# Concrete Pressure Pipe

MANUAL OF WATER SUPPLY PRACTICES — M9, Third Edition

**AWWA MANUAL M9**

*Third Edition*

American Water Works Association

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# Concrete Pressure Pipe



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*Chapter* 1

# Purpose and Scope

The use of concrete pressure pipe for conveying water and other liquids under pressure has dramatically increased in recent years. Its rugged construction and the natural corrosion resistance provided by embedment of the ferrous components in concrete or cement mortar offer the design engineer solutions to a wide range of structural and environmental problems (Figure 1-1).

The manufacture of four basic types of concrete pressure pipe are covered by the following American Water Works Association (AWWA) standards:

ANSI/AWWA C300 Standard for Reinforced Concrete Pressure Pipe, Steel-Cylinder Type

ANSI/AWWA C301 Standard for Prestressed Concrete Pressure Pipe, Steel-Cylinder Type

Figure 1-1 A typical installation in rugged terrain

ANSI/AWWA C302 Standard for Reinforced Concrete Pressure Pipe, Noncylinder Type

ANSI/AWWA C303 Standard for Concrete Pressure Pipe, Bar-Wrapped, Steel-Cylinder Type

Preparing pipeline project plans and specifications, however, requires many pipe-related decisions that are not covered by the respective AWWA standards.

This manual provides supplemental information to assist engineers and designers in achieving optimum field performance of concrete pressure pipelines. Information and guidelines are provided covering hydraulics, surge pressure, external loads, bedding, and backfilling; designing reinforced concrete pressure pipe, fittings and appurtenances, thrust restraints, pipe on piers, and subaqueous installations; design considerations for corrosive environments; transportation of pipe; trench and tunnel installation; and other pertinent subjects.

The information in this manual is not intended to supersede, nor should be regarded as superseding, any portion of any AWWA standard.

NOTE: This manual uses US customary units of measurement. Metric equivalents have been added where deemed appropriate.

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*Chapter* **2**

# Description of Concrete Pressure Pipe

Several types of concrete pressure pipe are manufactured and used in North America. These include prestressed concrete cylinder pipe, reinforced concrete cylinder pipe, reinforced concrete noncylinder pipe, and concrete bar-wrapped cylinder pipe. Each type will be discussed in this chapter. Because of their construction, some types of concrete pressure pipe are made for a specific type of service condition; others are constructed for a broader range of service conditions. The general description of these pipes is based on whether or not the pipe has a full-length steel cylinder and whether it is conventionally reinforced with deformed bar, wire, or smooth bar, or prestressed with high-strength wire (Table 2-1). It should be noted that not all concrete pressure pipe manufacturers make all of these types of pipe.

## PRESTRESSED CONCRETE CYLINDER PIPE (ANSI/AWWA C301-TYPE PIPE)

Prestressed concrete cylinder pipe (Figure 2-1) has been manufactured in the United States since 1942 and is the most widely used type of concrete pressure pipe. Applications for this product include transmission mains, distribution feeder mains, water intake and discharge lines, pressure siphons, low-head penstocks, industrial pressure lines (including power plant cooling-water lines), sewer force mains, gravity sewer lines, subaqueous lines (into both freshwater and salt water), and spillway conduits. This high-strength rigid pipe has also been extensively used for conduits under small dams, deep highway fills, and other high-earth-cover projects.

### Types of Construction

Prestressed concrete cylinder pipe has the following two general types of construction: (1) a steel cylinder lined with a concrete core, or (2) a steel cylinder embedded in a concrete core (Figure 2-1). In either type of construction, manufacturing begins with a full-length welded steel cylinder (Figure 2-2). Joint rings are attached to each end, and the pipe is hydrostatically tested to ensure watertightness. A concrete core with a minimum thickness of one-sixteenth times the pipe diameter is placed either

Table 2-1 General description of concrete pressure pipe

| Type of Pipe | ANSI/AWWA Standard | Steel Cylinder | Noncylinder | Mild Reinforcing Steel | Prestressed Wire | Design Basis* |
|---|---|---|---|---|---|---|
| Reinforced concrete cylinder pipe | C300 | X | | X | | Rigid |
| Prestressed concrete cylinder pipe | C301 | X | | | X | Rigid |
| Reinforced concrete non-cylinder pipe | C302 | | X | X | | Rigid |
| Concrete bar-wrapped cylinder pipe | C303 | X | | X | | Semirigid |

* The terms *rigid* and *semirigid* in this table and in the remainder of this manual are intended to differentiate between two design theories. Rigid pipe does not depend on the passive resistance of the soil adjacent to the pipe for support of vertical loads; semirigid pipe does require this passive soil resistance. The terms *rigid* and *semirigid* as used here should not be confused with the definitions stated by Marston (1930).

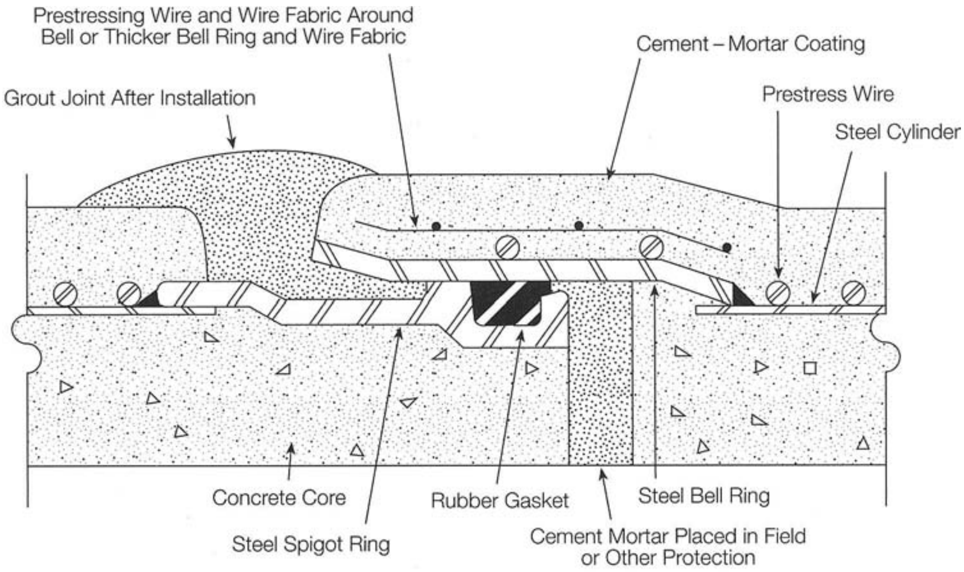
by the centrifugal process, radial compaction, or vertical casting. After the core is cured, the pipe is helically wrapped with high-strength, hard-drawn wire using a stress of 75 percent of the minimum specified tensile strength. The wrapping stress ranges between 150,000 and 189,000 psi (1,034 and 1,303 MPa) depending on the wire size and class. The wire spacing is accurately controlled to produce a predetermined residual compression in the concrete core. The wire is embedded in a thick cement slurry and coated with a dense mortar that has a high cement content.

## Size Range

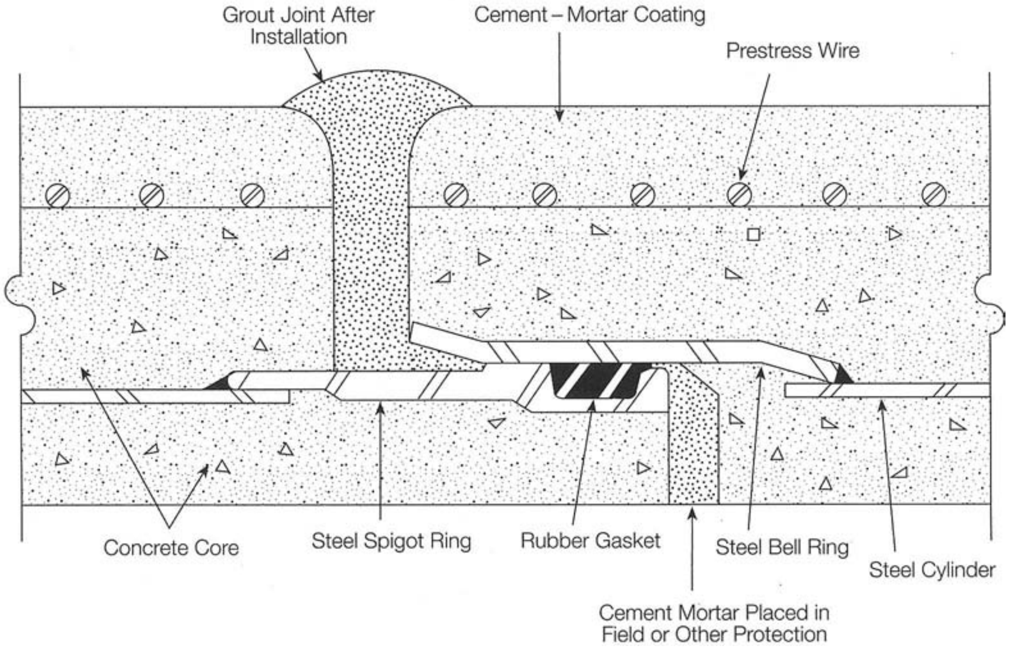
ANSI/AWWA C301 covers prestressed concrete cylinder pipe 16 in. (410 mm) in inside diameter and larger. Lined cylinder pipe is commonly available with inside diameters ranging from 16 through 48 in. (410 through 1,220 mm). Sizes up to 60 in. (1,520 mm) are available from some manufacturers. Embedded cylinder pipe larger than 250 in. (6,350 mm) in diameter has been manufactured and is commonly available with inside diameters 48 in. (1,220 mm) and larger. Lengths are generally 16 to 24 ft (4.9 to 7.3 m), although longer units can be furnished. Shipping requirements usually restrict the weight and length of pipe manufactured.

## Design Basis

Prestressed concrete cylinder pipe has been designed for operating pressures greater than 400 psi (2,758 kPa) and earth covers in excess of 100 ft (30 m) (Figure 2-3). The design of this pipe is covered in ANSI/AWWA C304, Standard for Design of Prestressed Concrete Cylinder Pipe. The design method is based on combined loading conditions — the most critical type of loading for rigid pipe — and includes surge pressure and live loads.




**A. Lined Cylinder Pipe**




**B. Embedded Cylinder Pipe**

Figure 2-1 Prestressed concrete cylinder pipe (ANS /AWWA C301-type pipe)

## Joints

The standard joint configuration for prestressed concrete cylinder pipe is sealed with a round rubber gasket and steel joint rings, as shown in Figure 2-1. The spigot ring is a hot-rolled steel shape containing a rectangular recess that holds a continuous solid ring gasket of circular cross section. The gasket is compressed by the cylindrical

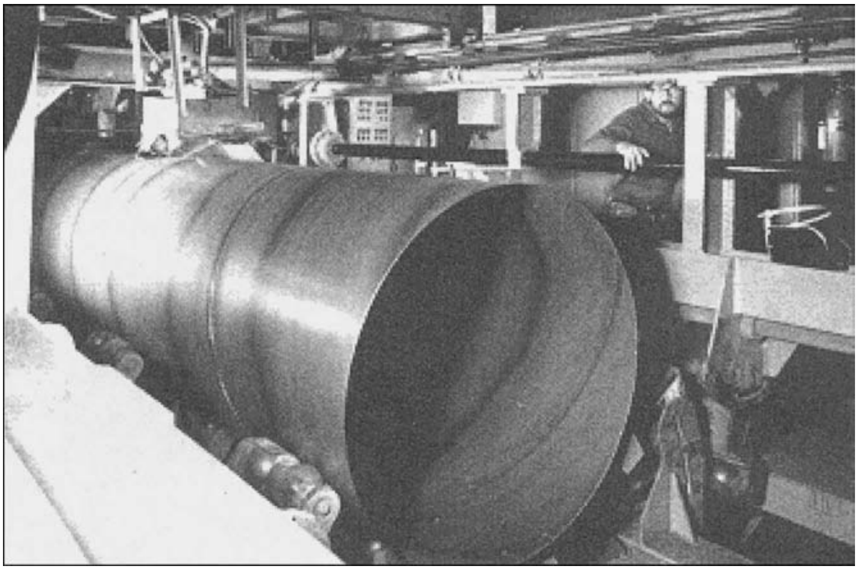


Figure 2-2 Fabrication of a steel cylinder on a "helical" machine

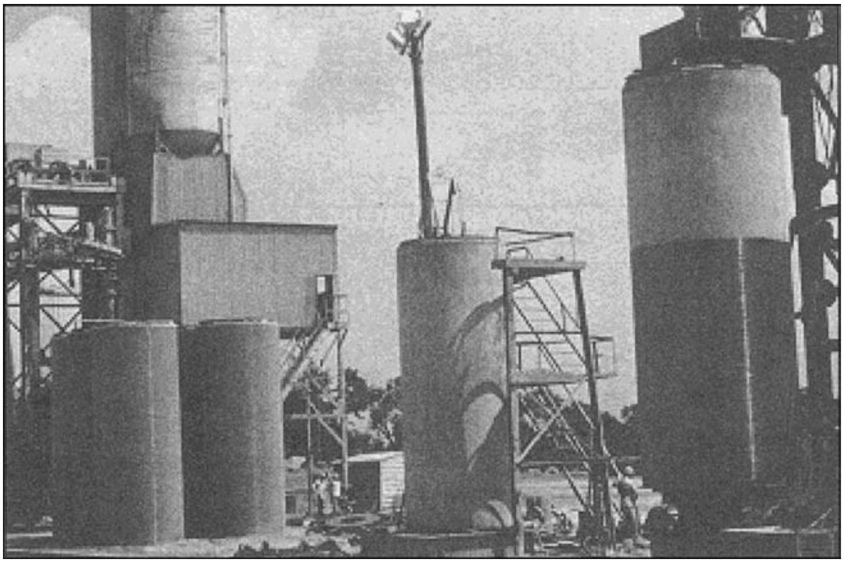


Figure 2-3 Prestressing concrete embedded cylinder pipe

portion of the steel bell ring when the spigot is pushed into the bell. Both joint rings are sized to close tolerances on a hydraulic press by expanding them beyond their elastic limit. The gasket diameter and volume are closely controlled to ensure a reliable high-pressure seal. After field assembly, the exterior joint recess is usually grouted to protect the steel joint rings, but other methods of protecting the joint rings are available. The internal joint may or may not be pointed with stiff mortar, depending on the type of liquid the pipe will carry and the type of protective coating applied to the joint rings during manufacture (Figure 2-4).

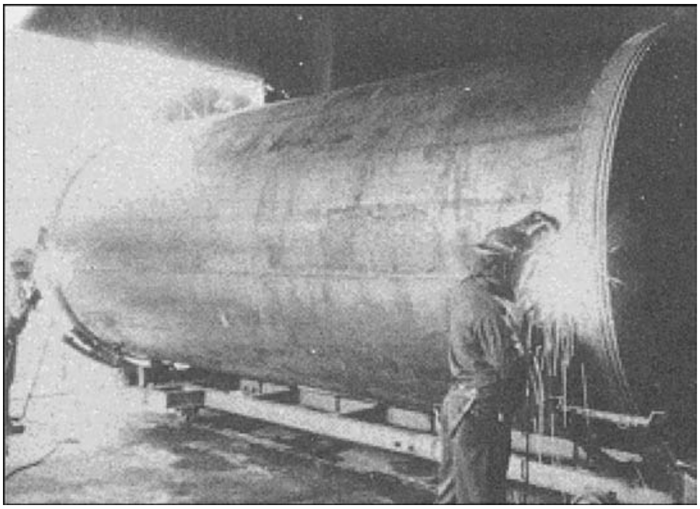


Figure 2-4 Welding joint rings to cylinder

## REINFORCED CONCRETE CYLINDER PIPE (ANSI/AWWA C300-TYPE PIPE)

Prior to the introduction of prestressed concrete cylinder pipe (ANSI/AWWA C301-type pipe) in the early 1940s, most of the concrete pressure pipe in the United States was reinforced concrete cylinder pipe. The construction of this pipe is similar to prestressed concrete cylinder pipe except that mild steel reinforcement is cast into the wall of the pipe instead of prestressing with high-strength wire. The minimum wall thickness is one-twelfth the inside diameter.

### Manufacture

The manufacture of reinforced concrete cylinder pipe (Figure 2-5) begins with a hydrostatically tested steel cylinder and attached steel joint rings. The cylinder assembly and one or more reinforcing cages are positioned between inside and outside forms, and the concrete is placed by vertical casting.

### Size Range

Reinforced concrete cylinder pipe is manufactured in diameters of 30 to 144 in. (760 to 3,660 mm), with larger sizes limited only by the restrictions of transportation to the job site. Standard lengths are 12 to 24 ft (3.7 to 7.3 m).

### Design Basis

Design of reinforced concrete cylinder pipe is covered in chapter 7 of this manual. The design procedure addresses external loads and internal pressures individually and in combinations. ANSI/AWWA C300 limits the reinforcing steel furnished in the cage(s) to no less than 40 percent of the total reinforcing steel in the pipe. The maximum loads and pressures for this type of pipe depend on the pipe diameter, wall thickness,

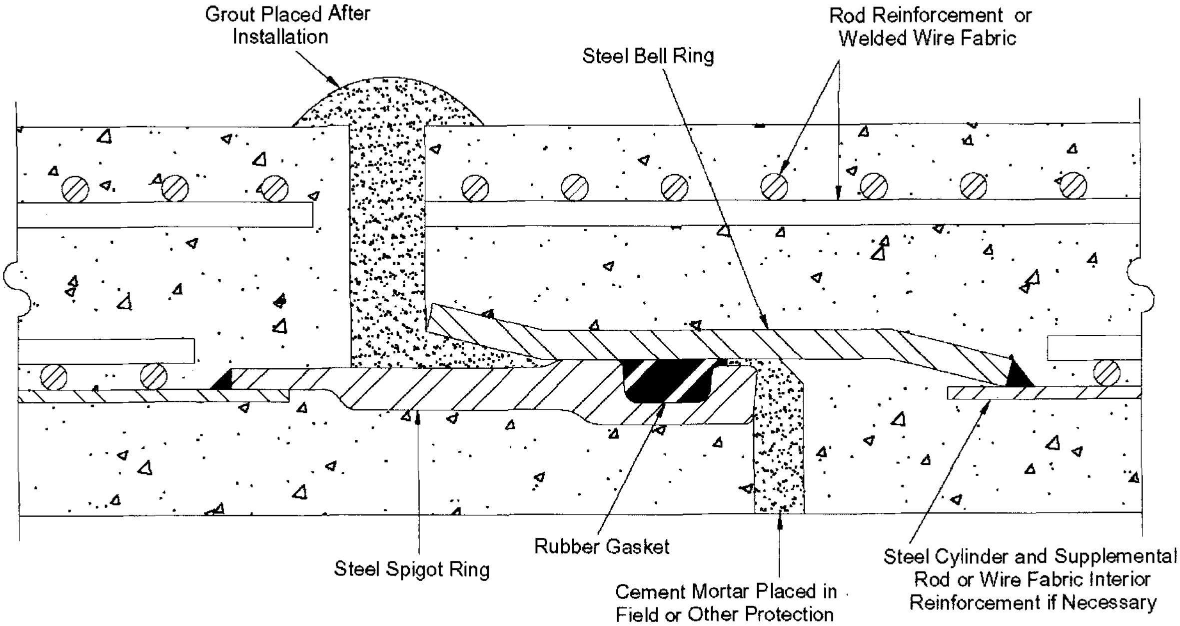



Figure 2-5 Reinforced concrete cylinder pipe (ANSI/AWWA C300-type pipe)

and strength limitations of the concrete and steel. The user should be aware that this type of pipe can be designed for high internal pressure, but is limited in external load capacity.

## Joints

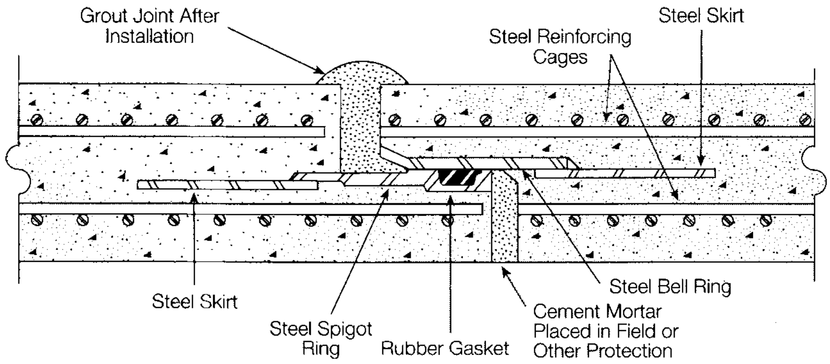
The standard joint configuration for reinforced concrete cylinder pipe, as shown in Figure 2-5, is identical to the joint for prestressed concrete cylinder pipe and consists of steel spigot and bell rings and a round rubber gasket. As with prestressed concrete cylinder pipe (ANSI/AWWA C301-type pipe), the external joint recess is normally grouted and the internal joint space may or may not be pointed with mortar, depending on the type of liquid the pipe will carry and the type of protective coating applied to the joint rings during manufacture.

# REINFORCED CONCRETE NONCYLINDER PIPE (ANSI/AWWA C302-TYPE PIPE)

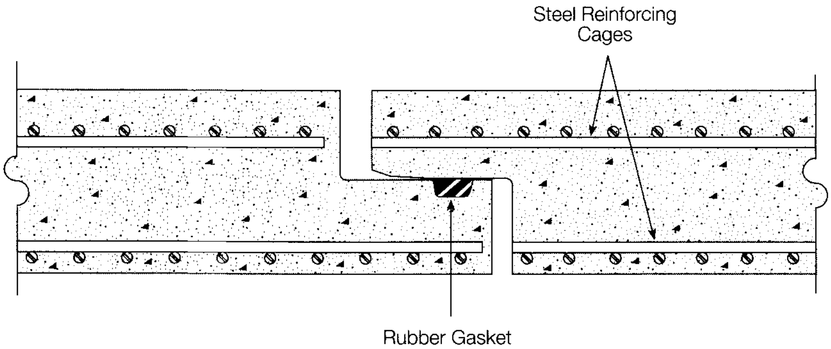
Reinforced concrete noncylinder pipe (Figure 2-6) is suitable for working pressures up to 55 psi (379 kPa). Typical applications are low-pressure transmission lines used for irrigation, industrial or domestic raw water supply and discharge lines, sanitary and storm sewers, and drainage culverts.

## Size Range and Design Basis

Reinforced noncylinder pipe is commonly furnished in diameters of 12 to 144 in. (300 to 3,660 mm), but larger diameters can be furnished if shipping limitations permit. Standard lengths are 8 to 24 ft (2.4 to 7.3 m), with ANSI/AWWA C302 limiting the maximum length that can be furnished for each pipe size. The wall thickness is a




**A. ANSI/AWWA C302-Type Pipe With Steel Joint Rings**




**B. ANSI/AWWA C302-Type Pipe With Concrete Joint Rings**

Figure 2-6 Reinforced concrete noncylinder pipe (ANSI/AWWA C302-type pipe)

minimum of one-twelfth the inside diameter. The design of reinforced concrete noncylinder pipe is covered in chapter 7 of this manual. The maximum working pressure is limited by ANSI/AWWA C302 to 55 psi (379 kPa).

## Joints

The standard joints for reinforced concrete noncylinder pipe are illustrated in Figure 2-6. The joint details are conceptual only and may vary based on design conditions and requirements. As with both reinforced and prestressed concrete cylinder pipe, these joints incorporate a continuous, round, solid rubber ring gasket placed in a spigot groove and compressed by the bell surface. However, the bell and spigot ends of reinforced concrete noncylinder pipe may be steel joint rings or formed concrete surfaces. It is recommended that the pressure sealing capacity of gasketed concrete joints be verified by testing the joint. It is also recommended that the structural capacity of concrete joints to withstand the installation and field loads be verified. If steel joint rings are used, as shown in Figure 2-6A, the grouting procedures are the same as described previously for prestressed concrete cylinder pipe and reinforced concrete cylinder pipe (ANSI/AWWA C301- and C300-type pipe, respectively). Concrete joints, as shown in Figure 2-6B, are normally not grouted.

### Manufacture

Manufacture of reinforced concrete noncylinder pipe begins with the fabrication of one or more reinforcing cages (depending on pipe size and wall thickness) and the fabrication of steel joint rings if that type of joint is used. The cages may be fabricated of smooth or deformed reinforcing bars, wire, or wire fabric. When concrete is to be placed by the vertical casting method, cages and joint rings are positioned between inner and outer steel forms. When concrete is to be placed by the centrifugal process, cages are positioned with respect to the outer form, and the entire assembly is rotated at high speed while the concrete is placed.

## CONCRETE BAR-WRAPPED CYLINDER PIPE (ANSI/AWWA C303-TYPE PIPE)

Concrete bar-wrapped cylinder pipe (Figure 2-7) is manufactured in Canada and in the western and southwestern areas of the United States. It is normally available in diameters of 10 to 72 in. (250 to 1,830 mm), although larger diameters can be manufactured. Standard lengths are generally 24 to 40 ft (7.3 to 12.2 m). Typical applications are cross-country transmission mains, distribution feeder mains, sewer force mains, water intake and discharge lines, and plant piping.

### Manufacture

Manufacture of concrete bar-wrapped cylinder pipe begins with a fabricated steel cylinder with joint rings, which is hydrostatically tested (Figure 2-8). A cement–mortar lining is placed by the centrifugal process inside the cylinder. The nominal lining thickness is ½ in. (13 mm) for sizes up to and including 16 in. (410 mm), and ¾ in. (19 mm) for larger sizes. After the lining is cured, the cylinder is wrapped with a smooth, hot-rolled steel bar, using moderate tension in the bar. The size and spacing of the bar, as well as the thickness of the steel cylinder, are proportioned to provide the required pipe strength. The cylinder and bar wrapping are covered with a cement slurry and a dense mortar coating that is rich in cement.

### Joints

The standard joint configuration shown in Figure 2-7 includes steel joint rings and a continuous, round, solid-rubber ring gasket. The exterior joint recess is normally grouted, and the internal joint space may or may not be pointed with mortar, depending on the type of liquid the pipe will carry and the type of protective coating applied to the joint rings during manufacture.

### Design Basis

The design procedure for concrete bar-wrapped cylinder pipe is presented in chapter 7 of this manual. This procedure is based on flexible pipe theory in which internal pressure and external load are considered separately but not in combination. Bar-wrapped cylinder pipe has been designed for operating pressures exceeding 400 psi (2,758 kPa). Several pipelines in Arizona and Colombia (South America) have been designed for pressures greater than 400 psi (2,758 kPa). Because the theory of flexible pipe design for earth loads is based on the passive soil pressure adjacent to the sides of the pipe, the design must be closely coordinated with the installation conditions.

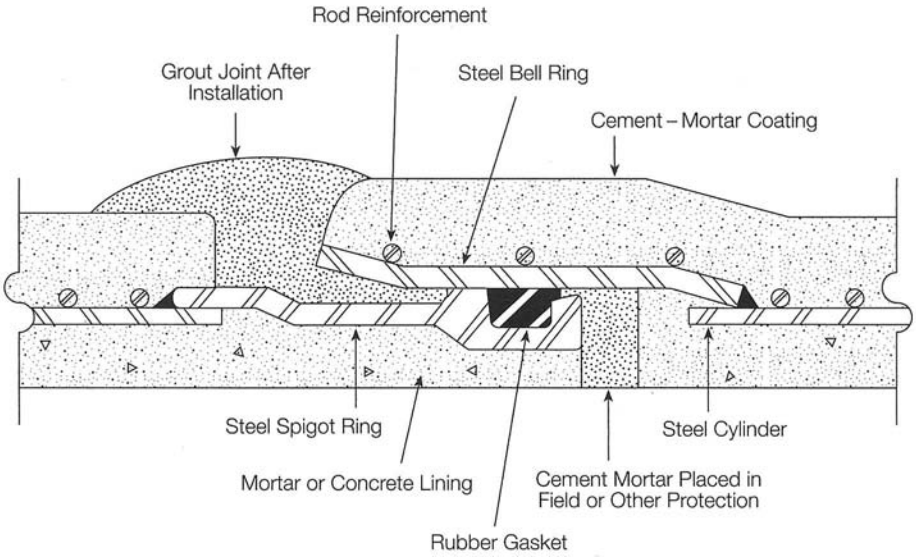



Figure 2-7 Concrete bar-wrapped cylinder pipe (ANSI/AWWA C303-type pipe)

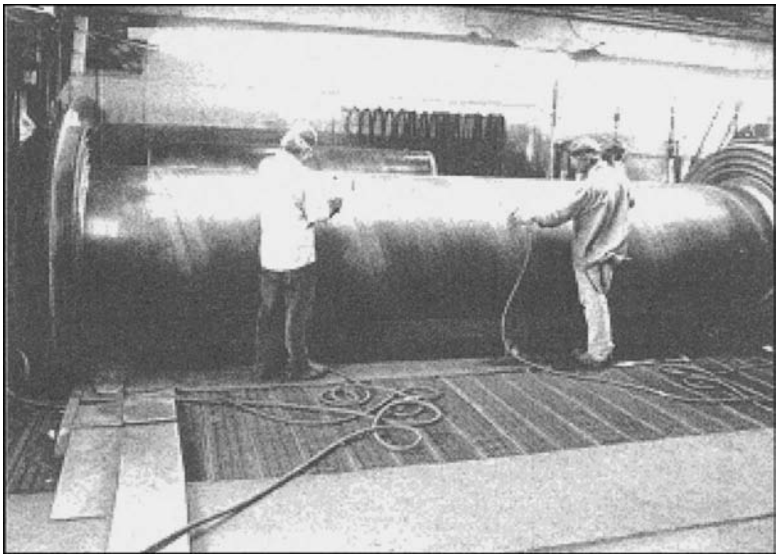
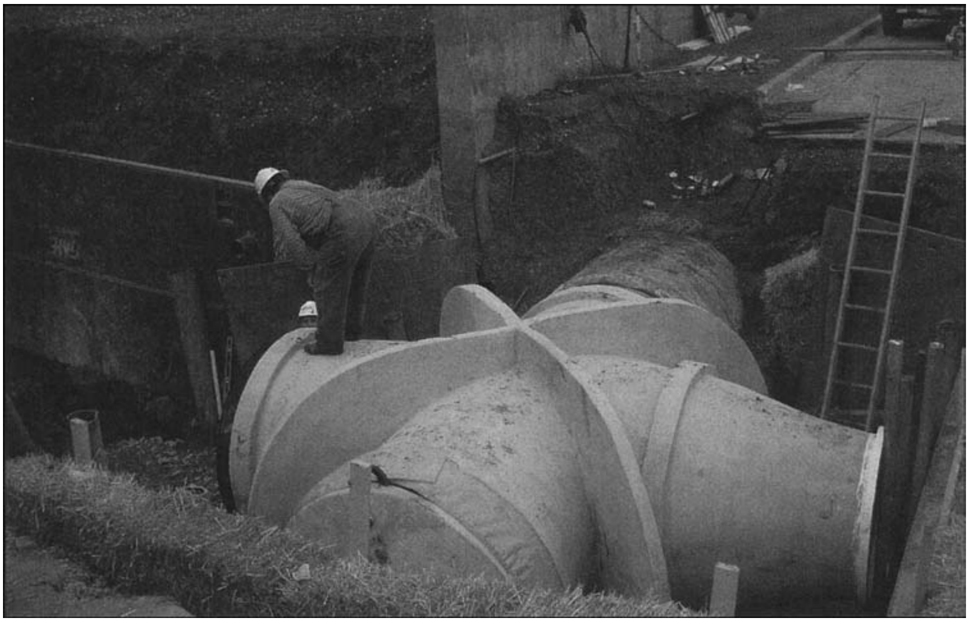


Figure 2-8 Steel cylinder being hydrostatically tested on a mandrel-type tester

## FITTINGS AND SPECIAL PIPE

All concrete pressure pipe manufacturers make a wide variety of fittings and special pipe, including bends (elbows), tees, wyes, crosses, manifolds, reducers, adapters, wall sleeves, closures, bulkheads, bevel pipe, and service outlets (Figure 2-9). Adapters and outlets are available to connect to all types of joints, including flanged, threaded (standard or Mueller), plain end (for mechanical coupling or field welding to steel



*Source:* Hanson Pressure Pipe

Figure 2-9 Installation of a large-diameter wye with reducer attached

pipe), iron pipe (bell or spigot for leaded joints), Victaulic, and various mechanical joints. Chapter 8 of this manual addresses the typical types of fittings that are manufactered and the basis of fittings design.

## Special Pipe

A special pipe is usually defined as a pipe in which the basic construction is the same as the standard pipe (prestressed cylinder, reinforced cylinder or noncylinder, or bar-wrapped cylinder), but contains some modification, such as an outlet or a beveled joint ring. Many types of outlets and beveled pipe can be furnished more economically in a special pipe than by using a fitting. Beveled pipe is manufactured with a joint ring that is deflected at a slight angle (up to 5° maximum). An individual beveled pipe or a series of beveled pipe can be used to deflect a line vertically or horizontally to avoid obstructions in the right-of-way, or a series of beveled pipe can be used to form a long sweeping curve to eliminate the need for a bend-and-thrust restraint. Most manufacturers make beveled pipe with more than one angle of deflection, which, together with standard lengths, provide many layout possibilities for offsets and curves without the use of bends. Beveled adapters (sections of beveled pipe with laying lengths generally less than 1 ft [305 mm]) are stocked by most manufacturers for contractors to use for deflections around unanticipated obstacles.

# REFERENCE

Marston, A. 1930. The Theory of External Loads on Closed Conduits in the Light of Latest Experiments. Bulletin 96, Iowa State Engineering Experiment Station. Iowa State University.

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Chapter 3

# Hydraulics

This chapter provides formulas and guidelines to aid in the hydraulic design of concrete pressure pipe. The three formulas typically used to determine pipeline capacity are presented and compared. Factors that contribute to the decrease with age of pipeline carrying capacity are also described. The methods used to determine minor and total head losses are presented, as well as the methodology for determining an economical pipe size. Throughout this chapter, discussions will be limited to combinations of pipe sizes and flows that provide a range of velocities and diameters most commonly used with concrete pressure pipe.

## FLOW FORMULAS

The formulas commonly used to determine the capacity of pipelines are the Hazen–Williams formula, the Darcy–Weisbach formula, and the Manning formula. Many other flow formulas may be found in technical literature, but these three are most frequently used. It is impossible to say that any one of these formulas is superior to the others for all pipe under all circumstances, and the reader is cautioned not to expect identical answers to a problem from all three formulas. Judgment must be used when selecting a flow formula and the roughness coefficient for a particular hydraulics problem.

### The Hazen–Williams Formula

The empirical Hazen–Williams formula is probably the most commonly used flow formula in the waterworks industry. The basic form of the equation is

$$V = 1.318 C_h R^{0.63} S^{0.54} \qquad \text{(Eq 3-1)}$$

Where:

- $V$ = velocity, ft/sec
- $C_h$ = Hazen–Williams roughness coefficient
- $R$ = hydraulic radius, ft, which is the cross-sectional area of the pipe divided by the wetted perimeter, i.e., for circular pipe flowing full, the internal diameter in feet divided by 4
- $S$ = slope of the hydraulic grade line, ft/ft calculated as $h_L/L$, where $h_L$ equals head loss in feet occurring in a pipe of length $L$ in ft

For circular conduits flowing full, the equation becomes

$$V = 0.550C_h d^{0.63}(h_L/L)^{0.54} \quad \text{(Eq 3-2)}$$

Where:

$V$ = velocity, ft/sec
$C_h$ = Hazen–Williams roughness coefficient
$d$ = inside diameter of the pipe, ft
$h_L$ = head loss, ft
$L$ = pipe length, ft

Table 3-1 lists the various forms of the Hazen–Williams formula. Figures 3-1 and 3-2 show the hydraulic grade line, head losses, and static grade line for gravity and pumped flow systems.

A detailed investigation of the available flow test data for concrete pipe was performed by Swanson and Reed (1963), who concluded that the Hazen–Williams formula most closely matches the test results for the range of velocities normally encountered in water transmission. A statistical analysis of Swanson and Reed's test data led to the development of the following equation for determining $C_h$:

$$C_h = 139.3 + 2.028d \quad \text{(Eq 3-3)}$$

Where:

$C_h$ = Hazen–Williams roughness coefficient
$d$ = inside diameter of pipe, ft

Table 3-2 compares theoretical Hazen–Williams $C_h$ values using Equation 3-3 to tested values for 67 tests reported by Swanson and Reed.

Equation 3-3 can be used to calculate a $C_h$ value for any size pipe; however, for design purposes the following conservative values are suggested:

| Diameter, *in.* | $C_h$ Value |
|---|---|
| 16 to 48 | 140 |
| 54 to 108 | 145 |
| 114 and larger | 150 |

Table 3-1 Various forms of the Hazen–Williams formula

| In Terms of: | Velocity, *fps* | Flow Rate, *cfs* |
|---|---|---|
| General equation | $V = 0.550C_h d^{0.63}(h_L/L)^{0.54}$ | $Q = 0.432C_h d^{2.63}(h_L/L)^{0.54}$ |
| Head loss, *ft* | $h_L = 3.021(L/d^{1.167})(V/C_h)^{1.852}$ | $h_L = 4.726(L/d^{4.870})(Q/C_h)^{1.852}$ |
| Pipe diameter, *ft* | $d = 2.580(V/C_h)^{1.587}(L/h_L)^{0.857}$ | $d = 1.376(Q/C_h)^{0.380}(L/h_L)^{0.205}$ |
| Pipe length, *ft* | $L = 0.331h_L d^{1.167}(C_h/V)^{1.852}$ | $L = 0.212h_L d^{4.870}(C_h/Q)^{1.852}$ |
| Roughness coefficient | $C_h = 1.817(V/d^{0.63})(L/h_L)^{0.54}$ | $C_h = 2.313(Q/d^{2.63})(L/h_L)^{0.54}$ |

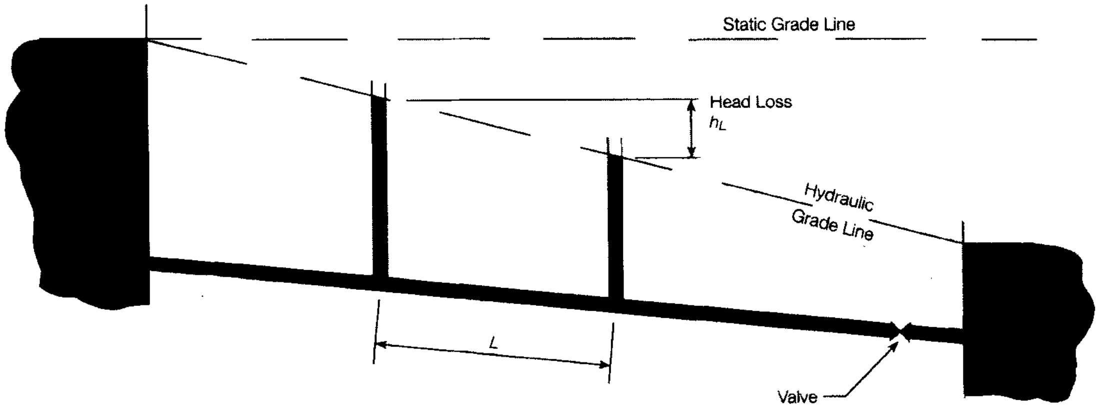



Figure 3-1 Hydraulic profile for a gravity flow system

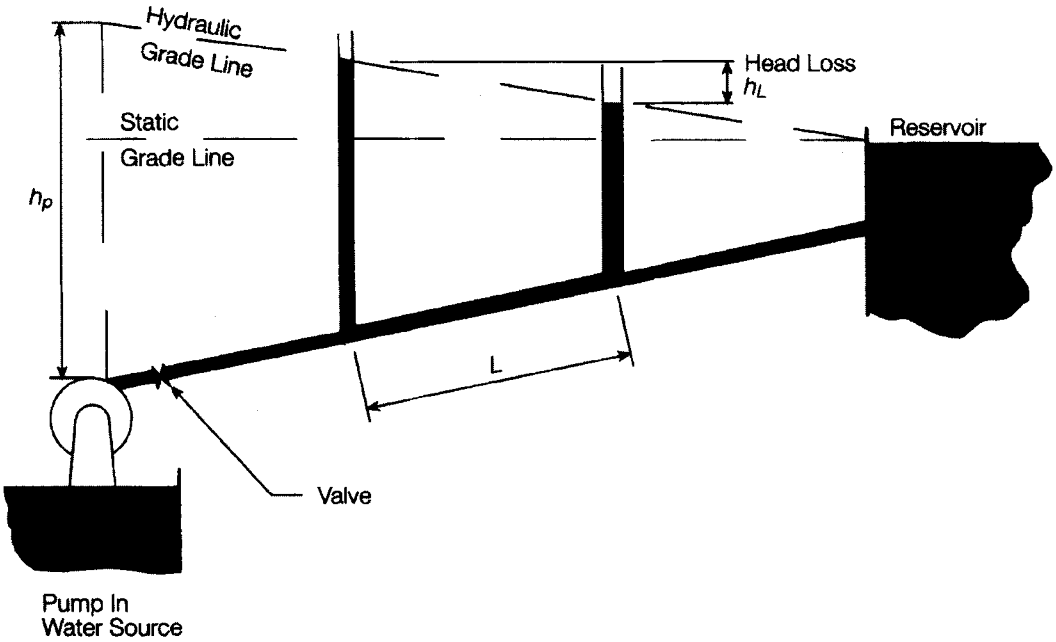



Figure 3-2 Hydraulic profile for a pumped flow system

Table 3-2 Comparison of theoretical Hazen–Williams $C_h$ values to tested $C_h$ values*

| Sample Number | Diameter *in.* | $C_h$ Measured | $C_h$ Theoretical | Percent of Theoretical |
|---|---|---|---|---|
| 1 | 24.0 | 145.0 | 143.4 | 101.1 |
| 2 | 30.0 | 147.0 | 144.4 | 101.8 |
| 3 | 30.0 | 145.0 | 144.4 | 100.4 |
| 4 | 31.5 | 147.0 | 144.6 | 101.6 |
| 5 | 31.8 | 152.0 | 144.7 | 105.1 |
| 6 | 36.0 | 143.0 | 145.4 | 98.4 |
| 7 | 36.0 | 150.0 | 145.4 | 103.2 |
| 8 | 36.0 | 141.5 | 145.4 | 97.3 |
| 9 | 36.0 | 142.5 | 145.4 | 98.0 |
| 10 | 36.0 | 150.0 | 145.4 | 103.2 |
| 11 | 39.0 | 147.0 | 145.9 | 100.8 |
| 12 | 42.0 | 147.5 | 146.4 | 100.8 |
| 13 | 42.0 | 142.0 | 146.4 | 97.0 |
| 14 | 42.0 | 149.0 | 146.4 | 101.8 |
| 15 | 42.0 | 149.0 | 146.4 | 101.8 |
| 16 | 42.0 | 142.0 | 146.4 | 97.0 |
| 17 | 42.0 | 147.0 | 146.4 | 100.4 |
| 18 | 46.0 | 144.0 | 147.1 | 97.9 |
| 19 | 48.0 | 146.0 | 147.4 | 99.0 |
| 20 | 48.0 | 150.5 | 147.4 | 102.1 |
| 21 | 48.0 | 144.0 | 147.4 | 97.7 |
| 22 | 51.0 | 142.0 | 147.9 | 96.0 |
| 23 | 54.0 | 152.0 | 148.4 | 102.4 |
| 24 | 54.0 | 141.0 | 148.4 | 95.0 |
| 25 | 54.0 | 140.5 | 148.4 | 94.7 |
| 26 | 60.0 | 145.5 | 149.4 | 97.4 |
| 27 | 60.0 | 152.5 | 149.4 | 102.0 |
| 28 | 46.0 | 148.5 | 147.1 | 101.0 |
| 29 | 54.0 | 150.0 | 148.4 | 101.1 |
| 30 | 36.0 | 138.0 | 145.4 | 94.9 |
| 31 | 36.0 | 137.0 | 145.4 | 94.2 |
| 32 | 36.0 | 142.0 | 145.4 | 97.7 |
| 33 | 48.0 | 149.0 | 147.4 | 101.1 |
| 34 | 54.0 | 151.5 | 148.4 | 102.1 |
| 35 | 54.0 | 145.0 | 148.4 | 97.7 |
| 36 | 54.0 | 146.0 | 148.4 | 98.4 |
| 37 | 54.0 | 147.5 | 148.4 | 99.4 |
| 38 | 60.0 | 147.0 | 149.4 | 98.4 |
| 39 | 60.0 | 154.0 | 149.4 | 103.1 |
| 40 | 60.0 | 156.0 | 149.4 | 104.4 |
| 41 | 60.0 | 143.5 | 149.4 | 96.0 |
| 42 | 66.0 | 142.0 | 150.5 | 94.4 |
| 43 | 48.0 | 156.5 | 147.4 | 106.2 |
| 44 | 48.0 | 152.0 | 147.4 | 103.1 |
| 45 | 48.0 | 149.5 | 147.4 | 101.4 |
| 46 | 48.0 | 153.5 | 147.4 | 104.1 |
| 47 | 54.0 | 154.0 | 148.4 | 103.8 |
| 48 | 54.0 | 155.5 | 148.4 | 104.8 |
| 49 | 54.0 | 151.0 | 148.4 | 101.7 |
| 50 | 54.0 | 152.0 | 148.4 | 102.4 |

*Table continued next page.*

Table 3-2 Comparison of theoretical Hazen–Williams $C_h$ values to tested $C_h$ values* (continued)

| Sample Number | Diameter *in.* | $C_h$ Measured | $C_h$ Theoretical | Percent of Theoretical |
|---|---|---|---|---|
| 51 | 54.0 | 143.0 | 148.4 | 96.3 |
| 52 | 72.0 | 154.0 | 151.5 | 101.7 |
| 53 | 12.0 | 145.0 | 141.3 | 102.6 |
| 54 | 18.0 | 147.0 | 142.3 | 103.3 |
| 55 | 20.0 | 134.0 | 142.7 | 93.9 |
| 56 | 24.0 | 143.5 | 143.4 | 100.1 |
| 57 | 30.0 | 141.0 | 144.4 | 97.7 |
| 58 | 30.0 | 143.0 | 144.4 | 99.1 |
| 59 | 33.0 | 147.0 | 144.9 | 101.5 |
| 60 | 33.0 | 146.0 | 144.9 | 100.8 |
| 61 | 33.0 | 146.0 | 144.9 | 100.8 |
| 62 | 36.0 | 134.0 | 145.4 | 92.2 |
| 63 | 31.4 | 138.0 | 144.6 | 95.4 |
| 64 | 36.0 | 147.0 | 145.4 | 101.1 |
| 65 | 36.0 | 150.0 | 145.4 | 103.2 |
| 66 | 41.9 | 150.0 | 146.4 | 102.5 |
| 67 | 42.0 | 150.0 | 146.4 | 102.5 |

* The theoretical values were calculated using Eq 3-3 and the test values are from Swanson and Reed (1963).

These values are applicable to concrete pipelines in which the fitting losses are a minor part of the total loss and to pipelines free from deposits or organic growths that can materially affect the pipe's carrying capacity.

## The Darcy–Weisbach Formula

An alternative to the Hazen–Williams formula is the Darcy–Weisbach formula, which is

$$h_L = \frac{fLV^2}{d2g} \qquad \text{(Eq 3-4)}$$

Where:

$h_L$ = head loss, in feet of water
$f$ = Darcy friction factor
$L$ = pipeline length, ft
$V$ = velocity, ft/sec
$d$ = inside diameter of the pipe, ft
$g$ = gravitational constant, 32.2 ft/sec$^2$

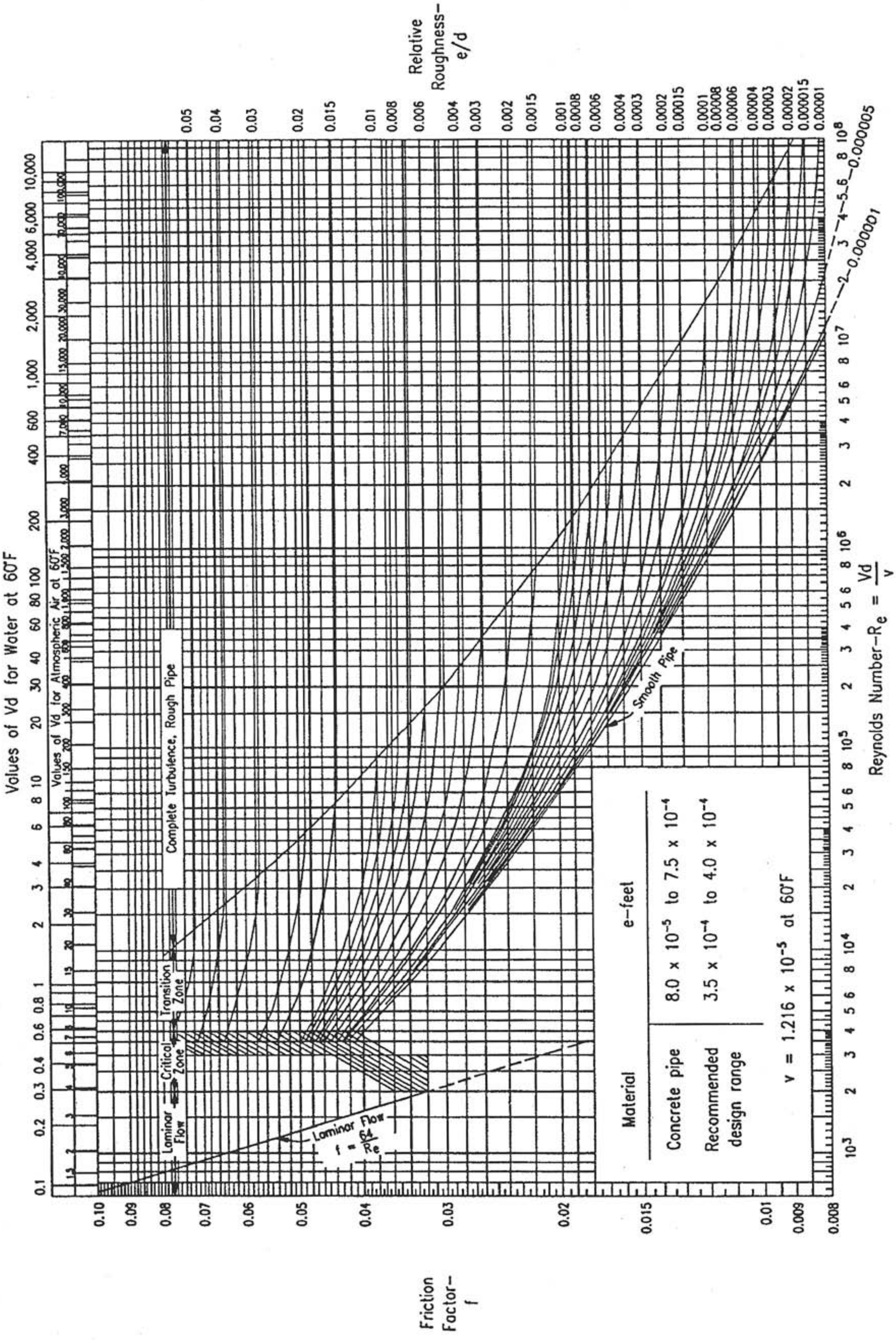



Figure 3-3 The Moody diagram for friction in pipe

The Darcy friction factor $(f)$ can be determined from the Moody diagram, presented in Figure 3-3. To use this diagram, the Reynolds number $(R_e)$ and the relative roughness $(e/d)$ must first be calculated. Both terms are defined in the paragraphs that follow.

The Reynolds number is a function of the flow in the pipe and may be calculated as

$$R_e = \frac{dV}{\nu} \tag{Eq 3-5}$$

Where:

$R_e$ = Reynolds number
$d$ = inside diameter of the pipe, ft
$V$ = velocity, ft/sec
$\nu$ = kinematic viscosity of the fluid, $ft^2$/sec

The kinematic viscosity of water at various temperatures from freezing to boiling is presented in Table 3-3.

The relative roughness $(e/d)$ of a pipe is a function of the absolute roughness $(e)$ of the interior surface of the pipe and the pipe diameter $(d)$. Values of the absolute roughness $(e)$ for concrete pipe range from $8.0 \times 10^{-5}$ to $7.5 \times 10^{-4}$, and the recommended design range is $3.5 \times 10^{-4}$ to $4.0 \times 10^{-4}$. Once the Reynolds number and the relative roughness are determined, the Moody diagram (Figure 3-3) can be used to determine values of the Darcy friction factor $(f)$, which may then be used to solve the Darcy–Weisbach formula.

If an analytical solution for the Darcy friction factor $(f)$ is preferred, it may be obtained by iteration from the Colebrook–White equation:

$$\frac{1}{\sqrt{f}} = -2\log_{10}\left[\frac{e}{3.7d} + \frac{2.51}{R_e\sqrt{f}}\right] \tag{Eq 3-6}$$

Where:

$f$ = Darcy friction factor
$e$ = absolute roughness, ft
$d$ = inside diameter of the pipe, ft
$R_e$ = Reynolds number

Table 3-3 Kinematic viscosity of water

| Temperature, °$F$ | Kinematic Viscosity ($\nu$), $ft^2/s$ |
|---|---|
| 32 | 1.931 E-05 |
| 40 | 1.664 E-05 |
| 50 | 1.410 E-05 |
| 60 | 1.216 E-05 |
| 70 | 1.059 E-05 |
| 80 | 09.30 E-06 |
| 90 | 08.23 E-06 |
| 100 | 07.36 E-06 |
| 120 | 06.10 E-06 |
| 150 | 04.76 E-06 |
| 180 | 03.85 E-06 |
| 212 | 03.19 E-06 |

### The Manning Formula

The Manning formula is more commonly used to determine the flow in partially filled gravity lines; however, it can be used for fully developed flow in conduits. The basic form of the equation is

$$V = 1.486R^{0.67}S^{0.5}/n_M \qquad \text{(Eq 3-7)}$$

Where:

- $V$ = velocity, ft/sec
- $R$ = hydraulic radius, in feet, which is the cross-sectional area of the pipe divided by the wetted perimeter (e.g., for circular pipe flowing full, the internal diameter divided by 4)
- $S$ = slope of the hydraulic grade line, ft/ft
- $n_M$ = Manning roughness coefficient

For pressure flow in round conduit, this equation becomes

$$V = (0.590d^{0.67}/n_M)\,(h_L/L)^{0.5} \qquad \text{(Eq 3-8)}$$

Where:

- $V$ = velocity, ft/sec
- $n_M$ = Manning roughness coefficient
- $d$ = inside diameter of the pipe, ft
- $h_L$ = head loss, in feet, occurring in a pipe of length $L$, ft

For concrete pressure pipe, the value for the Manning roughness coefficient $n_M$ should be approximately 0.011 when the velocity is 3 ft/s and 0.010 when the velocity is 5 ft/s.

## EFFECTS OF AGING ON CARRYING CAPACITY

The carrying capacity of concrete pressure pipe is usually not affected by age. However, some aggressive waters can roughen the lining surface, and there are several organic growths and some types of deposits that reduce the carrying capacity of pipe of any material, including concrete pressure pipe. These conditions are often associated with raw water transmission mains and can generally be prevented by chemical pretreatment. If left unchecked, however, excessive growths or deposits can develop. When this occurs, chemical treatment may not be adequate to remove the deposits if they have achieved a substantial buildup. In these cases, the growths or deposits can be removed by scraping or pigging the line. Industry has not witnessed deterioration of the mortar or concrete lining as a result of growths or deposits.

## HEAD LOSSES

The total head loss in a pipeline is the sum of minor losses due to changes in flow geometry added to the head loss created due to the friction caused by flow through the pipe. The head loss caused by pipe friction may be determined from the formulas already presented. The calculation procedure for determining minor losses is presented in the following section. A method to easily include minor losses with frictional losses to simplify pipe selection calculations is also presented.

## Minor Losses

Minor losses in pipelines are caused by turbulence resulting from changes in flow geometry. Minor losses, which are generally expressed as a function of the velocity head, will occur at entrances, outlets, contractions, enlargements, bends, and other fittings. In long pipelines, the minor losses are usually small compared to losses from pipe friction and may be neglected; however, in shorter lines or plant piping, these minor losses may become significant. The formula for calculating minor losses is

$$h_L = C_L V^2/(2g) \qquad \text{(Eq 3-9)}$$

Where:

$h_L$ = head loss, ft
$C_L$ = a dimensionless coefficient
$V$ = velocity, ft/sec
$g$ = gravitational constant, 32.2 ft/sec$^2$

Figure 3-4 presents values of $C_L$ for common flow configurations.

## Equivalent Length Method

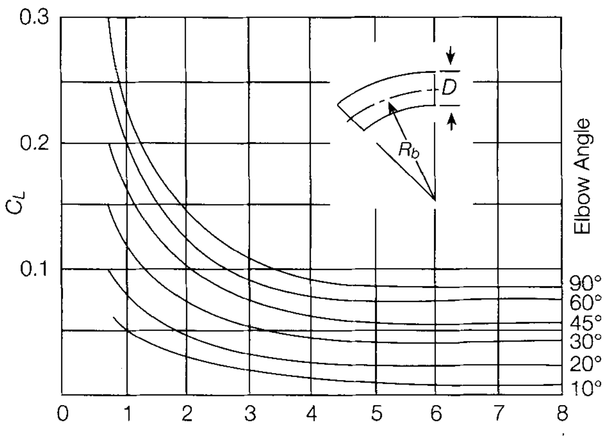
Each minor head loss in a piping system can be expressed in terms of an equivalent length of pipe $L_e$. This equivalent length of pipe is the number of feet of straight pipe that would have friction head loss equal to the minor losses created by the fitting. Table 3-4 lists the equivalent length formulas that correspond to the flow formulas presented earlier.

# DETERMINING AN ECONOMICAL PIPE DIAMETER

To determine the most economical pipe diameter, the first cost (purchase cost plus installation cost) must be compared to the pumping cost incurred during the design life of the pipe. While a greater diameter pipe will result in a greater first cost, the reduced head loss will lead to a lower pumping cost and provide capacity for subsequent increased flow. These costs should be determined for a number of pipe sizes and the results compared in order to select the pipe size that provides the lowest total cost during the life of the line.

