J. M. Doyle, J.M. and Fang, S.J. “Underground Pipe”
Structural Engineering Handbook
Ed. Chen Wai-Fah
Boca Raton: CRC Press LLC, 1999

Underground Pipe

J. M. Doyle and
S.J. Fang
Sargent & Lundy, LLC
Chicago, IL

25.1 Introduction

Throughout recorded history, works have been constructed for conveying water from one place to another. The Roman aqueducts are often mentioned as examples of great technical achievement; indeed, some of these early structures are still in use today. Although most of the early water carrying structures were open channels, conduits and pipes of various materials were also used in Roman times. It appears, though, that the effectiveness of the early pipes was limited because their materials were weak in tensile capacity. Therefore, the pipes could not carry fluid under any appreciable pressure. Beginning in the 17th century, wood and cast iron were used in water pipe applications in order to carry water under pressure from pumping, which was introduced about the same time. Since then, many materials have evolved for use in pipes. As a general rule, the goals for new pipe material development has been increased tensile strength, reduced weight, and, of course, reduced cost.

Pipe that is buried underground must sustain other loads besides the internal fluid pressure. That is, it must support the soil overburden, groundwater, loads applied at the ground surface, such as vehicular traffic, and forces induced by seismic motion. Buried pipe is, therefore, a structure as well as a conduit for conveying fluid. That being the case, special design procedures are required to insure that both functions are fulfilled. It is the purpose of this chapter to present techniques that are currently in use for the design of underground pipelines. Such lines are used for public water systems, sewers, drainage facilities, and many industrial processes. Pipe materials to be considered include steel, concrete, and fiberglass reinforced plastic. This selection provides examples of both flexible and rigid behavior. The methodologies presented here can be applied to other materials as well. Design procedures given are, for the most part, based on material contained in U.S. national standards or recommended practices developed by industry organizations. It is our intention to provide an exposition of the essential elements of the various design procedures. No claim is made to total inclusiveness for the methodologies discussed. Readers interested in the full range of refinements and subtleties of any of the approaches are encouraged to consult the cited works. For convenience when comparing references, the notations used in work by others will be maintained here. Attention is focused on large-diameter lines, generally greater than 24 in. Worked sample problems are included to illustrate the material presented.

25.2 External Loads

25.2.1 Overburden

The vertical load that the pipe supports consists of a block of soil extending from the ground surface to the top of the pipe plus (or minus) shear forces along the edges of the block. The shear forces are developed when the soil prism above the pipe or the soil surrounding the prism settle relative to each other. For example, the soil prism above the pipe in an excavated trench would tend to settle relative to the surrounding soil. The shear forces between the backfill and the undisturbed soil would resist the settlement, thus reducing the prism load to be carried by the pipe. For a pipe placed on the ground and covered by a new fill, the effect may be the same or opposite, in which case the load to be supported by the pipe would be greater than the soil prism. The difference in behavior depends on the difference in settlement between the pipe itself and the fill material. Sketches of typical methods of buried pipe installation are shown in Figure 25.1.

Methods developed by Marston and Spangler, and their co-workers, at Iowa State University [28, 29, 34, 35, 36, 39] over a period of about 50 years, are the accepted tools for evaluating overburden loads on buried conduits and are widely used in design practice. The general form of the expression, developed by this group, used to calculate the overburden load carried by the pipe is given as

$$W_c = CwB^2 \tag{25.1}$$

where:

W_c = total load on pipe, per unit of length
C = load coefficient, dependent on type of installation, trench or fill, on the soil type, and on relative rates of settlement of the pipe and surrounding soil
w = unit weight of soil supported by pipe
B = width of trench of outer diameter of pipe

Values for the load coefficient, C, for varying conditions of installation, are given in several standard references (see, e.g., [20]).

The American Water Works Association (AWWA) [21], in its design manual for steel pipe, recommends that the total overburden load on buried steel pipes be assumed equal to a soil prism with width equal to the outer diameter of the pipe and height equal to the cover depth. That is,

$$W_c = wB_ch \tag{25.2}$$

where

B_c = external pipe diameter
h = depth from ground surface to top of pipe

25.2.2 Surcharge at Grade

Besides the direct loads imposed by the soil overburden, underground pipes must also sustain loads applied on the ground surface. Typically, such loads occur as a result of vehicular traffic passing over the route of the pipe. However, they can be caused by static objects placed directly, or nearly so, above the pipe as well.

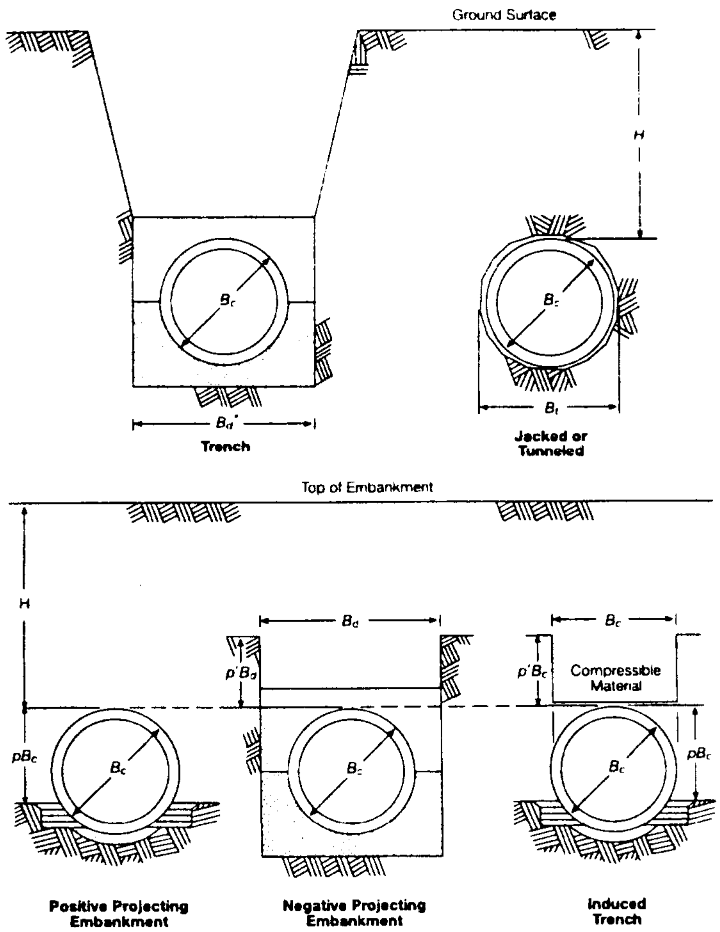

FIGURE 25.1: Typical underground pipe installations. (Reprinted from *Concrete Pressure Pipe*, M9, by permission. Copyright©1995, American Water Works Association.)

Experimental results, by the Iowa State University researchers and others [33, 37], have confirmed that the load intensity at the pipe depth, due to surface loads, can be predicted on the basis of the theory of elasticity. The effects of an arbitrary spatial distribution of surface load can be obtained by utilizing the well-known Boussinesq solution [41], for a point load on an elastic half space, as an influence function.

Since the Boussinesq solution provides a stress distribution for which magnitudes decay with distance from the load, it follows that the intensity of surface loads decreases with increased depth. Therefore, the consequence of traffic, or other surface loads, on deeply buried pipes is relatively minor. Conversely, surface loads applied over pipes with shallow cover can be quite serious. For this reason, a minimum cover is usually required in any place where vehicular traffic will operate over underground conduits.

