

Chapter 6

Design of PE Piping Systems

Introduction

Design of a PE piping system is essentially no different than the design undertaken with any ductile and flexible piping material. The design equations and relationships are well-established in the literature, and they can be employed in concert with the distinct performance properties of this material to create a piping system which will provide very many years of durable and reliable service for the intended application.

In the pages which follow, the basic design methods covering the use of PE pipe in a variety of applications are discussed.

The material is divided into four distinct sections as follows:

Section 1 covers Design based on Working Pressure Requirements. Procedures are included for dealing with the effects of temperature, surge pressures, and the nature of the fluid being conveyed, on the sustained pressure capacity of the PE pipe.

Section 2 deals with the hydraulic design of PE piping. It covers flow considerations for both pressure and non-pressure pipe.

Section 3 focuses on burial design and flexible pipeline design theory. From this discussion, the designer will develop a clear understanding of the nature of pipe/soil interaction and the relative importance of trench design as it relates to the use of a flexible piping material.

Finally, **Section 4** deals with the response of PE pipe to temperature change. As with any construction material, PE expands and contracts in response to changes in temperature. Specific design methodologies will be presented in this section to address this very important aspect of pipeline design as it relates to the use of PE pipe.

This chapter concludes with a fairly extensive appendix which details the engineering and physical properties of the PE material as well as pertinent pipe characteristics such as dimensions of product produced in accordance with the various industry standards.

Section 1

Design Based on Required Pressure Capacity

Pressure Rating

The methodology for arriving at the standard pressure rating, PR, for PE pipe is discussed in detail in Chapter 5. The terms pressure rating (PR), pressure class (PC), are used in various consensus standards from ASTM, AWWA, CSA and others to denote the pipe's capacity for safely resisting sustained pressure, and typically is inclusive of the capacity to resist momentary pressure increases from pressure surges such as from sudden changes in water flow velocity. Consensus standards may treat pressure surge capacity or allowances differently. That treatment may vary from the information presented in this handbook. The reader is referred to the standards for that specific information.

Equations 1-1 and 1-2 utilize the Hydrostatic Design Stress, HDS, at 73°F (23°C) to establish the performance capability of the pipe at that temperature. HDS's for various PE pipe materials are published in PPI TR-4, "PPI Listing of Hydrostatic Design Basis (HDB), Hydrostatic Design Stress (HDS), Strength Design Basis (SDB), Pressure Design Basis (PDB) and Minimum Required Strength (MRS) Ratings for Thermoplastic Piping Materials". Materials that are suitable for use at temperatures above 100°F (38°C) will also have elevated temperature Hydrostatic Design Basis ratings that are published in PPI TR-4.

The PR for a particular application can vary from the standard PR for water service. PR is reduced for pipelines operating above the base design temperatures, for pipelines transporting fluids that are known to have some adverse effect on PE, for pipelines operating under Codes or Regulations, or for unusual conditions. The PR may be reduced by application of a factor to the standard PR. For elevated temperature applications the PR is multiplied by a temperature factor, F_T . For special fluids such as hydrocarbons, or regulated natural gas, an environmental application factor, A_F , is applied. See Tables 1-2 and Appendix, Chapter 3.

The reader is alerted to the fact that the form of the ISO equation presented in Equations 1-1 and 1-2 has changed from the form of the ISO equation published in the previous edition of the PPI PE Handbook. The change is to employ HDS rather than HDB, and is necessitated by the additional ratings available for high performance materials. In the earlier form of the ISO equation, PR is given as a function of the HDB, not the HDS as in Equations 1-1 and 1-2. This difference is significant and can result in considerable error if the reader uses the Environmental Applications Factors given in Table 1-2 as the "Design Factor" in the HDB form of the ISO equation.

$$(1-1) \quad PR = \frac{2 HDS F_T A_F}{(DR-1)}$$

$$(1-2) \quad PR = \frac{2 HDS F_T A_F}{(IDR+1)}$$

WHERE

PR = Pressure rating, psi

HDS = Hydrostatic Design Stress, psi (Table 1-1)

A_F = Environmental Application Factor (Table 1-2)

NOTE: The environmental application factors given in Table 1-2 are not to be confused with the Design Factor, DF, used in previous editions of the PPI Handbook and in older standards.

F_T = Service Temperature Design Factor (See Appendix to Chapter 3)

DR = OD -Controlled Pipe Dimension Ratio

$$(1-3) \quad DR = \frac{D_o}{t}$$

D_O = OD-Controlled Pipe Outside Diameter, in.

t = Pipe Minimum Wall Thickness, in.

IDR = ID -Controlled Pipe Dimension Ratio

$$(1-4) \quad IDR = \frac{D_I}{t}$$

D_I = ID-Controlled Pipe Inside Diameter, in.

TABLE 1-1
Hydrostatic Design Stress and Service Temperatures

Property	Standard	PE 2606, PE2706	PE 2708, PE 3608, PE 3708, PE 4608	PE 3710, PE 4710
Hydrostatic Design Stress, HDS at 73°F (23°C)	ASTM D2837 & PPI TR-3	630 psi (4.6 MPa)	800 psi (5.5 MPa)	1000 psi (6.9 MPa)
Maximum recommended operating temperature for Pressure Service*	-	140°F (60°C)	140°F (60°C)	140°F (60°C)
Maximum recommended operating temperature for Non-Pressure Service	-	180°F (82°C)	180°F (82°C)	180°F (82°C)

* Some PE piping materials are stress rated at temperatures as high as 180°F. For more information regarding these materials and their use, the reader is referred to PPI, TR-4.

The Hydrostatic Design Stress, HDS, is the safe long-term circumferential stress that PE pipe can withstand. It is derived by applying an appropriate design factor, DF, to the Hydrostatic Design Basis, HDB. The method for establishing the Hydrostatic Design Stress for PE pipe is described in Chapters 3 and 5.

At the time of this printing, AWWA is in the process of revising AWWA C906 to incorporate PE4710 material and to use the HDS values in Table 1-1. The version in effect at the time of this printing, AWWA C906-07, limits the maximum Hydrostatic Design Stress to 800 psi for HDPE and to 630 psi for MDPE. AWWA C901-08 has been revised to incorporate the materials listed in Table 1-1.

The Environmental Application Factor is used to adjust the pressure rating of the pipe in environments where specific chemicals are known to have an effect on PE and therefore require derating as described in Chapter 3. Table 1-2 gives Environmental Applications Factors, A_F , which should only be applied to pressure equations (see Equations 1-1 and 1-2) based on the HDS, not the HDB.

TABLE 1-2
PE Pipe Environmental Application Factors (A_F)*

Pipe Environment	Environmental Application Factor (A_F) at 73°F (23°C)
Water: Aqueous solutions of salts, acids and bases; Sewage; Wastewater; Alcohols; Glycols (anti-freeze solutions)	1.0
Nitrogen; Carbon dioxide; Methane; Hydrogen sulfide; Non-Federally regulated applications involving dry natural gas or other non-reactive gases	1.0
Fluids such as solvating/permeating chemicals in pipe or soil (typically hydrocarbons) in 2% or greater concentration, natural or other fuel-gas liquids condensates, crude oil, fuel oil, gasoline, diesel, kerosene, hydrocarbon fuels, wet gas gathering, multiphase oilfield fluids, LVP liquid hydrocarbons, oilfield water containing >2% hydrocarbons.	0.5

* Certain codes and standards include prohibitions and/or strength reduction factors relating to the presence of certain constituents in the fluid being transported. In a code controlled application the designer must ensure compliance with all code requirements.

When choosing the environmental applications factor (A_F), consideration must be given to Codes and Regulations, the fluid being transported, the external environment, and the uncertainty associated with the design conditions of internal pressure and external loads.

The pressure rating (PR) for PE pipe in water at 73°F over the range of typical DR's is given in Tables 1-3 A and 1-3 B in this chapter.

Pressure Rating for Fuel Gas Pipe

Compared to other common thermoplastic pipes, PE pipe can be used over a broader temperature range. For pressure applications, it has been successfully used from -40°F (-40°C) to 140°F (60°C). In the case of buried non-pressure applications it has been used for conveying fluids that are at temperatures as high as 180°F (82°C). See Table 1-1. For pressure applications above 80°F (27°C) the Service Temperature Design Factor is applied to determine the pressure rating. See Table A.2 in the Appendix to Chapter 3.

The pressure rating for gas distribution and transmission pipe in US federally regulated applications is determined by Title 49, Transportation, of The Code of Federal Regulations. Part 192 of this code, which covers the transportation of natural and other gases, requires that the maximum pressure rating (PR) of a PE pipe be determined based on an HDS that is equal to the material's HDB times a DF of 0.32. (See Chapter 5 for a discussion of the Design Factor, DF.) This is the equivalent of saying that for high density PE pipe meeting the requirements of ASTM D2513 the HDS is 500 psi at 73°F and for medium density PE pipe meeting D2513 the HDS is 400 psi at 73°F . There are additional restrictions imposed by this Code, such as the maximum pressure at which a PE pipe may be operated (which at the time of this writing is 125 psi for pipe 12-in and smaller and 100 psi for pipe larger than 12-in through 24-in.) and the acceptable range of operating temperatures. The temperature design factors for federally regulated pipes are different than those given in Table A.2 in the Appendix to Chapter 3. Consult with the Federal Regulations to obtain the correct temperature design factor for gas distribution piping.

At the time of this writing, there is an effort underway to amend the US federal code to reflect changes already incorporated in ASTM F714 and D3035. When amended, these changes will increase the pressure rating (PR) of pipe made with high performance PR resins - those that meet the higher performance criteria listed in Chapter 5 (see "Determining the Appropriate Value of HDS"), to be 25% greater than pressure ratings of pipe made with 'traditional' resins.

In Canada gas distribution pipe is regulated per CSA Z662-07. CSA allows a design factor of 0.4 to be applied to the HDB to obtain the HDS for gas distribution pipe.

PE pipe meeting the requirements of ASTM D2513 may be used for the regulated distribution and transmission of liquefied petroleum gas (LPG). NFPA/ANSI 58 recommends a maximum operating pressure of 30 psig for LPG gas applications involving polyethylene pipe. This design limit is established in recognition of the higher condensation temperature for LPG as compared to that of natural gas and, thus, the maximum operating pressure is recommended to ensure that plastic pipe is not subjected to excessive exposure to LPG condensates. The Environmental Application Factor for LP Gas Vapors (propane, propylene, and butane) is 0.8 with

a maximum HDS of 800 psi at 73°F for HDPE and 630 psi for MDPE. For further information the reader is referred to PPI's TR-22, Polyethylene Piping Distribution Systems for Components of Liquid Petroleum Gases.

The pressure rating for PE gas gathering lines in the US may differ depending upon the class location (population density) of the gathering line. Gas gathering lines in Class 2, 3 and 4 locations are regulated applications and subject to US federal codes the same as gas distribution and transmission lines. Gas gathering lines in Class 1 locations are not regulated in accordance with US federal codes, and may be operated at service pressures determined using Equation 1-1. Non-regulated gas gathering lines may use PE pipe meeting ASTM F2619 or API 15LE, and may be larger than 24" diameter. PE pipe meeting ASTM D2513 is not required for non-regulated gas gathering lines.

In Canada, PE gas gathering lines are regulated in accordance with CSA Z662 Clause 13.3 and are required to meet API 15LE. PE gas gathering lines may be operated at service pressures equivalent to those determined using Equation 1-1.

Pressure Rating for Liquid Flow Surge Pressure

Surge pressure events, which give rise to a rapid and temporary increase in pressure in excess of the steady state condition, are the result of a very rapid change in velocity of a flowing liquid. Generally, it is the fast closing of valves and uncontrolled pump shutdowns that cause the most severe changes and oscillations in fluid velocity and, consequently in temporary major pressure oscillations. Sudden changes in demand can also lead to lesser but more frequent pressure oscillations. For many pipe materials repeated and frequent pressure oscillations can cause gradual and cumulative fatigue damage which necessitate specifying higher pressure class pipes than determined solely based on sustained pressure requirements. And, for those pipe materials a higher pressure class may also be required for avoiding pipe rupture under the effect of occasional but more severe high-pressure peaks. Two properties distinguish PE pipes from these other kinds of pipes. The first is that because of their lower stiffness the peak value of a surge pressures that is generated by a sudden change in velocity is significantly lower than for higher stiffness pipes such as metallic pipes. And, the second is that a higher pressure rating (PR), or pressure class (PC), is generally not required to cope with the effects of pressure surges. Research, backed by extensive actual experience, indicates that PE pipes can safely tolerate the commonly observed maximum peak temporary surge pressure of twice the steady state condition. Furthermore, the long-term strength of PE pipes is not adversely affected by repeated cyclic loading – that is, PE pipes are very fatigue resistant.

In the design of PE pipe, pressure surges are generally classified as Occasional pressure surges, Recurring pressure surges, and Negative pressures.

- Occasional surge pressures are caused by emergency operations such as fire flow or as a result of a malfunction, such as a power failure or system component failure, which includes pump seize-up, valve stem failure and pressure relief valve failure.
- Recurring surge pressures are inherent to the design and operation of a system. Recurring surge pressures can be caused by normal pump start up or shut down, normal valve opening and closing, and/or “background” pressure fluctuations associated with normal pipe operation.
- Negative pressure may be created by a surge event and cause a localized collapse by buckling. (Negative pressure may also occur inside flowing pipelines due to improper hydraulic design.)

In recognition of the performance behavior of PE pipes the following design principles have been adopted by AWWA for all PE pressure class (PC) rated pipes. These design principles, which are as follows, are also applicable to PE water pipes that are pressure rated (PR) in accordance with ASTM and CSA standards:

1. Resistance to Occasional Pressure Surges:

- The resultant total pressure – sustained plus surge – must not exceed 2.0 times the pipe’s temperature compensated pressure rating (PR). See Tables 1-3 A and 1-3 B for standard surge allowances when the pipe is operated at its full rated pressure.
- In the rare case where the resultant total pressure exceeds 2.0 times the pipe’s temperature adjusted PR, the pipe must be operated at a reduced pressure so that the above criterion is satisfied. In this event the pipe’s reduced pressure rating is sometimes referred to as the pipe’s “working pressure rating” (WPR), meaning that for a specific set of operating conditions (temperature, velocity, and surge) this is the pipe’s pressure rating. AWWA uses the term WPR not just for a reduced pressure rating but for any pressure rating based on application specific conditions. Where the total pressure during surge does not exceed the standard allowance of 2.0 (occasional) and 1.5 (recurring) the WPR equals the temperature adjusted PR.
- The maximum sustained pressure must never exceed the pipe’s temperature adjusted pressure rating (PR).

Example:

A PE pipe has a DR = 17 and is made from a PE4710 material. Accordingly, its standard pressure rating (PR) for water, at 73°F is 125 psi (See Table A.1 in Appendix to Chapter 3). The maximum sustained water temperature shall remain below 73°F. Accordingly, no temperature compensation is required and therefore, the pipe’s initial WPR is equal to its standard PR or, 125 psi.

Let us first assume that the maximum occasional surge pressure shall never exceed 120 psi. Since a WPR of 125 psi plus a surge of 120 psi is less than 2 times 125 psi the pipe's initial WPR of 125 psi remains at that value.

Now let us assume a second case in which the maximum occasional surge pressure can be as high as 150 psi. This pressure plus the pipe's initial WPR of 125 psi result in a total momentary pressure of 275 psi, which is 25 psi above the limit of $2 \times 125 \text{ psi} = 250 \text{ psi}$. To accommodate this 25 psi excess it is necessary to reduce the pipe's initial WPR of 125 to a final WPR of 100 psi.

2. Resistance to Recurring Pressure Surges:

- The resultant total momentary pressure – sustained plus surge – must not exceed 1.5 times the pipe's temperature adjusted pressure rating (PR). See Tables 1-3 A and 1-3 B for standard surge allowance when the pipe is operated at its full rated pressure.
- In the rare case where the resultant total pressure exceeds 1.5 times the pipe's temperature adjusted PR the pressure rating must be reduced to the pipe's WPR so that the above criterion is satisfied.
- The maximum sustained pressure must never exceed the pipe's temperature adjusted PR.

3. Resistance to Localized Buckling When Subjected to a Negative Pressure Generated by a Surge Event

A buried pipe's resistance to localized buckling while under the combined effect of external pressure and a very temporary full vacuum should provide an adequate margin of safety. The design for achieving this objective is discussed in a later section of this chapter. It has been shown that a DR21 pipe can withstand a recurring negative pressure surge equal to a full vacuum at 73°F. Higher DR pipes may also be able to withstand a recurring negative surge equal to full vacuum if they are properly installed and have soil support. Their resistance may be calculated using Luscher's Equation presented later in this chapter.

Estimating the Magnitude of Pressure Surges

Regardless of the type of pipe being used surge or water hammer problems can be complex especially in interconnected water networks and they are best evaluated by conducting a formal surge analysis (See References 25 and 32). For all water networks, rising mains, trunk mains and special pump/valve circumstances a detailed surge analysis provides the best way of anticipating and designing for surge.

Absent a formal surge analysis, an estimate of the magnitude of a surge pressure can be made by evaluating the surge pressure that results from an anticipated sudden change in velocity in the water flowing inside a PE pipe.

An abrupt change in the velocity of a flowing liquid in a pipe generates a pressure wave. The velocity of the wave may be determined using Equation 1-5.

$$(1-5) \quad a = \frac{4660}{\sqrt{1 + \frac{K_{BULK}}{E_d}(DR - 2)}}$$

WHERE

a = Wave velocity (celerity), ft/sec

K_{BULK} = Bulk modulus of fluid at working temperature (typically 300,000 psi for water at 73°F)

E_d = Dynamic instantaneous effective modulus of pipe material
(typically 150,000 psi for all PE pipe at 73°F (23°C)); see Appendix to Chapter 3

DR = Pipe dimension ratio

The resultant transient surge pressure, P_s , may be calculated from the wave velocity, a , and the sudden change in fluid velocity, ΔV .

$$(1-6) \quad P_s = a \left(\frac{\Delta V}{2.31g} \right)$$

WHERE

P_s = Transient surge pressure, psig

a = Wave velocity (celerity), ft/sec

ΔV = Sudden velocity change, ft/sec

g = Constant of gravitational acceleration, 32.2 ft/sec²

Figure 1-1 represents the pressure surge curves for all PE pipes as calculated using Equations 1-5 and 1-6 for Standard Dimension Ratios (SDR's).

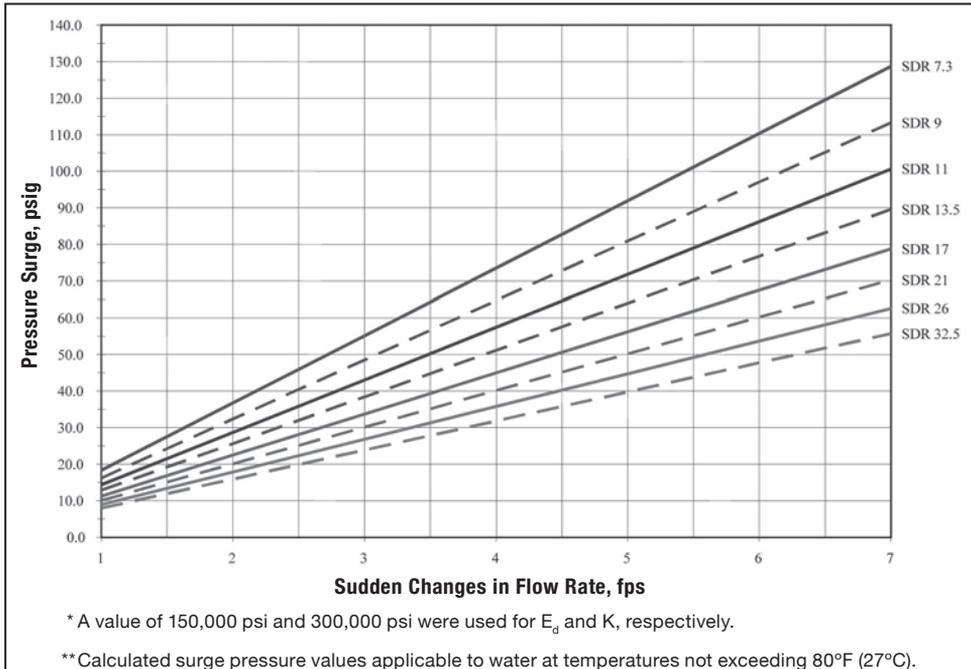


Figure 1-1 Sudden Velocity Change vs. Pressure Surge for All PE Pipes

The surge pressure values in Figure 1-1 are based on a sudden change in velocity, which may more often be the case for events like a sudden pump shut-down or a rapid valve closure. A sudden shut-down or a rapid closure occurs faster than the “critical time” (the time it takes a pressure wave initiated at the beginning of a valve closing to return again to the valve). Under ordinary operations, during which valve closings and pump shut-downs are slower than the “critical time”, the actual pressure surge is smaller than that in Figure 1-1. The “critical time” is determined by means of the following relationship:

$$(1-7) T_{CR} = 2L/a$$

WHERE

T_{CR} = critical time, seconds

L = distance within the pipeline that the pressure wave moves before it is reflected back by a boundary condition, ft

a = wave velocity (celerity) of pressure wave for the particular pipe, ft/s. (See Equation 1-5)

Generally, PE pipe’s capacity for safely tolerating occasional and frequently occurring surges is such that seldom are surge pressures large enough to require a de-rating of the pipe’s static pressure rating. Tables 1-3 A and 1-3 B show the maximum allowable sudden changes in water flow velocity (ΔV) that are safely tolerated without the need

to de-rate the pressure rating (PR) or, the pressure class (PC), of a PE pipe. If sudden changes in velocity are expected to be greater than the values shown in these Tables, they then must be accommodated by lowering the pipe’s static pressure rating. As previously discussed, the new rating is called the working pressure rating (WPR).The procedure for establishing a WPR has been discussed earlier in this Section.

TABLE 1-3A

Allowances for Momentary Surge Pressures Above PR or PC for Pipes Made From PE4710 and PE3710 Materials¹.

Pipe Standard Diameter Ratio (SDR)	Standard Static Pressure Rating (PR) or, Standard Pressure Class (PC) for water @ 73°F, psig	Standard Allowance for Momentary Surge Pressure Above the Pipe’s PR or PC			
		Allowance for Recurring Surge		Allowance for Occasional Surge	
		Allowable Surge Pressure, psig	Resultant Allowable Sudden Change in Velocity, fps	Allowable Surge Pressure, psig	Resultant Allowable Sudden Change in Velocity, fps
32.5	63	32	4.0	63	8.0
26	80	40	4.5	80	9.0
21	100	50	5.0	100	10.0
17	125	63	5.6	125	11.2
13.5	160	80	6.2	160	12.4
11	200	100	7.0	200	14.0
9	250	125	7.7	250	15.4
7.3	320	160	8.7	320	17.4

1. AWWA C906-07 limits the maximum Pressure Class of PE pipe to the values shown in Table B. At the time of this printing C906 is being revised to allow PC values in Table A to be used for PE3710 and PE4710 materials. Check the latest version of C906

TABLE 1-3 B

Allowances for Momentary Surge Pressures Above PR or PC for Pipes Made from PE 2708, PE3408, PE3608, PE3708 and PE4708 Materials.

Pipe Standard Diameter Ratio (SDR)	Standard Static Pressure Rating (PR) or, Standard Pressure Class (PC), for Water @ 73°F, psig	Standard Allowance for Momentary Surge Pressure Above the Pipe’s PR or PC			
		Allowance for Recurring Surge		Allowance for Occasional Surge	
		Allowable Surge Pressure, psig	Resultant Allowable Sudden Change in Velocity, fps	Allowable Surge Pressure, psig	Resultant Allowable Sudden Change in Velocity, fps
32.5	50	25	3.1	50	6.2
26	63	32	3.6	63	7.2
21	80	40	4.0	80	8.0
17	100	50	4.4	100	8.8
13.5	125	63	4.9	125	9.8
11	160	80	5.6	160	11.2
9	200	100	6.2	200	12.4
7.3	250	125	6.8	250	13.6

The surge pressure allowance in Table 1-3 A and 1-3 B are not the maximum surge limits that the pipe can safely withstand. Higher surge pressures can be tolerated in pipe where the working pressure rating (WPR) of the pipe is limited to a pressure less than the pressure rating (PR). This works because the combined total pressure for surge and for pumping pressure is limited to 1.5 times the PR (or PC) for recurring surge and 2.0 times the PR (or PC) for occasional surge. If the pumping pressure is less than the PR (or PC) then a higher surge than the standard allowance given in Table A and B is permitted. The maximum permitted surge pressure is equal to $1.5 \times PR - WP$ for recurring surge and $2.0 \times PR - WP$ for occasional surge, where WP is the pumping or working pressure of the pipeline. For example a DR21 PE4710 pipe with an operating pressure of 80 psi can tolerate a recurring surge pressure of $1.5 \times 100 \text{ psi} - 80 \text{ psi} = 70 \text{ psi}$. Note that in all cases WP must be equal or less than PR.

Controlling Surge Pressure Reactions

Reducing the rate at which a change in flow velocity occurs is the major means by which surge pressure rises can be minimized. Although PE pipe is very tolerant of such rises, other non-PE components may not be as surge tolerant; therefore, the prudent approach is to minimize the magnitude of surge pressures by taking reasonable precautions to minimize shock. Hydrants, large valves, pumps, and all other hydraulic appurtenances that may suddenly change the velocity of a column of water should be operated slowly, particularly during the portion of travel near valve closing which has the larger effect on rate of flow. If the cause of a major surge can be attributable to pump performance – especially, in the case of an emergency stoppage – then, proper pressure relief mechanisms should be included. These can include traditional solutions such as by providing flywheels or by allowing the pumps to run backwards.

In hilly regions, a liquid flow may separate at high points and cause surge pressures when the flow is suddenly rejoined. In such cases measures should be taken to keep the pipeline full at all times. These can consist of the reducing of the flow rate, of the use at high points of vacuum breakers or, of air relief valve.

Also, potential surge pressure problems should be investigated in the design of pumping station piping, force mains, and long transmission lines. Proven and suitable means should be provided to reduce the effect of surges to a minimum that is practicable and economical. Although PE pipe is much more tolerant of the effect of sudden pressure increases traditional measures should be employed for the minimizing of the occurrence of such increases.

Section 2

Hydraulic Design of PE Pipe

This section provides design information for determining the required flow diameter for PE pipe. It also covers the following topics: general fluid flows in pipe and fittings, liquid (water and water slurry) flow under pressure, non-pressure (gravity) liquid flow, and compressible gas flow under pressure. Network flow analysis and design is not addressed. ^(1,2)

The procedure for piping system design is frequently an iterative process. For pressure liquid flows, initial choice of pipe flow diameter and resultant combinations of sustained internal pressure, surge pressure, and head loss pressure can affect pipe selection. For non-pressure systems, piping design typically requires selecting a pipe size that provides adequate reserve flow capacity and a wall thickness or profile design that sufficiently resists anticipated static and dynamic earthloads. This trial pipe is evaluated to determine if it is appropriate for the design requirements of the application. Evaluation may show that a different size or external load capacity may be required and, if so, a different pipe is selected then reevaluated. The Appendix to Chapter 3 provides engineering data for PE pipes made to industry standards that are discussed in this chapter and throughout this handbook.