Experience has shown the most cost-effective water pipe diameter will usually yield design flow velocities in the range of 3 to 6 ft/s (0.9 to 1.8 m/s). Therefore, it is generally best to restrict pipe size evaluations to diameters that will deliver the required volume of water at a velocity between 2 and 7 ft/s (0.6 and 2.1 m/s), although in some rare circumstances involving short pipeline length, it may be cost effective to use velocities approaching 10 ft/s (3.0 m/s). The inside diameter of pipe for various flow velocities can be determined from

$$d = (1.27Q/V)^{0.5} \qquad \text{(Eq 3-10)}$$




Loss Coefficient $C_L$ for Elbows

| Flow Configurations | | $C_L$ |
|---|---|---|
| Elbow | | See figure above |
| Reentrant Entrance | | 0.80 |
| Square–Edged Entrance | | 0.50 |
| Slightly Rounded Entrance | | 0.23 |
| Well–Rounded Entrance | | 0.04 |
| Flow Into Reservoir | | 1.00 |
| Reducer With $\alpha \leq 15°$ | $\alpha$ | 0.04 |
| Enlarger: | | |
| $\beta$ = 10° | $\beta$ | 0.20 |
| $\beta$ = 20° | | 0.40 |
| $\beta$ = 30° | | 0.70 |
| Tee, Through Run | | 0.60 |
| Tee, Through Side Outlet | | 1.80 |
| Tee Into Side Outlet | | 1.50 |
| 45° Wye | or | 1.30 |
| Gate Valve, Fully Open | | 0.20 |
| Swing Check Valve, Fully Open | | 2.50 |
| Butterfly Valve, Fully Open | | 0.25 |

**Figure 3-4** Approximate loss coefficients for commonly encountered flow configurations

Table 3-4 Equivalent length formulas

| Friction Formula | Equivalent Length Formula |
|---|---|
| Hazen–Williams | $L_e = 0.331 C_L \frac{V^{0.148} d^{1.167} C_h^{1.852}}{2g}$ |
| Darcy–Weisbach | $L_e = C_L(d/f)$ |
| Manning | $L_e = 0.348 C_L \frac{d^{1.333}}{2g(n_M)^2}$ |

NOTE: Definition of variables:

$L_e$ = length of pipe that would have a frictional head loss equal to the minor loss created by fitting, ft
$C_L$ = loss coefficient
$V$ = velocity, ft/sec
$d$ = inside diameter of the pipe, ft
$C_h$ = Hazen–Williams roughness coefficient
$g$ = gravitational constant, 32.2 ft/sec$^2$
$f$ = Darcy friction factor
$n_M$ = Manning roughness coefficient

Where:
$d$ = inside diameter of the pipe, ft
$Q$ = flow rate, ft$^3$/sec
$V$ = flow velocity, ft/sec

For each pipeline diameter to be evaluated, minor losses should be converted to equivalent lengths of pipe as outlined in the preceding section, and the friction head should be calculated from one of the flow equations presented earlier. As an example, the Hazen–Williams formula, in the following form, can be used to calculate total head loss for the sum of actual pipe lengths and equivalent lengths:

$$h_f = (4.726 L' / d^{4.870})(Q / C_h)^{1.852} \quad \text{(Eq 3-11)}$$

Where:
$h_f$ = total head loss, ft
$d$ = pipe diameter, ft
$L'$ = the actual length of the pipeline ($L$) plus the sum of the calculated equivalent lengths ($L_e$), ft
$Q$ = flow rate, ft$^3$/sec
$C_h$ = Hazen–Williams roughness coefficient

The power required to overcome the total head loss may be determined from

$$P_p = (Q h_f \gamma)/550 \qquad \text{(Eq 3-12)}$$

Where:

$P_p$ = pump power requirements, hp
$Q$ = flow rate, ft³/sec
$h_f$ = total head loss, ft
$\gamma$ = unit weight of the fluid, lb/ft³; for water at 50°F, $\gamma$ = 62.41 lb/ft³

The preceding equations for total head loss and power requirements may be combined to yield the following equation for the annual pumping power cost relative to frictional head:

$$\text{Cost/year} = \frac{(0.745 \times P_p \times \text{power cost} \times \text{hours/year})}{(\text{pump eff.} \times \text{motor eff.})} \qquad \text{(Eq 3-13)}$$

Where:

Cost/year = pumping power cost per year, $
$P_p$ = pump power requirements, hp
power cost = power cost, $/kW·h
hours/year = number of hours per year of pump operation
pump eff. = the pump efficiency, expressed as a decimal
motor eff. = the motor efficiency, expressed as a decimal

It should be noted that the cost calculated using the preceding formulas is the cost of power required to overcome total head loss and does not include the cost of pumping required to overcome changes in elevation. Since the elevation change will be the same for all pipe diameters being evaluated, it can be ignored in the comparison of pipe diameters. If the designer desires to graphically present the hydraulic grade line for one or more of the pipe diameters to calculate total head or to calculate the total annual power cost, then the static head should be combined with the frictional head, and the sum of the two should be used in the preceding equations.

When selecting pipe diameters based on the preceding formulas, it is important that the designer consider both average annual pumping conditions and peak pumping conditions. The annual power cost equation, if used in conjunction with the peak pumping rate, will lead the designer to select a diameter that is larger than needed. In addition, the designer must consider that power factors or demand charges affect the rates paid for power and that power costs will increase with time. Therefore, it may be prudent to select an escalation rate so that a realistic comparison between the first cost of the pipe and the long-term pumping costs may be equitably made.

## AIR ENTRAPMENT AND RELEASE

Air entrapment in poorly vented pipelines can reduce pipeline carrying capacity, cause unexpected pressure surges, and produce objectionable "white water." The following three conditions must exist before air binding presents a serious problem:

- There must be a source of air in excess of that normally held in solution in the flow.
- The air must separate from the water and collect at high points or descending legs.
- The collected air must remain near the high points or in the descending legs.

It is desirable to vent high points and/or descending legs on transmission mains with automatic air-release valves to relieve this condition. Improperly sized air-release valves can contribute to surge pressure conditions as described in chapter 4.

## BLOWOFF OUTLETS

Blowoffs may be located at low points in the profile of a pipeline. They are mainly used to drain lines for maintenance or inspection. Occasionally, blowoffs are used to remove silt or sediment from a line. If there is the possibility of backsiphoning contaminated water into water transmission mains, the outlet for blowoff discharge piping should be elevated. Blowoffs should generally be sized so the main pipeline velocity change is limited to 1 ft/s (0.3 m/s) or less during blowoff operation.

## REFERENCES

Albertson, M.L., J.R. Barton, and D.B. Simons. 1960. *Fluid Mechanics for Engineers.* Englewood Cliffs, N.J.: Prentice-Hall Inc.

American Society of Civil Engineers (ASCE). 1975. *Pipeline Design for Water and Wastewater.* Report of the Task Committee on Engineering Practice in the Design of Pipelines. New York: ASCE.

Bradley, J.N., and L.K. Thompson. 1951. *Friction Factors of Large Conduits Flowing Full.* Engineering Monograph 7. US Bureau of Reclamation.

Edmunds, R.C. 1979. Air Binding in Pipes. *Jour. AWWA,* 71(5):272–277.

Russel, G.E. 1952. *Hydraulics,* 5th ed. New York: Henry Holt & Company.

Swanson, H.V., and M.D. Reed. 1963. Comparison of Flow Formulas and Friction Factors for Concrete Pressure Pipe. *Jour. AWWA,* 55(1):67–80.

Chapter 4

# Surge Pressure

Surge pressure is the result of a change in fluid velocity in a closed conduit. This change in flow causes waves to travel upstream and downstream from the point of origin. The waves, in turn, cause increases or decreases in pressure as they travel along the line. These pressure changes are variously referred to as water hammer, surge, or transient pressure.

Surge pressure or water hammer is independent of the internal working pressure created by the fluid in the line and is proportional to the velocity change and wave speed. Therefore, surge pressure must be added to the internal working pressure to determine the total pressure experienced by the pipe when surges occur.

The phenomenon of water hammer is extremely complex and cannot be covered in depth in this manual. Only the fundamentals of elastic wave theory and specific data pertaining to the properties of concrete pipe are discussed. For a more detailed understanding of water hammer, the references listed at the end of this chapter should be consulted.

## EQUATIONS AND VARIABLES

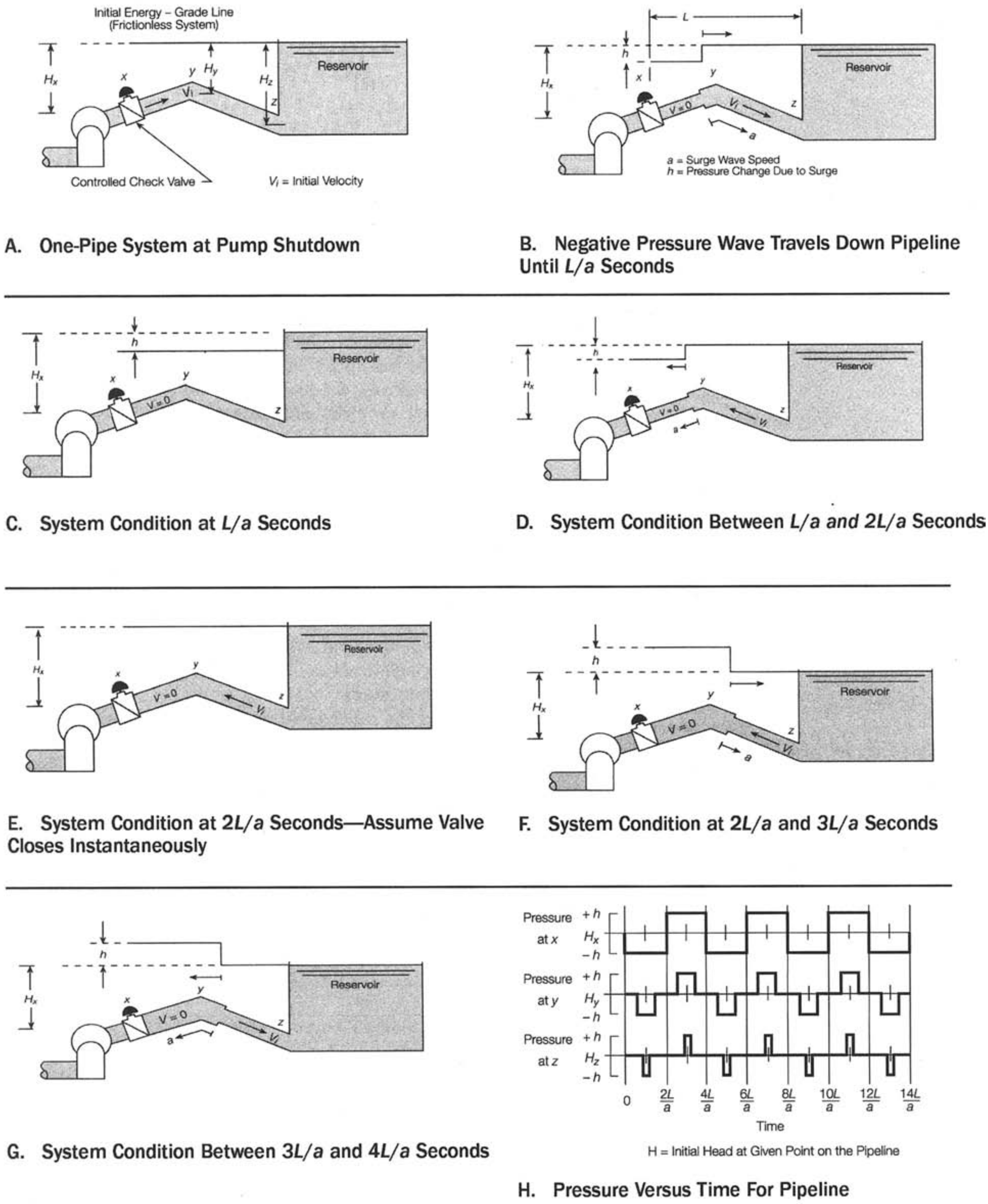
The critical time period $T_c$ for a piping system is the time required for a transient wave to travel in the piping system to a point of reflection and return to the point of origin. The value of $T_c$ may be determined from

$$T_c = 2L/a \quad \text{(Eq 4-1)}$$

Where:

$T_c$ = critical time period, sec
$L$ = the distance that a transient wave travels in the piping system between its origin and reflection point, ft
$a$ = wave speed, ft/sec

The term *critical period* is used because the change in flow velocity, which occurs in a time period equal to or less than $T_c$, will cause the maximum pressure rise for a particular surge event. The pressure rise from a fluid velocity change occurring within the critical period is illustrated in Figure 4-1. Pressure waves from changes




A. One-Pipe System at Pump Shutdown

B. Negative Pressure Wave Travels Down Pipeline Until $L/a$ Seconds

C. System Condition at $L/a$ Seconds

D. System Condition Between $L/a$ and $2L/a$ Seconds

E. System Condition at $2L/a$ Seconds—Assume Valve Closes Instantaneously

F. System Condition at $2L/a$ and $3L/a$ Seconds

G. System Condition Between $3L/a$ and $4L/a$ Seconds

H. Pressure Versus Time For Pipeline

NOTE: While the surge wave duration is shorter near the reservoir, the magnitude of the surge wave is the same throughout the line. The magnitude of the surge wave at any station on the line is not related to the operating pressure at that station when the surge occurs.

Figure 4-1 Idealized surge cycle for instantaneous pump shutdown in velocity occurring over longer time periods will be reduced by molecular frictional losses and by overlapping with the lower pressure troughs of returning surge waves.

The pressure rise from a change in fluid velocity occurring within the critical time period is calculated from

$$H_s = a(V_i - V_f)/g \tag{Eq 4-2}$$

Where:

$H_s$ = pressure rise due to surge, ft
$a$ = wave speed, ft/sec
$V_i$ = initial fluid velocity, ft/sec
$V_f$ = fluid velocity at time $t = T_c$, ft/sec
$g$ = gravitational constant, 32.2 ft/sec$^2$

If flow is completely stopped in $t < T_c$ seconds, $V_f$ is equal to zero and the formula reduces to

$$H_s = \frac{aV}{g} \tag{Eq 4-3}$$

Where:

$V$ = initial fluid velocity, ft/sec

and the other variables are as previously defined.

The maximum surge pressure that will be encountered by a line is a function of the maximum velocity that can be achieved by the system and the wave speed associated with the piping material. The maximum pressure surge will occur in situations in which flow is moving at the maximum velocity and is stopped by the closing of a quick-acting valve, the failure of the pump, or other actions that could stop the flow in a time that is less than the critical time $T_c$. The speed of the surge wave in a pipeline is determined by the type and thickness of the material from which the pipeline is constructed, the pipeline diameter, and the liquid being transported by the line.

The wave speed $a$ of a hydraulic transient may be determined from the following formula:

$$a = \left[\frac{(144E_L/\rho)}{1 + \frac{E_L D}{E_p t}}\right]^{0.5} \tag{Eq 4-4}$$

Where:

$a$ = wave speed, ft/sec
$E_L$ = bulk modulus of the liquid, psi (300,000 psi for water)
$\rho$ = mass density of the fluid, lb-s$^2$/ft$^4$ (1.94 lb-s$^2$/ft$^4$ for water)
$D$ = inside diameter of the pipe, in.
$E_p$ = modulus of elasticity of the pipe wall, psi
$t$ = transformed pipe wall thickness, in.

For water, $(144E_L/\rho)^{0.5}$ = 4,720 ft/s, and Eq. 4-4 becomes

$$a = \frac{4{,}720}{\left[1 + \frac{E_L D}{E_p t}\right]^{0.5}} \qquad \text{(Eq 4-5)}$$

with variables as previously defined.

Because concrete pressure pipe is a composite structure, the solution of this equation is not straightforward. The modulus of elasticity of the pipe wall $E_p$ is derived from the slope of the stress–strain curve of pipe wall material. These curves for concrete pressure pipe change in slope over the possible ranges of pressure the pipe may encounter. Kennison (1955) published a detailed explanation of this theory along with theoretical values and experimental results.

Calculated values of the surge wave speed for various types and classes of concrete pressure pipe range from 2,950 to 4,000 ft/sec (899 to 1,219 m/sec). Design values for different types of concrete pressure pipe are listed in Table 4-1. In piping systems where gas bubbles are dispersed throughout the liquid, the wave speed may be greatly reduced. The reader is referred to Wylie and Streeter (1983) for a more detailed explanation of this phenomenon.

## NEGATIVE PRESSURES

The conditions that produce pressure rises from water hammer also cause a reduction in pipeline pressure when the pressure wave reflects at a boundary. These pressure reductions, if large enough in magnitude, will result in negative pressures in all or part of the piping system. However, because of the inherent structural qualities of the concrete pipe wall, concrete pressure pipe does not collapse from negative pressures and does not require the special design considerations necessary to guard flexible pipe against collapse as a result of negative pressures.

If the pressure becomes lower than the vapor pressure of water, extensive vapor cavities may result. The subsequent positive surge pressures that result from the collapse of the vapor cavity can be extremely high. Pipeline high points are particularly susceptible to vapor cavity formation, and, therefore, vacuum-release valves are frequently installed at such locations. A surge tank or hydropneumatic chamber can also be used to prevent vapor cavity formation.

## CAUSES OF SURGE PRESSURE

Water hammer can result from any condition that causes a sudden change in the flow rate of the fluid in the pipeline. The following is a partial list of the causes of water hammer:

- normal pump startup and shutdown
- pump power interruption
- valve operations
- air venting from pipeline during filling
- improper operation of pressure-relief valves
- operation of large-orifice air and vacuum valves
- collapse of vapor cavities caused by negative pressures

Table 4-1 Design values—surge wave velocities, *ft/sec (m/sec)*

| AWWA Description | | Pipe Class, *psi (kPa)* | | | | | |
|---|---|---|---|---|---|---|---|
| | | 100 | (689) | 150 | (1,034) | 200 | (1,379) |
| Prestressed concrete pressure pipe, steel cylinder type (ANSI/ AWWA C301-type pipe) | Lined cylinder | 3,500–3,750 | (1,067–1,143) | 3,400–3,775 | (1,036–1,151) | 3,350–3,850 | (1,021–1,173) |
| | Embedded cylinder | 3,025–3,625 | (922–1,105) | 3,000–3,650 | (914–1,113) | 2,975–3,675 | (907–1,120) |
| Concrete water pipe, bar-wrapped steel cylinder type (ANSI/ AWWA C303-type pipe) | | 3,175–3,875 | (968–1,181) | 3,325–3,925 | (1,013–1,196) | 3,450–3,975 | (1,052–1,211) |

Pump startup and shutdown, pump power failure, and valve operations are sometimes the only operational situations investigated to determine the magnitude of water hammer. Pipeline filling and operation of its pressure-relief valves are occasionally overlooked water hammer sources that should be considered.

During pipeline filling, air in the line may escape through some type of a control structure (e.g., an air-release or discharge valve). The viscosity of air is much less than water, so air can escape at a high rate through the air-release valves, allowing the water to enter the pipeline at an equal rate. Air-release valves generally have a float valve that snaps shut instantaneously when all of the air has been released. Depending on the physical characteristics of the system, the magnitude of the velocity change can result in very high surge pressure. In most cases, pipeline surge-pressure damage will not occur if line filling is performed at a rate of flow equivalent to a full pipe velocity of 1 ft/sec (0.3 m/sec) or less.

The operation of a pressure-relief valve may cause water hammer. When the system becomes overpressured, the valve is designed to open and relieve line pressure by releasing fluid from the pipeline. Most relief valves are quick opening but close at a controlled rate. If the size of the valve is large in comparison to the pipeline size and is improperly adjusted, or if the valve closure rate setting is not correct (especially in pipelines during the testing phase), the quick action of the relief valve may cause a sudden and extreme change in fluid velocity, thus creating water hammer.

## CONTROL OF WATER HAMMER

Studies of surges should be conducted during the pipeline design stage. Once the general layout of the system has been completed, the length, diameter, thickness, material, and capacity of the pipe and the type and size of pumps can be established.

When analyzing surges, the designer generally first computes the hydraulic grade line for the ultimate operating mode of the system. The maximum working pressure at any point along the line is determined as the difference in elevation between the hydraulic grade line and the pipeline. Then, using the maximum volume of flow the pumping system can discharge or is expected to be able to discharge in the future, the maximum velocity is computed for the selected pipe diameter. Calculating surge pressure based on instantaneous stoppage of this flow will yield the maximum surge the system can expect to experience if the pipeline is designed without surge controls and if vapor cavity formation is prevented.

Concrete pressure pipe designs provide an allowance for water hammer that is generally adequate, but the design should be checked to be sure the pipe design provides an adequate allowance for surge. If not, either the pipe design should be adjusted to provide enough surge allowance to withstand the conditions that might be encountered or suitable remedial or control devices should be provided. The latter method is almost always less costly. It is important to note that there is no single device that will cure all surge difficulties. Only by a study of both normal operating conditions and possible emergency conditions can methods be determined to provide proper control.

It is not feasible to make general recommendations on the type, size, and application of surge-control equipment for all systems. Several possible solutions should be considered for any individual installation, and the one selected should be the one that gives the most protection for the least expenditure. Surges can often be reduced substantially by using bypasses around check valves, by cushioning check valves for the last 15–20 percent of the stroke, or by adopting a two-speed rate of valve stroke. Water hammer resulting from power failure to centrifugal pumps can sometimes be held to safe limits by providing flywheels or by allowing the pumps to run backward, as long as (1) the reverse pump flow does not produce motor backspin speed greater than recommended by the motor manufacturer, and (2) the motor cannot be reenergized during backspin. Otherwise, the electric motor coupling should be designed to automatically disengage if pump backspin is permitted. Air-inlet valves may be needed, or the preferred solution may be to use a surge tank, a surge damper, or a hydropneumatic chamber.

It is essential to coordinate all the elements of a system properly and to ascertain that operating practices conform to the requirements for safety. As changes take place in the system demand, it may be necessary to review and revise the surge conditions, particularly if the capacity is increased, additional pumpage or storage is added, or booster stations are planned.

The surge wave theory has been proven in actual practice, and design engineers should take the initiative in making surge studies and installing surge-control devices without waiting for serious failures to occur. The time and effort spent on a surge study in advance of the final design is the least expensive means of avoiding problems from excessive surge pressures.

## REFERENCES

Angus, R.W. 1937. Water Hammer in Pipes, Including Those Supplied by Centrifugal Pumps; Graphical Treatment. In *Proc. of the Inst. Mech. Engrs.*, 136:245.

Chaundhry, M.G. 1979. *Applied Hydraulic Transients*. New York: Van Nostrand Reinhold Company.

Kennison, H.G. 1955. Surge-Wave Velocity — Concrete Pressure Pipe. Paper presented at the ASME Annual Meeting in Chicago, Ill.

Kerr, S.L. 1948. Practical Aspects of Water Hammer. *Jour. AWWA*, 40(6):599–605.

———. 1951. Water Hammer Control. *Jour. AWWA*, 43(12):985–1000.

Parmakian, J. 1963. *Water Hammer Analysis*. New York: Dover Publications.

Rich, G.R. 1951. *Hydraulic Transients* (Engineering Societies Monographs). New York: McGraw-Hill Book Company.

Wylie, E.B., and V.L. Streeter. 1983. *Fluid Transients*. Ann Arbor, Mich.: Feb Press.

*Chapter* 5

# External Loads

This chapter provides information and methods to determine dead loads imposed on a conduit that is constructed in a trench or an embankment, as well as jacked or tunneled installations. Information and methods are also presented to determine live loads resulting from highway truck traffic and railways.

Many people have contributed to the development of associated load and supporting strength theories, but the basic load theory is rightly recognized as the Marston theory. The following presentation is based primarily on the results of studies made by the Engineering Experiment Station of Iowa State University, many of them in cooperation with the Public Roads Administration.

The Marston Theory of Loads on Underground Conduits demonstrates by rational principles of mechanics that the load on an underground structure is greatly affected by conditions of installation, as well as by the weight of fill over the conduit. The installation conditions determine the magnitude and direction of settlements of the prism of soil directly over the conduit relative to the settlements of the exterior prisms immediately adjacent. These relative settlements generate friction or shearing forces that are added to or subtracted from the weight of the central prism to produce the resultant load on the conduit.

## MAJOR INSTALLATION CLASSIFICATIONS

Because of the influence of these installation conditions and the importance of recognizing them when determining loads, underground conduit installations have been classified into groups and subgroups, as illustrated in Figure 5-1. Four common types are trench, positive projecting embankment, negative projecting embankment, and induced trench. Pipe is also installed by jacking or tunneling methods when deep installations are necessary or when conventional open excavation and backfill methods may not be feasible. The essential features of each of these installations are shown in Figure 5-2.

## TRENCH CONDUITS

Trench conduits are installed in relatively narrow excavations in passive or undisturbed soil and then covered with earth backfill to the original ground surface. The trench width must be sufficient to permit proper compaction or consolidation of the

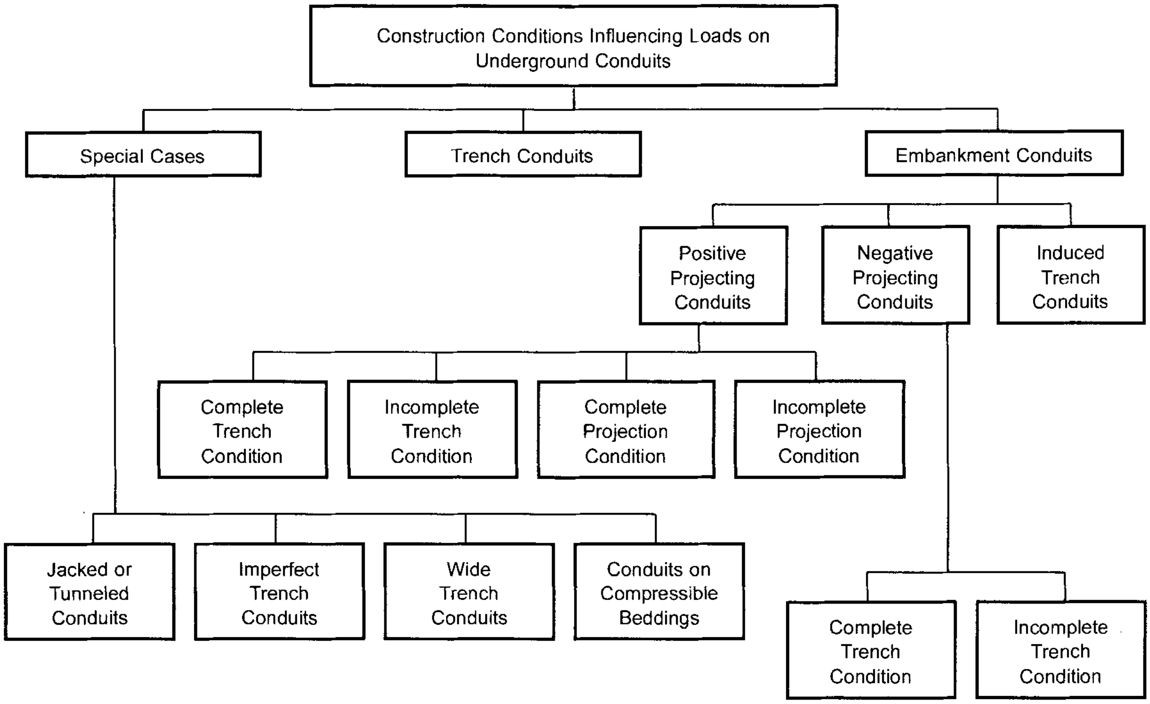


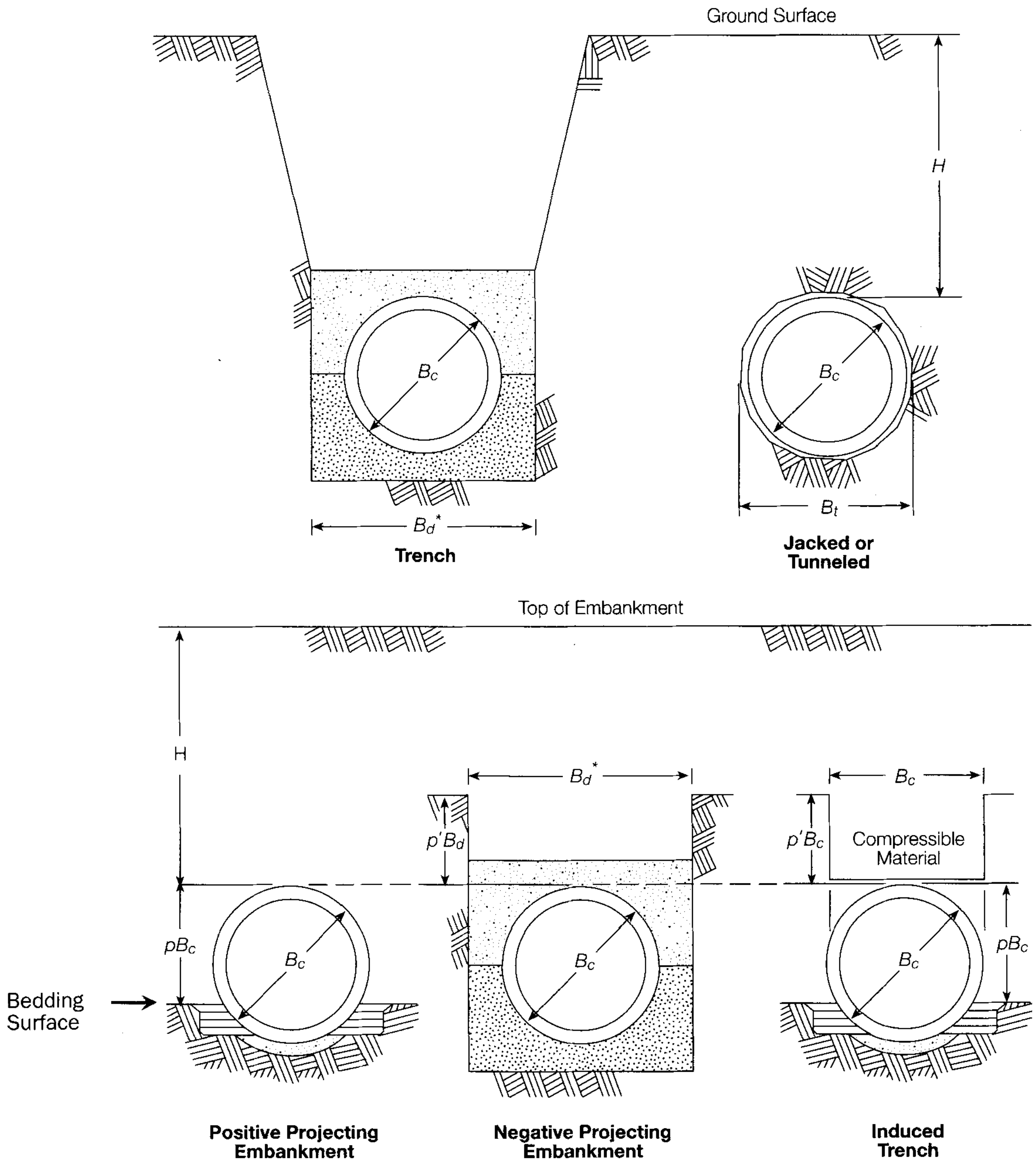

Figure 5-1 Underground conduit installation classifications

pipe bedding and backfill; widths 1.5 to 3.0 ft (0.45 to 0.91 m) greater than the outside diameter of the pipe are common. The trench load theory is based on the following assumptions:

- Load on the pipe develops as the backfill settles because the backfill is not compacted to the same density as the surrounding earth.
- The resultant load on an underground structure is equal to the weight of the material above the top of the conduit minus the shearing or friction forces on the sides of the trench. These shearing forces are computed in accordance with Rankine's theory.
- Cohesion is assumed to be negligible because (1) considerable time must elapse before effective cohesion between the backfill material and the sides of the trench can develop, and (2) the assumption of no cohesion yields the maximum probable load on the conduit.
- In the case of rigid pipe, the side fills may be relatively compressible, and the pipe itself will carry practically all the load developed over the entire width of the trench.

When a pipe is placed in a trench, the prism of backfill placed above it will tend to settle downward. Frictional forces will develop along the sides of the trench walls as the backfill settles and act upward against the direction of the settlement. The fill load on the pipe is equal to the weight of the mass of fill material less the summation of the frictional load transfer, and is expressed by the following formula:

$$W_d = C_d w B_d^2 \quad \text{(Eq 5-1)}$$




*Width of trench, in feet, at top of pipe. Trench may be wider than $B_d$ above top of pipe.

Figure 5-2 Essential features of types of installations

Where:

$W_d$ = trench load, lb/lin ft
$C_d$ = trench load coefficient
$w$ = unit weight of fill material, lb/ft$^3$
$B_d$ = width of trench at top of pipe, ft

$C_d$ is further defined as:

$$C_d = \frac{1 - e^{-2K\mu'(H/B_d)}}{2K\mu'} \quad \text{(Eq 5-2)}$$

Where:

$C_d$ = trench load coefficient
$e$ = base of natural logarithms
$K$ = $\tan\left(45° - \frac{\phi'}{2}\right)$ = Rankine's ratio of active lateral unit pressure to vertical unit pressure, with $\phi'$ = friction angle between backfill and soil
$\mu'$ = $\tan \phi'$ = coefficient of friction between fill material and sides of trench
$H$ = height of fill above top of pipe, ft
$B_d$ = width of trench at top of pipe, ft

Recommended values for the product $K\mu'$ for various soils are

$K\mu'$ = 0.1924 for granular materials without cohesion
$K\mu'$ = 0.1650 maximum for sand and gravel
$K\mu'$ = 0.1500 maximum for saturated top soil
$K\mu'$ = 0.1300 maximum for ordinary clay
$K\mu'$ = 0.1100 maximum for saturated clay

For very deep trenches, the load coefficient $C_d$ approaches a value of $1/(2K\mu')$, so an accurate selection of the appropriate $K\mu'$ value becomes more important. Generally, when the character of the soil is uncertain, it is adequate to assume $K\mu' = 0.150$ and $w$ = 120 lb/ft$^3$ (1,922 kg/m$^3$).

Study of the load formula shows that an increase in trench width $B_d$ (where $B_d$ is measured at the top of the pipe) will cause a marked increase in load. Consequently, the value of $B_d$ should be held to the minimum that is consistent with efficient construction operations and safety requirements. If the trench sides are sloped back, or if the width of the trench is large in comparison with the pipe, $B_d$ and the earth load on the pipe can be decreased by constructing a narrow subtrench at the bottom of the wider trench.

As trench width increases, the upward frictional forces become less effective in reducing the load on the pipe until the installation finally assumes the same properties as a positive projecting embankment condition. This situation is common when $B_d$ is approximately equal to or greater than $H$. The positive projection embankment condition represents the most severe load that the pipe can be subjected to; any further increase in trench width would have no effect. The maximum effective trench width, where transition to a positive projecting embankment condition occurs, is referred to as the *transition trench width*. The trench load formula does not apply when the transition trench width has been exceeded. The load generated (i.e., positive projecting) is referred to as the *transition load*. The transition load may be calculated using the positive projection embankment load formula. The transition trench width is found by setting the trench load $W_d$ equal to the transition load and solving for $B_d$.

## Calculated Loads

The preceding trench formula, with proper selection of the physical factors involved, predicts the maximum loads to which a particular pipe may be subjected when in service. Because of cohesion or other causes, these loads may never fully develop during the service life of the pipe.

## Trenches in Caving Soils

Some projects involve the decision as to whether to place sheeting in the trench or to allow the sides to cave in and thus to produce a trench with sloping sides. When sheeting is used, the load may be affected materially.

If the sheeting is left in place, the coefficient of sliding friction $\mu'$ may be reduced, increasing $C_d$ and the load. When the sheeting is to be retrieved, it should be pulled in 3- to 4-ft (0.9- to 1.2-m) increments to allow time for frictional forces to develop between the backfill and sides of the trench. This should result in the most favorable loading condition for a pipe in a sheeted trench. If the sheeting is pulled as the trench is backfilled, the value of $K\mu'$ is that for the fill material and the soil of the trench sides, and the value of $B_d$ is the width between the back faces of the sheeting. If the sheeting is pulled after all or most of the fill is completed, then the mass of fill material will probably retain its shape for a time, thus eliminating the frictional load transferences and substantially increasing the load on the pipe.

Calculation of earth load for trench conditions using the Marston theory is demonstrated in chapter 7 in the design examples for ANSI/AWWA C300-type and ANSI/AWWA C302-type pipe.

# EMBANKMENT CONDUITS

Embankment conduits are those that are covered by fills or embankments, such as railway embankments, highway embankments, and earth dams. Embankment installations are subdivided into the following three groups:

- Positive projection pipe is installed with the top of the pipe projecting above the surface of the natural ground or compacted fill and then covered with earth fill, as illustrated in Figure 5-3. This type also includes pipe installation in wide trenches, i.e., with $B_d$ equal to or greater than the transition width.
- Negative projection pipe is installed in relatively shallow trenches of such depth that the top of the pipe is below the level of the natural ground surface or compacted fill, as illustrated in Figure 5-4. The pipe is then covered with earth fill to a height appreciably greater than the distance from the natural ground surface or original compacted fill surface to the top of the pipe.
- Induced trench pipe is initially installed as positive projection. When the embankment fill has been placed to an elevation of at least one pipe diameter over the proposed top of the pipe, a trench is excavated over the pipe and backfilled with a more compressible material, simulating a negative projection installation. This installation condition is illustrated in Figure 5-5.

# POSITIVE PROJECTION INSTALLATIONS

In considering earth loads in positive projection installations, it is customary to designate the prism of fill directly above the pipe and bounded by vertical planes tangent to the side of the pipe as the interior prism. The exterior prisms are those adjacent to the vertical planes on both sides of the pipe and of indefinite width. The load transmitted to the top of the pipe is equal to the weight of the interior prism of soil, plus or minus the friction forces that develop along the two vertical planes bounding the interior prism.

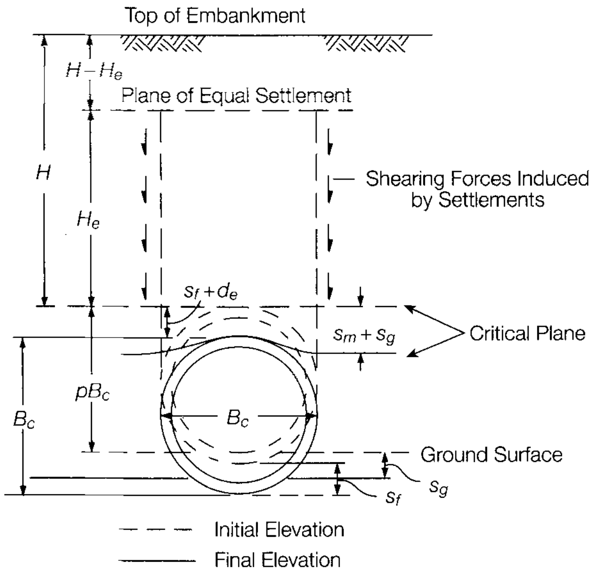
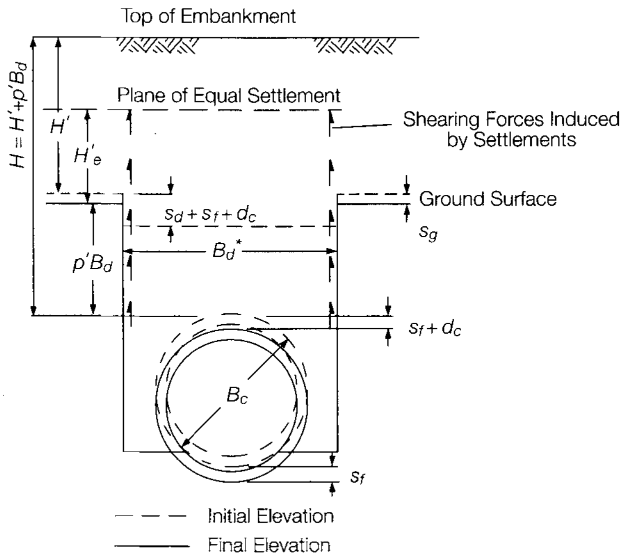



Figure 5-3 Settlements that influence loads—positive projecting embankment installation




*Width of trench, in feet, at top of pipe.

Figure 5-4 Settlements that influence loads—negative projecting embankment installation

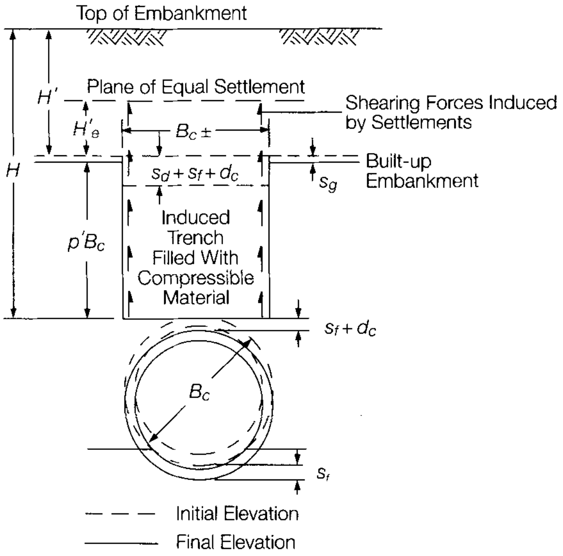



Figure 5-5 Settlements that influence loads — induced trench installation

The horizontal plane at the top of the pipe is defined as the critical plane. Unless the embankment material on each side of the pipe is compacted so as to have the same relative settlement as the pipe, the exterior prism of soil will compress more than the interior prism. In this case, the frictional forces on the vertical planes between the interior and exterior prisms will add to the load on the pipe, and the total load will be the weight of the interior prism of soil plus the frictional forces. Under these conditions, the critical plane will be deformed downward in the exterior prisms with respect to the top of the pipe and the interior prism, and the pipe is defined as being in the projection condition.

If the pipe is placed on a slightly yielding foundation, it will settle more than the adjacent soil as the fill is constructed. This condition would reverse the frictional forces and reduce the load on the pipe to a value less than the weight of the soil in the interior prism. In this condition, the critical plane will deform downward in the interior prism with respect to the top of the pipe and the exterior prisms, and the pipe is defined as being in the trench condition.

The frictional forces are the summation of the active lateral pressure on the two vertical planes multiplied by $\mu$, the coefficient of internal friction of the fill material. If the frictional forces extend up to the surface of the fill, the pipe is defined as being in the complete projection condition or the complete trench condition. If the frictional forces do not extend to the surface of the fill but cease at a horizontal plane below the top of the fill, the pipe is defined as being in the incomplete projection condition or the incomplete trench condition. The horizontal plane where the frictional forces cease is defined as the plane of equal settlement, and its height above the top of the pipe is the height of equal settlement, $H_e$.

The preceding concepts were considered by Marston, and the formula for earth loads in the projection and trench condition were mathematically derived using the same principles and notation as for the analysis of the trench installations.

The load on a positive projecting pipe is computed by the equation

$$W_c = C_c w B_c^2 \qquad \text{(Eq 5-3)}$$

Where:

$W_c$ = positive projection load, lb/lin ft
$C_c$ = positive projection load coefficient
$w$ = unit weight of fill material, lb/ft$^3$
$B_c$ = outside diameter of the pipe, ft

$C_c$ is further defined as

$$C_c = \frac{e^{\pm 2K\mu(H/B_c)} - 1}{\pm 2K\mu} \text{ when } H \leq H_e, \text{ and} \qquad \text{(Eq 5-4A)}$$

$$C_c = \frac{e^{\pm 2K\mu(H_e/B_c)} - 1}{\pm 2K\mu} + \left[(H/B_c) - (H_e/B_c)\right]e^{\pm 2K\mu(H_e/B_c)} \text{ when } H > H_e \qquad \text{(Eq 5-4B)}$$

Where:

$C_c$ = positive projection load coefficient
$e$ = base of natural logarithms
$K$ = Rankine's ratio
$\mu$ = coefficient of internal friction of the soil
$H$ = height of fill above top of pipe, ft
$B_c$ = outside diameter of pipe, ft
$H_e$ = height of the plane of equal settlement above the top of pipe, ft

In Eq 5-4A, the upper (positive) signs are used for the complete projection condition and the lower (negative) signs are used for the complete trench condition. In Eq 5-4B, the upper (positive) signs are used for the incomplete projection condition and the lower (negative) signs are used for the incomplete trench condition.

To evaluate the $H_e$ term in the equation for $C_c$, it is necessary to determine numerically the relationship between the pipe deflection and the relative settlement between the prism of fill directly above the pipe and the adjacent soil. This relationship is defined as the settlement ratio and is expressed as

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m} \qquad \text{(Eq 5-5)}$$

Where:

$r_{sd}$ = settlement ratio
$s_m$ = compression strain of the side columns of soil of height $pB_c$
$s_g$ = settlement of the natural ground surface
$s_f$ = total settlement of the pipe invert
$d_c$ = vertical deflection of the pipe

The formula used to determine the height of the plane of equal settlement is

$$\left[\frac{1}{2K\mu} \pm \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) \pm \frac{r_{sd}p}{3}\right]\frac{e^{\pm 2K\mu(H_e/B_c)} - 1}{\pm 2K\mu} \pm \frac{1}{2}\left(\frac{H_e}{B_c}\right)^2$$

$$\pm \frac{r_{sd}p}{3}\left(\frac{H}{B_c} - \frac{H_e}{B_c}\right)e^{\pm 2K\mu(H_e/B_c)} - \frac{1}{2K\mu} \times \frac{H_e}{B_c} \mp \frac{H}{B_c} \times \frac{H_e}{B_c} = \pm\, r_{sd}p\frac{H}{B_c} \qquad \text{(Eq 5-6)}$$

Where:

$p$ = projection ratio, which is the vertical distance between the outside top of pipe and the ground or bedding surface divided by the outside diameter of the pipe $B_c$

and the other variables were previously defined.

Again, the upper signs are used for the incomplete projection condition when $r_{sd}$ is positive, and the lower signs are for the incomplete trench condition when $r_{sd}$ is negative. Recommended design values for $r_{sd}$ are shown in Table 5-1.

The values of the product $K\mu$ for various soils are often considered the same as for the product $K\mu'$. These values range from less than 0.05 to a maximum of just over 0.19. When the character of the soil is unknown, a $K\mu$ design value of 0.19 is suggested.

The formula for $H_e$ requires a complex, iterative solution procedure, but a value for $H_e$ is required if $C_c$ is to be calculated. By using Eq 5-6 and setting $H_e = H$, the value of $H$ that occurs when $H = H_e$ can be calculated. If the design value of $H$ is less than this calculated value, $C_c$ is calculated using Eq 5-4A. Otherwise, the actual value of $H_e$ is calculated using the design value of $H$ in Eq 5-6, and $C_c$ is calculated using Eq 5-4B. Figure 5-6 provides a graphical solution for $C_c$ that circumvents the need to determine $H_e$. This figure permits reasonable estimates of $C_c$ for various conditions of $H/B_c$, $r_{sd}$, and $p$. Because the effect of $\mu$ is nominal, $K\mu$ is assumed to be 0.19 for the projection condition and 0.13 for the trench condition, as recommended by Spangler (1948). Figure 5-6 provides an estimate for $C_c$ that is well within the accuracy of the theoretical assumptions.

In Figure 5-6, the family of straight lines represents the incomplete conditions, while the curves represent the complete conditions. The straight lines intersect the curves where $H_e$ equals $H$. These diagrams can therefore be used to determine the minimum height of fill for which the plane of equal settlement will occur within the soil mass.

Calculation of earth load for positive projecting conditions using the Marston theory is demonstrated in chapter 7 in the design examples for ANSI/AWWA C300-type and ANSI/AWWA C302-type pipe.

## EARTH LOADS ON LARGE DIAMETER ANSI/AWWA C303 PIPE

Large-diameter bar-wrapped concrete cylinder pipe (greater than 54 in. [1,370 mm] in diameter) is considered to be flexible. Therefore, the earth load on the pipe can be calculated by using the prism load consisting of the weight of a prism of soil, with a width equal to the pipe outside diameter and a height equal to the depth of fill over the pipe. This load can be conveniently used for all installation conditions for both trench and embankment conditions as follows:

$$W_c = w\, H_c B_c \qquad \text{(Eq 5-7A)}$$

Where:

- $W_c$ = earth load on the conduit, lb/lin ft
- $w$ = unit weight of fill, lb/ft$^3$
- $H_c$ = height of fill above top of pipe, ft
- $B_c$ = outside diameter of pipe, ft

Table 5-1 Design values of settlement ratio

| Installation and Foundation Condition | | Settlement Ratio $r_{sd}$ Usual Range | Design Value |
|---|---|---|---|
| Positive projection | | 0.0 to +1.0 | |
| | Rock or unyielding soil | +1.0 | +1.0 |
| | Ordinary soil* | +0.5 to +0.8 | +0.7 |
| | Yielding soil | 0.0 to +0.5 | +0.3 |
| Zero projection | | | 0.0 |
| Negative projection† | | −1.0 to 0.0 | |
| | $p' = 0.5$ | | −0.1 |
| | $p' = 1.0$ | | −0.3 |
| | $p' = 1.5$ | | −0.5 |
| | $p' = 2.0$ | | −1.0 |
| Induced trench† | | −2.0 to 0.0 | |
| | $p' = 0.5$ | | −0.5 |
| | $p' = 1.0$ | | −0.7 |
| | $p' = 1.5$ | | −1.0 |
| | $p' = 2.0$ | | −2.0 |

* The value of the settlement ratio depends on the degree of compaction of the fill material adjacent to the sides of the pipe. With construction methods resulting in proper compaction of bedding and sidefill materials, a settlement ratio design value of +0.5 is recommended for rigid pipe and +0.3 for semirigid pipe.

† $p'$ = negative projection ratio, which is the depth of the top of pipe below the critical plane divided by the width of the trench $B_d$.

## NEGATIVE PROJECTION INSTALLATIONS

In negative projection installations, pipe is installed in shallow trenches of such depth that the top of the pipe is below the surface of the natural ground or compacted fill, and then covered with an embankment that extends above the ground level (Figure 5-4). In negative projection theory, load transmitted to the pipe is equal to the weight of the interior prism of soil above the pipe, minus frictional forces along the sides of that prism. The critical plane is the horizontal plane through the top of the subtrench, and the width of the interior prism is defined as the width of the subtrench.

The load for negative projecting pipe is computed by the equation

$$W_n = C_n w B_d^2 \qquad \text{(Eq 5-7B)}$$

Where:

$W_n$ = negative projection fill load, lb/lin ft
$C_n$ = the load coefficient for negative projecting conduits
$w$ = unit weight of fill material, lb/ft³
$B_d$ = width of trench at top of pipe, ft

$C_n$ is further defined as

$$C_n = \frac{e^{-2K\mu(H/B_d)} - 1}{-2K\mu} \text{ when } H \le H_e \text{, and} \qquad \text{(Eq 5-8A)}$$

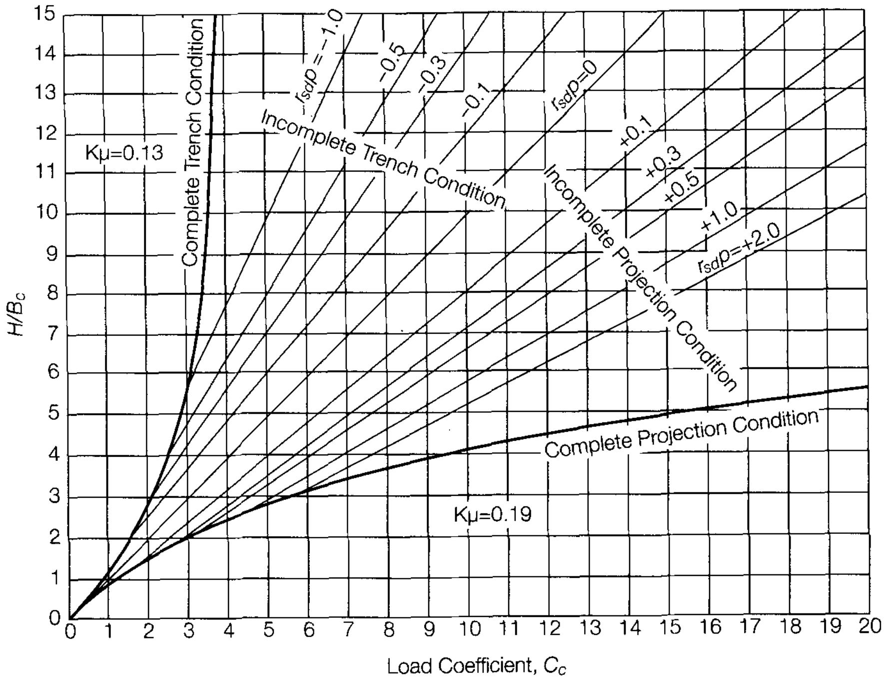



Figure 5-6 Load coefficient for positive projection embankment condition

$$C_n = \frac{e^{-2K\mu(H_e/B_d)} - 1}{-2K\mu} + [(H/B_d) - (H_e/B_d)]\, e^{-2K\mu(H_e/B_d)} \quad \text{when } H > H_e \qquad \text{(Eq 5-8B)}$$

Where:

$C_n$ = negative projection load coefficient
$e$ = base of natural logarithms
$K$ = Rankine's ratio
$\mu$ = coefficient of internal friction of the soil
$H$ = height of fill above top of pipe, ft
$B_d$ = width of trench at top of pipe, ft
$H_e$ = height of the plane of equal settlement above top of pipe, ft

To determine $H_e$, the settlement ratio $r_{sd}$ is defined as

$$r_{sd} = \frac{s_g - (s_d + s_f + d_c)}{s_d} \qquad \text{(Eq 5-9)}$$

Where:

$r_{sd}$ = settlement ratio
$s_g$ = settlement of the natural ground surface
$s_d$ = settlement of the soil from the original ground surface to the top of the pipe
$s_f$ = total settlement of the pipe invert
$d_c$ = vertical deflection of the pipe

If $H'$ is defined as $(H - p'B_d)$ and $H'_e$ is defined as $(H_e - p'B_d)$, then the equation for $H'_e$ is

$$\frac{e^{-2K\mu(H'_e/B_d)} - 1}{-2K\mu}\left(\frac{H'}{B_d} - \frac{H'_e}{B_d} - \frac{1}{2K\mu}\right) - \frac{H'_e}{B_d}\left[\left(\frac{H'}{B_d} - \frac{H'_e}{B_d}\right) + \frac{1}{2}\frac{H'_e}{B_d} - \frac{1}{2K\mu}\right]$$

$$= \frac{2}{3} r_{sd} p' \left[\frac{e^{-2K\mu(H'_e/B_d)} - 1}{-2K\mu} + \left(\frac{H'}{B_d} - \frac{H'_e}{B_d}\right) e^{-2K\mu(H'_e/B_d)}\right] \quad \text{(Eq 5-10)}$$

Where:

$p'$ = negative projection ratio, which is the depth of the top of pipe below the critical plane divided by the width of the trench $B_d$

and all other variables were previously defined.

Recommended design values for $r_{sd}$ are shown in Table 5-1.

As with $H_e$ for positive projection, the solution of the formula for $H'_e$ requires a complex, iterative procedure. To avoid the need to determine $H'_e$, values of $C_n$ versus $H/B_d$ for various values of $r_{sd}$ are provided in Figures 5-7 through 5-10 for values of $p'$ equal to 0.5, 1.0, 1.5, and 2.0. For other values of $p'$ between 0.5 and 2.0, values of $C_n$ may be obtained by interpolation. These figures are based on $K\mu = 0.13$, so they will give conservative values of $C_n$ if the actual $K\mu$ is greater than 0.13. The family of straight lines representing the incomplete condition intersects the curve for the complete condition. At these intersections, the height of the plane of equal settlement $H_e$ equals the height to the top of embankment $H$. These intersecting points can be used to determine the minimum height of fill for which the plane of equal settlement will occur within the soil mass.

## INDUCED TRENCH INSTALLATIONS

The induced trench method of construction is a practical method for relieving the load on pipe placed under high fills (Figure 5-5). The essential procedures of this method of construction are as follows:

1. Install pipe in a positive projection embankment condition.
2. Compact the fill material at each side of the pipe for a lateral distance equal to twice the outside diameter of the pipe or 12 ft (3.7 m), whichever is less. This fill is constructed to an elevation of at least one pipe diameter over the top of the pipe.
3. Excavate a trench in the compacted fill directly over the pipe. The depth of the trench should be at least one pipe diameter, and the width should coincide as nearly as possible with the outside diameter of the pipe.
4. Refill the trench with loose compressible material, such as straw, sawdust, or organic soil.
5. Complete the balance of the fill by normal methods.

An alternate method of induced trench construction is to partially construct the embankment before the pipe is installed to an elevation of at least one pipe diameter over the proposed top of the pipe. A trench is then excavated and the pipe installed. Compressible material is placed as loosely as possible directly above the pipe and the embankment completed by normal methods.

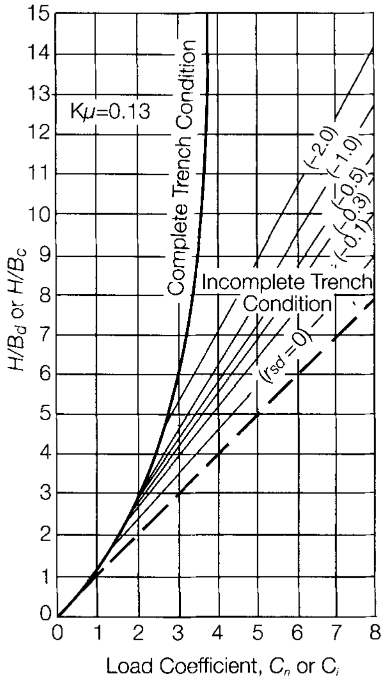



Figure 5-7 Load coefficient for negative projection and induced trench condition ($p'$ = 0.5)

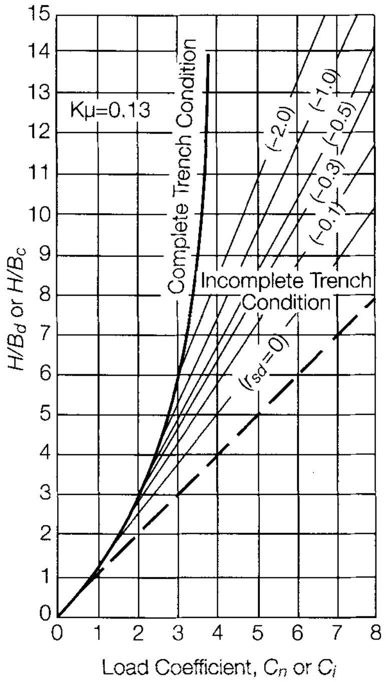



Figure 5-8 Load coefficient for negative projection and induced trench condition ($p'$ = 1.0)

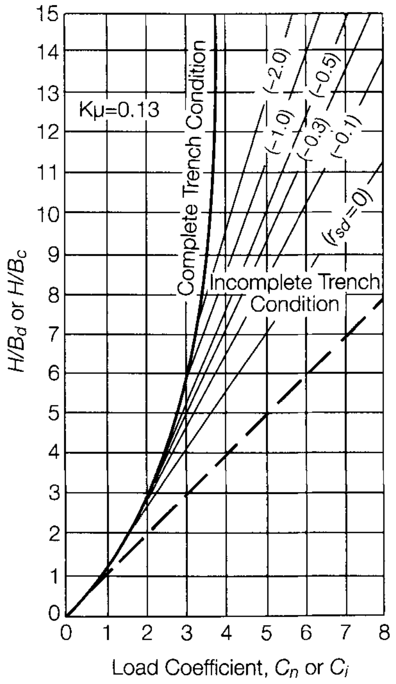



Figure 5-9 Load coefficient for negative projection and induced trench condition ($p'$ = 1.5)

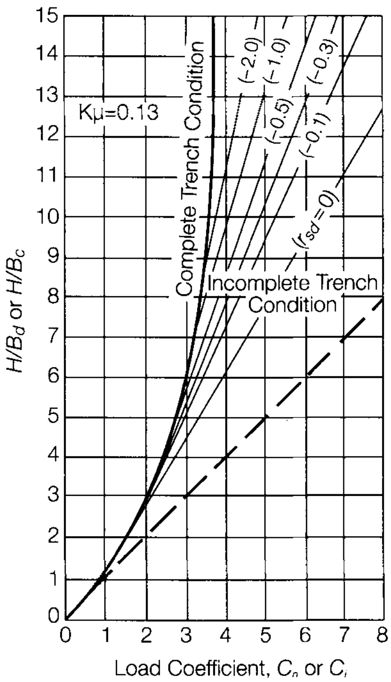



Figure 5-10 Load coefficient for negative projection and induced trench condition ($p'$ = 2.0)

This concept of installation was suggested by Marston (1930) and subsequently developed by Spangler (1948) as a special case of the negative projection condition. In this case, the width of the subtrench above the top of the pipe determines whether $B_c$ or $B_d$ is used to calculate the load on a pipe installed by the induced trench method in the design load equation

$$W_i = C_i w B_c^2 \tag{Eq 5-11}$$

Where:

$W_i$ = induced trench load, lb/lin ft
$C_i$ = induced trench load coefficient
$w$ = unit weight of fill material, lb/ft³
$B_c$ = outside diameter of pipe or width of trench, whichever is greater, ft

$C_i$ is further defined as

$$C_i = \frac{e^{-2K\mu(H/B_c)} - 1}{-2k\mu} \text{ when } H \leq H_e \text{, and} \qquad \text{(Eq 5-12A)}$$

$$C_i = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + [\,(H/B_c) - (H_e/B_c)\,]\, e^{-2K\mu(H_e/B_c)} \text{ when } H > H_e \qquad \text{(Eq 5-12B)}$$

Where:

$C_i$ = induced trench load coefficient
$e$ = base of natural logarithms
$K$ = Rankine's ratio
$\mu$ = coefficient of internal friction of the soil
$H$ = height of fill above top of pipe, ft
$B_c$ = outside diameter of pipe or width of trench, whichever is greater, ft
$H_e$ = height of the plane of equal settlement above top of pipe, ft

To avoid the need to determine $H_e$, values of $C_i$ versus $H/B_c$ for various values of $r_{sd}$ are provided in Figures 5-7 through 5-10 for values of $p'$ equal to 0.5, 1.0, 1.5, and 2.0. Negative projection ratio $p'$ for induced trench installations is defined as the depth of the excavated trench over the top of the pipe divided by the outside diameter of the pipe. Recommended design values for $r_{sd}$ were previously shown in Table 5-1.