Prior to development of present day computational tools, the evaluation of the Boussinesq equations to determine the total load on a buried pipe due to an arbitrary surface load was beyond the capability of most practitioners. For that reason, tables were developed, based on simple surface load distributions, and have been included in most design literature for buried pipe for many years. See, for example, the tables of values in the AWWA Manual M11 [21]. Loading configurations not covered by the previously developed tables can be investigated using available software programs. Mathcad [30], for example, can be utilized to carry out the analysis necessary to evaluate the effect of arbitrary surface loads on buried structures, including pipes.

25.2.3 Live Loads

The main source of design live loads on buried pipes is wheeled traffic from highway trucks, railroad locomotives, and aircraft. Loads transmitted to buried structures by the standard HS-20 truck loading [1] and the Cooper E-80 railroad loading have been evaluated using the Boussinesq solution and engineering judgment, for varying depths of cover, and are available, in different forms, in several publications (see, e.g., [6, 20]). Due to the wide variation in aircraft wheel loadings, it is usually necessary to evaluate each case separately. FAA Advisory Circular 150/5320-5B provides information on aircraft wheel loads. The load intensity at the depth of the pipe has been reported in numerous references. Simple load intensities for the HS-20 truck loads and for the Cooper E-80 locomotive loads, at varying depths, are given in Tables 25.1 and 25.2, respectively [6]. More comprehensive tables for truck and railroad loads have been published [20, 27]. In general, the intensities given in Tables 25.1 and 25.2 are close to the intensities given in the other tables, though some differences do exist. For examples in this chapter, live loads will be based on the intensities given in Tables 25.1 and 25.2. In case of doubt as to appropriate live load values to use in design of buried pipe, the advice of a geotechnical engineer should be obtained.

TABLE 25.1 HS-20 Live Load

Height of cover, ft	Live load, lb/ft^2
1	1800
2	800
3	600
4	400
5	250
6	200
7	175
8	100
Over 8	Neglect

From American Society for Testing and Materials. 1994. A796. *Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe-Arches and Arches for Storm and Sanitary Sewers and Other Buried Applications.* With permission.

25.2.4 Seismic Loads

In zones of high seismicity, buried conduits must be designed for the stresses imposed by earthquake ground motions. The American Society of Civil Engineers (ASCE) has developed procedures for evaluation of the magnitude of axial and flexural strains induced in underground lines by seismic motions [24]. The document reflects the research efforts of many of the leading seismic engineers in the country and the methodology is widely used for design of underground conduits of all kinds.

As a general rule, the stresses in pipe walls due to seismic motion–induced strains are quite small and do not adversely affect the design. Since most design codes allow for an increase in allowable stress, or a decrease in load factors, when seismic loads are included in a load combination, buried pipes that are sized to sustain other design loads usually have sufficient strength to resist seismic-imposed stresses.

TABLE 25.2 Cooper E-80 Live Load

Height of cover, ft	Live load, lb/ft^2
2	3800
5	2400
8	1600
10	1100
12	800
15	600
20	300
30	100
Over 30	Neglect

From American Society for Testing and Materials. 1994. A796. *Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe-Arches and Arches for Storm and Sanitary Sewers and Other Buried Applications.* With permission.

Consequently, the major consideration to be addressed in design of underground pipe is not strength but excessive relative movement. Unrestrained slip joints in buried pipe may be subject to relative movement, between the two segments meeting at a joint, that exceeds the limit of the joint's capacity to function. For that reason, slip joint pipe must be investigated for maximum relative movement when subject to seismic motion. Types of pipe commonly utilizing slip joints include ductile iron, reinforced and prestressed concrete, and fiberglass reinforced plastic.

25.3 Internal Loads

25.3.1 Internal Pressure and Vacuum

Underground pipe systems operate under varying levels of internal pressure. Gravity sewer lines normally operate under fairly low internal pressure whereas water supply mains and industrial process pipes may be subject to internal pressures of several hundred pounds per square inch. High-pressure pipelines are often designed for a continuous operating pressure and for a short-term transient pressure.

Certain operational events may cause a temporary vacuum in buried conduits. In most cases the duration of application of vacuum loading is extremely short and its effects can usually be examined separately from other live loads. For design, a hydraulic analysis of the system may be used to predict the magnitude and time variation of transients in both the positive and negative internal pressure.

25.3.2 Pipe and Contents

The effects of dead weight of the pipe wall and the fluid carried must be resisted by the structural capacity of the pipe. Neither of these loads contribute significantly to the overall stress state in most circumstances. In practice, loads from these two sources are often neglected in design of steel or plastic pipe, but they are usually included in design of prestressed and reinforced concrete pressure pipe and can be included in design of concrete nonpressure pipe as well. Formulas for determination of pipe wall bending moments and thrust forces, due to self-weight and fluid loads, are available in standard stress analysis references [43]. Since these loads are usually small compared to the overburden, they can be added to the vertical soil loads for simplicity and with conservatism.

25.4 Design Methods

25.4.1 General

The principal structural consideration in design of buried pipe is the ability to support all imposed loads. Other important items include the type of joints to be used and protection against environmental exposure. There are two fundamental approaches to design of buried pipe, based on the pipe's behavior under load [32, 40]. Pipe that undergoes relatively large deformations under its gravity loads, and obtains a large part of its supporting capacity from the passive pressure of the surrounding soil, is referred to as "flexible". As will be observed, the evaluation of the contribution of the soil to pipe strength is difficult due to varying conditions of pie installation. For that reason, prudence in design must be followed. However, as with most design problems, the engineer must, ultimately, balance conservatism with economic considerations.

Pipes with stiffer walls that resist most of the imposed load without much benefit of engagement of passive soil pressure, because deformation under load is restricted, are called "rigid". Steel, both corrugated and plain plate, ductile iron, and fiberglass reinforced plastic pipes are considered flexible; concrete pipe is considered rigid. Different methodologies are employed in assessing the strength of each type.

25.4.2 Flexible Design

Plain Steel

The structural capacity of flexible pipes is evaluated on the basis of resistance to buckling (compressive yield) and vertical diametrical deflection under load. Additionally, for flexible pipes, a nonstructural requirement in the form of a minimum stiffness to ensure that the pipe is not damaged during shipping and handling is normally imposed. In the case of steel pipes designed according to the recommendations of AWWA Manual M11 [21], the following two equations are used to choose pipe wall thickness sufficient to satisfy the handling requirement:

$$\begin{aligned} t &\geq \tfrac{D}{288} && \text{for diameter up to 54 in.} \\ t &\geq \tfrac{D+20}{400} && \text{for diameter greater than 54 in.} \end{aligned} \tag{25.3}$$

It is of interest to note that for many years, a minimum thickness of $D/200$ was used by pipe designers. In our experience, wall thicknesses meeting this ratio will usually result in designs that also satisfy the strength and deflection criteria discussed below. Tensile stresses due to internal pressure must be limited to a fraction of the tensile yield of the material. AWWA recommends limiting the tensile stress to 50% of yield.