Pipe ID for Flow Calculations

Thermoplastic pipes are generally produced in accordance with a dimension ratio (DR) system. The dimension ratio, DR or IDR, is the ratio of the pipe diameter to the respective minimum wall thickness, either OD or ID, respectively. As the diameter changes, the pressure rating remains constant for the same material, dimension ratio and application. The exception to this practice is production of thermoplastic pipe in accordance with the industry established SCH 40 and SCH 80 dimensions such as referenced in ASTM D 2447.

Flow Diameter for Outside Diameter Controlled Pipe

OD-controlled pipe is dimensioned by outside diameter and wall thickness. Several sizing systems are used including IPS, which specifies the same OD's as iron pipe sized (IPS) pipe; DIPS pipe which specifies the same OD's as ductile iron pipe; and CTS, which specifies the same OD's as copper tubing sizes. For flow calculations, inside diameter is calculated by deducting twice the average wall thickness from the specified outside diameter. OD-controlled pipe standards include ASTM D2513, ASTM D2737, ASTM D2447, ASTM D3035, ASTM F714, AWWA C901, AWWA C906 and API 15LE. ^(3,4,5,6,7,8,9,10) The Appendix to this chapter provides specific dimensional information for outside diameter controlled PE pipe and tubing that is made to

dimension ratio (DR) requirements in accordance with a number of different ASTM, AWWA, CSA and API standards.

The average inside diameter for such pipes has been calculated using Equation 2-1. Typically, wall thickness is specified as a minimum dimension, and a plus 12% tolerance is applied. In this equation, the average ID is determined by deducting twice the average wall thickness (minimum wall thickness plus a tolerance of 6%) from the average outside diameter.

$$(2-1) \quad D_I = D_O - 2.12 \left(\frac{D_O}{DR} \right)$$

WHERE

D_I = pipe average inside diameter, in

D_O = specified average value of pipe outside diameter, in

DR = dimension ratio

$$(2-2) \quad DR = \frac{D_O}{t}$$

t = pipe minimum wall thickness, in

Pipe Diameter for ID Controlled Pipe

Standards for inside diameter controlled pipes provide average dimensions for the pipe inside diameter that are used for flow calculations. ID-controlled pipe standards include ASTM D2104, ASTM D2239, ASTM F894 and AWWA C901. ^(11,12,13)

The terms “DR” and “IDR” identify the diameter to wall thickness dimension ratios for outside diameter controlled and inside diameter controlled pipe, respectively. When those ratios comply with standard values they are called “standard dimension ratios”, that is SDR or SIDR. A discussion of standard dimension ratios is included in Chapter 5.

Fluid Flow in PE Piping

Head Loss in Pipes – Darcy-Weisbach/Colebrook/Moody

Viscous shear stresses within the liquid and friction along the pipe walls create resistance to flow within a pipe. This resistance results in a pressure drop, or loss of head in the piping system.

The Darcy-Weisbach formula, Equation 2-3., and the Colebrook formula, Equation 2-6, are generally accepted methods for calculating friction losses due to liquids

flowing in full pipes.^(15,16) These formulas recognize dependence on pipe bore and pipe surface characteristics, liquid viscosity and flow velocity.

The Darcy-Weisbach formula is:

$$(2-3) \quad h_f = f \frac{L V^2}{d' 2g}$$

WHERE

h_f = friction (head) loss, ft. of liquid

L = pipeline length, ft.

d' = pipe inside diameter, ft.

V = flow velocity, ft./sec.

f = friction factor (dimensionless, but dependent upon pipe surface roughness and Reynolds number)

g = constant of gravitational acceleration (32.2ft/sec²)

The flow velocity may be computed by means of the following equation

$$(2-4) \quad V = \frac{0.4085 Q}{D_1^2}$$

WHERE

Q = flow rate, gpm

D_1 = pipe inside diameter, in

Liquid flow in pipes will assume one of three flow regimes. The flow regime may be laminar, turbulent or in transition between laminar and turbulent. In laminar flow (Reynolds number, Re, below 2000), the pipe's surface roughness has no effect and is considered negligible. As such, the friction factor, f , is calculated using Equation 2-5.

$$(2-5) \quad f = \frac{64}{\text{Re}}$$

WHERE

Re = Reynolds number, dimensionless = < 2000 for laminar flow, see Equation 2-7

> 4000 for turbulent flow, see Figure 2-1

For turbulent flow (Reynolds number, Re, above 4000), the friction factor, f , is dependent on two factors, the Reynolds number and pipe surface roughness. The resultant friction factor may be determined from Figure 2-1, the Moody Diagram. This factor applies to all kinds of PE's and to all pipe sizes⁽¹⁷⁾. In the Moody Diagram, relative roughness, ϵ/d (see Table 2-1 for ϵ) is used which is the ratio of absolute roughness to the pipe inside diameter. The friction factor may also be determined using the Colebrook formula. The friction factor can also be read from the Moody diagram with enough accuracy for calculation.

The Colebrook formula is:

$$(2-6) \quad \frac{1}{\sqrt{f}} = -2 \log_{10} \left\{ \frac{\varepsilon}{3.7 d'} + \frac{2.51}{\text{Re} \sqrt{f}} \right\}$$

For Formulas 2-5 and 2-6, terms are as previously defined, and:

ε = absolute roughness, ft. (see Table 2-1)

Re = Reynolds number, dimensionless (see Equation 2-5)

Liquid flow in a pipe occurs in one of three flow regimes. It can be laminar, turbulent or in transition between laminar and turbulent. The nature of the flow depends on the pipe diameter, the density and viscosity of the flowing fluid, and the velocity of flow. The numerical value of a dimensionless combination of these parameters is known as the Reynolds number and the resultant value of this number is a predictor of the nature of the flow. One form of the equation for the computing of this number is as follows:

$$(2-7) \quad Re = \frac{3160 Q}{k Di}$$

WHERE

Q = rate of flow, gallons per minute

k = kinematic viscosity, in centistokes (See Table 2-3 for values for water)

Di = internal diameter of pipe, in

When the friction loss through one size pipe is known, the friction loss through another pipe of different size may be found by:

$$(2-8) \quad h_{f2} = h_{f1} \left(\frac{d'_1}{d'_2} \right)^5$$

The subscripts 1 and 2 refer to the known and unknown pipes. Both pipes must have the same surface roughness, and the fluid must be the same viscosity and have the same flow rate.

TABLE 2-1
Surface Roughness for Various New Pipes

Type of Pipe	'E' Absolute Roughness of Surface, ft		
	Values for New Pipe Reported by Reference ⁽¹⁸⁾	Values for New Pipe and Recommended Design Values Reported by Reference ⁽¹⁹⁾	
		Mean Value	Recommended Design Value
Riveted steel	0.03 - 0.003	–	–
Concrete	0.01 – 0.001	–	–
Wood stave	0.0003 – 0.0006	–	–
Cast Iron – Uncoated	0.00085	0.00074	0.00083
Cast Iron – Coated	–	0.00033	0.00042
Galvanized Iron	0.00050	0.00033	0.00042
Cast Iron – Asphalt Dipped	0.0004	–	–
Commercial Steel or Wrought Iron	0.00015	–	–
Drawn Tubing	0.000005 corresponds to “smooth pipe”	–	–
Uncoated Steel	–	0.00009	0.00013
Coated Steel	–	0.00018	0.00018
Uncoated Asbestos – Cement	–		
Cement Mortar Relined Pipes (Tate Process)	–	0.00167	0.00167
Smooth Pipes (PE and other thermoplastics, Brass, Glass and Lead)	–	“smooth pipe” (0.000005 feet) (See Note)	“smooth pipe” (0.000005) (See Note)

Note: Pipes that have absolute roughness equal to or less than 0.000005 feet are considered to exhibit “smooth pipe” characteristics.

Pipe Deflection Effects

Pipe flow formulas generally assume round pipe. Because of its flexibility, buried PE pipe will deform slightly under earth and other loads to assume somewhat of an elliptical shape having a slightly increased lateral diameter and a correspondingly reduced vertical diameter. Elliptical deformation slightly reduces the pipe’s flow area. Practically speaking, this phenomenon can be considered negligible as it relates to pipe flow capacity. Calculations reveal that an elliptical deformation which reduces the pipe’s vertical diameter by 7% results in a flow reduction of approximately 1%.

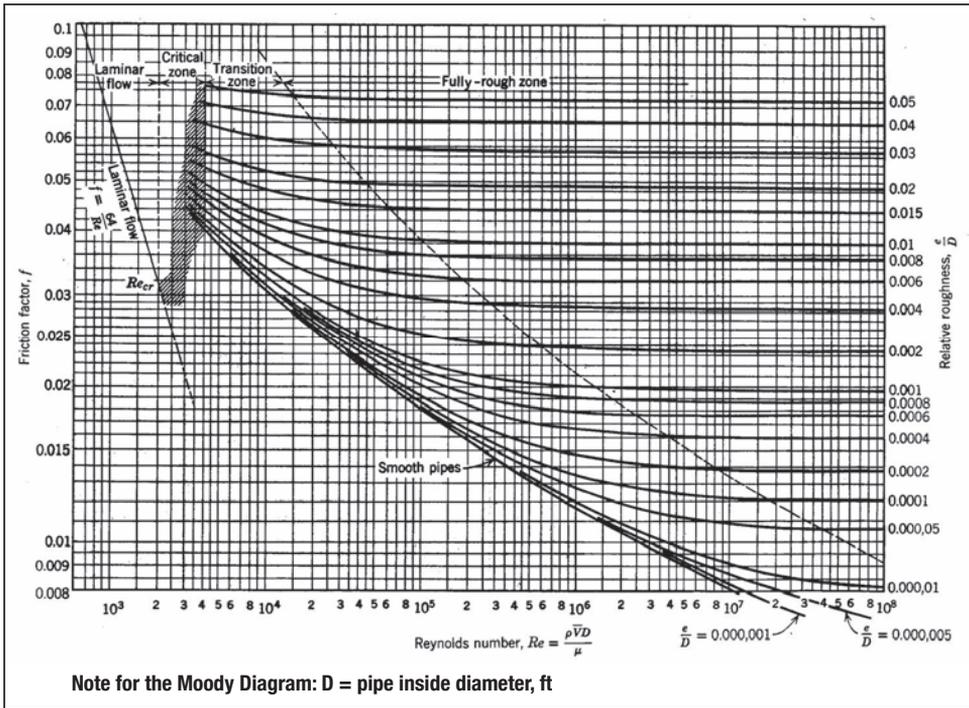


Figure 2-1 The Moody Diagram

Head Loss in Fittings

Fluids flowing through a fitting or valve will experience a friction loss that can be directly expressed using a resistance coefficient, K' , which represents the loss in terms of an equivalent length of pipe of the same diameter.⁽²⁰⁾ As shown in the discussion that follows, this allows the loss through a fitting to be conveniently added into the system flow calculations. Table 2-2 presents K' factors for various fittings.

Where a pipeline contains a large number of fittings in close proximity to each other, this simplified method of predicting flow loss may not be adequate due to the cumulative systems effect. Where this is a design consideration, the designer should consider an additional frictional loss allowance, or a more thorough treatment of the fluid mechanics.

The equivalent length of pipe to be used to estimate the friction loss due to fittings may be obtained by Eq. 2-9 where L_{EFF} = Effective Pipeline length, ft; D is pipe bore diameter in ft.; and K' is obtained from Table 2-2.

$$(2-9) L_{EFF} = K'D$$

TABLE 2-2
Representative Fittings Factor, K', To Determine Equivalent Length of Pipe

Piping Component	K'
90° Molded Elbow	40
45° Molded elbow	21
15° Molded Elbow	6
90° Fabricated Elbow (3 or more miters)	24
90° Fabricated Elbow (2 miters)	30
90° Fabricated Elbow (1 miters)	60
60° Fabricated Elbow (2 or more miters)	25
60° Fabricated Elbow (1 miters)	16
45° Fabricated Elbow (2 or more miters)	15
45° Fabricated Elbow (1 miters)	12
30° Fabricated Elbow (2 or more miters)	8
30° Fabricated Elbow (1 miters)	8
15° Fabricated Elbow (1 miters)	6
Equal Outlet Tee, Run/Branch	60
Equal Outlet Tee, Run/Run	20
Globe Valve, Conventional, Fully Open	340
Angle Valve, Conventional, Fully Open	145
Butterfly Valve, >8", Fully Open	40
Check Valve, Conventional Swing	135

- K values are based on Crane Technical Paper No 410-C

- K value for Molded Elbows is based on a radius that is 1.5 times the diameter.

- K value for Fabricated Elbows is based on a radius that is approximately 3 times the diameter.

Head Loss Due to Elevation Change

Line pressure may be lost or gained from a change in elevation. For liquids, the pressure for a given elevation change is given by:

$$(2-10) \ h_E = h_2 - h_1$$

WHERE

h_E = Elevation head, ft of liquid

h_1 = Pipeline elevation at point 1, ft

h_2 = Pipeline elevation at point 2, ft

If a pipeline is subject to a uniform elevation rise or fall along its length, the two points would be the elevations at each end of the line. However, some pipelines may have several elevation changes as they traverse rolling or mountainous terrain. These pipelines may be evaluated by choosing appropriate points where the pipeline slope changes, then summing the individual elevation heads for an overall pipeline elevation head.

In a pipeline conveying liquids and running full, pressure in the pipe due to elevation exists whether or not liquid is flowing. At any low point in the line, internal pressure will be equal to the height of the liquid above the point multiplied by the specific weight of the liquid. If liquid is flowing in the line, elevation head and head loss due to liquid flow in the pipe are added to determine the pressure in the pipe at a given point in the pipeline.

Pressure Flow of Water – Hazen-Williams Equation

The Darcy-Weisbach method of flow resistance calculation may be applied to liquid and gases, but its solution can be complex. For many applications, empirical formulas are available and, when used within their limitations, reliable results are obtained with greater convenience. For example, Hazen and Williams developed an empirical formula for the flow of water in pipes at 60° F.

The Hazen-Williams formula for water at 60° F (16°C) can be applied to water and other liquids having the same kinematic viscosity of 1.130 centistokes which equals 0.00001211 ft²/sec or 31.5 SSU (Saybolt Second Universal). The viscosity of water varies with temperature, so some error can occur at temperatures other than 60°F (16°C).

Hazen-Williams formula for friction (head) loss in feet of water head:

$$(2-11) \quad h_f = \frac{0.002083 L}{D_i^{4.8655}} \left(\frac{100 Q}{C} \right)^{1.85}$$

Hazen-Williams formula for friction (head) loss in psi:

$$(2-12) \quad p_f = \frac{0.0009015L}{D_i^{4.8655}} \left(\frac{100 Q}{C} \right)^{1.85}$$

Terms are as previously defined, and:

h_f = friction (head) loss, ft. of water.

p_f = friction (head) loss, psi

D_i = pipe inside diameter, in

C = Hazen-Williams Friction Factor, dimensionless $c = 150-155$ for PE, (not related to Darcy-Weisbach friction factor, f)

Q = flow rate, gpm

The Hazen-Williams Friction Factor, C , for PE pipe was determined in a hydraulics laboratory using heat fusion joined lengths of pipe with the inner bead present. Other forms of these equations are prevalent throughout the literature.⁽²¹⁾ The reader is referred to the references at the end of this chapter.

TABLE 2-3
Properties of Water

Temperature, °F/°C	Specific Weight, lb/ft ³	Kinematic Viscosity, Centistokes
32 / 0	62.41	1.79
60 / 15.6	62.37	1.13
75 / 23.9	62.27	0.90
100 / 37.8	62.00	0.69
120 / 48.9	61.71	0.57
140 / 60	61.38	0.47

Water flow through pipes having different Hazen-Williams factors and different flow diameters may be determined using the following equations:

(2-13)

$$\% \text{ flow} = 100 \left(\frac{D_{I2}}{D_{I1}} \right)^{2.63} \left(\frac{C_2}{C_1} \right)$$

Where the subscripts 1 and 2 refer to the designated properties for two separate pipe profiles, in this case, the pipe inside diameter (D_i in inches) of the one pipe (1) versus that of the second pipe (2) and the Hazen-Williams factor for each respective profile.

Pipe Flow Design Example

A PE pipeline conveying water at 60°F is 15,000 feet long and is laid on a uniform grade that rises 150 feet. What is the friction head loss in 4" IPS DR 17 PE 3408 pipe for a 50 gpm flow? What is the elevation head? What is the internal pressure at the bottom of the pipe when water is flowing uphill? When flowing downhill? When full but not flowing?

Using equation 2-12 and $C = 150$

$$p_f = \frac{0.0009015(15000)}{3.938^{4.8655}} \left(\frac{100(50)}{150} \right)^{1.85} = 11.3 \text{ psi}$$

To determine the elevation head, assume point 1 is at the bottom of the elevation, and point 2 is at the top. Using Equation 2-10,

$$h_E = 150 - 0 = 150 \text{ ft of water}$$

The specific weight of water at 60°F is 62.37 lb/ft³ (see Table 2-3), which, for each foot of head exerts a pressure of 62.37 lb over a 1 ft square area, or a pressure of 62.37/144 = 0.43 lb/in². Therefore, for a 150 ft. head,

$$h_E = (150 - 0)0.43 = 64.5 \text{ psig}$$

When water is flowing, elevation head and the friction head are added. The maximum friction head acts at the source point, and the maximum elevation head at the lowest point. Therefore, when flowing uphill, the pressure, P , at the bottom is elevation head plus the friction head because the flow is from the bottom to the top.

$$P = h_E + p_f = 64.5 + 11.3 = 75.8 \text{ psig}$$

When flowing downhill, water flows from the top to the bottom. Friction head applies from the source point at the top, so the pressure developed from the downhill flow is applied in the opposite direction as the elevation head. Therefore,

$$P = h_E - p_f = 64.5 - 11.3 = 53.2 \text{ psig}$$

When the pipe is full, but water is not flowing, no friction head develops.

$$P = h_E + p_f = 64.5 + 0 = 64.5 \text{ psig}$$

Pressure Flow of Liquid Slurries

Liquid slurry piping systems transport solid particles entrained in a liquid carrier. Water is typically used as a liquid carrier, and solid particles are commonly granular materials such as sand, fly-ash or coal. Key design considerations involve the nature of the solid material, its particle size and the carrier liquid.

Turbulent flow is preferred to ensure that particles are suspended in the liquid. Turbulent flow also reduces pipeline wear because particles suspended in the carrier liquid will bounce off the pipe inside surface. PE pipe has viscoelastic properties that combine with high molecular weight toughness to provide service life that can significantly exceed many metal piping materials. Flow velocity that is too low to maintain fully turbulent flow for a given particle size can allow solids to drift to the bottom of the pipe and slide along the surface. However, compared to metals, PE is a softer material. Under sliding bed and direct impingement conditions, PE may wear appreciably. PE directional fittings are generally unsuitable for slurry applications because the change of flow direction in the fitting results in direct impingement. Directional fittings in liquid slurry applications should employ hard materials that are resistant to wear from direct impingement.

Particle Size

As a general recommendation, particle size should not exceed about 0.2 in (5 mm), but larger particles are occasionally acceptable if they are a small percentage of the solids in the slurry. With larger particle slurries such as fine sand and coarser particles, the viscosity of the slurry mixture will be approximately that of the carrying liquid. However, if particle size is very small, about 15 microns or less, the slurry viscosity will increase above that of the carrying liquid alone. The rheology

of fine particle slurries should be analyzed for viscosity and specific gravity before determining flow friction losses. Inaccurate assumptions of a fluid's rheological properties can lead to significant errors in flow resistance analysis. Examples of fine particle slurries are water slurries of fine silt, clay and kaolin clay.

Slurries frequently do not have uniform particle size, and some particle size non-uniformity can aid in transporting larger particles. In slurries having a large proportion of smaller particles, the fine particle mixture acts as a more viscous carrying fluid that helps suspend larger particles. Flow analysis of non-uniform particle size slurries should include a rheological characterization of the fine particle mixture.

Solids Concentration and Specific Gravity

Equations 2-14 through 2-17 are useful in determining solids concentrations and mixture specific gravity. Tables 2-4, 2-5, and 2-6 provide information about specific gravity and particle size of some slurries.

$$(2-14) \quad C_V = \frac{S_M - S_L}{S_S - S_L}$$

$$(2-15) \quad C_W = \frac{C_V S_S}{S_M}$$

$$(2-16) \quad S_M = C_V (S_S - S_L) + S_L$$

$$(2-17) \quad S_M = \frac{S_L}{1 - \frac{C_W (S_S - S_L)}{S_S}}$$

WHERE

S_L = carrier liquid specific gravity

S_S = solids specific gravity

S_M = slurry mixture specific gravity

C_V = percent solids concentration by volume

C_W = percent solids concentration by weight

Critical Velocity

As pointed out above, turbulent flow is preferred to maintain particle suspension. A turbulent flow regime avoids the formation of a sliding bed of solids, excessive pipeline wear and possible clogging. Reynolds numbers above 4000 will generally insure turbulent flow.

Maintaining the flow velocity of a slurry at about 30% above the critical settlement velocity is a good practice. This insures that the particles will remain in suspension thereby avoiding the potential for excessive pipeline wear. For horizontal pipes, critical velocity may be estimated using Equation 2-18.

Individual experience with this equation varies. Other relationships are offered in the literature. See Thompson and Aude⁽²⁶⁾. A test section may be installed to verify applicability of this equation for specific projects.

$$(2-18) \quad V_C = F_L \sqrt{2gd' (S_s - 1)}$$

Where terms are previously defined and

V_C = critical settlement velocity, ft/sec

F_L = velocity coefficient (Tables 2-7 and 2-8)

d' = pipe inside diameter, ft

An approximate minimum velocity for fine particle slurries (below 50 microns, 0.05 mm) is 4 to 7 ft/sec, provided turbulent flow is maintained. A guideline minimum velocity for larger particle slurries (over 150 microns, 0.15 mm) is provided by Equation 2-19.

$$(2-19) \quad V_{\min} = 14\sqrt{d'}$$

WHERE

V_{\min} = approximate minimum velocity, ft/sec

Critical settlement velocity and minimum velocity for turbulent flow increases with increasing pipe bore. The relationship in Equation 2-20 is derived from the Darcy-Weisbach equation. (Equation 2-3)

$$(2-20) \quad V_2 = \frac{\sqrt{d'_2}}{\sqrt{d'_1}} V_1$$

The subscripts 1 and 2 are for the two pipe diameters.

TABLE 2-4
Scale of Particle Sizes

Tyler Screen Mesh	U.S. Standard Mesh	Inches	Microns	Class
–	–	1.3 – 2.5	33,000 – 63,500	Very coarse gravel
–	–	0.6 – 1.3	15,200 – 32,000	Coarse gravel
2.5	–	0.321	8,000	Medium gravel
5	5	0.157	4,000	Fine gravel
9	10	0.079	2,000	Very fine gravel
16	18	0.039	1,000	Very coarse sand
32	35	0.0197	500	Coarse sand
60	60	0.0098	250	Medium sand
115	120	0.0049	125	Fine sand
250	230	0.0024	62	Very fine sand
400	–	0.0015	37	Coarse silt
–	–	0.0006 – 0.0012	16 – 31	Medium silt
–	–	–	8 – 13	Fine silt
–	–	–	4 – 8	Very fine silt
–	–	–	2 – 4	Coarse clay
–	–	–	1 – 2	Medium clay
–	–	–	0.5 - 1	Fine clay

TABLE 2-5
Typical Specific Gravity and Slurry Solids Concentration (Water Slurries)

Material	Specific Gravity	Typical Solids Concentration	
		% by Weight	% by Volume
Gilsonite	1.05	40 – 45	39 – 44
Coal	1.40	45 – 55	37 – 47
Sand	2.65	43 – 43	23 – 30
Limestone	2.70	60 – 65	36 – 41
Copper Concentrate	4.30	60 – 65	26 – 30
Iron Ore	4.90	–	–
Iron Sands	1.90	–	–
Magnetite	4.90	60 - 65	23 - 27

TABLE 2-6
Water-Base Slurry Specific Gravities

Concentration by Weight Percent, C_w	Solid Specific Gravity, S_s									
	1.4	1.8	2.2	2.6	3.0	3.4	3.8	4.2	4.6	5.0
5	1.01	1.02	1.03	1.03	1.03	1.04	1.04	1.04	1.04	1.04
10	1.03	1.05	1.06	1.07	1.07	1.08	1.08	1.08	1.08	1.09
15	1.04	1.07	1.09	1.10	1.11	1.12	1.12	1.13	1.13	1.14
20	1.05	1.10	1.12	1.14	1.15	1.16	1.17	1.18	1.19	1.19
25	1.08	1.13	1.16	1.18	1.20	1.21	1.23	1.24	1.24	1.25
30	1.09	1.15	1.20	1.23	1.25	1.27	1.28	1.30	1.31	1.32
35	1.11	1.18	1.24	1.27	1.30	1.33	1.35	1.36	1.38	1.39
40	1.13	1.22	1.28	1.33	1.36	1.39	1.42	1.44	1.46	1.47
45	1.15	1.25	1.33	1.38	1.43	1.47	1.50	1.52	1.54	1.56
50	1.17	1.29	1.38	1.44	1.50	1.55	1.58	1.62	1.64	1.67
55	1.19	1.32	1.43	1.51	1.58	1.63	1.69	1.72	1.76	1.79
60	1.21	1.36	1.49	1.59	1.67	1.73	1.79	1.84	1.89	1.92
65	1.23	1.41	1.55	1.67	1.76	1.85	1.92	1.98	2.04	2.08
70	1.25	1.45	1.62	1.76	1.88	1.98	2.07	2.14	2.21	2.27

TABLE 2-7
Velocity Coefficient, F_L (Uniform Particle Size)

Particle Size, mm	Velocity Coefficient, F_L			
	$C_V = 2\%$	$C_V = 5\%$	$C_V = 10\%$	$C_V = 15\%$
0.1	.76	0.92	0.94	0.96
0.2	0.94	1.08	1.20	1.28
0.4	1.08	1.26	1.41	1.46
0.6	1.15	1.35	1.46	1.50
0.8	1.21	1.39	1.45	1.48
1.0	1.24	1.04	1.42	1.44
1.2	1.27	1.38	1.40	1.40
1.4	1.29	1.36	1.67	1.37
1.6	1.30	1.35	1.35	1.35
1.8	1.32	1.34	1.34	1.34
2.0	1.33	1.34	1.34	1.34
2.2	1.34	1.34	1.34	1.34
2.4	1.34	1.34	1.34	1.34
2.6	1.35	1.35	1.35	1.35
2.8	1.36	1.36	1.36	1.36
≥ 3.0	1.36	1.36	1.36	1.36

TABLE 2-8
Velocity Coefficient, F_L (50% Passing Particle Size)

Particle Size, mm	Velocity Coefficient, F_L			
	$C_V = 5\%$	$C_V = 10\%$	$C_V = 20\%$	$C_V = 30\%$
0.01	0.48	0.48	0.48	0.48
0.02	0.58	0.59	0.60	0.61
0.04	0.70	0.72	0.74	0.76
0.06	0.77	0.79	0.81	0.83
0.08	0.83	0.86	0.86	0.91
0.10	0.85	0.88	0.92	0.95
0.20	0.97	1.00	1.05	1.08
0.40	1.09	1.13	1.18	1.23
0.60	1.15	1.21	1.26	1.30
0.80	1.21	1.25	1.31	1.33
1.0	1.24	1.29	1.33	1.35
2.0	1.33	1.36	1.38	1.40
3.0	1.36	1.38	1.39	1.40

Equation 2-3, Darcy-Weisbach, and Equations 2-11 and 2-12, Hazen-Williams, may be used to determine friction head loss for pressure slurry flows provided the viscosity limitations of the equations are taken into account. Elevation head loss is increased by the specific gravity of the slurry mixture.

$$(2-21) \quad h_E = S_M (h_2 - h_1)$$

Compressible Gas Flow

Flow equations for smooth pipe may be used to estimate compressible gas flow through PE pipe.