The induced trench method results in less load on the pipe than either the positive projection or negative projection embankment conditions. The load reduction is obtained because the fill over the pipe settles downward relative to the adjacent fill, thus generating shearing forces that partially support the backfill. Because of the compressible material placed in the trench, a smaller (larger negative) $r_{sd}$ value is used to account for the larger relative settlement. The net result is a lower value for $W_i$ for any given height of fill and trench width.

## JACKED OR TUNNELED CONDUITS

This type of installation is used when surface conditions make it difficult to install pipe by conventional open excavation and backfill methods, or when it is necessary to install pipe under an existing embankment. The vertical load on the horizontal plane at the top of the bore, within the width of the excavation, is equal to the weight of the prism of earth above the bore minus the upward friction forces and minus the soil cohesion along the limits of the soil prism over the bore. This earth load is computed by the equation

$$W_t = C_t w B_t^2 - 2cC_t B_t \tag{Eq 5-13}$$

Where:

$W_t$ = earth load under tunneled or jacked conditions, lb/lin ft
$C_t$ = load coefficient for tunneled or jacked pipe
$w$ = unit weight of earth, lb/ft$^3$
$B_t$ = maximum width of bore excavation, ft
$c$ = cohesion of the soil above the excavation, lb/ft$^2$

$C_t$ is further defined as

$$C_t = \frac{1 - e^{-2K\mu(H/B_t)}}{2K\mu} \tag{Eq 5-14}$$

Where:

$C_t$ = load coefficient for tunneled and jacked pipe
$e$ = base of natural logarithms
$K$ = Rankine's ratio
$\mu$ = coefficient of internal friction of the soil
$H$ = height of cover above top of pipe, ft
$B_t$ = maximum width of bore excavation, ft

In Eq 5-13, the $C_t w B_t^2$ term is similar to the trench equation for trench loads, and the $2cC_t B_t$ term accounts for the cohesion of undisturbed soil. Conservative design values of the coefficient of cohesion for various soils are listed in Table 5-2.

## DETERMINATION OF LIVE LOAD

The most common live loads for which pipe must be designed are those from trucks, railroads, or construction equipment. The distribution of a live load at the surface on any horizontal plane in the subsoil is shown in Figure 5-11. The intensity of the load on any plane in the soil mass is greatest at the vertical axis directly beneath the point of application and decreases in all directions outward from the center of application. As the distance between the plane and the surface increases, the intensity of the load at any point on the plane decreases.

### Highways

If a rigid or flexible pavement designed for heavy-duty traffic is provided, and there is 1 ft (0.3 m) of cover between the top of the pipe and the bottom of the pavement, the intensity of a truck wheel load is usually reduced sufficiently so the live load transmitted to the pipe is negligible. In the case of flexible pavements designed for light-duty traffic but subjected to heavy truck traffic, the flexible pavement should be considered as fill material over the top of the pipe. In addition to considering light-duty pavements as fill material for calculating traffic loads, pipe should not be installed under light-duty pavements with less than 2 ft (0.6 m) of total cover between the bottom of the pavement and the top of the pipe.

The most critical AASHTO* loadings shown in Figure 5-12 should be used in either the single mode or the passing mode. Each of these loadings is assumed to be applied through dual wheel assemblies uniformly distributed over a surface area of 10 in. × 20 in. (254 mm × 508 mm) as shown in Figure 5-13. As recommended by AASHTO, the total wheel load is then assumed to be transmitted and uniformly distributed over a rectangular area on a horizontal plane at depth $H$ as shown in Figure 5-14 for a single HS-20 dual wheel. Distributed load areas for the alternate load and the passing mode for either loading are developed in a similar manner.

The average pressure intensity on the subsoil plane at the outside top of the pipe at depth $H$ is determined by the equation

$$w_L = \frac{WH(I_f)}{A_{LL}} \qquad \text{(Eq 5-15)}$$

Where:

$w_L$ = average pressure intensity, lb/ft²
$WH$ = total applied surface wheel loads, lb
$A_{LL}$ = distributed live load area, ft²
$I_f$ = impact factor

Table 5-3 presents AASHTO-recommended impact factors $I_f$ to be used in determining live loads imposed on pipe with less than 3 ft (0.9 m) of cover when subjected to dynamic traffic loads.

As depth $H$ increases, the critical loading configuration can either be one HS-20 wheel load, two HS-20 wheel loads in the passing mode, or the alternate load in the passing mode. The exact geometric relationship of individual or combinations of surface wheel loads cannot be anticipated; therefore, the most critical loading configurations and the outside dimensions of the distributed load areas within the indicated cover depths have been established, and these are summarized in Table 5-4.

Table 5-2 Design values of coefficient of cohesion

| Type of Soil | | Values of $c$ |
|---|---|---|
| Clay | Soft | 40 |
| | Medium | 250 |
| | Hard | 1,000 |
| Sand | Loose dry | 0 |
| | Silty | 100 |
| | Dense | 300 |
| Topsoil | Saturated | 100 |

* American Association of State Highway and Transportation Officials, 444 N. Capitol St., N.W., Washington, DC 20001.

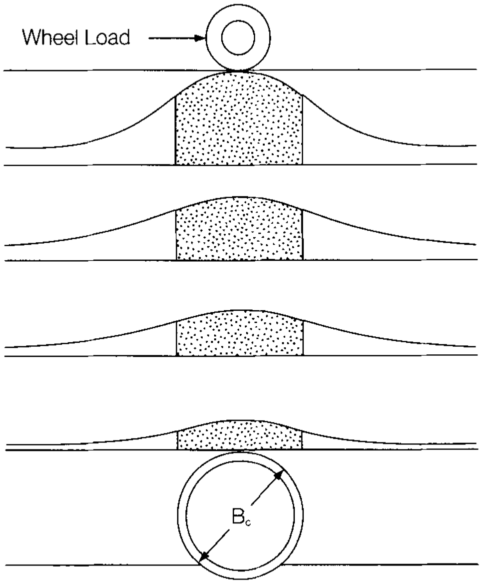



Figure 5-11 Live load distribution

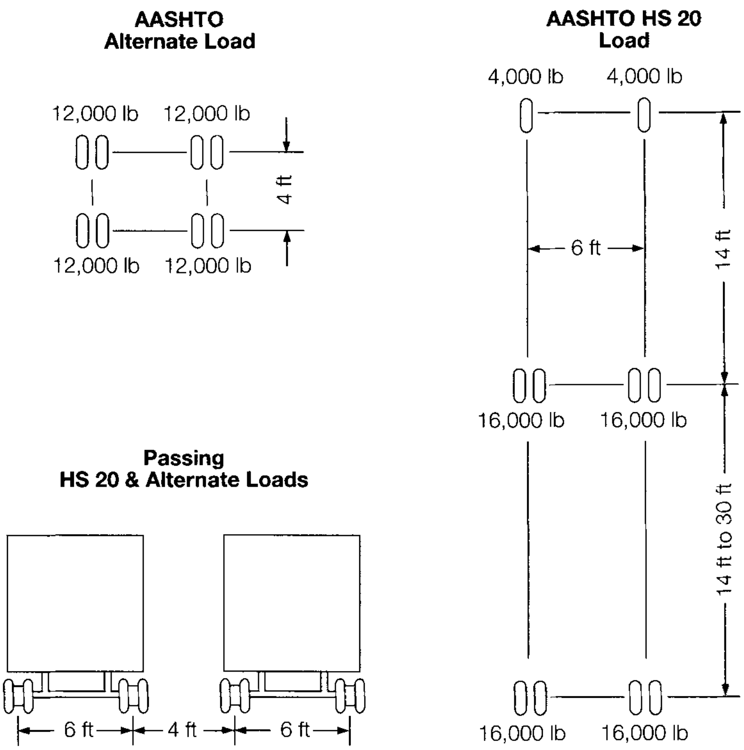



Figure 5-12 Live load spacing

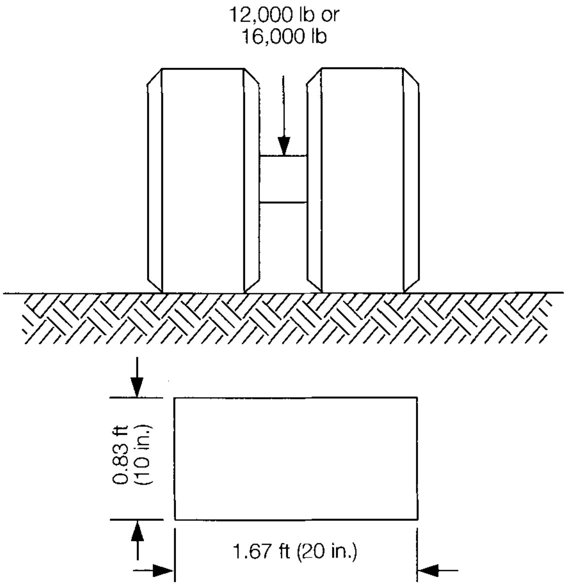



Figure 5-13 Wheel load surface contact area

Table 5-3 Impact factors for highway truck loads

| Height of Cover, $H$ | Impact Factor, $I_f$ |
|---|---|
| 0 to 1'-0" | 1.3 |
| 1'-1" to 2'-0" | 1.2 |
| 2'-1" to 2'-11" | 1.1 |
| 3'-0" and greater | 1.0 |

Table 5-4 Critical loading configurations

| Height of Cover, *ft* | Wheel Load, *lb* | $A_{LL}$, Distributed Load Area, *ft* × *ft* |
|---|---|---|
| $H < 1.33$ | 16,000 | (0.83 + 1.75H)(1.67 + 1.75H) |
| $1.33 < H < 4.10$ | 32,000 | (0.83 + 1.75H)(5.67 + 1.75H) |
| $4.10 < H$ | 48,000 | (4.83 + 1.75H)(5.67 + 1.75H) |

The total live load acting on the pipe is determined by the following formula:

$$W_T = w_L L S_L \qquad \text{(Eq 5-16)}$$

Where:

$W_T$ = total live load, lb
$w_L$ = average pressure intensity, lb/ft²
$L$ = length of $A_{LL}$, parallel to longitudinal axis of pipe, ft
$S_L$ = outside horizontal span of pipe or width of $A_{LL}$, transverse to longitudinal axis of pipe, whichever is less, ft

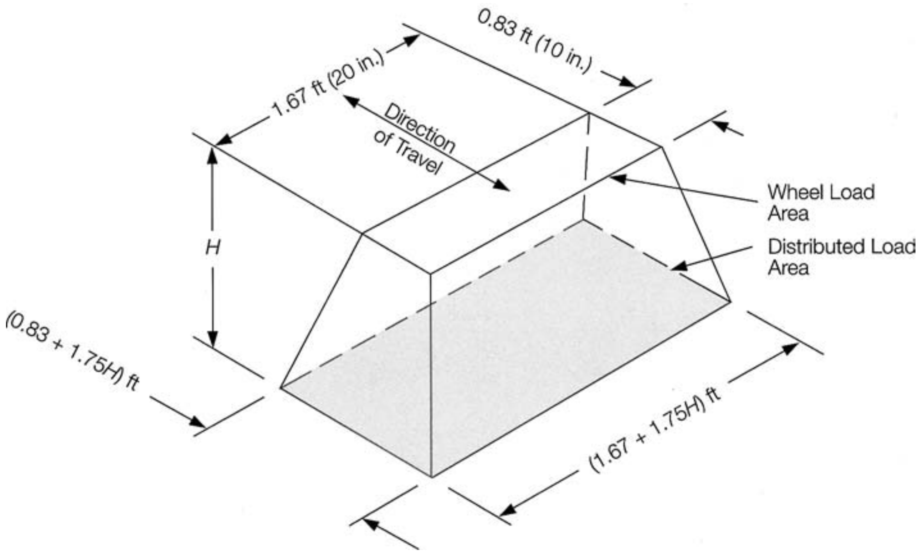



Figure 5-14 Distributed load area—single dual wheel

The live load acting on the pipe is determined by the following equation:

$$W_L = \frac{W_T}{L_e} \qquad \text{(Eq 5-17)}$$

Where:

$W_L$ = live load on pipe, lb/lin ft
$W_T$ = total live load, lb
$L_e$ = effective supporting length of pipe, ft

The buried concrete pipe is similar to a beam on continuous supports, and the effective supporting length of the pipe is assumed as in Figure 5-15 and determined by the following equation:

$$L_e = L + 1.75\,(3B_c/4) \qquad \text{(Eq 5-18)}$$

Where:

$L_e$ = effective supporting length of pipe, ft
$L$ = length of $A_{LL}$, parallel to longitudinal axis of pipe, ft
$B_c$ = outside diameter of the pipe, ft

Analysis of possible pipe alignments relative to load orientation confirms that the most critical loading can occur when the longitudinal pipe axis is either parallel or transverse to the direction of travel and centered under the distributed load area. Table 5-5 presents the maximum HS-20 highway live loads, including impact, imposed on circular pipe.

## Railroads

To determine the live load transmitted to a pipe installed under railroad tracks, the weight on the locomotive driver axles plus the weight of the track structure, including ballast, is considered to be uniformly distributed over an area equal to the length occupied by the drivers multiplied by the length of ties.

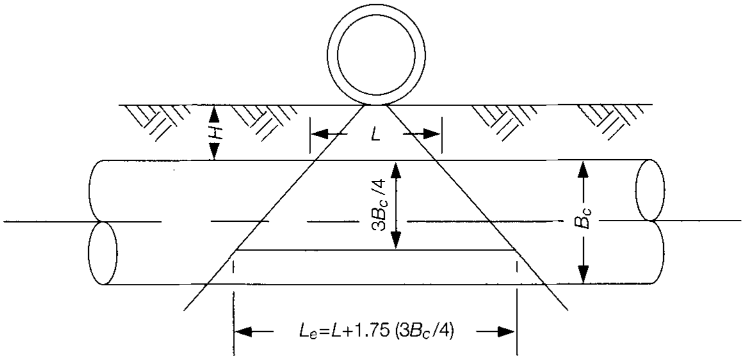



Figure 5-15 Effective supporting length of pipe

The AREMA* recommends a Cooper E80 loading with axle loads and axle spacing as shown in Figure 5-16. Based on a uniform load distribution at the bottom of the ties and through the soil mass, the live load transmitted to a pipe underground is computed by the equation

$$W_L = Cp_a B_c I_f \qquad \text{(Eq 5-19)}$$

Where:

$W_L$ = live load on pipe, lb/lin ft
$C$ = live load pressure coefficient (from Newmark's integration of the Boussinesq equation)
$p_a$ = intensity of the distributed load at the bottom of the ties, lb/ft$^2$
$B_c$ = outside diameter of the pipe, ft
$I_f$ = impact factor

Table 5-6 presents live loads in pounds per linear foot with a Cooper E80 design loading, track structure weighing 200 lb/lin ft (298 kg/m), and the locomotive load uniformly distributed over an area 8 ft × 20 ft (2.4 m × 6.1 m) yielding a uniform live load of 2,025 lb/ft$^2$ (97 kPa). In accordance with the AREMA *Manual for Railway Engineering,* an impact factor of 1.4 at zero cover decreasing to 1.0 at 10 ft (3 m) of cover is included in the table.

When design loading other than Cooper E80 is required for the design of a buried conduit, there is a simple procedure that can be used to determine the new live load figures for design. The procedures for both Cooper E72 and Cooper E90 design loads are as follows:

**Cooper E72 design load.** Select the live load figure from Table 5-6 that corresponds to the conduit diameter and depth of cover. Multiply this value by the ratio of 72/80 to obtain the Cooper E72 design load.

**Cooper E90 design load.** Select the live load figure from Table 5-6 that corresponds to the conduit diameter and depth of cover. Multiply this value by the ratio of 90/80 to obtain the Cooper E90 design load.

## Construction Loads

During grading operations, it may be necessary for heavy construction equipment to travel over an installed pipe. Unless adequate protection is provided, the pipe may be subjected to load concentrations in excess of the design loads. Before heavy

* American Railway Engineering and Maintenance-of-Way Association, 10003 Derekwood Lane, Suite 210, Lanham, MD 20706.

Table 5-5 Highway loads on circular pipe (pounds per linear foot)

| Pipe Size $D$ (*in.*) | Pipe OD $B_c$ (*ft*) | Height of Fill $H$ Above Top of Pipe, *ft* 0.5 | 1.0 | 1.5 | 2.0 | 2.5 | 3.0 | 3.5 | 4.0 | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | Pipe Size $D$ (*in.*) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 10 | 1.11 | 3,380 | 1,830 | 1,270 | 930 | 650 | 470 | 390 | 320 | 250 | 200 | 160 | 130 | 110 | 10 |
| 12 | 1.28 | 3,700 | 2,020 | 1,430 | 1,050 | 740 | 530 | 440 | 360 | 280 | 220 | 180 | 150 | 130 | 12 |
| 14 | 1.44 | 3,960 | 2,190 | 1,570 | 1,150 | 820 | 590 | 480 | 400 | 310 | 250 | 200 | 170 | 140 | 14 |
| 16 | 1.67 | 4,300 | 2,400 | 1,770 | 1,300 | 920 | 670 | 550 | 460 | 360 | 280 | 230 | 190 | 160 | 16 |
| 18 | 1.92 | 4,110 | 2,610 | 1,970 | 1,460 | 1,030 | 750 | 620 | 520 | 400 | 320 | 260 | 220 | 190 | 18 |
| 20 | 2.04 | 3,980 | 2,700 | 2,070 | 1,530 | 1,080 | 790 | 650 | 540 | 420 | 340 | 280 | 230 | 200 | 20 |
| 24 | 2.50 | 4,100 | 3,010 | 2,400 | 1,780 | 1,270 | 930 | 760 | 640 | 500 | 400 | 330 | 280 | 240 | 24 |
| 27 | 2.79 | 3,880 | 2,940 | 2,590 | 1,930 | 1,380 | 1,010 | 830 | 700 | 560 | 440 | 360 | 300 | 260 | 27 |
| 30 | 3.08 | 3,620 | 2,830 | 2,770 | 2,070 | 1,480 | 1,080 | 890 | 750 | 590 | 480 | 390 | 330 | 280 | 30 |
| 33 | 3.38 | 3,390 | 2,930 | 2,950 | 2,200 | 1,580 | 1,160 | 960 | 810 | 630 | 510 | 420 | 360 | 300 | 33 |
| 36 | 3.67 | 3,190 | 2,810 | 2,930 | 2,330 | 1,670 | 1,230 | 1,020 | 860 | 670 | 550 | 450 | 380 | 330 | 36 |
| 39 | 3.96 | 3,010 | 2,670 | 2,850 | 2,440 | 1,760 | 1,290 | 1,070 | 910 | 710 | 580 | 480 | 410 | 350 | 39 |
| 42 | 4.25 | 2,860 | 1,550 | 2,770 | 2,560 | 1,840 | 1,360 | 1,130 | 950 | 750 | 610 | 510 | 430 | 370 | 42 |
| 48 | 4.83 | 2,590 | 2,330 | 2,620 | 2,480 | 1,990 | 1,470 | 1,230 | 1,040 | 820 | 670 | 560 | 470 | 410 | 48 |
| 54 | 5.42 | 2,360 | 2,150 | 2,490 | 2,360 | 2,050 | 1,580 | 1,320 | 1,120 | 890 | 730 | 610 | 520 | 440 | 54 |
| 60 | 6.00 | 2,170 | 1,990 | 2,450 | 2,250 | 1,960 | 1,680 | 1,400 | 1,190 | 950 | 780 | 650 | 560 | 480 | 60 |
| 66 | 6.58 | 2,010 | 1,850 | 2,520 | 2,160 | 1,880 | 1,640 | 1,480 | 1,260 | 1,010 | 830 | 700 | 590 | 510 | 66 |
| 72 | 7.17 | 1,870 | 1,730 | 2,580 | 2,190 | 1,810 | 1,570 | 1,510 | 1,330 | 1,060 | 880 | 740 | 630 | 540 | 72 |
| 78 | 7.75 | 1,750 | 1,630 | 2,630 | 2,240 | 1,770 | 1,520 | 1,460 | 1,390 | 1,110 | 920 | 780 | 660 | 570 | 78 |
| 84 | 8.33 | 1,650 | 1,540 | 2,730 | 2,290 | 1,810 | 1,460 | 1,410 | 1,360 | 1,160 | 960 | 810 | 690 | 600 | 84 |
| 90 | 8.92 | 1,550 | 1,460 | 2,530 | 2,330 | 1,850 | 1,470 | 1,360 | 1,310 | 1,210 | 1,000 | 850 | 720 | 630 | 90 |
| 96 | 9.50 | 1,470 | 1,380 | 2,410 | 2,290 | 1,880 | 1,500 | 1,330 | 1,270 | 1,250 | 1,040 | 880 | 750 | 650 | 96 |
| 102 | 10.08 | 1,390 | 1,320 | 2,300 | 2,190 | 1,910 | 1,530 | 1,350 | 1,240 | 1,290 | 1,070 | 910 | 780 | 680 | 102 |
| 108 | 10.67 | 1,320 | 1,260 | 2,200 | 2,090 | 1,830 | 1,560 | 1,380 | 1,230 | 1,330 | 1,110 | 940 | 810 | 700 | 108 |
| 114 | 11.25 | 1,260 | 1,200 | 2,110 | 2,010 | 1,760 | 1,540 | 1,410 | 1,260 | 1,362 | 1,140 | 970 | 830 | 730 | 114 |
| 120 | 11.83 | 1,210 | 1,150 | 2,020 | 1,930 | 1,700 | 1,480 | 1,420 | 1,280 | 1,400 | 1,170 | 990 | 860 | 750 | 120 |
| 126 | 12.42 | 1,160 | 1,100 | 1,940 | 1,860 | 1,640 | 1,430 | 1,380 | 1,300 | 1,430 | 1,200 | 1,020 | 880 | 770 | 126 |
| 132 | 13.00 | 1,110 | 1,060 | 1,870 | 1,800 | 1,580 | 1,380 | 1,330 | 1,290 | 1,460 | 1,220 | 1,040 | 900 | 790 | 132 |
| 138 | 13.58 | 1,070 | 1,020 | 1,880 | 1,730 | 1,530 | 1,340 | 1,290 | 1,250 | 1,490 | 1,250 | 1,070 | 920 | 810 | 138 |
| 144 | 14.17 | 1,020 | 980 | 1,740 | 1,670 | 1,480 | 1,300 | 1,250 | 1,210 | 1,470 | 1,280 | 1,090 | 940 | 830 | 144 |

Data:
1. Unsurfaced roadway.
2. Loads — AASHTO HS-20, two 16,000-lb, dual-tired wheels, 4 ft on centers, or alternate loading, four 12,000-lb, dual-tired wheels, 4 ft on centers, with impact included.

Notes:
1. Interpolate for intermediate pipe sizes and/or fill heights.
2. Critical loads:
   a. For $H$ = 0.5 and 1.0 ft, a single 16,000-lb dual tired wheel.
   b. For $H$ = 1.5 through 4.0 ft, two 16,000-lb dual tired wheels, 4 ft on centers.
   c. For $H$>4.0 ft, alternate loading.
3. Truck live loads for $H$ = 10.0 ft or more are insignificant.

Table 5-6 Cooper E80 railroad loads on circular pipe (pounds per linear foot)

| Pipe Size *D* (in.) | Height of Fill *H* Above Top of Pipe (*ft*) | | | | | | | | | | | | | | | | | Pipe Size *D* (in.) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 12 | 14 | 16 | 18 | 20 | 25 | 30 | |
| 10 | 3,020 | 2,830 | 2,540 | 2,250 | 1,950 | 1,670 | 1,430 | 1,230 | 1,050 | 910 | 730 | 600 | 490 | 410 | 350 | 240 | 170 | 10 |
| 12 | 3,490 | 3,260 | 2,930 | 2,600 | 2,250 | 1,930 | 1,650 | 1,420 | 1,210 | 1,050 | 840 | 690 | 560 | 470 | 400 | 280 | 200 | 12 |
| 14 | 3,930 | 3,680 | 3,310 | 2,920 | 2,530 | 2,180 | 1,860 | 1,600 | 1,360 | 1,180 | 950 | 780 | 640 | 530 | 450 | 310 | 230 | 14 |
| 16 | 4,540 | 4,240 | 3,810 | 3,380 | 2,930 | 2,510 | 2,150 | 1,850 | 1,580 | 1,360 | 1,100 | 900 | 740 | 610 | 520 | 360 | 260 | 16 |
| 18 | 5,220 | 4,890 | 4,390 | 3,890 | 3,370 | 2,890 | 2,480 | 2,130 | 1,820 | 1,570 | 1,260 | 1,030 | 840 | 710 | 610 | 410 | 300 | 18 |
| 20 | 5,560 | 5,200 | 4,670 | 4,130 | 3,580 | 3,070 | 2,630 | 2,260 | 1,940 | 1,670 | 1,340 | 1,100 | 900 | 750 | 650 | 440 | 310 | 20 |
| 24 | 6,810 | 6,370 | 5,730 | 5,070 | 4,390 | 3,770 | 3,230 | 2,770 | 2,370 | 2,050 | 1,640 | 1,350 | 1,100 | 920 | 790 | 540 | 390 | 24 |
| 27 | 7,610 | 7,120 | 6,400 | 5,660 | 4,900 | 4,210 | 3,610 | 3,100 | 2,650 | 2,290 | 1,830 | 1,500 | 1,230 | 1,030 | 890 | 600 | 430 | 27 |
| 30 | 8,400 | 7,860 | 7,070 | 6,250 | 5,420 | 4,640 | 3,980 | 3,420 | 2,930 | 2,520 | 2,020 | 1,660 | 1,360 | 1,140 | 980 | 670 | 480 | 30 |
| 33 | 9,200 | 8,610 | 7,740 | 6,840 | 5,930 | 5,080 | 4,360 | 3,740 | 3,200 | 2,760 | 2,220 | 1,820 | 1,480 | 1,240 | 1,070 | 730 | 520 | 33 |
| 36 | 10,000 | 9,350 | 8,400 | 7,430 | 6,450 | 5,520 | 4,740 | 4,070 | 3,480 | 3,000 | 2,410 | 1,980 | 1,610 | 1,350 | 1,160 | 790 | 570 | 36 |
| 42 | 11,600 | 10,800 | 9,740 | 8,610 | 7,470 | 6,400 | 5,500 | 4,710 | 4,030 | 3,480 | 2,790 | 2,290 | 1,870 | 1,570 | 1,350 | 920 | 660 | 42 |
| 48 | 13,200 | 12,300 | 11,100 | 9,790 | 8,500 | 7,280 | 6,250 | 5,360 | 4,590 | 3,960 | 3,170 | 2,600 | 2,130 | 1,780 | 1,530 | 1,040 | 750 | 48 |
| 54 | 14,800 | 13,800 | 12,400 | 11,000 | 9,520 | 8,160 | 7,000 | 6,000 | 5,140 | 4,440 | 3,560 | 2,920 | 2,380 | 2,000 | 1,720 | 1,160 | 840 | 54 |
| 60 | 16,400 | 15,300 | 13,800 | 12,200 | 10,500 | 9,040 | 7,750 | 6,650 | 5,690 | 4,910 | 3,940 | 3,230 | 2,640 | 2,210 | 1,900 | 1,290 | 930 | 60 |
| 66 | 17,900 | 16,800 | 15,100 | 13,300 | 11,600 | 9,920 | 8,510 | 7,300 | 6,250 | 5,390 | 4,320 | 3,550 | 2,900 | 2,430 | 2,090 | 1,420 | 1,020 | 66 |
| 72 | 19,500 | 18,300 | 16,400 | 14,500 | 12,600 | 10,800 | 9,260 | 7,950 | 6,800 | 5,870 | 4,710 | 3,860 | 3,150 | 2,640 | 2,270 | 1,540 | 1,110 | 72 |
| 78 | 21,100 | 19,800 | 17,800 | 15,700 | 13,600 | 11,700 | 10,000 | 8,590 | 7,350 | 6,350 | 5,090 | 4,180 | 3,410 | 2,860 | 2,460 | 1,670 | 1,200 | 78 |
| 84 | 22,700 | 21,200 | 19,100 | 16,900 | 14,600 | 12,600 | 10,800 | 9,240 | 7,900 | 6,820 | 5,470 | 4,490 | 3,670 | 3,070 | 2,640 | 1,800 | 1,290 | 84 |
| 90 | 24,300 | 22,700 | 20,400 | 18,100 | 15,700 | 13,400 | 11,500 | 9,890 | 8,460 | 7,300 | 5,860 | 4,800 | 3,920 | 3,290 | 2,820 | 1,920 | 1,380 | 90 |
| 96 | 25,900 | 24,200 | 21,800 | 19,300 | 16,700 | 14,300 | 12,300 | 10,500 | 9,010 | 7,780 | 6,240 | 5,120 | 4,180 | 3,500 | 3,010 | 2,050 | 1,470 | 96 |
| 102 | 27,500 | 25,700 | 23,100 | 20,400 | 17,700 | 15,200 | 13,000 | 11,200 | 9,560 | 8,250 | 6,620 | 5,430 | 4,440 | 3,720 | 3,200 | 2,180 | 1,560 | 102 |
| 108 | 29,100 | 27,200 | 24,500 | 21,600 | 18,800 | 16,100 | 13,800 | 11,800 | 10,100 | 8,740 | 7,010 | 5,750 | 4,690 | 3,940 | 3,380 | 2,300 | 1,650 | 108 |
| 114 | 30,200 | 28,300 | 25,400 | 22,500 | 19,500 | 16,700 | 14,300 | 12,300 | 10,500 | 9,070 | 7,280 | 5,970 | 4,880 | 4,090 | 3,510 | 2,390 | 1,710 | 114 |
| 120 | 31,800 | 29,800 | 26,800 | 23,700 | 20,500 | 17,600 | 15,100 | 12,900 | 11,100 | 9,560 | 7,660 | 6,290 | 5,130 | 4,300 | 3,700 | 2,520 | 1,800 | 120 |
| 126 | 33,400 | 31,200 | 28,100 | 24,800 | 21,500 | 18,500 | 15,800 | 13,600 | 11,600 | 10,000 | 8,040 | 6,600 | 5,390 | 4,520 | 3,880 | 2,640 | 1,890 | 126 |
| 132 | 35,000 | 32,700 | 29,400 | 26,000 | 22,500 | 19,300 | 16,600 | 14,200 | 12,200 | 10,500 | 8,420 | 6,910 | 5,650 | 4,730 | 4,060 | 2,770 | 1,980 | 132 |
| 138 | 36,600 | 34,200 | 30,800 | 27,200 | 23,600 | 20,200 | 17,300 | 14,900 | 12,700 | 11,000 | 8,810 | 7,230 | 5,900 | 4,950 | 4,250 | 2,890 | 2,070 | 138 |
| 144 | 38,200 | 35,700 | 32,100 | 28,400 | 24,600 | 21,100 | 18,100 | 15,500 | 13,300 | 11,500 | 9,190 | 7,540 | 6,160 | 5,160 | 4,430 | 3,020 | 2,160 | 144 |

NOTE: Cooper E80 design loading consisting of four 80,000-lb axles spaced 5 ft on centers. Locomotive load assumed uniformly distributed over an area 8 ft × 20 ft. Weight of track structure assumed to be 200 lb/lin ft. Impact included. Height of fill measured from top of pipe to bottom of ties. Interpolate for intermediate pipe sizes and/or fill heights.

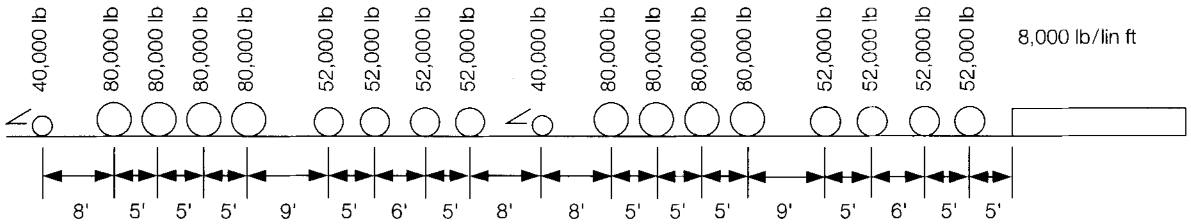



Figure 5-16 Cooper E80 design load

construction equipment is permitted to cross over a pipe, a temporary earth fill should be constructed to an elevation at least 3 ft (0.9 m) over the top of the pipe. The fill should be of sufficient width to prevent possible lateral displacement of the pipe.

## REFERENCES


American Concrete Pipe Association. 1990. *Concrete Pipe Design Manual.* Vienna, Va.: American Concrete Pipe Association.

———. 1988. *Concrete Pipe Handbook.* Vienna, Va.: American Concrete Pipe Association.

American Pipe and Construction Company. 1964. *Soils and Pipe Backfill,* Engineering Topics No. 5. Monterey Park, Calif.: American Pipe and Construction Company.

ARMCO Drainage and Metal Products Inc. 1948. *Handbook of Culvert and Drainage Practice.* Middleton, Ohio: ARMCO Drainage and Metal Products Inc.

Babcock, D.P. 1955. Simplified Design Methods for Reinforced Concrete Pipe Under Earth Fills. In *Proc. of Highway Research Board*. Washington, D.C.

Marston, A. 1930. The Theory of External Loads on Closed Conduits in the Light of Latest Experiments. Bulletin 96, Iowa State Engineering Experiment Station. Iowa State University.

Portland Cement Association. 1975. *Concrete Culverts and Conduits.* Skokie, Ill.: Portland Cement Association.

———. 1951a. *Concrete Sewers*. Skokie, Ill.: Portland Cement Association.

———. 1951b. Vertical Pressure on Culverts Under Wheel Loads on Concrete Pavement Slabs. No. ST65. Skokie, Ill.: Structural Bureau, Portland Cement Association.

Spangler, M.G. 1948. Underground Conduits, An Appraisal of Modern Research. Trans. ASCE. Paper No. 2337, Vol. 113.

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

US Department of Agriculture. 1958. Structural Design of Underground Conduits, Technical Release No. 5. Soil Conservation Service.

US Department of Commerce. 1963. Reinforced Concrete Pipe Culverts. Bureau of Public Records.

Water Pollution Control Federation. 1969. *Design and Construction of Sanitary and Storm Sewers.* WPCF Manual of Practice No. 9, ASCE Manuals and Reports on Engineering Practice No. 37. Washington, D.C.: Water Pollution Control Federation.

*Chapter* 6

# Bedding and Backfilling

## INTRODUCTION

The bedding of pipe has an important effect on its external-load-carrying capacity and can be a controlling factor in the design of the pipe. How loads from pipe, water, and cover will be supported by the bedding must be determined when the pipe is designed. If the cover or other external load on the pipe is high, the degree and uniformity of bedding support can have a substantial influence on the required pipe strength.

The exterior pipe mortar or concrete provides the primary corrosion protection for the pipe reinforcing steel. It is important to remove large rocks and debris from backfill placed near the pipe to avoid damage to the pipe exterior and the consequent reduction in corrosion resistance. The backfill and trench should also be free of metallic debris to avoid electrical interference if corrosion monitoring is performed in the future.

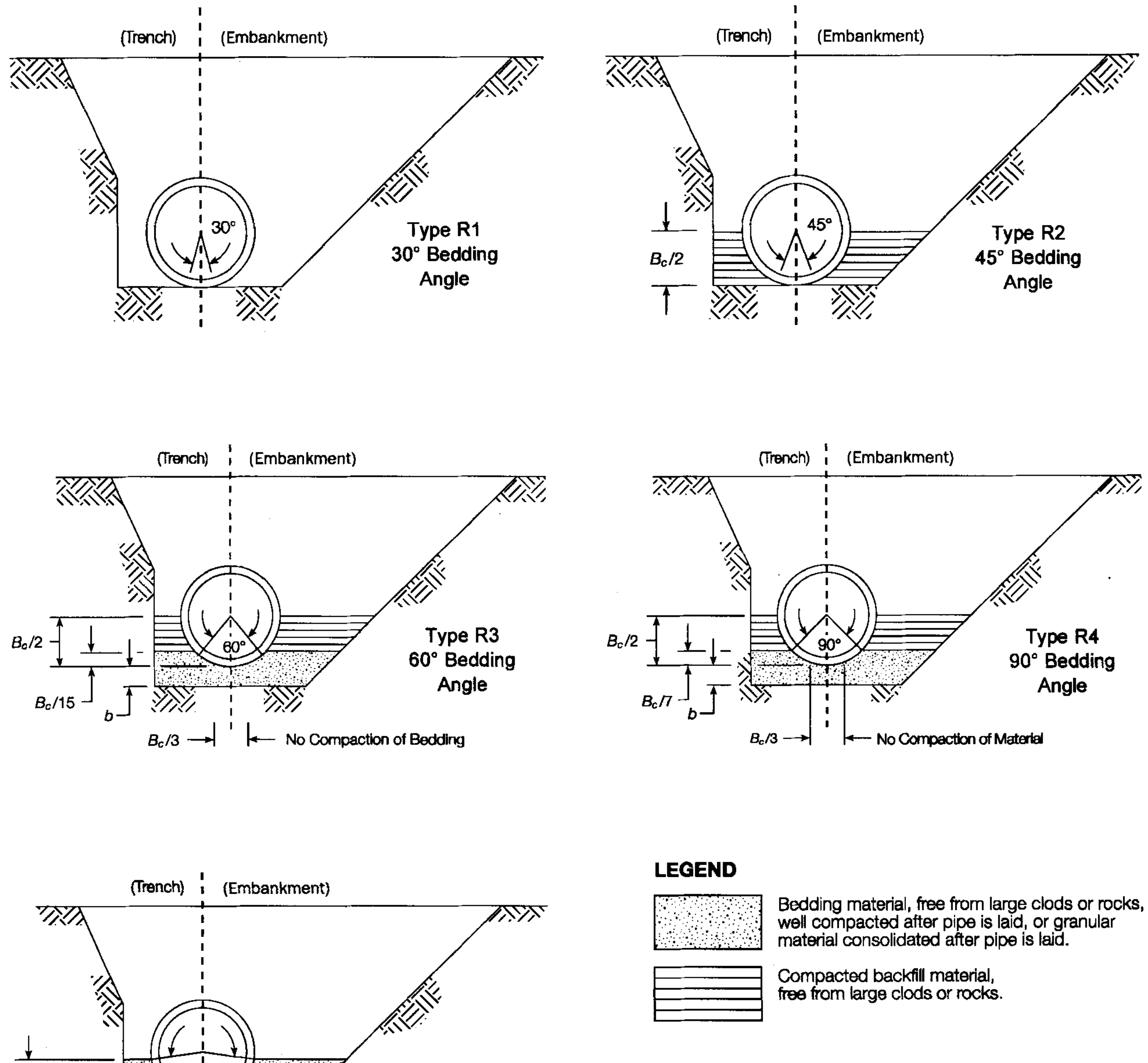
### Trench Width

Trench width is measured at the top of the pipe. The minimum specified trench width should allow sufficient access on each side of the pipe for any required consolidation of bedding under the pipe haunches. The minimum clearance between the pipe and the trench wall should be no less than 9 in. (229 mm). If soil and installation conditions and safety standards permit, the maximum trench width should also be limited to reduce the soil load that the pipe must support.

## RIGID PIPE

In general, it is more economical to design rigid pipe (ANSI/AWWA C300-, C301-, and C302-type pipe) to accommodate external loading with minimal support than it is to require the pipe to be installed with a highly compacted bedding. It is important to avoid laying the pipe on a hard or unyielding surface, such as rock, hard clay, or shale. Usually, fines from the excavation can be placed in the trench bottom to provide sufficient support for the pipe. When appropriate material is not available from the excavation, it may be necessary to import material to provide a cushion.

Typical bedding conditions and the approximate angle of support for each are shown in Figure 6-1. These various conditions of bedding and backfilling affect the




**Depth of Bedding Material Below Pipe**

| Pipe ID | *b* (minimum) |
|---|---|
| 27 in. and smaller | 3 in. |
| 30 to 60 in. | 4 in. |
| 66 in. and larger | 6 in. |

$B_c$ = outside diameter of pipe

NOTES:
1. Embankment condition indicates the trench width at the top of the pipe exceeds transition width.
2. For bedding types R1 and R2, the trench bottom shall be overexcavated and bedding material shall extend to depth *b* below the bottom of pipe if subgrade is rock or other unyielding material.

Figure 6-1 Bedding and backfill for rigid pipe structural design of the pipe as shown in chapter 7. When rigid concrete pipelines less than 72 in. (1,830 mm) in diameter are subjected to moderate earth covers (10 ft [3 m] or less), the type R1 or R2 beddings represent a cost-effective support when both pipe and installation costs are considered. For more severe external loading situations, types R3, R4, and R5 reflect beddings that provide additional pipe support. Designers should consider the cost of increasing pipe-load supporting strength, the cost of increasing the supporting strength of the bedding, or combinations of pipe strength and bedding support that provide the most economical installation.

# SEMIRIGID PIPE

Concrete pressure pipe manufactured in accordance with ANSI/AWWA C303 is considered semirigid in larger diameters because as it deflects it develops its ability to support external loads both from the support of the surrounding soil and from its inherent strength.

The smaller diameters of ANSI/AWWA C303-type pipe (24 in. [610 mm] and smaller) are designed as semirigid, but may be considered as rigid when selecting bedding because they have sufficient strength to support large external loads without injurious deflection or dependence on support from surrounding soil. As the diameter of ANSI/AWWA C303-type pipe becomes larger, support from the surrounding soil becomes increasingly important, as demonstrated in chapter 7.

The surrounding soil support for semirigid pipe is measured in terms of the bedding constant $k$ and the modulus of soil reaction $E'$. Bedding constant $k$ is dependent on the angle of support beneath the pipe.* Modulus of soil reaction $E'$ is a measure of the lateral support provided by the soil. Values of $E'$ vary widely depending on the native soils, depth of cover, backfill materials, and the degree of densification of the backfill.†

## Soil Compaction

When specifying the amount of compaction required, it is very important to consider the degree of soil compaction that is economically obtainable in the field for a particular installation. The density and supporting strength of the native soil must be equal to or greater than that of the compacted backfill. The densification of the backfill envelope must include the haunches under the pipe to control both the horizontal and vertical pipe deflections. Specifying an unobtainable soil compaction value can result in inadequate support and injurious deflection. Therefore, a conservative assumption of the supporting capability of a soil is recommended, and good field inspection should be provided to verify that design assumptions are met.

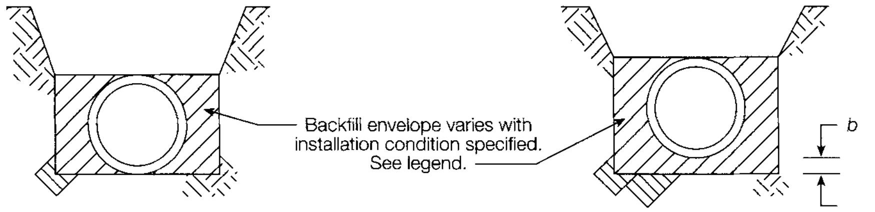
## Bedding Details

Bedding details for ANSI/AWWA C303-type pipe are shown in Figure 6-2, together with design values for $k$. Concrete cradles are not ordinarily used for semirigid pipe.

The type of bedding may be chosen after determining how much soil support must be provided to limit pipe deflection as required in chapter 7. On longer projects, where depths of cover vary significantly, it may be most economical to require less bedding support in the areas of less cover and more bedding support only where necessary. When safety standards permit, it is recommended that trench widths be kept as narrow as possible to minimize the external load while still providing adequate working space.

* For a discussion of the correlation between bedding angle and bedding constant $k$, see Bull. 153 (December 1941), Iowa State Experiment Station, by Spangler (1982).

† For correlation of $E'$ values with depth of cover, degree of densification, and soil types, reference is made to the papers "$E'$ and Its Variation with Depth," by Hartley and Duncan (1987), and "Modulus of Soil Reaction Values for Buried Flexible Pipe," by Howard (1977).




Yielding Material or Ordinary Soil for $D \leq 24$ in.

In Rock or Unyielding Material for $D \leq 24$ in. or All Installations for $D > 24$ in.

**Depth of Bedding Material Below Pipe**

| *D* | *b (minimum)* |
|---|---|
| 27 in. and smaller | 3 in. |
| 30 to 60 in. | 4 in. |
| 66 in and larger | 6 in. |

Note: *b* includes 1-in.-layer loose material below the pipe.

**Bedding Design Values**

| | |
|---|---|
| S1 Bedding: | $k = 0.105$ |
| S2 Bedding: | $k = 0.098$ |
| S3 Bedding: | $k = 0.090$ |
| S4 Bedding: | $k = 0.085$ |

**Backfill Envelope Legend**

S1 Bedding: Fine-grained, low-plasticity backfill, CL, ML, CL–ML, free from large clods or rocks larger than *b*/4, lightly consolidated to remove large voids. Soil density less than 85% Standard Proctor (ASTM D698). (Not recommended for pipe larger than 24 in.)

S2 Bedding: Fine-grained, low-plasticity backfill, CL, ML, CL–ML, free from large clods or rocks larger than *b*/4, placed with light to moderate compaction. Minimum density = 85% Standard Proctor (ASTM D698). (Not recommended for pipe larger than 36 in.)

S3 Bedding: Fine-grained, low-plasticity backfill, CL, ML, CL–ML, free from large clods or rocks larger than *b*/4, placed with moderate to heavy compaction. Minimum density = 90% Standard Proctor (ASTM D698).

S4 Bedding: Fine-grained, low-plasticity backfill, CL, ML, CL–ML, free from large clods or rocks larger than *b*/4, placed with moderate to heavy compaction. Minimum density = 95% Standard Proctor (ASTM D698).

NOTE: Backfill envelope material is assumed to be fine-grained soils with less than 25% sand content, which is generally of lower structural quality. For the modulus of soil reaction E′ of the fine-grained material previously described and the E′ of backfill material of higher structural quality, refer to the Hartley and Duncan (1987) and the Howard (1977) references cited in this chapter.

Figure 6-2 Bedding and backfill for semirigid pipe

## UNSTABLE FOUNDATIONS

Saturated soils, such as those found in many areas along coastlines or in swamps, are often unstable or nearly liquid. Pile-supported piers or other pipe foundations should be considered but are not necessarily required for pipe installed in saturated soils. The composite unit weight of concrete pressure pipe filled with water ranges from about

80 to 95 lb/ft$^3$ (1,281 to 1,522 kg/m$^3$), which is usually less than in-situ soil material. If an unstable soil is in equilibrium, a ditch may be dug in it and refilled with pipe and soil with a total weight approximately equal to the weight of the excavated material. The pipe would then "float" in the unstable soil at the installed depth and would not sink or move. For larger pipe or lower covers, consideration should always be given to the differences in full versus empty pipe buoyancy. See Eq 11-1 in chapter 11 for checking the buoyancy of empty pipe.

Pipe in unstable swamp or bog material can be supported on bedding that will spread the pipe load over the trench bottom instead of concentrating it along the invert. Successful installations using a spread bedding of seashells or granular material 1½ to 2 pipe diameters wide have been used, as have light timber mats with seashells or granular material over the mat for pipe bedding. A geotextile envelope surrounding a seashell or granular bedding can also spread the pipe load over the unstable trench bottom.

A timber mat, crushed rock, shell, sheet pile, or geotextile fabric foundation may be required to support construction equipment or to hold the ditch open. The pipe may not need special support in unstable soils because loading is more radially compressive or hydrostatic rather than vertically downward.

"Floating" installations work well in soils that will remain in equilibrium. If the soil can be expected to shift or not remain static, other installation procedures must be used. Loads and moments applied to pipe by shifting soils can become extreme, so elevating the pipe on a bridge or piers or otherwise rerouting the pipeline around the shifting soil may provide the most economical installation.

## REFERENCES


American Concrete Pipe Association. 1974. *Concrete Pipe Handbook.* Vienna, Va.: American Concrete Pipe Association.

Hartley, J.D., and J.M. Duncan. 1987. $E'$ and Its Variation with Depth. *Jour. of Transportation Engineering*, ASCE, Vol. 113, No. 5.

Howard, A.K. 1977. Modulus of Soil Reaction Values for Buried Flexible Pipe. *Jour. of the Geotechnical Engineering Division*, ASCE, Vol. 103, No. GT1, Proceedings Paper 12,700.

Spangler, M.G. 1982. *Soil Engineering.* 4th ed. Harper & Row Publishers.

Water Pollution Control Federation. 1969. *Design and Construction of Sanitary and Storm Sewers.* WPCF Manual of Practice No. 9, ASCE Manuals and Reports on Engineering Practice No. 37. Washington, D.C.: Water Pollution Control Federation.

Journal of Transportation Engineering, ASCE, Vol. 113, No. 5, September, 1987, pp. 538–553, and "Modulus of Soil Reaction Values for Buried Flexible Pipe," by Amster K. Howard, Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, No. GT 1, Proceedings Paper 12,700, January 1977, pp. 33-43.

Chapter 7

# Design of Reinforced Concrete Pressure Pipe

This chapter presents three design procedures with examples, one for each of the reinforced concrete pressure pipe types described in chapter 2, i.e., ANSI/AWWA C300-, ANSI/AWWA C302-, and ANSI/AWWA C303-type pipe. The design of prestressed concrete pressure pipe, ANSI/AWWA C301-type pipe, is covered separately in ANSI/AWWA C304.

The method of design for reinforced concrete pressure pipe includes two basic procedures. The first is to establish the amount of wall strength required to resist internal hydrostatic pressure acting alone. The second is to evaluate the influence of external loads. ANSI/AWWA C300- and C302-type pipe are designed for external loads in various combinations with internal pressures using rigid pipe concepts with controlled steel and concrete stresses. ANSI/AWWA C303-type pipe is designed for external loads using semirigid pipe concepts and controlled pipe deflections.

## INFORMATION REQUIRED FOR PIPE DESIGN

The design of reinforced concrete pressure pipe requires the following information:

- pipe diameter $D$
- internal pressure criteria (chapters 3 and 4)
  - working pressure $P_w$
  - field test pressure $P_{ft}$
  - surge pressure $P_t$
- external earth load criteria (chapter 5)
  - weight of earth cover $H$
  - trench or projecting condition $B_d$ or $p$
  - soil properties $w$, $K\mu$, $K\mu'$, and $r_{sd}$

- external live load criteria (chapter 5)
  - type of load: highway, rail, etc.
  - impact factor
- installation criteria (chapter 6)
  - compaction of bedding and backfill
  - bedding angle
  - moment, thrust, and shear coefficients, Paris or Olander (for ANSI/AWWA C300- and C302-type pipe)
  - modulus of soil reaction $E'$ (for ANSI/AWWA C303-type pipe)

To make the pipe as economical as possible, a pipeline is usually subdivided, either by the engineer or the pipe supplier, into sections requiring different design pressures and external-load capacities.

Pipe wall minimum dimensions, steel material and placement requirements, and minimum concrete design strengths are determined by each AWWA standard. Any special requirements must be stated in the project specification. Some special designs may only be feasible for longer pipelines due to the availability and cost of unusual manufacturing equipment.

## DESIGN PROCEDURE FOR RIGID PIPE (ANSI/AWWA C300- AND C302-TYPE PIPE)

The rigid-pipe design procedure involves the following steps for each subdivision of the pipeline:

Step 1 Calculate the total circumferential steel area required to resist internal pressure only using the hoop tension equation for working pressure and working pressure plus surge pressure. The hydrostatic design steel area is the maximum $A_s$ obtained from Eq 7-1 and 7-2 or Eq 7-3 and 7-4. Eq. 7-2 and 7-4 may be used to check $A_s$ for field test pressure by replacing $(P_w + P_t)$ with $P_{ft}$.

For ANSI/AWWA C300-type pipe

$$A_s = 6P_wD_{yi}/f_s \qquad \text{(Eq 7-1)}$$

$$A_s = 6(P_w + P_t)D_{yi}/f_{st} \qquad \text{(Eq 7-2)}$$

For ANSI/AWWA C302-type pipe

$$A_s = 6P_wD/f_s \qquad \text{(Eq 7-3)}$$

$$A_s = 6(P_w + P_t)D/f_{st} \qquad \text{(Eq 7-4)}$$

Where:

$A_s$ = cross-sectional area of circumferential steel reinforcement, including any steel cylinder, in square inches per foot of pipe wall

$P_w$ = internal working pressure established by the hydraulic gradient or by specified static pressure, whichever is greater, psi

$D_{yi}$ = inside diameter of the steel cylinder, in.

$f_s$ = average circumferential stress, psi, in the steel reinforcement, including any cylinder, when the pipe is subjected to the internal working pressure

For ANSI/AWWA C300-type pipe

$f_s$ = 16,500 psi

For ANSI/AWWA C302-type pipe

$f_s$ = $16{,}500 - 75P_w$

$D$ = pipe inside diameter, in.

$P_t$ = surge pressure in excess of working pressure, psi

$P_{ft}$ = field test pressure, psi

$f_{st}$ = average circumferential stress, psi, in the steel reinforcement, including any cylinder, when the pipe is subjected to working plus surge pressure, or field test pressure

For ANSI/AWWA C300-type pipe

$f_{st}$ = 21,000 psi

For ANSI/AWWA C302-type pipe

$f_{st}$ = 16,500 psi

Step 2 (ANSI/AWWA C302-type pipe only) For the selected wall thickness of ANSI/AWWA C302-type pipe, use the hoop tension equation to calculate the circumferential tensile stress in the concrete of the pipe wall resulting from working pressure plus surge pressure. The concrete strength, $f'_c$, or the wall thickness must be increased if the tensile stress exceeds the allowable.

For ANSI/AWWA C302-type pipe

$$f_{ct} = (P_w + P_t)D/(2t) \qquad \text{(Eq 7-5)}$$

Where:

$f_{ct}$ = average circumferential tensile stress in the pipe wall concrete, with no allowance for steel reinforcement, psi, not to exceed $4.5\sqrt{f'_c}$ psi

$f'_c$ = 28-day compressive strength of the pipe wall concrete, psi

$t$ = pipe wall thickness, in.

and other variables are as defined under step 1.

Step 3 Calculate the pipe weight and water weight.

Step 4 Calculate the external earth load on the pipe.

Step 5 Calculate the external live load, if any, on the pipe.

External dead loads and live loads must be computed in accordance with recognized and accepted theories, such as those presented in chapter 5.

Step 6 Calculate moments and thrusts for each load on the pipe, including internal pressure. Values at the invert and at the side are required. For normal loading conditions, the crown values do not control the design.

Check the radial tension and shear capacity of the pipe wall chosen (Eq 7-21 and 7-22) for loading condition 4. For the definition of radial tension (slabbing) and shear (diagonal tension), refer to Heger and McGrath (1983) and Olander (1950), respectively, in the references. Experience indicates that for normal loading conditions, radial tension or shear stresses do not control the design if the minimum wall thickness is equal to or greater than the minimum wall thicknesses shown in Table 1 and Table 4 of ANSI/AWWA standards C300 and C302, respectively.

For ANSI/AWWA C300-type pipe, radial tension and shear capacities of the pipe wall should be checked for the inner cage with zero cylinder thickness except for microtunneling applications or shallow covers, where the cylinder alone may provide the inner steel.

The coefficients for moments, thrusts, and shear must be from recognized and accepted theories, such as those presented by Paris (1921) and Olander (1950). The bedding angle used in design must be compatible with the installation criteria specified by the purchaser.

Step 7 Calculate the required circumferential steel area for the invert and the side for each of the first three conditions shown below. Calculate minimum bar area, $A_{bi}$, for condition 4 assuming zero cylinder thickness for ANSI/AWWA C300-type pipe.

Combined load design means the pipe is designed to resist the flexural and axial stresses from each of the following conditions:

- Condition 1: a combination of working pressure, dead loads (earth, pipe, and water), and live loads
- Condition 2: a combination of dead loads (earth, pipe, and water) and live loads with zero internal pressure
- Condition 3: a combination of working pressure, surge pressure, and dead loads (earth, pipe, and water). This condition may be used for field test pressure also.
- Condition 4: a combination of earth load, pipe weight, and live load.

The reinforced concrete design shall be according to the applicable provisions of ACI* 318, Building Code Requirements for Structural Concrete, with either the Strength Design method or the Alternate Design method (with permissible service load stresses) being used.

For the Alternate Design method, the service (design) loads are used and the permissible service load stresses are as follows: the calculated compressive stress in the concrete shall not exceed $0.45f'_c$ (the specified 28-day strength), and the allowable tensile stress in the reinforcement shall not exceed 22,000 psi (152 MPa) at the sides of the pipe and 20,000 psi (138 MPa) for the crown and invert of ANSI/AWWA C300-type pipe, or 22,000 psi (152 MPa) for the crown and invert of ANSI/AWWA C302-type pipe.

The Strength Design method procedures for ANSI/AWWA C300 and C302 are demonstrated in this chapter.

For ANSI/AWWA C300-type pipe, the load factor shall be 1.8 for conditions 1 and 2, 1.35 for condition 3, and 1.2 for condition 4. For ANSI/AWWA C302-type pipe, the load factor shall be 1.8 for conditions 1, 2, and 3, and 1.5 for condition 4. For both ANSI/AWWA C300- and C302-type pipe, the capacity reduction factor shall be 1.0, and an equivalent rectangular concrete stress distribution shall be used.

* American Concrete Institute, P.O. Box 9094, Farmington Hills, MI 48333.

For ANSI/AWWA C300-type pipe, for conditions 1, 2, and 3, the design yield strength of the steel reinforcement shall not exceed 40,000 psi (276 MPa) for the sides of the pipe, and 36,000 psi (248.3 MPa) for the crown and invert of the pipe; for condition 4 the yield strength shall not exceed 40,000 psi (276 MPa).

For ANSI/AWWA C302-type pipe, for all four conditions, the design yield strength of the steel reinforcement shall not exceed 40,000 psi (276 MPa).

The reinforcement for ANSI/AWWA C300-type pipe consists of a steel cylinder and one or more cages. A typical pipe wall cross section is shown in chapter 2. The cages are either circular or elliptical in shape and may be used singly or in combination. The cross-sectional area of the circumferential bar or wire reinforcement per linear foot of pipe shall be no less than 40 percent of the total area of reinforcement per linear foot of pipe. When the ANSI/AWWA C300-type pipe reinforcement consists of a combination containing an elliptical cage, the cross-sectional area of the circular reinforcement, including the steel cylinder, shall be no less than that determined by Eq 7-1, using an allowable steel stress $f_s$ of 25,000 psi (172 MPa).