Collapse, or buckling, of flexible pipes is difficult to predict theoretically because of the indeterminate nature of the load pattern. AWWA has published an expression for the determination of capacity of a given pipe to support imposed loads. The equation, given as Equation 6-7 in AWWA Manual M11 [21], incorporates the effects of the passive soil resistance, the buoyant effect of groundwater, and the stiffness of the pipe itself. Allowable buckling pressure is given by:

$$q_a = \left(\frac{1}{FS}\right)\left(32 R_w B' E' \frac{EI}{D^3}\right)^{1/2} \tag{25.4}$$

where

q_a = allowable buckling pressure (psi)
FS = factor of safety
= 2.5 for $(12h/D) \geq 2$ and

	$=$	3.0 for $(12h/D) < 2$
R_w	$=$	water buoyancy factor
	$=$	1-0.33 (h_w/h)
h	$=$	height of ground surface above top of pipe (ft)
h_w	$=$	height of groundwater surface above top of pipe (ft)
D	$=$	diameter of pipe (in.)
B'	$=$	coefficient of elastic support
	$=$	$\frac{1}{1+4e^{-0.065h}}$
E'	$=$	modulus of soil reaction (psi)
E	$=$	modulus of elasticity of pipe wall (psi)
I	$=$	moment of inertia per inch length of pipe wall (in.^{3})

In case vacuum load and surface live load are both included in the design conditions, AWWA recommends that separate load combinations be considered for each. That is because vacuum loads usually occur only for a short time and the probability of vacuum and maximum surface load occurring simultaneously is very small. In particular the following two load cases should be considered. For traffic live load:

$$q_a \geq \gamma_w h_w + R_w \frac{W_c}{D} + \frac{W_L}{D} \tag{25.5}$$

where

γ_w	$=$	specific weight of water (0.0361 lb/in.^{3})
W_L	$=$	live load on pipe (lb/in. length of pipe)
W_c	$=$	vertical soil load on pipe (lb/in. length of pipe)

For vacuum load:

$$q_a \geq \gamma_w h_w + R_w \frac{W_c}{D} + P_v \tag{25.6}$$

where

P_v	$=$	internal vacuum pressure (psi)

Deflection is determined by the Spangler formula:

$$\Delta y = D_l \left(\frac{K W_c r^3}{EI + 0.061 E' r^3} \right) \tag{25.7}$$

where

Δy	$=$	deflection of pipe (in.)
D_l	$=$	deflection lag factor (1.0 to 1.5)
K	$=$	bedding constant (0.1)
r	$=$	pipe radius (in.)

This form of the deflection equation was obtained by ordinary bending theory of a ring subject to an assumed pattern of applied vertical load, width of vertical reaction, and distribution of horizontal passive pressure [38, 42] and has been used in pipe design for over 50 years. According to the formula, deflection is limited by the stiffness of the pipe wall itself and by the effect of the passive pressure. It is significant to note that in the sizes of steel pipes often encountered, the ratio of the two components of resistance is on the order of 1:20, with the pipe wall stiffness being the smaller. Therefore, it is obvious that the passive resistance, which is closely related to the type of backfill and its degree of compaction, is the dominant influence on the vertical deflection of flexible pipes. That being the case, it becomes apparent that increasing the strength of a flexible pipe will probably be an inefficient way to properly limit deflection of underground pipe in most cases. The pipe installation must be completed as specified in order for this to be achieved.

Efforts to quantify the modulus of soil reaction, E', have continued since the initial development of the deflection equation. Suggested values are published in numerous references, including AWWA Manual M11 [21]. Values given there range from 200 to 3000 psi. The values depend on the type and level of compaction of the surrounding soil. Since pipe designers often have little control over the installation of pipe, historically, a value of E' in the range of 700 to 1000 psi has been assumed representative of average installations for estimating deflection at time of design.

In a recent work, engineers at the U.S. Bureau of Reclamation addressed the question of deflection of flexible pipe [27]. Their work, which is based on the wide experience of the Bureau of Reclamation in construction of all kinds of underground pipes, discusses appropriate values of E' based on not only the backfill and compaction used, but also the native soil. In addition to the soil modulus values, the authors also give a modified form of the deflection equation that includes a factor to account for long-term deflection, T_f (which replaces the factor D_l in Equation 25.7), and an additional multiplier on the soil modulus, called a design factor, F_d, with values ranging from 0.3 to 1.0. The combined effect of these two changes is, generally, to predict larger deflections than with the original Spangler equation. The revised equation becomes:

$$\Delta y = T_f \left(\frac{K W_c r^3}{EI + 0.061 F_d E' r^3} \right) \tag{25.8}$$

where

T_f = time lag factor

F_d = design factor

Values for Spangler's deflection lag factor, D_l, of 1.0 to 1.5 are recommended; designers usually use the 1.5 value for conservatism. Since the minimum recommended value for T_f is 1.5, the deflections by the modified equation will be higher. Values of the design factor, F_d, are presented for three cases, A, B, and C. The value for case A is 1.0; case B values, which are recommended for design, vary from 0.5 to 1.0; and case C values, which are recommended for designs in which deflection is critical, range from 0.3 to 0.75. In all cases the values of F_d increase with quality and level of compaction of the backfill.

It follows that control of bedding and backfill of flexible pipes during construction is critical to their performance. The required passive pressure can be developed only in high-quality fill material, compacted to the proper density. The material surrounding the pipe and extending above the pipe for at least 12 in. should be a well-graded granular stone. Coarse-grained material provides much higher passive resistance and, therefore, limits pipe deflection, in flexible pipe systems, more than fine-grained soil types. Compaction in the lower levels of the pipe is critical. Hand tampers or similar equipment are necessary to ensure that adequate density is obtained in the region below the lower haunches of the pipe. Historically, failure to achieve the proper level of compaction in this area of difficult accessibility has been identified as a major contributing cause to excessive deformations in flexible pipe construction.

It is common practice to limit the final vertical deflection of unlined pipes to less than 5% of the diameter. Deflection of pipes with cement mortar coatings should be limited to 2% of the diameter. Field observations of steel pipes in service indicate that once the deflection reaches 20% of the diameter, collapse is imminent.

EXAMPLE 25.1:

A 96-in.-diameter steel pipe with a 1/2-in. wall is installed with its top 15 ft below the ground surface. The local water table is located 7 ft below the surface. Assume that the soil has a modulus of reaction, E', of 1000 psi, and that it has a unit weight of 120 pcf.

2. Verify that the pipe will satisfy the buckling and deflection criteria given in AWWA Manual M11 [21].
3. Determine the amount of vacuum load that can be supported by the pipe.

Solution

1. The weight of soil bearing on the pipe is calculated from the prism of soil from the top of the pipe to the ground surface:

$$\begin{aligned} W_c &= \gamma_s h D \\ &= 120 \times 15 \times (96/12) = 14{,}400 \ \text{lb/ft} \end{aligned}$$

Determine the h/D ratio to obtain the appropriate factor of safety:

$$\frac{h}{D} = \frac{15}{8} = 1.875 < 2; \qquad \text{therefore } \ FS = 3$$

The groundwater surface is $15 - 7 = 8$ ft above the top of the pie. The water buoyancy factor (R_w) and the coefficient of elastic support (E') are calculated on the basis of the depth of cover and groundwater:

$$\begin{aligned} h_w &= 8ft \\ R_w &= 1 - 0.33\frac{h_w}{h} = 1 - 0.33 \times \frac{8}{15} = 0.824 \\ B' &= \frac{1}{1 + 4e^{-0.065 \times 15}} = 0.399 \end{aligned}$$

The modulus of elasticity for steel is 29×10^6 psi; the moment of inertia per inch length of pipe is

$$I = \frac{t^3}{12} = \frac{0.5^3}{12} = 0.0104 \ \text{in.}^3; \qquad \text{hence the product } \ EI = 302{,}083 \ \text{in.-lb}$$

Therefore, by Equation 25.4, the allowable buckling pressure is

$$q_a = \left(\frac{1}{3}\right)\left(32 \times 0.824 \times 0.399 \times 1{,}000 \times \frac{302{,}083}{96^3}\right)^{1/2} = 19.968 \ \text{psi}$$

The total applied load intensity, Q, is given by

$$Q = \gamma_w h_w + R_w \frac{W_c}{D} + \frac{W_L}{D} = 0.0361 \times 96 + 0.824 \times \frac{14{,}400}{12 \times 96} + 0 = 13.766 \ \text{psi}$$

Since $Q < q_a$, the pipe is safe against buckling. Check deflection:

$$\Delta y = 1.5\left(\frac{0.1 \times \frac{14{,}400}{12} \times 48^3}{302{,}083 + 0.061 \times 1{,}000 \times 48^3}\right) = 2.824 \ \text{in.}$$

The calculated deflection is approximately 3% of the diameter, less than the 5% usually specified as the limit for unlined pipe.