Empirical Equations for High Pressure Gas Flow

Equations 2-22 through 2-25 are empirical equations used in industry for pressure greater than 1 psig. Calculated results may vary due to the assumptions inherent in the derivation of the equation.

Mueller Equation

$$(2-22) \quad Q_h = \frac{2826 D_I^{2.725}}{S_g^{0.425}} \left(\frac{p_1^2 - p_2^2}{L} \right)^{0.575}$$

Weymouth Equation

$$(2-23) \quad Q_h = \frac{2034 D_I^{2.667}}{S_g^{0.5}} \left(\frac{p_1^2 - p_2^2}{L} \right)^{0.5}$$

IGT Distribution Equation

$$(2-24) \quad Q_h = \frac{2679 D_I^{2.667}}{S_g^{0.444}} \left(\frac{p_1^2 - p_2^2}{L} \right)^{0.555}$$

Spitzglass Equation

$$(2-25) \quad Q_h = \frac{3410}{S_g^{0.5}} \left(\frac{p_1^2 - p_2^2}{L} \right)^{0.5} \left(\frac{D_I^5}{1 + \frac{3.6}{D_I} + 0.03 D_I} \right)^{0.5}$$

WHERE

(Equations 2-22 through 2-25)

Q_h = flow, standard ft³/hour

S_g = gas specific gravity

p_1 = inlet pressure, lb/in² absolute

p_2 = outlet pressure, lb/in² absolute

L = length, ft

D_I = pipe inside diameter, in

Empirical Equations for Low Pressure Gas Flow

For applications where internal pressures are less than 1 psig, such as landfill gas gathering or wastewater odor control, Equations 2-26 or 2-27 may be used.

Mueller Equation

$$(2-26) \quad Q_h = \frac{2971 D_I^{2.725}}{S_g^{0.425}} \left(\frac{h_1 - h_2}{L} \right)^{0.575}$$

Spitzglass Equation

$$(2-27) \quad Q_h = \frac{3350}{S_g^{0.5}} \left(\frac{h_1 - h_2}{L} \right)^{0.5} \left(\frac{D_I^5}{1 + \frac{3.6}{D_I} + 0.03 D_I} \right)^{0.5}$$

Where terms are previously defined, and

h_1 = inlet pressure, in H₂O

h_2 = outlet pressure, in H₂O

Gas Permeation

Long distance pipelines carrying compressed gasses may deliver slightly less gas due to gas permeation through the pipe wall. Permeation losses are small, but it may be necessary to distinguish between permeation losses and possible leakage. Equation 2-28 may be used to determine the volume of a gas that will permeate through PE pipe of a given wall thickness:

$$(2-28) \quad q_P = \frac{K_P A_s \Theta P_A}{t'}$$

WHERE

q_P = volume of gas permeated, cm^3 (gas at standard temperature and pressure)

K_P = permeability constant (Table 2-9); units: $\frac{\text{Cm}^3 \text{ mil}}{100 \text{ in}^2 \text{ atm day}}$

A_s = pipe outside wall area in units of 100 square inches

P_A = pipe internal pressure, atmospheres (1 atmosphere = 14.7 lb/in^2)

Θ = elapsed time, days

t' = wall thickness, mils

TABLE 2-9
Permeability Constants⁽²⁸⁾

Gas	K_P
Methane	85
Carbon Monoxide	80
Hydrogen	425

TABLE 2-10
Physical Properties of Gases (Approx. Values at 14.7 psi & 68°F)

Gas	Chemical Formula	Molecular Weight	Weight Density, lb/ft ³	Specific Gravity, (Relative to Air) S _g
Acetylene (ethylene)	C ₂ H ₂	26.0	0.0682	0.907
Air	–	29.0	0.0752	1.000
Ammonia	NH ₃	17.0	0.0448	0.596
Argon	A	39.9	0.1037	1.379
Butane	C ₄ H ₁₀	58.1	0.1554	2.067
Carbon Dioxide	CO ₂	44.0	0.1150	1.529
Carbon Monoxide	CO	28.0	0.0727	0.967
Ethane	C ₂ H ₆	30.0	0.0789	1.049
Ethylene	C ₂ H ₄	28.0	0.0733	0.975
Helium	He	4.0	0.0104	0.138
Hydrogen Chloride	HCl	36.5	0.0954	1.286
Hydrogen	H	2.0	0.0052	0.070
Hydrogen Sulphide	H ₂ S	34.1	0.0895	1.190
Methane	CH ₄	16.0	0.0417	0.554
Methyl Chloride	CH ₃ Cl	50.5	0.1342	1.785
Natural Gas	–	19.5	0.0502	0.667
Nitric Oxide	NO	30.0	0.0708	1.037
Nitrogen	N ₂	28.0	0.0727	0.967
Nitrous Oxide	N ₂ O	44.0	0.1151	1.530
Oxygen	O ₂	32.0	0.0831	1.105
Propane	C ₃ H ₈	44.1	0.1175	1.562
Propene (Propylene)	C ₃ H ₆	42.1	0.1091	1.451
Sulfur Dioxide	SO ₂	64.1	0.1703	2.264
Landfill Gas (approx. value)	–	–	–	1.00
Carbureted Water Gas	–	–	–	0.63
Coal Gas	–	–	–	0.42
Coke-Oven Gas	–	–	–	0.44
Refinery Oil Gas	–	–	–	0.99
Oil Gas (Pacific Coast)	–	–	–	0.47
“Wet” Gas (approximate value)	–	–	–	0.75

Gravity Flow of Liquids

In a pressure pipeline, a pump of some sort, generally provides the energy required to move the fluid through the pipeline. Such pipelines can transport fluids across a level surface, uphill or downhill. Gravity flow lines, on the other hand, utilize the energy associated with the placement of the pipeline discharge below the inlet. Like pressure flow pipelines, friction loss in a gravity flow pipeline depends on viscous shear stresses within the liquid and friction along the wetted surface of the pipe bore.

Some gravity flow piping systems may become very complex, especially if the pipeline grade varies, because friction loss will vary along with the varying grade. Sections of the pipeline may develop internal pressure, or vacuum, and may have varying liquid levels in the pipe bore.

Manning Flow Equation

For open channel water flow under conditions of constant grade, and uniform channel cross section, the Manning equation may be used.^(29,30) Open channel flow exists in a pipe when it runs partially full. Like the Hazen-Williams formula, the Manning equation is applicable to water or liquids with a kinematic viscosity equal to water.

Manning Equation

$$(2-29) \quad V = \frac{1.486}{n} r_H^{2/3} S_H^{1/2}$$

WHERE

V = flow velocity, ft/sec

n = roughness coefficient, dimensionless

r_H = hydraulic radius, ft

S_H = hydraulic slope, ft/ft

$$(2-30) \quad r_H = \frac{A_C}{P_W}$$

A_C = cross-sectional area of flow bore, ft²

P_W = perimeter wetted by flow, ft

$$(2-31) \quad S_H = \frac{h_U - h_D}{L} = \frac{h_f}{L}$$

h_U = upstream pipe elevation, ft

h_D = downstream pipe elevation, ft

h_f = friction (head) loss, ft of liquid

L = length, ft

It is convenient to combine the Manning equation with

$$(2-32) \quad Q = A_C V$$

To obtain

$$(2-33) \quad Q = \frac{1.486 A_C}{n} r_H^{2/3} S_H^{1/2}$$

Where terms are as defined above, and

Q = flow, ft³/sec

When a circular pipe is running full or half-full,

$$(2-34) \quad r_H = \frac{d'}{4} = \frac{D_I}{48}$$

WHERE

d' = pipe inside diameter, ft

D_I = pipe inside diameter, in

Full pipe flow in ft³ per second may be estimated using:

$$(2-35) \quad Q_{FPS} = \left(6.136 \times 10^{-4}\right) \frac{D_I^{8/3} S_H^{1/2}}{n}$$

Full pipe flow in gallons per minute may be estimated using:

$$(2-36) \quad Q' = 0.275 \frac{D_I^{8/3} S_H^{1/2}}{n}$$

Nearly full circular pipes will carry more liquid than a completely full pipe. When slightly less than full, the perimeter wetted by flow is reduced, but the actual flow area is only slightly lessened. This results in a larger hydraulic radius than when the pipe is running full. Maximum flow is achieved at about 93% of full pipe flow, and maximum velocity at about 78% of full pipe flow. Manning’s n is often assumed to be constant with flow depth. Actually, n has been found to be slightly larger in non-full flow.

TABLE 2-11
Values of n for Use with Manning Equation

Surface	n , typical design
PE pipe	0.009
Uncoated cast or ductile iron pipe	0.013
Corrugated steel pipe	0.024
Concrete pipe	0.013
Vitrified clay pipe	0.013
Brick and cement mortar sewers	0.015
Wood stave	0.011
Rubble masonry	0.021

Note: The n -value of 0.009 for PE pipe is for clear water applications. An n -value of 0.010 is typically utilized for applications such as sanitary sewer, etc.

Comparative Flows for Slipliners

Deteriorated gravity flow pipes may be rehabilitated by sliplining with PE pipe. This process involves the installation of a PE liner inside of the deteriorated original pipe as described in subsequent chapters within this manual. For conventional sliplining, clearance between the liner outside diameter and the existing pipe bore is required to install the liner; thus after rehabilitation, the flow channel is smaller than that of the original pipe. However, it is often possible to rehabilitate with a PE slipliner, and regain all or most of the original flow capacity due to the extremely smooth inside surface of the PE pipe and its resistance to deposition or build-up. Because PE pipe is mostly joined by means of butt-fusion, this results in no effective reduction of flow diameter at joint locations. Comparative flow capacities of circular pipes may be determined by the following:

(2-37)

$$\% \text{ flow} = 100 \frac{Q_1}{Q_2} = 100 \frac{\left(\frac{D_{I1}^{8/3}}{n_1} \right)}{\left(\frac{D_{I2}^{8/3}}{n_2} \right)}$$

Table 2-12 was developed using Equation 2-36 where D_{I1} = the inside diameter (ID) of the liner, and D_{I2} = the original inside diameter of the deteriorated host pipe.

TABLE 2-12
Comparative Flows for Slipliners

Existing Sewer ID, in	Liner OD, in.	Liner DR 32.5			Liner DR 26			Liner DR 21			Liner DR 17		
		Liner ID, in.†	% flow vs. concrete	% flow vs. clay	Liner ID, in.†	% flow vs. concrete	% flow vs. clay	Liner ID, in.†	% flow vs. concrete	% flow vs. clay	Liner ID, in.†	% flow vs. concrete	% flow vs. clay
4	3.500	3.272	97.5%	84.5%	3.215	93.0%	80.6%	3.147	87.9%	76.2%	3.064	81.8%	70.9%
6	4.500	4.206	64.6%	56.0%	4.133	61.7%	53.5%	4.046	58.3%	50.5%	3.939	54.3%	47.0%
6	5.375	5.024	103.8%	90.0%	4.937	99.1%	85.9%	4.832	93.6%	81.1%	4.705	87.1%	75.5%
8	6.625	6.193	84.2%	73.0%	6.085	80.3%	69.6%	5.956	75.9%	65.8%	5.799	70.7%	61.2%
8	7.125	6.660	102.2%	88.6%	6.544	97.5%	84.5%	6.406	92.1%	79.9%	6.236	85.8%	74.4%
10	8.625	8.062	93.8%	81.3%	7.922	89.5%	77.6%	7.754	84.6%	73.3%	7.549	78.8%	68.3%
12	10.750	10.049	103.8%	90.0%	9.873	99.1%	85.9%	9.665	93.6%	81.1%	9.409	87.1%	75.5%
15	12.750	11.918	90.3%	78.2%	11.710	86.1%	74.6%	11.463	81.4%	70.5%	11.160	75.7%	65.6%
15	13.375	12.503	102.5%	88.9%	12.284	97.8%	84.8%	12.025	92.4%	80.1%	11.707	86.1%	74.6%
16	14.000	13.087	97.5%	84.5%	2.858	93.0%	80.6%	12.587	87.9%	76.2%	12.254	81.8%	70.9%
18	16.000	14.956	101.7%	88.1%	14.695	97.0%	84.1%	14.385	91.7%	79.4%	14.005	85.3%	74.0%
21	18.000	16.826	92.3%	80.0%	16.532	88.1%	76.3%	16.183	83.2%	72.1%	15.755	77.5%	67.1%
24	20.000	18.695	85.6%	74.2%	18.369	81.7%	70.8%	17.981	77.2%	66.9%	17.506	71.9%	62.3%
24	22.000	20.565	110.4%	95.7%	20.206	105.3%	91.3%	19.779	99.5%	86.2%	19.256	92.6%	80.3%
27	24.000	22.434	101.7%	88.1%	22.043	97.0%	84.1%	21.577	91.7%	79.4%	21.007	85.3%	74.0%
30	28.000	26.174	115.8%	100.4%	25.717	110.5%	95.8%	25.173	104.4%	90.5%	24.508	97.2%	84.2%
33	30.000	28.043	108.0%	93.6%	27.554	103.0%	89.3%	26.971	97.3%	84.3%	26.259	90.6%	78.5%
36	32.000	29.913	101.7%	88.1%	29.391	97.0%	84.1%	28.770	91.7%	79.4%	28.009	85.3%	74.0%
36	34.000	31.782	119.5%	103.6%	31.228	114.1%	98.9%	30.568	107.7%	93.4%	29.760	100.3%	86.9%
42	36.000	33.652	92.3%	80.0%	33.065	88.1%	76.3%	32.366	83.2%	72.1%	31.511	77.5%	67.1%
48	42.000	39.260	97.5%	84.5%	38.575	93.0%	80.6%	37.760	87.9%	76.2%	36.762	81.8%	70.9%
54	48.000	44.869	101.7%	88.1%	44.086	97.0%	84.1%	43.154	91.7%	79.4%	42.014	85.3%	74.0%
60	54.000	50.478	105.1%	91.1%	49.597	100.3%	86.9%	48.549	94.8%	82.1%	47.266	88.2%	76.5%

† Liner ID calculated per Equation 2-1.

Flow Velocity

Acceptable flow velocities in PE pipe depend on the specific details of the system. For water systems operating at rated pressures, velocities may be limited by surge allowance requirements. See Tables 1-3A and 1-3B. Where surge effects are reduced, higher velocities are acceptable, and if surge is not possible, such as in many gravity flow systems, water flow velocities exceeding 25 feet per second may be acceptable.

Liquid flow velocity may be limited by the capabilities of pumps or elevation head to overcome friction (head) loss and deliver the flow and pressure required for the application. PE pipe is not eroded by water flow. Liquid slurry pipelines may be subject to critical minimum velocities that ensure turbulent flow and maintain particle suspension in the slurry.

Gravity liquid flows of 2 fps (0.6 m/s) and higher can help prevent or reduce solids deposition in sewer lines. When running full, gravity flow pipelines are subject to the same velocity considerations as pressure pipelines.

Flow velocity in compressible gas lines tends to be self-limiting. Compressible gas flows in PE pipes are typically laminar or transitional. Fully turbulent flows are possible in short pipelines, but difficult to achieve in longer transmission and distribution lines because the pressure ratings for PE pipe automatically limit flow capacity and, therefore, flow velocity.

Pipe Surface Condition, Aging

Aging acts to increase pipe surface roughness in most piping systems. This in turn increases flow resistance. PE pipe resists typical aging effects because PE does not rust, rot, corrode, tuberculate, or support biological growth, and it resists the adherence of scale and deposits. In some cases, moderate flow velocities are sufficient to prevent deposition, and where low velocities predominate, occasional high velocity flows will help to remove sediment and deposits. As a result, the initial design capabilities for pressure and gravity flow pipelines are retained as the pipeline ages.

Where cleaning is needed to remove depositions in low flow rate gravity flow pipelines, water-jet cleaning or forcing a “soft” (plastic foam) pig through the pipeline are effective cleaning methods. Bucket, wire and scraper-type cleaning methods will damage PE pipe and must not be used.

Section 3

Buried PE Pipe Design

Introduction

This section covers basic engineering information for calculating earth and live-load pressures on PE pipe, for finding the pipe's response to these pressures taking into account the interaction between the pipe and its surrounding soil, and for judging that an adequate safety factor exists for a given application.

Soil pressure results from the combination of soil weight and surface loads. As backfill is placed around and over a PE pipe, the soil pressure increases and the pipe deflects vertically and expands laterally into the surrounding soil. The lateral expansion mobilizes passive resistance in the soil which, in combination with the pipe's inherent stiffness, resists further lateral expansion and consequently further vertical deflection.

During backfilling, ring (or hoop) stress develops within the pipe wall. Ring bending stresses (tensile and compressive) occur as a consequence of deflection, and ring compressive stress occurs as a consequence of the compressive thrust created by soil compression around the pipe's circumference. Except for shallow pipe subject to live load, the combined ring stress from bending and compression results in a net compressive stress.

The magnitude of the deflection and the stress depends not only on the pipe's properties but also on the properties of the surrounding soil. The magnitude of deflection and stress must be kept safely within PE pipe's performance limits. Excessive deflection may cause loss of stability and flow restriction, while excessive compressive stress may cause wall crushing or ring buckling. Performance limits for PE pipe are given in Watkins, Szpak, and Allman⁽¹⁾ and illustrated in Figure 3-1.

The design and construction requirements can vary somewhat, depending on whether the installation is for pressure or non-pressure service. These differences will be addressed later in this chapter and in Chapter 7, "Underground Installation of PE Pipe."

Calculations

Section 3 describes how to calculate the soil pressure acting on PE pipe due to soil weight and surface loads, how to determine the resulting deflection based on pipe and soil properties, and how to calculate the allowable (safe) soil pressure for wall compression (crushing) and ring buckling for PE pipe.

Detailed calculations are not always necessary to determine the suitability of a particular PE pipe for an application. Pressure pipes that fall within the Design Window given in AWWA M-55 “PE Pipe – Design and Installation” regarding pipe DR, installation, and burial depth meet specified deflection limits for PE pipe, have a safety factor of at least 2 against buckling, and do not exceed the allowable material compressive stress for PE. Thus, the designer need not perform extensive calculations for pipes that are sized and installed in accordance with the Design Window.

AWWA M-55 Design Window

AWWA M-55, “PE Pipe – Design and Installation”, describes a Design Window. Applications that fall within this window require no calculations other than constrained buckling per Equation 3-15. It turns out that if pipe is limited to DR 21 or lower as in Table 3-1, the constrained buckling calculation has a safety factor of at least 2, and no calculations are required.

The design protocol under these circumstances (those that fall within the AWWA Design Window) is thereby greatly simplified. The designer may choose to proceed with detailed analysis of the burial design and utilize the AWWA Design Window guidelines as a means of validation for his design calculations and commensurate safety factors. Alternatively, he may proceed with confidence that the burial design for these circumstances (those outlined within the AWWA Design Window) has already been analyzed in accordance with the guidelines presented in this chapter.

The Design Window specifications are:

- Pipe made from stress-rated PE material.
- Essentially no dead surface load imposed over the pipe, no ground water above the surface, and provisions for preventing flotation of shallow cover pipe have been provided.
- The embedment materials are coarse-grained, compacted to at least 85% Standard Proctor Density and have an E' of at least 1000 psi. The native soil must be stable; in other words the native soil must have an E' of at least 1000 psi. See Table 3-7.
- The unit weight of the native soil does not exceed 120 pcf.
- The pipe is installed in accordance with manufacturer’s recommendations for controlling shear and bending loads and minimum bending radius, and installed

in accordance with ASTM D2774 for pressure pipes or ASTM D2321 for non-pressure pipes.

- Minimum depth of cover is 2 ft (0.61 m); except when subject to AASHTO H20 truck loadings, in which case the minimum depth of cover is the greater of 3 ft (0.9 m) or one pipe diameter.
- Maximum depth of cover is 25 ft (7.62 m).

TABLE 3-1
AWWA M-55 Design Window Maximum and Minimum Depth of Cover Requiring No Calculations

DR	Min. Depth of Cover With H2O Load	Min. Depth of Cover Without H2O Load	Maximum Depth of Cover
7.3	3 ft	2 ft	25 ft
9	3 ft	2 ft	25 ft
11	3 ft	2 ft	25 ft
13.5	3 ft	2 ft	25 ft
17	3 ft	2 ft	25 ft
21	3 ft	2 ft	25 ft

* Limiting depths where no calculations are required. Pipes are suitable for deeper depth provided a sufficient E' (1,000 psi or more) is accomplished during installations. Calculations would be required for depth greater than 25 ft.

Installation Categories

For the purpose of calculation, buried installations of PE pipe can be separated into four categories depending on the depth of cover, surface loading, groundwater level and pipe diameter. Each category involves slightly different equations for determining the load on the pipe and the pipe's response to the load. The boundaries between the categories are not definite, and engineering judgment is required to select the most appropriate category for a specific installation. The categories are:

- 1. Standard Installation-Trench or Embankment** installation with a maximum cover of 50 ft with or without traffic, rail, or surcharge loading. To be in this category, where live loads are present the pipe must have a minimum cover of at least one diameter or 18" whichever is greater. Earth pressure applied to the pipe is found using the prism load (geostatic soil stress). The Modified Iowa Formula is used for calculating deflection. Crush and buckling are performance limits as well. The Standard Installation section also presents the AWWA "Design Window."
- 2. Shallow Cover Vehicular Loading Installation** applies to pipes buried at a depth of at least 18" but less than one pipe diameter. This installation category

uses the same equations as the Standard Installation but with an additional equation relating wheel load to the pipe’s bending resistance and the soil’s supporting strength.

3. **Deep Fill Installation** applies to embankments with depths exceeding 50 ft. The soil pressure calculation may be used for profile pipe in trenches less than 50 ft. The Deep Fill Installation equations differ from the Standard Installation equations by considering soil pressure based on armored, calculating deflection from the Watkins-Gaube Graph, and calculating buckling with the Moore-Selig Equation.
4. **Shallow Cover Flotation Effects** applies to applications where insufficient cover is available to either prevent flotation or hydrostatic collapse. Hydrostatic buckling is introduced in this chapter because of its use in subsurface design.

Section 3 of the Design Chapter is limited to the design of PE pipes buried in trenches or embankments. The load and pipe reaction calculations presented may not apply to pipes installed using trenchless technologies such as pipe bursting and directional drilling. These pipes may not develop the same soil support as pipe installed in a trench. The purveyor of the trenchless technology should be consulted for piping design information. See the Chapter on “PE Pipe for Horizontal Directional Drilling” and ASTM F1962, *Use of Maxi-Horizontal Directional Drilling (HDD) for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings* for additional information on design of piping installed using directional drilling.

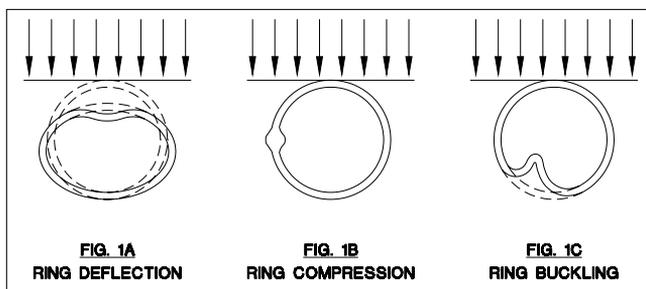


Figure 3-1 Performance Limits for Buried PE Pipe

Design Process

The interaction between pipe and soil, the variety of field-site soil conditions, and the range of available pipe Dimension Ratios make the design of buried pipe seem challenging. This section of the Design Chapter has been written with the intent of easing the designer’s task. While some very sophisticated design approaches for

buried pipe systems may be justified in certain applications, the simpler, empirical methodologies presented herein have been proven by experience to provide reliable results for virtually all PE pipe installations.

The design process consists of the following steps:

1. Determine the **vertical soil pressure** acting at the crown of the pipe due to earth, live, and surcharge loads.
2. Select a **trial pipe**, which means selecting a **trial dimension ratio (DR)** or, in the case of profile pipe, a **trial profile**.
3. Select an embedment material and degree of compaction. As will be described later, soil type and compaction are relatable to a specific **modulus of soil reaction value (E')** (Table 3-8). (As deflection is proportional to the combination of pipe and soil stiffness, pipe properties and embedment stiffness can be traded off to obtain an optimum design.)
4. For the trial pipe and trial modulus of soil reaction, **calculate** the deflection due to the vertical soil pressure. **Compare** the pipe deflection to the deflection limit. If deflection exceeds the limit, it is generally best to look at increasing the modulus of soil reaction rather than reducing the DR or changing to a heavier profile. Repeat step 4 for the new E' and/or new trial pipe.
5. For the trial pipe and trial modulus of soil reaction, **calculate** the allowable soil pressure for wall crushing and for wall buckling. **Compare** the allowable soil pressure to the applied vertical pressure. If the allowable pressure is equal to or higher than the applied vertical pressure, the design is complete. If not, select a different pipe DR or heavier profile or different E' , and repeat step 5.

Since design begins with calculating vertical soil pressure, it seems appropriate to discuss the different methods for finding the vertical soil pressure on a buried pipe before discussing the pipe's response to load within the four installation categories.