Steel reinforcement for ANSI/AWWA C302-type pipe consists of a single elliptical cage, one or more circular cages, or a combination of an elliptical cage and one or more circular cages. At least one of the cages must be circular in ANSI/AWWA C302-type pipe designed for a working pressure of more than 22 psi (152 kPa) or in all ANSI/AWWA C302-type pipe larger than 72 in. (1,830 mm) in diameter. An inner circular cage and an outer circular cage must be used and may be combined with an elliptical cage in ANSI/AWWA C302-type pipe designed for working pressures of more than 45 psi (310 kPa). When the reinforcement for ANSI/AWWA C302-type pipe consists of a combination of circular and elliptical cages and the working pressure exceeds 22 psi (152 kPa), the total cross-sectional area of the circular cage or cages shall not be less than that determined by Eq 7-3, using an allowable steel stress $f_s$ of 25,000 psi (172 MPa).

For design of single-cage circular reinforcement in ANSI/AWWA C302-type pipe, the centroid of the steel reinforcement is assumed as the centerline of the pipe wall.

Step 8 Select the controlling maximum steel area for the invert (inner) and side (outer). The total steel area must be equal to or greater than the steel area required for the hydrostatic design. Increase the inner area, outer area, or both sides to meet the required total.

Step 9 Select appropriate bars or fabric to meet the design circumferential steel areas and spacing. For ANSI/AWWA C300-type pipe, check to ensure the area of rod reinforcement is at least 40 percent of the total circumferential steel area. Check concrete cover over steel.

The minimum design clear concrete cover for either an outer circular cage or for an elliptical cage at the horizontal axis for ANSI/AWWA C300-type pipe is 1⅛ in. (29 mm). This provides a concrete cover manufacturing tolerance of ⅛ in. (3 mm). For the steel cylinder, the design clear cover shall be the nominal lining thickness designated by the manufacturer, but no less than the minimum lining thickness shown in ANSI/AWWA C300.

The minimum clear spacing between the circumferential reinforcing members in ANSI/AWWA C300-type pipe is 1¼ in. (32 mm) or 1⅓ times the maximum aggregate size, whichever is greater. The maximum center-to-center spacing of circumferential reinforcing members is ¾ times the wall thickness or 4 in. (102 mm), whichever is smaller.

For ANSI/AWWA C302-type pipe with either double circular cages, an elliptical cage, or a combination of an elliptical cage and one or more circular cages, the design depth to the centroid of the tensile steel reinforcement shall provide a minimum clear concrete cover of ⅛ in. (3 mm) (manufacturing tolerance) more than the minimum specified in ANSI/AWWA C302.

The minimum clear spacing between the circumferential reinforcing members in ANSI/AWWA C302-type pipe is 1¼ in. (32 mm) or 1⅓ times the maximum aggregate size, whichever is greater. The maximum center-to-center spacing of circumferential reinforcing members is 2 in. (51 mm) for pipe with wall thicknesses less than 3 in. (76 mm). For pipe with wall thicknesses 3 in. (76 mm) or more, the maximum center-to-center spacing is ¾ times the wall thickness or 4 in. (102 mm), whichever is smaller.

Step 10 Check the reinforcement areas to be sure that the minimum requirement of ACI 318 for flexural reinforcement (Sec. 10.5) is being provided. The ACI 318 minimum reinforcement requirement is the larger of $3\sqrt{f'_c}\,bd/f_{yb}$ and $200bd/f_{yb}$, where $b$ = 12 in.

Step 11 If ANSI/AWWA C302-type pipe is to be installed on supports or in any other condition that would create longitudinal bending, refer to chapter 10, Design of Pipe on Piers.

## Example Calculations for ANSI/AWWA C300-Type Pipe

**Project requirements**

- pipe diameter $D$, in. = 60
- internal pressure
  - working pressure $P_w$, psi = 75
  - surge pressure $P_t$, psi = 25
- external earth load
  - height of earth cover $H$, ft = 5
  - trench width $B_d$, ft = $B_c$ + 3.5
  - soil properties $K\mu' = K\mu$ = 0.165
  - unit weight of soil $w$, lb/ft$^3$ = 120
- external live load $W_L$
  - AASHTO highway load = HS-20
  - AASHTO impact factor $I_f$ = 1.0
- installation criteria
  - bedding angle, in degrees = 90
  - moment, thrust, and shear coefficients = Olander

**Design information from specifications or manufacturer**

- pipe wall thickness $t$, in. = 6
- cylinder inside diameter (ID) $D_{yi}$, in. = 63.38
- minimum cylinder thickness $t_y$ = 16 ga. (0.0598 in.)
- design yield strength for steel in crown and invert $f_{yi}$, psi = 36,000
- design yield strength for steel at springline $f_{yb}$, psi = 40,000
- average circumferential stress in steel with pipe at working pressure $f_s$, psi = 16,500
- average circumferential stress in steel with pipe at working plus surge pressure $f_{st}$, psi = 21,000
- minimum design concrete strength $f'_c$, psi = 4,500

Step 1 Calculate the total circumferential steel area required to resist internal pressure.

$$f_s = 16{,}500 \text{ psi}$$

Using Eq 7-1 $$A_s = \frac{6P_w D_y i}{16,500} = \frac{6(75)(63.38)}{16,500} = 1.73 \text{ in.}^2/\text{lin ft}$$

Using Eq 7-2

$$A_s = \frac{6(P_w + P_t)D_{yi}}{21,000} = \frac{6(75 + 25)(63.38)}{21,000} = 1.81 \text{ in.}^2/\text{lin ft}$$

Step 2 (Used for ANSI/AWWA C302-type pipe only.)

Step 3 Calculate the pipe weight and water weight.

Pipe weight, $W_p = (\pi/4)[(\text{OD pipe})^2 - (\text{ID pipe})^2]$ (pipe material unit weight)

$$W_p = (\pi/4)[(72/12)^2 - (60/12)^2]\,(160 \text{ lb/ft}^3) = 1{,}382 \text{ lb/lin ft}$$

Water weight, $W_w = (\pi/4)(\text{ID pipe})^2$ (water unit weight)

$$W_w = (\pi/4)(60/12)^2\,(62.4 \text{ lb/ft}^3) = 1{,}225 \text{ lb/lin ft}$$

Step 4 Calculate the external earth load on the pipe. Refer to Equations 5-1 and 5-3 for formulas and definitions of variables.

OD pipe, $B_c$ = 60 + 2(6) = 72 in. or 6.0 ft

$B_d = B_c + 3.5 = 6.0 + 3.5 = 9.5$ ft

$W_d = C_d w B_d^2,\ C_d = (1 - e^{-2K\mu'(H/B_d)})/(2K\mu')$

$W_d = [(1 - e^{-2(0.165)(5/9.5)})/(2(0.165))](120)(9.5)^2$

$W_d$ = 5,231 lb/lin ft

$W_c = C_c w B_c^2$, must find where height of plane of equal settlement $H_e$ equals depth of cover $H$ before solving for $C_c$. Set $H = H_e$ in Eq 5-6 and use $r_{sd} = 0.5$ and $p = 1.0$.

$$\left[\frac{1}{2(0.165)} + \left(\frac{H_e}{6.0} - \frac{H_e}{6.0}\right) + \frac{0.5}{3}\right]\frac{e^{2(0.165)(H_e/6.0)} - 1}{2(0.165)}$$
$$+\frac{1}{2}\left(\frac{H_e}{6.0}\right)^2 + \frac{0.5}{3}\left(\frac{H_e}{6.0} - \frac{H_e}{6.0}\right)e^{2(0.165)(H_e/6.0)}$$
$$-\frac{1}{2(0.165)}\left(\frac{H_e}{6.0}\right) - \frac{H_e}{6.0}\left(\frac{H_e}{6.0}\right) = 0.5\left(\frac{H_e}{6.0}\right)$$

Solving Equation 5-6, $H_e$ = 12.0 ft > $H$ = 5 ft. Therefore, Eq 5-4A is used to calculate $C_c$.

$$C_c = \frac{e^{2K\mu(H/B_c)} - 1}{2K\mu}$$

$$= \frac{e^{2(0.165)(5/6.0)} - 1}{2(0.165)}$$

$$= 0.959$$

$W_c = 0.959\,(120)(6.0)^2$

$W_c = 4{,}144$ lb/lin ft

$W_c \le W_d$ (transition trench width has been exceeded)

$\therefore W_d = 4{,}144$ lb/lin ft

Step 5 Calculate the external live load, if any, on the pipe.

From Table 5-5, live load $W_L = 950$ lb/lin ft at $H = 5.0$ ft.

Step 6 Calculate moments and thrusts for each load on the pipe, including internal pressure. Calculate shear for pipe weight, earth load, and live load.

Using the coefficients from Figure 7-1, calculate the moment, thrust, and shear in the pipe wall as follows:

Moment = (Coefficient)(Load)(Radius)

Thrust = (Coefficient)(Load)

Shear = (Coefficient)(Load)

The radius is calculated as follows:

Radius = $r = 0.5(D + t) = 0.5(60 + 6) = 33$ in.

At invert (180° from crown):

| Component | Load *lb/lin ft* (1) | Coefficients for 90° Bedding: Thrust (2) | Coefficients for 90° Bedding: Moment (3) | *F* Thrust *lb/lin ft* Col. 1 × Col. 2 | *M* Moment *in.-lb/lin ft* Col. 1 × Col. 3 × *r* |
|---|---|---|---|---|---|
| Pipe | $W_p = 1{,}382$ | 0.207 | 0.121 | 286 | 5,518 |
| Water | $W_w = 1{,}225$ | –0.270 | 0.121 | –331 | 4,891 |
| Earth | $W_d = 4{,}144$ | 0.326 | 0.125 | <u>1,351</u> | <u>17,094</u> |
| Total dead load | | | | 1,306 | 27,503 |
| Live load | $W_L = 950$ | 0.326 | 0.125 | 310 | 3,919 |

Pressure thrust, $F_{pt} = -(D_{yi})(P_w \text{ or } P_t)\,[(12 \text{ in./ft})/2]$

$P_w = 75$ psi: $F_{pt} = -(63.38)(75)(6) = -28{,}521$ lb/lin ft

$P_t = 25$ psi: $F_{pt} = -(63.38)(25)(6) = -9{,}507$ lb/lin ft

At side (104° from crown for maximum moment for 90° bedding):

| Component | Load *lb/lin ft* (1) | Coefficients for 90° Bedding: Thrust (2) | Coefficients for 90° Bedding: Moment (3) | *F* Thrust *lb/lin ft* Col. 1 × Col. 2 | *M* Moment *in.-lb/lin ft* Col. 1 × Col. 3 × *r* |
|---|---|---|---|---|---|
| Pipe | $W_p = 1{,}382$ | 0.295 | 0.088 | 408 | 4,013 |
| Water | $W_w = 1{,}225$ | –0.062 | 0.088 | –76 | 3,557 |
| Earth | $W_d = 4{,}144$ | 0.539 | 0.089 | <u>2,234</u> | <u>12,171</u> |
| Total dead load | | | | 2,566 | 19,741 |
| Live load | $W_L = 950$ | 0.539 | 0.089 | 512 | 2,790 |

Pressure thrust, $F_{pt}$, is calculated in the previous table and results in the same values at $P_w$ and $P_t$.

At maximum shear (146° from crown for 90° bedding):

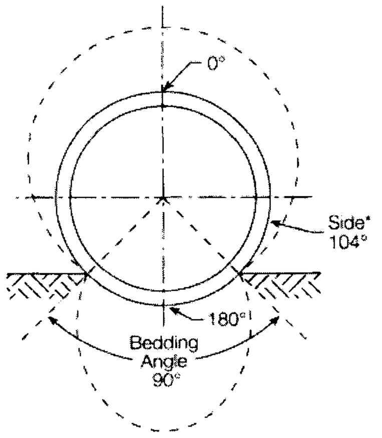
| Component | Load *lb/lin ft* (1) | Shear Coefficient for 90° Bedding (2) | Shear Load *lb/lin ft* Col. 1 × Col. 2 |
|---|---|---|---|
| Pipe | $W_p$ = 1,382 | 0.264 | 365 |
| Earth | $W_d$ = 4,144 | 0.276 | 1,144 |
| Live load | $W_L$ = 950 | 0.276 | 262 |
| Total shear load | | | 1,771 |

Step 7 Calculate the required circumferential steel area for the invert and the side for each of the three combinations of loads (condition 1, condition 2, and condition 3) as defined on page 66. Calculate minimum bar area for condition 4 with zero cylinder thickness.

The ACI 318 Strength Design method will be used. The following table summarizes moments and thrusts from step 6 for the three combinations of load conditions.

| | Components | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Condition | Dead Loads | + | $W_L$ | + | $P_w$ | + | $P_t$ | = | Total |
| **Condition 1** | | | | | | | | | |
| (No surge pressure) | | | | | | | | | |
| Invert moment *M* | 27,503 | | 3,919 | | | | | | 31,422 in.-lb/lin ft |
| Invert thrust *F* | 1,306 | | 310 | | –28,521 | | 0 | | –26,905 lb./lin ft |
| Side moment *M* | 19,741 | | 2,790 | | | | | | 22,531 in.-lb/lin ft |
| Side thrust *F* | 2,566 | | 512 | | –28,521 | | 0 | | –25,443 lb/lin ft |
| **Condition 2** | | | | | | | | | |
| (No pressure) | | | | | | | | | |
| Invert moment *M* | 27,503 | | 3,919 | | | | | | 31,422 in.-lb/lin ft |
| Invert thrust *F* | 1,306 | | 310 | | 0 | | 0 | | 1,616 lb/lin ft |
| Side moment *M* | 19,741 | | 2,790 | | | | | | 22,531 in.-lb/lin ft |
| Side thrust *F* | 2,566 | | 512 | | 0 | | 0 | | 3,078 lb/lin ft |
| **Condition 3** | | | | | | | | | |
| (No live load) | | | | | | | | | |
| Invert moment *M* | 27,503 | | 0 | | | | | | 27,503 in.-lb/lin ft |
| Invert thrust *F* | 1,306 | | 0 | | –28,521 | | –9,507 | | –36,722 lb/lin ft |
| Side moment *M* | 19,741 | | 0 | | | | | | 19,741 in.-lb/lin ft |
| Side thrust *F* | 2,566 | | 0 | | –28,521 | | –9,507 | | –35,462 lb/lin ft |
| **Condition 4 *** | | | | | | | | | |
| Invert moment *M* | 22,612 | | 3,919 | | 0 | | 0 | | 26,531 in.-lb/lin ft |
| Invert thrust *F* | 1,637 | | 310 | | 0 | | 0 | | 1,947 lb./lin ft |
| Side moment *M* | 16,184 | | 2,790 | | | | | | 18,974 in.-lb/lin ft |
| Side thrust *F* | 2,642 | | 512 | | | | | | 3,154 lb/lin ft |
| Shear | 1,509 | | 262 | | | | | | 1,771 lb/lin ft |

* This condition does not control the design. The steel area for condition 4 is used to check against the maximum steel area for radial tension capacity.




Note: For 90° bedding angle the maximum moment and shear side coefficients are 104° from the top and maximum shear coefficients are 146° from the top.

| | Moment* | | Thrust | | Shear |
|---|---|---|---|---|---|
| | Side | Invert | Side | Invert | 146° from top |
| Pipe Weight | 0.088 | 0.121 | 0.295 | 0.207 | -0.264 |
| Water Weight | 0.088 | 0.121 | –0.062 | –0.270 | -0.264 |
| External Load | 0.089 | 0.125 | 0.539 | 0.326 | -0.276 |

Sign conventions: Positive moment = Tension in external face at side
= Tension in internal face at invert
Positive thrust = Compression

* The sign of the moment coefficients at the invert in the Olander reference was changed from negative to positive because the sign convention in the Olander reference is different from the sign convention defined in the table; in the Olander reference, the moment coefficient is positive when the moment creates compressive stress on the inside face of the pipe, and negative when the moment creates tensile stress on the inside face of the pipe.

Figure 7-1 Olander moment, thrust, and shear coefficients for 90° bedding angle

The moments and thrusts are factored to establish the ultimate design conditions. The following factors are used:

| | |
|---|---|
| Load factor, conditions 1 and 2, | $L_f = 1.8$ |
| Load factor, condition 3, | $L_f = 1.35$ |
| Load factor, condition 4, | $L_f = 1.20$ |
| Strength reduction factor, | $\phi = 1.0$ |

The strength reduction factor $\phi$ is 1.0 for precast manufacturing; thus, $\phi$ is omitted from the design equations.

Figure 7-2 pictorially defines variables used in the calculation of the ultimate moment $M_{ult}$.

$$M_{ult} = L_f M + L_f F d'' \tag{Eq 7-6}$$

Summing moments at the centerline of the tensile reinforcement shown in Figure 7-2 yields

$$+M_{ult} - 0.85 f'_c ba(d - a/2) = 0 \tag{Eq 7-7}$$

Solving Eq 7-7 for the compression block depth $a$

$$a = d\left[1 - \sqrt{1 - \left(\frac{2M_{ult}}{0.85f'_c\,bd^2}\right)}\right] \quad \text{(Eq 7-8)}$$

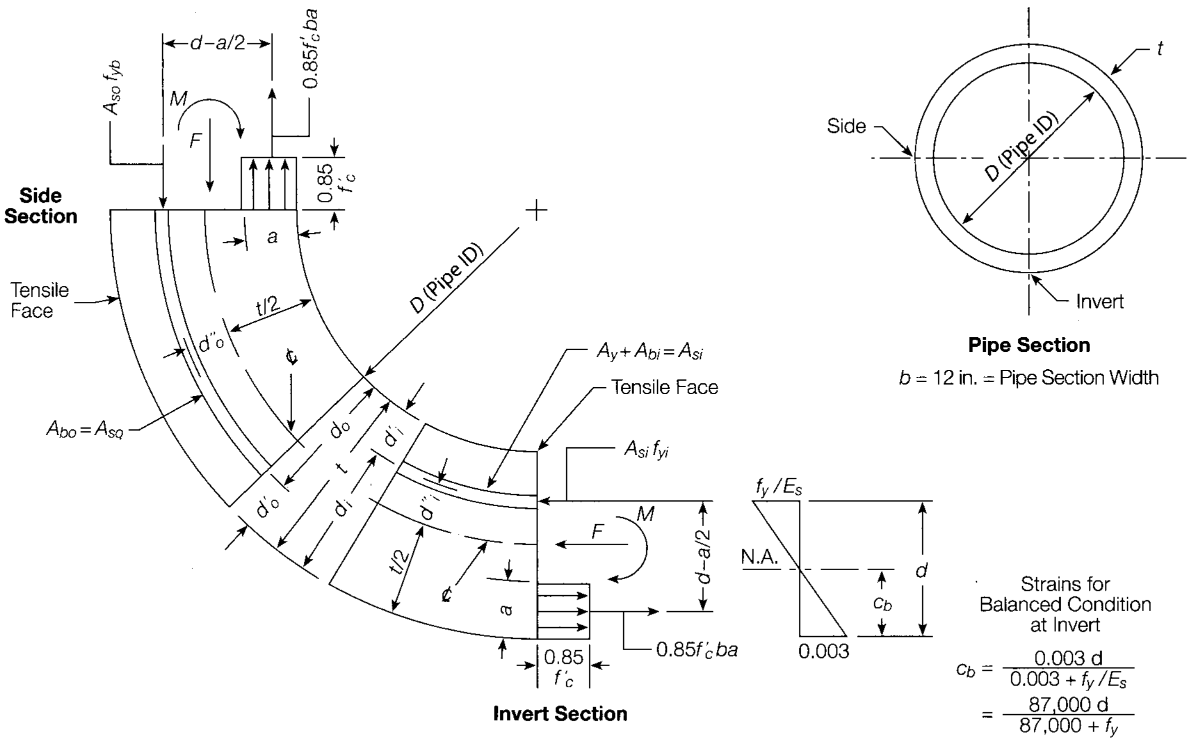
When dimension $a$ is negative, tension exists across the entire pipe wall and the tensile steel is proportioned between the inner and outer steel reinforcement. The method of proportioning tensile steel that follows is described by Harris (1975). See Figure 7-3 for the dimensions necessary for proportioning.

Proportioning steel for tension across the entire pipe wall is accomplished as follows:

The inner $A_{si}$ is obtained by taking moments about $s$ at the invert of the pipe.

$$A_{si}f_{yi}(d_o - d'_i) = P_u i(d_i - d'_o - e_i) \quad \text{(Eq 7-9)}$$

$$A_{si} = [P_u i(d_i - d'_o - e_i)] / [f_{yi}(d_o - d'_i)] \quad \text{(Eq 7-10)}$$




NOTE: For configurations where multiple cages are used for either the inner or outer reinforcement, or where the inner reinforcement is a cage plus a steel cylinder for pipe manufactured to ANSI/AWWA C300, the effective depth must be adjusted to accommodate the corrected neutral axis for that reinforcement combination.

Figure 7-2 Force diagrams for reinforced concrete pipe design

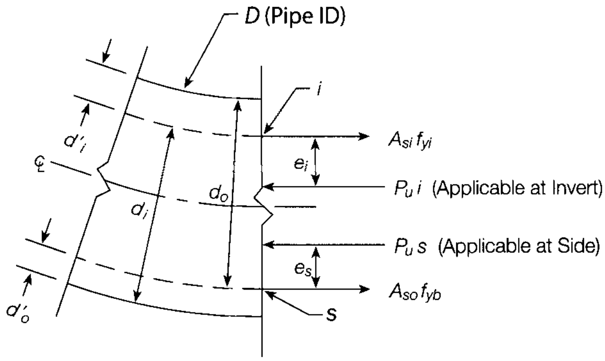



Figure 7-3 Section for proportioning tensile steel

Where:

$$e_i = M_{ult} i / P_u i \qquad \text{(Eq 7-11)}$$

The outer $A_{so}$ is obtained by taking moments about $i$ at the side of the pipe:

$$A_{so} f_{yb} (d_i - d'_o) = P_u s (d_o - d'_i - e_s) \qquad \text{(Eq 7-12)}$$

$$A_{so} = [P_u s (d_o - d'_i - e_s)] / [f_{yb} (d_i - d'_o)] \qquad \text{(Eq 7-13)}$$

Where:

$$e_s = M_{ult} s / P_u s \qquad \text{(Eq 7-14)}$$

Refer back to Figure 7-2. The stress in the steel reinforcement is set at the steel yield strength for ultimate design conditions as required by ACI 318, the thrust force $F$ in Figure 7-2 can be replaced by the ultimate design thrust $P_u$, where $P_u = L_f F$, and thrust forces at the sections shown in Figure 7-2 can be summed as follows:

$$+P_u + A_s f_y - 0.85 f'_c ab = 0 \qquad \text{(Eq 7-15)}$$

Solving Eq 7-15 for $A_s$

$$A_s = [(0.85 f'_c ab)/f_y)] - (P_u / f_y) \qquad \text{(Eq 7-16)}$$

ACI 318 also limits the maximum area of reinforcing steel for flexural members, and for members subject to combined flexure and compressive axial load when the design axial load is less than certain limits, to not more than 75 percent of the area of steel that would produce balanced strain conditions for the section under flexure without axial load. This limitation is applied to concrete pressure pipe. Balanced strain in a section is defined by ACI 318 as the condition when tension reinforcement reaches the strain corresponding to its specified yield strength $f_y$ just as concrete in compression reaches its assumed maximum strain of 0.003. As shown in Figure 7-2, this condition exists when the dimension $c$ from the neutral axis to the fiber of maximum compressive strain is set to the value

$$c_b = 87{,}000\, d / (87{,}000 + f_y) \qquad \text{(Eq 7-17)}$$

ACI 318 also defines the compression block depth $a$ equal to $\beta_1 c$, where $\beta_1$ is defined by ACI 318 as

$$\begin{aligned} \beta_1 &= 0.85 \text{ for } f'_c \leq 4{,}000 \text{ psi} \\ &= 0.85 - 0.05\,[(f'_c - 4{,}000)/1{,}000] \text{ for } 4{,}000 \text{ psi} < f'_c < 8{,}000 \text{ psi} \\ &= 0.65 \text{ for } f'_c \geq 8{,}000 \text{ psi} \end{aligned} \quad \text{(Eq 7-18)}$$

Replacing the compression block depth $a$ in Eq 7-16 with $\beta_1 c_b$, where $c_b$ is defined by Eq 7-17, and multiplying the resulting area of steel for balanced strain conditions by 75 percent, gives the following ACI 318 maximum limit for steel area:

$$A_s\text{max} = 0.75\{[(0.85 f'_c b\beta_1)/f_y][87{,}000\, d/(87{,}000 + f_y)] - (P_u/f_y)\} \quad \text{(Eq 7-19)}$$

If $A_s$ is greater than $A_s$max, compression steel is required. This condition rarely, if ever, applies to concrete pressure pipe, and design for such is not illustrated by the selected examples.

The maximum flexural reinforcement without stirrups for condition 4 is limited by radial tension. For US customary units, the following equation must be satisfied:

$$A_{si} \text{ max} = 16\, r_s F_{rt} F_{rp} (\Phi_r/\Phi_f)(f'_c)^{1/2}/f_y \quad \text{(Eq 7-20)}$$

Where:

| | | |
|---|---|---|
| $A_{si}$ max | = | maximum flexural reinforcement without stirrups, in.$^2$/ft |
| $r_s$ | = | radius of inside bar reinforcement, in. |
| $F_{rt}$ | = | $1 + 0.008333(72 - D)^2$ for $D \leq 72$ in. |
| | = | $0.8 + (144 - D)^2/26{,}000$ for 72 in. $< D \leq 144$ in. |
| | = | 0.8 for $D > 144$ in. |
| $F_{rp}$ | = | 1.0 unless a higher value is established |
| $\Phi_r/\Phi_f$ | = | ratio of resistance factors for radial tension and flexure |
| | = | 1.0 |
| $f'_c$ | = | compressive strength of concrete, psi (maximum 7,000 psi) |
| $f_y$ | = | yield strength of reinforcement, psi |
| | = | 40,000 psi |

Equation 7-20 can be rewritten as:

$$A_{si} \text{ max} = 16 r_s F_{rt} (f'_c)^{1/2}/40{,}000 \quad \text{(Eq 7-21)}$$

Shear load for condition 4 can be determined by multiplying the earth, pipe weight, and live load by Olander's maximum shear coefficient.

The shear stress can be determined as follows:

$$v_s = (V_e + V_L + V_p)/bd \quad \text{(Eq 7-22)}$$

Where:

$V_e$ = earth shear load, lb/ft

$V_L$ = live load shear, lb/ft

$V_p$ = pipe shear load, lb/ft

$b$ = 12 in.

$d$ = effective depth for inner cage, in.

Shear capacity or allowable shear stress is:

$$V_c = \beta (f'_c)^{1/2} \quad \text{(Eq 7-23)}$$

Where:

$\beta$ = approximately 2, for additional information refer to AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, 2002.

$f'_c$ = concrete compressive strength, psi (maximum 7,000 psi)

Table 7-1 provides a convenient view of the required strength method calculations for each of the design conditions. Because $a$ is negative for conditions 1 and 3, tensile steel must be proportioned between the inner and outer reinforcement using Eq 7-10 through Eq 7-14 as follows:

| | Condition 1 | Condition 3 |
|---|---|---|
| **Invert** | | |
| $e_i$ (Eq 7-11) | $\frac{-3,976}{-48,429} = 0.082$ in. | $\frac{-24,840}{-49,575} = 0.501$ in. |
| $A_{si}$ (Eq 7-10) | $\frac{48,429}{36,000}\left(\frac{4.25-1.375-0.082}{4.625-1.75}\right)$ | $\frac{49,575}{36,000}\left(\frac{4.25-1.375-0.501}{4.625-1.75}\right)$ |
| $A_{si}$ | 1.31 in.$^2$/lin ft | 1.14 in.$^2$/lin ft |
| **Side** | | |
| $e_s$ (Eq 7-14) | $\frac{-33,864}{-45,797} = 0.739$ in. | $\frac{-51,145}{-47,874} = 1.068$ in. |
| $A_{so}$ (Eq 7-13) | $\frac{45,797}{40,000}\left(\frac{4.625-1.75-0.739}{4.25-1.375}\right)$ | $\frac{47,874}{40,000}\left(\frac{4.625-1.75-1.068}{4.25-1.375}\right)$ |
| $A_{so}$ | 0.86 in.$^2$/lin ft | 0.76 in.$^2$/lin ft |

Step 8 Select the controlling maximum steel area for the invert and side. Check to see that the total steel area will be sufficient for the hydro static design. Check radial tension and shear capacity.

*Design Summary, ANSI/AWWA C300-type pipe*

Combined Load Designs

Condition 1 $A_{si}$ = 1.31 in.$^2$/lin ft (controls)

$A_{so}$ = +0.86 in.$^2$/lin ft (controls)

$A_s$ = 2.17 in.$^2$/lin ft

## Table 7-1 Tabulation of strength method design for ANSI/AWWA C300-type pipe

| Cond. | Invert $(i)$ or Side $(s)$ | Load Factor $L_f$ | Moment $M_u = L_f M$ *in.-lb/lin ft* | Thrust $P_u = L_f F$ *lb/lin ft* | $d''$ *in.* | $M_{ult} = M_u + P_u d''$ *in.-lb/lin ft* | $d$ *in.* | $\frac{2M_{ult}}{0.85f'_c bd^2}$ | $a$ *in.* | $\frac{0.85f'_c b\beta_1}{f_y}$ | $\frac{87,000d}{87,000 + f_y}$ | $\frac{P_u}{f_y}$ | $A_s$max *in.²/lin ft* | $\frac{0.85f'_c ab}{f_y}$ | $A_s$ *in.²/lin ft* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Invert $(i)$ | 1.80 | 56,560 | –48,429 | 1.250 | –3,976 | 4.250 | –0.0096 | –0.020 | | | * | | | 1.31 |
| 1 | Side $(s)$ | 1.80 | 40,556 | –45,797 | 1.625 | –33,864 | 4.625 | –0.0690 | –0.157 | | | * | | | 0.86 |
| 2 | Invert $(i)$ | 1.80 | 56,560 | +2,909 | 1.250 | +60,196 | 4.250 | +0.1452 | +0.321 | 1.148 | 3.081 | 0.088 | 2.59 | 0.446 | 0.33 |
| 2 | Side $(s)$ | 1.80 | 40,556 | +5,540 | 1.625 | +49,559 | 4.625 | +0.1010 | +0.240 | 0.947 | 3.168 | 0.139 | 2.15 | 0.275 | 0.14 |
| 3 | Invert $(i)$ | 1.35 | 37,129 | –49,575 | 1.250 | –24,840 | 4.250 | –0.0599 | –0.125 | | | * | | | 1.14 |
| 3 | Side $(s)$ | 1.35 | 26,650 | –47,874 | 1.625 | –51,145 | 4.625 | –0.1042 | –0.235 | | | * | | | 0.76 |
| 4 | Invert $(i)$ | 1.20 | 31,837 | 2,336 | 1.000 | 34,173 | 4.000 | +0.093 | +0.191 | 0.947 | 2.740 | 0.058 | 1.90 | 0.219 | 0.16 |
| 4 | Side $(s)$ | 1.20 | 22,769 | 3,785 | 1.625 | 28,920 | 4.625 | +0.059 | +0.139 | 0.947 | 3.168 | 0.095 | 2.17 | 0.160 | 0.07 |

Pipe ID, $D = 60$

Wall dimensions:

$t = 6.00$ in.
$b = 12$ in.
$d_i = 4.250$ in. at Cyl. OD
$d_o = 4.625$ in.
$d'_i = 1.750$ in.
$d'_o = 1.375$ in.
$d''_i = d_i - 0.5t = 4.250 - 3 = 1.250$ in.
$d''_o = d_o - 0.5t = 4.625 - 3 = 1.625$ in.

$f_{yi} = 36,000$ psi
$f_{yb} = 40,000$ psi
$f'_c = 4,500$ psi
$\beta_1 = 0.85 - 0.05/2 = 0.825$ in.

For condition 4:

$d'_{i4} = 1.75 + 0.25 = 2.0$ in.
$d_{i4} = 4.0$ in.
$d''_{i4} = 4 - 3 = 1.0$ in.
$r_s = 2 + 30 = 32$ in.

$$a = d\left(1 - \sqrt{1 - (2M_{ult})/(0.85f'_c bd^2)}\right)$$

If $a \leq 0$, there is tension through the wall.

$$A_s\text{max} = 0.75\left[\frac{0.85f'_c b\beta_1}{f_y}\left(\frac{87,000d}{87,000 + f_y}\right) - \frac{P_u}{f_y}\right]$$

$$A_s = \frac{0.85f'_c ab}{f_y} - \frac{P_u}{f_y}$$

If $A_s > A_s$max, compression steel is required.

*See end of step 7 of C300 design example.

| | | | |
|---|---|---|---|
| Condition 2 | $A_{si}$ | = | 0.33 in.²/lin ft |
| | $A_{so}$ | = | 0.14 in.²/lin ft |
| Condition 3 | $A_{si}$ | = | 1.14 in.²/lin ft |
| | $A_{so}$ | = | 0.76 in.²/lin ft |
| Condition 4 | $A_{bi}$ | = | 0.16 in.²/lin ft |
| | $A_{so}$ | = | 0.07 in.²/lin ft |

Hydrostatic Designs

Eq 7-1: $A_s =$ 1.73 in.²/lin ft

Eq 7-2: $A_s =$ 1.81 in.²/lin ft

Radial Tension Check, Eq 7-21: $A_{si}$ max = 1.89 in.²/lin ft > 0.16 in.²/lin ft (condition 4)

The wall has adequate radial tension capacity for this design.

Shear Check, Eq 7-22 and 7-23:

$$v_s = \frac{1771\,(1.2)}{12\,(d)} = 44.3 \text{ psi}$$

$$V_c = \beta\sqrt{f'_c} = 2\sqrt{4500} = 134 \text{ psi}$$

The wall has adequate shear capacity for this design.

If the pipe wall is not adequate for the radial tension and/or shear loads, either the pipe wall thickness, the concrete compressive strength ($f'_c$), or both must be increased. The addition of stirrups is impractical in an ANSI/AWWA C300 pipe. The pipe supplier should be contacted concerning the practicality and limits for increasing either the pipe wall thickness or the compressive strength of the concrete.

Step 9 Select appropriate bars or fabric to meet the design circumferential steel areas. For ANSI/AWWA C300-type pipe, check to see that the area of bar reinforcement is at least 40 percent of the total circumferential steel area. Check concrete cover over steel.

Condition 1 controls the total steel area.

| | | |
|---|---|---|
| Inner steel area $A_{si}$ | = | 1.31 in.²/lin ft |
| Outer steel area $A_{so}$ | = | +0.86 in.²/lin ft |
| Total steel area $A_s$ | = | 2.17 in.²/lin ft |

Inner steel distribution:

$$A_{si} = A_y + A_{bi}$$

Choose $A_y$ = 0.90 in.²/lin ft (cylinder area of a 14-ga cylinder)

$A_{bi} = A_{si} - A_y$

$= 1.31 - 0.90$

$A_{bi}$ = 0.41 in.²/lin ft (bar area) > 0.16 (condition 4)

Note that if the calculated $A_{bi}$ is less than 0.16 or negative, the required value for $A_{bi}$ defaults to the required value from condition 4 (i.e., 0.16 in.²/lin ft).

$d_{bi}$ = ⅜ in. (bar diameter)

$c\text{–}c$ = 3.23 in. (bar spacing)

Now that the values of $A_y$, $A_{bi}$, and $d_{bi}$ have been selected, the actual centroid of the inner line of reinforcing steel must be checked against the assumed centroid ($d_i$).

From the invert design for conditions 1, 2, and 3, the value of $d_i$ used for design was 4.25 in. (see Table 7-1). From the actual steel cylinder diameter, steel cylinder thickness, inside bar area, and bar diameter, the actual $d_i$ value can be calculated as follows:

$$d_i = t - \frac{A_y\left(\dfrac{D_{yi} - D + t_y}{2}\right) + A_{bi}\left(\dfrac{D_{yi} - D + d_{bi}}{2} + t_y\right)}{A_y + A_{bi}}$$

$$= 6 - \frac{0.90\left(\dfrac{63.38 - 60 + 0.0747}{2}\right) + 0.41\left(\dfrac{63.38 - 60 + 0.5}{2} + 0.0747\right)}{0.90 + 0.41}$$

$$= 4.18 \text{ in.} \approx 4.25 \text{ in.}$$

If the actual value of $d_i$ were significantly different from the value used in design, the design conditions using $d_i$ in the calculation of $A_{si}$ would have to be recalculated until the assumed $d_i$ and the actual $d_i$ were in close agreement.

Outer steel:

Choose $\quad d_{bo} = $ ½ in. (bar diameter)

$\quad c\text{–}c = $ 2.74 in. (bar spacing)

Design concrete cover over outer cage is:

$t - d_o - (d_{bo}/2) = $ 1.125 in., allowable design minimum, meets requirements

The center-to-center ($c$–$c$) spacing of 3.23 in. for the inner bar reinforcement and 2.74 in. for the outer bar reinforcement result in a clear spacing greater than 1.25 in. and are both less than the maximum $c$–$c$ spacing of 4.00 in., therefore meeting the requirements of the criteria in step 9 on page 67.

Percentage of bar reinforcement to total reinforcement:

(0.41 + 0.86)/(0.41 + 0.90 + 0.86) = 0.59 > 0.40, meets requirements.

Step 10 Check the $A_{si}$ and $A_{so}$ values against the minimum reinforcement requirement of ACI 318, Sec. 10.5:

For the check of $A_{si}$, $d = d_i = 4.25$ in.

ACI 318 min. $A_s = 3\sqrt{f'_c}\,bd \div f_{yb} = 3\sqrt{4,500} \times 12 \times 4.25 \div 40,000$

$= 0.257$ in.²/lin ft

or min. $A_s = 200bd \div f_{yb} = 200 \times 12 \times 4.25 \div 40,000$

$= 0.255$ in.²/lin ft

Calculated $A_s = 1.31 > 0.257$ OK

For the check of $A_{so}$, $d = d_o = 4.625$ in.

ACI 318 min. $A_s = 3\sqrt{f'_c}\,bd \div f_{yb} = 3\sqrt{4500} \times 12 \times 4.625 \div 40,000$

$= 0.279$ in.²/lin ft

or min. $A_s = 200bd \div f_{yb} = 200 \times 12 \times 4.625 \div 40,000$

$= 0.277$ in.²/lin ft

Calculated $A_{so}$ = 0.86 > 0.279 OK

Figures 7-2 and 7-3 show dimensions of $d$, $d_i$, and $d_o$.

# Example Calculations for ANSI/AWWA C302-Type Pipe

**Project requirements**

| | |
|---|---|
| • pipe diameter $D$, in. | = 60 |
| • internal pressure | |
| – working pressure $P_w$, psi | = 22 |
| – surge pressure $P_t$, psi | = 9 |
| • external earth load | |
| – height of earth cover $H$, ft | = 5 |
| – trench width $B_d$, ft | = $B_c + 3.5$ |
| – soil properties $K\mu' = K\mu$ | = 0.165 |
| – unit weight of soil $w$, lb/ft³ | = 120 |
| • external live load $W_L$ | |
| – AASHTO highway load | = HS-20 |
| – AASHTO impact factor $I_f$ | = 1.0 |
| • installation criteria | |
| – bedding angle, in degrees | = 90 |
| – moment, thrust, and shear coefficients | = Olander |

**Design information from specifications or manufacturer**

| | |
|---|---|
| • pipe wall thickness $t$, in. | = 6 |
| • design yield strength for steel reinforcement $f_{yb}$, psi | = 40,000 |
| • average circumferential stress in steel with pipe at working pressure $f_s$, psi | = $16{,}500 - 75P_w$ |
| • average circumferential stress in steel with pipe at working plus surge pressure $f_{st}$, psi | = 16,500 |
| • average circumferential tensile stress in ANSI/AWWA C302-type pipe wall concrete $f_{ct}$, psi | $\leq 4.5\sqrt{f'_c}$ |
| • minimum design concrete strength $f'_c$, psi | = 4,500 |

Step 1 Calculate the total circumferential steel area required to resist internal pressure.

Using Eq 7-3

$$A_s = \frac{6P_wD}{16{,}500 - 75P_w} = \frac{6(22)60}{16{,}500 - 75(22)} = 0.53 \text{ in.}^2/\text{lin ft}$$

Using Eq 7-4

$$A_s = \frac{6(P_w + P_t)D}{16{,}500} = \frac{6(22 + 9)60}{16{,}500} = 0.68 \text{ in.}^2/\text{lin ft}$$

Step 2 Calculate the circumferential tensile stress in the pipe wall concrete resulting from working plus surge pressure to determine if the assumed wall thickness is acceptable.

Using Eq 7-5

$$f_{ct} = \frac{(P_w + P_t)D}{2t} = \frac{(22 + 9)\,60}{2\,(6)} = 155 \text{ psi}$$

maximum $f_{ct} = 4.5\sqrt{4{,}500} = 302$ psi, wall thickness is acceptable

Step 3 Calculate the pipe weight and water weight.

Pipe weight, $W_p = (\pi/4)[(\text{OD pipe})^2 - (\text{ID pipe})^2]$ (pipe material unit weight)
$W_p = (\pi/4)[(72/12)^2 - (60/12)^2]\,(160 \text{ lb/ft}^3) = 1{,}382$ lb/lin ft

Water weight, $W_w = (\pi/4)(\text{ID pipe})^2$ (water unit weight)
$W_w = (\pi/4)(60/12)^2\,(62.4 \text{ lb/ft}^3) = 1{,}225$ lb/lin ft

Step 4 Calculate the external earth load on the pipe. Refer to Eq 5-1 and Eq 5-3 for formulas and definitions of variables.

OD pipe, $B_c = 60 + 2(6) = 72$ in. or 6.0 ft
$B_d = B_c + 3.5 = 6.0 + 3.5 = 9.5$ ft
$W_d = C_d w B_d^{\,2}$, $C_d = (1 - e^{-2K\mu'(H/B_d)})/(2K\mu')$
$W_d = [(1 - e^{-2(0.165)(5/9.5)})/(2(0.165))](120)(9.5)^2$
$W_d = 5{,}231$ lb/lin ft
$W_c = C_c w B_c^{\,2}$, find where height of plane of equal settlement ($H_e$) equals depth of cover ($H$) before solving for $C_c$. Set $H = H_e$ in Eq 5-6 and use $r_{sd} = 0.5$ and $p = 1.0$.
$W_c = 0.959\,(120)(6.0)^2$
$W_c = 4{,}144$ lb/lin ft
$W_c \leq W_d$ (transition trench width has been exceeded.)
$\therefore W_d = 4{,}144$ lb/lin ft

Step 5 Calculate the external live load, if any, on the pipe.

From Table 5-5, live load $W_L = 950$ lb/lin ft at $H = 5.0$ ft.

Step 6 Calculate moments and thrusts for each load on the pipe, including internal pressure. Calculate shear for pipe weight, earth load, and live load.

Using the coefficients from Figure 7-1, calculate the moment, thrust, and shear in the pipe wall as follows:

Moment = (Coefficient) (Load) (Radius)
Thrust = (Coefficient) (Load)
Shear = (Coefficient) (Load)

The radius is calculated as follows:
Radius = $r = 0.5(D + t) = 0.5(60 + 6) = 33$ in.

At invert (180° from crown):

| Component | Load, *lb/lin ft* (1) | Coefficients for 90° Bedding: Thrust (2) | Coefficients for 90° Bedding: Moment (3) | $F$ Thrust, *lb/lin ft* Col. 1 × Col. 2 | $M$ Moment, *in.-lb/lin ft* Col. 1 × Col. 3 × $r$ |
|---|---|---|---|---|---|
| Pipe | $W_p$ = 1,382 | 0.207 | 0.121 | 286 | 5,518 |
| Water | $W_w$ = 1,225 | –0.270 | 0.121 | –331 | 4,891 |
| Earth | $W_d$ = 4,144 | 0.326 | 0.125 | 1,351 | 17,094 |
| Total Dead Load | | | | 1,306 | 27,503 |
| Live Load | $W_L$ = 950 | 0.326 | 0.125 | 310 | 3,919 |

Pressure thrust, $F_{pt} = -(D)(P_w \text{ or } P_t)\ [(12 \text{ in./ft})/2]$

$P_w$ = 22 psi: $F_{pt} = -(60)(22)(6) = -7{,}920$ lb/lin ft

$P_t$ = 9 psi: $F_{pt} = -(60)(9)(6) = -3{,}240$ lb/lin ft

At side (104° from crown for maximum moment for 90° bedding):

| Component | Load, *lb/lin ft* (1) | Coefficients for 90° Bedding: Thrust (2) | Coefficients for 90° Bedding: Moment (3) | $F$ Thrust, *lb/lin ft* Col. 1 × Col. 2 | $M$ Moment, *in.-lb/lin ft* Col. 1 × Col. 3 × $r$ |
|---|---|---|---|---|---|
| Pipe | $W_p$ = 1,382 | 0.295 | 0.088 | 408 | 4,013 |
| Water | $W_w$ = 1,225 | –0.062 | 0.088 | –76 | 3,557 |
| Earth | $W_d$ = 4,144 | 0.539 | 0.089 | 2,234 | 12,171 |
| Total dead load | | | | 2,566 | 19,741 |
| Live load | $W_L$ = 950 | 0.539 | 0.089 | 512 | 2,790 |

Pressure thrust, $F_{pt}$, is calculated in previous table and results in the same values at $P_w$ and $P_t$.

At maximum shear (146° from crown for 90° bedding):

| Component | Load, *lb/lin ft* (1) | Shear Coefficients for 90° Bedding (2) | Shear Load *lb/lin ft* Col. 1 × Col. 2 |
|---|---|---|---|
| Pipe | $W_p$ = 1,382 | 0.264 | 365 |
| Earth | $W_d$ = 4,144 | 0.276 | 1,144 |
| Live load | $W_L$ = 950 | 0.276 | 262 |
| Total shear load | | | 1,771 |

Step 7 Calculate the required circumferential steel area for the invert and the side for each of the three combinations of loads (condition 1, condition 2, and condition 3) shown on page 66.

The ACI 318 Strength Design method will be used. The following table summarizes moments and thrusts from step 6 for the three combinations of load conditions.

| Condition | Dead Loads + | $W_L$ + | $P_w$ + | $P_t$ = | Total |
|---|---|---|---|---|---|
| | Components | | | | |
| **Condition 1** | | | | | |
| (No surge pressure) | | | | | |
| Invert moment $M$ | 27,503 | 3,919 | | | 31,422 in.-lb/lin ft |
| Invert thrust $F$ | 1,306 | 310 | –7,920 | 0 | –6,304 lb./lin ft |
| Side moment $M$ | 19,741 | 2,790 | | | 22,531 in.-lb/lin ft |
| Side thrust $F$ | 2,566 | 512 | –7,920 | 0 | –4,842 lb/lin ft |
| **Condition 2** | | | | | |
| (No pressure) | | | | | |
| Invert moment $M$ | 27,503 | 3,919 | | | 31,422 in.-lb/lin ft |
| Invert thrust $F$ | 1,306 | 310 | 0 | 0 | 1,616 lb/lin ft |
| Side moment $M$ | 19,741 | 2,790 | | | 22,531 in.-lb/lin ft |
| Side thrust $F$ | 2,566 | 512 | 0 | 0 | 3,078 lb/lin ft |
| **Condition 3** | | | | | |
| (No live load) | | | | | |
| Invert moment $M$ | 27,503 | 0 | | | 27,503 in.-lb/lin ft |
| Invert thrust $F$ | 1,306 | 0 | –7,920 | –3,240 | –9,854 lb/lin ft |
| Side moment $M$ | 19,741 | 0 | | | 19,741 in.-lb/lin ft |
| Side thrust $F$ | 2,566 | 0 | –7,920 | –3,240 | –8,594 lb/lin ft |
| **Condition 4 *** | | | | | |
| (No water) | | | | | |
| Invert moment $M$ | 22,612 | 3,919 | | | 26,531 in.-lb/lin ft |
| Invert thrust $F$ | 1,637 | 310 | 0 | 0 | 1,947 lb/lin ft |
| Side moment $M$ | 16,184 | 2,790 | | | 18,974 in.-lb/lin ft |
| Side thrust $F$ | 2,642 | 512 | 0 | 0 | 3,154 lb/lin ft |
| Shear | 1,509 | 262 | | | 1,771 lb/lin ft |

* This condition does not control the design. The steel area for condition 4 is used to check against the maximum steel area for radial tension capacity.

The moments and thrusts are factored to establish the ultimate design conditions. The following factors are used:

| | | | |
|---|---|---|---|
| Load factor, conditions 1, 2, and 3, | $L_f$ | = | 1.8 |
| Load factor, condition 4, | $L_f$ | = | 1.5 |
| Strength reduction factor, | $\phi$ | = | 1.0 |

The capacity (strength) reduction factor $\phi$ is 1.0 for precast concrete manufacturing; thus, $\phi$ is omitted from the design equations.

The reinforced concrete design assumptions, procedures, and equations in step 7 of the ANSI/AWWA C300 example calculations shown previously in this chapter also apply to ANSI/AWWA C302-type pipe. Table 7-2 provides a convenient view of the required strength method design calculations for each of the conditions.

Step 8 Select the controlling maximum steel area for the invert and side. Check that the total steel area is sufficient for the hydrostatic design. Check the radial tension and shear capacity.

Table 7-2 Tabulation of strength method design for ANSI/AWWA C302-type pipe

| Cond. | Invert $(i)$ or Side $(s)$ | Load Factor $L_f$ | Moment $M_u = L_f M$ in.-lb/lin ft | Thrust $P_u = L_f F$ lb/lin ft | $d''$ in. | $M_{ult} = M_u + P_u d''$ in.-lb/lin ft | $\frac{2M_{ult}}{0.85f'_c bd^2}$ | $a$ in. | $\frac{0.85f'_c b\beta_1}{f_y}$ | $\frac{87{,}000d}{87{,}000 + f_y}$ | $\frac{P_u}{f_y}$ | $A_s$max in.²/lin ft | $\frac{0.85f'_c ab}{f_y}$ | $A_S$ in.²/lin ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Invert $(i)$ | 1.8 | 56,560 | −11,347 | 1.625 | 38,121 | 0.0777 | 0.183 | 0.947 | 3.168 | −0.284 | 2.46 | 0.210 | 0.50 |
| 1 | Side $(s)$ | 1.8 | 40,556 | −8,716 | 1.625 | 26,393 | 0.0538 | 0.126 | 0.947 | 3.168 | −0.218 | 2.41 | 0.145 | 0.37 |
| 2 | Invert $(i)$ | 1.8 | 56,560 | +2,909 | 1.625 | 61,287 | 0.1248 | 0.298 | 0.947 | 3.168 | −0.073 | 2.20 | 0.342 | 0.27 |
| 2 | Side $(s)$ | 1.8 | 40,556 | +5,540 | 1.625 | 49,559 | 0.1010 | 0.240 | 0.947 | 3.168 | −0.139 | 2.15 | 0.275 | 0.14 |
| 3 | Invert $(i)$ | 1.8 | 49,505 | −17,737 | 1.625 | 20,682 | 0.0421 | 0.098 | 0.947 | 3.168 | −0.443 | 2.58 | 0.112 | 0.56 |
| 3 | Side $(s)$ | 1.8 | 35,534 | −15,469 | 1.625 | 10,397 | 0.0212 | 0.049 | 0.947 | 3.168 | −0.387 | 2.54 | 0.056 | 0.45 |
| 4 | Invert $(i)$ | 1.5 | 39,797 | 2,920 | 1.625 | 44,542 | 0.081 | 0.191 | 0.947 | 3.168 | 0.073 | 2.20 | 0.219 | 0.15 |
| 4 | Side $(s)$ | 1.5 | 28,461 | 4,731 | 1.625 | 36,149 | 0.058 | 0.136 | 0.947 | 3.168 | 0.118 | 2.16 | 0.156 | 0.04 |

Pipe ID, $D = 60$ in.

Wall dimensions:

$t$ = 6.00 in.
$b$ = 12 in.
$d_i$ = 4.625 in.
$d_o$ = 4.625 in.
$d'_i$ = 1.375 in.
$d'_o$ = 1.375 in.
$d''_i = d_i - 0.5t$ = 1.625 in.
$d''_o = d_o - 0.5t$ = 1.625 in.

$f_{yb}$ = 40,000 psi

$f'_c$ = 4,500 psi
$\beta_1$ = 0.85 − (0.05/2) = 0.825 in.

$$a = d\left(1 - \sqrt{1 - (2M_{ult}) / (0.85f'_c\, bd^2)}\right)$$

If $a \leq 0$, there is tension through the wall.

$$A_s\text{max} = 0.75\left[\frac{0.85f'_c b\beta_1}{f_y}\left(\frac{87{,}000d}{87{,}000 + f_y}\right) - \frac{P_u}{f_y}\right]$$

$$A_s = \frac{0.85f'_c ab}{f_y} - \frac{P_u}{f_y}$$

For condition 4:

$r_s$ = 1.375 + 30 = 31.375 in.

If $A_s > A_s$max, compression steel is required.

*Design Summary, ANSI/AWWA C302-type pipe*

Combined Load Designs

| | | | |
|---|---|---|---|
| Condition 1 | $A_{si}$ | = | 0.50 in.$^2$/lin ft |
| | $A_{so}$ | = | 0.37 in.$^2$/lin ft |
| Condition 2 | $A_{si}$ | = | 0.27 in.$^2$/lin ft |
| | $A_{so}$ | = | 0.14 in.$^2$/lin ft |
| Condition 3 | $A_{si}$ | = | 0.56 in.$^2$/lin ft (controls) |
| | $A_{so}$ | = | +0.45 in.$^2$/lin ft (controls) |
| | $A_s$ | = | 1.01 in.$^2$/lin ft |
| Condition 4 | $A_{si}$ | = | 0.15 in.$^2$/lin ft |
| | $A_{so}$ | = | +0.04 in.$^2$/lin ft |

Hydrostatic Designs

| | | | |
|---|---|---|---|
| Eq 7-3: | $A_s$ | = | 0.53 in.$^2$/lin ft |
| Eq 7-4: | $A_s$ | = | 0.68 in.$^2$/lin ft |

Radial Tension Check, Eq 7-21: $A_{si}$ max = 1.85 in.$^2$/lin ft

The wall has adequate radial tension capacity for this design.

Shear Check, Eq 7-22 and 7-23:

$$v_s = \frac{1,771\,(1.5)}{12\,(4.625)} = 47.9 \text{ psi}$$

$$V_c = \beta\sqrt{f'_c} = 2\sqrt{4,500} = 134 \text{ psi}$$

The wall has adequate shear capacity for this design.

If the pipe wall is not adequate for the radial tension and/or shear loads, either stirrup reinforcement must be added, the pipe wall thickness must be increased, the concrete compressive strength ($f'_c$) must be increased, or some combination of these three solutions must be used. The pipe supplier should be contacted concerning the practicality of any of these changes.

Step 9 Select appropriate bars or fabric to meet the design circumferential steel areas and spacing. Check concrete cover over steel.

For condition 3, $A_s$ = 1.01 in.$^2$/lin ft is greater than the hydrostatic design control $A_s$ = 0.68 in.$^2$/lin ft. Therefore, the final design areas are from condition 3. A bar cage for the inner area must have

$$A_{si} = 0.56 \text{ in.}^2/\text{lin ft} > 0.15 \text{ in.}^2/\text{lin ft (condition 4)}$$

Choose $^7/_{16}$-in. diameter bar. For $d_{bi}$ = $^7/_{16}$ in., each bar area is 0.150 in.$^2$ and the spacing $c$–$c$ is 3.21 in. The clear spacing between bars is 2.77 in. (3.21 – 0.44), which meets the requirements of step 9 on page 67.

Check the design clear cover for a bar diameter of 7⁄16 in. to be sure it is a minimum of 1 in. required by ANSI/AWWA C302 plus the ⅛-in. manufacturing tolerance required:

$$t - d_i - (d_{bi}/2) = 1.156 \text{ in.} > 1.125 \text{ in., meets requirements}$$

For the outer steel area

$$A_{so} = 0.45 \text{ in.}^2/\text{lin ft}$$

Choose the 7⁄16-in. diameter bar. For $d_{bo}$ = 7⁄16 in., the spacing $c–c$ is 4.00 in.

The $c–c$ spacing of 3.21 in. for the inner bar reinforcement is less than 4.00 in., and the $c–c$ spacing of 4.00 in. for the outer bar reinforcement is equal to or less than 4.00 in. Therefore, both bar cages meet the criteria for $c–c$ spacing in step 9 on page 67.

Design clear concrete cover for the outer cage is also 1.156 in. > 1.125 in., and therefore meets requirements.

The steel areas could be supplied as reinforcing steel cages made from bar reinforcement or wire fabric.

Step 10 Check the $A_{si}$ and $A_{so}$ values against the minimum reinforcement requirement of ACI 318, Sec. 10.5:

For the check of $A_{si}$ and $A_{so}$, $d$ = 4.625 in.

$$\text{ACI 318 min. } A_s = 3\sqrt{f'_c}\,bd \div f_{yb} = 3\sqrt{4,500} \times 12 \times 4.625 \div 40,000$$
$$= 0.279 \text{ in.}^2/\text{lin ft}$$
$$\text{or min. } A_s = 200bd \div f_{yb} = 200 \times 12 \times 4.625 \div 40,000$$
$$= 0.277 \text{ in.}^2/\text{lin ft}$$
$$\text{Calculated } A_{si} = 0.56 > 0.279 \text{ OK}$$
$$\text{Calculated } A_{so} = 0.45 > 0.279 \text{ OK}$$

## DESIGN PROCEDURE FOR SEMIRIGID PIPE (ANSI/AWWA C303-TYPE PIPE)

The semirigid pipe design procedure for ANSI/AWWA C303-type pipe involves the following steps:

Step 1 Select a steel cylinder thickness equal to or greater than the ANSI/AWWA C303 minimum. Calculate the total circumferential steel area required to resist internal pressure using the hoop tension Eq 7-1 for working pressure and Eq 7-2 for working pressure plus surge pressure.

The hydrostatic design steel area is the maximum $A_s$ obtained from Eq 7-1 and Eq 7-2. For ANSI/AWWA C303-type pipe, $f_s$ = 18,000 psi or 0.5 times the minimum specified yield strength of the cylinder steel, whichever is less, and $f_{st}$ = 27,000 psi or 0.75 times the minimum specified yield strength of the cylinder steel, whichever is less.

Step 2 Calculate the cylinder steel area and place the remaining required steel area in the bar by selecting a bar diameter and bar spacing within the following limits established in ANSI/AWWA C303:

- The area of bar reinforcement shall not exceed 60 percent of the total area of circumferential reinforcement.
- The area of bar reinforcement shall not be less than 0.23 in.²/lin ft.
- The *c–c* bar spacing shall not exceed 2 in.
- The area of bar reinforcement in square inches per linear foot of pipe wall shall be numerically equal to at least 1 percent of the inside diameter of the pipe, in inches.
- The design clear space between bars shall not be less than the diameter of the bar used.
- The bar diameter shall not be less than 7⁄32 in.

Step 3 Calculate the total external load on the pipe.

External loads must be computed in accordance with recognized and accepted theories, such as those presented in chapter 5.

Step 4 Determine if the total external load is less than either the maximum allowable external load for minimum designs in Table 7-3 or the maximum allowable external load for the actual design as calculated by Eq 7-24. If either condition is met, then the selected pipe design meets the project requirements.

The maximum allowable external load *W* for a given semirigid (bar-wrapped) pipe design is the load producing the limiting pipe deflection $D^2/4{,}000$, where *D* is the inside diameter of the pipe in inches. Experimental and field observations have shown Spangler's Iowa deflection equation for flexible pipe may be applied to semi-rigid design. The formula for deflection is

$$\Delta x = \frac{D_l k (W/12) r^3}{EI + 0.061 E' r^3} \qquad \text{(Eq 7-24)}$$

Where:

$\Delta x$ = horizontal deflection of pipe, in.