2. The vacuum pressure that can be supported within the buckling capacity of the pipe is the difference between the calculated critical buckling capacity, q_a, and the applied load intensity, Q:

$$P_v = q_a - Q = 19.968 - 13.766 = 6.202 \ \text{psi}$$

Corrugated Steel

Corrugated steel has the advantage of greater flexural strength per unit weight of material than plain steel, and has been widely used in surface drainage systems and to a lesser extent in process water systems. In this form, the pipe is assembled from corrugated sheets, rolled to radius and bolted or riveted together.

Corrugated steel pipes can be designed according to ASTM standard practice A796 (American Society for Testing and Materials). The practice covers both curve and tangent ("sinusoidal") walls and smooth walls with helical ribs of rectangular section at regular intervals for increased strength. As with plain steel pipe, this design procedure requires a minimum stiffness in the pipe wall for shipping and handling. To make a quantitative evaluation of the degree of stiffness, a flexibility factor, defined for all wall configurations as

$$FF = \frac{D^2}{EI} \tag{25.9}$$

where

FF = flexibility factor (in.-lb^{-1})
D = pipe diameter (in.)
E = modulus of elasticity (psi)
I = moment of inertia of wall cross-section per inch (in.^{3})

is subject to limits depending on the corrugation configuration and the type of installation. For example, in configurations of sinusoidal corrugations, specified in ASTM A760 and A761 [4, 5], values of the flexibility factor are restricted to 0.020 to 0.060.

The phenomenon of buckling of buried corrugated pipes has been investigated, through prototype testing, by Watkins [3]. Design curves utilizing the results of that research were originally published in an American Iron and Steel Institute (AISI) design manual [3] and have been continued into the current edition of the book. The curves provide buckling loads for corrugated steel–walled pipes as a function of diameter-to-radius of gyration ratio. The design equations given in ASTM A796 [6] are of the same general form as the design curves developed by AISI. That is, there are three ranges of behavior—elastic buckling, inelastic buckling, and yield—and the dependence of the expressions on the independent variable, D/r, is the same in the two regimes of the formulas in both documents. The principal difference between the two approaches is the inclusion of an explicit dependence on soil stiffness in the ASTM A796 equations. The AISI formulas, on the other hand, account for soil stiffness by reduction in applied load for well-compacted backfills.

The applicable formulas for critical buckling stress, as given in ASTM A796 [6], and their applicable ranges of diameter-to-radius of gyration ratio are given below:

$$\begin{aligned} f_c &= f_u - \frac{f_u^2}{48E}\left(\frac{kD}{r}\right)^2 &\quad \text{for } \frac{kD}{r} \le \sqrt{\frac{24E}{f_u}} \\ f_c &= \frac{12E}{\left(\frac{kD}{r}\right)^2} &\quad \text{for } \frac{kD}{r} > \sqrt{\frac{24E}{f_u}} \end{aligned} \tag{25.10}$$

subject to the provision that:

$$f_c \le f_y \tag{25.11}$$

where

f_c = critical buckling stress (psi)
f_y = specified minimum yield stress (psi)
f_u = specified minimum ultimate stress (psi)
k = soil stiffness factor = 0.22 for material at 90% density

E = modulus of elasticity (psi)
D = pipe diameter (in.)
r = radius of gyration of pipe wall (in.)

The buckling formulas in ASTM A796 [6] also are given in the AASHTO (American Association of State Highway and Transportation Officials) specification for design of highway bridges [1].

The corresponding equations provided in the AISI handbook [3] are

$$
\begin{aligned}
f_c &= 40{,}000 - 0.081\left(\frac{D}{r}\right)^2 && \text{for } 294 < \frac{D}{r} < 500 \\
f_c &= \frac{4.93 \times 10^9}{\left(\frac{D}{r}\right)^2} && \text{for } \frac{D}{r} > 500
\end{aligned}
\tag{25.12}
$$

with, again, the provision that the critical buckling stress cannot exceed the yield stress, f_y.

A comparison of the two sets of formulas can be made to determine the maximum variation. The soil parameter, k, obviously affects the results. In ASTM A796 [6], a value of $k = 0.22$ is recommended for good fill material compacted to 90% of standard density; no suggestions are provided for other backfill conditions. The AISI expressions for critical buckling stress (Equation 25.12) do not contain any dependence on the degree of compaction of the surrounding soil. However, the handbook does recommend load factors, which multiply the applied loads, that are related to the degree of compaction. For example, the recommended load factor for 90% compaction is 0.75. Use of a load factor of 0.75 has the same effect as increasing the allowable stress by 1.33. If the results from the ASTM and AISI equations are compared on that basis, the values are within 10%. On the other hand, if a load factor of 1.0, which corresponds to a density of only 80% standard, is used, the soil stiffness, k, must be increased to 0.26 for the two sets of formulas to give approximately the same results. Clearly, use of the higher value for k results in a slightly more conservative design, and that may be desirable, in view of the normally unknown character of the actual installed backfill.

In either case, the appropriate wall cross-section must be selected to satisfy

$$
\frac{W_c}{2A} \leq \frac{f_c}{(SF)}
\tag{25.13}
$$

where

A = cross-section area of wall per unit length
SF = safety factor = 2
W_c = vertical load per unit length of pipe (*Note:* W_c must be multiplied by the appropriate load factor if the AISI equations are used)

It is of interest to note the D/r ratios that form the transition between elastic and inelastic and between buckling and yield behavior in the ASTM A796 equations. For pipe meeting ASTM A760, maximum thickness of 0.168 in., the specified minimum yield and ultimate stress are 33 ksi and 45 ksi, respectively. For those values, elastic buckling controls design for D/r ratios greater than 478 and yield controls for D/r ratios less than 350, for k equal to 0.26. These values correspond fairly closely with the AISI limits of 500 and 294, respectively.

EXAMPLE 25.2:

Consider the pipe in Example 25.1. Determine the minimum wall thickness of 3 in. x 1 in. corrugated pipe that will satisfy the buckling expressions of Equation 25.10 and the handling requirement of Equation 25.9 with FF limited to a maximum value of 0.033. Assume a value of k of 0.26. Also, the minimum specified yield (f_y) and ultimate (f_u) stresses for the material are 33,000 and 45,000 psi, respectively.

Solution The radius of gyration for all thicknesses in the 3×1 series range from 0.3410 to 0.3499 in. (see Table 25.3).

Therefore, in the calculations use

$$r = 0.34 \text{ in.}$$

It follows that

$$\frac{kD}{r} = \frac{0.26 \times 96}{0.34} = 52.8 \qquad \sqrt{\frac{24E}{f_u}} = \sqrt{\frac{24 \times 29 \times 10^6}{45{,}000}} = 124.4$$

Since $\frac{kD}{r} < \sqrt{\frac{24E}{f_u}}$, the first of Equation 25.10 must be used:

$$\begin{aligned} f_c &= f_u - \frac{f_u^2}{48E}\left(\frac{kD}{r}\right)^2 = 45{,}000 - \frac{45{,}000^2}{48 \times 29 \times 10^6}\left(\frac{0.26 \times 96}{0.34}\right)^2 \\ &= 37{,}160 \text{ psi} > f_y \end{aligned}$$

Since the calculated buckling stress exceeds the yield, the critical stress is the yield stress, 33,000 psi.