Earth, Live, and Surcharge Loads on Buried Pipe

Vertical Soil Pressure

The weight of the earth, as well as surface loads above the pipe, produce soil pressure on the pipe. The weight of the earth or "earth load" is often considered to be a "dead-load" whereas surface loads are referred to as "surcharge loads" and may be temporary or permanent. When surcharge loads are of short duration they are usually referred to as "live loads." The most common live load is vehicular load. Other common surcharge loads include light structures, equipment, and piles of stored materials or debris. This section gives formulas for calculating the vertical

soil pressure due to both earth and surcharge loads. The soil pressures are normally calculated at the depth of the pipe crown. The soil pressures for earth load and each surcharge load are added together to obtain the total vertical soil pressure which is then used for calculating deflection and for comparison with wall crush and wall buckling performance limits.

Earth Load

In a uniform, homogeneous soil mass, the soil load acting on a horizontal plane within the mass is equal to the weight of the soil directly above the plane. If the mass contains areas of varying stiffness, the weight of the mass will redistribute itself toward the stiffer areas due to internal shear resistance, and arching will occur. Arching results in a reduction in load on the less stiff areas. Flexible pipes including PE pipes are normally not as stiff as the surrounding soil, so the resulting earth pressure acting on PE pipe is reduced by arching and is less than the weight of soil above the pipe. (One minor exception to this is shallow cover pipe under dynamic loads.) For simplicity, engineers often ignore arching and assume that the earth load on the pipe is equal to the weight of soil above the pipe, which is referred to as the “prism load” or “geostatic stress.” Practically speaking, the prism load is a conservative loading for PE pipes. It may be safely used in virtually all designs. Equation 3-1 gives the vertical soil pressure due to the prism load. The depth of cover is the depth from the ground surface to the pipe crown.

$$(3-1) P_E = wH$$

WHERE

P_E = vertical soil pressure due to earth load, psf

w = unit weight of soil, pcf

H = depth of cover, ft

UNITS CONVENTION: To facilitate calculations for PE pipes, the convention used with rigid pipes for taking the load on the pipe as a line load along the longitudinal axis in units of **lbs/lineal-ft** of pipe length is not used here. Rather, the load is treated as a soil pressure acting on a horizontal plane at the pipe crown and is given in units of **lbs/ft² or psf**.

Soil weight can vary substantially from site to site and within a site depending on composition, density and load history. Soil weights are often found in the construction site geotechnical report. The saturated unit weight of the soil is used when the pipe is below the groundwater level. For design purposes, the unit weight of dry soil is commonly assumed to be 120 pcf, when site-specific information is not available.

Generally, the soil pressure on profile pipe and on DR pipe in deep fills is significantly less than the prism load due to arching. For these applications, soil pressure is best calculated using the calculations that account for arching in the “Deep Fill Installation” section.

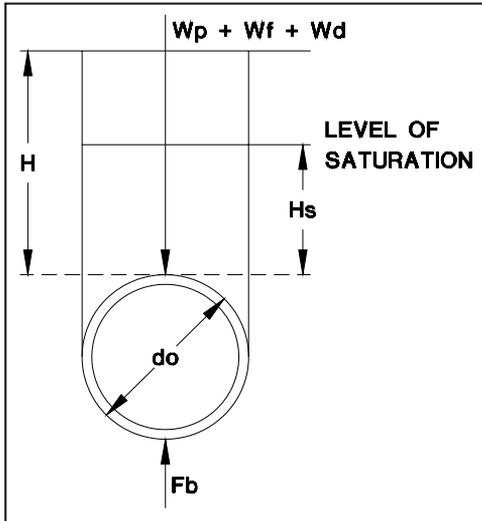


Figure 3-2 Prism Load

Live Load

Even though wheel loadings from cars and other light vehicles may be frequent, these loads generally have little impact on subsurface piping compared to the less frequent but significantly heavier loads from trucks, trains, or other heavy vehicles. For design of pipes under streets and highways, only the loadings from these heavier vehicles are considered. The pressure transmitted to a pipe by a vehicle depends on the pipe's depth, the vehicle's weight, the tire pressure and size, vehicle speed, surface smoothness, the amount and type of paving, the soil, and the distance from the pipe to the point of loading. For the more common cases, such as AASHTO, H20 HS20 truck traffic on paved roads and E-80 rail loading, this information has been simplified and put into Table 3-3, 3-4, and 3-5 to aid the designer. For special cases, such as mine trucks, cranes, or off-road vehicles, Equations 3-2 and 3-4 may be used.

The maximum load under a wheel occurs at the surface and diminishes with depth. PE pipes should be installed a minimum of one diameter or 18", whichever is greater, beneath the road surface. At this depth, the pipe is far enough below the wheel load to significantly reduce soil pressure and the pipe can fully utilize the embedment soil for load resistance. Where design considerations do not permit installation with

at least one diameter of cover, additional calculations are required and are given in the section discussing “Shallow Cover Vehicular Loading Installation.” State highway departments often regulate minimum cover depth and may require 2.5 ft to 5 ft of cover depending on the particular roadway.

During construction, both permanent and temporary underground pipelines may be subjected to heavy vehicle loading from construction equipment. It may be advisable to provide a designated vehicle crossing with special measures such as temporary pavement or concrete encasement, as well as vehicle speed controls to limit impact loads.

The following information on AASHTO Loading and Impact Factor is not needed to use Tables 3-3 and 3-4. It is included to give the designer an understanding of the surface loads encountered and typical impact factors. If the designer decides to use Equations 3-2 or 3-4 rather than the tables, the information will be useful.

AASHTO Vehicular Loading

Vehicular loads are typically based on The American Association of State Highway and Transportation Officials (AASHTO) standard truck loadings. For calculating the soil pressure on flexible pipe, the loading is normally assumed to be an H20 (HS20) truck. A standard H20 truck has a total weight of 40,000 lbs (20 tons). The weight is distributed with 8,000 lbs on the front axle and 32,000 lbs on the rear axle. The HS20 truck is a tractor and trailer unit having the same axle loadings as the H20 truck but with two rear axles. See Figure 3-3. For these trucks, the maximum wheel load is found at the rear axle(s) and equals 40 percent of the total weight of the truck.

The maximum wheel load may be used to represent the static load applied by either a single axle or tandem axles. Some states permit heavier loads. The heaviest tandem axle loads normally encountered on highways are around 40,000 lbs (20,000 lbs per wheel). Occasionally, vehicles may be permitted with loads up to 50 percent higher.

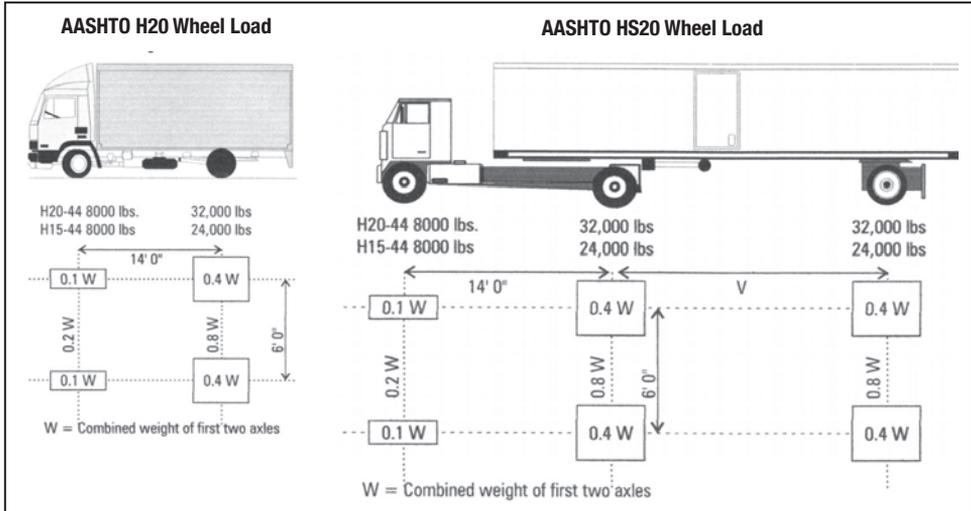


Figure 3-3 AASHTO H20 and HS20 Vehicle Loads

Impact Factor

Road surfaces are rarely smooth or perfectly even. When vehicles strike bumps in the road, the impact causes an instantaneous increase in wheel loading. Impact load may be found by multiplying the static wheel load by an impact factor. The factor varies with depth. Table 3-2 gives impact factors for vehicles on paved roads. For unpaved roads, impact factors of 2.0 or higher may occur, depending on the road surface.

TABLE 3-2
Typical Impact Factors for Paved Roads

Cover Depth, ft	Impact Factor, I_f
1	1.35
2	1.30
3	1.25
4	1.20
6	1.10
8	1.00

Derived from Illinois DOT dynamic load formula (1996).

Vehicle Loading through Highway Pavement (Rigid)

Pavement reduces the live load pressure reaching a pipe. A stiff, rigid pavement spreads load out over a large subgrade area thus significantly reducing the vertical

soil pressure. Table 3-3 gives the vertical soil pressure underneath an H20 (HS20) truck traveling on a paved highway (12-inch thick concrete). An impact factor is incorporated. For use with heavier trucks, the pressures in Table 3-3 can be adjusted proportionally to the increased weight as long as the truck has the same tire area as an HS20 truck.

TABLE 3-3
Soil Pressure under H20 Load (12" Thick Pavement)

Depth of cover, ft.	Soil Pressure, lb/ft ²
1	1800
1.5	1400
2	800
3	600
4	400
5	250
6	200
7	175
8	100
Over 8	Neglect

Note: For reference see ASTM F7906. Based on axle load equally distributed over two 18 by 20 inch areas, spaced 72 inches apart. Impact factor included.

Vehicle Loading through Flexible Pavement or Unpaved Surface

Flexible pavements (or unpaved surfaces) do not have the bridging ability of rigid pavement and thus transmit more pressure through the soil to the pipe than given by Table 3-3. In many cases, the wheel loads from two vehicles passing combine to create a higher soil pressure than a single dual-tire wheel load. The maximum pressure may occur directly under the wheels of one vehicle or somewhere in between the wheels of the two vehicles depending on the cover depth. Table 3-4 gives the largest of the maximum pressure for two passing H20 trucks on an unpaved surface. No impact factor is included. The loading in Table 3-3 is conservative and about 10% higher than loads found by the method given in AASHTO Section 3, LRFD Bridge Specifications Manual based on assuming a single dual-tire contact area of 20 x 10 inches and using the equivalent area method of load distribution.

TABLE 3-4
Soil Pressure Under H20 Load (Unpaved or Flexible Pavement)

Depth of cover, ft.	Soil Pressure, lb/ft ²
1.5	2000
2.0	1340
2.5	1000
3.0	710
3.5	560
4.0	500
6.0	310
8.0	200
10.0	140

Note: Based on integrating the Boussinesq equation for two H20 loads spaced 4 feet apart or one H20 load centered over pipe. No pavement effects or impact factor included.

Off-Highway Vehicles

Off-highway vehicles such as mine trucks and construction equipment may be considerably heavier than H20 trucks. These vehicles frequently operate on unpaved construction or mine roads which may have very uneven surfaces. Thus, except for slow traffic, an impact factor of 2.0 to 3.0 should be considered. For off-highway vehicles, it is generally necessary to calculate live load pressure from information supplied by the vehicle manufacturer regarding the vehicle weight or wheel load, tire footprint (contact area) and wheel spacing.

The location of the vehicle's wheels relative to the pipe is also an important factor in determining how much load is transmitted to the pipe. Soil pressure under a point load at the surface is dispersed through the soil in both depth and expanse. Wheel loads not located directly above a pipe may apply pressure to the pipe, and this pressure can be significant. The load from two wheels straddling a pipe may produce a higher pressure on a pipe than from a single wheel directly above it.

For pipe installed within a few feet of the surface, the maximum soil pressure will occur when a single wheel (single or dual tire) is directly over the pipe. For deeper pipes, the maximum case often occurs when vehicles traveling above the pipe pass within a few feet of each other while straddling the pipe, or in the case of off-highway vehicles when they have closely spaced axles. The minimum spacing between the centerlines of the wheel loads of passing vehicles is assumed to be four feet. At this spacing for H20 loading, the pressure on a pipe centered midway between the two passing vehicles is greater than a single wheel load on a pipe at or below a depth of about four feet.

For design, the soil pressure on the pipe is calculated based on the vehicle location (wheel load locations) relative to the pipe that produces the maximum pressure. This

generally involves comparing the pressure under a single wheel with that occurring with two wheels straddling the pipe. The Timoshenko Equation can be used to find the pressure directly under a single wheel load, whereas the Boussinesq Equation can be used to find the pressure from wheels not directly above the pipe.

Timoshenko’s Equation

The Timoshenko Equation gives the soil pressure at a point directly under a distributed surface load, neglecting any pavement.

$$(3-2) \quad P_L = \frac{I_f W_w}{a_C} \left(1 - \frac{H^3}{(r_T^2 + H^2)^{1.5}} \right)$$

WHERE

P_L = vertical soil pressure due to live load, lb/ft²

I_f = impact factor

W_w = wheel load, lb

a_C = contact area, ft²

r_T = equivalent radius, ft

H = depth of cover, ft

The equivalent radius is given by:

$$(3-3) \quad r_T = \sqrt{\frac{a_C}{\pi}}$$

For standard H2O and HS20 highway vehicle loading, the contact area is normally taken for dual wheels, that is, 16,000 lb over a 10 in. by 20 in. area.

Timoshenko Example Calculation

Find the vertical pressure on a 24" PE pipe buried 3 ft beneath an unpaved road when an R-50 off-road truck is over the pipe. The manufacturer lists the truck with a gross weight of 183,540 lbs on 21X35 E3 tires, each having a 30,590 lb load over an imprint area of 370 in².

SOLUTION: Use Equations 3-2 and 3-3. Since the vehicle is operating on an unpaved road, an impact factor of 2.0 is appropriate.

$$r_T = \sqrt{\frac{370 / 144}{\pi}} = 0.90ft \quad P_L = \frac{(2.0)(30,590)}{\frac{370}{144}} \left(1 - \frac{3^3}{(0.90^2 + 3^2)^{1.5}} \right)$$

$$P_L = 2890lb / ft^2$$

Boussinesq Equation

The Boussinesq Equation gives the pressure at any point in a soil mass under a concentrated surface load. The Boussinesq Equation may be used to find the pressure transmitted from a wheel load to a point that is not along the line of action of the load. Pavement effects are neglected.

$$(3-4) \quad P_L = \frac{3I_f W_w H^3}{2\pi r^5}$$

WHERE

P_L = vertical soil pressure due to live load lb/ft²

W_w = wheel load, lb

H = vertical depth to pipe crown, ft

I_f = impact factor

r = distance from the point of load application to pipe crown, ft

$$(3-5) \quad r = \sqrt{X^2 + H^2}$$

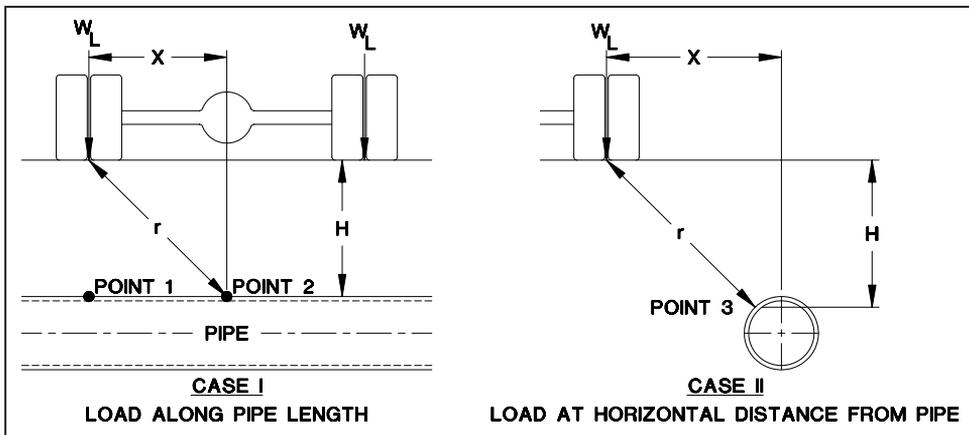


Figure 3-4 Illustration of Boussinesq Point Loading

Example Using Boussinesq Point Loading Technique

Determine the vertical soil pressure applied to a 12" pipe located 4 ft deep under a dirt road when two vehicles traveling over the pipe and in opposite lanes pass each other. Assume center lines of wheel loads are at a distance of 4 feet. Assume a wheel load of 16,000 lb.

SOLUTION: Use Equation 3-4, and since the wheels are traveling, a 2.0 impact factor is applied. The maximum load will be at the center between the two wheels, so $X = 2.0$ ft. Determine r from Equation 3-5.

$$r = \sqrt{4^2 + 2.0^2} = 4.47 \text{ ft}$$

Then solve Equation 3-4 for P_L , the load due to a single wheel.

$$P_L = \frac{3(2.0)(16,000)(4)^3}{2\pi(4.47)^5}$$

$$P_L = 548 \text{ lb} / \text{ft}^2$$

The load on the pipe crown is from both wheels, so

$$2 P_L = 2(548) = 1096 \text{ lb} / \text{ft}^2$$

The load calculated in this example is higher than that given in Table 3-4 for a comparable depth even after correcting for the impact factor. Both the Timoshenko and Boussinesq Equations give the pressure applied at a point in the soil. In solving for pipe reactions it is assumed that this point pressure is applied across the entire surface of a unit length of pipe, whereas the actual applied pressure decreases away from the line of action of the wheel load. Methods that integrate this pressure over the pipe surface such as used in deriving Table 3-4 gives more accurate loading values. However, the error in the point pressure equations is slight and conservative, so they are still effective equations for design.

Railroad Loads

The live loading configuration used for pipes under railroads is the Cooper E-80 loading, which is an 80,000 lb load that is uniformly applied over three 2 ft by 8 ft areas on 5 ft centers. The area represents the 8 ft width of standard railroad ties and the standard spacing between locomotive drive wheels. Live loads are based on the axle weight exerted on the track by two locomotives and their tenders coupled together in doubleheader fashion. See Table 3-5. Commercial railroads frequently require casings for pressure pipes if they are within 25 feet of the tracks, primarily for safety reasons in the event of a washout. Based upon design and permitting requirements, the designer should determine whether or not a casing is required.

TABLE 3-5
Live Load Pressure for E-80 Railroad Loading

Depth of cover, ft.	Soil Pressure*, lb/ft ²
2.0	3800
5.0	2400
8.0	1600
10.0	1100
12.0	800
15.0	600
20.0	300
30.0	100
Over 30.0	Neglect

For reference see ASTM A796. *The values shown for soil pressure include impact.

Surcharge Load

Surcharge loads may be distributed loads, such as a footing, foundation, or an ash pile, or may be concentrated loads, such as vehicle wheels. The load will be dispersed through the soil such that there is a reduction in pressure with an increase in depth or horizontal distance from the surcharged area. Surcharge loads not directly over the pipe may exert pressure on the pipe as well. The pressure at a point beneath a surcharge load depends on the load magnitude and the surface area over which the surcharge is applied. Methods for calculating vertical pressure on a pipe either located directly beneath a surcharge or located near a surcharge are given below.

Pipe Directly Beneath a Surcharge Load

This design method is for finding the vertical soil pressure under a rectangular area with a uniformly distributed surcharge load. This may be used in place of Tables 3-3 to 3-5 and Equations 3-3 and 3-5 to calculate vertical soil pressure due to wheel loads. This requires knowledge of the tire imprint area and impact factor.

The point pressure on the pipe at depth, H , is found by dividing the rectangular surcharge area (ABCD) into four sub-area rectangles (a, b, c, and d) which have a common corner, E, in the surcharge area, and over the pipe. The surcharge pressure, P_L , at a point directly under E is the sum of the pressure due to each of the four sub-area loads. Refer to Figure 3-5 A.

The pressure due to each sub-area is calculated by multiplying the surcharge pressure at the surface by an Influence Value, I_v . Influence Values are proportionality constants that measure what portion of a surface load reaches the subsurface point in question. They were derived using the Boussinesq Equation and are given in Table 3-6.

$$(3-6) \quad P_L = p_a + p_b + p_c + p_d$$

WHERE

P_L = vertical soil pressure due to surcharge pressure, lb/ft²

p_a = pressure due to sub-area a, lb/ft²

p_b = pressure due to sub-area b, lb/ft²

p_c = pressure due to sub-area c, lb/ft²

p_d = pressure due to sub-area d, lb/ft²

Pressure due to the surcharge applied to the i-th sub-area equals:

$$(3-7) \quad p_i = I_V w_S$$

WHERE

I_V = Influence Value from Table 3-6

w_S = distributed pressure of surcharge load at ground surface, lb/ft²

If the four sub-areas are equivalent, then Equation 3-7 may be simplified to:

$$(3-8) \quad P_L = 4I_V w_S$$

The influence value is dependent upon the dimensions of the rectangular area and upon the depth to the pipe crown, H. Table 3-6 Influence Value terms depicted in Figure 3-6, are defined as:

H = depth of cover, ft

M = horizontal distance, normal to the pipe centerline, from the center of the load to the load edge, ft

N = horizontal distance, parallel to the pipe centerline, from the center of the load to the load edge, ft

Interpolation may be used to find values not given in Table 3-6. The influence value gives the portion (or influence) of the load that reaches a given depth beneath the corner of the loaded area.

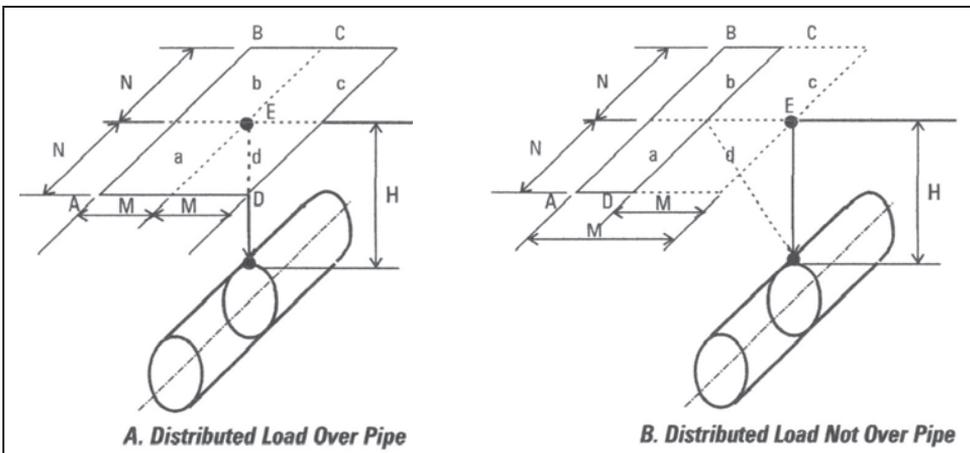


Figure 3-5 Illustration of Distributed Loads

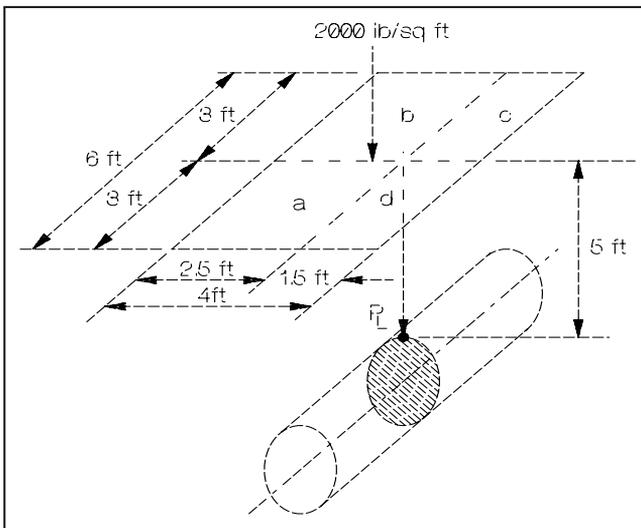
TABLE 3-6
Influence Values, I_V for Distributed Loads*

M/H	N/H													
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	∞
0.1	0.005	0.009	0.013	0.017	0.020	0.022	0.024	0.026	0.027	0.028	0.029	0.030	0.031	0.032
0.2	0.009	0.018	0.026	0.033	0.039	0.043	0.047	0.050	0.053	0.055	0.057	0.060	0.061	0.062
0.3	0.013	0.026	0.037	0.047	0.056	0.063	0.069	0.073	0.077	0.079	0.083	0.086	0.089	0.090
0.4	0.017	0.033	0.047	0.060	0.071	0.080	0.087	0.093	0.098	0.101	0.106	0.110	0.113	0.115
0.5	0.020	0.039	0.056	0.071	0.084	0.095	0.103	0.110	0.116	0.120	0.126	0.131	0.135	0.137
0.6	0.022	0.043	0.063	0.080	0.095	0.107	0.117	0.125	0.131	0.136	0.143	0.149	0.153	0.156
0.7	0.024	0.047	0.069	0.087	0.103	0.117	0.128	0.137	0.144	0.149	0.157	0.164	0.169	0.172
0.8	0.026	0.050	0.073	0.093	0.110	0.125	0.137	0.146	0.154	0.160	0.168	0.176	0.181	0.185
0.9	0.027	0.053	0.077	0.098	0.116	0.131	0.144	0.154	0.162	0.168	0.178	0.186	0.192	0.196
1.0	0.028	0.055	0.079	0.101	0.120	0.136	0.149	0.160	0.168	0.175	0.185	0.194	0.200	0.205
1.2	0.029	0.057	0.083	0.106	0.126	0.143	0.157	0.168	0.178	0.185	0.196	0.205	0.209	0.212
1.5	0.030	0.060	0.086	0.110	0.131	0.149	0.164	0.176	0.186	0.194	0.205	0.211	0.216	0.223
2.0	0.031	0.061	0.088	0.113	0.135	0.153	0.169	0.181	0.192	0.200	0.209	0.216	0.232	0.240
∞	0.032	0.062	0.089	0.116	0.137	0.156	0.172	0.185	0.196	0.205	0.212	0.223	0.240	0.250

* H, M, and N are per Figure 3-5.

Vertical Surcharge Example # 1

Find the vertical surcharge load for the 4' x 6', 2000 lb/ft² footing shown below.



SOLUTION: Use equations 3-6 and 3-7, Table 3-6, and Figure 3-5. The 4 ft x 6 ft footing is divided into four sub-areas, such that the common corner of the sub-areas is directly over the pipe. Since the pipe is not centered under the load, sub-areas a

and b have dimensions of 3 ft x 2.5 ft, and sub-areas c and d have dimensions of 3 ft x 1.5 ft.

Determine sub-area dimensions for M, N, and H, then calculate M/H and N/H. Find the Influence Value from Table 3-6, then solve for each sub area, p_a , p_b , p_c , p_d , and sum for P_L .