$D_l$ = deflection lag factor = 1.0*

$k$ = bedding constant

$W$ = total external dead plus live load, lb/lin ft of pipe length

$r$ = mean radius of pipe wall, in., calculated as $0.5(D + t)$, where $D$ is the inside diameter of the pipe, in., and $t$ is the pipe wall thickness, in.

$EI$ = pipe wall stiffness, in.-lb, where, for ANSI/AWWA C303-type pipe, $E$ is the modulus of elasticity of cement mortar, taken as 4,000,000 psi, and $I$ is 25 percent of the transverse moment of inertia of the composite wall section of the pipe, in.⁴/in. of pipe length

$E'$ = modulus of soil reaction, psi

* A value of 1.0 for $D_l$ is used in Eq 7-24 in recognition of the conservative values being used for pipe stiffness ($EI$) and allowable deflection ($\Delta x$) in the same equation. The $I$-value is 25 percent of the pipe wall's moment of inertia and the allowable deflection in inches is $D^2/4{,}000$, which is less than 2 percent deflection for all pipe sizes permitted by ANSI/AWWA C303.

## Table 7-3 Allowable external load for ANSI/AWWA C303-type pipe* of minimum class

| | | | | | Bedding Type S1 $E'$ used: 200 psi† | | | Bedding Type S2 $E'$ used: 400 psi† | | | Bedding Type S3 $E'$ used: 700 psi† | | | Bedding Type S4 $E'$ used: 1,000 psi† | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ID *in.* | Pipe Class *psi* | Cyl. thk. *ga or in.* | Bar Dia. *in.* | Wrap $A_s$ *in.²/ft* | $W$ *(lb/lin ft)* | $H_T$ *ft* | $H_E$ *ft* | $W$ *lb/lin ft* | $H_T$ *ft* | $H_E$ *ft* | $W$ *lb/lin ft* | $H_T$ *ft* | $H_E$ *ft* | $W$ *lb/lin ft* | $H_T$ *ft* | $H_E$ *ft* |
| 10 | 251 | 16 | 7/32 | 0.23 | 5,930 | >50 | 29 | 6,390 | >50 | 31 | 7,020 | >50 | 35 | 7,500 | >50 | 37 |
| 12 | 213 | 16 | 7/32 | 0.23 | 5,340 | >50 | 23 | 5,770 | >50 | 25 | 6,370 | >50 | 27 | 6,840 | >50 | 29 |
| 14 | 187 | 16 | 7/32 | 0.23 | 4,450 | 32 | 17 | 4,840 | >50 | 18 | 5,390 | >50 | 20 | 5,840 | >50 | 22 |
| 16 | 164 | 16 | 7/32 | 0.23 | 4,440 | 22 | 15 | 4,860 | 32 | 16 | 5,450 | >50 | 18 | 5,930 | >50 | 20 |
| 18 | 144 | 16 | 7/32 | 0.23 | 5,440 | 34 | 16 | 5,960 | >50 | 18 | 6,680 | >50 | 20 | 7,290 | >50 | 22 |
| 20 | 130 | 16 | 7/32 | 0.23 | 5,080 | 21 | 14 | 5,590 | 28 | 15 | 6,330 | >50 | 17 | 6,960 | >50 | 19 |
| 21 | 124 | 16 | 7/32 | 0.23 | 4,920 | 18 | 13 | 5,430 | 23 | 14 | 6,190 | 37 | 16 | 6,840 | >50 | 18 |
| 24 | 133 | 14 | 7/32 | 0.24 | 4,410 | 12 | 10 | 4,950 | 15 | 11 | 5,740 | 21 | 13 | 6,450 | 29 | 15 |
| 27 | 122 | 14 | 7/32 | 0.27 | | | | 4,750 | 12 | 10 | 5,620 | 17 | 12 | 6,430 | 22 | 14 |
| 30 | 113 | 14 | 1/4 | 0.30 | | | | 4,810 | 11 | 9 | 5,790 | 15 | 11 | 6,710 | 20 | 13 |
| 33 | 106 | 14 | 1/4 | 0.33 | | | | 4,640 | 10 | 8 | 5,720 | 13 | 10 | 6,760 | 18 | 12 |
| 36 | 114 | 13 | 1/4 | 0.36 | | | | 4,530 | 9 | 7 | 5,730 | 12 | 9 | 6,910 | 16 | 11 |
| 39 | 108 | 13 | 1/4 | 0.39 | Not Recommended for These Diameters | | | | | | 5,800 | 11 | 9 | 7,130 | 15 | 11 |
| 42 | 115 | 12 | 1/4 | 0.42 | | | | | | | 5,930 | 11 | 8 | 7,430 | 15 | 10 |
| 45 | 109 | 12 | 1/4 | 0.45 | | | | | | | 6,110 | 10 | 8 | 7,780 | 15 | 10 |
| 48 | 104 | 12 | 1/4 | 0.48 | | | | | | | 6,330 | 10 | 8 | 8,200 | 15 | 10 |
| 51 | 121 | 10 | 1/4 | 0.51 | | | | | | | 6,620 | 10 | 8 | 8,700 | 15 | 10 |
| 54 | 116 | 10 | 1/4 | 0.54 | | | | | | | 6,920 | 10 | 8 | 9,230 | 15 | 10 |
| | | | | | | | | | | | | $H_P$, ft | | | $H_P$, ft | |
| 57 | 111 | 10 | 1/4 | 0.57 | | | | | | | 7,660 | 11 | | 9,750 | 15 | |
| 60 | 115 | 8 | 5/16 | 0.60 | | | | | | | 8,360 | 12 | | 10,680 | 16 | |
| 66 | 130 | 3/16 | 3/8 | 0.66 | | | | | | | 8,690 | 11 | | 11,250 | 15 | |
| 72 | 121 | 3/16 | 3/8 | 0.72 | | | | | | | 8,820 | 10 | | 11,610 | 14 | |

$H_T$ indicates depth of cover for trench installations.
$H_E$ indicates depth of cover for embankment (positive projection) installations.
$H_P$ indicates depth of cover for prism of soil design.

Soil Load Parameters:

$w$ = 120 lb/ft³
$K\mu'$ = 0.15
$K\mu$ = 0.19
$B_d$ = outside diameter of pipe + 2 ft
$D_l$ = 1.0
$r_{sd}p$ = 0.50 for $D \leq 21$ in. and 0.30 for 24 in. $\leq D \leq 54$ in. and 0.0 for $D \geq 57$ in.

Bedding Parameters: (See chapter 6)

Type S1: $k$ = 0.105
Type S2: $k$ = 0.098
Type S3: $k$ = 0.090
Type S4: $k$ = 0.085

Highway loads are minimal for cover depths shown but should be included for depths of 6 ft or less.

* This table is intended only as a design example and is based on a limiting deflection of $D^2$/4,000 in., where $D$ is the inside nominal pipe diameter in inches. All pipe shown are minimum class and backfill envelope material is defined in chapter 6. Increasing the pipe class provides disproportionately small increases in allowable external load. If an increased allowable external load is desired, it can be achieved by increasing the effective moment of inertia of the longitudinal pipe section or by improving the bedding material or compaction requirements.

† The E′ values used are based on backfill envelope of lower structural quality. For E′ values of backfill material of higher structural quality, refer to the references cited in chapter 6.

Replacing the deflection $\Delta x$ with the $D^2/4{,}000$ allowable for ANSI/AWWA C303-type pipe, setting $D_l = 1.0$, and solving for the allowable external load in pounds per linear foot, the equation becomes

$$W = \frac{D^2(EI + 0.061E'r^3)}{333kr^3} \qquad \text{(Eq 7-25)}$$

where all variables were previously defined.

The maximum allowable external loads for minimum design of ANSI/AWWA C303-type pipe installed in various beddings are shown in Table 7-3.

Step 5 If required, provide additional external-load capacity by increasing the effective moment of inertia of the longitudinal pipe section or by improving the bedding material or compaction requirements. The effective moment of inertia may be increased by increasing the area or diameter of the bar reinforcement or by increasing the coating thickness to a maximum of 1.25 in. over the bar wrap.

## Example Calculations for ANSI/AWWA C303-Type Pipe

**Project requirements**

| | | |
|---|---|---|
| • pipe diameter $D$, in. | = | 36 |
| • internal pressure | | |
| – working pressure $P_w$, psi | = | 150 |
| – surge pressure $P_t$, psi | = | 50 |
| • external earth load | | |
| – height of earth cover $H$, ft | = | 14 |
| – trench width $B_d$, ft | = | $B_c + 2.0$ |
| – soil properties $K\mu'$ | = | 0.165 |
| – unit weight of soil $w$, lb/ft$^3$ | = | 120 |
| • external live load $W_L$ | | |
| – AASHTO highway load | = | HS-20 |
| – AASHTO impact factor $I_f$ | = | 1.0 |
| • installation criteria | | |
| – S3 bedding, moderate to heavy compaction | | |
| – soil modulus $E'$, psi | = | 700 |
| – Rankine's ratio $K$ | = | 0.090 |
| – deflection lag factor $D_l$ | = | 1.0 |

**Design information from specification or manufacturer**

| | | |
|---|---|---|
| • cylinder OD for ANSI/AWWA C303-type pipe $D_{yo}$, in. | = | 37.88 |
| – min. $t_y$ | = | 13 gauge (0.090 in.) |
| • $f_y$, psi | = | 36,000 |
| • $f_{yb}$, psi | = | 40,000 |
| • $E_s$, psi | = | 30,000,000 |
| • $E_c$, psi | = | 4,000,000 |

Step 1 Select a steel cylinder thickness equal to or greater than the ANSI/AWWA C303 minimum. Calculate the total circumferential steel area required to resist internal pressure.

Choose: $t_y$ = 12 gauge (0.105 in.)

$D_{yi} = D_{yo} - 2t_y = 37.88 - 2(0.105) = 37.67$ in.

Eq 7-1:

$$A_s = \frac{6P_w D_{yi}}{f_s} \qquad \text{For } P_w,\ f_s = 18{,}000 \text{ psi}$$

$$A_s = \frac{6(150)(37.67)}{18{,}000} = 1.88 \text{ in.}^2/\text{lin ft}$$

Eq 7-2:

$$A_s = \frac{6(P_w + P_t)D_{yi}}{f_{st}} \qquad \text{For } P_w + P_t,\ f_{st} = 27{,}000 \text{ psi}$$

$$A_s = \frac{6(150 + 50)(37.67)}{27{,}000} = 1.67 \text{ in.}^2/\text{lin ft}$$

Working pressure controls, so $A_s$ = 1.88 in.²/lin ft for hydrostatic design.

Step 2 Calculate the cylinder steel area and place the remaining area of steel in the bar wrap by selecting a bar diameter and bar spacing within the limits established in ANSI/AWWA C303.

$$\begin{aligned} A_s &= A_y + A_b \\ 1.88 &= 0.105(12) + A_b \\ A_b &= 1.88 - 1.26 = 0.62 \text{ in.}^2/\text{lin ft} \\ \text{Minimum } A_b &= (36)(0.01) = 0.36 \text{ in.}^2/\text{lin ft} \end{aligned}$$

Choose ⁵⁄₁₆-in. diameter bar. Calculate *c–c* spacing for ⁵⁄₁₆-in. diameter bar reinforcement (area = 0.0767 in.²):

Using $A_b$ = 0.62 in.²/lin ft

$$c\text{–}c = \frac{12(0.0767)}{0.62} = 1.48 \text{ in.} < 2.00 \text{ in., meets requirements}$$

Spacing for ⁵⁄₁₆-in. bar is acceptable per ANSI/AWWA C303.

Step 3 Calculate the total external load on the pipe. Refer to Eqs 5-1 and 5-3 for trench load and positive projecting embankment load formulae and definition of variables.

Calculate the pipe OD = $B_c$ and wall thickness $t$. ANSI/AWWA C303 requires mortar coating thickness to provide a minimum ¾-in. cover over the bar reinforcement or 1-in. cover over the cylinder, whichever results in the greater coating thickness.

$$\begin{aligned} B_c &= D_{yo} + 2(0.3125) + 2(0.75) \\ &= 40.01 \text{ in. } (3.33 \text{ ft}) \\ t &= 0.5(40.01 - 36) = 2.0025 \text{ in. (to be used in step 4)} \\ B_d &= B_c + 2.0 \end{aligned}$$

$= 3.33 + 2.0 = 5.33$ ft

$$W_d = C_d w B_d^2 = \left[\frac{1 - e^{-2K\mu'(H/B_d)}}{2K\mu'}\right] w B_d^2$$

$$W_d = \left[\frac{(1 - e^{-2(0.165)(14/5.33)})}{2(0.165)}\right](120)(5.33)^2$$

$W_d = 5{,}989$ lb/lin ft

$W_c = C_c w B_c^{\ 2}$

$H/B_c = 14/3.33 = 4.20$

Using Figure 5-6 with $r_{sd}p = +0.3$, then $C_c = 6.0$.

$W_c = 6.0(120)(3.33^2) = 7{,}984$ lb/lin ft

$W_c > W_d$ (trench load control)

$\therefore W_d = 5{,}989$ lb/lin ft

From Table 5-5, highway live load is negligible. Therefore, total external load for 14-ft cover is

$$W_T = W_d + W_L = 5{,}989 + 0 = 5{,}989 \text{ lb/lin ft}$$

Step 4 Determine if the total external load is less than either the maximum allowable external load for minimum designs in Table 7-3 or the maximum allowable external load for the actual design, as calculated by Eq 7-25. If either condition is met, the selected pipe design meets the project requirements.

From Table 7-3, allowable load for minimum design 36-in. pipe with

Type S3 bedding: $W = 5{,}730$ lb/lin ft

For 14 ft, required $W_T = 5{,}989$ lb/lin ft

The allowable external load for the minimum class 36-in. ANSI/AWWA C303-type pipe is less than required. However, the allowable external load for a 36-in. ANSI/AWWA C303-type pipe with $P_w = 150$ psi may be sufficient. Determine the allowable $W$ for the actual material dimensions by first calculating $\bar{I}_x$.

To determine $\bar{I}_x$, the ANSI/AWWA C303-type pipe longitudinal cross section (Figure 7-4) is transformed into an equivalent cement–mortar section by multiplying steel areas by the modular ratio $n$. First moments $AY$ and moments of inertia $I_x$ are taken about the $x$-axis. Moments of inertia of the steel cylinder and bar reinforcement about their own centers of gravity are ignored. The section properties determined are

$\bar{Y}$ = distance from $x$-axis to the section center of gravity, in.

$\bar{I}_x$ = moment of inertia of the transformed section about its center of gravity parallel to the $x$-axis, in.$^4$/lin ft

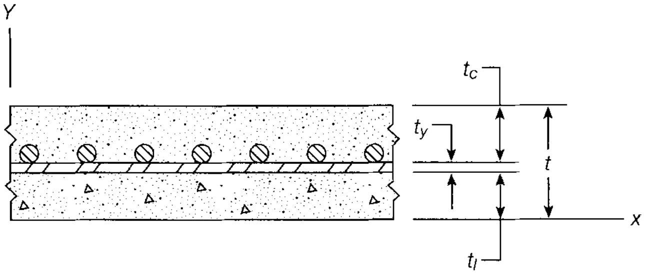



Figure 7-4 Section through AWWA C303-type pipe wall

The general solution in tabular format is as follows:

| Component | $A$ in.²/lin ft | $Y$ in. | $AY$ in.³/lin ft | $I_x$ in.⁴/lin ft |
|---|---|---|---|---|
| Total section | $12t$ | $0.5t$ | $6t^2$ | $4t^3$ |
| Steel cylinder | $(n-1)A_y$ | $t_1 + 0.5t_y = Y_y$ | $(n-1)A_yY_y$ | $(n-1)A_yY_y^2$ |
| Steel bar | $(n-1)A_b$ | $t_1 + t_y + 0.5d_b = Y_b$ | $(n-1)A_bY_b$ | $(n-1)A_bY_b^2$ |
| | $\Sigma A$ | | $\Sigma AY$ | $\Sigma I_x$ |
| $\bar{Y} = \Sigma AY/\Sigma A$ | | | $\bar{I}_x = \Sigma I_x - \bar{Y}^2\Sigma A$ | |

For this example of a 36-in. ANSI/AWWA C303-type pipe with $P_w$ = 150 psi

$t_c$ = 1.0625 in. $A_y$ = 1.26 in.²/lin ft
$t_y$ = 0.105 in. $A_b$ = 0.62 in.²/lin ft
$t_l$ = 0.835 in. $d_b$ = 0.3125 in.
$t$ = 2.0025 in. $n$ = 7.5

| Component | $A$, in.²/lin ft | $Y$, in. | $AY$, in.³/lin ft | $I_x$, in.⁴/lin ft |
|---|---|---|---|---|
| Total section | 24.03 | 1.001 | 24.05 | 32.12 |
| Steel cylinder | 8.19 | 0.888 | 7.27 | 6.46 |
| Steel bar | 4.03 | 1.096 | 4.42 | 4.84 |
| | 36.25 | | 35.74 | 43.42 |

$\bar{Y}$ = 35.74/36.25 = 0.986 in.

$\bar{I}_x$ = 43.42 – (0.986)² (36.25) = 8.18 in.⁴/lin ft = 0.682 in.⁴/lin in.

The wall stiffness $EI$ is calculated using the modulus of elasticity of mortar, $4 \times 10^6$ psi, and $I$ as $0.25\,\bar{I}_x$:

$$EI = (4{,}000{,}000)(0.25)(0.682)$$
$$= 682{,}000 \text{ lb-in.}^2\text{/in.}$$

Calculating the allowable load

$$W = \frac{D^2(EI + 0.061E'r^3)}{333kr^3} \qquad \text{(Eq 7-25)}$$

Where:

$$\begin{aligned} r &= 0.5(D + t) \\ &= 0.5(36 + 2.0025) \\ &= 19.001 \text{ in.} \end{aligned}$$

$$W = \frac{(36)^2\left[682,000 + 0.061(700)(19.001)^3\right]}{333(0.090)(19.001)^3}$$

$$= 6,146 \text{ lb/lin ft} > 5,989 \text{ lb/lin ft, OK}$$

The pipe designed for $P_w = 150$ will withstand at least 14 ft of earth cover when installed with a type S3 bedding having an $E' \geq 700$ psi.

## REFERENCES


American Concrete Institute. 2008. ACI 318-08, *Building Code Requirements for Structural Concrete.* Farmington Hills, Mich.: ACI.

American Concrete Pipe Association, 1993, *Concrete Pipe Technology Handbook*.

American Water Works Association. 2004. ANSI/AWWA C300-04, *Standard for Reinforced Concrete Pressure Pipe, Steel-Cylinder Type.* Denver, Colo.: AWWA.

———. 2004. ANSI/AWWA C302-04, *Standard for Reinforced Concrete Pressure Pipe, Noncylinder Type*. Denver, Colo.: AWWA.

———. 2002. ANSI/AWWA C303-02, *Standard for Concrete Pressure Pipe, Bar-Wrapped Steel Cylinder Type*. Denver, Colo.: AWWA.

Ameron. 1988. *Concrete Cylinder Pipe—Design Manual 303.* Monterey Park, Calif.: Ameron.

Harris, E.C. 1975. Design of Members Subject to Combined Bending and Tension. *ACI Journal*.

Heger, F.J., and T.J. McGrath. 1983. Radial Tension Strength of Pipe and Other Curved Flexural Members. *ACI Journal.* January–February 1983.

Olander, H.C. 1950. Stress Analysis of Concrete Pipe. Engineering Monograph No. 6, US Bureau of Reclamation. Washington, D.C.: US Department of Interior.

Paris, J.M. 1921. Stress Coefficients for Large Horizontal Pipes. *Engineering News–Record*, 87:19:768.

Spangler, M.G. 1941. The Structural Design of Flexible Pipe Culverts. Bulletin 153. Ames, Iowa: Iowa Engineering Experimental Station.

Chapter 8

# Design of Fittings and Appurtenances

Concrete pressure pipe fittings and specials provide variations and adaptability from the straight course of a pipeline. Fittings and specials are designed and fabricated to satisfy specific project requirements and to minimize the need for cutting and fitting in the field.

## FITTINGS

Fittings are fabricated from welded steel sheet or plate, have steel joint connections welded to the ends, and are lined and coated with cement mortar or concrete. As with concrete pressure pipe, this combination of materials results in an efficient use of the tensile strength of steel with the compressive strength and corrosion-inhibiting properties of cement mortar.

A wide variety of standard fittings is available to satisfy most project requirements. If the layout is so complicated or unique that standard fittings will not serve the purpose, then fittings can be custom-made to virtually any size or configuration needed. Common fittings are illustrated in Figure 8-1.

Bends (elbows) are usually designed for deflections ranging from 10° to 90° (Figure 8-2). Bends smaller than 10° or larger than 90° can be supplied when required. Custom bends can be manufactured to the requirements of specific projects.

A reducer or increaser makes a gradual change in the inside diameter (ID) of the line and may be either concentric or eccentric, depending on the design requirement (Figure 8-3). A desirable taper for both eccentric and concentric reducers is 3 in. in diameter per ft of length. Greater tapers may be required because of space limitations and other practical considerations.

A tee or a cross is used for 90° lateral connections (Figure 8-4), and a wye is used for an angle other than 90°.

Bifurcations (Figure 8-5) are used to split the fluid flow between two pipeline branches.

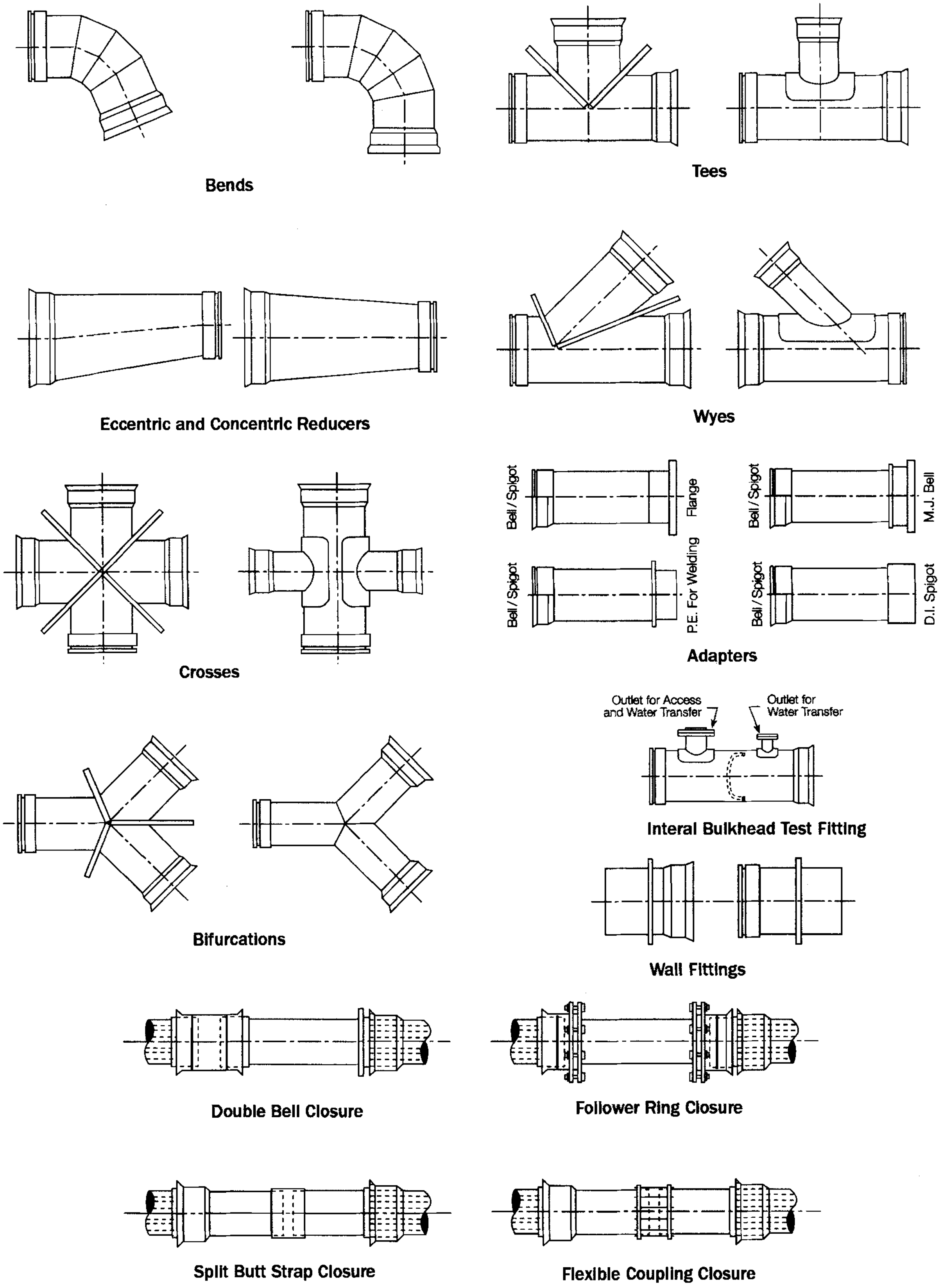



Figure 8-1 Common fittings

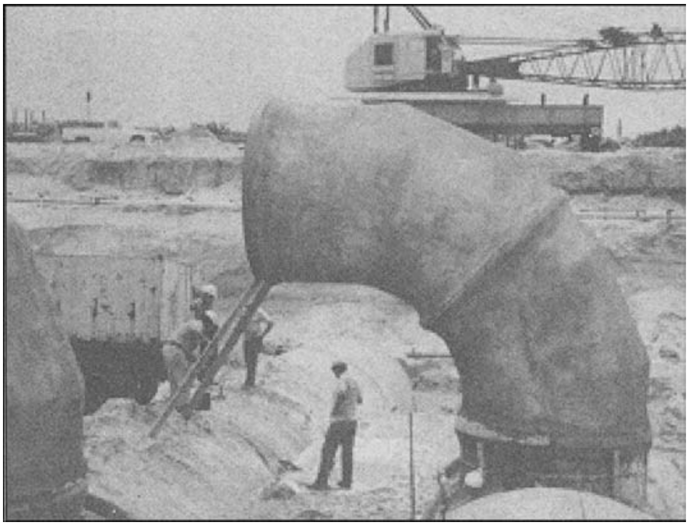


Figure 8-2 A 90-in. (2,290-mm) bend ready to install

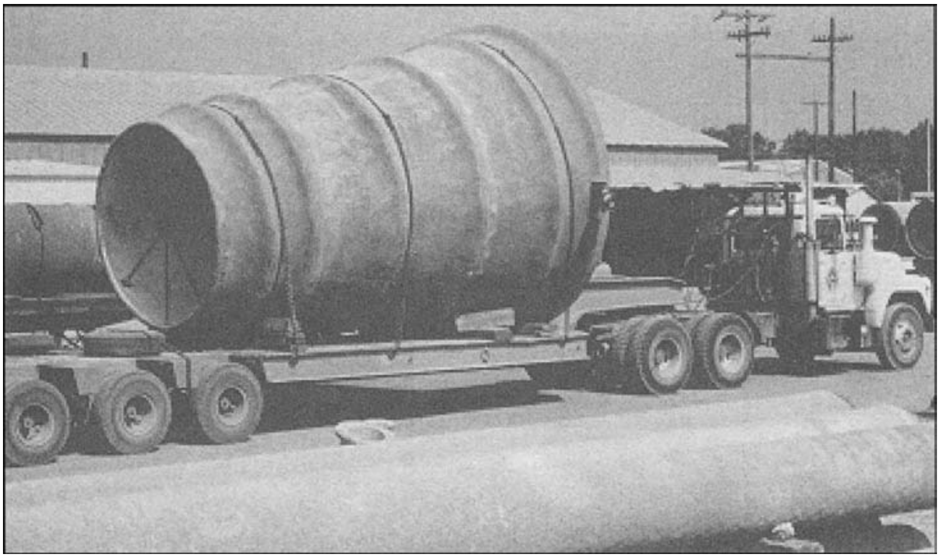


Figure 8-3 A reducer or increaser makes a gradual change in ID of the line

Concrete pressure pipe tees, crosses, wyes, and bifurcations may have either collar, wrapper, or crotch-plate type reinforcing (Figure 8-6).

Adapters are used to connect concrete pressure pipe to valves, couplings, or pipe of a different material (Figure 8-7). A bevel adapter is made by welding together beveled bell and spigot rings and can be coupled to a standard pipe to produce an immediate deflection of up to 5°. To circumvent unexpected obstacles encountered during installation, bevel adapters and random lengths of special short pipe are readily available in a variety of sizes.

A wall piece or a shorter-than-standard piece of pipe is used in a concrete structure wall where the pipeline connects to or passes through the structure wall and is cast/anchored into the wall. To allow for possible differential settlement between the

*Source:* Hanson Pressure Pipe

Figure 8-4 A tee or cross is used for 90° lateral connections

structure and the pipe, the distance between the first joint outside the structure and the exterior face of the structure should not exceed one-half the pipe diameter or 18 in., whichever is larger. When a pipe is installed inside an oversized wall sleeve with compressible material in the annular space between the pipe and the sleeve, a wall piece or joint within one-half a pipe diameter from the exterior face of the structure is not necessary.

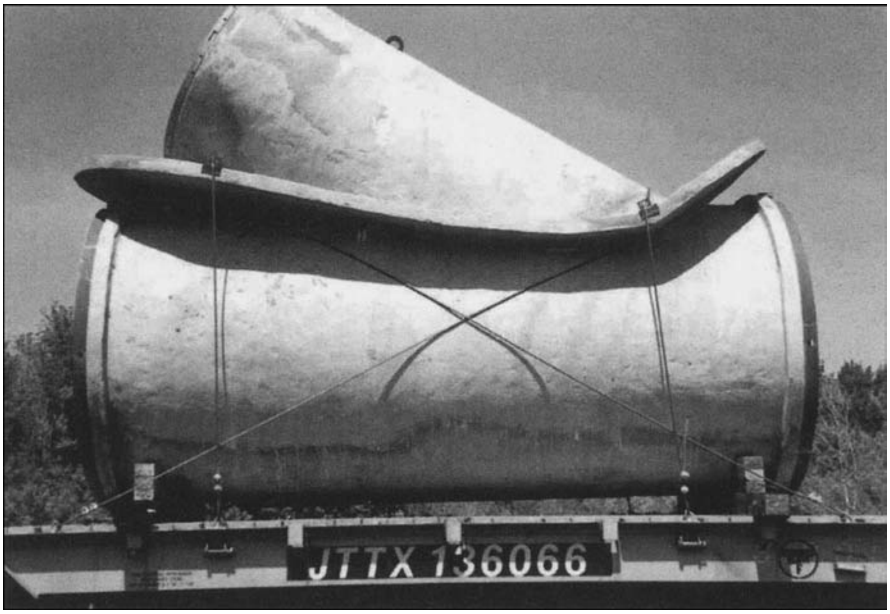
A bulkhead provides a permanent end for a line or a temporary end for hydrostatic testing. It is secured in place by either thrust blocking or restrained joints. Bulkheads are either flat or dish-shaped, depending on the types of thrust restraint employed. An internal bulkhead may be used to temporarily isolate a section of pipeline (36 in. [910 mm] ID or larger) for hydrostatic testing and has the advantage of usually eliminating the necessity of blocking or tying joints for thrust. Nightcaps are flatheaded bulkheads used to temporarily close the ends of a pipeline under construction.

Closures are used to connect installed pipeline sections and are either fabricated to the exact dimension in the plant or cut to fit in the field. Several types of closures are available.

Fittings may be welded together or welded to straight pipe to decrease the number of field joints and expedite installation. Typical end configurations include flange, mechanical joint (bell or spigot), plain steel end (for welding, Victaulic coupling, sleeve-type couplings, and asbestos–cement connections), iron pipe (bell or spigot), or concrete pressure pipe (bell or spigot). Figure 8-8 illustrates typical end configurations.

## FITTINGS DESIGN

Fittings are first designed for internal pressure and then reviewed for external-load capacity. Where additional external capacity is required, the steel cylinder thickness may be increased or supplemental reinforcement added.




*Source:* Hanson Pressure Pipe

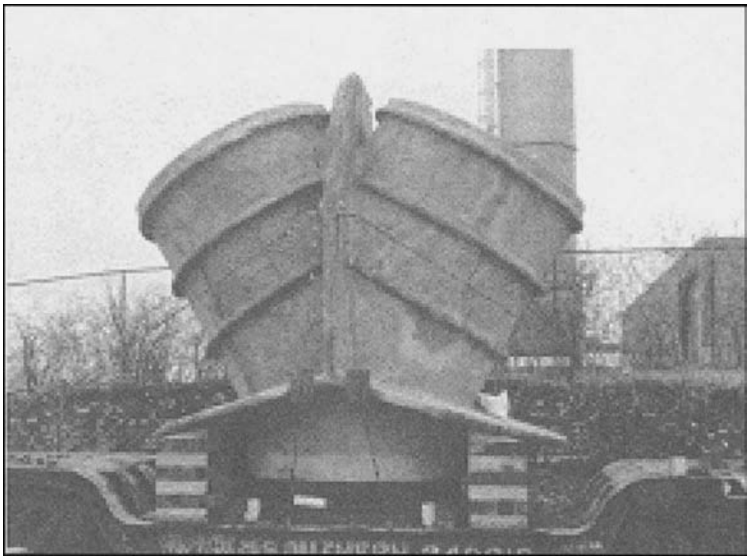
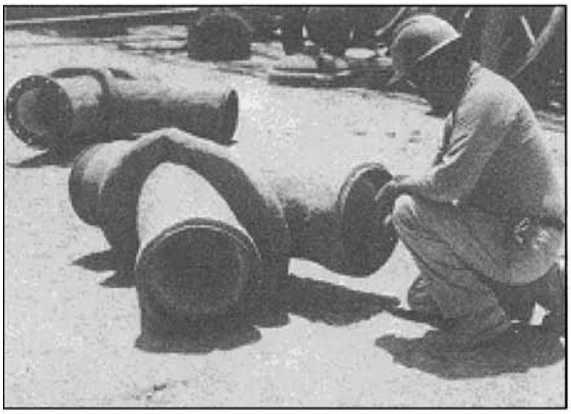


Figure 8-5 Bifurcations for splitting flow

The design thickness of the steel cylinder used to manufacture fittings other than short radius bends is calculated by the "Hoop Stress" equation (also known as the Barlow equation):

$$T_r = \frac{P_w D_{yi}}{2 f_s} \qquad \text{(Eq 8-1)}$$



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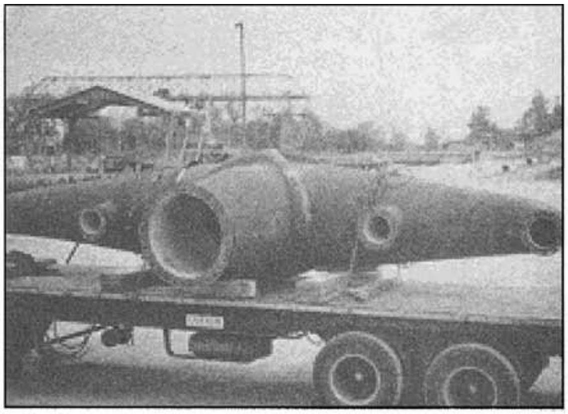

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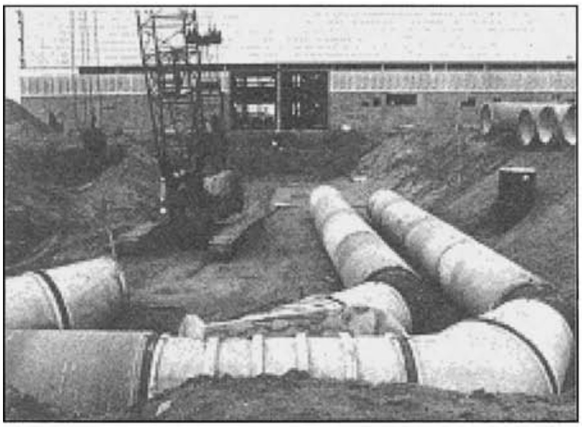

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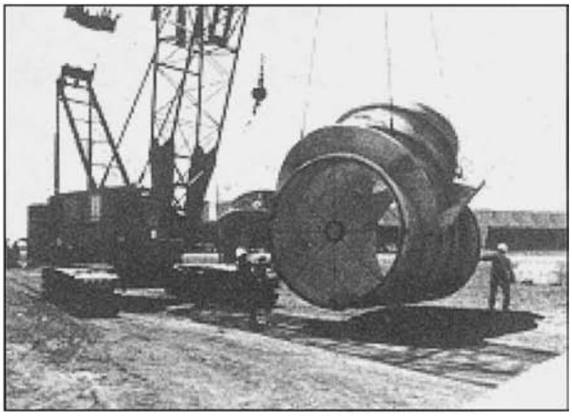

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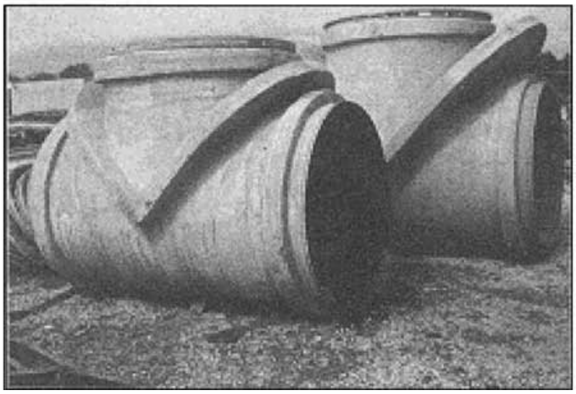

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Figure 8-6 Various types of reinforcing

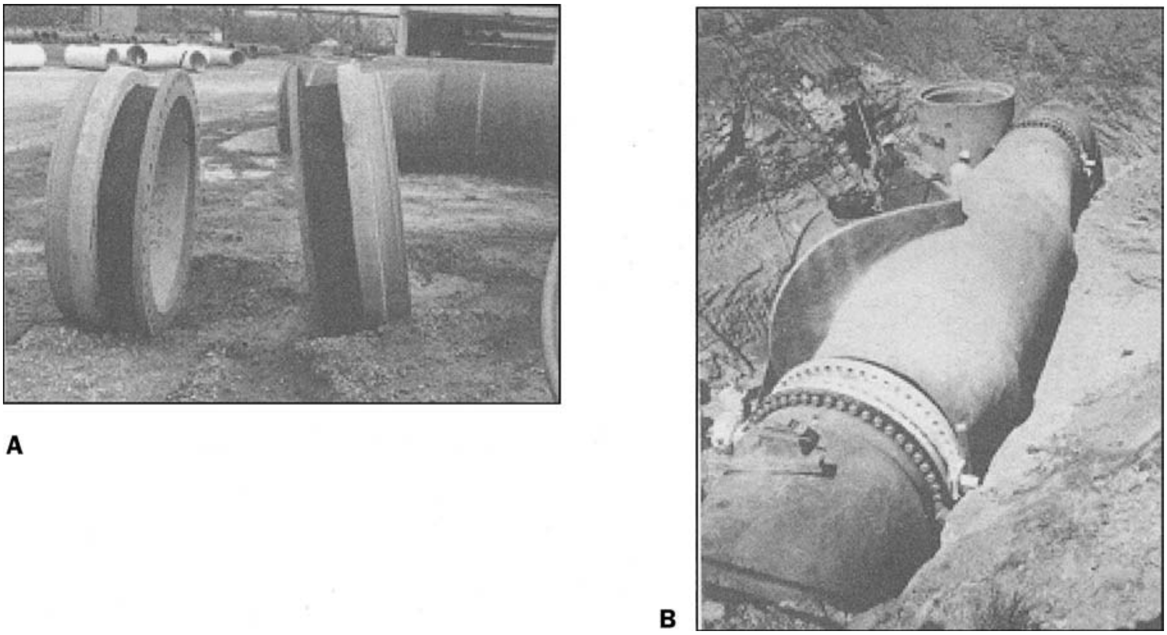


Figure 8-7 Adapters used to connect pipe to valves, couplings, or pipe of a different material

Where:

$T_r$ = required steel cylinder thickness, in.

$P_w$ = working pressure, psi

$D_{yi}$ = inside diameter of the cylinder, in. (For bends, tees, and wyes, $D_{yi}$ is taken conservatively as the OD)

$f_s$ = circumferential stress in the steel cylinder at working pressure, psi, not to exceed 16,500 psi

For reducers, $D_{yi}$ is taken as the ID of the cylinder at the large end.

## Design for External Load

Fittings are generally designed for external load in the manner used for semirigid pipe. This approach is conservative because it ignores the rigidity resulting from the fitting diameter-to-length ratio and from end support provided by the connecting pipe.

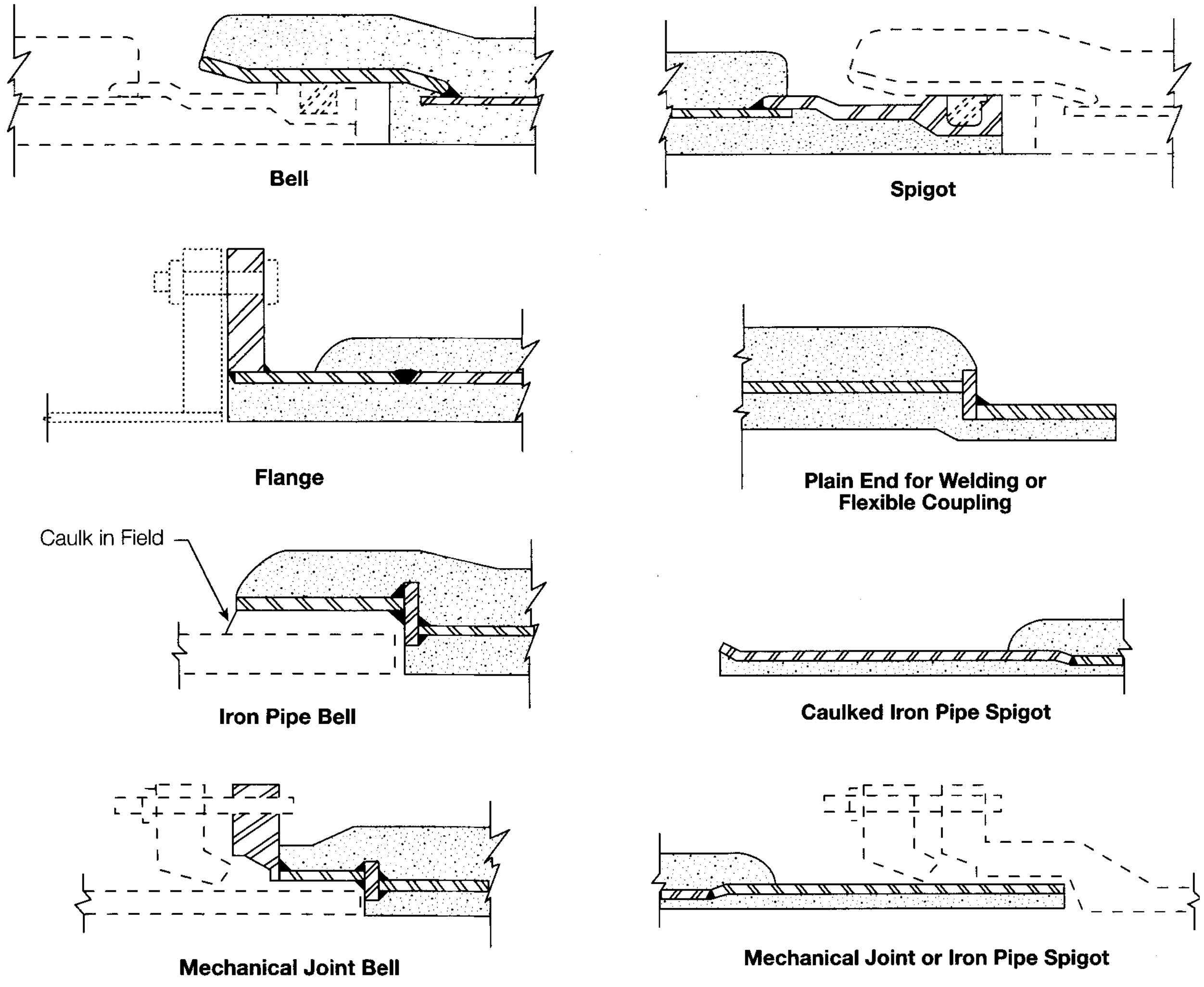
The allowable external load on a semirigid steel fitting is limited by the deflection that can be safely sustained without causing detrimental cracks in the cement–mortar lining and coating. The limiting horizontal or vertical deflection is

$$\Delta x = \frac{D^2}{4{,}000} \text{ or } 0.02D\text{, whichever is less} \qquad \text{(Eq 8-2)}$$

Where:

$\Delta x$ = vertical deflection, in.

$D$ = nominal pipe diameter, in.




NOTE: For clarity, not all welds are shown.

Figure 8-8 Typical end configurations

Fitting deflection under external load can be determined from Spangler's equation, as follows:

$$\Delta x = \frac{D_l k (W/12) r^3}{EI + 0.061E'r^3} \qquad \text{(Eq 8-3)}$$

Where:

- $\Delta x$ = horizontal deflection of fitting, in.
- $D_l$ = deflection lag factor
- $k$ = bedding constant
- $W$ = total external dead plus live load, lb/lin ft
- $r$ = mean radius of fitting wall, in., calculated as $0.5(D + t)$, where $D$ is the inside diameter of the fitting, in., and $t$ is the fitting wall thickness, in.

$EI$ = fitting wall stiffness, in.-lb

$E'$ = modulus of soil reaction, psi

The deflection lag factor $D_l$ for concrete pressure pipe and fittings is taken as 1.0. Values of $k$ for various beddings are provided in chapter 6.

Rewriting Spangler's equation and setting $D_l = 1$, the allowable external load per linear foot of fitting is

$$W = \frac{12(\Delta x)(EI + 0.061E'r^3)}{kr^3} \quad \text{(Eq 8-4)}$$

where all variables are as previously defined, and pipe wall stiffness is calculated as follows:

**Pipe wall stiffness, *EI*.** A cement–mortar-lined-and-coated steel fitting behaves as a three-part laminar ring, and its effective stiffness is

$$EI = (E_c t_l^3 + E_s I_s + E_c t_c^3)/12 \quad \text{(Eq 8-5)}$$

Where:

$EI$ = effective fitting wall stiffness, in.-lb per inch of fitting length

$t_l$ = lining thickness, in.

$t_c$ = coating thickness, in., not to exceed 1.25 in.

$I_s$ = moment of inertia of the steel, in.$^4$ per linear foot

$E_c$ = modulus of elasticity of cement mortar, taken as 4,000,000 psi

$E_s$ = modulus of elasticity of the steel cylinder, taken as 30,000,000 psi

For unreinforced steel cylinders, $I_s$ would be equal to the cube of the cylinder thickness (in inches). When stiffening rings or similar reinforcements are used, $I_s$ may be calculated as the combined moment of inertia of the reinforcement and the steel cylinder.

## Cylinder Stress at Stiffeners

Under high external loads, the load capacity of standard fittings may be less than required. Stiffening rings may be welded to the cylinder to provide additional support. The moments of inertia of the stiffening rings and the cylinder act together to resist the external loads. However, the pressure acting on an unstiffened cylinder will cause greater circumferential strain than will occur immediately under the stiffeners. The differential straining causes longitudinal rim-bending stresses in the cylinder, which may be calculated as follows:

$$f_b = \frac{1.82(A_s - ct_y)}{\left[A_s + 1.56t_y(r_y t_y)^{0.5}\right]} \times \frac{P_w r_y}{t_y} \quad \text{(Eq 8-6)}$$

Where:

$f_b$ = longitudinal rim-bending cylinder stresses at stiffening rings, psi

$A_s$ = cross-sectional area of stiffening ring, in.$^2$

$c$ = width of stiffening ring against cylinder, in.

$t_y$ = cylinder thickness, in.

$P_w$ = working pressure, psi

$r_y$ = mean radius of fitting cylinder, in.

For fittings with unrestrained rubber gasket joints, the longitudinal bending stresses will generally not affect design. However, the rim-bending stresses are additive to longitudinal stresses from thrust restraint or pier-supported pipe and must be considered if stiffening rings are used for such installations.

## Bends

Long-radius curves and small angular changes in pipe alignment are formed by deflecting joints, by beveling pipe ends or using beveled adapters to provide deflection angles up to 5° between adjacent pipe, or by a combination of these. Short-radius curves are formed by pipe bends, which are usually constructed of mitered steel-cylinder segments. The deflection angle between adjacent segments of a bend is limited to 22½°. Adjacent segments are joined by complete-penetration, butt-joint welds.

Bends are fabricated from mitered segments of steel cylinders. A typical four-piece bend and related terminology are shown schematically in Figure 8-9A. From this figure, it can be seen that the projected area from the centerline to the outside of a bend is greater than the projected area from the centerline to the inside of the bend. This difference in projected areas, together with the difference in segment lengths at the inside (throat, as defined in Figure 8-9A) and the outside of the bend, causes hoop stress at the inside to be greater than $P_w D_y/(2t)$ (hoop stress in a straight cylinder) by an amount that is dependent on the ratio of the centerline radius $R$ of the bend (as defined in Figure 8-9A) to its outside cylinder diameter $D_y$.

$$\sum ForceVert = P_w A_{trap} - T_i S_i = 0 \qquad \text{(Eq 8-7A)}$$

Where:

$P_w$ = working pressure, psi

$A_{trap}$ = trapezoid area (cross-hatched in Figures 8-9B and 8-9C), in.$^2$

$T_i$ = stress resultant at the short side of the miter segment, psi

Note that from Figure 8-9C,

$$S_i = 2(R - r_y)\tan\frac{\theta}{2} \qquad \text{(Eq 8-7B)}$$

$$A_{trap} = \frac{r_y}{2}(S + S_i) = r_y(2R - r_y)\tan\frac{\theta}{2} \qquad \text{(Eq 8-7C)}$$

Where:

$S$ = $2R\tan\frac{\theta}{2}$

$T_i$ = $f_s t$ (Eq 8-7D)

$r_y$ = cylinder radius, in. = $D_y/2$

$S$ = length of miter segment at the centerline of the bend, in.

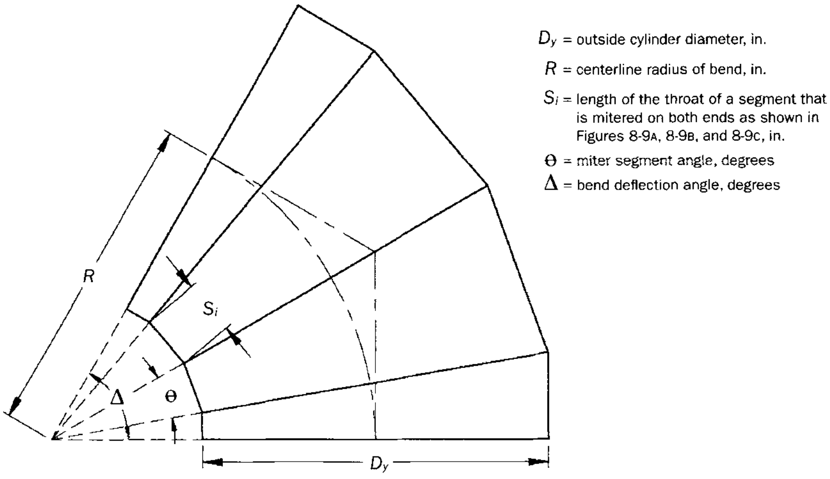



Figure 8-9A Typical four-piece bend

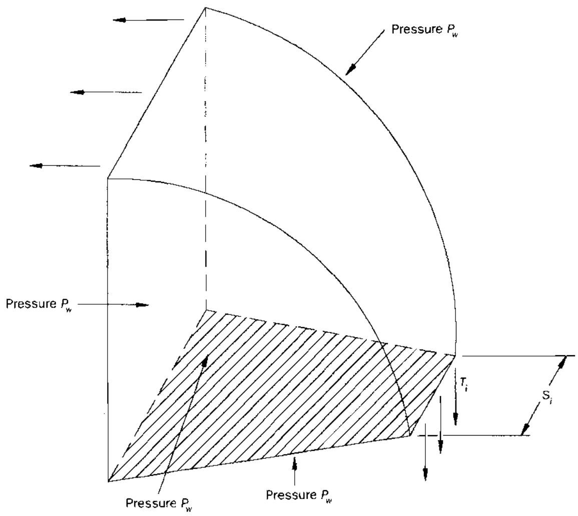



Figure 8-9B Free body diagram of one-fourth of a miter segment and fluid

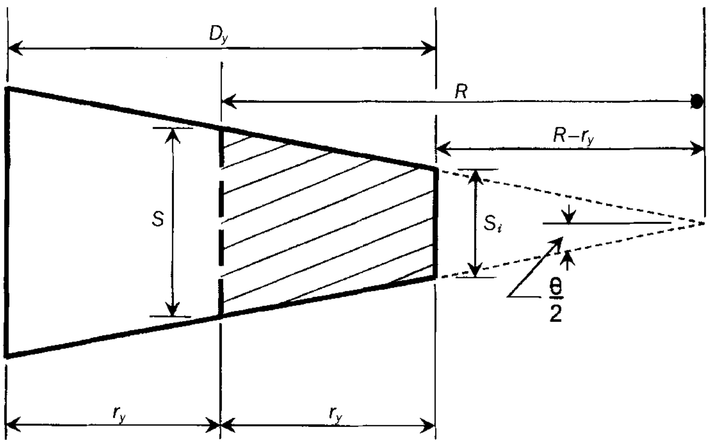



Figure 8-9C Schematic of $A_{trap}$

$f_s$ = hoop membrane stress in the wall section of length $S_i$ due to working pressure, psi

$t$ = minimum wall thickness required for mitered bend, in.

Substituting Eq 8-7B, 8-7C, and 8-7D into Eq 8-7A and solving for $t$ results in:

$$t = \frac{P_w D_y}{2f_s}\left(\frac{R/D_y - 0.25}{R/D_y - 0.50}\right) = \frac{P_w D_y}{2f_s} S_f \qquad \text{(Eq 8-8)}$$

which is recognizable as the Barlow equation multiplied by a shape factor ($S_f$) where:

$S_f$ = shape factor = $\dfrac{K - 0.25}{K - 0.50}$

$K$ = $R/D_y$

When $f_s$ is equal to the maximum allowable membrane stress for working pressure, Eq 8-8 gives the minimum required thickness of steel cylinder for the mitered segment.

The recommended minimum centerline radius of a mitered bend is equal to $D$, the finished (mortar or concrete) inside diameter of the bend. It is recognized, however, that in some cases the centerline radius may be required to be less than $D$. This is acceptable provided that throat lengths are not less than the minimum required throat lengths stated as follows.

The minimum required throat length of segments of bends mitered on both ends is:

$S_{imin}$ = 1.5 in. or $6t$, whichever is greater

The minimum required throat length of segments of bends mitered on one end only is $S_{imin}$ / 2.

Equation 8-8, used for calculation of the minimum required wall thickness of mitered bends, is based on membrane-stress theory. It is appropriate for bends in water-supply transmission, water treatment and distribution pipe lines where stress reversals leading to fatigue, high or low fluid temperatures, space-frame piping arrangements, large miter angles or hazardous fluid conveyance are not involved. If these or other special conditions are encountered, process piping codes should be followed in the design of mitered bends.

## Procedure for the Design of Mitered Bends

1. Using the preferred centerline radius $R$, calculate $S_i$ from:

$$S_i = 2\left(R - \frac{D_y}{2}\right)\tan\frac{\theta}{2}$$

If $S_i$ is less than $S_{imin}$, calculate the required $R$ using:

$$R = \frac{S_{i\,min}}{2\tan\frac{\theta}{2}} + \frac{D_y}{2}$$

2. For two-segment bends only (each segment mitered on one end only), use the throat length of the shorter segment ($S_m$) to compute $R$ using:

$$R = \frac{S_m}{\tan\frac{\theta}{2}} + \frac{D_y}{2}$$

3. Using the bend centerline radius $R$ from step 1 or 2, calculate the minimum required bend cylinder thickness from Eq 8-8.
4. Any cylinder extensions beyond $S_i$ or $S_i/2$ must have plate thicknesses equal to or greater than $t$ calculated in step 3.

## Tees and Wyes

The cylinder thickness required for tees and wyes must be increased or the fitting must be reinforced to compensate for material removed from the pipe wall at the branch. Typical types of reinforcement are collars, wrappers, and crotch plates (Figure 8-6). The selection of the appropriate type of reinforcement and the decision of whether to add cylinder thickness depend on the relative size of the branch diameter versus the run diameter and on a factor known as the *pressure–diameter value*, which is calculated as

$$PDV = \frac{P_w}{D_y}(d_{yo} / \sin\Delta)^2 \qquad \text{(Eq 8-9)}$$

Where:

| | | |
|---|---|---|
| $PDV$ | = | pressure–diameter value |
| $P_w$ | = | working pressure, psi |
| $d_{yo}$ | = | branch cylinder outside diameter, in. |
| $D_y$ | = | run cylinder outside diameter, in. |
| $\Delta$ | = | branch angle of deflection, in degrees |

When $PDV$ is greater than 6,000, crotch plates should be used to reinforce tees and wyes. When $PDV$ is less than or equal to 6,000, either a wrapper or a collar may be used.

## Design of Wrappers and Collars for Tees and Wyes

Wrappers and collars are designed to replace that area of steel removed from the run cylinder that was required to resist internal pressure, multiplied by the dimensionless factor $M$, which is defined as

$$M = \frac{PDV}{4{,}000} \text{ or 1.0, whichever is greater} \qquad \text{(Eq 8-10)}$$

The required replacement steel area $A_r$ is then defined as

$$A_r = \frac{P_w D_{yi}}{2f_s}\frac{d_{yi}}{\sin\Delta}M \qquad \text{(Eq 8-11)}$$

Where:

| | | |
|---|---|---|
| $A_r$ | = | required cross-sectional area of outlet reinforcement, in.$^2$ |
| $P_w$ | = | working pressure, psi |
| $D_{yi}$ | = | run cylinder inside diameter, in. |

$d_{yi}$ = branch cylinder inside diameter, in.

$f_s$ = allowable tensile stress at working pressure, psi, not to exceed 16,500 psi

$\Delta$ = branch angle of deflection, in degrees

The area $A_r$ may be provided all or in part by areas in the run and branch walls in excess of those required to resist internal pressure. These areas are depicted as $A_1$ and $A_2$ in Figure 8-10 and are given by

$$A_1 = \frac{d_{yo} + 2T_y}{\sin \Delta}\left(t_y - t_r\right) \qquad \text{(Eq 8-12)}$$

$$A_2 = 2(2.5T_y)(T_y - T_r) \qquad \text{(Eq 8-13)}$$

Where:

$A_1$ = excess steel area in run cylinder, in.$^2$

$A_2$ = excess steel area in branch wall, in.$^2$

$d_{yo}$ = branch cylinder outside diameter, in.

$t_y$ = actual steel thickness in run cylinder, in.

$t_r$ = required steel thickness in run cylinder for internal pressure, in.

$T_y$ = actual steel thickness in branch wall, in.

$T_r$ = required steel thickness in branch wall for internal pressure, in.

When $A_r > (A_1 + A_2)$, additional steel area must be provided by a collar, wrapper, or increased cylinder thickness. The additional area required is

$$2wT = A_r - (A_1 + A_2) \qquad \text{(Eq 8-14)}$$

Where:

$w$ = effective shoulder width of added steel, in., within the following limits: $d_{yo}/(3 \sin \Delta) \leq w \leq d_{yo}/(2 \sin \Delta)$

$T$ = added steel thickness, in.