The required wall area per foot of length can be determined by rearrangement of Equation 25.13:

$$A = \frac{W_c}{2\frac{f_c}{(SF)}} = \frac{14{,}400}{2 \times \frac{33{,}000}{2}} = 0.437 \text{ in.}^2$$

The lightest section, thickness of 0.052 in., will satisfy the stress requirement. Check the handling requirement. The moment of inertia per inch of wall is 6.892×10^{-3} in.^{3}:

$$FF = \frac{D^2}{EI} = \frac{96^2}{(29 \times 10^6) \times (6.892 \times 10^{-3})} = 0.046 > 0.033$$

Since the flexibility factor is too large, try the next section in the series, the 0.064-in. thickness, and $I = 8.658 \times 10^{-3}$ in.^{3}:

$$FF = \frac{D^2}{EI} = \frac{96^2}{(29 \times 10^6) \times (8.658 \times 10^{-3})} = 0.037 > 0.033$$

Since the flexibility is still too great, the next section in the series, with a thickness of 0.079 in. and flexibility factor of 0.029 is chosen. This example demonstrates a condition that occurs quite often in the design of flexible pipes; if the handling and installation minimum stiffness requirements are met, the strength requirements are automatically taken care of.

Fiberglass Reinforced Plastic

Fiberglass reinforced plastic (FRP) pipe is fabricated by winding glass strands into a matrix of organic resin on a mandrel of the desired diameter. A variation on the fiberglass-resin matrix utilizes cement of polymer mortar incorporated into the structure to add stiffness and reduce cost of materials. ASTM standards D3262 [9], D3517 [10], and D3754 [11], and AWWA standard C950 [19] provide requirements for manufacture of both the fiberglass-resin and the mortar pipe in diameters up to 144 in. Structural strength and rigidity against external loads for this type of pipe are established by load tests performed as specified by ASTM D2412 [8]. In the load test, equal and opposite concentrated loads are applied on opposite ends of a diameter. Load deflection data are obtained from which stiffness and related buckling strength of the pipe can be determined.

Each of the mentioned pipe specifications provides for levels of pipe stiffness (PS) of 9, 18, 36, and 72 psi. These values represent applied force per unit length of pipe divided by deflection. Use of

TABLE 25.3 Sectional Properties for 3×1 in. Corrugated Sheets

Thickness in.	Area in.^{2}/ft	Moment of inertia in.$^3 \times 10^{-3}$	Radius of gyration in.
0.052	0.711	6.892	0.3410
0.064	0.890	8.658	0.3417
0.079	1.113	10.883	0.3427
0.109	1.560	15.458	0.3448
0.138	2.008	20.175	0.3472
0.168	2.458	25.083	0.3499

From American Society for Testing and Materials. 1994. A796. *Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe-Arches and Arches for Storm and Sanitary Sewers and Other Buried Applications.* With permission.

the pipe stiffness and the formula for deflection of a point-loaded circular ring allows determination of the product, EI, of the composite pipe wall. In FRP construction, the modulus of elasticity (E) depends on several variables: the moduli of the resin and the glass reinforcement, the relative amounts of glass 20 and resin, and the angle of the filament winding. For that reason, it is convenient to utilize the experimentally determined overall pipe stiffness in design rather than to base calculations on the composite modulus of elasticity of the material.

In particular, the buckling formula (Equation 25.4) and Spangler's equation for deflection (Equation 25.7) can be recast in terms of the pipe stiffness, as shown in the following steps. The formula for deflection of a concentrated loaded pipe can be rearranged to provide an expression for EI:

$$EI = 0.149\frac{F}{\Delta}r^3 \tag{25.14}$$

where

F = concentrated load per inch of pipe (lb/in.)
D = pipe deflection (in.)
r = pipe radius (in.)

Since the pipe stiffness is defined as

$$PS = \frac{F}{\Delta} \tag{25.15}$$

the relationship between EI and PS becomes

$$EI = 0.149(PS)r^3 \tag{25.16}$$

The allowable buckling stress expression, therefore, can be rewritten:

$$q_a = \left(\frac{1}{FS}\right)\left(0.596R_w B' E' PS\right)^{1/2} \tag{25.17}$$

and the deflection formula is

$$\Delta y = D_l\left(\frac{KW_c}{0.149PS + 0.061E'}\right) \tag{25.18}$$

For underground installations, many fiberglass pipe manufacturers recommend a minimum pipe stiffness of 36 psi in order to ensure sufficient stiffness to perform backfilling properly. Deflections are normally restricted to 5% of the diameter. In contrast to design of steel pipe, it is normal practice to consider the bending stresses induced in the wall by deflection of the pipe. Methods for evaluating these stresses in combination with the stresses due to internal pressure have been developed by a committee of AWWA and will be included in one chapter of a manual on design of fiberglass reinforced water pipe, scheduled for publication in the near future [22].

25.4.3 Rigid Design

Concrete Pressure Pipe

Concrete pipe can be used for both pressure and nonpressure applications. It offers the advantage of being corrosion resistant in conditions where steel might be attacked, and in some instances it may be a more cost-effective solution than steel or plastic.

When concrete pipe is used in high-pressure systems, prestressed concrete pipe is the type most often selected. The pipe is manufactured in diameters from 24 to 144 in. and is fabricated with walls from 4 to 12 in. thick. A steel cylinder, usually of 16-gal thickness, is embedded in the wall for leak protection. The outer surface of the wall is wrapped with high-strength wire under tensile stresses in the 175–200 ksi range. The prestressing places the concrete wall into compression of sufficient magnitude so that it will not be fully relieved under design internal pressure loadings. Finally, a coat of sand-cement mortar is applied over the prestressing wires to provide corrosion protection. A comprehensive design procedure for this type of pipe is contained in AWWA C304 [17]. Prestressed pipe is normally designed only by pipe manufacturers. The design provisions meet AWWA C304 and the actual pipe is fabricated according to AWWA C301 [14]. Pipe purchasers must indicate the design pressures, including transients, installation conditions, and surface loads.

Reinforced concrete pipe can be designed to sustain internal pressure loads, but the maximum pressures that can be carried are significantly less than with prestressed pipe and its use in such applications is limited. AWWA C300, C302, and C303 [13, 15, 16] are all specifications covering the design and fabrication of reinforced concrete pressure pipe in differing configurations of reinforcing and with or without the steel cylinder pressure boundary. Design procedures for all three specifications are presented in AWWA Manual M9 [20] and the interested reader is encouraged to review that manual for details. As with prestressed pipe, the pipe specifier usually supplies only the performance attributes and the pipe fabricator performs the design to meet the appropriate specification.

Concrete Nonpressure Pipe

ASTM C76 [7] contains specification requirements for reinforced concrete pipe not intended for pressure applications. Five classes of pipe, classes I–V, respectively, representing five levels of structural strength, are specified. The strength is characterized by the concentrated load required to cause a crack of 0.01 in. width and the ultimate concentrated load. Load values are determined experimentally by the three-edge-bearing test. The test simulates concentrated loads applied at opposite ends of a pipe diameter. These loads are referred to as D-loads ($D_{0.01}$ and D_{ult}): the concentrated force per unit length of pipe per unit length of diameter necessary to cause either the 10-mil-width crack or ultimate failure of the pipe. D-load values for the five pipe classes included in ASTM C76 are shown in Table 25.4.

