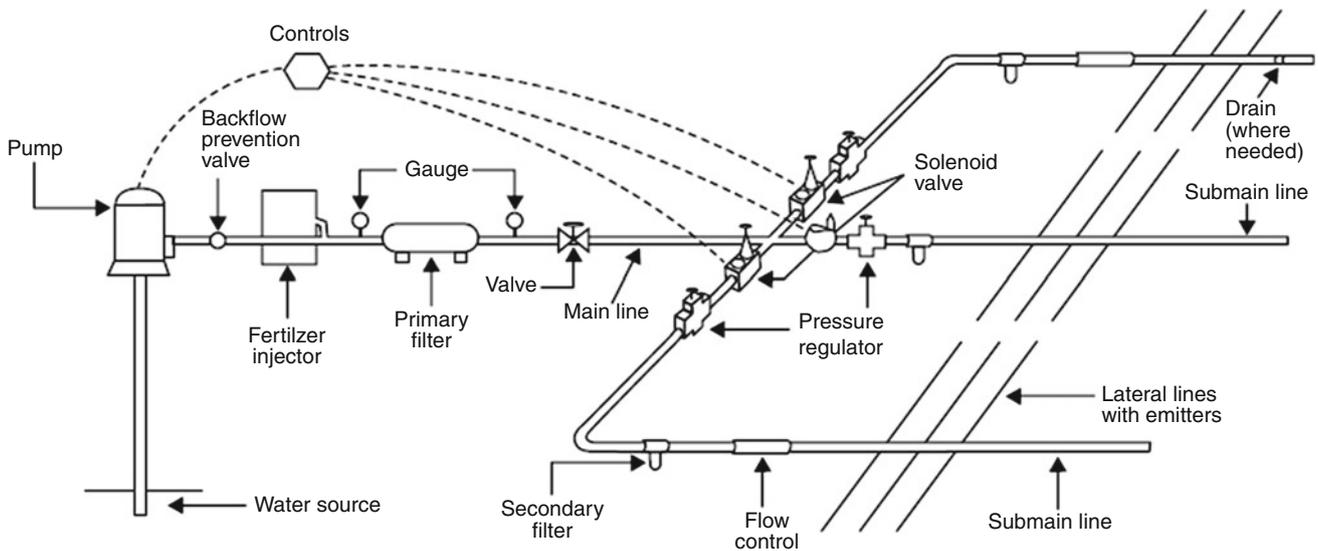
PVC Pipe				
Energy Cost >>	Pipe Length (m)	800	C 140	
	Q (LPS)	191.016	← Q (GPM) 3028	
	Design Pressure (kPa)	531.3	← PSI 77	
	% max due to temperature and fittings	78.3%		
		<i>Option 1</i>	<i>Option 2</i>	<i>Option 3</i>
	PVC pipe diameter and class (mm)	SEE CLASS	SEE SCH 40	SEE SCH 40
	Nominal diameter (in)	12"	14"	16"
	ID (mm)	293.12	333.8	380.6
	OD (mm)	323.9	356	406
	Wall thickness (mm)	15.39	11.10	12.7
	SDR	21.05	32.07	31.97
	Head loss (m)	16.78	8.91	4.70
	Velocity (m/s)	2.831	2.183	1.679
	C1 (this value must be entered)	1.00	1.00	1.00
	Surge pressure (kPa)	1043	647	499
	Design + surge pressure (kPa)	1574.4	1178.8	1030.2
	Max surge + design (1.5) (% max)	1617	1043	1047
	Surge pressure < 1.5 * design pressure	Yes	No	Yes
	Velocity < 1.5 m/sec	No	No	No
	Acceptable based on surge and velocity?	No	No	No
	Pressure wave velocity a (m/sec)	369	297	297
	Minimum valve closure time (seconds)	4.34	5.39	5.38
	Volume of PVC (m3/m)	0.014916	0.012027	0.015692
	Cost of PVC (\$/m3)	3000.00	3000.00	3000.00
	Cost of PIPE (\$/m)	\$ 44.749	\$ 36.082	\$ 47.076
	Head loss (m/m)	0.020978	0.011140	0.005880