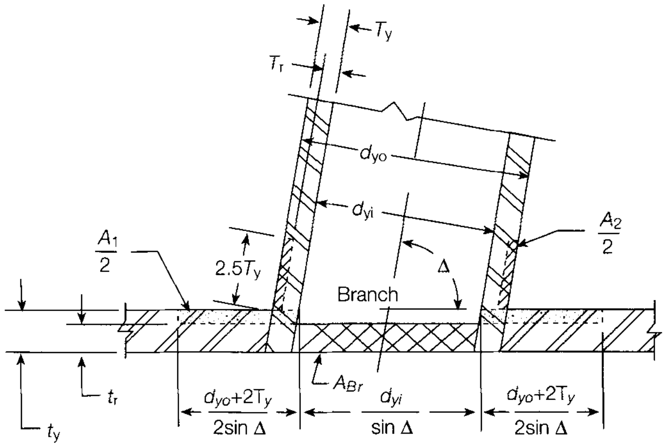
and other variables were previously defined.

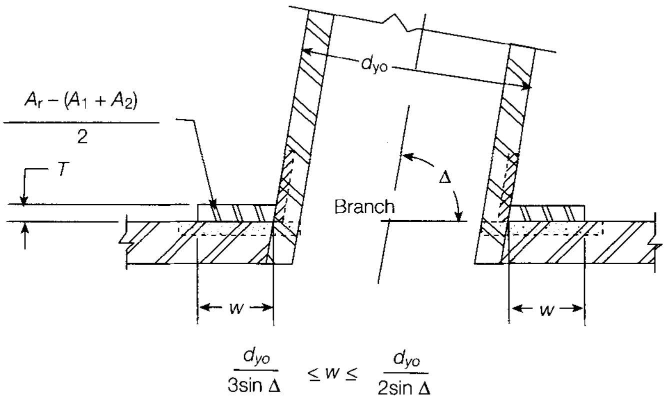
## Design of Crotch Plates for Tees and Wyes

Crotch plates are designed as curved beams carrying the unbalanced hydrostatic loads in the area of the opening made for branching. Figure 8-11 illustrates the assumed load distribution for an equal-diameter wye with two-plate reinforcement. For simplification, a nomograph design procedure has been developed based on these assumed loadings.

The nomograph design, based on design working pressure plus surge allowance, includes a safety factor that keeps stresses well below the yield point of steel. The minimum yield strength of the steel used in this procedure is 30,000 psi (207 MPa).

The design pressure used in the nomograph is kept to 1.5 times the working pressure in order to approximate an allowable stress of 20,000 psi (138 MPa).




**NOTE:**

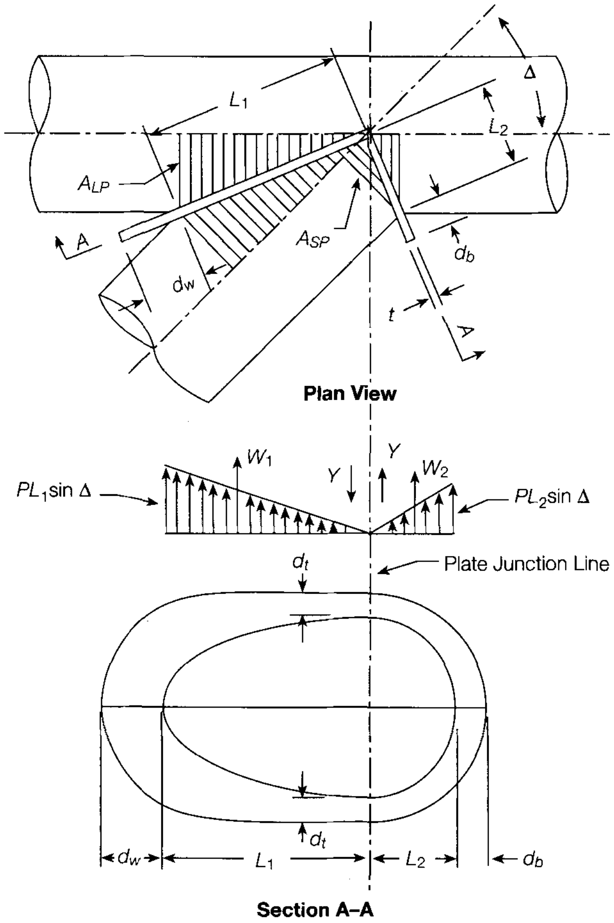
Thicknesses $t_r$ and $T_r$ are required to resist internal pressure in the run and branch, respectively, as shown in diagram (A). The required replacement steel area $A_r$ is equal to $M$ (from Eq 8-10) times the area $A_{Br}$ in diagram (A). $A_r$ may be provided all or in part by area $A_1$ and $A_2$, which represent areas in excess of those required for internal pressure as shown in diagram (A). When $(A_1 + A_2) < A_r$, additional steel must be provided in the form of a collar or wrapper having a cross-sectional area of $2wT = A_r - (A1 + A2)$ as shown in diagram (B).

Figure 8-10 Replacement of steel at openings in fabricated fittings requiring collar or wrapped type of reinforcement

Step 1 Lay a straightedge across the nomograph (Figure 8-12) through the appropriate points on the run cylinder inside diameter and internal pressure scales, and then read the depth of plate from its scale. This reading is the crotch depth for a 1-in. thick, two-plate reinforcement for a 90° wye branch that is the same diameter as the run.

Step 2a If the wye-branch deflection angle is other than 90°, use the $N$-factor curve (Figure 8-13) to get the factors which, when multiplied by the depth of plate found in step 1, will give the wye depth $d_w$ and the base depth $d_b$ for the new wye branch.

Step 2b If the wye branch has unequal diameter pipe, the larger run cylinder diameter will have been used in steps 1 and 2a, and these results




$A_{LP}$ = area of hydrostatic load to be carried by large plate
$A_{SP}$ = area of hydrostatic load to be carried by small plate
$L_1$ = length of axis of large plate
$L_2$ = length of axis of small plate
$P$ = internal pressure
$\Delta$ = branch angle
$W_1$ = total hydrostatic load on large plate
$W_2$ = total hydrostatic load on small plate
$Y$ = load necessary to cause equal deflection of plates
$d_w$ = depth of wye section
$d_b$ = depth of base section
$d_t$ = depth of top section
$t$ = crotch-plate thickness

Figure 8-11 Assumed hydrostatic load distribution for an equal-diameter wye with two-crotch-plate reinforcement

should be multiplied by the $Q$ factors found on the single-plate stiffener curves (Figure 8-14) to give $d'_w$ and $d'_b$. These factors vary with the ratio of the ID of the small cylinder to the ID of the large cylinder.

Step 3 If the wye depth $d_w$ found so far is greater than 30 times the thickness of the plate (1 in.), then $d_w$ and $d_b$ should be converted to conform to a greater thickness $t$ by use of the general equation

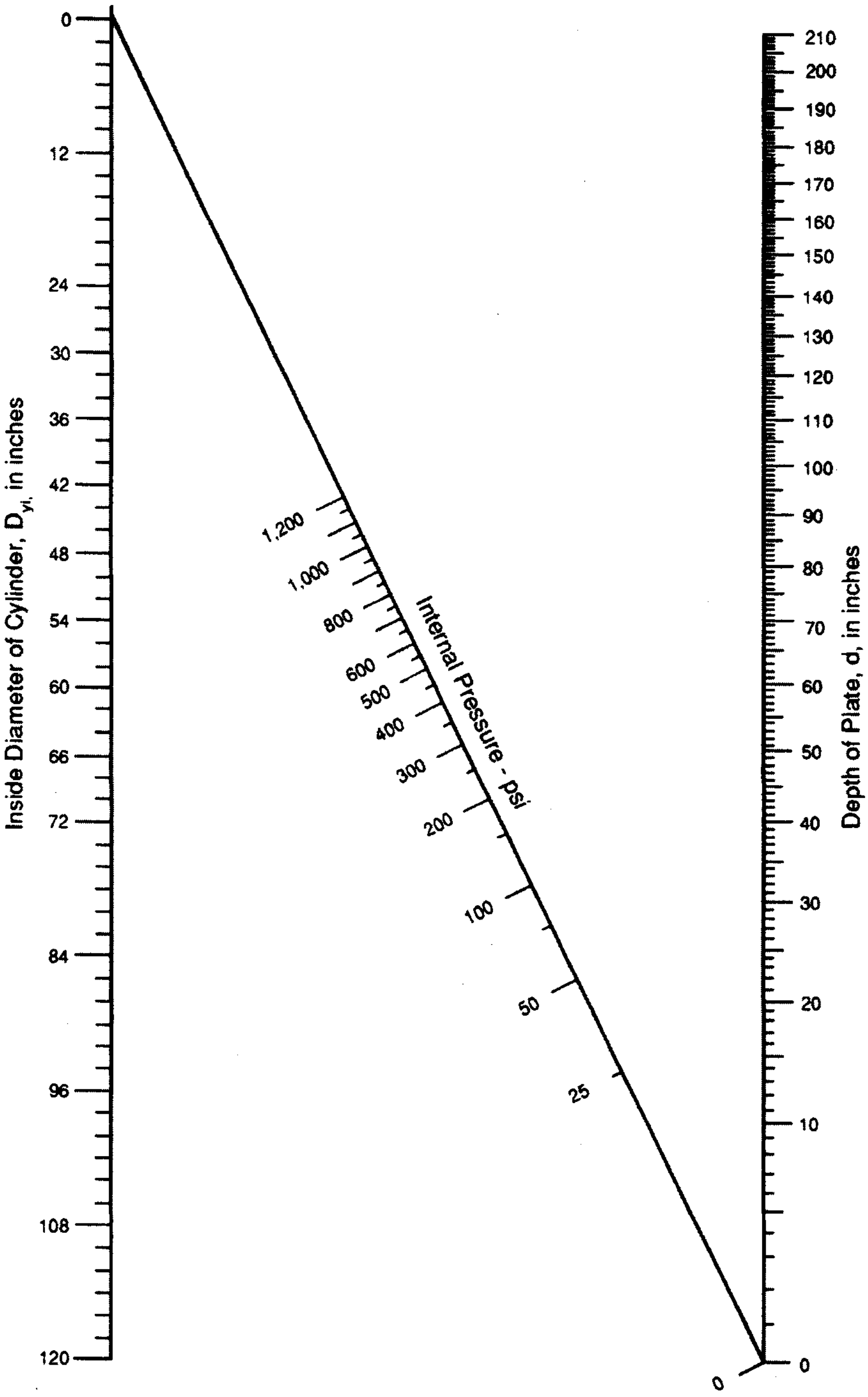



Plate thickness: 1 inch; deflection angle: 90°

Figure 8-12 Nomograph for selecting reinforcement plate depths of equal-diameter pipes

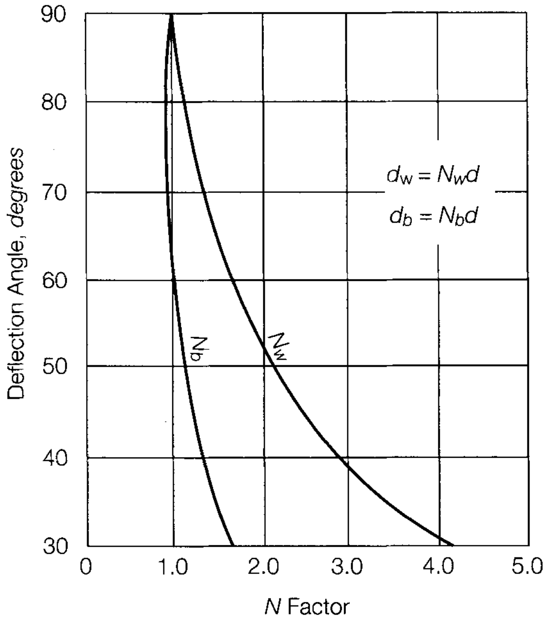



**Figure 8-13** $N$-factor curves for branch deflection angles less than 90°

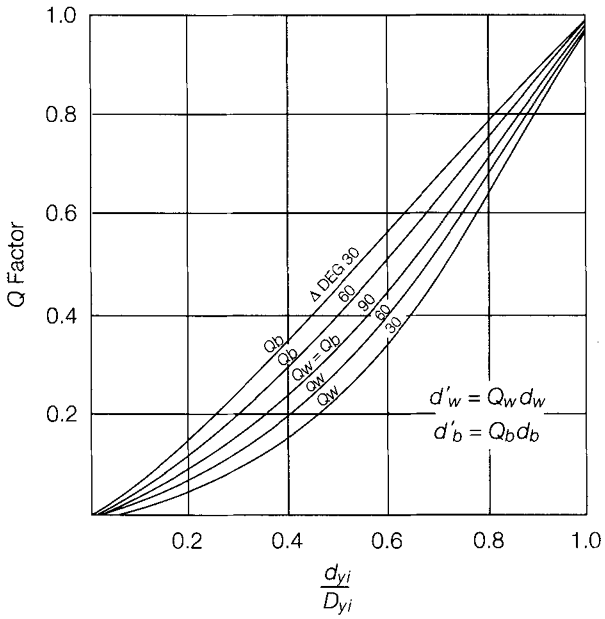


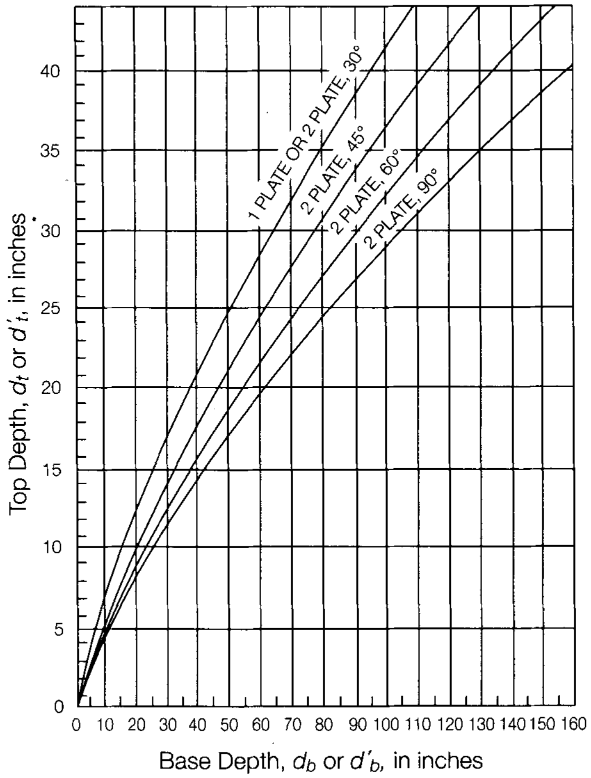

**Figure 8-14** $Q$-factor curves for branch diameter smaller than run diameter

$$d = d_1 \, (t_1/t)^{[0.917 - (\Delta/360)]} \qquad \text{(Eq 8-15)}$$

Where:

$d$ = new depth of plate, in.

$d_1$ = existing depth of plate, in.




$d'_t$ and $d'_b$ are one plate design dimensions;
$d_t$ and $d_b$ are two plate design dimensions.

Figure 8-15 Selection of top depth

$t_1$ = existing thickness of plate, in.

$t$ = new thickness of plate selected, in.

$\Delta$ = deflection angle of the wye branch, in degrees

Equation 8-15 can also be used to convert to a thinner crotch plate.

Step 4 To find the top depth $d_t$ or $d'_t$, use Figure 8-15, in which $d_t$ or $d'_t$ is plotted against $d_b$ or $d'_b$. This dimension gives the top and bottom depths of plate at 90° from the crotch depths.

Step 5 The interior curves follow the cut of the fitting cylinder, but the outside crotch radius in both crotches should equal $d_t$ plus the inside radius of the smaller cylinder, or in the single plate design, $d'_t$ plus the inside radius of the smaller cylinder. Tangents connected between these curves complete the outer shape.

## Example 1 — One-Plate Reinforcement Design (Figure 8-16A)

$D_{yi}$ = 61.5 in.

$d_{yi}$ = 43.5 in.

$\Delta$ = 45°

$P_w$ = 150 psi

Design pressure = 150(1.5) = 225 psi

Step 1 With the larger cylinder diameter of 61.5 in. and the design pressure of 225 psi, read the critical plate depth $d$ from the nomograph ($t$ = 1 in., $\Delta$ = 90°).

$d$ = 37 in.

Step 2a Using the deflection angle 45°, find the factors on the $N$-factor curve that will convert the depth found in step 1 to apply to a 45° wye branch ($t$ = 1 in.).

$d_w = N_w d = 2.45(37) = 90.7$ in.

$d_b = N_b d = 1.23(37) = 45.6$ in.

Step 2b With the ratio of the smaller cylinder inside diameter divided by the larger cylinder inside diameter, $d_{yi}/D_{yi}$ = 43.5/61.5 = 0.71, and the deflection angle, $\Delta$ = 45°, use Figure 8-14 to find the $Q$ factors that give the crotch depths for a single-plate wye stiffener ($t$ = 1 in.).

$Q_w$ = 0.52

$Q_b$ = 0.66

$d'_w$ = 0.52(90.7) = 47.2 in.

$d'_b$ = 0.66(45.6) = 30.1 in.

Step 3 Because the depth $d'_w$ is greater than 30 times the thickness $t$, the conversion Eq 8-15 should be used:

$$d = d_1 (t_1/t)^{[0.917 - (\Delta/360)]}$$

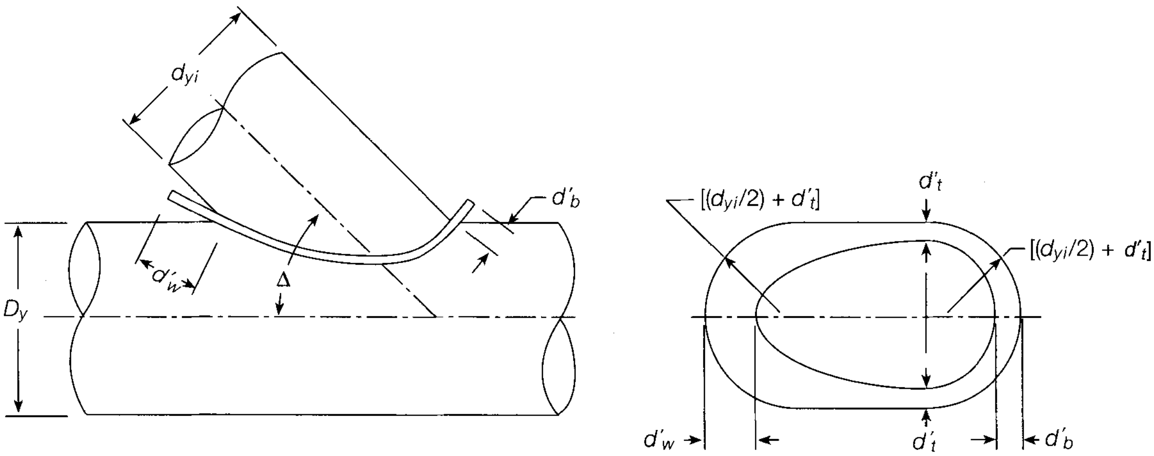
Try a thickness of 1½ in.

$$d = d_1(1/1.5)^{[0.917 - (45/360)]} = d_1(2/3)^{0.792}$$

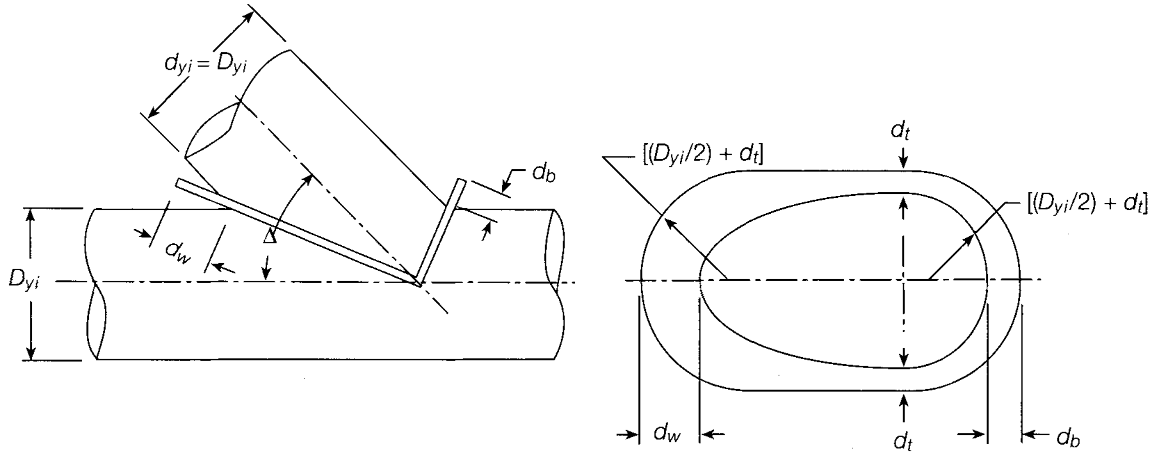
$$d = d_1(0.725)$$

$$d'_w = 47.2(0.725) = 35 \text{ in.}$$

$$d'_b = 30.1(0.725) = 22 \text{ in.}$$




**A. One-Plate Stiffener**




**B. Two-Plate Stiffener**

Figure 8-16 Wye-branch reinforcement plan and layout

Step 4 Find the top depth $d'_t$ from the curve for one-plate design in Figure 8-15.

For $d'_b$ = 22 in., $d'_t$ = 15 in.

Final results:

| | | |
|---|---|---|
| Thickness of reinforcing plate | $t$ | = 1½ in. |
| Depth of plate at acute crotch | $d'_w$ | = 35 in. |
| Depth of plate at obtuse crotch | $d'_b$ | = 22 in. |
| Depth of plate at top and bottom | $d'_t$ | = 15 in. |

Outside radius of plate at both crotches equals the top depth plus the inside radius of the small cylinder $d'_t + (d_{yi}/2) = 15 + (43.5/2) = 36.75$ in.

## Example 2 — Two-Plate Reinforcement Design (Figure 8-16B)

$$D_{yi} = d_{yi} = 73.5 \text{ in.}$$
$$\Delta = 53°$$
$$P_w = 150 \text{ psi}$$
$$\text{Design pressure} = 150(1.5) = 225 \text{ psi}$$

Step 1 With a cylinder diameter of 73.5 in. and a pressure of 225 psi, read the critical depth of plate from the nomograph ($t$ = 1 in., $\Delta$ = 90°).

$$d = 52 \text{ in.}$$

Step 2 From the $N$-factor curve, find the two factors at $\Delta$ = 53°; then, at $t$ = 1 in.

$$d_w = 1.97(52) = 102.5 \text{ in.}$$
$$d_b = 1.09(52) = 56.7 \text{ in.}$$

Step 3 Because $d_w$ is greater than 30 times the thickness of the plate, try $t$ = 2.25 in. in the conversion equation:

$$d = d_1 (t_1/t)^{[0.917 - (\Delta/360)]} = d_1 (1/2.25)^{0.770}$$
$$d = d_1(0.536)$$
$$d_w = 102.5(0.536) = 55 \text{ in.}$$
$$d_b = 56.7(0.536) = 31 \text{ in.}$$

Step 4 Read the top depth $d_t$ from the two-plate design curve in Figure 8-15.

$$d_t = 15 \text{ in.}$$

Final results:

| | | |
|---|---|---|
| Thickness of reinforcing plate | $t$ | = 2.25 in. |
| Depth of plate at acute crotch | $d_w$ | = 55 in. |
| Depth of plate at obtuse crotch | $d_b$ | = 31 in. |
| Depth of plate at top and bottom | $d_t$ | = 15 in. |
| Outside radius of plate at both crotches | 15 + (73.5/2) | = 51.75 in. |

## Three-Plate Design

The three-plate wye branch design (Figure 8-17) is very similar to the two-plate design. The function of the third plate is to act as a clamp in holding down the deflection of the two main plates. In doing so, it accepts part of the stresses of the other plates and permits a smaller design. This decrease in the depths of the two main plates is small enough to make it practical to simply add a third plate to a two-plate design. The additional plate should be considered a means of reducing the deflection at the junction of the plates. The two factors that dictate the use of a third plate are diameter of pipe and internal pressure. When the diameter is greater than 60 in. ID and the internal

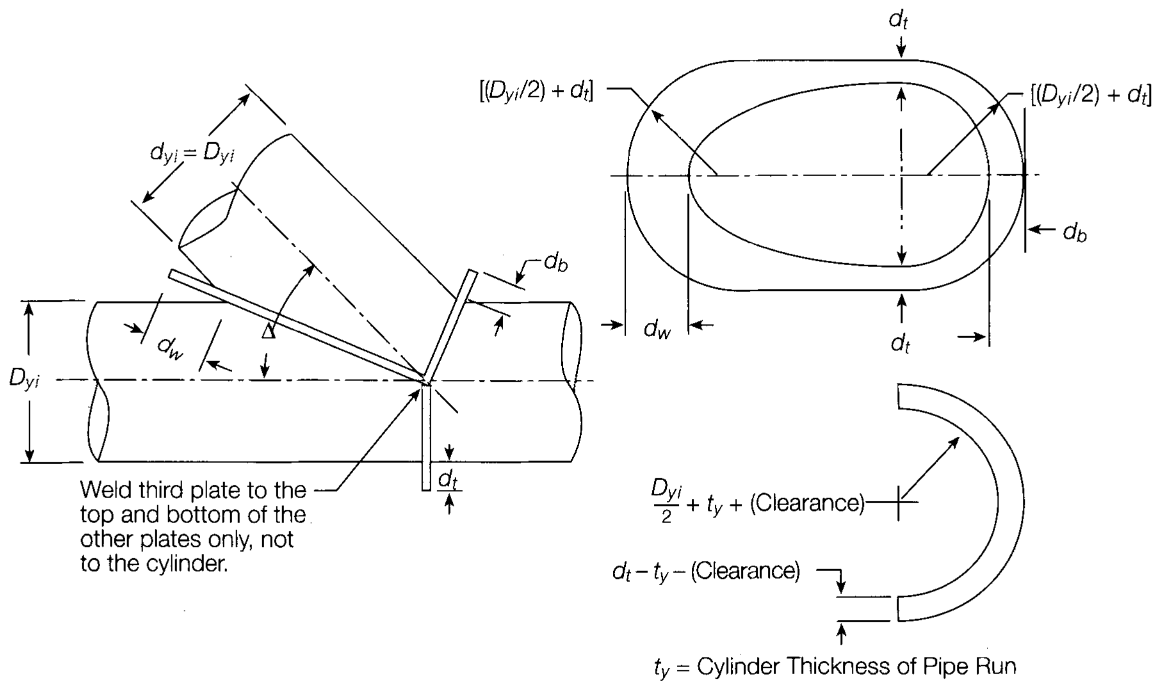



Figure 8-17 Three-plate wye-branch reinforcement plan and layout

pressure is greater than 300 psi, a ring plate can be advantageous. If either of these factors is below the limit, the designer may desire to choose a third plate.

If a third plate is desired as an addition to the two-plate design, its size should be dictated by the top depth $d_t$. Because the other two plates are flush with the inside surface of the steel cylinder, however, the cylinder-plate thickness plus clearance should be subtracted from the top depth. This dimension should be constant throughout, and the plate should be placed at right angles to the axis of the pipe, giving it a half-ring shape. Its thickness should equal that of the main plates.

The third plate should be welded to the other reinforcement plates only at the top and bottom, being left free from the pipe cylinder so that none of the cylinder stresses will be transferred to the ring plate.

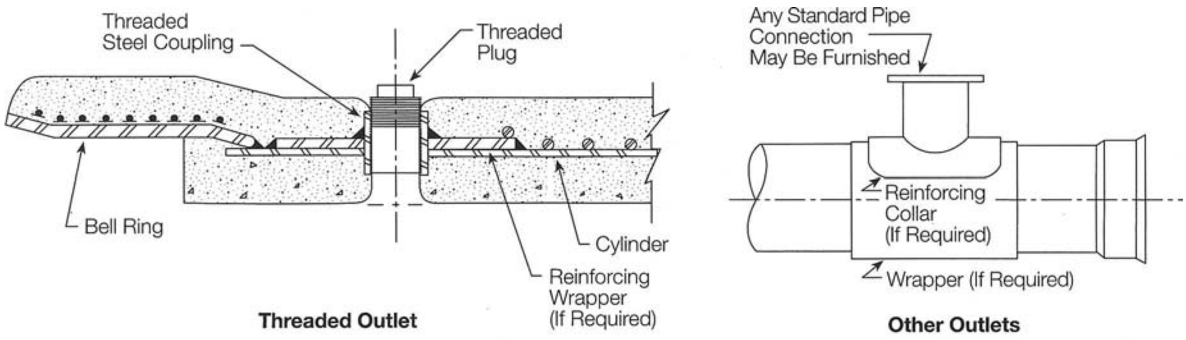
## SPECIALS

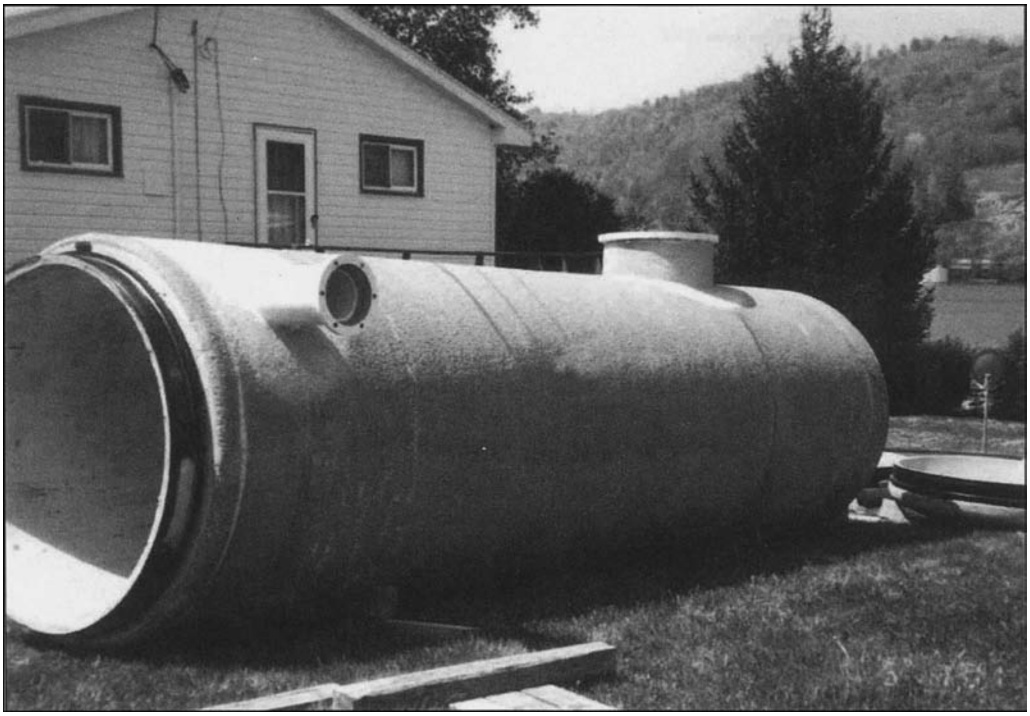
A special has the same basic construction as standard concrete pressure pipe but has some modification, such as a shorter length, beveled end, or a built-in outlet.

A short pipe, as the name implies, is made shorter than the standard length and is used so that a bend, outlet, line valve, and so forth, can be exactly located at a specific station when required.

A beveled pipe has the spigot joint ring attached to the steel cylinder at an angle up to 5° to negotiate minor changes in pipeline direction. Long radius curves and small angular changes in pipe alignment are executed by joint deflection of standard pipe, beveled pipe, bevel adaptors, or a combination of these.

Outlets for manholes, air valves, blowoffs, and lateral connections (Figure 8-18) may be fabricated into special pipe more economically than furnishing fittings, and



*Source:* Hanson Pressure Pipe

Figure 8-18 Typical outlets built in to pipe

the available end configurations for outlets are identical to those for fittings. Threaded outlets up to 3 in. in diameter can also be built into the pipe. The steel cylinder at the outlet opening on pipe is reinforced, in a manner similar to the reinforcement of outlets on fittings, with either a steel collar, saddle plate, wrapper plate, or crotch plate, depending on the design conditions.

## REFERENCES


American Water Works Association (AWWA). 2004. *Steel Pipe — A Guide for Design and Installation*. 4th ed. AWWA Manual M11. Denver, Colo.: AWWA.

Ameron. 1986. *Prestressed Concrete Cylinder Pipe—Design Manual 301.* Monterey Park, Calif.: Ameron.

Den Hartog, J.P. 1952. *Advanced Strength of Materials*. New York: McGraw Hill.

Swanson, H.S. et al. 1955. Design of Wye Branches for Steel Pipe. *Jour. AWWA*, 47:6:581.

Troitsky, M.S. 1982. *Tubular Steel Structures — Theory and Design*. Cleveland, Ohio: James F. Lincoln Arc Welding Foundation.

Young, W.C., and Budynas, R.G. 2002. *Roark's Formulas for Stress and Strain*, 7th ed.

AWWA MANUAL M9

*Chapter* 9

# Design of Thrust Restraints for Buried Pipe

## THRUST FORCES

Thrust forces are resultant forces from internal pressure in fittings at changes in direction (such as in bends, wyes, tees, etc.), at changes in cross-sectional area (such as in reducers), or at pipeline terminations (such as at bulkheads). If not adequately restrained, these forces tend to disengage joints, as illustrated in Figure 9-1, or result in circumferential cracks. Thrust forces of primary importance are (1) hydrostatic thrust due to internal pressure of the pipeline, and (2) hydrodynamic thrust due to changing momentum of flowing water. Because most waterlines operate at relatively low velocities, the dynamic force is insignificant and is usually ignored when computing thrust. For example, the hydrodynamic force created by water flowing at 8 ft/s (2.4 m/s) is less than the hydrostatic force created by 1 psi (6.9 kPa). If the flow velocity is deemed to be large enough that hydrodynamic force should be considered, a textbook on hydraulics that shows the calculation of such forces should be consulted.

For pipelines that are not buried or are buried in soil with zero stiffness, the thrust forces and/or the thrust resistances must be calculated differently than shown in this chapter.

## HYDROSTATIC THRUST

Typical examples of hydrostatic thrust are shown in Figure 9-2. The thrust in buried dead ends, outlets, laterals, and reducers is a function of internal pressure $P$ and cross-sectional area $A$ at the pipe joint as defined by the following equations:

Branch Wye: $T = PA_O$ (Eq 9-1A)

Dead End: $T = PA$ (Eq 9-1B)

Tee: $T = PA_O$ (Eq 9-1C)

Reducer: $T = P(A_1 - A_2)$ (Eq 9-1D)

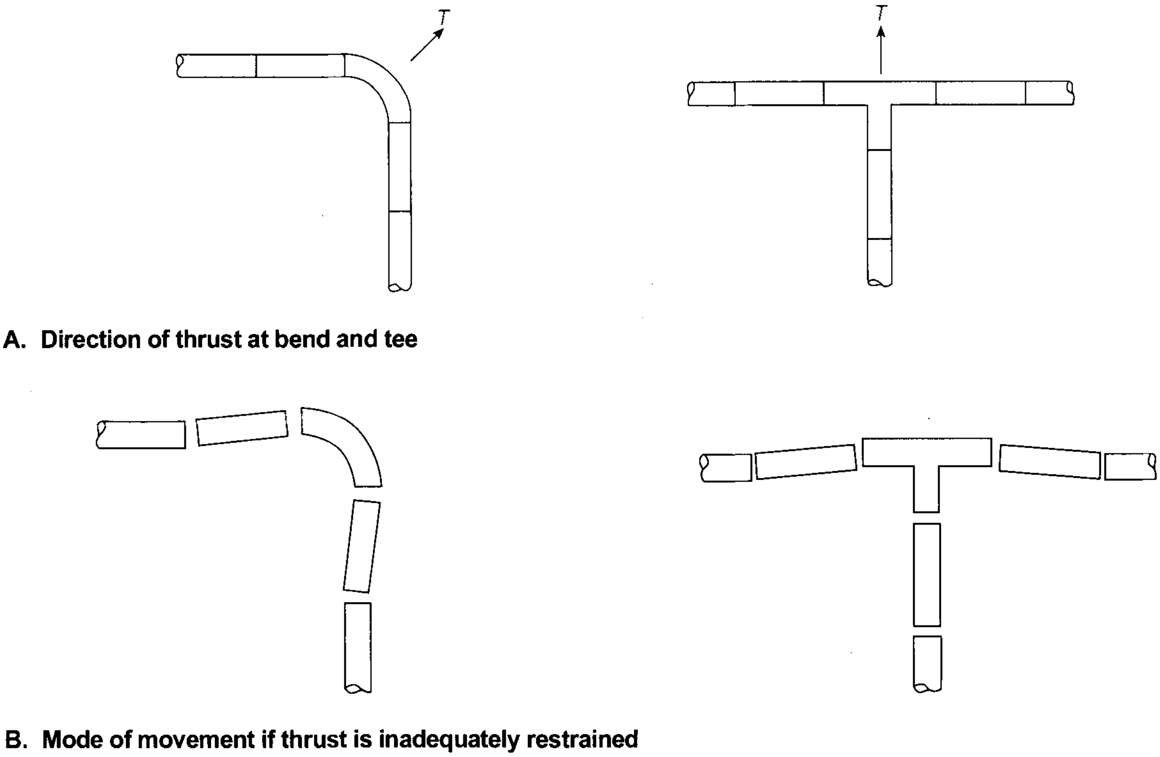



Figure 9-1 Thrusts and movement in pipeline

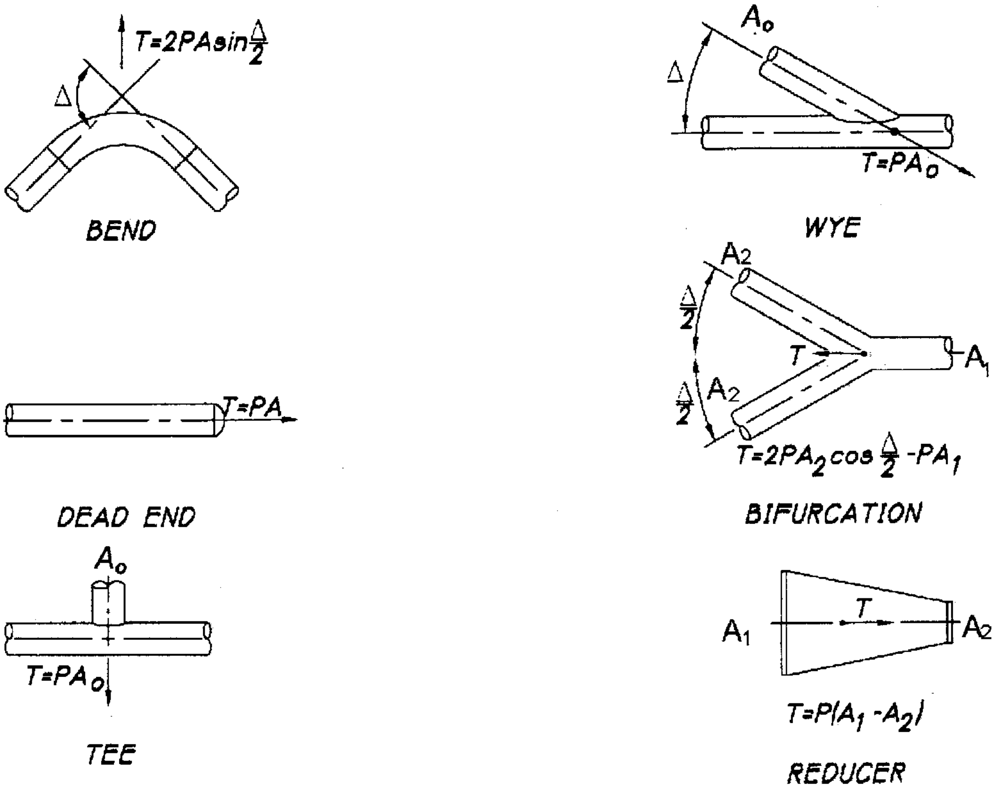



Figure 9-2 Hydrostatic thrust $T$ applied by fluid pressure to typical fittings

The resultant thrust at buried bifurcations and bends is also a function of the deflection angle Δ and is given by:

Bend: $$T = 2PA\sin(\Delta/2) \qquad \text{(Eq 9-1E)}$$

Bifurcation: $$T = 2PA_2\cos(\Delta/2) - PA_1 \qquad \text{(Eq 9-1F)}$$

Where:

$T$ = hydrostatic thrust, lb

$P$ = internal pressure, psi

$A$ = $(\pi/4)D_j^2$ = cross-sectional area of the pipe joint, in square inches, where $D_j$ is the pipe joint diameter, in.

$A_o$ = cross-sectional area of branch pipe joint (refer to Figure 9-2), in.$^2$

$A_1$ = cross-sectional area of larger pipe joint (refer to Figure 9-2), in.$^2$

$A_2$ = cross-sectional area of smaller pipe joint (refer to Figure 9-2), in.$^2$

Δ = deflection angle of bend or bifurcation, in degrees

The equation for the resultant thrust at a bend (Eq 9-1E) is based on forces due to internal pressure only (no hydrodynamic forces), no pressure drop across the bend, and no changes in diameter from one end of the bend to the other end (i.e., a nonreducing bend). Information for calculating the pressure drop over the length of a bend can be found in chapter 3 (Eq 3-9).

## THRUST RESISTANCE

For buried pipelines, thrust resulting from angular deflections at standard and beveled pipe with rubber-gasket joints is resisted by passive resistance of the soil against lateral motion of the pipe and by frictional drag from the dead weight of the pipe, and additional restraint is usually not needed. Thrust at in-line fittings, such as valves, reducers, or internal test plugs, is usually restrained by frictional drag on the longitudinally compressed downstream pipe. Tied joints or thrust blocks are usually not required provided that any in-line appurtenances, such as valves, are designed to transmit the compressive force and the joints are grouted with a grout mix of sufficient strength to safely transmit the compressive force. Other fittings subjected to unbalanced horizontal thrust have two inherent sources of resistance: (1) frictional drag from the dead weight of the fitting, earth cover, and contained water, and (2) passive resistance of soil against the fitting and the pipe. If frictional drag and passive resistance forces are not adequate to resist the thrust involved, they must be supplemented by either increasing the supporting area on the bearing side of the fitting with a thrust block or "tying" adjacent pipe to the fitting to increase passive soil pressure and frictional drag of the line.

Unbalanced uplift thrust at a point of vertical deflection is resisted by the dead weight of the fitting, earth cover, and contained water. If that is not adequate to resist the thrust involved, then it must be supplemented either by increasing the dead weight with a gravity-type thrust block or by increasing the dead weight of the line by "tying" adjacent pipe to the fitting.

When a high water table or submerged conditions are encountered, the effects of buoyancy on all materials should be considered.

One particular concern in pipeline design, installation, and maintenance is the possibility of substantial excavation close to previously installed thrust blocks or restrained pipe and fittings. During installation, attention must be paid to the soil behind thrust blocks and the backfill around restrained pipes and fittings where soil stiffness and friction will be providing thrust resistance. During operation, excavation of this soil while the pipeline is under pressure should be avoided. This is especially important in parallel lines installed close to each other. The pipeline owner must take action to preserve the soil system's ability to resist the thrust forces. The function of thrust blocks and restrained pipes is to transmit thrust forces to the soil structure. If that soil is removed or significantly disturbed with the pipeline under pressure, the safety and stability of the system may be compromised. Proper systems management, engineering, and construction judgment must be exercised in these conditions.

## THRUST BLOCKS

Thrust blocks increase the ability of fittings to resist movement by increasing the bearing area. Typical thrust blocking of a horizontal bend (elbow) is shown in Figure 9-3.

### Calculation of Size

Thrust-block size can be estimated based on the transverse lateral or bearing resistance of the soil.

$$\text{Area of block} = L_B \times H_B = T/\sigma \qquad \text{(Eq 9-2)}$$

Where:

$L_B \times H_B$ = area of bearing surface of thrust block, ft²

$T$ = thrust force, lb

$\sigma$ = transverse lateral or bearing resistance value for soil, lb/ft²

If it is impractical to design the block so that the thrust force passes through the geometric center of the soil-bearing area, then the design should be evaluated for stability.

After establishing preliminary thrust-block size based on the soil parameters, the shear resistance of the passive soil wedge behind the thrust block should be checked to confirm that it can support the weight of the thrust block without significant settlement relative to the pipe. Determining the transverse lateral or bearing resistance of soils is beyond the scope of this manual. It is recommended that a qualified geotechnical expert be consulted.

### Vertical Bends With Large Deflections

The design of thrust blocks for vertical bends with the pressure-induced thrust pushing down into the soil is as follows. The applied load is the sum of the thrust $T$, the weight of the bend $W_p$, the weight of the fluid $W_f$, and the weight of the soil above the bend $W_e$. The force must not exceed the allowable bearing capacity of the soil times the projected area of the bend. If that value is exceeded, a thrust block must be designed to spread the load to a greater area below the elbow. If that is impractical, then the joints

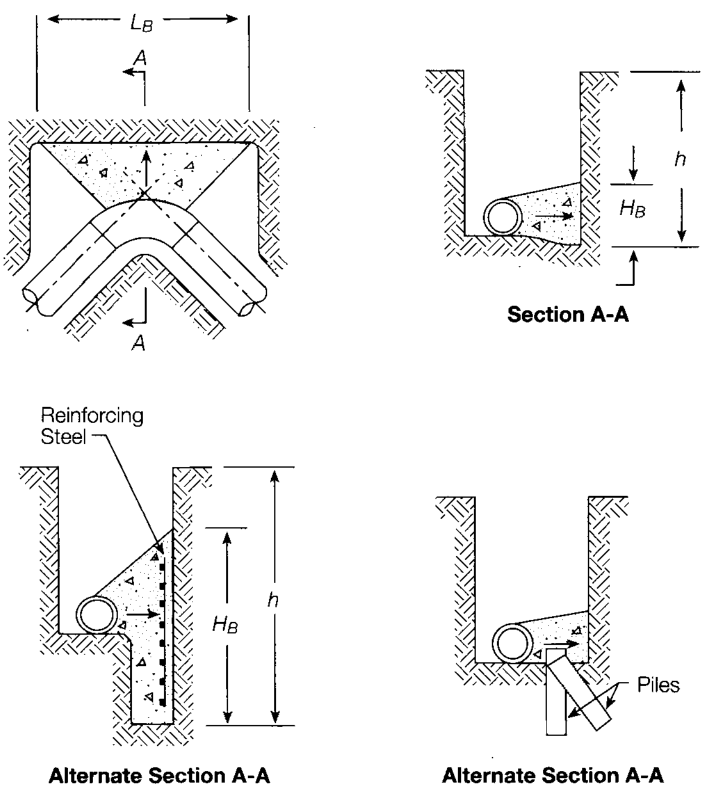



Figure 9-3 Typical thrust blocking of a horizontal bend

must be tied, using the same calculation methods as for horizontal bends described later in this chapter (see Horizontal Bends and Bulkheads beginning on page 132).

The design of thrust blocks for vertical bends with the pressure-induced thrust pushing up out of the soil must be done differently. The thrust $T$ must be resisted by the weight of the bend $W_p$, the weight of the fluid $W_f$, and the weight of the soil above the bend $W_e$. If these weights are exceeded by the thrust $T$, then additional weight, usually in the form of a concrete block, should be added above or below the elbow. As noted in the previous paragraph, if that is impractical, then the joints must be tied, as described later in this chapter (see Tied Joints beginning on page 132).

## Typical Configurations

Knowledge of local soil conditions is necessary for proper sizing of thrust blocks. Figure 9-3 shows several details for distributing thrust at a horizontal bend. Section A-A is the more common detail, but the other methods shown in the alternate sections may be necessary in weaker soils. Figure 9-4 shows typical thrust blocking of vertical bends. The design of the block for a bottom bend is based on the vertical bearing capacity of the soil. The block for a top bend must be sized to adequately resist the vertical component of thrust with consideration of the dead weight of the block, bend, water in the bend, and overburden.

## Proper Construction Essential

Most thrust-block failures can be attributed to inadequate design or improper construction. Even a correctly sized block can fail if it is not properly constructed. A block must be placed against undisturbed soil, and the face of the block must be perpen-

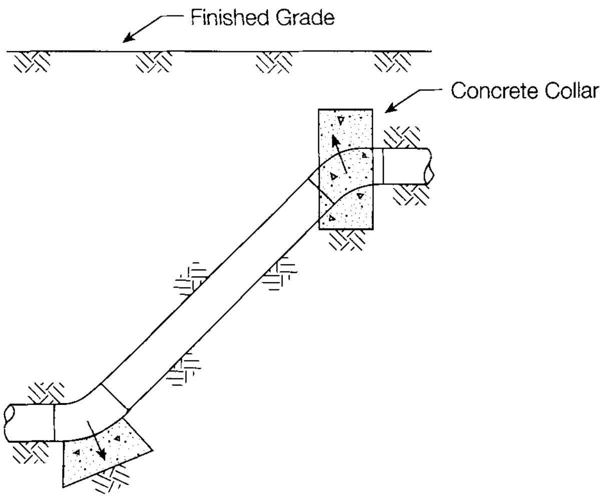



Figure 9-4 Typical profile of vertical-bend thrust blocking

dicular to, and centered on, the line of action of the thrust (Figure 9-5). Many construction and design professionals do not realize the magnitude of the thrusts involved. As an example, a thrust block behind a 36-in. (910-mm), 90° bend operating at 100 psi (689 kPa) must resist a thrust force of nearly 150,000 lb (667 kN). Furthermore, the thrust increases in proportion to the square of the pipe diameter. A 36-in. (910-mm) pipe produces about four times the thrust produced by an 18-in. (460-mm) pipe operating at the same internal pressure.

## JOINTS WITH SMALL DEFLECTIONS

The thrust at beveled pipe or standard pipe installed with small angular deflections is usually so low that supplemental restraint is not required. Two approaches can be taken in the analysis of thrust at small angular deflections. In one approach, a small angle bend is demonstrated to be locked in place in the pipeline by forces transferred through joints to adjacent pipe sections. In the other approach, single or multiple pipe joints with small deflection angles are assumed to be completely free to rotate, and soil friction and passive soil resistance are shown to be more than adequate to resist the thrust. These two analyses demonstrate why some small deflections under moderate pressure do not require thrust blocks or restrained joints.

### Locking of Pipe at Small Deflections

Thrust at small angle bends can be locked in a pipeline by the geometric configuration shown in Figure 9-6. Diagonals *AC* and *BD* form the angle $\theta$ with their respective tangents of the deflection angle. By observation, it is evident that for a bend to move in the direction of the thrust *T*, the two adjacent pipes must rotate about points *A* and *B* on arcs *Cm* and *Dn*. Whenever the angle $\theta$ is larger than $\Delta/2$, the corners *C* and *D* of the two rotating pipes begin to jam and resist movement. The two pipes *E* and *H*, backed by the pipe behind each of them along the tangents, act as abutments supporting pipes *F* and *G* from movement and begin to create a beam from point *A* to point *B* provided that a grout mix of sufficient strength is used to safely transmit the compressive forces. The interlocking bell and spigot joint rings provide the shear resistance necessary at the joints as demonstrated by joint shear tests. As the angle $\Delta/2$ becomes larger than $\theta$, the corners *C* and *D* do not jam and permit the two pipes to rotate and release the bend. Because it is possible for the point of rotation to move out to the second joint, allowing two pipe lengths to act as a straight pipe length

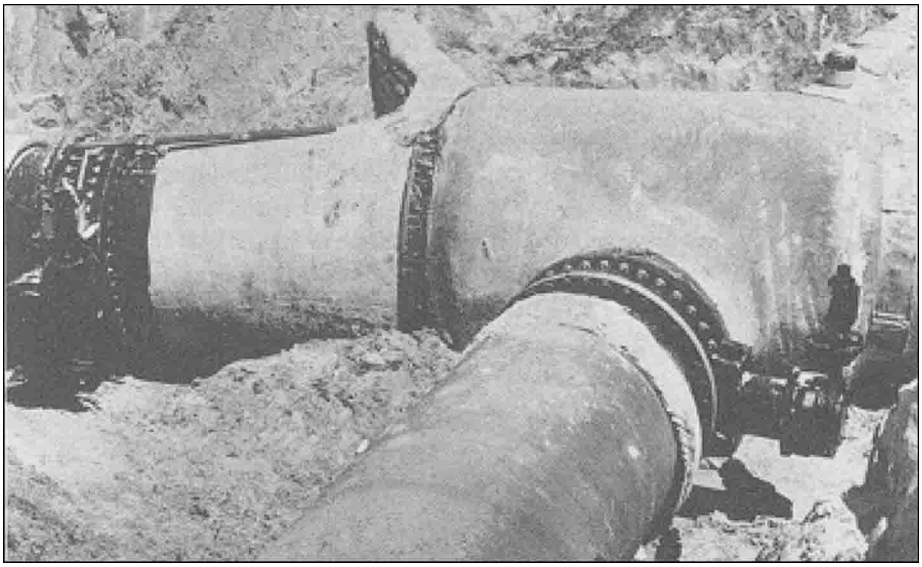


Figure 9-5 Tee and reducer on large-diameter line. Note sandbags behind tee as forms for placement of thrust block.

and to rotate on each side of the bend, bends greater than 5° should be restrained.

Joint locking by beam or bridging action has been demonstrated numerous times in the field. In 1978 a collapsed gravity sewer pipe completely washed out the support from underneath a 36-in. (910-mm) prestressed pipeline in Evansville, Ind. (Figure 9-7). Four lengths of 20-ft (6.1-m) long pipe were left spanning the washout and supporting several large slabs of concrete pavement. In 1957 high water completely washed out the support from under four consecutive 12-ft (3.7-m) lengths of 84-in. (2,130-mm) reinforced concrete pipe providing the entire raw water supply for the City of Dayton, Ohio. In 1988, a sinkhole in Orange County, Fla., left 50 ft (15 m) of 54-in. (1,370-mm) embedded cylinder pipe (2½ pipe lengths) spanning the hole.

In all of these examples, the pipelines, although totally unsupported, continued to operate and carried their own weight, the weight of water inside, and additional external load. These examples demonstrate the ability of concrete pipe joints to lock up in a pipeline and resist load or thrust.

## Small Horizontal Deflections With Joints Free to Rotate

The thrust $T$ at deflected joints on long-radius horizontal curves, which do not have tied joints, is resisted by soil along the outside of the curve as shown in Figure 9-8A. The thrust is resisted by the passive resistance of the soil and the transverse friction at the top and bottom of the pipe. Additional restraint, in the form of tied joints, is not necessary when

$$\frac{T}{L_p} \leq \left[\gamma_s H N_{qh}\left(\frac{D_o}{12}\right) + \mu(1+\beta)(W_e) + \mu\left(W_p + W_f\right)\right] \quad \text{for sand} \qquad \text{(Eq 9-3A)}$$

and

$$\frac{T}{L_p} \leq \left[S_u N_{ch}\left(\frac{D_o}{12}\right) + \mu(1+\beta)(W_e) + \mu\left(W_e + W_f\right)\right] \quad \text{for clay} \qquad \text{(Eq 9-3B)}$$

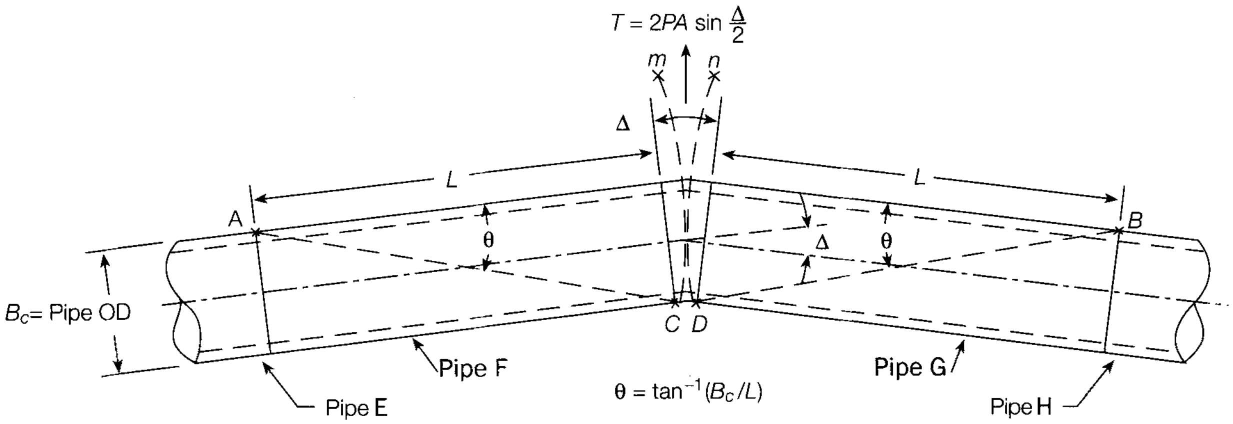



Figure 9-6 Locking of pipe at small deflections

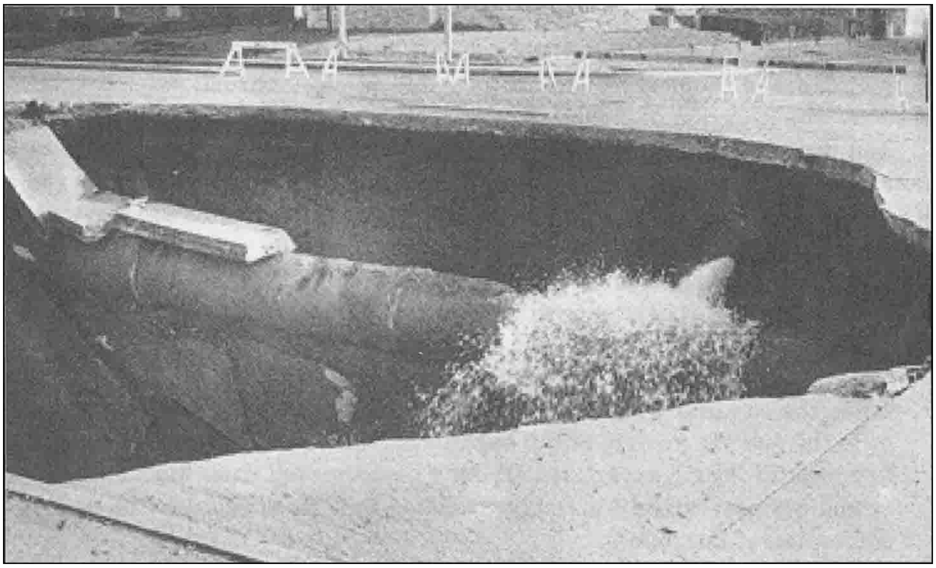


Figure 9-7 Four sections of unrestrained 36-in. (910-mm) prestressed concrete pipe span an 80-ft (24-m) wide washout in Evansville, Ind.