	Sub-area			
	a	b	c	d
M	2.5	2.5	1.5	1.5
N	3.0	3.0	3.0	3.0
M/H	0.5	0.5	0.3	0.3
N/H	0.6	0.6	0.6	0.6
I_v	0.095	0.095	0.063	0.063
p_i	190	190	126	126

Therefore: $P_L = 632 \text{ lbs/ft}^2$

Pipe Adjacent to, but Not Directly Beneath, a Surcharge Load

This design method may be used to find the surcharge load on buried pipes near, but not directly below, uniformly distributed loads such as concrete slabs, footings and floors, or other rectangular area loads, including wheel loads that are not directly over the pipe.

The vertical pressure is found by first adding an imaginary loaded area that covers the pipe, then determining the surcharge pressure due to the overall load (actual and imaginary) based on the previous section, and finally by deducting the pressure due to the imaginary load from that due to the overall load.

Refer to Figure 3-5 B. Since there is no surcharge directly above the pipe centerline, an imaginary surcharge load, having the same pressure per unit area as the actual load, is applied to sub-areas c and d. The surcharge pressure for sub-areas a+d and b+c are determined, then the surcharge loads from the imaginary areas c and d are deducted to determine the surcharge pressure on the pipe.

$$(3-9) P_L = p_{a+d} + p_{b+c} - p_d - p_c$$

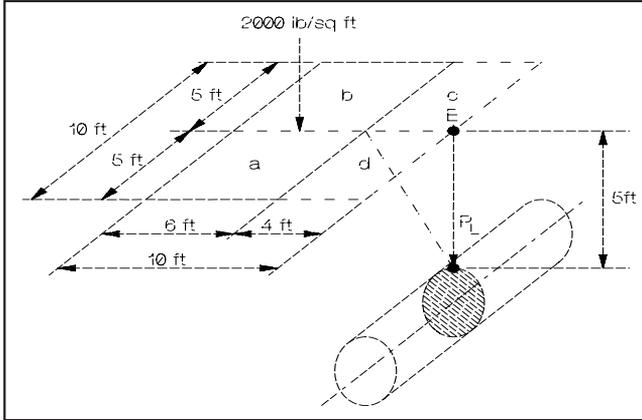
Where terms are as previously defined above, and

P_{a+d} = surcharge load of combined sub-areas a and d, lb/ft²

P_{b+c} = surcharge load of combined sub-areas b and c, lb/ft²

Vertical Surcharge Example # 2

Find the vertical surcharge pressure for the 6' x 10', 2000 lb/ft² slab shown below.



SOLUTION: Use Equations 3-7 and 3-9, Table 3-6, and Figure 3-5 B. The surcharge area is divided into two sub-areas, a and b. The area between the surcharge and the line of the pipe crown is divided into two sub-areas, c and d, as well. The imaginary load is applied to sub-areas c and d. Next, the four sub-areas are treated as a single surcharge area. Unlike the previous example, the pipe is located under the edge of the surcharge area rather than the center. So, the surcharge pressures for the combined sub-areas a+d and b+c are determined, and then for the sub-areas c and d. The surcharge pressure is the sum of the surcharge pressure due to the surcharge acting on sub-areas a+d and b+c, less the imaginary pressure due to the imaginary surcharge acting on sub-areas c and d.

	Sub-area			
	a + d	b + c	c	d
M	10	10	4	4
N	5	5	5	5
M/H	2.0	2.0	0.8	0.8
N/H	1.0	1.0	1.0	1.0
l_v	0.200	0.200	0.160	0.160
p_i	400	400	(320)	(320)

Therefore $P_L = 160 \text{ lb/ft}^2$

Installation Category 1: Standard Installation - Trench or Embankment

Pipe Reaction to Earth, Live, and Surcharge Loads

Now might be a good time to review the “Design Process” that appeared earlier in Section 3. After calculating the vertical pressure applied to the pipe the next design step is to choose a trial pipe (DR or profile). Then, based on the Installation Category and the selected embedment and compaction, calculate the anticipated deflection and resistance to crush and buckling.

The Standard Installation category applies to pipes that are installed between 18 inches and 50 feet of cover. Where surcharge, traffic, or rail load may occur, the pipe must have at least one full diameter of cover. If such cover is not available, then the application design must also consider limitations under the Shallow Cover Vehicular Loading Installation category. Where the cover depth exceeds 50 ft an alternate treatment for dead loads is given under the Deep Fill Installation category. Where ground water occurs above the pipe’s invert and the pipe has less than two diameters of cover, the potential for the occurrence of flotation or upward movement of the pipe may exist. See Shallow Cover Flotation Effects.

While the Standard Installation is suitable for up to 50 feet of cover, it may be used for more cover. The 50 feet limit is based on A. Howard’s⁽³⁾ recommended limit for use of E' values. Above 50 feet, the E' values given in Table B.1.1 in Chapter 3 Appendix are generally thought to be overly conservative as they are not corrected for the increase in embedment stiffness that occurs with depth as a result of the higher confinement pressure within the soil mass. In addition, significant arching occurs at depths greater than 50 feet.

The Standard Installation, as well as the other design categories for buried PE pipe, looks at a ring or circumferential cross-section of pipe and neglects longitudinal loading, which is normally insignificant. They also ignore the re-rounding effect of internal pressurization. Since re-rounding reduces deflection and stress in the pipe, ignoring it is conservative.

Ring Deflection

Ring deflection is the normal response of flexible pipes to soil pressure. It is also a beneficial response in that it leads to the redistribution of soil stress and the initiation of arching. Ring deflection can be controlled within acceptable limits by the selection of appropriate pipe embedment materials, compaction levels, trench width and, in some cases, the pipe itself.

The magnitude of ring deflection is inversely proportional to the combined stiffness of the pipe and the embedment soil. M. Spangler⁽⁴⁾ characterized this relationship

in the Iowa Formula in 1941. R. Watkins ⁽⁵⁾ modified this equation to allow a simpler approach for soil characterization, thus developing the Modified Iowa Formula. In 1964, Burns and Richards ⁽⁶⁾ published a closed-form solution for ring deflection and pipe stress based on classical linear elasticity. In 1976 M. Katona et. al. ⁽⁷⁾ developed a finite element program called CANDE (Culvert Analysis and Design) which is now available in a PC version and can be used to predict pipe deflection and stresses.

The more recent solutions may make better predictions than the Iowa Formula, but they require detailed information on soil and pipe properties, e.g. more soil lab testing. Often the improvement in precision is all but lost in construction variability. Therefore, the Modified Iowa Formula remains the most frequently used method of determining ring deflection.

Spangler's Modified Iowa Formula can be written for use with solid wall PE pipe as:

$$(3-10) \quad \frac{\Delta X}{D_M} = \frac{1}{144} \left(\frac{K_{BED} L_{DL} P_E + K_{BED} P_L}{\frac{2E}{3} \left(\frac{1}{DR - 1} \right)^3 + 0.061 F_S E'} \right)$$

and for use with ASTM F894 profile wall pipe as:

$$(3-11) \quad \frac{\Delta X}{D_I} = \frac{P}{144} \left(\frac{K_{BED} L_{DL}}{\frac{1.24(RSC)}{D_M} + 0.061 F_S E'} \right)$$

WHERE

ΔX = Horizontal deflection, in

K_{BED} = Bedding factor, typically 0.1

L_{DL} = Deflection lag factor

P_E = Vertical soil pressure due to earth load, psf

P_L = Vertical soil pressure due to live load, psf

E = Apparent modulus of elasticity of pipe material, lb/in²

E' = Modulus of Soil reaction, psi

F_S = Soil Support Factor

RSC = Ring Stiffness Constant, lb/ft

DR = Dimension Ratio, OD/t

D_M = Mean diameter ($D_I + 2z$ or $D_O - t$), in

z = Centroid of wall section, in

t = Minimum wall thickness, in

D_I = pipe inside diameter, in

D_O = pipe outside diameter, in

Deflection is reported as a percent of the diameter which can be found by multiplying 100 times $\Delta X/D_M$ or $\Delta X/D_I$. (When using RSC, the units of conversion are accounted for in Equation 3-11.)

Apparent Modulus of Elasticity for Pipe Material, E

The apparent modulus of PE is dependent on load-rate or, duration of loading and temperature. Apparent elastic modulus values for high and medium density PE may be found in Table B.1.1 in Chapter 3 Appendix. These values can be used in Spangler's Iowa Formula. It has long been an industry practice to use the short-term modulus in the Iowa Formula for thermoplastic pipe. This is based on the idea that, in granular embedment soil, deformation is a series of instantaneous deformations consisting of rearrangement and fracturing of grains while the bending stress in the pipe wall is decreasing due to stress relaxation. Use of the short-term modulus has proven effective and reliable for corrugated and profile wall pipes. These pipes typically have pipe stiffness values of 46 psi or less when measured per ASTM D2412. Conventional DR pipes starting with DR17 or lower have significantly higher stiffness and therefore they may carry a greater proportion of the earth and live load than corrugated or profile pipe; so it is conservative to use the 50-year modulus for DR pipes that have low DR values when determining deflection due to earth load.

Vehicle loads are generally met with a higher modulus than earth loads, as load duration may be nearly instantaneous for moving vehicles. The deflection due to a combination of vehicle or temporary loads and earth load may be found by separately calculating the deflection due to each load using the modulus appropriate for the expected load duration, then adding the resulting deflections together to get the total deflection. When doing the deflection calculation for vehicle load, the Lag Factor will be one. An alternate, but conservative, method for finding deflection for combined vehicle and earth load is to do one calculation using the 50-year modulus, but separate the vertical soil pressure into an earth load component and a live load component and apply the Lag Factor only to the earth load component.

Ring Stiffness Constant, RSC

Profile wall pipes manufactured to ASTM F894, "Standard Specification for Polyethylene (PE) Large Diameter Profile Wall Sewer and Drain Pipe," are classified on the basis of their Ring Stiffness Constant (RSC). Equation 3-12 gives the RSC.

$$(3-12) \quad RSC = \frac{6.44 EI}{D_M^2}$$

WHERE **E** = Apparent modulus of elasticity of pipe material @73°F (See Chapter 3 Appendix) **I** = Pipe wall moment of inertia, in⁴/in (t³/12, if solid wall construction) **z** = Pipe wall centroid in **DI** = Pipe inside diameter in **D_M** = Mean diameter ($DI + 2z$ or $D_0 - t$), in **t** = Minimum wall thickness, inModulus of Soil Reaction, E'

The soil reaction modulus is proportional to the embedment soil's resistance to the lateral expansion of the pipe. There are no convenient laboratory tests to determine the soil reaction modulus for a given soil. A. Howard⁽⁸⁾ determined E' values empirically from numerous field deflection measurements by substituting site parameters (i.e. depth of cover, soil weight) into Spangler's equation and "back-calculating" E' . Howard developed a table for the Bureau of Reclamation relating E' values to soil types and compaction efforts. See Table 3-7. In back-calculating E' , Howard assumed the prism load was applied to the pipe. Therefore, Table 3-7 E' values indirectly include load reduction due to arching and are suitable for use only with the prism load. In 2006, Howard published a paper reviewing his original 1977 publication from which Table 3-7 is taken. For the most part the recent work indicates that the E' values in Table 3-7 are conservative.

Due to differences in construction procedures, soil texture and density, pipe placement, and insitu soil characteristics, pipe deflection varies along the length of a pipeline. Petroff⁽⁹⁾ has shown that deflection measurements along a pipeline typically fit the Normal Distribution curve. To determine the anticipated maximum deflection using Eq. 3-10 or 3-11, variability may be accommodated by reducing the Table 3-7 E' value by 25%, or by adding to the calculated deflection percentage the correction for 'accuracy' percentage given in Table 3-7.

In shallow installations, the full value of the E' given in Table 3-7 may not develop. This is due to the lack of "soil confining pressure" to hold individual soil grains tightly together and stiffen the embedment. Increased weight or equivalently, depth, increases the confining pressure and, thus, the E' . J. Hartley and J. Duncan⁽¹⁰⁾ published recommended E' values based on depth of cover. See Table 3-8. These are particularly useful for shallow installations.

Chapter 7, "Underground Installation of PE Pipe" covers soil classification for pipe embedment materials and preferred methods of compaction and installation for selected embedment materials. Some of the materials shown in Table 3-7 may not be appropriate for all pipe installation. One example would be fine-grained soils in wet ground, which would not be appropriate embedment, under most circumstances, for either profile pipe or pipes with high DR's. Such limitations are discussed in Chapter 7.

TABLE 3-7
Values of E' for Pipe Embedment (See Howard ⁽⁸⁾)

Soil Type-pipe Embedment Material (Unified Classification System) ¹	E' for Degree of Embedment Compaction, lb/in ²			
	Dumped	Slight, <85% Proctor, <40% Relative Density	Moderate, 85%-95% Proctor, 40%-70% Relative Density	High, >95% Proctor, >70% Relative Density
Fine-grained Soils (LL > 50) ² Soils with medium to high plasticity; CH, MH, CH-MH	No data available: consult a competent soils engineer, otherwise, use E' = 0.			
Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with less than 25% coarse grained particles.	50	200	400	1000
Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with more than 25% coarse grained particles; Coarse-grained Soils with Fines, GM, GC, SM, SC ³ containing more than 12% fines.	100	400	1000	2000
Coarse-grained soils with Little or No Fines GW, GP, SW, SP ³ containing less than 12% fines	200	1000	2000	3000
Crushed Rock	1000	3000	3000	3000
Accuracy in Terms of Percentage Deflection ⁴	±2%	±2%	±1%	±0.5%

¹ ASTM D-2487, USBR Designation E-3

² LL = Liquid Limit

³ Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC).

⁴ For ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Note: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only; appropriate Deflection Lag Factor must be applied for long-term deflections. If embedment falls on the borderline between two compaction categories, select lower E' value, or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using 12,500 ft-lb/cu ft (598,000 J/m²) (ASTM D-698, AASHTO T-99, USBR Designation E-11). 1 psi = 6.9 KPa.

TABLE 3-8
Values of E' for Pipe Embedment (See Duncan and Hartley⁽¹⁰⁾)

Type of Soil	Depth of Cover, ft	E' for Standard AASHTO Relative Compaction, lb/in ²			
		85%	90%	95%	100%
Fine-grained soils with less than 25% sand content (CL, ML, CL-ML)	0-5	500	700	1000	1500
	5-10	600	1000	1400	2000
	10-15	700	1200	1600	2300
	15-20	800	1300	1800	2600
Coarse-grained soils with fines (SM, SC)	0-5	600	1000	1200	1900
	5-10	900	1400	1800	2700
	10-15	1000	1500	2100	3200
	15-20	1100	1600	2400	3700
Coarse-grained soils with little or no fines (SP, SW, GP, GW)	0-5	700	1000	1600	2500
	5-10	1000	1500	2200	3300
	10-15	1050	1600	2400	3600
	15-20	1100	1700	2500	3800

Soil Support Factor, F_s

Ring deflection and the accompanying horizontal diameter expansion create lateral earth pressure which is transmitted through the embedment soil and into the trench sidewall. This may cause the sidewall soil to compress. If the compression is significant, the embedment can move laterally, resulting in an increase in pipe deflection. Sidewall soil compression is of particular concern when the insitu soil is loose, soft, or highly compressible, such as marsh clay, peat, saturated organic soil, etc. The net effect of sidewall compressibility is a reduction in the soil-pipe system's stiffness. The reverse case may occur as well if the insitu soil is stiffer than the embedment soil; e.g. the insitu soil may enhance the embedment giving it more resistance to deflection. The Soil Support Factor, F_s , is a factor that may be applied to E' to correct for the difference in stiffness between the insitu and embedment soils. Where the insitu soil is less stiff than the embedment, F_s is a reduction factor. Where it is stiffer, F_s is an enhancement factor, i.e. greater than one.

The Soil Support Factor, F_s , may be obtained from Tables 3-9 and 3-10 as follows:

- Determine the ratio B_d/D_O , where B_d equals the trench width at the pipe springline (inches), and D_O equals the pipe outside diameter (inches).
- Based on the native insitu soil properties, find the soil reaction modulus for the insitu soil, E'_N in Table 3-9.
- Determine the ratio E'_N/E' .
- Enter Table 3-10 with the ratios B_d/D_O and E'_N/E' and find F_s .

TABLE 3-9
Values of E'_N , Native Soil Modulus of Soil Reaction, Howard ⁽³⁾

Native In Situ Soils				
Granular		Cohesive		E'_N (psi)
Std. Penetration ASTM D1586 Blows/ft	Description	Unconfined Compressive Strength (TSF)	Description	
> 0 - 1	very, very loose	> 0 - 0.125	very, very soft	50
1 - 2	very loose	0.125 - 0.25	very soft	200
2 - 4	very loose	0.25 - 0.50	soft	700
4 - 8	loose	0.50 - 1.00	medium	1,500
8 - 15	slightly compact	1.00 - 2.00	stiff	3,000
15 - 30	compact	2.00 - 4.00	very stiff	5,000
30 - 50	dense	4.00 - 6.00	hard	10,000
> 50	very dense	> 6.00	very hard	20,000
Rock	–	–	–	50,000

TABLE 3-10
Soil Support Factor, F_s

E'_N/E'	B_d/D_0 1.5	B_d/D_0 2.0	B_d/D_0 2.5	B_d/D_0 3.0	B_d/D_0 4.0	B_d/D_0 5.0
0.1	0.15	0.30	0.60	0.80	0.90	1.00
0.2	0.30	0.45	0.70	0.85	0.92	1.00
0.4	0.50	0.60	0.80	0.90	0.95	1.00
0.6	0.70	0.80	0.90	0.95	1.00	1.00
0.8	0.85	0.90	0.95	0.98	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.30	1.15	1.10	1.05	1.00	1.00
2.0	1.50	1.30	1.15	1.10	1.05	1.00
3.0	1.75	1.45	1.30	1.20	1.08	1.00
5.0	2.00	1.60	1.40	1.25	1.10	1.00

Lag Factor and Long-Term Deflection

Spangler observed an increase in ring deflection with time. Settlement of the backfill and consolidation of the embedment under the lateral pressure from the pipe continue to occur after initial installation. To account for this, he recommended applying a lag factor to the Iowa Formula in the range of from 1.25 to 1.5. Lag occurs in installations of both plastic and metal pipes. Howard ^(3, 11) has shown that the lag factor varies with the type of embedment and the degree of compaction. Many plastic pipe designers use a Lag Factor of 1.0 when using the prism load as it

accounts for backfill settlement. This makes even more sense when the Soil Support Factor is included in the calculation.

Vertical Deflection Example

Estimate the vertical deflection of a 24" diameter DR 26 pipe produced from a PE4710 material that is installed under 18 feet of cover. The embedment material is a well-graded sandy gravel, compacted to a minimum 90 percent of Standard Proctor density, and the native ground is a saturated, soft clayey soil. The anticipated trench width is 42".

SOLUTION: Use the prism load, Equation 3-1, Tables 3-7, 3-9, and 3-10, and Equation 3-10. Table 3-7 gives an E' for a compacted sandy gravel or GW-SW soil as 2000 lb/in². The Short-Term Apparent Modulus of Elasticity for PE 4710 material obtained from Table B.2.1 equals 130,000 psi. To estimate maximum deflection due to variability, this value will be reduced by 25%, or to 1500 lb/in². Table 3-9 gives an E'_N of 700 psi for soft clay. Since B_d/D equals 1.75 and E'_N/E' equals 0.47, F_s is obtained by interpolation and equal 0.60.

The prism load on the pipe is equal to:

$$P_E = (120)(18) = 2160 \text{ lb} / \text{ft}^2$$

Substituting these values into Equation 3-10 gives:

$$\frac{\Delta X}{D_M} = \frac{2160}{144} \left(\frac{(0.1)(1.0)}{\frac{2(130,000)}{3} \left(\frac{1}{26-1}\right)^3 + (0.061)(0.60)(1500)} \right)$$

$$\frac{\Delta X}{D_M} = 0.025 = 2.5 \%$$

Deflection Limits

The designer limits ring deflection in order to control geometric stability of the pipe, wall bending strain, pipeline hydraulic capacity and compatibility with cleaning equipment, and, for bell-and-spigot jointed pipe, its sealing capability. Only the limits for geometric stability and bending strain will be discussed here. Hydraulic capacity is not impaired at deflections less than 7.5%.

Geometric stability is lost when the pipe crown flattens and loses its ability to support earth load. Crown flattening occurs with excessive deflection as the increase in horizontal diameter reduces crown curvature. At 25% to 30% deflection, the

crown may completely reverse its curvature inward and collapse. See Figure 3-1A. A deflection limit of 7.5% provides at least a 3 to 1 safety factor against reverse curvature.

Bending strain occurs in the pipe wall as a result of ring deflection—outer-fiber tensile strain at the pipe springline and outer-fiber compressive strain at the crown and invert. While strain limits of 5% have been proposed, Jansen ⁽¹²⁾ reported that, on tests of PE pipe manufactured from pressure-rated resins and subjected to soil pressure only, “no upper limit from a practical design point of view seems to exist for the bending strain.” In other words, as deflection increases, the pipe’s performance limit will not be overstraining but reverse curvature collapse.

Thus, for non-pressure applications, a 7.5 percent deflection limit provides a large safety factor against instability and strain and is considered a safe design deflection. Some engineers will design profile wall pipe and other non-pressure pipe applications to a 5% deflection limit, but allow spot deflections up to 7.5% during field inspection.

The deflection limits for pressurized pipe are generally lower than for non-pressurized pipe. This is primarily due to strain considerations. Hoop strain from pressurization adds to the outer-fiber tensile strain. But the internal pressure acts to reround the pipe and, therefore, Eq. 3-10 overpredicts the actual long-term deflection for pressurized pipe. Safe allowable deflections for pressurized pipe are given in Table 3-11. Spangler and Handy ⁽¹³⁾ give equations for correcting deflection to account for rerounding.

TABLE 3-11
Safe Deflection Limits for Pressurized Pipe

DR or SDR	Safe Deflection as % of Diameter
32.5	7.5
26	7.5
21	7.5
17	6.0
13.5	6.0
11	5.0
9	4.0
7.3	3.0

* Based on Long-Term Design Deflection of Buried Pressurized Pipe given in ASTM F1962.

Compressive Ring Thrust

Earth pressure exerts a radial-directed force around the circumference of a pipe that results in a compressive ring thrust in the pipe wall. (This thrust is exactly opposite to the tensile hoop thrust induced when a pipe is pressurized.) See Figure 3-1B.

Excessive ring compressive thrust may lead to two different performance limits: crushing of the material or buckling (loss of stability) of the pipe wall. See Figure 3-1C. This section will discuss crushing, and the next section will discuss buckling.

As is often the case, the radial soil pressure causing the stress is not uniform around the pipe's circumference. However, for calculation purposes it is assumed uniform and equal to the vertical soil pressure at the pipe crown.

Pressure pipes often have internal pressure higher than the radial pressure applied by the soil. As long as there is pressure in the pipe that exceeds the external pressure, the net thrust in the pipe wall is tensile rather than compressive, and wall crush or buckling checks are not necessary. Whether one needs to check this or not can be quickly determined by simply comparing the internal pressure with the vertical soil pressure.

Crushing occurs when the compressive stress in the wall exceeds the compressive yield stress of the pipe material. Equations 3-13 and 3-14 give the compressive stress resulting from earth and live load pressure for conventional extruded DR pipe and for ASTM F894 profile wall PE Pipe:

$$(3-13) \quad S = \frac{(P_E + P_L) DR}{288}$$

$$(3-14) \quad S = \frac{(P_E + P_L) D_O}{288A}$$

WHERE

P_E = vertical soil pressure due to earth load, psf

P_L = vertical soil pressure due to live-load, psf

S = pipe wall compressive stress, lb/in²

DR = Dimension Ratio, D_0/t

D_O = pipe outside diameter (for profile pipe $D_0 = D_I + 2H_P$), in

D_I = pipe inside diameter, in

H_P = profile wall height, in

A = profile wall average cross-sectional area, in²/in

(Obtain the profile wall area from the manufacturer of the profile pipe.)

(Note: These equations contain a factor of 144 in the denominator for correct units conversions.)

Equation 3-14 may overstate the wall stress in profile pipe. Ring deflection in profile wall pipe induces arching. The “Deep Fill Installation” section of this chapter discusses arching and gives equations for calculating the earth pressure resulting from arching, P_{RD} . P_{RD} is given by Equation 3-23 and may be substituted for PE to determine the wall compressive stress when arching occurs.

The compressive stress in the pipe wall can be compared to the pipe material allowable compressive stress. If the calculated compressive stress exceeds the allowable stress, then a lower DR (heavier wall thickness) or heavier profile wall is required.

Allowable Compressive Stress

Allowable long-term compressive stress values for the several PE material designation codes can be found in Appendix, Chapter 3.

The long-term compressive stress value should be reduced for elevated temperature pipeline operation. Temperature design factors used for hydrostatic pressure may be used. See temperature re-rating or adjustment factors in the Appendix, Chapter 3.

Ring Compression Example

Find the pipe wall compressive ring stress in a DR 32.5 PE4710 pipe buried under 46 ft of cover. The ground water level is at the surface, the saturated weight of the insitu silty-clay soil is 120 lbs/ft³.

SOLUTION: Find the vertical earth pressure acting on the pipe. Use Equation 3-1.

Although the net soil pressure is equal to the buoyant weight of the soil, the water pressure is also acting on the pipe. Therefore the total pressure (water and earth load) can be found using the saturated unit weight of the soil.

Next, solve for the compressive stress.

$$P_E = (120 \text{ pcf})(46 \text{ ft}) = 5520 \text{ psf}$$

$$S = \frac{(5520 \text{ lb} / \text{ft}^2)(32.5)}{288} = 623 \text{ lb} / \text{inch}^2$$

The compressive stress is well below the allowable limit of 1150 psi for the PE4710 material given in the Appendix, Chapter 3.

Constrained (Buried) Pipe Wall Buckling

Excessive compressive stress (or thrust) may cause the pipe wall to become unstable and buckle. Buckling from ring compressive stress initiates locally as a large “dimple,” and then grows to reverse curvature followed by structural collapse. Resistance to buckling is proportional to the wall thickness divided by the diameter

raised to a power. Therefore the lower the DR, the higher the resistance. Buried pipe has an added resistance due to support (or constraint) from the surrounding soil.

Non-pressurized pipes or gravity flow pipes are most likely to have a net compressive stress in the pipe wall and, therefore, the allowable buckling pressure should be calculated and compared to the total (soil and ground water) pressure. For most pressure pipe applications, the fluid pressure in the pipe exceeds the external pressure, and the net stress in the pipe wall is tensile. Buckling needs only be considered for that time the pipe is not under pressure, such as during and immediately after construction and during system shut-downs and, in cases in which a surge pressure event can produce a temporary negative internal pressure. Under these circumstances the pipe will react much stiffer to buckling as its modulus is higher under short term loading. When designing, select a modulus appropriate for the duration of the negative external pressure. For pipe that are subjected to negative pressure due to surge, consideration should be given to selecting a DR that gives the pipe sufficient unconstrained collapse strength to resist the full applied negative pressure without support for the soil. This is to insure against construction affects that result in the embedment material not developing its full design strength.