In determination of the strength required to resist external loads, the total pipe load is estimated by standard methods. Bedding factors, based on the type of installation, the soil type, and its level of compaction, have been developed by the American Concrete Pipe Association [2]. These factors represent the ratio of the maximum bending moment due to a concentrated load to the moment caused by the actual live and dead load of the same magnitude as the concentrated load.

ACPA has defined four standard installation types, for which relevant information is shown in Tables 25.5 and 25.6. Bedding factors for embankment installations are given in Table 25.7. Other bedding factors, for trench installations and for live load effects, have also been obtained by ACPA but are not reproduced here. It is noted that ACPA recommends using the dead load factor for live load contributions as well, if the tabulated live load factor is larger than the dead load factor. The calculation methodology used to obtain the various factors is described in the ACPA design data [2].

Use of the appropriate bedding factor allows the conversion of the actual load to an equivalent point load. Comparison of that equivalent load with standard D-loads is used to establish the appropriate class of pipe with sufficient capacity to support the design loads. Normal procedure is to utilize a

TABLE 25.4 D-Loads for ASTM C76 Concrete Pipe

Pipe class	$D_{0.01}$ load	D_{ult} load
I	800	1200
II	1000	1500
III	1350	2000
IV	2000	3000
V	3000	3750

From American Society for Testing and Materials. 1994. C76. *Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe.* With permission.

design factor of safety of 1.0 against the D-load required to cause a 0.01-in. crack. Details of the procedure are illustrated in the following example problem for embankment installation. The design procedure is similar for trench installation.

EXAMPLE 25.3:

Consider a 48-in.-diameter reinforced concrete pipe to be installed beneath a railroad for surface drainage. The pipe is to be installed in an embankment with a depth of cover of 5 ft. For the purpose of this example, assume that the overburden load is equal to the prism of soil above the pipe. Soil unit weight is 120 pcf and the backfill conditions are such that a standard installation type 3 exists.

Solution For a 48-in. pipe, the wall thickness of a pipe meeting ASTM C76 is 5 in. The soil dead weight is given by

$$W_E = whB_c = 120 \times 5 \times \frac{48+10}{12} = 2900 \text{ lb/ft}$$

The live load intensity is obtained from Table 25.2:

$$W_L = w_{LL}B_c = 2400 \times \left(\frac{48+10}{12}\right) = 11{,}600 \text{ lb/ft}$$

The total overburden plus live load is

$$W_E + W_L = 2{,}900 + 11{,}600 = 14{,}500 \text{ lb/ft}$$

From Table 25.7, the bedding factor, B_{fe}, is found to be 2.2; the live load bedding factor (not tabulated here) is also 2.2 for this installation. Therefore, use a bedding factor of 2.2 for the total load:

$$\frac{\text{Total load}}{\text{Bedding factor}} = \frac{14{,}500}{2.2} = 6{,}591$$

To obtain required D-load, divide this result by the pipe diameter:

$$D_{0.01} \text{ required } = \frac{6591}{4} = 1648$$

Using class IV pipe, $D_{0.01} = 2000, \quad D_{\text{ult}} = 3000.$

TABLE 25.5 Equivalent USCS and AASHTO Soil Classifications for SIDD Soil Designations (ACPA)

	Representative Soil Types	
SIDD[a] soil	USCS	AASHTO
Gravelly sand (category I)	SW, SP, GW, GP	A1, A3
Sandy silt (category II)	GM, SM, ML Also GC, SC with less then 20% passing #200 sieve	A2, A4
Silty clay (category III)	CL, MH, GC, SC CH	A5, A6 A7

[a] Standard Installations Direct Design, ACPA

From American Concrete Pipe Association. 1995. Design Data 40. *Standard Installations and Bedding Factors for the Indirect Design Method.* With permission.

TABLE 25.6 Standard Embankment Installation Soils and Minimum Compaction Requirements (ACPA)

Installation type	Haunch and outer bedding	Lower side
Type 1	95% Category I	90% Category I, 95% category II, or 100% category III
Type 2	90% Category I or 95% category II	85% Category I, 90% category II, or 95% category III
Type 3	85% Category I, 90% category II, or 95% category III	85% Category I, 90% category II, or 95% category III
Type 4	No compaction required, except if category III, use 85% category III	No compaction required except if category III, use 85% category III

Note: Bedding thickness for all types: $D_o/24$ minimum, not less than 3 in. If rock foundation, use $D_o/12$ minimum, not less than 6 in. Compaction is standard Proctor.

From American Concrete Pipe Association. 1995. Design Data 40. *Standard Installations and Bedding Factors for the Indirect Design Method.* With permission.

TABLE 25.7 Bedding Factors, Embankment Condition, B_{fe} (ACPA)

Pipe	Standard installation			
diameter, (in.)	Type 2	Type 1	Type 3	Type 4
12	3.2	4.4	2.5	1.7
24	3.0	4.2	2.4	1.7
36	2.9	4.0	2.3	1.7
72	2.8	3.8	2.2	1.7
144	2.8	3.6	2.2	1.7

From American Concrete Pipe Association. 1995. Design Data 40. *Standard Installations and Bedding Factors for the Indirect Design Method.* With permission.

25.5 Joints

25.5.1 General

In order to form a continuous conduit from the individual pipe sections, it is necessary to connect the sections together in such a way that the pressure-containing and load-resisting capability is preserved in the completed assembly. Each type of pipe discussed previously utilizes special types of joints as explained in the following.

25.5.2 Joint Types

Plain Steel

In plain steel plate pipe, the individual pipe sections are fabricated from plate, rolled to the proper radius, and welded together. Joints, in fabricated sections, are either continuous helical or longitudinal. When installed, the sections are welded together using either bell-and-spigot or butt joints.

In plain steel pipe, full penetration butt welds are used extensively for field joints. In water works construction, welding of pipelines is covered by AWWA C206 [12]. That standard requires that welding procedures and welding operators be prequalified before use on a job. In addition, tolerances on fit-up are specified and inspection requirements are set out. Strict adherence to project specifications is necessary to guarantee that the desired continuity is obtained at the junction.

Although connecting pipe segments by use of full-penetration butt welds has enjoyed wide acceptance in pipeline construction, bell-and-spigot joints, fillet welded, may also be used. These joints require only fillet welds and are generally considered to be less expensive to install than the full-penetration butt weld. However, due to the inherent eccentricity in such joints, a potential for failure exists under certain temperature conditions when longitudinal tensile stresses are developed. Some failures of welded bell-and-spigot joints were reported in the technical literature a few years ago [26, 31]. Since that time, requirements for welding of bell-and-spigot joints in steel pipe in AWWA C206 [12] have been revised to minimize the potential for failure in this type of joint.

Corrugated Steel

The field joints used in corrugated steel are usually made by bolting, either in lap joints or with coupling bands that fit over two adjacent sections. In most cases, gaskets should be used at joints to provide leak tightness.

Fiberglass Reinforced Plastic

Several types of joints are used in fiberglass pipe. Coupling or bell-and-spigot joints with O-ring gaskets (see Figure 25.2) and mechanical couplings, for unrestrained joints, are specified by the ASTM fiberglass pipe specifications mentioned previously. These joints can be used with restraining devices, such as tie rods, if necessary. In addition, continuous hand lay-up joints consisting of alternating layers of glass fabric and resin or adhesive-bonded bell-and-spigot joints are used for joints that must resist longitudinal force as well as contain the pressure exerted by the fluid carried.

FIGURE 25.2: Bell-and-spigot and coupling joints for fiberglass pipe. (From American Society for Testing and Materials. 1991. D3517. *Standard Specification for "Fiberglass" Glass-Fiber-Reinforced Thermosetting-Resin Pressure Pipe.* With permission.)