Where:

$T$ = $2PA \sin\frac{\Delta}{2}$, lb

$L_p$ = length of standard or beveled pipe, ft

$\gamma_s$ = unit weight of the soil, lb/ft$^3$

$N_{qh}$ = horizontal bearing capacity factor for sand, as shown in Figure 9-8B

$N_{ch}$ = horizontal bearing capacity factor for clay (equal to $N_{qh}$ for granular soil with friction angle $\varphi$ = 30°)

$S_u$ = undrained shear strength of clay, lb/ft$^2$

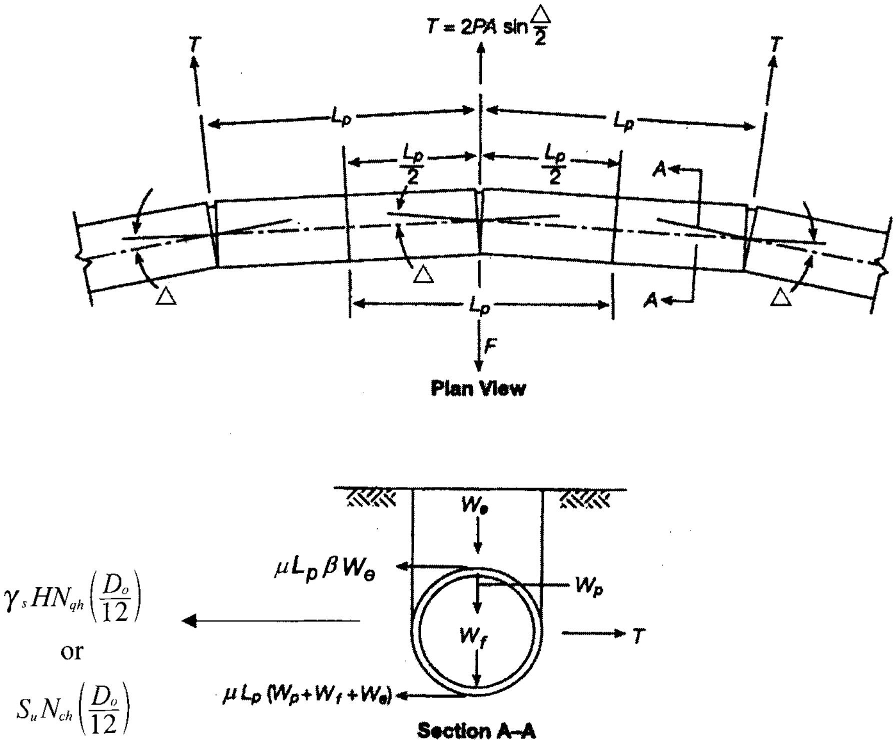



Figure 9-8A Restraint of thrust at deflected, nontied joints on long-radius horizontal curves

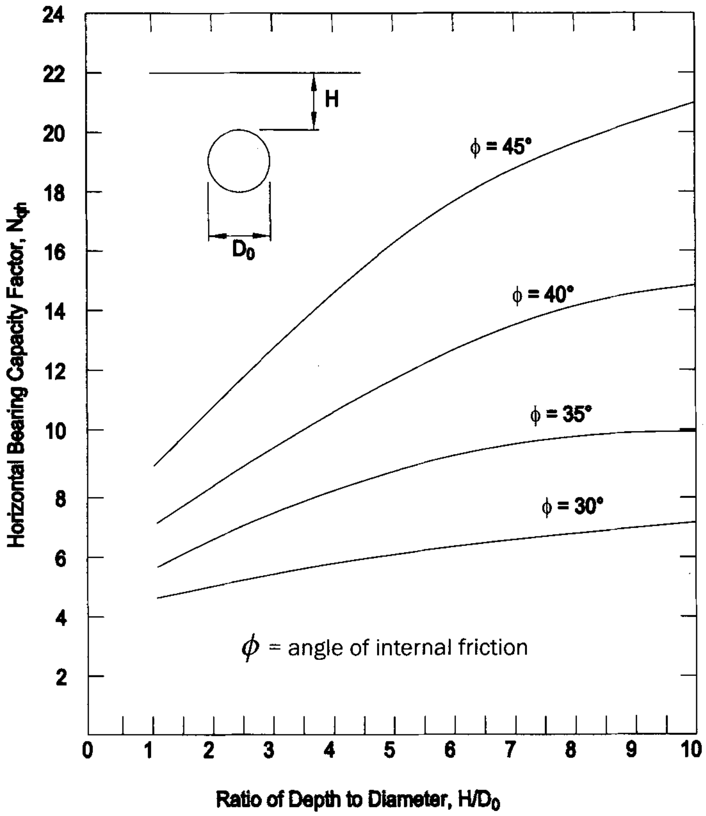



Figure 9-8B Horizontal bearing factor for sand vs. depth-to-diameter ratio ($H/D_0$)(adapted from O'Rourke and Liu [1999])

$H$ = depth of pipe, as shown in Figure 9-8B, ft

$D_o$ = outside diameter of pipe, in.

$\mu$ = coefficient of friction at the pipe–soil interface

$\beta$ = shallow cover factor, Eq 9-8

$W_p$ = weight of pipe, lb/lin ft

$W_f$ = weight of fluid, lb/lin ft

$W_e$ = earth load, lb/lin ft

$\Delta$ = deflection angle, in degrees

## Small Vertical Deflections With Joints Free to Rotate

Uplift thrust at deflected joints on long-radius vertical curves is resisted by the combined dead weight $W_p + W_f + W_e$, as shown in Figure 9-9A. Additional restraint is not required when

$$T \leq L_p(W_p + W_f + W_e)\cos(\alpha - \Delta/2) \quad \text{(Eq 9-5A)}$$

Where:

$T$ = $2PA\sin\frac{\Delta}{2}$, lb

$L_p$ = length of standard or beveled pipe, ft

$W_p$ = weight of pipe, lb/lin ft

$W_f$ = weight of fluid in pipe, lb/lin ft

$W_e$ = earth load, lb/lin ft

$\alpha$ = slope angle, in degrees

$\Delta$ = deflection angle, created by angular deflection of joint, in degrees

The determination of earth load should be based on a backfill density and height of cover consistent with what can be expected when the line is pressurized. Values of the unit weight of backfill vary from 110 to 140 lb/ft$^3$ (1,762 to 2,243 kg/m$^3$), depending on the type of soil and the degree of compaction. When the pipe is laid under the water table, the unit weight of the backfill below the water table must be reduced to $\gamma_s - \gamma_w$, where $\gamma_s$ is the unit weight of the soil, and $\gamma_w$ is the unit weight of the water. Earth load $W_e$ may be computed from Marston's trench and embankment equations or, more conservatively, it can be assumed that $W_e$ is equal to the weight of backfill on top of the pipe, including the upper haunch areas (see Figure 9-9B). This weight is calculated by the following formula:

$$W_e = \gamma\left[\frac{D_o H}{12} + \left(1 - \frac{\pi}{4}\right)\left(\frac{D_o}{12 \times 2}\right)^2\right] \quad \text{(Eq 9-5B)}$$

Where:

$W_e$ = earth load, lb/lin ft

$\gamma$ = unit weight of backfill, lb/ft$^3$, adjusted for reduced weight of backfill, if all or a portion of it is under the water table as described in the preceding paragraph

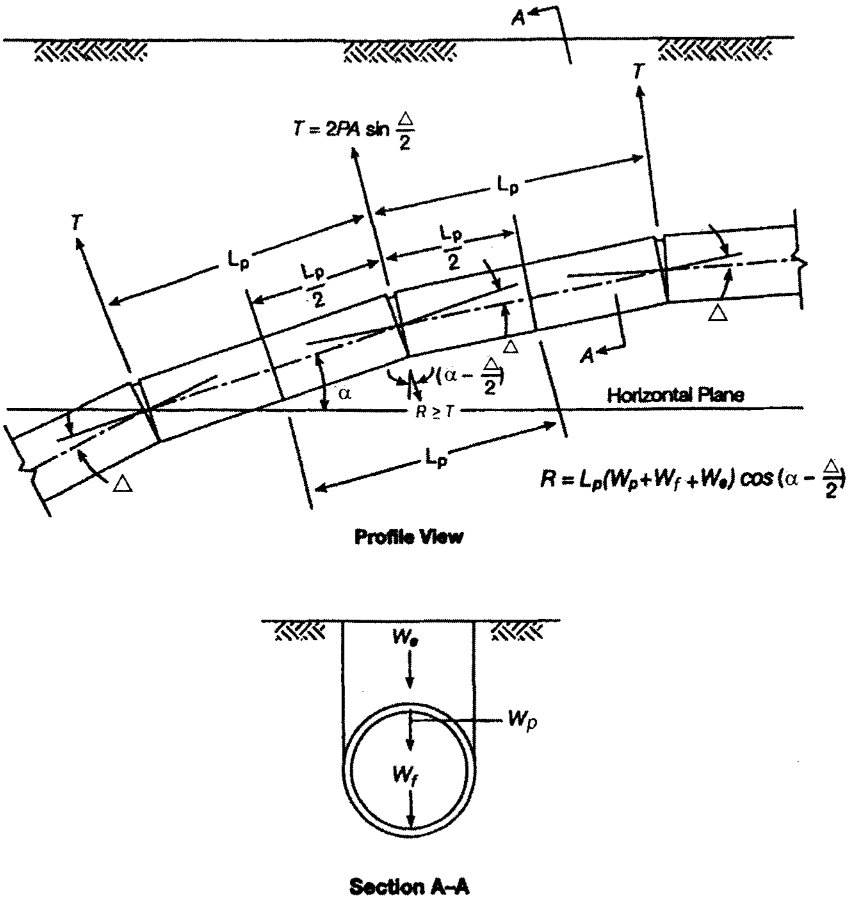



Figure 9-9A Restraint of uplift thrust at nontied, deflected joints on long-radius vertical curves

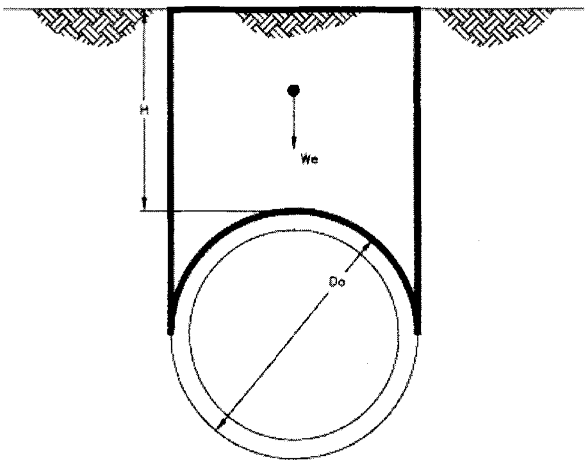



Figure 9-9B Earth load for restrained joint calculations

$D_o$ = pipe outside diameter, ft

$H$ = minimum earth cover, ft

Downward thrust at deflected joints on long-radius vertical curves is resisted by the bearing of the trench against the bottom of the pipe.

# TIED JOINTS

Many engineers choose to restrain thrust from buried fittings by tying adjacent pipe joints. This method fastens a number of pipe joints on each side of the fitting to increase the frictional drag of the connected pipe and resist the fitting thrust. Friction forces on the exterior of the tied pipe act in the opposite direction of the motion of the pipe relative to the soil. Section A–A in Figure 9-8A shows lateral and horizontal friction forces $\mu L_p \beta W_e$ and $\mu L_p(W_p + W_f + W_e)$, which resist thrust $T$. The term $\beta$ is the shallow cover factor represented in Eq 9-8.

Uplift thrust restraint provided by gravity-type thrust blocks shown for the top bend in Figure 9-4 may be supplemented or replaced by the dead weight of the adjacent pipe tied to the vertical bend together with the earth and fluid weights acting on the tied pipe. Section A–A in Figure 9-9A shows vertical forces that act on a buried vertical bend to resist uplift thrust.

## Horizontal Bends and Bulkheads

The thrust at a buried horizontal bend is resisted by the frictional resistance of the soil against axial movement of the pipe as well as the passive resistance of the soil to the transverse movement of the pipe. The frictional resistance of the soil against axial movement of the pipe gives rise to an axial force, and the passive resistance of the soil to the transverse movement of the pipe gives rise to shear and bending in the pipe. The true axial force and the shear in the pipe must satisfy both equilibrium of forces and compatibility of deformations.

The thickness of the steel cylinder and the restrained length depends on whether the joint has mechanical or welded restraint. Pipelines with welded joints typically require thicker steel cylinders and longer tied lengths. For a pipeline designed with mechanically restrained joints, it may be necessary to weld one or more joints because of local installation conditions. Such welding is not expected to alter the behavior of a pipeline with mechanically restrained joints. However, if a large number of joints in a bend need to be welded, a re-evaluation of the thrust restraint system at that bend with welded joints and actual field conditions, as well as the actual soil properties, may be required (Zarghamee et al. 2004).

Figure 9-10 shows the free body diagram of forces acting on the bend and the displacement of the bend. From the equilibrium of forces acting on the bend

$$T = 2PA\sin\left(\frac{\Delta}{2}\right) = 2F_o\sin\left(\frac{\Delta}{2}\right) + 2V_o\cos\left(\frac{\Delta}{2}\right) + 2k\delta\, l_b\cos\left(\frac{\Delta}{2}\right) \qquad \text{(Eq 9-6)}$$

Where:

$P$ = internal pressure, psi

$A$ = $(\pi/4)D_j^2$ = cross-sectional area of the pipe joint, in.$^2$, where $D_j$ is the pipe joint diameter, in.

$\Delta$ = deflection angle of the bend, in degrees

$F_o$ = axial force in the pipe at the fitting, lb

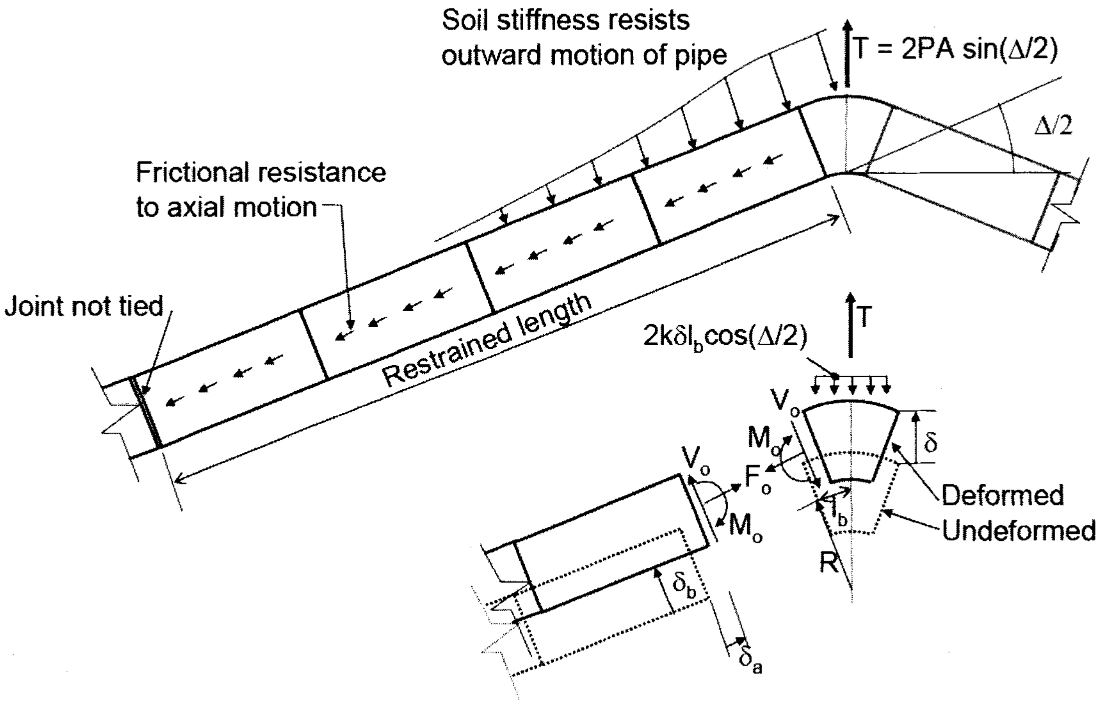



Figure 9-10 Free body diagram of forces and deformations at a bend

$V_o$ = shear in the pipe at the fitting, lb

$l_b$ = centerline distance, in., from the point of intersection of the bend to the end of the mortar or concrete lining at the bell end of the first restrained joint. This dimension will vary among manufacturers due to differences in centerline bend radius, joint ring geometry, and extensions of bend length. If $l_b$ exceeds $2.5D_y \tan(\Delta/2)$ plus the distance from the end of the bend cylinder to the end of the mortar or concrete lining of the bell, use $l_b = 2.5D_y \tan(\Delta/2)$ and consider the bend beyond $l_b$ to be a restrained pipe attached to the bend by a welded joint.

$\delta$ = outward movement of fitting, in.

$k$ = soil stiffness against outward movement of the pipe or fitting, lb/in./in.

Note that Eq 9-6 differs from the equation used in Zarghamee et al. (2004) in that the last term is a conservative representation of the resistance of soil resulting from outward movement of the bend.

The frictional resistance, $f_\mu$, of a buried pipe, in pounds per linear foot, is expressed by

$$f_\mu = \mu\,[(1+\beta)W_e + W_p + W_f] \qquad \text{(Eq 9-7)}$$

Where:

$\mu$ = coefficient of friction between pipe and soil

$W_e$ = earth load, lb/lin ft

$W_p$ = weight of pipe, lb/lin ft

$W_f$ = weight of fluid in pipe, lb/lin ft

$\beta$ = shallow cover factor

When the pipe has low cover, the soil on the top of the pipe may move with the pipe. Soil resistance against movement of the pipe is provided, in part, along the sides of the soil block directly above the pipe, rather than at the pipe-to-soil interface along the top surface of the pipe. Hence, the shallow cover factor, $\beta$, may be expressed by the following (Zarghamee et al. 2004):

$$\beta = \frac{K_o \tan\varphi}{\mu} \frac{\left(\frac{12H}{D_o} + 0.5\right)^2}{\left(\frac{12H}{D_o} + 0.107\right)} \leq 1 \qquad \text{(Eq 9-8)}$$

Where:

$K_o$ = 1 – sin$\varphi$ = coefficient of lateral soil pressure

$\varphi$ = angle of internal friction, in degrees

$H$ = depth of cover, ft

$D_o$ = outside diameter of pipe, in.

NOTE: When $H = 0$, use $W_e = 0$.

Figure 9-10 shows the displacements of the pipe. Compatibility of displacements is ensured by expressing the axial deformation of the pipe, $\delta_a$(in inches), and the transverse deformation of the pipe, $\delta_b$(in inches), in terms of the outward movement of the fittings, $\delta$, as follows:

$$\delta_a = \delta \sin\left(\frac{\Delta}{2}\right) = \frac{12F_o L_{ft}}{2E_c A_t} = \frac{6F_o L_{ft}}{E_c A_t} \qquad \text{(Eq 9-9A)}$$

$$\delta_b = \delta \cos\left(\frac{\Delta}{2}\right) = \frac{V_o \lambda}{k} \qquad \text{(Eq 9-9B)}$$

Where:

$D_y$ = steel cylinder outside diameter, in.

$E_c$ = modulus of elasticity of concrete, psi

$A_t$ = transformed area of the pipe wall cross section = $A_c + nA_y$, in.$^2$

$A_c$ = $\frac{\pi}{4}(D_o^2 - ID^2) - t_y \pi(D_y - t_y)$, in.$^2$

$A_y$ = $\pi(D_y - t_y)(t_y)$, in.$^2$

$\lambda$ = beam on elastic foundation parameter

$L_{ft}$ = length of restrained pipe, ft

$n$ = modular ratio of steel to concrete

$ID$ = inside diameter of pipe, in.

$D_o$ = outside diameter of pipe, in.

$t_y$ = thickness of steel cylinder, in.

The value of $\lambda$ is expressed by

$$\lambda = \left( \frac{k}{4E_c I_{eff}} \right)^{1/4} \qquad \text{(Eq 9-10)}$$

Where

$I_{eff} = \left[ \frac{\pi}{64} \left( D_o^4 - ID^4 \right) - I_s \right] \psi + nI_s$

= effective moment of inertia of the pipe wall cross section, in.$^4$

$I_s$ = moment of inertia of the steel cylinder, in.$^4$

$\psi$ = 0.2 = moment of inertia reduction factor*

The restrained length of pipe, $L_{ft}$ (in ft), measured from the bend or fitting, is calculated as

$$L_{ft} = \frac{F_o}{f_\mu} \qquad \text{(Eq 9-11A)}$$

Eq 9-6, 9-9A, 9-9B, and 9-11A are solved simultaneously to compute $F_o$; $V_o$, $L_{ft}$, and $\delta$. The bending moment $M$ is calculated from shear $V_o$ using the beam on elastic foundation equation $M = \frac{V_o}{2\lambda}\left[ e^{\lambda x} (\cos \lambda x - \sin \lambda x) \right]$. These values are then used to determine the stress resultants along the pipeline with welded joints.

Tests have demonstrated the axial extensibility of a grouted mechanically harnessed joint even when installed fully extended. Therefore, pipelines with mechanically harnessed joints have an effective axial slack, $\Omega$. Subjected to thrust, the pipe segments will move axially and in the direction transverse to the pipeline against the soil. Such transverse movement causes lateral pressure against the pipe, and transverse shear and bending moment in the pipe. As the thrust increases, the pipe will extend and rotate at the joints until the slack is consumed, starting with the first joint and gradually extending to the joints farther away from the elbow. This will result in axial force in addition to bending moment in the pipe segments.

An analysis procedure for both types of restrained joints was verified against a detailed finite element model of the pipe that accounts for the actual geometry of the pipe and the joints, including the slack in mechanically harnessed joints, the nonlinearities in the concrete, the steel stress–strain relationship, and joint flexibility. For details of the finite-element model of the pipe at bends and the behavior of mechanically harnessed joints, see Zarghamee et al. 2004. A conservative estimate of slack of $\Omega$ = 1⁄16 in. is used for mechanically harnessed joints.

For a bulkhead, $F_o = PA$ and $V_o = 0$. The thickness design of the steel cylinder for transmitting the axial force is based on an allowable stress of $0.50f_y$ at the effective working pressure $P_{weff}$ or the field test pressure $P_{ft}$, whichever is greater, since the hydrostatic thrust imparts no bending moment into the pipe. For this discussion,

* The factor $\psi$ accounts for (1) tensile softening of concrete in tension, and (2) axial flexibility of the joint as joint rings and steel cylinder in tension slide relative to the core concrete over a bond development length when subjected to the design internal pressure $P = 1.25P_{weff}$. The value of $\psi$ was determined conservatively from a comparison of the maximum effects of combined axial force and bending moment in bends calculated using the procedure described here with those of the axial force and bending moment determined from a finite-element procedure that accounts for tensile softening of concrete, axial flexibility of the joint, and side friction between the pipe and soil near the bend at the design internal pressure $P = 1.25P_{weff}$ (as described in Zarghamee et al. 2004).

## Table 9-1 Soil type selection guide*

| Soil Types | In-Situ Soil | Backfill Soil, Standard Proctor Percent Compaction | Soil Properties |
|---|---|---|---|
| I | Coarse-grained soils—very dense | Coarse-grained soils containing less than 12% fines—high compaction (≥ 95% standard Proctor) | $k$ = 7,000 lb/in./in. (48.3 MPa)<br>$\mu$ = 0.5<br>$\gamma$ = 140 pcf (2,243 kg/m$^3$)<br>$\phi$ = 48° |
| II | Coarse-grained soils—dense and medium dense | Coarse-grained soils containing less than 12% fines—moderate and high compaction (≥ 90% standard Proctor) | $k$ = 3,400 lb/in./in. (23.4 MPa)<br>$\mu$ = 0.5 |
| | | Coarse-grained soils containing more than 12% fines—high compaction (≥ 95% standard Proctor) | $\gamma$ = 120 pcf (1,922 kg/m$^3$)<br>$\phi$ = 34° |
| | Fine-grained soils (LL < 50) with more than 25% coarse-grained particles—hard, very stiff, and stiff | Fine-grained soils (LL < 50) with more than 25% coarse-grained particles—high compaction (95% standard Proctor) | |
| III | Coarse-grained soils | Coarse-grained soils containing less than 12% fines—moderate compaction (≥ 85% standard Proctor) | $k$ = 1,900 lb/in./in. (13.1 MPa)<br>$\mu$ = 0.5 |
| | | Coarse-grained soils containing more than 12% fines—moderate compaction (85%–95% standard Proctor) | $\gamma$ = 114 pcf (1,826 kg/m$^3$)<br>$\phi$ = 30° |
| | Fine-grained soils (LL < 50)—hard, very stiff, and stiff | Fine-grained soils (LL < 50) with more than 25% coarse-grained particles—moderate compaction (85%–95% standard Proctor) | |
| | | Fine-grained soils (LL < 50) with less than 25% coarse-grained particles—high compaction (≥ 95% standard Proctor) | |
| IV | Coarse-grained soils | Coarse-grained soils—slight compaction (< 85% standard Proctor) | $k$ = 1,100 lb/in./in. (7.6 MPa)<br>$\mu$ = 0.5 |
| | | Fine-grained soils (LL < 50) with more than 25% coarse-grained particles—slight compaction (< 85% standard Proctor) | $\gamma$ = 112 pcf (1,794 kg/m$^3$)<br>$\phi$ = 20° |
| | Fine-grained soils (LL < 50)—medium stiff | Fine-grained soils (LL < 50) with less than 25% coarse-grained particles—moderate compaction (85%–95% standard Proctor) | |
| V† | Fine-grained soils (LL < 50)—soft and very soft | Fine-grained soils (LL < 50) with less than 25% coarse-grained particles—slight compaction (< 85% standard Proctor) | $k$ = 425 lb/in./in. (2.9 MPa)<br>$\mu$ = 0.3 |
| | Fine-grained soils (LL > 50) | Fine-grained soils (LL > 50) | $\gamma$ = 110 pcf (1,762 kg/m$^3$)<br>$\phi$ = 20° |

* Zarghamee et al., 2004.

† The properties for soil type V are selected very conservatively, as some highly plastic soils in a saturated state and under cyclic loading can lose stiffness and become very soft with very low coefficient of friction, while others may remain stiff and/or with a high coefficient of pipe-to-soil friction. For pipelines with soil type V, use of appropriate backfill materials and improved compaction verified by geotechnical, or by direct testing of soil stiffness or friction coefficient may imrpove the soil stiffness and friction coefficients allowing the use of higher soil type classification. Higher friction coefficients, but not more than 0.5, maybe be used if confirmed by testing that accounts for the expected range of soil moisture and sustained loading condition.

$f_y$ = steel cylinder yield strength, psi

$P_{weff}$ = $P_w$, $\frac{P_{ft}}{1.25}$, or $\frac{P_w + P_t}{1.4}$, whichever is greatest

Where:

$P_w$ = working pressure, psi

$P_{ft}$ = field test pressure, psi

$P_t$ = surge pressure, psi

The restrained length, $L_{ft}$ (in ft), is calculated as follows:

$$L_{ft} = 1.1\left[\frac{1.25P_{weff}A}{f_\mu}\right] \quad \text{(Eq 9-11B)}$$

When the design pressure $P_d \geq 1.25P_{weff}$, then $P_d$ replaces the term $1.25P_{weff}$ in Eq 9-11B and elsewhere that the term $1.25P_{weff}$ appears.

The 1.1 factor in Eq 9-11B corresponds to a resistance factor of 0.9.

## Backfill Soil and In-Situ Soil Types

Backfill and in-situ soil types are classified into five different groups, referred to as soil types I through V (see Table 9-1). This classification system is based on the simultaneous consideration of backfill soil classification and compaction levels and in-situ soil types according to ASTM D2487, Standard for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

For in-situ coarse-grained soils, this classification system distinguishes between dense and medium dense soils where the standard penetration test performed according to ASTM D1586, Standard Test Method for Penetration Test and Split Barrel Sampling of Soils, shows a blow count of more than 8 per foot; for loose and very loose soils, the blow count is less than or equal to 8 per foot.

For in-situ fine-grained soils, this classification system distinguishes between hard, very stiff, and stiff clays with an unconfined compressive strength, $q_u$, greater than 1 ton/ft$^2$, medium stiff clays with $q_u$ greater than 0.5 but less than or equal to 1 ton/ft$^2$, and soft and very soft soils with $q_u$ less than or equal to 0.5 ton/ft$^2$.

For backfill soils, the compaction level is expressed in terms of a percent of maximum Proctor density as determined by testing in accordance with ASTM D698, Standard Test Method for Laboratory Compaction Characteristics of Soils Using Standard Effort (12,400 ft-lbf/ft$^3$ [600 kN-m/m$^3$]).

Soil stiffness, $k$, accounts for the combined stiffnesses of the in-situ and the backfill soils. Values of $k$ are based on finite-element soil-structure interaction analyses of pipe and soil systems. In these analyses, the program CANDE (1989) was used with the Selig soil model (1988). The soil model allows for construction sequence and the at-rest strains in the soil, volumetric strains due to the motion into the soil, and shearing strains at the top and bottom of the pipe, thus eliminating the need for separate consideration of lateral soil friction. Analysis was performed for a combination of in-situ soils and various backfill types with different compaction levels. The results were combined into five groups, representing conservative soil properties (see Table 9-1).

The coefficient of friction, μ, depends on the type and compaction of the soil and the roughness of the pipe exterior. In identical soils, the coefficient of friction at the surface of concrete pressure pipe with impact-applied mortar coating (ANSI/AWWA C301 or C303) will be greater than the coefficient of friction of a similar pipe with a smooth exterior surface. The value of $\mu = 0.5$ for compacted granular soils is based on results of tests performed by industry on pipe with mortar coating in granular backfills. In 1965, tests were performed on the time-dependent frictional movement of mortar-coated pipe installed in loose granular and clayey backfills. In 1992 and 1993, industry performed tests on mortar-coated pipe laid over fine sands in dry and wet conditions, and in 1998 on the time-dependent frictional movement of mortar- and concrete-coated pipe on granular material compacted to 85, 90, and 95 percent standard Proctor compaction. The values of $\mu$ in Table 9-1 are based on the results of these industry tests. For clayey soils, the cohesion further increases the effective coefficient of friction, although saturated plastic clays may, in fact, have lower soil-to-pipe friction coefficients. Pipe with a smooth exterior surface, such as ANSI/AWWA C300 and C302 pipes, and applications of exterior barrier coating, require the use of a lower coefficient of friction for the pipe-soil interface movement. When in-situ soil and backfill soil are very different, existing rules for combining stiffnesses or moduli may be used (AWWA 2005).

## Pipe Wall Design

The pipe wall for concrete pressure pipe must be designed using the interaction diagram for the combined effects of axial load and bending moment as a reinforced concrete section. The interaction diagram for a pipe cross section is a plot of the calculated moment capacity against the axial tensile force. The interaction diagram can be plotted for the ultimate moment capacity, $M_{ultimate}$, versus axial force ranging from zero to the ultimate tensile capacity of the steel cylinder, $F_{ultimate}$. The ultimate capacities can be calculated using the stress–strain relationships for the steel and the simplified Whitney block for concrete. It can also be plotted for the onset of yielding of steel cylinder, $M_{yield}$, as axial force ranges from zero to the onset of yield strength of the steel cylinder, $F_{yield}$, using the stress–strain relationships for steel and concrete. In general, the design process involves an initial estimate of the thickness of the steel cylinder for a known total wall thickness and cylinder location, calculating the stresses, and then iterating to an acceptable solution.

For prestressed concrete cylinder pipe, the design process accounts for both serviceability and strength limit states through the use of conservative load factors. For serviceability, tensile strength and softening of concrete are considered in the analysis, but are neglected for ultimate strength calculations, as in ANSI/AWWA C304 for circumferential pipe wall design. For serviceability, the stresses in the pipe wall are computed assuming a bilinear stress–strain relationship for the steel cylinder and a trilinear stress–strain relationship for concrete and mortar (see Figure 2 of Zarghamee et al. 2004). For simplicity, the trilinear relationship for concrete is also used for mortar coating. The location of the neutral axis and the strains and stresses in the pipe, schematically shown in Figure 9-11, are determined from equilibrium of internal and external forces and moments. The design process used for prestressed concrete steel-cylinder pipe can also be applied to the other concrete pressure pipes, such as reinforced concrete steel-cylinder pipe (ANSI/AWWA C300) and bar-wrapped steel-cylinder pipe (ANSI/AWWA C303).

The ultimate strength of the pipe section, when subjected to an axial tensile force and a moment, is conservatively determined assuming a bilinear stress–strain relationship for the steel cylinder and a Whitney block stress–strain diagram for concrete or mortar in compression and neglecting the tensile strength of concrete or mortar (see Figure 9-12). The compressive effect of mortar coating is included in calculations of serviceability limit states for lined-cylinder and embedded-cylinder prestressed pipes (ANSI/AWWA C301) and for bar-wrapped pipe (ANSI/AWWA C303).

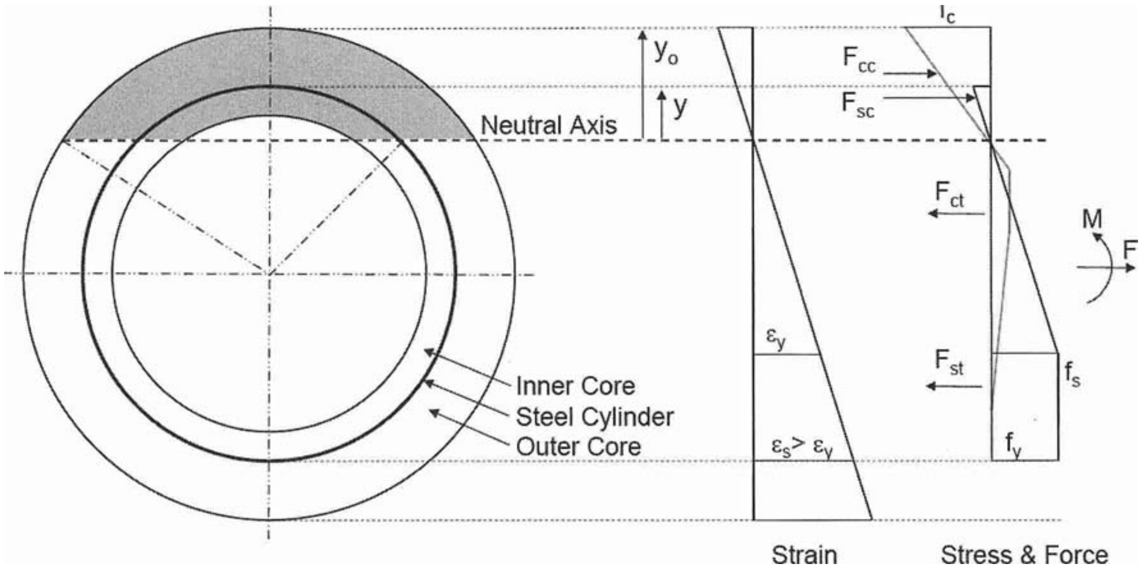



Figure 9-11 Schematic strain and stress distributions at cylinder yield design criterion ($\varepsilon_y$ = steel cylinder yield strain, $f_y$ = steel cylinder yield stress, $f_s$ = steel cylinder stress, $f_c$ = concrete stress)

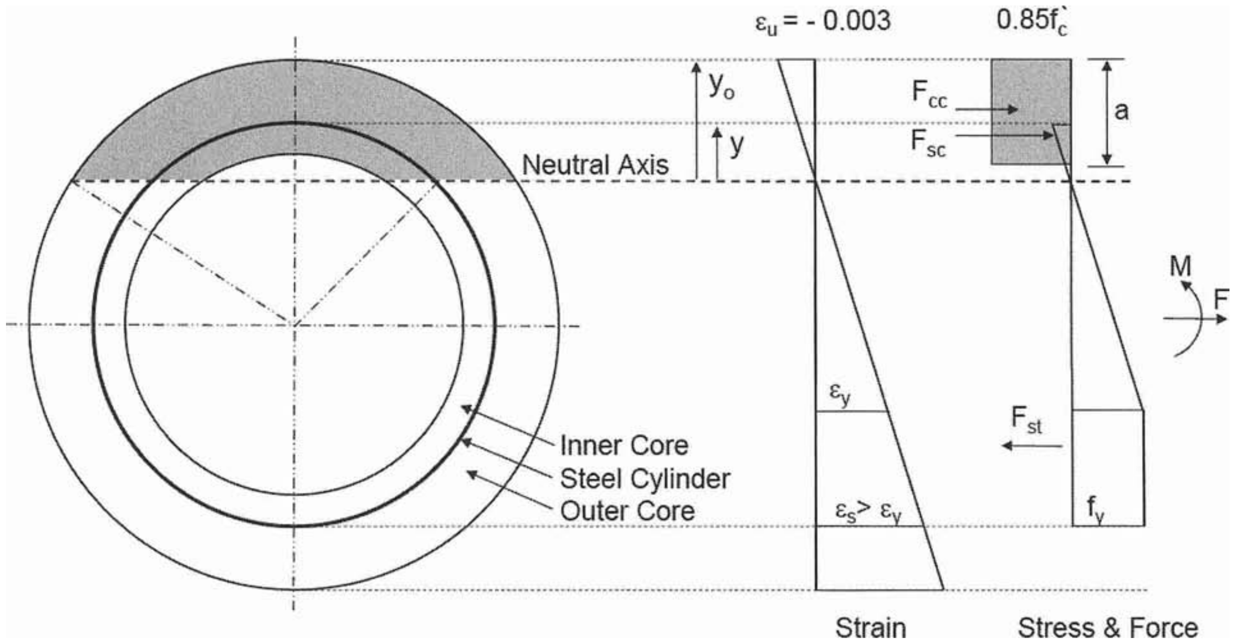



Figure 9-12 Schematic strain and stress distributions at ultimate strength limit state ($\varepsilon_y$ = steel cylinder yield strain, $f_y$ = steel cylinder yield stress, $f_s$ = steel cylinder stress, $f'_c$ = concrete strength)

The following design criteria are used for pipe sections subjected to the combined effects of axial force and bending moment:

**Design Criterion 1:**

At 1.3 times the effective field test pressure, $1.3P_{fteff}$, where $P_{fteff} = 1.25P_{weff}$, the strain in the steel cylinder may not exceed $\varepsilon_y = f_y / E_s$. In calculating the cylinder strain, the tensile strength of the concrete or mortar inside the steel cylinder and at the joint is neglected because the pipe may have circumferential or helical cracks in its inner core as permitted by ANSI/AWWA C300, C301, and C303. As a result, the minimum load factor applied to $P_w$ is 1.63 to reach the onset of cylinder yield.

**Design Criterion 2:**

At 1.56 times the effective field test pressure ($1.56P_{fteff}$), the axial force and bending moment may not exceed the ultimate strength of the pipe under combined loads. (The factor of 1.56 is based on a load factor of 1.4 and a material resistance factor of 0.9, consistent with the requirements of ACI 318.) The minimum load factor applied to $P_w$ to reach the ultimate compressive strength of the concrete is 1.95.

Due to the iterative nature of the design process for restraining buried horizontal bends with welded or mechanically harnessed joints, computer software is virtually mandatory. The number of concrete pressure pipe sizes and the variety of wall thicknesses available make it impractical to provide a comprehensive set of tables in this manual.

In concrete cylinder pipe (i.e., ANSI/AWWA C300, C301, or C303 type), the steel cylinder is used to transmit longitudinal forces, since it is continuous over the full pipe length and is welded to the joint rings at each end. In noncylinder pipe (ANSI/AWWA C302 type) with restrained joints, thrust is transmitted through the full-length bars, which must be welded to the steel joint rings at both ends of the pipe. For noncylinder pipe with restrained joints, the minimum area of longitudinal reinforcement must not be less than the required area of a steel cylinder determined by the calculation methods described in this chapter.

## Effects of Temperature and Poisson's Ratio

In addition to internal pressure, restrained joint pipe is subjected to the effects of temperature and Poisson's ratio. Thermal stresses are produced in the pipe wall when the temperature of the water is significantly different from the temperature of the pipe at the time of installation. The Poisson's ratio effect results in shortening of the pipe length due to radial expansion of the pipe wall under pressure. When the temperature of the pipe becomes less than its temperature at the time of its installation, both of these effects tend to shorten the pipe adjacent to the bend and result in an increase in axial force and a reduction of bending moment at the bend. When the temperature of the pipe becomes greater than its temperature at the time of installation, the Poisson's ratio effect counteracts a part of the temperature effect. The net effect is a reduction of axial force and an increase in bending moment.

Analyses performed using the detailed finite-element model of the pipe near a bend, accounting for tensile softening of concrete and the nonlinear behavior of the joints, show that the temperature change and Poisson's effect do not increase the maximum stress resultants that occur at a bend compared to the maximum stress due to pressure-induced thrust alone. They do affect the stresses away from the bend; however, the assumed linear reduction in required cylinder thickness by Eq 9-12 is sufficiently conservative to account for the stresses in the pipe wall due to temperature and Poisson's ratio effects (Zarghamee et al. 2004).

## Vertical Bends

The design of vertical bends with the pressure-induced thrust pushing down into the soil may be performed using the same design procedure as for horizontal bends. However, when the pressure-induced thrust is pushing up and out of the soil, the thrust must be resisted by the dead weight of the pipe, fluid, and soil.

$$\text{Dead weight resistance per foot of pipe} = (W_p + W_f + W_e)\cos\left(\Delta - \frac{\alpha}{2}\right)$$

Where:

$W_e$ = earth load, lb/lin ft

$W_f$ = fluid weight, lb/lin ft

$W_p$ = pipe weight, lb/lin ft

$\alpha$ = slope angle, in degrees

$\Delta$ = bend angle, in degrees

The length of pipe, $L_{ft}$, to be tied to each leg of a vertical uplift bend is calculated as follows:

$$L_{ft} = \frac{PA\sin\frac{\Delta}{2}}{(W_p + W_f + W_e)\cos\left(\Delta - \frac{\alpha}{2}\right)} \qquad \text{(Eq 9-14)}$$

**Restrained Joint Strength.** Restrained joints of either type (welded or mechanical) must have component strengths in the load path that exceed the yield strength of the steel cylinder:

$$F_j \geq F_y = \pi(D_y - t_{yreq})\,t_{yreq}f_y \qquad \text{(Eq 9-15)}$$

Where:

$F_j$ = force the joint can support without material failure or pipe leakage, lb

$F_y$ = force that results in yielding of the steel cylinder, lb

$t_{yreq}$ = pipe cylinder thickness required for thrust restraint where the restrained joint will be used, in.

$D_y$ = steel cylinder outside diameter, in.

$f_y$ = steel cylinder yield strength, psi

Evidence that a restrained joint meets this requirement can be provided by calculations or by test results.

# DESIGN EXAMPLES

Example design calculations are presented for three different pipe types, namely, two bend angles and a bulkhead. The design conditions, pipe properties, and installation parameters used for each of these examples are presented in Table 9-2. The input and results from five additional examples are provided in Table 9-3 without accompanying calculations. These results were obtained from specially developed computer software.

The pipe material properties and characteristics presented in Table 9-2 and Table 9-3 have been selected for illustrative purposes and should not be used as actual design values without verification of their appropriateness.

## Design Example 1: 36 in. Diameter LCP (ANSI/AWWA C301) —Bulkhead

1. Determine effective working pressure:

$$P_{weff} = \max\left(\frac{P_{ft}}{1.25}, \frac{P_w + P_t}{1.4}\right) = \max\left(\frac{200}{1.25}, \frac{150 + 80}{1.4}\right) = \max(160, 164) = 164\,\text{psi} < 200\,\text{psi} = P_{ft}$$

Therefore, use $P_{ft}$ to calculate cylinder thickness required.

2. Determine thrust in pipe:

Pressurized area of pipe:

$$A = \pi \frac{D_i^{\,2}}{4} = \pi \frac{(41\ \text{in.})^2}{4} = 1,320\ \text{in.}^2$$

Thrust in pipe at $P_{ft}$:

$$F = P_{ft} A = 200\ \text{psi} \times 1,320\ \text{in.}^2 = 264\ \text{kip}$$

3. Determine the pipe cylinder area requirement at the bulkhead:

Required pipe cylinder area:

$$A_{yo} = \frac{F}{0.5 f_y} = \frac{264\ \text{kip}}{0.5\,(36\ \text{ksi})} = 14.67\ \text{in.}^2$$

4. Determine frictional resistance of pipe:

Weight of fluid:

$$W_f = \gamma_f \frac{\pi}{4} ID^2 = (62.4\ \text{pcf})\left(\frac{\pi}{4}\right)(36\ \text{in.})^2 \div \left(12 \frac{\text{in.}}{\text{ft}}\right)^2 = 441 \frac{\text{lb}}{\text{ft}}$$

Weight of soil above pipe (Eq 9-5B):

$$W_e = \gamma_s \left[\frac{D_o H}{12} + \left(1 - \frac{\pi}{4}\right)\left(\frac{D_o}{12 \times 2}\right)^2\right]$$

$$= 114\ \text{pcf}\left[\frac{42.5\ \text{in.}}{12\ \text{in./ft}}(4\ \text{ft}) + \left(1 - \frac{\pi}{4}\right)\frac{(42.5\ \text{in.})^2}{(2)(12\ \text{in./ft})^2}\right] = 1,768 \frac{\text{lb}}{\text{ft}}$$

Shallow cover factor to account for reduced friction if the soil-to-soil friction is less than that of pipe-to-soil (Eq 9-8):

$$\beta = \frac{K_o \tan\phi}{\mu} \frac{\left(\frac{12H}{D_o} + 0.5\right)^2}{\left(\frac{12H}{D_o} + 0.107\right)}$$

Table 9–2 Summary of design examples and results of detailed calculation

| Conditions and Parameters | Example 1 | Example 2 | Example 3 |
|---|---|---|---|
| **Bend Conditions:** | | | |
| Fitting | Bulkhead | 45° bend | 75° bend |
| Joint type | Any | Welded | Mech. restr. |
| Bend length, $l_b$, in. | — | 24.6 | 38.64 |
| **Pipe Properties:** | | | |
| Pipe type | LCP (C301) | BWP (C303) | ECP (C301) |
| Nominal pipe inside diameter, *ID*, in. | 36 | 54 | 42 |
| Pipe weight, $W_p$*, lb/ft | 404 | 518 | 662 |
| Core outside diameter, *OD*, in. | 40.5 | 55.875 | 49.0 |
| Pipe outside diameter, $D_o$, in. | 42.50 | 58.38 | 51.00 |
| Core thickness, $h_c$, in. | 2.25 | 0.9375 | 3.5 |
| Mortar coating thickness, $t_m$, in. | 1.0 | 1.25 | 1.0 |
| Effective core thickness, $h_c$, in. | 2.25 | 2.188 | 3.5 |
| Joint diameter, $D_j$, in. | 41.0 | 56.375 | 45.0 |
| Cylinder outside diameter, $D_y$, in. | 40.5 | 55.875 | 44.5 |
| Minimum cylinder thickness | 16 ga | $^3/_{16}$ in. | 16 ga |
| Pipe laying length, $L_p$, ft | 20 ft | 40 ft | 20 ft |
| **Pressure Conditions:** | | | |
| Working pressure, $P_w$, psi | 150 | 200 | 140 |
| Transient pressure, $P_t$, psi | 80 | 100 | 60 |
| Field test pressure, $P_{ft}$, psi | 200 | 250 | 180 |
| Effective working pressure, $P_{weff}$, psi | 164 | 214 | 144 |
| **Soil Conditions:** | | | |
| Cover depth, *H*, ft | 4 | 6 | 5 |
| Soil type | III | II | III |
| Soil weight, $\gamma_s$, pcf | 114 | 120 | 114 |
| Soil stiffness, *k*, psi | 1,900 | 3,400 | 1,900 |
| Pipe-to-soil friction coefficient, μ | 0.5 | 0.5 | 0.5 |
| Angle of internal friction, φ, in degrees | 30 | 34 | 30 |
| **Material Properties:** | | | |
| Concrete strength, $f'_c$, psi | 6,000 | 4,500 | 4,500 |
| Modulus of elasticity for concrete, $E_c$, psi | 3,942,500 | 3,616,500 | 3,616,500 |
| Concrete tensile strength, $f_t$, psi | 387 | 335 | 335 |
| Concrete weight, $\gamma_c$, pcf | 145 | 145 | 145 |
| Steel cylinder yield strength, $f_y$, psi | 36,000 | 36,000 | 36,000 |
| Modulus of elasticity for steel, $E_y$, psi | 30,000,000 | 30,000,000 | 30,000,000 |
| Modular ratio, *n* | 7.609 | 8.295 | 8.295 |
| Steel yield strain, $\varepsilon_y$ | 0.0012 | 0.0012 | 0.0012 |
| Concrete strength at steel yield strain, $f_{cy}$, psi | 123 | 82 | 82 |
| **Results:** | | | |
| Selected cylinder thickness, $t_y$ | 10 ga | 0.1875 in. | 12 ga |
| Ultimate moment strength, $M_{ultimate}$, in.-kip | — | 27,921 | 11,673 |
| Ultimate tensile strength, $F_{ultimate}$, kip | — | 1,181 | 525 |
| Yield moment strength, $M_{yield}$, in.-kip | — | 19,611 | 9,221 |
| Yield tensile strength, $F_{yield}$, kip | — | 1,199 | 565 |
| Pipe moment at first joint, *M*, in.-kip | — | 7,392 | 3,528 |
| Pipe axial force at first joint, *F*, kip | — | 373 | 179 |
| Thrust dissipation length, $L_{ft}$, ft | 136 | 82 | 55 |
| Margin of safety, ultimate | — | 1.10 | 1.00 |
| Margin of safety, yield | — | 1.12 | 1.10 |

* Pipe weight for ANSI/AWWA C300, C301, and C302 pipe is calculated from page 69, step 3. Pipe weight for ANSI/AWWA C303 pipe is obtained from the manufacturer.

## Table 9–3 Summary of additional design examples

| Conditions and Parameters | Example 4 | Example 5 | Example 6 | Example 7 | Example 8 |
|---|---|---|---|---|---|
| **Bend Conditions:** | | | | | |
| Fitting | 22.5° Bend | 90° Bend | 30° Bend | 22.5° Bend | 90° Bend |
| Joint type | Mech. Restr. | Mech. Restr. | Welded | Welded | Mech. Restr. |
| Bend length, $l_b$, in. | 15.6 | 34.8 | 27.8 | 19.2 | 98.4 |
| **Pipe Properties:** | | | | | |
| Pipe type | ECP (C301) | BWP (C303) | ECP (C301) | (C300) | ECP (C301) |
| Nominal pipe inside diameter, *ID*, in. | 72 | 30 | 96 | 84 | 96 |
| Pipe weight, $W_p$*, lb/ft | 1,614 | 245 | 2,456 | 2,328 | 2,456 |
| Core outside diameter, *OD*, in. | 83 | 31.88 | 109 | 100 | 109 |
| Pipe outside diameter, $D_o$, in. | 85.00 | 34.13 | 111.00 | 100.00 | 111.00 |
| Core thickness, $h_c$, in. | 5.5 | 0.9375 | 6.5 | 8.0 | 6.5 |
| Mortar coating thickness, $t_m$, in. | 1 | 1.125 | 1 | NA | 1 |
| Joint diameter, $D_j$, in. | 76.375 | 32.375 | 101.125 | 88.75 | 101.125 |
| Cylinder outside diameter, $D_y$, in. | 75.5 | 31.875 | 100.25 | 87.875 | 100.25 |
| Minimum cylinder thickness | 16 ga | 10 ga | 16 ga | 12 ga | 16 ga |
| Pipe laying length, $L_p$, ft | 20 | 40 | 20 | 20 | 20 |
| **Pressure Conditions** | | | | | |
| Working pressure, $P_w$, psi | 125 | 250 | 175 | 72 | 150 |
| Transient pressure, $P_t$, psi | 50 | 0 | 85 | 0 | 75 |
| Field test pressure, $P_{ft}$, psi | 150 | 250 | 225 | 72 | 180 |
| Effective working pressure, $P_{weff}$, psi | 125 | 250 | 185.7 | 72 | 160.7 |
| **Soil Conditions:** | | | | | |
| Cover depth, *H*, ft | 4 | 3 | 4 | 5 | 4 |
| Soil type | IV | II | V | III | III |
| Soil weight, $\gamma_s$, pcf | 112 | 120 | 110 | 114 | 114 |
| Soil stiffness, *k*, psi | 1,100 | 3,400 | 425 | 1,900 | 1,900 |
| Pipe-to-soil friction coefficient, $\mu$ | 0.4 | 0.5 | 0.3 | 0.5 | 0.5 |
| Angle of internal friction, $\varphi$, degrees | 20 | 34 | 20 | 30 | 30 |
| **Material Properties:** | | | | | |
| Concrete strength, $f'_c$, psi | 4,500 | 4,500 | 4,500 | 4,500 | 4,500 |
| Modulus of elasticity for concrete, $E_c$, psi | 3,616,500 | 3,616,500 | 3,616,500 | 3,616,500 | 3,616,500 |
| Concrete tensile strength, $f_t$, psi | 335 | 335 | 335 | 335 | 335 |
| Concrete weight, $\gamma_c$, pcf | 145 | 145 | 145 | 145 | 145 |
| Steel cylinder yield strength, $f_y$, psi | 36,000 | 36,000 | 36,000 | 36,000 | 36,000 |
| Modulus of elasticity for steel, $E_y$, psi | 30,000,000 | 30,000,000 | 30,000,000 | 30,000,000 | 30,000,000 |
| Modular ratio, *n* | 8.295 | 8.295 | 8.295 | 8.295 | 8.295 |
| Steel yield strain, $\varepsilon_y$ | 0.0012 | 0.0012 | 0.0012 | 0.0012 | 0.0012 |
| Concrete strength at steel yield strain, $f_{cy}$, psi | 82.6 | 82.6 | 82.6 | 82.6 | 82.6 |
| **Results:** | | | | | |
| Selected cylinder thickness, $t_y$ | 12 ga | 10 ga | 0.3125 in. | 12 ga | 0.3125 in. |
| Ultimate moment strength, $M_{ultimate}$, in.-kip | 34,210 | 6,750 | 166,348 | 47,220 | 166,348 |
| Ultimate tensile strength, $F_{ultimate}$, kip | 892 | 483 | 3,532 | 1,038 | 3,532 |
| Yield moment strength, $M_{yield}$, in.-kip | 29,301 | 4,736 | 121,544 | 43,072 | 121,544 |
| Yield tensile strength, $F_{yield}$, kip | 990 | 492 | 3,679 | 1,186 | 3,679 |
| Pipe moment at first joint, *M*, in.-kip | 11,473 | 2,128 | 34,200 | 4,147 | 50,953 |
| Pipe axial force at first joint, *F*, kip | 133 | 150 | 1,293 | 381 | 1,146 |
| Thrust dissipation length, $L_{ft}$, ft | 30 | 107 | 274 | 49 | 145 |
| Margin of safety, ultimate | 1.32 | 1.02 | 1.12 | 1.41 | 1.02 |
| Margin of safety, yield | 1.46 | 1.02 | 1.22 | 1.84 | 1.05 |

* Pipe weight for ANSI/AWWA C300, C301, and C302 pipe is calculated from page 69, step 3. Pipe weight for ANSI/AWWA C303 pipe is obtained from the manufacturer.

$$= \frac{(1 - \sin 30°) \tan 30°}{0.5} \frac{\left[\left(\frac{4\text{ ft}}{42.5\text{ in.}}\right)\left(12\frac{\text{in.}}{\text{ft}}\right) + 0.5\right]^2}{\left[\left(\frac{4\text{ ft}}{42.5\text{ in.}}\right)\left(12\frac{\text{in.}}{\text{ft}}\right) + 0.107\right]} = 1.24 > 1 \therefore \text{ use } 1$$

Soil frictional force (Eq 9-7):

$$f_\mu = \mu\left[(1+\beta)W_e + W_p + W_f\right] = 0.5\left[(1+1)\left(1768\frac{\text{lb}}{\text{ft}}\right) + 404\frac{\text{lb}}{\text{ft}} + 441\frac{\text{lb}}{\text{ft}}\right] = 2,196\frac{\text{lb}}{\text{ft}}$$

5. Thrust dissipation length required (from Eq. 9-11B):

$$L_{ft} = 1.1\left[\frac{1.25P_{weff}A}{f_\mu}\right] = 1.1\left[\frac{1.25(164\text{ psi})(1320\text{ in.}^2)}{2196\frac{\text{lb}}{\text{ft}}}\right] = 136\text{ ft}$$

6. Determine required cylinder thicknesses over the thrust dissipation length:

Pipe cylinder areas:

$$A_{s16ga} = (D_y - t_y)(\pi)(t_y) = (40.5\text{ in.} - 0.0598\text{ in.})(\pi)(0.0598\text{ in.}) = 7.60\text{ in.}^2$$
$$A_{s14ga} = (D_y - t_y)(\pi)(t_y) = (40.5\text{ in.} - 0.0747\text{ in.})(\pi)(0.0747\text{ in.}) = 9.49\text{ in.}^2$$
$$A_{s12ga} = (D_y - t_y)(\pi)(t_y) = (40.5\text{ in.} - 0.1046\text{ in.})(\pi)(0.1046\text{ in.}) = 13.27\text{ in.}^2$$
$$A_{s10ga} = (D_y - t_y)(\pi)(t_y) = (40.5\text{ in.} - 0.1345\text{ in.})(\pi)(0.1345\text{ in.}) = 17.06\text{ in.}^2$$

Because the required area is 14.67 in.$^2$ [$t_{yreq}$ = (14.67/17.06) × 0.1345 = 0.1157 in.], a 10 ga cylinder thickness is selected for the pipe at the bulkhead.

The pipe axial force, and therefore the required cylinder thickness, diminishes on a straight-line basis to zero at the thrust dissipation length from the bulkhead.

$$L_{10ga} = L_{ft} - \frac{A_{s12ga}}{A_{yo}}L_{ft} = 136\text{ ft} - \frac{13.27\text{ in.}^2}{14.67\text{ in.}^2}(136\text{ ft}) = 13\text{ ft}$$

$$L_{12ga} = L_{ft} - L_{10ga} - \frac{A_{s14ga}}{A_{yo}}L_{ft} = 136\text{ ft} - 13\text{ ft} - \frac{9.49\text{ in.}^2}{14.67\text{ in.}^2}(136\text{ ft}) = 35\text{ ft}$$

$$L_{14ga} = L_{ft} - L_{10ga} - L_{12ga} - \frac{A_{s16ga}}{A_{yo}}L_{ft} = 136\text{ ft} - 13\text{ ft} - 35\text{ ft} - \frac{7.60\text{ in.}^2}{14.67\text{ in.}^2}(136\text{ ft}) = 18\text{ ft}$$

$$L_{16ga} = L_{ft} - L_{10ga} - L_{12ga} - L_{14ga} = 136\text{ ft} - 13\text{ ft} - 35\text{ ft} - 18\text{ ft} = 70\text{ ft}$$

See Figure 9-23A for the minimum restrained footage and minimum cylinder thickness requirements for this design example. Typical restrained footages and cylinder thicknesses based on the use of 20 ft lengths are also shown.

The minimum restrained joint strength ($F_j$) required without material failure or joint leakage is calculated from Eq 9-15:

$$F_j = \pi(D_y - t_{yreq})t_{yreq}f_y = A_{yo}f_y$$

Where:

$F_j$ = (14.67 in.$^2$)(36,000 psi)

= 528,100 lb

Chapter 10

# Design of Pipe on Piers

Unstable soil conditions or conditions requiring aerial installations may be encountered along a pipeline route. In such cases, piers may be used to support the pipe (Figure 10-1), but the pipe must then be designed to span the supports and to resist the load applied to the pipe at the support.

The standard rubber-gasketed joint is suitable for pier-supported installations in either aerial crossings or in unstable soils. Piers are not usually required for pipe in unstable soil (refer to chapter 6). For all pier-supported installations, the joints may be placed over the support, in which case the longitudinal bending moments may be computed on the basis of a simply supported beam; or, preferably, they may be offset from the support, with moments being computed on the basis of a continuous, pin-connected span. Regardless of the location selected for pier placement, there should be only one pier per length of pipe.

The method of supporting pipe on a pier may vary from wood chocks to shaped saddles. The strength required of the pipe will be related to the type of support provided.

## LOADS

Pipe supported on piers must be designed to resist the loads produced by the pipe, the weight of water, and, if buried, the weight of backfill over the pipe. The weight of backfill, or earth load, on the pipe will depend on the soil conditions and installation procedures, but in all cases, the pipe is assumed to be completely unsupported between piers. If live loads are applicable to supported pipe, they should be placed on the span so as to produce the maximum beam loading.