This chapter gives two equations for calculating buckling. The modified Luscher Equation is for buried pipes that are beneath the ground water level, subject to vacuum pressure, or under live load with a shallow cover. These forces act to increase even the slightest eccentricity in the pipe wall by following deformation inward. While soil pressure alone can create instability, soil is less likely to follow deformation inward, particularly if it is granular. So, dry ground buckling is only considered for deep applications and is given by the Moore-Selig Equation found in the section, "Buckling of Pipes in Deep, Dry Fills".

Luscher Equation for Constrained Buckling Below Ground Water Level

For pipes below the ground water level, operating under a full or partial vacuum, or subject to live load, Luscher's equation may be used to determine the allowable constrained buckling pressure. Equation 3-15 and 3-16 are for DR and profile pipe respectively.

$$(3-15) \quad P_{WC} = \frac{5.65}{N} \sqrt{RB'E' \frac{E}{12(DR-1)^3}}$$

$$(3-16) \quad P_{WC} = \frac{5.65}{N} \sqrt{RB'E' \frac{EI}{D_M^3}}$$

WHERE

P_{WC} = allowable constrained buckling pressure, lb/in²

N = safety factor

$$(3-17) \quad R = 1 - 0.33 \frac{H_{GW}}{H}$$

WHERE

R = buoyancy reduction factor

H_{GW} = height of ground water above pipe, ft

H = depth of cover, ft

$$(3-18) \quad B' = \frac{I}{1 + 4e^{-0.065H}}$$

WHERE

e = natural log base number, 2.71828

E' = soil reaction modulus, psi

E = apparent modulus of elasticity, psi

DR = Dimension Ratio

I = pipe wall moment of inertia, in⁴/in (t³/12, if solid wall construction)

D_M = Mean diameter ($D_1 + 2z$ or $D_0 - t$), in

Although buckling occurs rapidly, long-term external pressure can gradually deform the pipe to the point of instability. This behavior is considered viscoelastic and can be accounted for in Equations 3-15 and 3-16 by using the apparent modulus of elasticity value for the appropriate time and temperature of the loading. For instance, a vacuum event is resisted by the short-term value of the modulus whereas continuous ground water pressure would be resisted by the 50 year value. For modulus values see Appendix, Chapter 3.

For pipes buried with less than 4 ft or a full diameter of cover, Equations 3-15 and 3-16 may have limited applicability. In this case the designer may want to use Equations 3-39 and 3-40.

The designer should apply a safety factor commensurate with the application. A safety factor of 2.0 has been used for thermoplastic pipe.

The allowable constrained buckling pressure should be compared to the total vertical stress acting on the pipe crown from the combined load of soil, and ground water or floodwater. It is prudent to check buckling resistance against a ground water level for a 100-year-flood. In this calculation the total vertical stress is typically taken as the prism load pressure for saturated soil, plus the fluid pressure of any floodwater above the ground surface.

For DR pipes operating under a vacuum, it is customary to use Equation 3-15 to check the combined pressure from soil, ground water, and vacuum, and then to use the unconstrained buckling equation, Equation 3-39, to verify that the pipe can operate with the vacuum independent of any soil support or soil load, in case construction does not develop the full soil support. Where vacuum load is short-term, such as during water hammer events two calculations with Equation 3-14 are necessary. First determine if the pipe is sufficient for the ground water and soil pressure using a long-term modulus; then determine if the pipe is sufficient for the combined ground water, soil pressure and vacuum loading using the short-term modulus.

Constrained Buckling Example

Does a 36" SDR 26 PE4710 pipe have satisfactory resistance to constrained buckling when installed with 18 ft of cover in a compacted soil embedment? Assume ground water to the surface and an E' of 1500 lb/in².

SOLUTION: Solve Equation 3-15. Since this is a long-term loading condition, the 50 year stress relaxation modulus for PE4710 material is given in the Appendix to Chapter 3 as 29,000 psi. Soil cover, H , and ground water height, H_{GW} are both 18 feet. Therefore, the soil support factor, B' , is found as follows;

$$B' = \frac{1}{1 + 4e^{-(0.065)(18)}} = 0.446$$

and the bouyancy reduction factor, R , is found as follows:

$$R = 1 - 0.33 \frac{18}{18} = 0.67$$

Solve Equation 3-15 for the allowable long-term constrained buckling pressure:

$$P_{WC} = \frac{5.65}{2} \sqrt{\frac{0.67(0.446)1500(29,000)}{12(26-1)^3}}$$

$$P_{WC} = 23.5 \text{ psi} = 3387 \text{ psf}$$

The earth pressure and ground water pressure applied to the pipe is found using Equation 3-1 (prism load) with a saturated soil weight. The saturated soil weight being the net weight of both soil and water.

$$P_E = (120)(18) = 2160 \frac{\text{lb}}{\text{ft}^2}$$

Compare this with the constrained buckling pressure. Since P_{WC} exceeds P_E , DR 26 has satisfactory resistance to constrained pipe buckling.

Installation Category #2: Shallow Cover Vehicular Loading

The Standard Installation methodology assumes that the pipe behaves primarily as a “membrane” structure, that is, the pipe is almost perfectly flexible with little ability to resist bending. At shallow cover depths, especially those less than one pipe diameter, membrane action may not fully develop, and surcharge or live loads place a bending load on the pipe crown. In this case the pipe’s flexural stiffness carries part of the load and prevents the pipe crown from dimpling inward under the load. Equation 3-19, published by Watkins⁽¹⁴⁾ gives the soil pressure that can be supported at the pipe crown by the combination of the pipe’s flexural stiffness (bending resistance) and the soil’s internal resistance against heaving upward. In addition to checking Watkins’ formula, the designer should check deflection using Equations 3-10 or 3-11, pipe wall compressive stress using Equations 3-13 or 3-14, and pipe wall buckling using Equations 3-15 or 3-16.

Watkins’ equation is recommended only where the depth of cover is greater than one-half of the pipe diameter and the pipe is installed at least 18 inches below the road surface. In other words, it is recommended that the pipe regardless of diameter always be at least 18” beneath the road surface where there are live loads present; more may be required depending on the properties of the pipe and installation. In some cases, lesser cover depths may be sufficient where there is a reinforced concrete cap or a reinforced concrete pavement slab over the pipe. Equation 3-19 may be used for both DR pipe and profile pipe. See definition of “A” below.

$$(3-19) \quad P_{WAT} = \frac{12w(KH)^2}{N_s D_o} + \frac{7387(I)}{N_s D_o^2 c} \left(S_{MAT} - \frac{w D_o H}{288A} \right)$$

WHERE

P_{WAT} = allowable live load pressure at pipe crown for pipes with one diameter or less of cover, psf

w = unit weight of soil, lb/ft³

D_o = pipe outside diameter, in

H = depth of cover, ft

I = pipe wall moment of inertia (t³/12 for DR pipe), in⁴/in

A = profile wall average cross-sectional area, in²/in, for profile pipe or wall thickness (in) for DR pipe
(obtain the profile from the manufacturer of the profile pipe.)

c = outer fiber to wall centroid, in

c = $H_p - z$ for profile pipe and c = 0.5t for DR pipe, in

H_p = profile wall height, in

z = pipe wall centroid, in

S_{MAT} = material yield strength, lb/in², Use 3000 PSI for PE3408

N_S = safety factor

K = passive earth pressure coefficient

$$(3-20) \quad K = \frac{1 + \text{SIN}(\phi)}{1 - \text{SIN}(\phi)}$$

ϕ = angle of internal friction, deg

Equation 3-19 is for a point load applied to the pipe crown. Wheel loads should be determined using a point load method such as given by Equations 3-2 (Timoshenko) or 3-4 (Boussinesq).

When a pipe is installed with shallow cover below an unpaved surface, rutting can occur which will not only reduce cover depth, but also increase the impact factor.

Shallow Cover Example

Determine the safety factor against flexural failure of the pipe accompanied by soil heave, for a 36" RSC 100 F894 profile pipe 3.0 feet beneath an H20 wheel load. Assume an asphalt surface with granular embedment.

SOLUTION: The live load pressure acting at the crown of the pipe can be found using Equation 3-4, the Boussinesq point load equation. At 3.0 feet of cover the highest live load pressure occurs directly under a single wheel and equals:

$$P_L = \frac{(3)(2.0)(16000)(3.0)^3}{2\pi(3.0)^5} = 1697 \text{ psf}$$

WHERE

$I_f = 2.0$

$W = 16,000 \text{ lbs}$

$H = 3.0 \text{ ft}$

$w = 120 \text{ pcf}$

The live load pressure is to be compared with the value in Equation 3-19. To solve Equation 3-19, the following parameters are required:

$I = 0.171 \text{ in}^4/\text{in}$

$A = 0.470 \text{ in}^2/\text{in}$

$H_P = 2.02 \text{ in (Profile Wall Height)}$

$D_O = D_I + 2 \cdot h = 36.00 + 2 \cdot 2.02 = 40.04 \text{ in}$

$Z = 0.58 \text{ in}$

$C = h - z = 1.44 \text{ in}$

$S = 3000 \text{ psi}$

$\phi = 30 \text{ deg.}$

Determine the earth pressure coefficient:

$$K = \frac{1 + \sin(30)}{1 - \sin(30)} = \frac{1 + 0.5}{1 - 0.5} = 3.0$$

The live load pressure incipient to failure equals:

$$P_{WAT} = \frac{(12)120(3.0 * 3.0)^2}{40.04} + \frac{7387 * 0.171}{40.04^2 (1.44)} \left(3000 - \frac{120(40.04)3.0}{288 * 0.470} \right)$$

$$P_{WAT} = 2904 + 1584 = 4498 \text{ psf}$$

The resulting safety factor equals:

$$N = \frac{P_{WAT}}{p_L} = \frac{4498}{1697} = 2.65$$

Installation Category #3: Deep Fill Installation

The performance limits for pipes in a deep fill are the same as for any buried pipe. They include:

1. Compressive ring thrust stress
2. Ring deflection
3. Constrained pipe wall buckling

The suggested calculation method for pipe in deep fill applications involves the introduction of design routines for each performance limit that are different than those previously given.

Compressive ring thrust is calculated using soil arching. The arching calculation may also be used for profile pipe designs in standard trench applications. Profile pipes are relatively low stiffness pipes where significant arching may occur at relatively shallow depths of cover.

At a depth of around 50 feet or so it becomes impractical to use Spangler's equation as published in this chapter because it neglects the significant load reduction due to arching and the inherent stiffening of the embedment and consequential increase in E' due to the increased lateral earth pressure applied to the embedment. This section gives an alternate deflection equation for use with PE pipes. It was first introduced by Watkins et al. ⁽¹⁾ for metal pipes, but later Gaube extended its use to include PE pipes. ⁽¹⁵⁾

Where deep fill applications are in dry soil, Luscher's equation (Eq. 3-15 or 3-16) may often be too conservative for design as it considers a radial driving force from ground water or vacuum. Moore and Selig⁽¹⁷⁾ developed a constrained pipe wall buckling equation suitable for pipes in dry soils, which is given in a following section.

Considerable care should be taken in the design of deeply buried pipes whose failure may cause slope failure in earthen structures, or refuse piles or whose failure may have severe environmental or economical impact. These cases normally justify the use of methods beyond those given in this Chapter, including finite element analysis and field testing, along with considerable professional design review.

Compressive Ring Thrust and the Vertical Arching Factor

The combined horizontal and vertical earth load acting on a buried pipe creates a radially-directed compressive load acting around the pipe's circumference. When a PE pipe is subjected to ring compression, thrust stress develops around the pipe hoop, and the pipe's circumference will ever so slightly shorten. The shortening permits "thrust arching," that is, the pipe hoop thrust stiffness is less than the soil hoop thrust stiffness and, as the pipe deforms, less load follows the pipe. This occurs much like the vertical arching described by Marston.⁽¹⁸⁾ Viscoelasticity enhances this effect. McGrath⁽¹⁹⁾ has shown thrust arching to be the predominant form of arching with PE pipes.

Burns and Richard⁽⁶⁾ have published equations that give the resulting stress occurring in a pipe due to arching. As discussed above, the arching is usually considered when calculating the ring compressive stress in profile pipes. For deeply buried pipes McGrath⁽¹⁹⁾ has simplified the Burns and Richard's equations to derive a vertical arching factor as given by Equation 3-21.

$$(3-21) \quad VAF = 0.88 - 0.71 \frac{S_A - 1}{S_A + 2.5}$$

WHERE

VAF = Vertical Arching Factor

S_A = Hoop Thrust Stiffness Ratio

$$(3-22) \quad S_A = \frac{1.43 M_s r_{CENT}}{EA}$$

WHERE

r_{CENT} = radius to centroidal axis of pipe, in

M_s = one-dimensional modulus of soil, psi

E = apparent modulus of elasticity of pipe material, psi (See Appendix, Chapter 3)

A = profile wall average cross-sectional area, in²/in, or wall thickness (in) for DR pipe

One-dimensional modulus values for soil can be obtained from soil testing, geotechnical texts, or Table 3-12 which gives typical values. The typical values in Table 3-12 were obtained by converting values from McGrath⁽²⁰⁾.

TABLE 3-12
Typical Values of M_s , One-Dimensional Modulus of Soil

Vertical Soil Stress ¹ (psi)	Gravelly Sand/Gravels 95% Std. Proctor (psi)	Gravelly Sand/Gravels 90% Std. Proctor (psi)	Gravelly Sand/Gravels 85% Std. Proctor (psi)
10	3000	1600	550
20	3500	1800	650
40	4200	2100	800
60	5000	2500	1000
80	6000	2900	1300
100	6500	3200	1450

* Adapted and extended from values given by McGrath⁽²⁰⁾. For depths not shown in McGrath⁽²⁰⁾, the M_s values were approximated using the hyperbolic soil model with appropriate values for K and n where $n=0.4$ and $K=200$, $K=100$, and $K=45$ for 95% Proctor, 90% Proctor, and 85% Proctor, respectively.

¹ Vertical Soil Stress (psi) = [soil depth (ft) x soil density (pcf)]/144

The radial directed earth pressure can be found by multiplying the prism load (pressure) by the vertical arching factor as shown in Eq. 3-23.

$$(3-23) P_{RD} = (VAF)wH$$

WHERE

P_{RD} = radial directed earth pressure, lb/ft²

w = unit weight of soil, pcf

H = depth of cover, ft

The ring compressive stress in the pipe wall can be found by substituting P_{RD} from Equation 3-23 for P_E in Equation 3-13 for DR pipe and Equation 3-14 for profile wall pipe.

Earth Pressure Example

Determine the earth pressure acting on a 36" profile wall pipe buried 30 feet deep. The following properties are for one unique 36" profile pipe made from PE3608 material. Other 36" profile pipe may have different properties. The pipe's cross-sectional area, A , equals 0.470 inches²/inch, its radius to the centroidal axis is 18.00 inches plus 0.58 inches, and its apparent modulus is 27,000 psi. Its wall height is 2.02 in and its D_o equals 36 in +2 (2.02 in) or 40.04 in. Assume the pipe is installed in a clean granular soil compacted to 90% Standard Proctor ($M_s = 1875$ psi), the insitu soil is as stiff as the embedment, and the backfill weighs 120 pcf. (Where the excavation

is in a stable trench, the stiffness of the insitu soil can generally be ignored in this calculation.) The following series of equations calculates the hoop compressive stress, S , in the pipe wall due to the earth pressure applied by the soil above the pipe. The earth pressure is reduced from the prism load by the vertical arching factor.

(From Equation 3-22)

$$S_A = \frac{1.43(1875 \frac{lbs}{inch^2})(18.58 inch)}{(28250 \frac{lbs}{inch^2})(0.470 \frac{inch^2}{inch})} = 3.93$$

(From Equation 3-21)

$$VAF = 0.88 - 0.71 \frac{3.75 - 1}{3.75 + 2.5} = 0.56$$

(From Equation 3-23)

$$P_{RD} = 0.57(120 pcf)(30 ft) = 2016 \frac{lb}{ft^2}$$

(From Equation 3-14)

$$S = \frac{P_{RD} D_O}{288A} = \frac{2052 psf (40.04 in)}{288 (0.470 in^2 / in)} = 596 psi \leq 1000 psi$$

(Allowable compressive stress per Table C.1, Appendix to Chapter 3)

Ring Deflection of Pipes Using Watkins-Gaube Graph

R. Watkins⁽¹⁾ developed an extremely straight-forward approach to calculating pipe deflection in a fill that does not rely on E' . It is based on the concept that the deflection of a pipe embedded in a layer of soil is proportional to the compression or settlement of the soil layer and that the constant of proportionality is a function of the relative stiffness between the pipe and soil. Watkins used laboratory testing to establish and graph proportionality constants, called Deformation Factors, D_F , for the stiffness ranges of metal pipes. Gaube^(15, 16) extended Watkins' work by testing to include PE pipes. In order to predict deflection, the designer first determines the amount of compression in the layer of soil in which the pipe is installed using conventional geotechnical equations. Then, deflection equals the soil compression multiplied by the D_F factor. This bypasses some of the inherent problems associated with using the soil reaction modulus, E' , values. The designer using the Watkins-Gaube Graph (Figure 3-6) should select conservative soil modulus values to accommodate variance due to installation. Two other factors to consider when using

this method is that it assumes a constant Deformation Factor independent of depth of cover and it does not address the effect of the presence of ground water on the Deformation Factor.

To use the Watkins-Gaube Graph, the designer first determines the relative stiffness between pipe and soil, which is given by the Rigidity Factor, R_F . Equation 3-24 and 3-25 are for DR pipe and for profile pipe respectively:

$$(3-24) \quad R_F = \frac{12 E_S (DR - 1)^3}{E}$$

$$(3-25) \quad R_F = \frac{E_S D_m^3}{EI}$$

WHERE

DR = Dimension Ratio

E_S = Secant modulus of the soil, psi

E = Apparent modulus of elasticity of pipe material, psi

I = Pipe wall moment of inertia of pipe, in⁴/in

D_m = Mean diameter ($D_1 + 2z$ or $D_0 - t$), in

The secant modulus of the soil may be obtained from testing or from a geotechnical engineer’s evaluation. In lieu of a precise determination, the soil modulus may be related to the one-dimensional modulus, M_s , from Table 3-12 by the following equation where μ is the soil’s Poisson ratio.

$$(3-26) \quad E_S = M_s \frac{(1 + \mu)(1 - 2\mu)}{(1 - \mu)}$$

TABLE 3-13
Typical range of Poisson’s Ratio for Soil (Bowles⁽²¹⁾)

Soil Type	Poisson’s Ratio, μ
Saturated Clay	0.4-0.5
Unsaturated Clay	0.1-0.3
Sandy Clay	0.2-0.3
Silt	0.3-0.35
Sand (Dense)	0.2-0.4
Coarse Sand (Void Ratio 0.4-0.7)	0.15
Fine-grained Sand (Void Ratio 0.4-0.7)	0.25

Next, the designer determines the Deformation Factor, D_F , by entering the Watkins-Gaube Graph with the Rigidity Factor. See Fig. 3-6. The Deformation Factor is the proportionality constant between vertical deflection (compression) of the soil layer containing the pipe and the deflection of the pipe. Thus, pipe deflection can be obtained by multiplying the proportionality constant D_F times the soil settlement. If D_F is less than 1.0 in Fig. 3-6, use 1.0.

The soil layer surrounding the pipe bears the entire load of the overburden above it without arching. Therefore, settlement (compression) of the soil layer is proportional to the prism load and not the radial directed earth pressure. Soil strain, ϵ_S , may be determined from geotechnical analysis or from the following equation:

$$(3-27) \quad \epsilon_S = \frac{wH}{144E_S}$$

WHERE

w = unit weight of soil, pcf

H = depth of cover (height of fill above pipe crown), ft

E_S = secant modulus of the soil, psi

The designer can find the pipe deflection as a percent of the diameter by multiplying the soil strain, in percent, by the deformation factor:

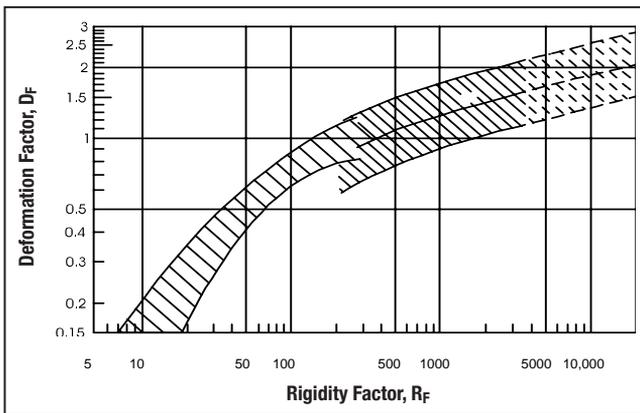


Figure 3-6 Watkins-Gaube Graph

$$(3-28) \quad \frac{\Delta X}{D_M}(100) = D_F \epsilon_S$$

WHERE

$\Delta X/D_M$ multiplied by 100 gives percent deflection.

Example of the Application of the Watkins-Gaube Calculation Technique

Find the deflection of a 6" SDR 11 pipe made from PE4710 materials under 140 ft of fill with granular embedment containing 12% or less fines, compacted at 90% of standard proctor. The fill weighs 75 pcf.

SOLUTION: First, calculate the vertical soil pressure equation, Eq. 3-1.

Eq. 3-1: $P_E = wH$

$$P_E = (75\text{lb/ft}^3)(140\text{ ft})$$

$$P_E = 10,500\text{ lb/ft}^2\text{ or }72.9\text{ psi}$$

The M_S is obtained by interpolation from Table 3-12 and equals 2700. The secant modulus can be found assuming a Poisson's Ratio of 0.30.

$$E_S = \frac{2700\text{ psi}(1 + 0.30)(1 - 2(0.30))}{(1 - 0.30)} = 2005\text{ psi}$$

The rigidity factor is obtained from Equation 3-24.

$$R_F = \frac{12(2005)(11 - 1)^3}{29,000} = 830$$

Using Figure 3-6, the average value of the deformation factor is found to be 1.2. The soil strain is calculated by Equation 3-27.

$$\epsilon_S = \frac{75\text{pcf} * 140\text{ft}}{144 * 2005 \frac{\text{lbs}}{\text{inch}^2}} * 100 = 3.6\%$$

The deflection is found by multiplying the soil strain by the deformation factor:

$$\frac{\Delta X}{D_M}(100) = 1.2 * 3.6 = 4.4\%$$

Moore-Selig Equation for Constrained Buckling in Dry Ground

As discussed previously, a compressive thrust stress exists in buried pipe. When this thrust stress approaches a critical value, the pipe can experience a local instability or large deformation and collapse. In an earlier section of this chapter, Luscher's equation was given for constrained buckling under ground water. Moore and Selig⁽¹⁷⁾ have used an alternate approach called the continuum theory to develop design equations for constrained buckling due to soil pressure (buckling of embedded pipes). The particular version of their equations given below is more appropriate for dry applications than Luscher's equation. Where ground water is present, Luscher's equation should be used.

The Moore-Selig Equation for critical buckling pressure follows: (Critical buckling pressure is the pressure at which buckling will occur. A safety factor should be provided.)

$$(3-29) \quad P_{CR} = \frac{2.4 \phi R_H}{D_M} (EI)^{\frac{1}{3}} (E_S^*)^{\frac{2}{3}}$$

WHERE

P_{CR} = Critical constrained buckling pressure, psi

ϕ = Calibration Factor, 0.55 for granular soils

R_H = Geometry Factor

E = Apparent modulus of elasticity of pipe material, psi

I = Pipe wall moment of Inertia, in⁴/in (t³/12, if solid wall construction)

E_S^* = $E_S / (1 - \mu)$

E_S = Secant modulus of the soil, psi

μ_S = Poisson's Ratio of Soil (Consult a textbook on soil for values. Bowles (1982) gives typical values for sand and rock ranging from 0.1 to 0.4.)

The geometry factor is dependent on the depth of burial and the relative stiffness between the embedment soil and the insitu soil. Moore has shown that for deep burials in uniform fills, R_H equals 1.0.

Critical Buckling Example

Determine the critical buckling pressure and safety factor against buckling for the 6" SDR 11 pipe (5.987" mean diameter) in the previous example.

SOLUTION:

$$E_S^* = \frac{2000}{(1-0.3)} = 2860 \frac{lbs}{inch^2}$$

$$P_{CR} = \frac{2.4 * 0.55 * 1.0}{5.987} (29000 * 0.018)^{\frac{1}{3}} (2860)^{\frac{2}{3}} = 358 \frac{lbs}{in^2}$$

Determine the Safety Factor against buckling:

$$S.F. = \frac{P_{CR}}{P_E} = \frac{358 * 144}{140 * 75} = 4.9$$

Installation Category #4: Shallow Cover Flotation Effects

Shallow cover presents some special considerations for flexible pipes. As already discussed, full soil structure interaction (membrane effect) may not occur, and live loads are carried in part by the bending stiffness of the pipe. Even if the pipe has sufficient strength to carry live load, the cover depth may not be sufficient to prevent

the pipe from floating upward or buckling if the ground becomes saturated with ground water. This section addresses:

- Minimum soil cover requirements to prevent flotation
- Hydrostatic buckling (unconstrained)

Design Considerations for Ground Water Flotation

High ground water can float buried pipe, causing upward movement off-grade as well as catastrophic upheaval. This is not an issue for plastic pipes alone. Flotation of metal or concrete pipes may occur at shallow cover when the pipes are empty.

Flotation occurs when the ground water surrounding the pipe produces a buoyant force greater than the sum of the downward forces provided by the soil weight, soil friction, the weight of the pipe, and the weight of its contents. In addition to the disruption occurring due to off-grade movements, flotation may also cause significant reduction of soil support around the pipe and allow the pipe to buckle from the external hydrostatic pressure.

Flotation is generally not a design consideration for buried pipe where the pipeline runs full or nearly full of liquid or where ground water is always below the pipe invert. Where these conditions are not met, a quick “rule of thumb” is that pipe buried in soil having a saturated unit weight of at least 120 lb/ft³ with at least 1½ pipe diameters of cover will not float. However, if burial is in lighter weight soils or with lesser cover, ground water flotation should be checked.