Prestressed Concrete

In straight runs of prestressed concrete pressure pipe, the most common joint type is the bell-and-spigot slip-on joint with a rubber O-ring gasket (see Figure 25.3). When making the joint, care should be used to ensure that the gasket is in its proper place and that the mating ends are properly located with respect to each other. The exterior of the joint should be filled with flowable sand-cement grout, contained by a suitable appliance. Grouting of the inside joint gap may be required, depending on water chemistry. When it is, the grouting should be completed after the backfill is compacted. The interior surface of each joint must be smoothed to allow unrestricted flow. When axial tension forces must be transmitted across a joint, locking variations of the basic slip-on joint are available.

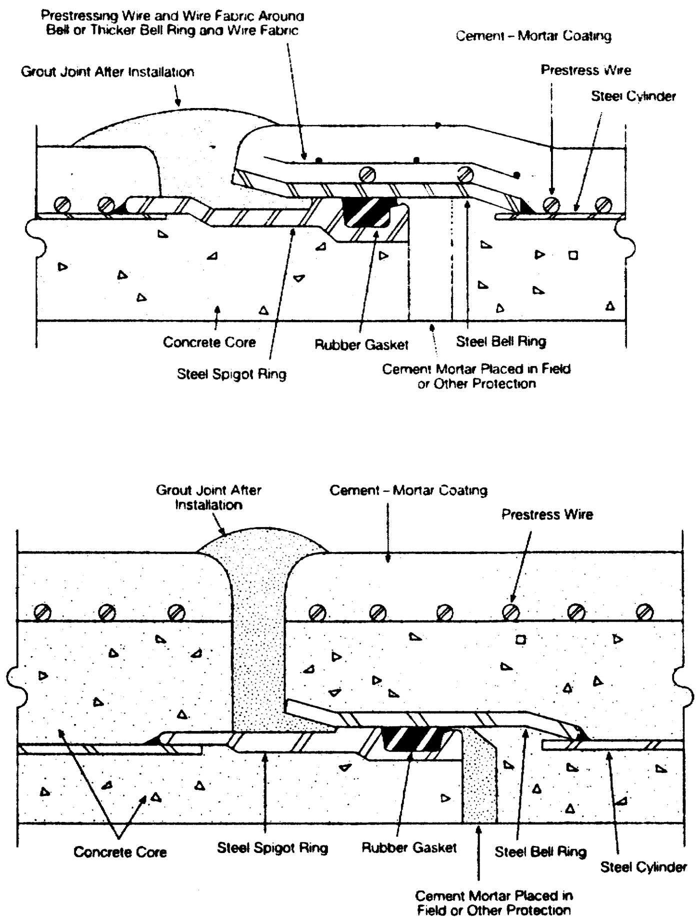

FIGURE 25.3: Prestressed concrete joint details. (Reprinted from *Concrete Pressure Pipe*, M9, by permission. Copyright©1995, American Water Works Association.)

Reinforced Concrete

Typical joints for reinforced concrete pressure pipe are shown in Figure 25.4. Joints for concrete nonpressure pipe are similar to the concrete-only joints in Figure 25.4. In some cases, gaskets are not used in nonpressure pipe.

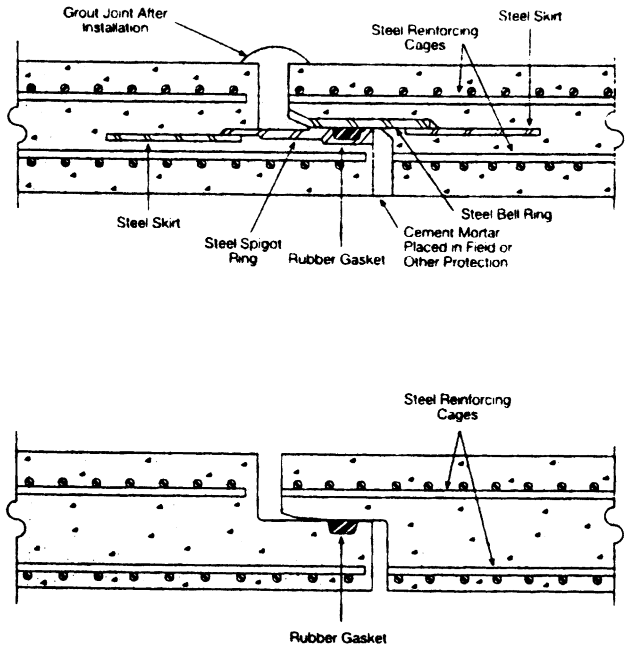

FIGURE 25.4: Joint details for reinforced concrete pipe. (Reprinted from *Concrete Pressure Pipe*, M9, by permission. Copyright©1995, American Water Works Association.)

25.5.3 Hydrostatic Testing

A field hydrostatic test is usually performed to verify that all joints are watertight. Test pressure should exceed the maximum design pressure, including transients, by at least 25%. Leakage through welded joints should be virtually nonexistent. It is common to allow a slight leakage rate for O-ring gasketed joints (AWWA, C600).

25.6 Corrosion Protection

There are three environmental agents that exert strong influence on corrosion of the pipe wall material in buried installations. These are the water, or other fluid, carried, the soil in contact with the pipe, and the groundwater. In the case of certain process water systems, such as power plant condenser cooling systems, the water may be circulated continuously within a closed loop using cooling towers, lakes, or other means of exhausting heat. When closed systems are used, even in fresh water environments, the concentrations of certain compounds in the water may increase and cause elevated corrosion rates in steel pipes. Chlorides are generally believed to be the most aggressive compounds, normally found in water sources, in regard to corrosion of steel. Chlorides can also be harmful to concrete pipes, posing threats to the concrete itself and to steel reinforcing. Sulfates are not usually associated with steel corrosion, but they can be detrimental to concrete.

Once-through systems, on the other hand, are usually less corrosive for steel pipes and less harmful to concrete than the closed-cycle systems. When brackish water is used for cooling, positive steps must be taken to ensure that corrosion is controlled.

While the process water carried may promote corrosion or other damage on the inside of the pipe, the outside surface may be attacked by the surrounding soil, the groundwater, or both. Soils with low electrical resistivity may help advance corrosion in steel. Soils that have sulfate compounds above certain critical levels can cause damage to concrete pipe. The groundwater can have the same effects on the exterior of the pipe as the process fluid has on the inside. Specifically, groundwater with high chloride or sulfate contents may be harmful to the pipe material.

Because of the wide range of possibilities for the existence of detrimental chemical action, it is essential that the nature of the external and internal environments of the pipe be evaluated in the design process. Chemical analyses of the process water and the groundwater are essential. Also, chemical analysis of the soil and resistivity survey results must be available in order to make the best choice of pipe system to withstand the exposure throughout the design life of the facility.

25.6.1 Coatings

Coatings can be used to inhibit corrosion or other forms of deterioration in both concrete and steel pipes. Type and extent of coatings depends on the service environment. Steel pipes are almost always coated externally. Coal tar enamel and wrapping has been used successfully in the U.S. for decades. Epoxies and urethanes, among others, have become popular in more recent times.

Cement mortar coatings may be applied to both the interior and exterior of steel pipes. This type of coating offers several advantages and has an extensive record of satisfactory service. When exterior coating of prestressed concrete pipes is desired, certain epoxies are acceptable. Because prevention of corrosion in the prestressing wires is so important, pipe designers sometimes specify an additional coating to supplement the protection furnished by the cement mortar coating.