**Table 8.17** Pipe selection comparison for Example 8.10

Pipe Diam.	Capital	Low flow rate	High flow rate	Total
12 inch	44.75	138.63	141.75	Astronomical
14 inch	36.08	73.62	75.27	Astronomical
16 inch	47.08	38.86	39.73	\$126/m
18 inch	59.86	21.84	22.33	\$104/m
20 inch	70.15	12.84	13.12	\$95/m***
21 inch	76.54	10.13	10.36	\$96/m

**Questions**

1. On the following drawing of a drip irrigation system network, label the type in the well, pump station, submains, mainlines, and laterals.



2. What is the hydraulic head and total energy of water in a pipe that is 5 m above the datum with pressure 350 kPa and water velocity 1.5 m/sec?
3. When does nonsteady state flow occur, and what are the possible hazards associated with nonsteady state flow?
4. Maximum allowable flow velocity in PVC irrigation pipes is typically specified as 1.5 m/sec. What is the kinetic energy of water at this velocity? Express your answer in terms of m (length) and kPa (pressure). If the irrigation system operates at 350 kPa, then what percent of the energy is kinetic?

5. What two forces are included in the Reynolds number, and why is turbulent flow observed at higher Reynolds numbers?
6. Calculate the friction loss in 1,000 m of 50 mm nominal diameter class 125 pipe. Calculate for flow velocities of 0.05, 1, and 3 m/sec with the Hazen-Williams and Darcy-Weisbach equations. Use  $C = 140$  and  $150$  in the Hazen-Williams equation. Show your work.
7. Redo Example 8.2, but the pipe discharges into a pond with a water surface elevation that is 100 meters below the upper pond water surface elevation. The pipe inlet projects into the upper pond. Use the Hazen-Williams equation.
8. For the problem described in Example 8.3, find the flow rate with a known diameter pipe with the Hazen-Williams equation: the nominal diameter is 10 in (250 mm) Class 125 pipe, the pipe length is 300 m,

- and the Hazen-Williams  $C$  value is 150. Recalculate flow rate with the Darcy-Weisbach equation. The inlet pipe projects into the reservoir.
9. Calculate the pressure rating in metric units and convert to psi for 4 inch (100 mm) class 160 PVC (1120) pipe.
10. Ten inch (250 mm) Class 160 bell end pipe has expansion joints. Calculate the velocity and magnitude of the pressure wave if the operating velocity is 2.3 m/sec, and a valve suddenly closes. If the operating pressure is 50 psi (345 kPa), then what is the maximum surge pressure?

11. Flow rate is 10 GPM (37.9 L/min) and the design pressure is 50 psi (345 kPa). Select a pipe class and diameter that does not exceed the maximum allowable surge pressure. Use the surge equations in this case and not just the 1.5 m/s rule. Also perform an economic analysis for the best pipe diameter. Project parameters are \$3,000/m<sup>3</sup> PVC, 20 year, 8 %, 1440 hr/year, \$0.1/kW-hr, pump efficiency = 80 %.
12. Calculate the maximum surge pressure in a long (100 m) drip irrigation tube (12 mm ID polyethylene). Solve the problem in two ways: assume an air vent and no air vent. Inlet pressure is 210 kPa. Conclude by discussing whether pressure relief valves and air vents are needed on long drip irrigation tubes and state the reason for your answer. Also consider sprinkler laterals (PVC) with and without sprinklers along the pipe.
13. For the following parameters, find the best economic pipe size. The price of PVC is \$3,000/m<sup>3</sup>. Project parameters are 20 year, 8 %, \$0.1/kW-hr, pump efficiency = 80 %.

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