Mathematically the relationship between the buoyant force and the downward forces is given in Equation 3-30. Refer to Figure 3-7. For an empty pipe, flotation will occur if:

$$(3-30) \quad F_B > W_P + W_S + W_D + W_L$$

WHERE

F_B = buoyant force, lb/ft of pipe

W_P = pipe weight, lb/ft of pipe

W_S = weight of saturated soil above pipe, lb/ft of pipe

W_D = weight of dry soil above pipe, lb/ft of pipe

W_L = weight of liquid contents, lb/ft of pipe

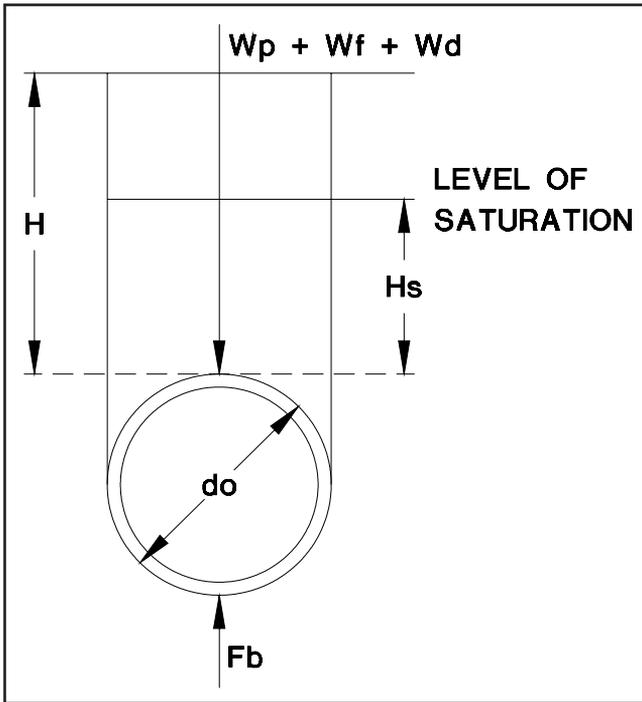


Figure 3-7 Schematic of Ground Water Flotation Forces

For a 1 ft length of pipe running empty and submerged, the upward buoyant force is:

$$(3-31) \quad F_B = \omega_G \frac{\pi}{4} d_o^2$$

WHERE

d_o = pipe outside diameter, ft

ω_G = specific weight of ground water

(fresh water = 62.4 lb/ft³)

(sea water = 64.0 lb/ft³)

The average pipe weight, W_P in lbs/ft may be obtained from manufacturers' literature or from Equation 3-32 or from the Table of Weights in the Appendix to this Chapter. This calculation is based on the use of a pipe material density of 0.955 gm/cc.

$$(3-32) \quad W_P = \pi d_o^2 \frac{(1.06 DR - 1.12)}{DR^2} 59.6$$

Equation 3-33 gives the weight of soil per lineal foot of pipe.

$$(3-33) \quad W_D = \omega_d(H - H_S)d_o$$

WHERE

ω_d = unit weight of dry soil, pcf (See Table 3-14 for typical values.)

H = depth of cover, ft

H_S = level of ground water saturation above pipe, ft

TABLE 3-14
Saturated and Dry Soil Unit Weight

Soil Type	Unit Weight, lb/ft ³	
	Saturated, unit weight of ground water, pcf ω_s	Dry, the weight of saturated soil above the pipe, lbs per foot of pipe ω_d
Sands & Gravel	118-150	93-144
Silts & Clays	87-131	37-112
Glacial Till	131-150	106-144
Crushed Rock	119-137	94-125
Organic Silts & Clay	81-112	31-94

$$(3-34) \quad W_S = (\omega_s - \omega_G) \left(\frac{d_o^2(4 - \pi)}{8} + d_o H_S \right)$$

WHERE

ω_s = saturated unit weight of soil, pcf

When an area is submerged, the soil particles are buoyed by their immersion in the ground water. The effective weight of submerged soil, ($W_s - W_G$), is the soil's saturated unit weight less the density of the ground water. For example, a soil of 120 pcf saturated unit weight has an effective weight of 57.6 pcf when completely immersed in water ($120 - 62.4 = 57.6$ pcf).

Equation 3-35 gives the weight per lineal foot of the liquid in a full pipe.

$$(3-35) \quad W_L = \omega_L \frac{\pi d^2}{4}$$

WHERE

W_L = weight of the liquid in the pipe, lb/ft

ω_L = unit weight of liquid in the pipe, pcf

and if half-full, the liquid weight is

$$(3-36) \quad W_L = \omega_L \frac{\pi d'^2}{8}$$

WHERE

ω_L = unit weight of the liquid in the pipe, lb/ft³

d' = pipe inside diameter, ft

For liquid levels between empty and half-full (0% to 50%), or between half-full and full (50% to 100%), the following formulas provide an approximate liquid weight with an accuracy of about ±10%. Please refer to Figure 3-8.

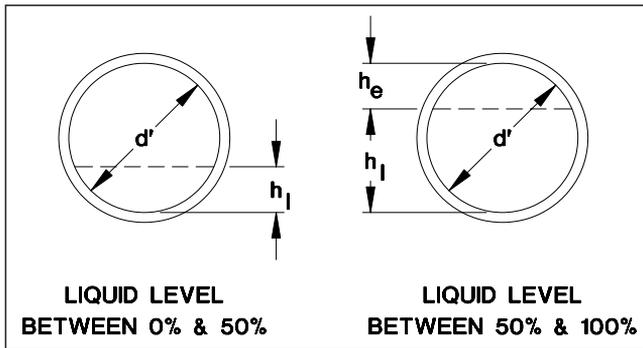


Figure 3-8 Flotation and Internal Liquid Levels

For a liquid level between empty and half-full, the weight of the liquid in the pipe is approximately

$$(3-37) \quad W_L = \omega_L \frac{4 h_l^3}{3} \sqrt{\frac{d' - h_l}{h_l} + 0.392}$$

WHERE

h_l = liquid level in pipe, ft

For a liquid level between half-full and full, the weight of the liquid in the pipe is approximately

$$(3-38) \quad W_L = \omega_L \left(\frac{\pi d'^2}{4} - 1.573 h_e \right)$$

WHERE

$$h_e = d' - h_i$$

Unconstrained Pipe Wall Buckling (Hydrostatic Buckling)

The equation for buckling given in this section is here to provide assistance when designing shallow cover applications. However, it may be used to calculate the buckling resistance of above grade pipes subject to external air pressure due to an internal vacuum, for submerged pipes in lakes or ponds, and for pipes placed in casings without grout encasement.

Unconstrained pipe are pipes that are not constrained by soil embedment or concrete encasement. Above ground pipes are unconstrained, as are pipes placed in a casing prior to grouting. Buried pipe may be considered essentially unconstrained where the surrounding soil does not significantly increase its buckling resistance beyond its unconstrained strength. This can happen where the depth of cover is insufficient to prevent the pipe from floating slightly upward and breaking contact with the embedment below its springline. Ground water, flooding, or vacuum can cause buckling of unconstrained pipe.

A special case of unconstrained buckling referred to as “upward” buckling may happen for shallow buried pipe. Upward buckling occurs when lateral pressure due to ground water or vacuum pushes the sides of the pipe inward while forcing the pipe crown and the soil above it upward. (Collapse looks like pipe deflection rotated 90 degrees.) A pipe is susceptible to upward buckling where the cover depth is insufficient to restrain upward crown movement. It has been suggested that a minimum cover of four feet is required before soil support contributes to averting upward buckling; however, larger diameter pipe may require as much as a diameter and a half to develop full support.

A conservative design for shallow cover buckling is to assume no soil support, and to design the pipe using the unconstrained pipe wall buckling equation. In lieu of this, a concrete cap, sufficient to resist upward deflection, may also be placed over the pipe and then the pipe may be designed using Luscher’s equation for constrained buckling.

Equations 3-39 and 3-40 give the allowable unconstrained pipe wall buckling pressure for DR pipe and profile pipe, respectively.

$$(3-39) \quad P_{WU} = \frac{f_o}{N_s} \frac{2E}{(1 - \mu^2)} \left(\frac{1}{DR - 1} \right)^3$$

$$(3-40) \quad P_{WU} = \frac{f_o}{N_S} \frac{24EI}{(1 - \mu^2) D_M^3}$$

WHERE

P_{WU} = allowable unconstrained pipe wall buckling pressure, psi

DR = Dimension Ratio

E = apparent modulus of elasticity of pipe material, psi

f_o = Ovality Correction Factor, Figure 3-9

N_S = safety factor

I = Pipe wall moment of inertia, in⁴/in

μ = Poisson's ratio

D_M = Mean diameter, $(D_I + 2z \text{ or } D_O - t)$, in

D_I = pipe inside diameter, in

z = wall-section centroidal distance from inner fiber of pipe, in (obtain from pipe producer)

Although buckling occurs rapidly, long-term external pressure can gradually deform the pipe to the point of instability. This behavior is considered viscoelastic and can be accounted for in Equations 3-39 and 3-40 by using the apparent modulus of elasticity value for the appropriate time and temperature of the specific application as given in the Appendix, Chapter 3. For Poisson's ratio, use a value of 0.45 for all PE pipe materials.

Ovality or deflection of the pipe diameter increases the local radius of curvature of the pipe wall and thus reduces buckling resistance. Ovality is typically reported as the percentage reduction in pipe diameter or:

$$(3-41) \quad \%DEFLECTION = 100 \left(\frac{D_I - D_{MIN}}{D_I} \right)$$

WHERE

D_I = pipe inside diameter, in

D_{MIN} = pipe minimum inside diameter, in

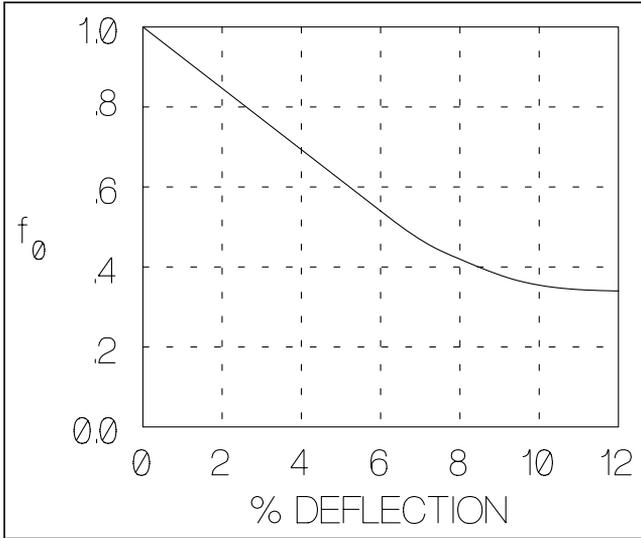


Figure 3-9 Ovality Compensation Factor, f_0

The designer should compare the critical buckling pressure with the actual anticipated pressure, and apply a safety factor commensurate with their assessment of the application. A safety factor of 2.5 is common, but specific circumstances may warrant a higher or lower safety factor. For large-diameter submerged pipe, the anticipated pressure may be conservatively calculated by determining the height of water from the pipe invert rather than from the pipe crown.

Ground Water Flotation Example

Find the allowable flood water level above a 10" DR 26 PE4710 pipe installed with only 2 ft of cover. Assume the pipe has 3 percent ovality due to shipping, handling, and installation loads.

SOLUTION: Use Equation 3-39. The pipe wall buckling pressure depends upon the duration of the water level above the pipe. If the water level is long lasting, then a long-term value of the stress relaxation modulus should be used, but if the water level rises only occasionally, a shorter term elastic modulus may be applied.

Case (a): For the long lasting water above the pipe, the stress relaxation modulus at 50 year, 73°F is approximately 29,000 lb/in² for a typical PE4710 material. Assuming 3% ovality (f_0 equals 0.76) and a 2.5 to 1 safety factor, the allowable long-term pressure, P_{WU} is given by:

$$P_{WU} = \frac{(0.76) 2(29,000)}{2.5 (1 - 0.45^2)} \left(\frac{1}{26 - 1} \right)^3 = 1.4 \text{ psig (3.2 ft-hd)}$$

Case (b): Flooding conditions are occasional happenings, usually lasting a few days to a week or so. However, ground water elevations may remain high for several weeks following a flood. The 1000 hour elastic modulus value has been used to approximate the expected flood duration.

$$P_{wu} = \frac{(0.76) 2(46,000)}{2.5 (1 - 0.45^2)} \left(\frac{1}{26 - 1} \right)^3 = 2.2 \text{ psi} = 5.2 \text{ ft. (of head)}$$

Section 4 Thermal Design Considerations

Introduction

Similar to all thermoplastics, the engineering behavior of PE can be significantly affected by temperature. An increase in temperature causes a decrease in strength and in apparent modulus. A decrease in temperature results in opposite effects. For effective pipeline design these effects must be adequately recognized.

In the case of pressure pipe the highest operating temperature is limited by the practical consideration of retaining sufficient long-term strength or maintaining the pressure rating that is sufficient for the intended application. That maximum temperature is generally 140°F (60°C). De-rating factors for up to 140°F are presented in the Appendix to Chapter 3. If higher temperatures are being considered, the pipe supplier should be consulted for additional information.

In the case of buried applications of non-pressure pipe, in which the embedment material provides a significant support against pipe deformation, the highest operating temperature can be higher –sometimes, as high 180°F (~82°C). The temperature re-rating factors for apparent modulus of elasticity, which are presented in the Appendix, Chapter 3, can be used for the re-rating of a pipe's 73°F pipe stiffness for any other temperature between -20 to 140°F (-29 to 60°C). For temperatures above 140°F the effect is more material dependent and the pipe supplier should be consulted.

A beneficial feature of PE pipe is that it retains much of its toughness even at low temperatures. It can be safely handled, installed and operated even in sub-freezing conditions. The formation of ice in the pipe will restrict or, stop flow but not cause pipe breakage. Although under sub-freezing conditions PE pipe is somewhat less tough it is still much tougher than most other pipe materials.

Strength and Stress/Strain Behavior

As discussed earlier in this Handbook, the engineering properties of PE material are affected by the magnitude of a load, the duration of loading, the environment and the operating temperature. And, also as discussed earlier, the standard convention is to report the engineering properties of PE piping materials based on a standard environment – which is water – and, a standard temperature – which is 73°F (23°C). A design for a condition that departs from this convention requires that an appropriate accommodation be made. This Section addresses the issue of the effect of a different temperature than that of the base temperature.

To properly consider the affect of temperature on strength and, on stress/strain properties this must be done based on actually observed long-term strength behavior. Tables which are presented in an Appendix to Chapter 3, list temperature adjustment factors that have been determined based on long-term evaluations.

Thermal Expansion/Contraction Effects

Fused PE pipe joints are fully restrained. The pipe and the fused joints can easily accommodate the stress induced by changes in temperature. In general thrust restraints and mechanical expansion joints are not required in a fully fused PE piping system. However, thrust restraint may be necessary where PE pipe is connection to other 'bell and spigot' end pipe. Design for this condition is addressed later in this chapter and in PPI's TN-36.

Because the coefficient of thermal expansion for PE is significantly larger than that of non-plastics, considerations relating to the potential effects of thermal expansion/contraction may include:

- Piping that is installed when it is warm may cool sufficiently after installation to generate significant tensile forces. Thus, the final connection should be made after the pipe has equilibrated to its operating temperature.
- Unrestrained pipe may shrink enough so that it pulls out from a mechanical joint that does not provide sufficient pull-out resistance. Methods used to connect PE pipe should provide restraint against pull-out that is either inherent to the joint design or additional mechanical restraint. See Chapter 9. (Note –specially designed thrust blocks may be needed to restrain movement when mechanical joints are in line with PE pipes.)
- Unrestrained pipe that is exposed to significant temperature swings will in some combination, expand and contract, deflect laterally, or apply compressive or tensile loads to constraints or supports.

A mitigating factor is PE's relatively low modulus of elasticity, which greatly reduces the thrust that is generated by a restrained expansion/contraction. This thrust imposes no problem on thermal fusion connections.

See Chapter 8 for additional information on designing above grade pipelines for thermal effects.

Unrestrained Thermal Effects

The theoretical change in length for an unrestrained pipe placed on a frictionless surface can be determined from Equation 4-1.

$$(4-1) \Delta L = L \alpha \Delta T$$

WHERE

ΔL = pipeline length change, ft

L = pipe length, ft

α = thermal expansion coefficient, in/in/°F

ΔT = temperature change, °F

The coefficient of thermal expansion for PE pipe material is approximately 1×10^{-4} in/in/°F. As a “rule of thumb,” temperature change for *unrestrained* PE pipe is about “1/10/100,” that is, 1 inch for each 10°F temperature change for each 100 foot of pipe. A temperature rise results in a length increase while a temperature drop results in a length decrease.

End Restrained Thermal Effects

A length of pipe that is restrained or anchored on both ends and one placed on a frictionless surface will exhibit a substantially different reaction to temperature change than the unrestrained pipe discussed above. If the pipe is restrained in a straight line between two points and the temperature decreases, the pipe will attempt to decrease in length. Because the ends are restrained or anchored, length change cannot occur, so a longitudinal tensile stress is created along the pipe. The magnitude of this stress can be determined using Equation 4-2.

$$(4-2) \sigma = E \alpha \Delta T$$

Where terms are as defined above, and

σ = longitudinal stress in pipe, psi

E = apparent modulus elasticity of pipe material, psi

The value of the apparent modulus of elasticity of the pipe material has a large impact on the calculated stress. As with all thermoplastic materials, PE’s modulus, and therefore its stiffness, is dependent on temperature and the duration of the applied load. Therefore, the appropriate elastic modulus should be selected based on these two variables. When determining the appropriate time interval, it is important to consider that heat transfer occurs at relatively slow rates through the wall of PE pipe; therefore temperature changes do not occur rapidly. Because the temperature change does not happen rapidly, the average temperature is often chosen for the modulus selection.

$$(4-3) F = \sigma A_p$$

Where terms are as defined above, and

F = end thrust, lb

A_p = area of pipe cross section, $(\pi/4)(D_0^2 - D_i^2)$ in²

Equations 4-2 and 4-3 can also be used to determine the compressive stress and thrust (respectively) from a temperature increase.

Although the length change of PE pipe during temperature changes is greater than many other materials, the amount of force required to restrain the movement is less because of its lower modulus of elasticity.

As pipeline temperature decreases from weather or operating conditions, a longitudinal tensile stress develops along the pipe that can be determined using Equation 4-2. The allowable tensile stress for pipe operating at its pressure rating is determined by the HDS for that temperature. The HDS is that of the pipe material for the base temperature at 73°F (23°C) times the temperature adjustment factor listed in Appendix, Chapter 3.

$$(4-4) \sigma_{allow} = HDS \times F_T$$

WHERE

HDS = Hydrostatic Design Stress, psi (Table 1-1)

F_T = Temperature factor (See Appendix, Chapter 3)

Equation 4-3 is used to determine the thrust load applied to structural anchoring devices.

During temperature increase, the pipeline attempts to increase in length, but is restrained by mechanical guides that direct longitudinal compressive thrust to structural anchors that prevent length increase. This in turn creates a longitudinal compressive stress in the pipe and a thrust load against the structural anchors. The compressive stress that develops in the pipe and is resisted by the structural anchors is determined using Equation 4-2. Compressive stress should not exceed the allowable compressive stress per the Appendix in Chapter 3.

Above Ground Piping Systems

The design considerations for PE piping systems installed above ground are extensive and, therefore, are addressed separately in the Handbook chapter on above ground applications for PE pipe.

Buried Piping Systems

A buried pipe is generally well restrained by soil loads and will experience very little lateral movement. However, longitudinal end loads may result that need to be addressed.

Transitions to other pipe materials that use the bell and spigot assembly technique will need to be calculated using the thrust load as delivered by the pressure

plus the potential of the load due to temperature changes. Merely fixing the end of the PE to the mating material may result in up stream joints pulling apart unless those connections are restrained. The number of joints that need to be restrained to prevent bell and spigot pull out may be calculated using techniques as recommended by the manufacturer of the alternate piping material. Equation 4-3 may be used to calculate the total thrust load due to temperature change.

Low thrust capacity connections to manholes or other piping systems as will be present in many no pressure gravity flow systems may be addressed via a longitudinal thrust anchor such as shown in Fig. 4-1. The size of the thrust block will vary depending on soil conditions and the thrust load as calculated via Equation 4-3.

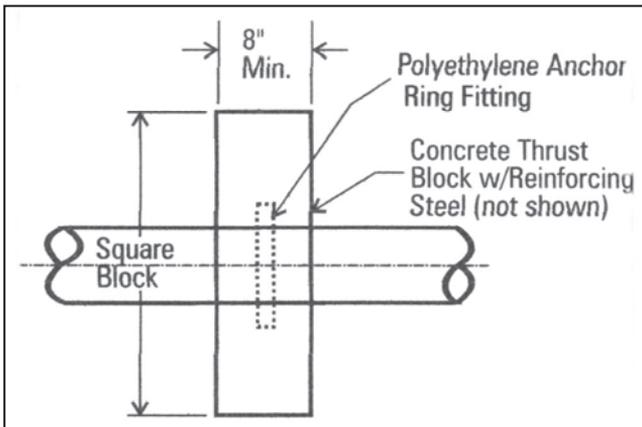


Figure 4-1 Longitudinal Thrust Anchor

Conclusion

The durability and visco-elastic nature of modern PE piping materials makes these products ideally suited for a broad array of piping applications such as: potable water mains and service lines, natural gas distribution, oil and gas gathering, force main sewers, gravity flow lines, industrial and various mining piping. To this end, fundamental design considerations such as fluid flow, burial design and thermal response were presented within this chapter in an effort to provide guidance to the piping system designer on the use of these tough piping materials in the full array of potential piping applications.

For the benefit of the pipeline designer, a considerable amount of background information and/or theory has been provided within this chapter. However, the designer should also keep in mind that the majority of pipeline installations fall within the criteria for the AWWA Design Window approach presented in Section 3

of this chapter. Pipeline installations that fall within the guidelines for the AWWA Window, may be greatly simplified in matters relating to the design and use of flexible PE piping systems.

While every effort has been made to be as thorough as possible in this discussion, it also should be recognized that these guidelines should be considered in light of specific project, installation and/or service needs. For this reason, this chapter on pipeline design should be utilized in conjunction with the other chapters of this Handbook to provide a more thorough understanding of the design considerations that may be specific to a particular project or application using PE piping systems. The reader is also referred to the extensive list of references for this chapter as additional resources for project and or system analysis and design.

References for Section 1

- Kerr, Logan, "Water Hammer Problems in Engineering Design", Consulting Engineering, May 1985
- Howard; A., "The Reclamation E' Table, 25 Years Later", Plastic Pipe XIII, Washington, D. C., Oct. 2-5, 2006.
- Marshall, G. P., Brogden, S., "Evaluation of the Surge and Fatigue Resistances of PVC and PE Pipe Materials for use in the U.K. Water Industry", Plastics Pipe X, September '98, Gotenberg, Sweden.
- UK Water Industry IG N 4-37-02, "Design Against Surge And Fatigue Conditions For Thermoplastics Pipes", March 1999.

References for Section 2

1. Jeppson, Roland W., Analysis of Flow in Pipe Networks, Ann Arbor Science, Ann Arbor, MI.
2. Distribution Network Analysis, *AWWA Manual M32*, American Water Works Association, Denver, CO.
3. ASTM D 2513, Standard Specification for Thermoplastic Gas Pipe, Tubing and Fittings, American Society for Testing and Materials, West Conshohocken, PA.
4. ASTM D 2737, Standard Specification for PE (PE) Tubing, American Society for Testing and Materials, West Conshohocken, PA.
5. ASTM D 2447, Standard Specification for PE (PE) Plastic Pipe, Schedules 40 and 80, Based on Outside Diameter, American Society for Testing and Materials, West Conshohocken, PA.
6. ASTM D 3035, Standard Specification for PE (PE) Plastic Pipe (DR-PR) Based on Controlled Outside Diameter, American Society for Testing and Materials, West Conshohocken, PA.
7. ASTM F 714, Standard Specification for PE (PE) Plastic Pipe (SDR-PR) Based on Controlled Outside Diameter, American Society for Testing and Materials, West Conshohocken, PA.
8. ANSI/AWWA C901, AWWA Standard for PE (PE) Pressure Pipe and Tubing, 1/2 In.(13 mm) Through 3 In. (76 mm) for Water Service, American Water Works Association, Denver, CO
9. ANSI/AWWA C906, AWWA Standard for PE (PE) Pressure Pipe and Fittings, 4 In. Through 63 In. for Water Distribution, American Water Works Association, Denver, CO.
10. API Specification 15LE, Specification for PE Line Pipe (PE), American Petroleum Institute, Washington DC.
11. ASTM D 2104, Standard Specification for PE (PE) Plastic Pipe, Schedule 40, American Society for Testing and Materials, West Conshohocken, PA.
12. ASTM D 2239, Standard Specification for PE (PE) Plastic Pipe (SIDR-PR) Based on Controlled Inside Diameter, American Society for Testing and Materials, West Conshohocken, PA.
13. ASTM F 894, Standard Specification for PE (PE) Large Diameter Profile Wall Sewer and Drain Pipe, American Society for Testing and Materials, West Conshohocken, PA.
14. PPI TR-22, PE Piping Distribution Systems for Components of Liquid Petroleum Gases, Plastics Pipe Institute, Irving, TX.
15. Nayyar, Mohinder L. (1992). *Piping Handbook*, 6th Edition, McGraw-Hill, New York, NY.
16. Iolelchick, I.E., Malyavskaya O.G., & Fried, E. (1986). *Handbook of Hydraulic Resistance*, Hemisphere Publishing Corporation.
17. Moody, L.F. (1944). *Transactions*, Volume 6, American Society of Mechanical Engineers (ASME), New York, NY.
18. Swierzawski, Tadeusz J. (2000). Flow of Fluids, Chapter B8, *Piping Handbook*, 7th edition, Mohinder L. Nayyar, McGraw-Hill, New York, NY.
19. Lamont, Peter A. (1981, May). Common Pipe Flow Formulas Compared with the Theory of Roughness, *Journal of the American Water Works Association*, Denver, CO.
20. Flow of Fluids through Valves, Fittings and Pipe. (1957). Crane Technical Paper No 410, the Crane Company, Chicago, IL.
21. Chen, W.F., & J.Y. Richard Liew. (2002). *The Civil Engineering Handbook*, 2nd edition, CRC Press, Boca Raton, FL.

22. Bowman, J.A. (1990). The Fatigue Response of Polyvinyl Chloride and PE Pipe Systems, Buried Plastics Pipe Technology, ASTM STP 1093, American Society for Testing and Materials, Philadelphia.
23. Marshall, GP, S. Brogden, & M.A. Shepherd, Evaluation of the Surge and Fatigue Resistance of PVC and PE Pipeline Materials for use in the UK Water Industry, Proceedings of Plastics Pipes X, Goteborg, Sweden.
24. Fedossof, F.A., & Szpak, E. (1978, Sept 20-22). Cyclic Pressure Effects on High Density PE Pipe, Paper presented at the Western Canada Sewage Conference, Regian, Saskatoon, Canada.
25. Parmakian, John. (1963). *Waterhammer Analysis*, Dover Publications, New York, NY.
26. Thompson, T.L., & Aude, T.C. (1980). Slurry Pipelines, *Journal of Pipelines*, Elsevier Scientific Publishing Company, Amsterdam.
27. *Handbook of Natural Gas Engineering*. (1959). McGraw-Hill, New York, NY.
28. *AGA Plastic Pipe Manual for Gas Service*. (2001). American Gas Association, Washington DC.
29. ASCE Manuals and Reports on Engineering Practice No. 60. (1982). Gravity Sewer Design and Construction, American Society of Civil Engineers, New York, NY.
30. Hicks, Tyler G. (1999). *Handbook of Civil Engineering Calculations*, McGraw-Hill, New York, NY.
31. PPI TR-14, Water Flow Characteristics of Thermoplastic Pipe, Plastics Pipe Institute, Irving, TX.
32. Kerr, Logan, Water Hammer Problems in Engineering Design, Consulting Engineering, May 1985.