25.6.2 Cathodic Protection

Protection against corrosion may, in certain circumstances, require a cathodic protection system. For example, cathodic protection has proven to be very successful in providing leak-free high-pressure oil and natural gas pipelines throughout the U.S. Power plant sites have widely dispersed grounding systems, which can cause unpredictable stray currents that may promote steel corrosion. Cathodic protection must be designed by competent engineers based on information regarding the extent of buried facilities, the soil resistivity measurements, and the plant grounding system. Electrical continuity should be provided on prestressed concrete pipe if present or future installation of cathodic protection is a possibility.

References

[1] American Association of State Highway and Transportation Officials (AASHTO). 1992. *Standard Specifications for Highway Bridges.* 15th ed.

[2] American Concrete Pipe Association (ACPA). 1995. Design Data 40. *Standard Installations and Bedding Factors for the Indirect Design Method.*

[3] American Iron and Steel Institute (AISI). 1977. *Handbook of Steel Drainage & Highway Construction Products.*

[4] American Society for Testing and Materials (ASTM). 1993. A760. *Standard Specification for Corrugated Steel Pipe, Metallic Coated for Sewers and Drains.*

[5] American Society for Testing and Materials (ASTM). 1990. A761. *Standard Specification for Corrugated Steel Structural Plate, Zinc-Coated, for Field-Bolted Pipe, Pipe-Arches, and Arches.*

[6] American Society for Testing and Materials (ASTM). 1994. A796. *Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe-Arches and Arches for Storm and Sanitary Sewers and Other Buried Applications.*

[7] American Society for Testing and Materials (ASTM). 1994. C76. *Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe.*

[8] American Society for Testing and Materials (ASTM). 1993. C2412. *Standard Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading.*

[9] American Society for Testing and Materials (ASTM). 1993. D3262. *Standard Specification for "Fiberglass" (Glass-Fiber-Reinforced Thermosetting-Resin Sewer Pipe).*

[10] American Society for Testing and Materials (ASTM). 1991. D3517. *Standard Specification for "Fiberglass" (Glass-Fiber-Reinforced Thermosetting-Resin Pressure Pipe).*

[11] American Society for Testing and Materials (ASTM). 1991. D3754. *Standard Specification for "Fiberglass" (Glass-Fiber-Reinforced Thermosetting-Resin Sewer and Industrial Pressure Pipe).*

[12] American Water Works Association (AWWA). 1991. C206. *Standard for Field Welding of Steel Water Pipe.*

[13] American Water Works Association (AWWA). 1989. C300. *Standard for Reinforced Concrete Pressure Pipe, Steel-Cylinder Type, for Water and Other Liquids.*

[14] American Water Works Association (AWWA). 1992. C301. *Standard for Prestressed Concrete Pressure Pipe, Steel-Cylinder Type, for Water and Other Liquids.*

[15] American Water Works Association (AWWA). 1987. C302. *Standard for Reinforced Concrete Pressure Pipe, Noncylinder Type, for Water and Other Liquids.*

[16] American Water Works Association (AWWA). 1987. C303. *Standard for Reinforced Concrete Pressure Pipe, Steel Cylinder Type, Pretensioned, for Water and Other Liquids.*

[17] American Water Works Association (AWWA). 1992. C304. *Standard for Design of Prestressed Concrete Cylinder Pipe.*

[18] American Water Works Association (AWWA). 1987. C303. *Standard for Installation of Ductile-Iron Water Mains and Their Appurtenances.*

[19] American Water Works Association (AWWA). 1988. C950. *Standard for Fiberglass Pressure Pipe.*

[20] American Water Works Association (AWWA). 1995. Manual M9. *Concrete Pressure Pipe.*

[21] American Water Works Association (AWWA). 1989. Manual M11. *Steel Pipe-A Guide for Design and Installation.*

[22] American Water Works Association (AWWA). 1997 (projected). Manual M45. *Fiberglass Pressure Pipe.*

[23] Anton, W.F., J.E. Herold, R.T. Dailey, and W.J. Cichanski. 1990. *Investigation & Rehabilitation of Seattle's Tolt Pipeline.* Proceedings of the International Conference on Pipeline Design and Installation. ASCE. Las Vegas. pp. 213–229.

[24] Committee on Seismic Analysis of the ASCE Structural Division Committee on Nuclear Structures and Materials. 1983. *Seismic Response of Buried Pipes and Structural Components.* American Society of Civil Engineers (ASCE).

[25] Doyle, J.M. and S.L. Chu. 1968. Plastic Design of Flexible Conduits. *J. of Structural Division (ASCE),* 94, 1935–1944.

[26] Eberhardt, A. 1990. 108-in. Diameter Steel Water Conduit Failure and Assessment of AWWA Practice. *J. of Performance of Constructed Facilities (ASCE),* 4, 30–50.

[27] Howard, A.K., L.A. Kinney, and R.P. Fuerst. 1995. *Method for Prediction of Flexible Pipe Deflection.* Report M-25 (M0250000.995). U.S. Bureau of Reclamation. Denver, CO.

[28] Marston, A. and A.O. Anderson. 1913. The Theory of Loads on Pipes in Ditches and Tests of Cement and Clay Drain Tile and Sewer Pipe. Bull. 31. Iowa Engineering Experiment Station. Ames, IA.

[29] Marston, A. 1930. The Theory of External Loads on Closed Conduits in the Light of the Latest Experiments. Bull. 96. Iowa Engineering Experiment Station. Ames, IA.

[30] MathSoft Inc. 1994. *Mathcad PLUS 5.0.* MathSoft Inc. Cambridge, MA.

[31] Moncarz, P.D., J.C. Shyne, and G.K. Derbalian. 1987. Failures of 108-inch Steel Pipe Water Main. *J. of Performance of Constructed Facilities (ASCE)*, 1, 168–187.

[32] Netzel, R.J. and J.M. Doyle. 1983. Design of Underground Pipelines. *Proc. American Power Conference*, 45, 792.

[33] Newmark, N.M. 1935. Simplified Computation of Vertical Pressures in Elastic Foundations. Circular No. 24. Engineering Experiment Station. University of Illinois. Urbana, IL.

[34] Schlick, W.J. 1932. Loads on Pipe in Wide Ditches. Bull. 108. Iowa Engineering Station. Ames, IA.

[35] Schlick, W.J. 1952. Loads on Negative Projecting Conduits. *Proc. Highway Research Board*, 31, 308.

[36] Spangler, M.G., R. Winfrey, and C. Mason. 1926. Experimental Determination of Static and Impact Loads Transmitted to Culverts. Bull. 76. Iowa Engineering Experiment Station. Ames, IA.

[37] Spangler, M.G. and R.L. Hennessy. 1946. A Method of Computing Live Loads Transmitted to Underground Conduits. *Proc. Highway Research Board*, 26, 83.

[38] Spangler, M.G. 1941. The Structural Design of Flexible Pipe Culverts. Bull. 153. Iowa Engineering Experiment Station. Ames, IA.

[39] Spangler, M.G. 1950. A Theory of Loads on Negative Projecting Conduits. *Proc. Highway Research Board*, 29, 153.

[40] Spangler, M.G. and R.L. Handy. 1982. *Soil Engineering.* 4th ed. Harper & Row.

[41] Timoshenko, S. and J.N. Goodier. 1951. *Theory of Elasticity.* McGraw-Hill, New York.

[42] Watkins, R.K. and M.G. Spangler. 1958. Some Characteristics of the Modulus of Passive Resistance of Soil: A Study in Similitude. *Proc. Highway Research Board*, 37, 576.

[43] Young, W.C.1989. *Roark's Formulas for Stress & Strain.* 6th ed. McGraw-Hill, New York.