References for Section 3

1. Watkins, R.K., Szpak, E., & Allman, W.B. (1974). Structural Design of PE Pipes Subjected to External Loads, Engr. Experiment Station, Utah State Univ., Logan.
2. AWWA (2006), PE Pipe Design and Installation, M55, American Water Works Association, Denver, CO.
3. Howard, A.K. (1996). *Pipeline Installation*, Relativity Printing, Lakewood, Colorado, ISBN 0-9651002-0-0.
4. Spangler, M.G. (1941). The Structural Design of Flexible Pipe Culverts, Bulletin 153, Iowa Engineering Experiment Station, Ames, IA.
5. Watkins, R.K., & Spangler, M.G. (1958). Some Characteristics of the Modulus of Passive Resistance of Soil—A Study in Similitude, Highway Research Board, Proceedings 37:576-583, Washington.
6. Burns, J.Q., & Richard, R.M. (1964). Attenuation of Stresses for Buried Cylinders, Proceedings of the Symposium on Soil Structure Interaction, pp.378-392, University of Arizona, Tucson.
7. Katona, J.G., Forrest, F.J., Odello, & Allgood, J.R. (1976). CANDE—A Modern Approach for the Structural Design and Analysis of Buried Culverts, Report FHWA-RD-77-5, FHWA, US Department of Transportation.
8. Howard, A.K. (1977, January). Modulus of Soil Reaction Values for Buried Flexible Pipe, *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 103, No GT 1.
9. Petroff, L.J. (1995). Installation Technique and Field Performance of PE, Profile Pipe, Proceedings 2nd Intl. Conference on the Advances in Underground Pipeline Engineering, ASCE, Seattle.
10. Duncan, J.M., & Hartley, J.D. (1982). Evaluation of the Modulus of Soil Reaction, E', and Its Variation with Depth, Report No. UCB/GT/82-02, University of California, Berkeley.
11. Howard, A.K. (1981). The USBR Equation for Predicting Flexible Pipe Deflection, Proceedings Intl. Conf. On Underground Plastic Pipe, ASCE, New Orleans, LA.
12. Janson, L.E. (1991). Long-Term Studies of PVC and PE Pipes Subjected to Forced Constant Deflection, Report No. 3, KP-Council, Stockholm, Sweden.
13. Spangler, M.G., & Handy, R.L. (1982). *Soil Engineering*, 4th ed., Harper & Row, New York.
14. Watkins, R.K. (1977). Minimum Soil Cover Required Over Buried Flexible Cylinders, Interim Report, Utah State University, Logan, UT.
15. Gaube, E. (1977, June). Stress Analysis Calculations on Rigid PE and PVC Sewage Pipes, *Kunststoffe*, Vol.67, pp. 353-356, Germany.
16. Gaube, E., & Muller, W. (1982, July). Measurement of the long-term deformation of PE pipes laid underground, *Kunststoffe*, Vol. 72, pp. 420-423, Germany.
17. Moore, I. D., & Selig, E. T. (1990). Use of Continuum Buckling Theory for Evaluation of Buried Plastic Pipe Stability, Buried Plastic Pipe Technology, ASTM STP 1093, Philadelphia.
18. Marston, A. (1930). Iowa Engineering Experiment Station, Bulletin No. 96.
19. McGrath, T. (1994). *Analysis of Burns & Richard Solution for Thrust in Buried Pipe*, Simpson Gumpertz & Heger, Inc, Cambridge, Mass.
20. McGrath, T.J. (1998). Replacing E' with the Constrained Modulus in Flexible Pipe Design, proceedings Pipeline Div. Conf. Pipelines in the Constructed Environment, ASCE, San Diego, CA.
21. Bowles, J.E. (1982). *Foundation Analysis and Design*, 3rd ed., McGraw-Hill Book Company, New York.

Appendix A.1

PIPE WEIGHTS AND DIMENSIONS (DIPS) (Black)

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR*	in.	in.	lb. per foot
		7	2.76	0.566	2.621
		9	3.03	0.440	2.119
		11	3.20	0.360	1.776
		13.5	3.34	0.293	1.476
3	3.960	15.5	3.42	0.255	1.299
		17	3.47	0.233	1.192
		21	3.56	0.189	0.978
		26	3.64	0.152	0.798
		32.5	3.70	0.122	0.644
		7	3.35	0.686	3.851
		9	3.67	0.533	3.114
		11	3.87	0.436	2.609
		13.5	4.05	0.356	2.168
4	4.800	15.5	4.14	0.310	1.909
		17	4.20	0.282	1.752
		21	4.32	0.229	1.436
		26	4.41	0.185	1.172
		32.5	4.49	0.148	0.946
		7	4.81	0.986	7.957
		9	5.27	0.767	6.434
		11	5.57	0.627	5.392
		13.5	5.82	0.511	4.480
6	6.900	15.5	5.96	0.445	3.945
		17	6.04	0.406	3.620
		21	6.20	0.329	2.968
		26	6.34	0.265	2.422
		32.5	6.45	0.212	1.954

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR*	in.	in.	lb. per foot
		7	6.31	1.293	13.689
		9	6.92	1.006	11.069
		11	7.31	0.823	9.276
		13.5	7.63	0.670	7.708
8	9.050	15.5	7.81	0.584	6.787
		17	7.92	0.532	6.228
		21	8.14	0.431	5.106
		26	8.31	0.348	4.166
		32.5	8.46	0.278	3.361
		7	7.74	1.586	20.593
		9	8.49	1.233	16.652
		11	8.96	1.009	13.955
		13.5	9.36	0.822	11.595
10	11.100	15.5	9.58	0.716	10.210
		17	9.72	0.653	9.369
		21	9.98	0.529	7.681
		26	10.19	0.427	6.267
		32.5	10.38	0.342	5.056
		7	9.20	1.886	29.121
		9	10.09	1.467	23.548
		11	10.66	1.200	19.734
		13.5	11.13	0.978	16.397
12	13.200	15.5	11.39	0.852	14.439
		17	11.55	0.776	13.250
		21	11.87	0.629	10.862
		26	12.12	0.508	8.863
		32.5	12.34	0.406	7.151
		7	10.67	2.186	39.124
		9	11.70	1.700	31.637
		11	12.35	1.391	26.513
		13.5	12.90	1.133	22.030
14	15.300	15.5	13.21	0.987	19.398
		17	13.39	0.900	17.801
		21	13.76	0.729	14.593
		26	14.05	0.588	11.907
		32.5	14.30	0.471	9.607

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR*	in.	in.	lb. per foot
		7	12.13	2.486	50.601
		9	13.30	1.933	40.917
		11	14.05	1.582	34.290
		13.5	14.67	1.289	28.492
16	17.400	15.5	15.02	1.123	25.089
		17	15.23	1.024	23.023
		21	15.64	0.829	18.874
		26	15.98	0.669	15.400
		32.5	16.26	0.535	12.425
		7	13.59	2.786	63.553
		9	14.91	2.167	51.390
		11	15.74	1.773	43.067
		13.5	16.44	1.444	35.785
18	19.500	15.5	16.83	1.258	31.510
		17	17.07	1.147	28.916
		21	17.53	0.929	23.704
		26	17.91	0.750	19.342
		32.5	18.23	0.600	15.605
		7	15.06	3.086	77.978
		9	16.51	2.400	63.055
		11	17.44	1.964	52.842
		13.5	18.21	1.600	43.907
20	21.600	15.5	18.65	1.394	38.662
		17	18.91	1.271	35.479
		21	19.42	1.029	29.085
		26	19.84	0.831	23.732
		32.5	20.19	0.665	19.147
		11	20.83	2.345	75.390
		13.5	21.75	1.911	62.642
		15.5	22.27	1.665	55.159
24	25.800	17	22.58	1.518	50.618
		21	23.20	1.229	41.495
		26	23.70	0.992	33.858
		32.5	24.12	0.794	27.317

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR*	in.	in.	lb. per foot
		13.5	26.97	2.370	96.367
		15.5	27.62	2.065	84.855
30	32.000	17	28.01	1.882	77.869
		21	28.77	1.524	63.835
		26	29.39	1.231	52.086
		32.5	29.91	0.985	42.023

*These DRs (7.3, 9, 11, 13.5, 17, 21, 26, 32.5) are from the standard dimension ratio (SDR) series established by ASTM F 412.51

Appendix A.2

PIPE WEIGHTS AND DIMENSIONS (IPS) (BLACK)

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR	in.	in.	lb. per foot
		7	0.59	0.120	0.118
		7.3	0.60	0.115	0.114
1/2	0.840	9	0.64	0.093	0.095
		9.3	0.65	0.090	0.093
		11	0.68	0.076	0.080
		11.5	0.69	0.073	0.077
		7	0.73	0.150	0.184
		7.3	0.75	0.144	0.178
3/4	1.050	9	0.80	0.117	0.149
		9.3	0.81	0.113	0.145
		11	0.85	0.095	0.125
		11.5	0.86	0.091	0.120
		7	0.92	0.188	0.289
		7.3	0.93	0.180	0.279
1	1.315	9	1.01	0.146	0.234
		9.3	1.02	0.141	0.227
		11	1.06	0.120	0.196
		11.5	1.07	0.114	0.188
		7	1.16	0.237	0.461
		7.3	1.18	0.227	0.445
		9	1.27	0.184	0.372
1 1/4	1.660	9.3	1.28	0.178	0.362
		11	1.34	0.151	0.312
		11.5	1.35	0.144	0.300
		13.5	1.40	0.123	0.259

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR	in.	in.	lb. per foot
		7	1.32	0.271	0.603
		7.3	1.35	0.260	0.583
		9	1.45	0.211	0.488
1 1/2	1.900	9.3	1.47	0.204	0.474
		11	1.53	0.173	0.409
		11.5	1.55	0.165	0.393
		13.5	1.60	0.141	0.340
		15.5	1.64	0.123	0.299
		7	1.66	0.339	0.943
		7.3	1.69	0.325	0.911
		9	1.82	0.264	0.762
		9.3	1.83	0.255	0.741
2	2.375	11	1.92	0.216	0.639
		11.5	1.94	0.207	0.614
		13.5	2.00	0.176	0.531
		15.5	2.05	0.153	0.467
		17	2.08	0.140	0.429
		7	2.44	0.500	2.047
		7.3	2.48	0.479	1.978
		9	2.68	0.389	1.656
		9.3	2.70	0.376	1.609
		11	2.83	0.318	1.387
3	3.500	11.5	2.85	0.304	1.333
		13.5	2.95	0.259	1.153
		15.5	3.02	0.226	1.015
		17	3.06	0.206	0.932
		21	3.15	0.167	0.764
		26	3.21	0.135	0.623

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR	in.	in.	lb. per foot
		7	3.14	0.643	3.384
		7.3	3.19	0.616	3.269
		9	3.44	0.500	2.737
		9.3	3.47	0.484	2.660
		11	3.63	0.409	2.294
4	4.500	11.5	3.67	0.391	2.204
		13.5	3.79	0.333	1.906
		15.5	3.88	0.290	1.678
		17	3.94	0.265	1.540
		21	4.05	0.214	1.262
		26	4.13	0.173	1.030
		32.5	4.21	0.138	0.831
		7	3.88	0.795	5.172
		7.3	3.95	0.762	4.996
		9	4.25	0.618	4.182
		9.3	4.29	0.598	4.065
		11	4.49	0.506	3.505
5	5.563	11.5	4.54	0.484	3.368
		13.5	4.69	0.412	2.912
		15.5	4.80	0.359	2.564
		17	4.87	0.327	2.353
		21	5.00	0.265	1.929
		26	5.11	0.214	1.574
		32.5	5.20	0.171	1.270
		7	4.62	0.946	7.336
		7.3	4.70	0.908	7.086
		9	5.06	0.736	5.932
		9.3	5.11	0.712	5.765
		11	5.35	0.602	4.971
6	6.625	11.5	5.40	0.576	4.777
		13.5	5.58	0.491	4.130
		15.5	5.72	0.427	3.637
		17	5.80	0.390	3.338
		21	5.96	0.315	2.736
		26	6.08	0.255	2.233
		32.5	6.19	0.204	1.801

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR	in.	in.	lb. per foot
		7	6.01	1.232	12.433
		7.3	6.12	1.182	12.010
		9	6.59	0.958	10.054
		9.3	6.66	0.927	9.771
		11	6.96	0.784	8.425
8	8.625	11.5	7.04	0.750	8.096
		13.5	7.27	0.639	7.001
		15.5	7.45	0.556	6.164
		17	7.55	0.507	5.657
		21	7.75	0.411	4.637
		26	7.92	0.332	3.784
		7	7.49	1.536	19.314
		7.3	7.63	1.473	18.656
		9	8.22	1.194	15.618
		9.3	8.30	1.156	15.179
		11	8.68	0.977	13.089
10	10.750	11.5	8.77	0.935	12.578
		13.5	9.06	0.796	10.875
		15.5	9.28	0.694	9.576
		17	9.41	0.632	8.788
		21	9.66	0.512	7.204
		26	9.87	0.413	5.878
		32.5	10.05	0.331	4.742
		7	8.89	1.821	27.170
		7.3	9.05	1.747	26.244
		9	9.75	1.417	21.970
		9.3	9.84	1.371	21.353
		11	10.29	1.159	18.412
12	12.750	11.5	10.40	1.109	17.693
		13.5	10.75	0.944	15.298
		15.5	11.01	0.823	13.471
		17	11.16	0.750	12.362
		21	11.46	0.607	10.134
		26	11.71	0.490	8.269
		32.5	11.92	0.392	6.671

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR	in.	in.	lb. per foot
		7	9.76	2.000	32.758
		7.3	9.93	1.918	31.642
		9	10.70	1.556	26.489
		9.3	10.81	1.505	25.745
		11	11.30	1.273	22.199
14	14.000	11.5	11.42	1.217	21.332
		13.5	11.80	1.037	18.445
		15.5	12.09	0.903	16.242
		17	12.25	0.824	14.905
		21	12.59	0.667	12.218
		26	12.86	0.538	9.970
		32.5	13.09	0.431	8.044
		7	11.15	2.286	42.786
		7.3	11.35	2.192	41.329
		9	12.23	1.778	34.598
		9.3	12.35	1.720	33.626
		11	12.92	1.455	28.994
16	16.000	11.5	13.05	1.391	27.862
		13.5	13.49	1.185	24.092
		15.5	13.81	1.032	21.214
		17	14.00	0.941	19.467
		21	14.38	0.762	15.959
		26	14.70	0.615	13.022
		7	12.55	2.571	54.151
		7.3	12.77	2.466	52.307
		9	13.76	2.000	43.788
		9.3	13.90	1.935	42.558
		11	14.53	1.636	36.696
18	18.000	11.5	14.68	1.565	35.263
		13.5	15.17	1.333	30.491
		15.5	15.54	1.161	26.849
		17	15.76	1.059	24.638
		21	16.18	0.857	20.198
		26	16.53	0.692	16.480
		32.5	16.83	0.554	13.296

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR	in.	in.	lb. per foot
		7	13.94	2.857	66.853
		7.3	14.19	2.740	64.576
		9	15.29	2.222	54.059
		9.3	15.44	2.151	52.541
		11	16.15	1.818	45.304
20	20.000	11.5	16.31	1.739	43.535
		13.5	16.86	1.481	37.643
		15.5	17.26	1.290	33.146
		17	17.51	1.176	30.418
		21	17.98	0.952	24.936
		26	18.37	0.769	20.346
		32.5	18.70	0.615	16.415
		9	16.82	2.444	65.412
		9.3	16.98	2.366	63.574
		11	17.76	2.000	54.818
		11.5	17.94	1.913	52.677
22	22.000	13.5	18.55	1.630	45.548
		15.5	18.99	1.419	40.107
		17	19.26	1.294	36.805
		21	19.78	1.048	30.172
		26	20.21	0.846	24.619
		32.5	20.56	0.677	19.863
		9	18.35	2.667	77.845
		9.3	18.53	2.581	75.658
		11	19.37	2.182	65.237
		11.5	19.58	2.087	62.690
24	24.000	13.5	20.23	1.778	54.206
		15.5	20.72	1.548	47.731
		17	21.01	1.412	43.801
		21	21.58	1.143	35.907
		26	22.04	0.923	29.299
		32.5	22.43	0.738	23.638

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR	in.	in.	lb. per foot
		11	22.60	2.545	88.795
		11.5	22.84	2.435	85.329
		13.5	23.60	2.074	73.781
		15.5	24.17	1.806	64.967
28	28.000	17	24.51	1.647	59.618
		21	25.17	1.333	48.874
		26	25.72	1.077	39.879
		32.5	26.17	0.862	32.174
		11	24.22	2.727	101.934
		11.5	24.47	2.609	97.954
		13.5	25.29	2.222	84.697
		15.5	25.90	1.935	74.580
30	30.000	17	26.26	1.765	68.439
		21	26.97	1.429	56.105
		26	27.55	1.154	45.779
		32.5	28.04	0.923	36.934
		13.5	26.97	2.370	96.367
		15.5	27.62	2.065	84.855
32	32.000	17	28.01	1.882	77.869
		21	28.77	1.524	63.835
		26	29.39	1.231	52.086
		32.5	29.91	0.985	42.023
		15.5	31.08	2.323	107.395
		17	31.51	2.118	98.553
36	36.000	21	32.37	1.714	80.791
		26	33.06	1.385	65.922
		32.5	33.65	1.108	53.186
		15.5	36.26	2.710	146.176
		17	36.76	2.471	134.141
42	42.000	21	37.76	2.000	109.966
		26	38.58	1.615	89.727
		32.5	39.26	1.292	72.392

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	DR	in.	in.	lb. per foot
		17	42.01	2.824	175.205
48	48.000	21	43.15	2.286	143.629
		26	44.09	1.846	117.194
		32.5	44.87	1.477	94.552
		21	48.55	2.571	181.781
54	54.000	26	49.60	2.077	148.324
		32.5	50.48	1.662	119.668

Appendix A.3

List of Design Chapter Variables

ν	=	kinematic viscosity, ft ² /sec
ρ	=	fluid density, lb/ft ³
μ	=	dynamic viscosity, lb-sec/ft ²
Δv	=	Sudden velocity change, ft/sec
a	=	Wave velocity (celerity), ft/sec
A_C	=	Cross-sectional area of pipe bore, ft ²
a_c	=	contact area, ft ²
A	=	profile wall average cross-sectional area, in ² /in, for profile pipe or wall thickness (in) for DR pipe
A_S	=	Area of pipe cross-section or $(\pi/4)(D_o^2 - D_i^2)$, in ²
A_P	=	area of the outside wall of the pipe, 100 in ²
C	=	Hazen-Williams Friction Factor, dimensionless ,see table 1-7.
c	=	outer fiber to wall centroid, in
C_V	=	percent solids concentration by volume
C_W	=	percent solids concentration by weight
D_A	=	pipe average inside diameter, in
DF	=	Design Factor, from Table 1-2
d'	=	Pipe inside diameter, ft
D_I	=	Pipe inside diameter, in
D_M	=	Mean diameter $(D_I + 2z \text{ or } D_o - t)$, in
D_{MIN}	=	pipe minimum inside diameter, in
D_o	=	pipe outside diameter, in
d_o	=	pipe outside diameter, ft
DR	=	Dimension Ratio, D_o/t
E	=	Apparent modulus of elasticity for pipe material, psi
e	=	natural log base number, 2.71828
E'	=	Modulus of soil reaction, psi
E_d	=	Dynamic instantaneous effective modulus of pipe material (typically 150,000 psi for PE pipe)
E_N	=	Native soil modulus of soil reaction, psi
E_S	=	Secant modulus of the soil, psi
E_S^*	=	$E_S/(1-\mu)$
f	=	friction factor (dimensionless, but dependent upon pipe surface roughness and Reynolds number)
F	=	end thrust, lb
F_B	=	buoyant force, lb/ft
F_L	=	velocity coefficient (Tables 1-14 and 1-15)
f_o	=	Ovality Correction Factor, Figure 2-9
F_S	=	Soil Support Factor
F_T	=	Service Temperature Design Factor, from Table 1-11
g	=	Constant gravitational acceleration, 32.2 ft/sec ²
H_P	=	profile wall height, in
H	=	height of cover, ft
h_l	=	liquid level in the pipe, ft
H_{GW}	=	ground water height above pipe, ft
h_1	=	pipeline elevation at point 1, ft

h_1	=	inlet pressure, in H ₂ O
h_U	=	upstream pipe elevation, ft
h_2	=	pipeline elevation at point 2, ft
h_2	=	outlet pressure, in H ₂ O
d_D	=	downstream pipe elevation, ft
HDB	=	Hydrostatic Design Basis, psi
h_E	=	Elevation head, ft of liquid
h_f	=	friction (head) loss, ft. of liquid
H_S	=	level of ground water saturation above pipe, ft
I_V	=	Influence Value from Table 2-5
I	=	Pipe wall moment of inertia, in ⁴ /in
IDR	=	ID -Controlled Pipe Dimension Ratio
I_f		impact factor
k	=	kinematic viscosity, centistokes
K_{BULK}	=	Bulk modulus of fluid at working temperature
K_{BED}	=	Bedding factor, typically 0.1
K	=	passive earth pressure coefficient
K'	=	Fittings Factor, Table 1-5
K_P	=	permeability constant (Table 1-13)
L_{EFF}	=	Effective Pipeline length, ft.
L	=	Pipeline length, ft
L_{DL}	=	Deflection lag factor
ΔL	=	pipeline length change, in
M	=	horizontal distance, normal to the pipe centerline, from the center of the load to the load edge, ft
M_s	=	one-dimensional modulus of soil, psi
n	=	roughness coefficient, dimensionless
N	=	horizontal distance, parallel to the pipe centerline, from the center of the load to the load edge, ft
N_S	=	safety factor
P	=	Internal Pressure, psi
P_W	=	perimeter wetted by flow, ft
p_1	=	inlet pressure, lb/in ² absolute
p_2	=	outlet pressure, lb/in ² absolute
P_A	=	pipe internal pressure, atmospheres (1 atmosphere = 14.7 lb/in ²)
PC	=	Pressure Class
P_{CR}	=	Critical constrained buckling pressure, psi
P_E	=	vertical soil pressure due to earth load, psf
P_f	=	friction (head) loss, psi
P_L	=	vertical soil pressure due to live load, psf
P_{OS}	=	Occasional Surge Pressure
P_{RD}	=	radial directed earth pressure, lb/ft ²
P_{RS}	=	Recurring Surge Pressure
P_s	=	Transient surge pressure, psig
P_{WAT}	=	Allowable live load pressure at pipe crown for pipes with one diameter or less of cover, psf
P_{WC}	=	allowable constrained buckling pressure, lb/in ²
P_{WU}	=	allowable unconstrained pipe wall buckling pressure, psi

p_i	=	Pressure due to sub-area i lb/ft ²
Q	=	flow rate, gpm
Q_{FPS}	=	flow, ft ³ /sec
Q_h	=	flow, standard ft ³ /hour
q_P	=	volume of gas permeated, cm ³ (gas at standard temperature and pressure)
r_H	=	hydraulic radius, ft
r	=	distance from the point of load application to pipe crown, ft
R	=	buoyancy reduction factor
r_{CENT}	=	radius to centroidal axis of pipe, in
Re	=	Reynolds number, dimensionless
R_H	=	Geometry Factor
RSC	=	Ring Stiffness Constant, lb/ft
r_T	=	equivalent radius, ft
RF	=	Rigidity factor, dimensions
s	=	liquid density, gm/cm ³
S_H	=	hydraulic slope, ft/ft
S	=	pipe wall compressive stress, lb/in ²
S_{MAT}	=	material yield strength, lb/in ²
S_A	=	Hoop Thrust Stiffness Ratio
S_g	=	gas specific gravity
S_L	=	carrier liquid specific gravity
S_M	=	slurry mixture specific gravity
S_S	=	solids specific gravity
t	=	minimum wall thickness, in
t'	=	wall thickness, mils
T_{CR}	=	Critical time, seconds
V	=	flow velocity, ft/sec
VAF	=	Vertical Arching Factor
V_C	=	critical settlement velocity, ft/sec
ν	=	kinematic viscosity, ft ² /sec
V_{Min}	=	approximate minimum velocity, ft/sec
w	=	unit weight of soil, pcf
w	=	unit weight of soil, lb/ft ³
W_D	=	weight of dry soil above pipe, lb/ft of pipe
W_w	=	wheel load, lb
W_L	=	weight of liquid contents, lb/ft of pipe
W_L	=	weight of the liquid in contacts, lb/ft of pipe
WP	=	Working Pressure, psi
W_P	=	pipe weight, lb/ft of pipe
WPR	=	Working Pressure Rating, psi
w_S	=	distributed surcharge pressure acting over ground surface, lb/ft ²
W_S	=	weight of saturated soil above pipe, lb/ft of pipe
ζ	=	dynamic viscosity, centipoises
Z	=	Centroid of wall section, in
Z	=	Pipe wall centroid, in
Z_i	=	wall-section centroidal distance from inner fiber of pipe, in
α	=	thermal expansion coefficient, in/in/°F

ΔL	=	length change, in
ΔT	=	temperature change, °F
ΔX	=	Horizontal deflection, in
ΔV	=	Sudden velocity change., ft/sec
ϵ	=	absolute roughness, ft.
ϵ_s	=	Soil strain
Θ	=	elapsed time, days
μ_s	=	Poisson's Ratio of Soil
μ	=	Poisson's ratio
σ	=	longitudinal stress in pipe, psi
σ_{allow}	=	Allowable tensile stress at 73°F, lb/in
ϕ	=	Calibration Factor, 0.55 for granular soils change in psi
ω_D	=	unit weight of dry soil, lb/ft ³ (See Table 2-16 for typical values.)
ω_G	=	unit weight of groundwater lb/ft ³
ω_L	=	unit weight of liquid in the pipe, lb/ft ³
ω_S	=	unit weight of saturated soil, pcf lb/ft ³
ϕ	=	angle of internal friction, deg
Γ	=	Dynamic viscosity, lb-sec/ft ²