## CYLINDER PIPE

The beam strength of cylinder pipe can be computed by considering the pipe to be a reinforced concrete beam, circular in section, in which the transformed steel cylinder in tension and the transformed steel cylinder plus concrete and mortar elements in compression resist bending moments from pipe and water weights and external loads. To ensure a conservative design, longitudinal steel reinforcement other than the steel cylinder is ignored. Bending stresses are computed from

$$f_s = nMc_1/I_T \qquad \text{(Eq 10-1)}$$

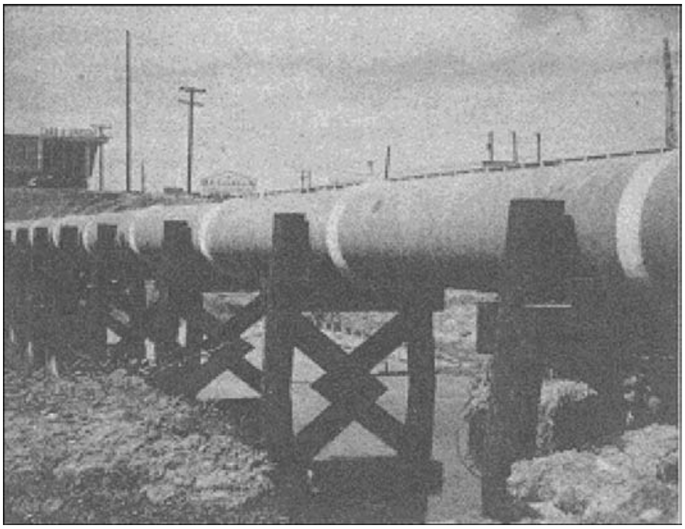


Figure 10-1 Pipe on piers

and

$$f_c = Mc_2/I_T \qquad \text{(Eq 10-2)}$$

Where:

- $f_s$ = tensile stress in the outer fiber of the longitudinal steel, psi
- $n$ = ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete, generally taken as 7.5
- $M$ = bending moment, in.-lb
- $c_1$ = dimension from the neutral axis to the outer tensile fiber of the longitudinal steel, in.
- $I_T$ = transformed moment of inertia, in.$^4$
- $f_c$ = compressive stress in the outer fiber of the concrete or mortar, psi
- $c_2$ = dimension from the neutral axis to the outer compressive fiber of the concrete or mortar, in.

For a transverse cross section of the pipe (Figure 10-2), $\alpha$ is defined as the angle from the top vertical centerline to the point of intersection of the outside of the steel cylinder and the neutral axis.

The location of the neutral axis can be found iteratively using the formula

$$\tan \alpha = (\alpha\, \pi/180) + [nt_y \pi/(t - t_y)] \qquad \text{(Eq 10-3)}$$

Where:

- $\alpha$ = angle from the top vertical centerline to the point of intersection between the outside of the steel cylinder and the neutral axis, in degrees

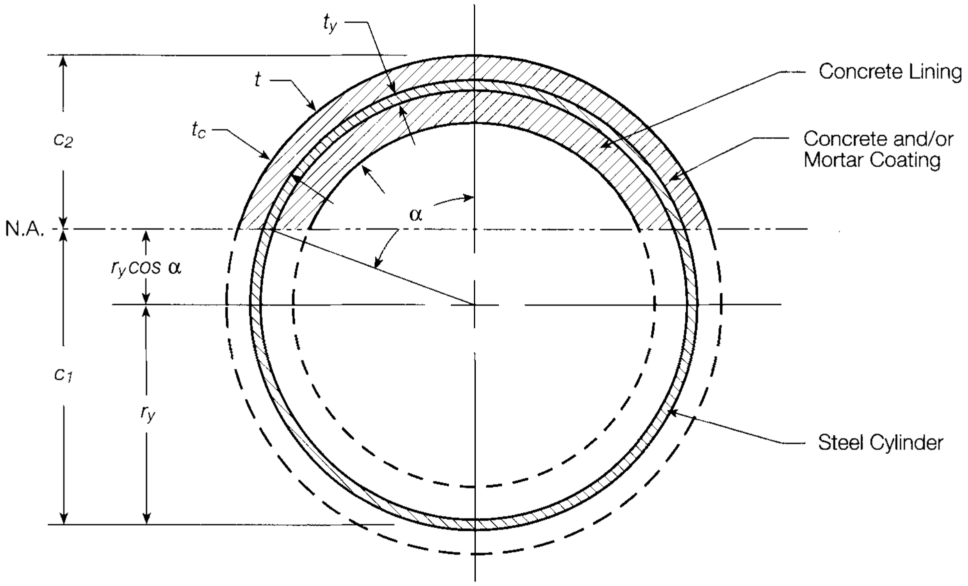



Figure 10-2 Pipe elements assumed effective in resisting bending

$n$ = ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete, generally taken as 7.5

$t_y$ = thickness of the steel cylinder, in.

$t$ = total pipe wall thickness, in.

$I_T$, $c_1$, and $c_2$ can then be calculated by the formulas

$$I_T = 2r_y^3\{(t - t_y)\,[(\alpha\pi/360) + (\alpha\pi/180)\cos^2\alpha - 0.75 \sin 2\alpha] + nt_y\pi(0.5 + \cos^2\alpha)\} \qquad \text{(Eq 10-4)}$$

$$c_1 = r_y\,(1 + \cos\alpha) \qquad \text{(Eq 10-5)}$$

$$c_2 = r_y\,(1 - \cos\alpha) + t_c \qquad \text{(Eq 10-6)}$$

Where:

$r_y$ = outside radius of the steel cylinder, in.

$t_c$ = thickness of the mortar and/or concrete that is outside the steel cylinder, in.

All other variables were previously defined.

A design stress of $0.45f'_c$ for concrete in compression is recommended, where $f'_c$ is the 28-day concrete strength. Under certain optimum conditions, concrete stress as high as $0.6f'_c$ has been used. Total longitudinal tensile stress in the steel from beam loading, thrust, or other loads should not exceed 20,000 psi for cylinder pipe.

## Noncylinder Pipe

The beam strength of noncylinder pipe should be determined by analyzing the pipe with both a cracked and uncracked section.

Noncylinder pipe should be designed so that the beam strength of pipe with cracked and uncracked sections will be sufficient to withstand bending moments from pipe and water weights and externally applied loads.

**Cracked section.** A sufficient number and area of longitudinal reinforcing bars must be provided to resist tension in a cracked section. Longitudinal reinforcing bars should be equally spaced in both inner and outer reinforcing steel cages.

For a cracked section, the location of the neutral axis may be found using Eq 10-3, but with $t_y$ determined from the following equations:

$$t_y = \frac{N_i a_i + N_o a_o}{2\pi r_y} \qquad \text{(Eq 10-7)}$$

and

$$r_y = \frac{N_i a_i r_i + N_o a_o r_o}{N_i a_i + N_o a_o} \qquad \text{(Eq 10-8)}$$

Where:

$a_i$ = cross-sectional area of a longitudinal bar in the inner cage, in.$^2$

$a_o$ = cross-sectional area of a longitudinal bar in the outer cage, in.$^2$

$N_i$ = number of longitudinal bars in the inner cage

$N_o$ = number of longitudinal bars in the outer cage

$r_i$ = radius of the centerline of longitudinal bars in the inner cage, in.

$r_o$ = radius of the centerline of longitudinal bars in the outer cage, in.

$r_y$ = radius of a steel cylinder of thickness $t_y$, with a cross-sectional area equal to the sum of the cross-sectional areas of the longitudinal bars in the inner and outer cages, in.

Bending stresses are computed from

$$f_s = nMc_1/I_T \qquad \text{(Eq 10-9)}$$

and

$$f_c = Mc_2/I_T \qquad \text{(Eq 10-10)}$$

Where:

$f_s$ = tensile stress in the outer fiber of the longitudinal steel, psi

$n$ = ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete, generally taken as 7.5

$M$ = bending moment, in.-lb

$c_1$ = dimension from the neutral axis to the outer tensile fiber of the longitudinal steel, in.

$I_T$ = transformed moment of inertia, in.$^4$

$f_c$ = compressive stress in the outer fiber of the concrete or mortar, psi

$c_2$ = dimension from the neutral axis to the outer compressive fiber of concrete or mortar, in.

$I_T$, $c_1$, and $c_2$ may be calculated using Eq 10-4, 10-5, and 10-6 with values of $t_y$ and $r_y$ obtained from Eq 10-7 and 10-8, respectively.

The design compressive stress in the concrete should not exceed $0.45f'_c$ and the design tensile stress in the longitudinal steel bars should not exceed 10,000 psi.

**Uncracked section.** The moment of inertia of the uncracked section is calculated with no allowance for longitudinal steel reinforcement using Eq 10-11.

$$I_u = \pi/4[(D/2 + t)^4 - (D/2)^4] \qquad \text{(Eq 10-11)}$$

Where:

$I_u$ = moment of inertia of the uncracked section of concrete, in.$^4$

$D$ = inside diameter of pipe, in.

$t$ = pipe wall thickness, in.

Tensile stress in the concrete is computed from

$$f_t = Mc_3/I_u \qquad \text{(Eq 10-12)}$$

Where:

$f_t$ = flexural tensile stress in the concrete, psi

$M$ = bending moment, in.-lb

$c_3$ = distance from the centerline of the pipe to the outside pipe surface, in.

$I_u$ = moment of inertia of the uncracked section of concrete, in.$^4$

Concrete flexural tensile stress should not exceed $4.5\sqrt{f'_c}$ psi.

The method of calculation of beam strength of cylinder and noncylinder pipe presented in this section is simple to implement and produces conservative results. A more detailed analysis using more exact methods may be beneficial under special circumstances.

## LOCATION OF PIERS

Piers for aerial crossings are typically located with a span length $L$ equal to the pipe laying length. The laying length is usually the manufacturer's standard length; however, longer lengths may be provided by welding two or more joints together.

Moments in the pipe can be minimized by offsetting the flexible pipe joints from the supports a distance $O_L$ as shown in Figure 10-3. With flexible joints offset from supports a distance $O_L = 0.146L$, the maximum negative and maximum positive moments are numerically equal and are each half the maximum moment (midspan) for a simply supported span, where the joints are placed directly on the supports. The uniformly loaded simple span maximum moment is:

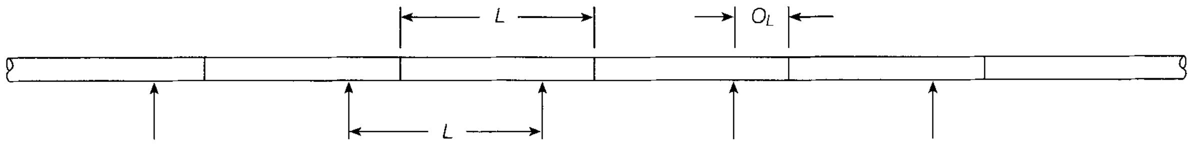



Figure 10-3 Configuration of pile-supported installations with joints offset from the supports

$$M_{MAX} = \frac{wL^2}{8} = 0.125wL^2 \quad \text{(Eq 10-13)}$$

and the maximum moments when the joint is offset by 0.146$L$ from the support are:

$$M_{MAX} = \pm wL^2/16 = \pm 0.0625wL^2 \quad \text{(Eq 10-14)}$$

Where:

$M_{MAX}$ = maximum bending moment, ft-lb

$w$ = weight of pipe, water, and external load, lb/lin ft

$L$ = span length, ft

For some typical lengths of pipe, the computed distance $O_L$ is as follows:

| $L$, *ft* | $O_L$, *ft* |
|---|---|
| 8 | 1.17 |
| 16 | 2.34 |
| 20 | 2.93 |
| 24 | 3.51 |
| 32 | 4.69 |
| 36 | 5.27 |
| 40 | 5.86 |
| 48 | 7.03 |

Small deviations in the dimension $O_L$ in Figure 10-3 caused by misplacement of the pipe on the pier will increase the maximum bending moment in the pipe. A misplacement of 0.5 ft (0.15 m) is conceivable under some construction field conditions. The table that follows presents a comparison of bending moments for a 0.5-ft (0.15-m) misplacement versus the correct placement. The largest increase in the bending moment occurs when the long overhang is increased 0.5 ft (0.15 m), which decreases the short overhang by the same amount. This increases the positive bending moment in the long overhang to the value shown in the second column of the table for various spans. If good construction practices and inspection are provided with pipe properly placed at $O_L = 0.146L$, the maximum positive and negative bending moments are equal and are shown in the third column of the table.

| Bending Moment *M, ft-lb* | | |
|---|---|---|
| *L*, *ft* | Misplaced 0.5 ft, Pier at $O_L = 0.146L - 0.5$ ft | Correct Placement, Pier at $O_L = 0.146L$ |
| 8 | 5.5*W* | 4.0*W* |
| 16 | 19.0*W* | 16.0*W* |
| 20 | 28.7*W* | 25.0*W* |
| 24 | 40.4*W* | 36.0*W* |
| 32 | 69.8*W* | 64.0*W* |
| 36 | 87.5*W* | 81.0*W* |
| 40 | 107.2*W* | 100.0*W* |
| 48 | 152.6*W* | 144.0*W* |
| 60 | 235.7*W* | 225.0*W* |

NOTE: *W* = weight of pipe, water, and external load in pounds per linear foot.

The support system shown in Figure 10-3 assumes balanced loads. To ensure stability, end spans must be anchored by being buried in earth or by structural anchorage. In overhead crossings with multiple spans, pipe should be secured with straps to the pier to prevent joints from opening.

## DESIGN OF PIPE AND SUPPORTS AT PIERS

When pipelines are placed on piers, stresses are induced in the wall of the pipe that may require increasing the pipe design strength. The magnitude of these stresses for a given installation depends on the type and/or dimensions of the supports and the type of pipe, i.e., whether the pipe is rigid or semirigid. Standard concrete pressure pipe joints are easily capable of transferring shear forces in aerial crossings, but where heavy earth loads are carried by the pipe or pier, the joint design should be analyzed to be sure it is adequate for the expected shear stresses.

### Rigid Pipe

Supports for rigid pipe may be either cradles or two-point supports. For pipe without earth cover, supports are usually specified to be 1 to 2 ft (0.3 to 0.6 m) in length. Buried installations may require the use of cradle-type supports with longer supporting lengths. Some typical support configurations are shown in Figure 10-4.

The pier reaction $wL$ (Figure 10-5A) is imposed on the pipe over the support length $b$, but the interaction of the pipe and the support produces radial strains in the pipe over a length defined as the effective length $EL$, which may be taken conservatively to be equal to the length of the support plus the inside diameter (ID) of the pipe.

To determine the stresses in the pipe that accompany its radial strains at the piers, the weight of the effective length of the pipe must be increased by the weight of the pipe in the remaining span and any weight that the span itself supports. The sums of these weights are transferred to the effective length of pipe as shear forces $V$ at the ends of the effective length (as shown in Figure 10-5B).

For computing pipe stresses in the support area, the effect of the shear forces $V$ is assumed to be equivalent to an increased weight $\Delta W_p$ of the effective length of pipe. For equally loaded spans of equal length, this apparent increase in pipe weight in the effective length is

$$\Delta W_p = 2V/EL = (L - b - d)w/(b + d) \quad \text{(Eq 10-15)}$$

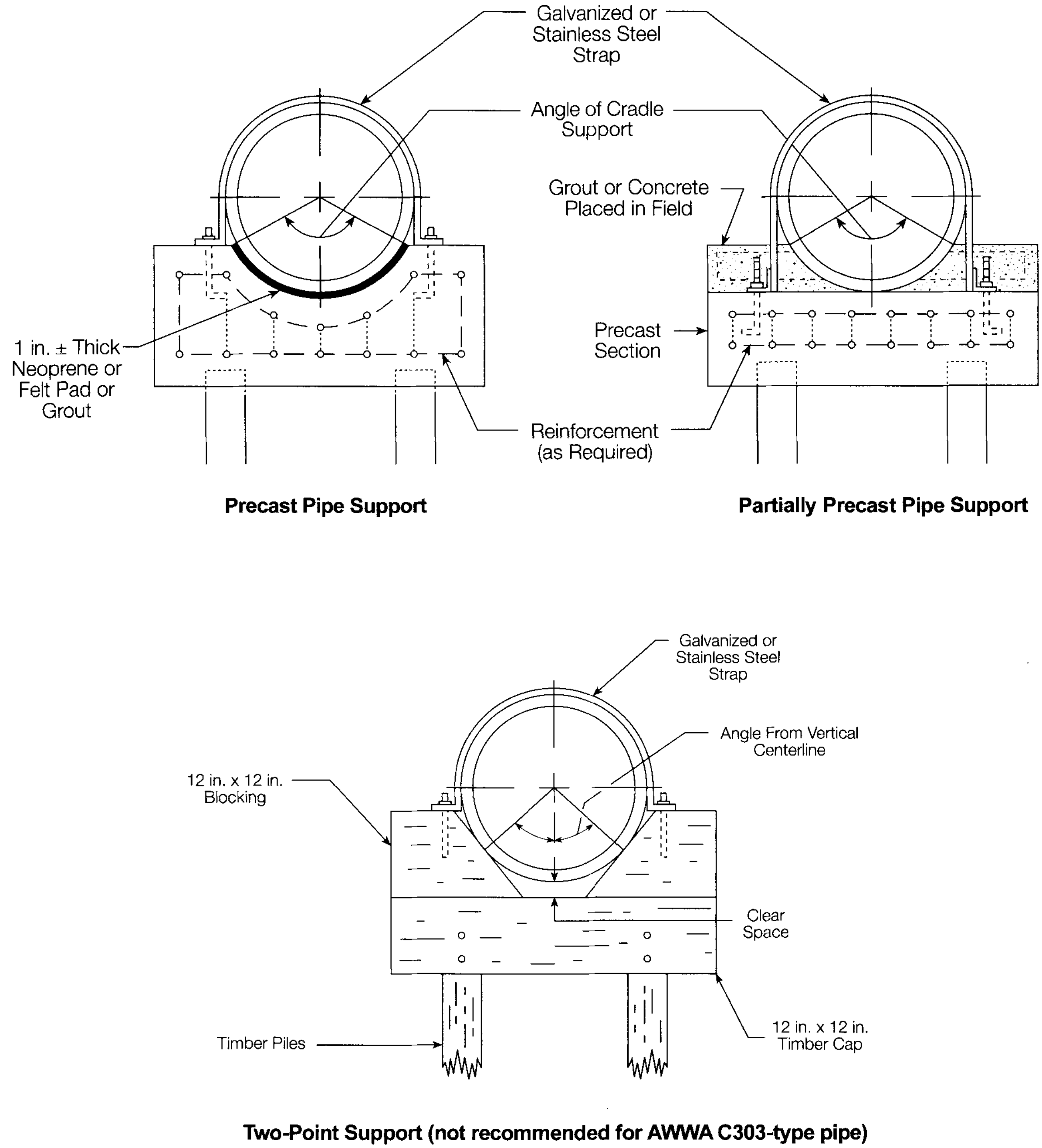
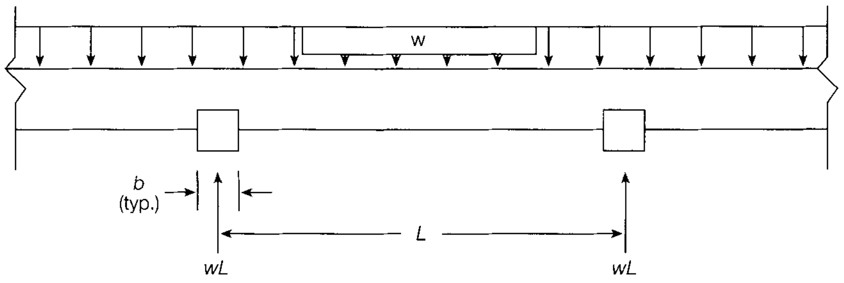
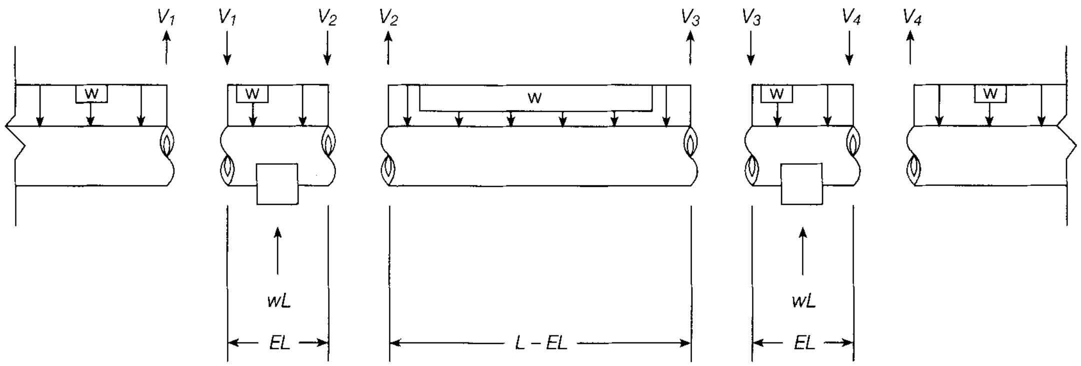


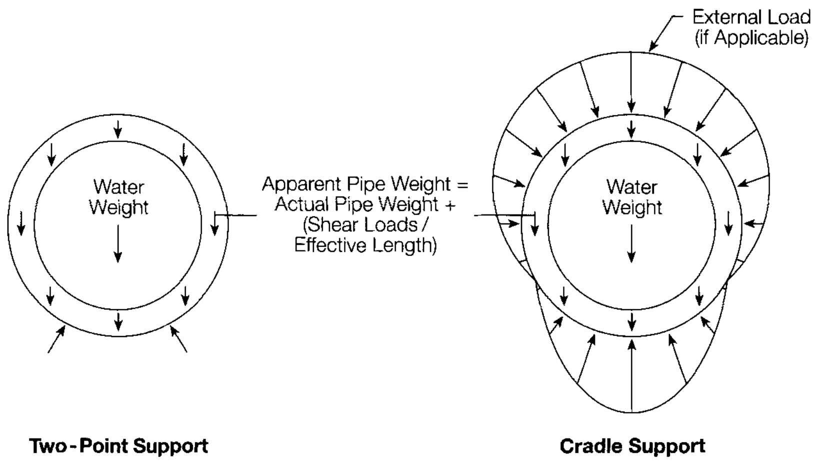

Figure 10-4 Typical supports for pipe on piling




**A. Support Configuration**




**B. Shear Forces in Pipe Wall Support Span Weight**




**C. Load Configuration for Design of Pipe in Effective Length**

Figure 10-5 Assumed load and force configuration for design of rigid pipe on piers

Where:

$\Delta W_p$ = apparent increase in pipe weight in the effective length, lb/lin ft

$V$ = the weight of one half of the span outside the effective length plus any weight supported by that part of the span, lb

$EL$ = effective length, ft

$L$ = span length between pier support centerlines, ft

$w$ = actual total weight of pipe, water, and external load, lb/lin ft

$b$ = pier support width, ft

$d$ = inside diameter of pipe, ft

To determine pipe wall stresses in the effective length, moments and thrusts in the pipe wall are determined using coefficients for cradle support from Olander (1950) and for two-point supports from Paris (1921). These coefficients are applied to the actual pipe and water weights and to the actual external load on the pipe as well as to the apparent increase in pipe weight $\Delta W_p$ as follows:

$$T = C_{tp}(W_p + \Delta W_p) + C_{tw}W_w + C_{te}W_e \qquad \text{(Eq 10-16)}$$

$$M = [C_{mp}(W_p + \Delta W_p) + C_{mw}W_w + C_{me}W_e]R_m \qquad \text{(Eq 10-17)}$$

Where:

$T$ = thrust, lb/lin ft

$C_{tp}$ = thrust coefficient for pipe weight

$W_p$ = pipe weight, lb/lin ft

$C_{tw}$ = thrust coefficient for water weight

$W_w$ = water weight, lb/lin ft

$C_{te}$ = thrust coefficient for external load

$W_e$ = external load, lb/lin ft

$M$ = moment, in.-lb/lin ft

$C_{mp}$ = moment coefficient for pipe weight

$C_{mw}$ = moment coefficient for water weight

$C_{me}$ = moment coefficient for external load

$R_m$ = mean radius of pipe, in.

$\Delta W_p$ is as defined by Eq 10-15.

The wall stresses calculated are then used to determine the required circumferential prestressing wire or steel reinforcement needed for the pipe.

## Semirigid Pipe

Two-point supports should not be used for semirigid (ANSI/AWWA C303-type) pipe. This type of pipe should only be installed on cradle supports. The support length required will depend on the load per foot of span, the pipe design, the span length, and the angle of cradle support. For minimum pipe designs, spans up to 40 ft (12 m) without earth load or surcharge, and 120° cradle supports, the following support lengths are sufficient:

| ANSI/AWWA C303-Type Pipe Diameter, *in.* | Support Length, *ft* |
|---|---|
| ≤ 42 | 1.0 |
| 45 | 1.5 |
| 48 | 2.0 |
| 51 | 2.5 |
| 54 | 3.0 |

For the design of semirigid pipe on piers, the load $Q$ that the pipe must support over the effective length is calculated as

$$Q = \frac{wL}{EL} \qquad \text{(Eq 10-18)}$$

Where:

$Q$ = required design load for semirigid pipe on piers, lb/lin ft

$w$ = total weight of pipe, water, and external load, lb/lin ft

$L$ = span length between pier support centerlines, ft

$EL$ = effective length, equal to the pipe inside diameter plus the pier support width, ft

The allowable supporting strength of the pipe is calculated in accordance with chapter 7, except that the computed wall stiffness is taken as one half of the value derived from the composite wall section of the pipe to account for the additional support of the concrete cradle. Spangler's deflection equation is used to determine the allowable load $W$ that causes a deflection of $D^2/4{,}000$ with the deflection lag factor equal to 1.0 and $E'$ equal to zero. The bedding constant $k$ is determined from the angle of cradle support. As long as the required load $Q$ is less than the allowable load $W$, the design is acceptable.

# PROTECTION OF EXPOSED CONCRETE PIPELINES

Buried concrete pressure pipelines are usually in relatively static, favorable environments where the pipe will not be subjected to large temperature fluctuations, wetting and drying, freeze–thaw conditions, or atmospheric carbonation of concrete. When pipe is installed in an aerial crossing (Figure 10-6) or in a situation where the depth of cover is not sufficient to ensure static conditions (usually 3 to 5 ft [0.9 to 1.5 m] of cover is adequate), proper consideration should be given to protecting the pipe from exposure.

Differential expansion or contraction of the component materials of the pipe will not occur, even when the pipe is exposed in aerial crossings, because the coefficient of thermal expansion for steel and concrete are essentially the same. However, ambient

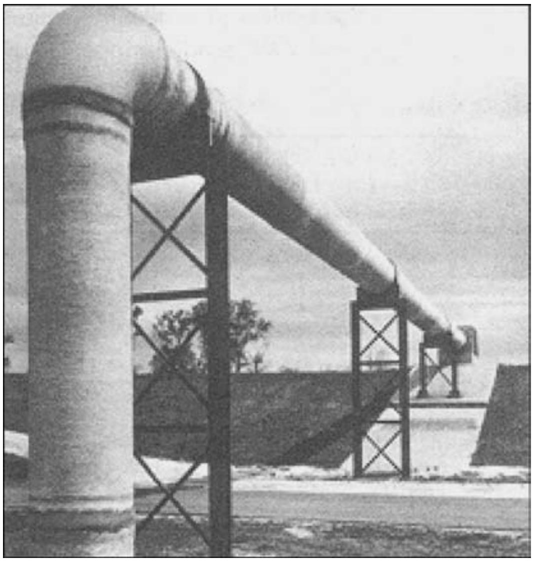


Figure 10-6 50-ft (15-m) spans of 42-in. (1,070-mm) concrete pressure pipe were used in this aerial line crossing of a highway and river levee

temperature changes will cause expansion or contraction of unrestrained pipe or will cause internal stresses in restrained pipe. If exposed pipe is laid with standard rubber-gasket joints poured with cement mortar, the joints will restrain compressive forces, but they will not restrain tensile forces due to pipe contraction. This contraction often causes cracks in the mortar in unrestrained joints, but these cracks can be avoided by welding or otherwise restraining the joint prior to mortaring or by deleting the mortar on unrestrained joints. If mortar is omitted from unrestrained exposed joints, the joint rings should be protected from corrosion by a durable paint system.

Laying unrestrained pipe on piers provides flexible joints, and if the pipe is left unsecured at the pipe supports, it can "walk" over supports during repeated temperature cycles, possibly resulting in a disengaged joint from the accumulation of several joints' flexibility at one location. Therefore, the pipe should be firmly secured at pipe supports to prevent such a disengaged joint and to secure against the forces from hydrostatic thrust and flotation. Bands over the pipe attached to the cradle are sufficient for this purpose.

Pier supports should be designed and installed to support the pipe without damaging it. Cradle supports can be made of concrete cast against the pipe or be premanufactured and used with a compressible bearing pad between the cradle and the pipe. If two-point supports are used, they should be of a material or design that will not dig into the pipe during repeated temperature cycles.

Application of a good-quality paint to the pipe exterior is often recommended for installations of concrete pressure pipe continuously exposed to the atmosphere. Such a coating acts as a barrier to water and atmospheric carbon dioxide, thereby preventing both physical damage to the mortar coating due to wetting–drying or freeze–thaw cycles and loss of alkalinity due to mortar carbonation. The use of white or light-colored paint systems reflects sun heat and reduces temperature stresses. The paint system should be of good quality and renewed periodically to maintain its integrity.

Concrete pipelines should be protected in the transition zone (the point at which the pipe exits the ground or a body of water) to prevent the accumulation of soluble chemicals by wicking (capillary) action in the mortar coating. Application of a high-build epoxy coating is recommended from 3 ft (0.9 m) below to 2 ft (0.6 m) above the ground surface or splash zone.

## REFERENCES

Ameron. 1988. *Concrete Cylinder Pipe-Designs Manual* 202. Monterey Park, Calif.

Olander, H.C. 1950. *Stress Analysis of Concrete Pipe.* USBR Engineering Monograph No. 6. Denver, Colo.: US Bureau of Reclamation.

Paris, J.H. 1921. Coefficients for Large Horizontal Pipes. *Engineering News-Record*, 87:19 (Nov. 10, 1921).

Chapter 11

# Design of Subaqueous Installations

## APPLICATION

Subaqueous concrete pressure pipelines (Figure 11-1) can be located in either freshwater or seawater environments, but all underwater lines require special consideration in design and installation. Concrete pressure pipelines can be used in a wide range of subaqueous applications, including

- water or sewer lines crossing under rivers, estuaries, or lakes
- industrial cooling water intake and discharge lines
- sewer outfalls
- intake lines for potable, irrigation, or other water supplies

## PIPE DESIGN FEATURES

### Evaluation of Forces on Subaqueous Pipe

Earth loads on subaqueous pipe are generally less than would be expected for ordinary installations since the unit weight of backfill is significantly reduced under water. The radially compressive external pressure from water generally is not a significant design consideration because it either (1) is completely offset by internal water pressure if the pipe is full, or (2) causes only a small additional compressive stress in the concrete pipe wall if the pipe is empty. Subaqueous pipe is usually installed in a very uniform support envelope underneath and around the pipe because it is easier to consolidate the bedding under water.

Proposed underwater routes should be analyzed using hydrographic data specific for each site to ensure the line is not jeopardized by an unstable shoreline. Also, when the designer is determining adequate cover and protection for the pipeline and appurtenant structures, consideration should be given to potential problems caused by beach erosion, ships that drag anchor, wave action, scouring, ice jams in rivers, and shore ice in lakes. Sections of pipeline that are installed in deep water not affected by waves, tidal action, or current are not subjected to significant external forces and can be installed with only sufficient cover to prevent buoyancy, if flotation is a design

Figure 11-1 54-in. (1,370-mm) diameter pipe, preassembled to a 32-ft (9.75-m) unit, being lowered with a double bridge sling in a sewer outfall installation

consideration. However, sections of pipeline affected by waves, tidal action, and currents require the most critical treatment and should always be buried in trenches to prevent movement.

## Buoyancy

The possibility of pipeline flotation must always be considered in the design of subaqueous lines, and buoyancy should be analyzed assuming the pipeline is empty unless complete dewatering of the line is impossible. The specific gravity of the material in which the line is to be buried must be considered in buoyancy calculations. Experienced consultants have observed the following:

- A pipeline with a bulk specific gravity of 0.5 or less will work its way through beach sand or other similarly fine sand.
- A pipeline with a bulk density between that of water and sand will have little or no tendency to ascend through fine sand.
- A pipeline is very likely to float out of sand or silt that is subject to wave action if the line is buoyant in water.
- If a pipeline is buoyant in a liquified sediment, then agitation of the sediment will cause the pipe to rise. Cyclic changes in bottom pressure caused by storm waves can sometimes liquify slightly cohesive material and cause flotation. The same effect can also be caused by a variable water table, underground percolation, and unstable clay that is subjected to high water content.
- If a pipeline is to float, then (1) the sediment must act like a fluid, and (2) the bulk specific gravity of the pipeline must be less than the specific gravity of the sediment.

Dead weight is generally the most practical method to anchor an offshore pipeline, with coarse backfill or large riprap being the most commonly used materials. Pipelines with heavy walls may also be used to overcome buoyancy.

## Pipe Buoyancy Analysis

To ensure empty, buried subaqueous pipe will not float, the following equation must be satisfied:

$$\frac{\pi}{4}(B_c^2 - d^2)w_p + HB_c\left(1 - \frac{1}{g_e}\right)w_e \geq SF\left(\frac{\pi}{4}B_c^2 w_w\right) \qquad \text{(Eq 11-1)}$$

Where:

$B_c$ = outside pipe diameter, ft

$d$ = inside pipe diameter, ft

$w_p$ = unit weight of pipe material in air, lb/ft$^3$

$H$ = soil cover over pipe, ft

$g_e$ = specific gravity of backfill particles

$w_e$ = bulk unit weight of dry backfill, lb/ft$^3$

$SF$ = safety factor

$w_w$ = unit weight of water, lb/ft$^3$

The following design values are suggested:

$w_p$ = 150 lb/ft$^3$

$g_e$ = 2.65

$w$ = 110 lb/ft$^3$

$SF$ = 1.5 if overburden is used to offset buoyancy; 1.1 if increased pipe wall thickness is used

$w_w$ = 62.4 lb/ft$^3$ for fresh water, or 64 lb/ft$^3$ for seawater

Using these suggested design values and solving for height of cover, the pipe buoyancy equation (Eq 11-1) reduces to the following:

for fresh water

$$H \geq 1.72(d^2/B_c) - 0.65B_c \qquad \text{(Eq 11-2)}$$

for seawater

$$H \geq 1.72(d^2/B_c) - 0.62B_c \qquad \text{(Eq 11-3)}$$

Where:

$H$ = soil cover over pipe, ft

$d$ = inside pipe diameter, ft

$B_c$ = outside pipe diameter, ft

Also by using the suggested design values, substituting $d$ plus twice the wall thickness for $B_c$, and setting $H$ equal to zero, the pipe buoyancy equation can be used to determine the minimum wall thickness (in feet) that will offset the buoyant force on empty pipe. That formula is as follows:

for fresh water

$$t_{min} = 0.179d \tag{Eq 11-4}$$

for seawater

$$t_{min} = 0.186d \tag{Eq 11-5}$$

Where:

$t_{min}$ = minimum wall thickness, ft

$d$ = inside pipe diameter, ft

## SUBAQUEOUS PIPE DETAILS

Standard concrete pressure pipe is usually used for subaqueous lines, but the following section discusses modifications that can be made to the pipe to provide for joint engaging assemblies, longer lengths, special joints, and joint protection.

### Joint Engagement

Subaqueous joint engagement can be accomplished by a number of different methods, with the most commonly used methods falling into two basic categories—joint-engaging assemblies and hydraulic suction. Joint-engaging assemblies generally consist of heavy metal anchor sockets solidly attached to the wall at the end of the pipe at each springline (Figure 11-2). Joints with these assemblies can be engaged using either draw bolts, as shown in Figure 11-2, or by using powered clamping equipment. This bolting is only temporary and cannot be relied on to harness the joint against thrust. Because the bolting is temporary, it does not require corrosion protection. After the joint is engaged, the drawbolts are either removed or the nuts are backed off 1 in. (25 mm) to facilitate joint movement. The same joint assembly can be used with powered clamping equipment by placing the clamping equipment over the flat faces of the adjacent lugs and then powering the clamping equipment to pull the joint together.

On larger projects, joints can be economically engaged using a patented hydraulic suction method. This method incorporates two bulkheads and a high-volume, low-pressure pump. The shore end of the pipeline is bulkheaded. The pipe to be laid is bulkheaded on one end and the open end is positioned against the end of the previously laid pipe string. When started, the pump removes water at a high rate from the pipeline and the external water pressure forces the bulkheaded pipe into place. This method can be used with standard pipe joints and does not require attaching anchor sockets to the pipe wall.

### Preassembled Lengths

To reduce the number of joints made under water, two or more lengths of pipe can be preassembled before laying. The length of the preassembled pipe depends on available lifting equipment and the beam strength of the pipe. Using a strongback, large-diameter pipe with preassembled lengths up to 100 ft (30 m) have been laid successfully in deep water with proper equipment (Figure 11-3).

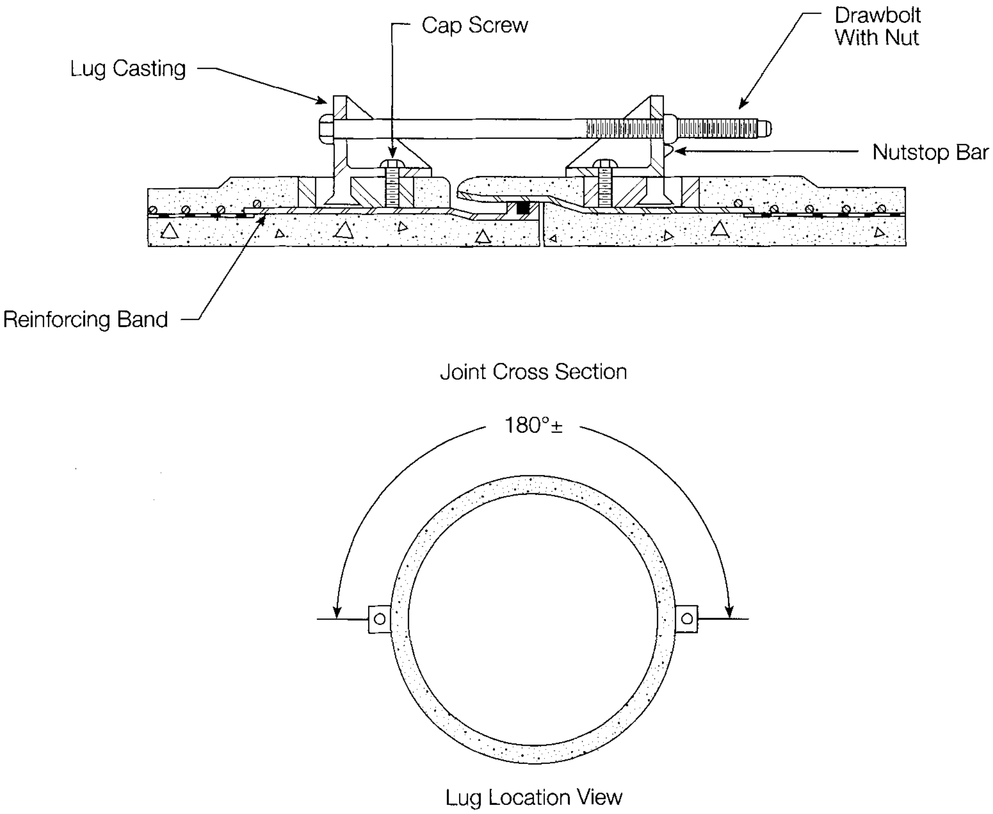



Figure 11-2 Typical subaqueous joint-engaging assembly

## Joints

The conventional bell-and-spigot joint with a rubber gasket is used for most subaqueous pipe joints, but when pipe with conventional joints would be too difficult to install, special ball joints may be cost-effective (Figure 11-4). Ball joints will withstand large joint deflections and longitudinal forces without leakage. Ball-joint pipe up to 40 ft (12 m) in length has been used for numerous projects.

## Joint Protection

Steel joint-ring surfaces on subaqueous pipe can be protected by one of several methods. Pipe that is preassembled and welded before submergence can be protected by filling the interior and exterior joint recesses with cement mortar. Joints assembled subaqueously may be protected with a zinc-rich or other primer and epoxy topcoat or by application of a mastic material that is compressed and encircles the joint when the joint is engaged.

# INSTALLATION

The installation of subaqueous pipe requires special techniques and equipment that vary depending on the size of the pipe, depth of water, and type of bottom.

## Trenching

For soft, mucky material that is stable, "floating" installations can be used (see chapter 6), but for shifting material in shallow water, pier-supported installations are preferable (see chapter 10). In rock or other solid material, timber or concrete sills

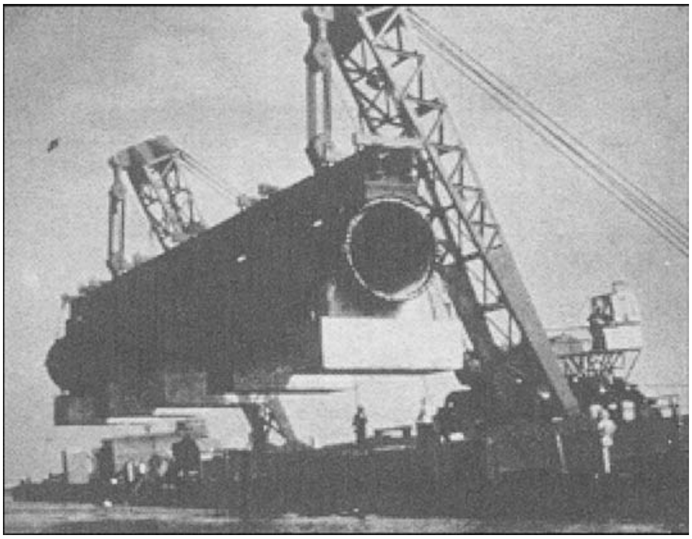


Figure 11-3 Five 20-ft (6-m) sections of 84-in. (2,130-mm) diameter pipe were assembled on deck into a 100-ft (30-m) unit. The 100-ft (30-m) unit was then mounted on precast concrete caps and cradles and installed as a single unit on piles. Two derricks were used to lower the strongback, cradles, caps, and the 100-ft (30-m) pipe unit.

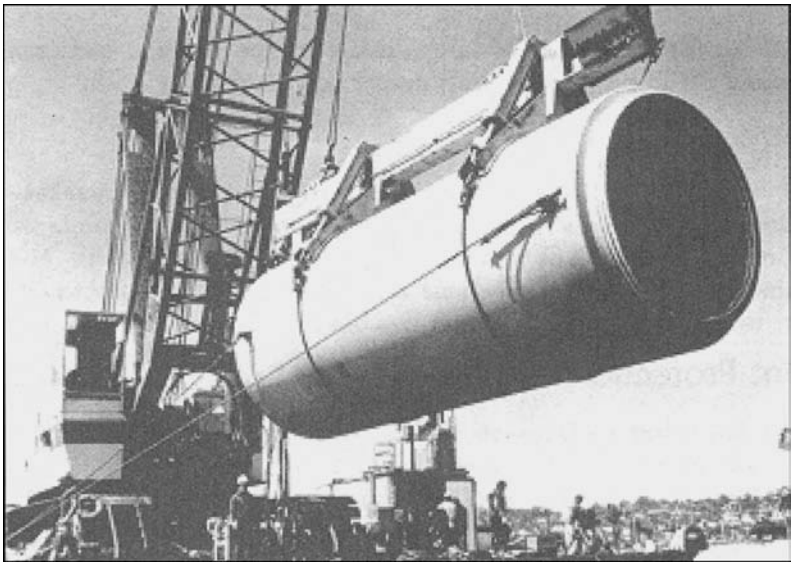


Figure 11-4 Large-diameter pipe being lifted with a special double bridge sling. Joint-engaging assemblies were installed at the ends of the pipe at the springline.

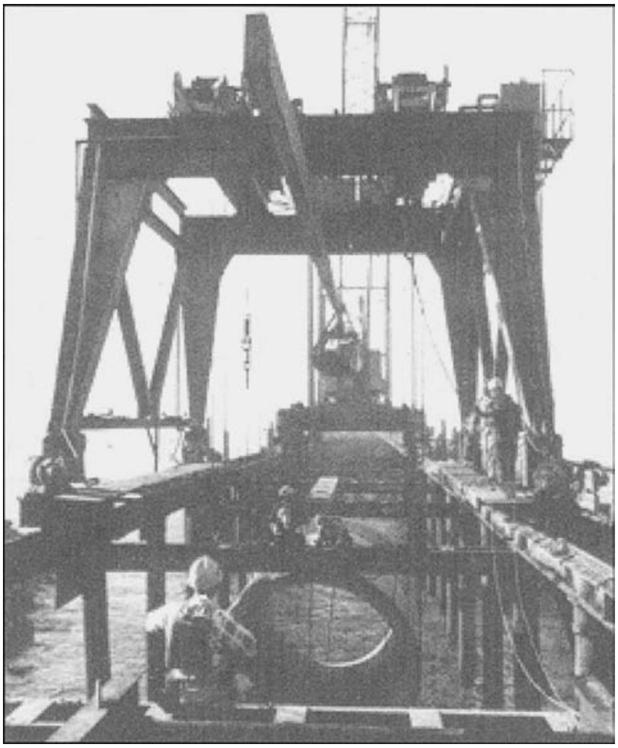


Figure 11-5 Rail-mounted gantry crane used for installing sections of ocean outfall near shore

can be used. Concrete sills should have an embedded timber so shimming wedges can be attached when necessary. In dredged trenches, burlap bags filled with lean dry-mix concrete or sand can be packed into the space between the pipe and the trench to form a support cradle. As with pier-supported installations, usually only one sill or other rigid support is needed per length of pipe, and the support should preferably be located as outlined in chapter 10.

## Pipe Laying

A derrick scow or crane on a barge that is capable of handling the pipe is needed for deep water installations (Figure 11-5), but in shallow water, a temporary causeway may be used. The pipe should be lifted with a double sling. If subaqueous lugs are used, care must be taken to ensure that the lugs are on the horizontal centerline of the pipe. This can be accomplished by laying a spirit level across the ends of the pipe in a direct line with the lugs.

Before the gasket is placed in the annular groove on the spigot ring, it must be coated with a silicone lubricant or a water-resistant vegetable soap, as should the surfaces of both the bell and spigot. With large-diameter pipe, two divers normally direct the positioning of the pipe. The pipe can then be brought together. If this is accomplished with draw bolts, the nuts must be backed off a few turns after joining to allow flexing of the joint. To prevent movement during the backfilling operation, the pipe can be held in place with bags filled with dry concrete mix.

## Backfilling

It is important that backfill is raised symmetrically on each side of the pipe, thereby ensuring that the pipe is evenly supported throughout its length and not pushed off its alignment by the weight of backfill placed on one side. Placing backfill as soon as possible after installing the pipe is especially important if external forces could possibly cause movement. In areas subject to currents, coarse material should be used for backfill. If ice forces, waves, or strong currents are expected, armoring the backfill or weighting the line with rock fill, concrete bags, or concrete weights may be necessary.

## TESTING

Subaqueous pressure water lines are generally tested in the same manner as pipelines buried on land, i.e., by applying pressure between closed valves or bulkheads.

Water intake and outfall lines, which have negligible working pressure requirements, are generally tested by "infiltration" methods. If the pipe is heavy enough to prevent flotation, the line may be dewatered and inspected visually. However, if dewatering will cause flotation, the line can be tested by connecting a suction pump to the high point of each test section and leakage can be determined by measuring the amount of water that can be removed from the line in a prescribed period of time.

Another system frequently used for testing subaqueous intakes and outfalls is the testable joint. The joint consists of two gaskets sealing against the bell ring. Air or water can be used to test the space between the gaskets.

*Chapter* 12

# Design Considerations for Corrosive Environments

## HISTORY

The corrosive-prevention properties of cement mortar and concrete coatings and linings are well known, as these materials have been used since the early 1800s to minimize corrosion of pipe. Experience has shown that cement–mortar protection of ferrous pipe surfaces is still maintained after many years of continuous service.

One of the earliest documented uses of cement–mortar linings was to prevent tuberculation of cast-iron water pipe in France in 1836. Subsequently, Jonathan Ball patented a method of cement–mortar lining of metallic pipe in the United States in 1843, and a wrought-iron water line was installed with a cement–mortar lining in Jersey City, N.J., in 1845. In addition, in 1855, a mortar-lined-and-coated steel pipeline was constructed for the city of St. John, N.B. By the turn of the century, cement–mortar coatings and linings were being used in many cities in North America to protect buried metallic water pipe from corrosion.

## INHERENT PROTECTIVE PROPERTIES

Cement–mortar coatings and linings for concrete pressure pipe are relatively thick and strong, contribute substantially to the structural strength of pipe, and provide resistance to physical damage. Unlike paints and tape coatings, however, they do not insulate the substrate from the environment. Cement mortar protects ferrous elements of concrete pipe (wire, cylinder, rebar, etc.) principally by providing an alkaline environment that passivates the steel.

### Passivation

The unique corrosion-inhibiting properties of cement mortar are due to the chemical nature of portland cement, which has a pH in the range of 12.5 to 13.5. This high alkalinity is created primarily by calcium hydroxide, which is produced from free lime in the cement reacting with moisture in the mortar. In such an alkaline environment, ferrous metal surfaces develop a passivating iron oxide film before the mortar hardens, and even small cracks in the mortar do not harm the protective film in most natural soil and water environments.

# SPECIAL ENVIRONMENTAL CONDITIONS

The concrete and mortar linings and coatings specified in AWWA standards for concrete pressure pipe provide ample corrosion protection in most environments. However, under certain conditions, the ability of the mortar to maintain a passivating environment around the steel components may be compromised, and supplemental corrosion protection may be needed. These unusual environmental conditions include the following:

- High chloride environments and stray current interference that may cause corrosion of the embedded steel
- High sulfate or severe acid conditions or aggressive carbon dioxide in the soil or groundwater that may be potentially damaging to the portland cement matrix in the concrete or mortar coating
- Atmospheric exposure where carbonation of the concrete and mortar surfaces may occur or where the exterior may be subject to freezing and thawing cycles

Low soil resistivities are an indication that high concentrations of chloride, sulfate, or other ions may be present. Since soil resistivity is more easily measured than soil chemistry, it is recommended that a soil resistivity survey be performed along the pipeline right-of-way to locate soils that are potentially aggressive. As a guideline, where soil resistivity, with the soil moistened, is less than 1,500 ohm-cm, soil chemistry should be determined. Wherever moist soil resistivities are not obtained first, soil chemistry should be determined regardless of the dry soil resistivity readings.

## High-Chloride Environments

Chloride ions in sufficient concentration can destroy the passivation of steel embedded in concrete and initiate corrosion if oxygen is also present at the steel surface. For corrosion to continue, the oxygen at the steel surface must be replenished. Lines continuously submerged in seawater do not experience damaging corrosion despite chloride concentrations in excess of 20,000 ppm due to the extremely low rate of oxygen diffusion through the mortar coating. Continuously submerged pipelines do not require supplemental protection over the mortar coating. However, a protective coating system should be applied to the exposed portions of steel joint rings. One effective system is to encase the joint rings in portland cement concrete or grout.

Concrete cylinder pipe buried in soils with significant water soluble chloride concentrations must be evaluated differently than pipe continuously submerged in fresh water or seawater. Where pipe is to be buried in soils with resistivity readings below 1,500 ohm-cm and the water soluble chloride contents exceeds 400 ppm at those same locations, one of the following protective measures should be used:

- A moisture barrier should be used to protect the exterior surfaces.
- Silica fume in an amount equal to 8 to 10 percent of the cement weight or a corrosion inhibitor should be included in the exterior mortar or concrete.
- Cathodic protection should be installed if monitoring of the pipeline detects the onset of corrosion.

## Stray Current Interference

Stray currents from nearby direct current sources, such as impressed current cathodic protection systems and electric railways or subway systems, can be picked up and discharged by buried metallic pipelines. Discharge of such current through pinholes in organically coated steel pipelines results in pitting and perforation of the pipe at the discharge points. Discharge of stray currents from steel encased in concrete or mortar is opposed by polarization effects and is distributed over large surface areas. The discharge consumes excess alkalinity before attacking the embedded steel. However, when stray current interference is anticipated, one of the following measures should be considered:

- Eliminate the source of the interfering stray current if possible.
- Apply a supplementary coating with dielectric properties to the pipe exterior. This increases the concrete pipe's electrical resistance so the surrounding soil will provide a lower resistance path than the pipe for the stray current.
- Provide electrical continuity bonding, if not already provided, and monitor to determine if stray current is picked up and discharged by the pipe. If concrete pipe does pick up a stray current, the current must be drained from the concrete pipe by (1) connecting directly to the source of current, or (2) connecting properly designed sacrificial anodes to the pipeline so the current discharges through the sacrificial anodes. Any electrical current drain should be designed as a site-specific solution to the interference problem.

## High-Sulfate Environments

Instances of sulfate attack on concrete or mortar are generally limited to partially buried structures in soils containing high concentrations of sodium, magnesium, or calcium sulfate. Capillary action and surface evaporation can build up high sulfate concentrations in the concrete and lead to deterioration. Underground concrete pipelines are not normally subject to this type of sulfate buildup. Also, high cement contents typical of the mortar coatings on concrete pressure pipe (in excess of 950 lb/yd$^3$ ([565 kg/m$^3$]) have been shown to increase resistance to sulfate attack. As a result, instances of sulfate attack on buried lines are rare. For buried concrete pipe installed in soils with more than 2,000 ppm water soluble sulfate or in areas with groundwater with more than 2,000 ppm sulfate, cement with a tricalcium aluminate ($C_3A$) content not exceeding 5 percent should be specified. For concrete pipe continuously submerged in seawater, which contains approximately 2,700 ppm sulfate ions, standard type II cement with a maximum of 8 percent $C_3A$ should be specified. A barrier coating or envelope to isolate the concrete or mortar from the sulfates can be used instead of special cement. For installations of mortar-coated pipe in soils with more than 5,000 ppm water-soluble sulfates, use of both low $C_3A$ cement and a barrier material should be considered.

## Severe Acid Conditions

Exterior acid attack on concrete pipe is extremely rare. Documented cases have been associated with unnatural conditions, such as mine wastes, acid spills, or industrial dumps. The acids involved are often mineral acids, such as sulfuric acid mine wastes, with very low pH* and high total acidity† values. Naturally occurring acidic soils frequently are clays and contain milder, less aggressive acids, such as humic acids from decomposed organic materials.

Replenishment of soil water to the surface of the concrete or cement mortar is the most important factor in determining how aggressive a given soil environment can be. This is particularly true of soils that drop in pH after a period of exposure to the atmosphere. Generally, such soils are fine-grained silts or clays, the imperviousness of which greatly reduces the replenishment of the acid soil water to the pipe surface. Even though after a period of atmospheric exposure such soils may have a low pH, the

* pH measures the intensity of an acid. The pH scale is logarithmic from pH 0 to pH 14. pH 7.0 is neutral with an equal number of hydrogen (H+) ions and hydroxyl (OH–) ions active in the solution. As the pH numbers decrease from 7.0, the solution becomes more acidic with more hydrogen ions than hydroxyl ions. As the pH numbers increase above 7.0, the solution becomes more alkaline with more hydroxyl ions than hydrogen ions.

† Total acidity measures the quantity of acid ions (H+) in the soil or soil water. The unit of measure for total acidity is milligram equivalents of acid per 100 g of dry soil (meq/100 g).

total acidity is also low. In this case, the limited amount of acid soil water at the pipe surface is neutralized by the alkalinity of the exterior surface mortar or concrete.

One investigation of two existing pipelines in acid soil conditions found the long-term effect on the cement–mortar coating to be inconsequential (Willett 1981). One pipeline was in soil with pH values immediately after sample excavation ranging from 3.70 to 6.82 and total acidity ranging from 6.4 to 114 meq/100 g of dry soil.* The other pipeline was in soil with pH values immediately after sample excavation ranging from 4.1 to 6.7 and total acidity ranging from 1.6 to 31.4 meq/100 g of dry soil. Careful examination of the steel components of both of these pipelines, at 17 separate locations, found them to be in excellent condition. These two pipelines have been in service in excess of 30 years and 50 years, respectively, with no service interruptions.

Evaluation of aggressive acid soils and potential remedial measures can be generalized as follows:

1. In clay soils, supplemental precautions against acid attack generally are not needed.
2. In granular soils, when the soil pH immediately after sample excavation is greater than 5, supplemental precautions against acid attack of the mortar coating generally are not needed.
3. In granular soils, when the soil pH immediately after sample excavation is less than 5, the total acidity should be determined. If the total acidity is greater than 25 meq/100 g of dry soil, one of the following precautions should be taken:
   - Backfill the pipe zone with consolidated clay material.
   - Backfill the pipe zone with limestone.
   - Provide an acid-resistant membrane on or around the pipe exterior.
   - Add silica fume in an amount equal to 8 to 10 percent of the cement weight to the mortar coating.
4. In all soils, when the soil pH immediately after the sample excavation is less than 4, the pipe should be installed with an acid-resistant membrane or in an envelope of nonaggressive consolidated clay.

## Aggressive Carbon Dioxide

Carbonation is the reaction between calcium hydroxide and other hydration products in mortar or concrete and environmental carbon dioxide to produce calcium carbonate. Calcium carbonate has a lower pH than calcium hydroxide, so steel embedded in completely carbonated concrete or mortar will not maintain the passivating iron-oxide film. Surface carbonation of concrete and mortar may occur in the presence of carbon dioxide and moisture but is not detrimental to the pipe. However, penetrating carbonation sufficient to damage buried concrete pipe can occur under some conditions. These conditions include carbon dioxide generation in the soil from sources such as decaying vegetation or geothermal activity, relatively soft groundwater, groundwater with high dissolved carbon dioxide from rainwater, and highly permeable soil. If such conditions are suspected to exist, testing of groundwater for aggressive carbon dioxide should be performed. A consensus on limiting values for soil or groundwater conditions beyond which protective measures are recommended does not exist. When history or current soils and groundwater analyses indicate a potential problem, either a clay backfill should encase the pipe, a membrane should be placed around the pipe, or the pipe exterior should be treated with a sealer or protective coating.

## Atmospheric Exposure

Mortar coatings on pipe installed above ground are subject to an environment different from buried lines, which are in conditions of moderate change in temperature and

* meq is the abbreviation for milliequivalent.

moisture. An exposed line may be subjected to large temperature fluctuations, wetting and drying cycles, freezing and thawing cycles, and atmospheric carbonation (the reaction between calcium hydroxide in the mortar and carbon dioxide in the atmosphere), which may, over a period of time, detrimentally affect the protective properties of the mortar coating. For such installations, a light-colored barrier coating over the pipe exterior may be used initially and periodically renewed if deemed necessary to maintain the integrity of the underlying mortar coating.

### Connections to Other Pipelines

Concrete or mortar encasement of steel or iron results in a potential that is approximately 300 to 500 millivolts (mV) more noble than the bare or organically coated metal. If a connection is made between concrete-encased steel and bare or organically coated steel or iron, the concrete pipe may be protected by sacrificial corrosion of the bare or organically coated pipeline. This problem is mitigated over time by polarization of the potential of the concrete pipeline toward the potential of the steel or iron pipeline. The current flow is then reduced to a much lower value, and the problem becomes inconsequential in most cases. If it is desired to completely eliminate the problem, an insulating connection should be used between the concrete pipe and the ferrous pipeline, or the ferrous pipeline should be concrete or mortar encased to bring both pipe materials to the same potential.

## BONDING PIPELINES

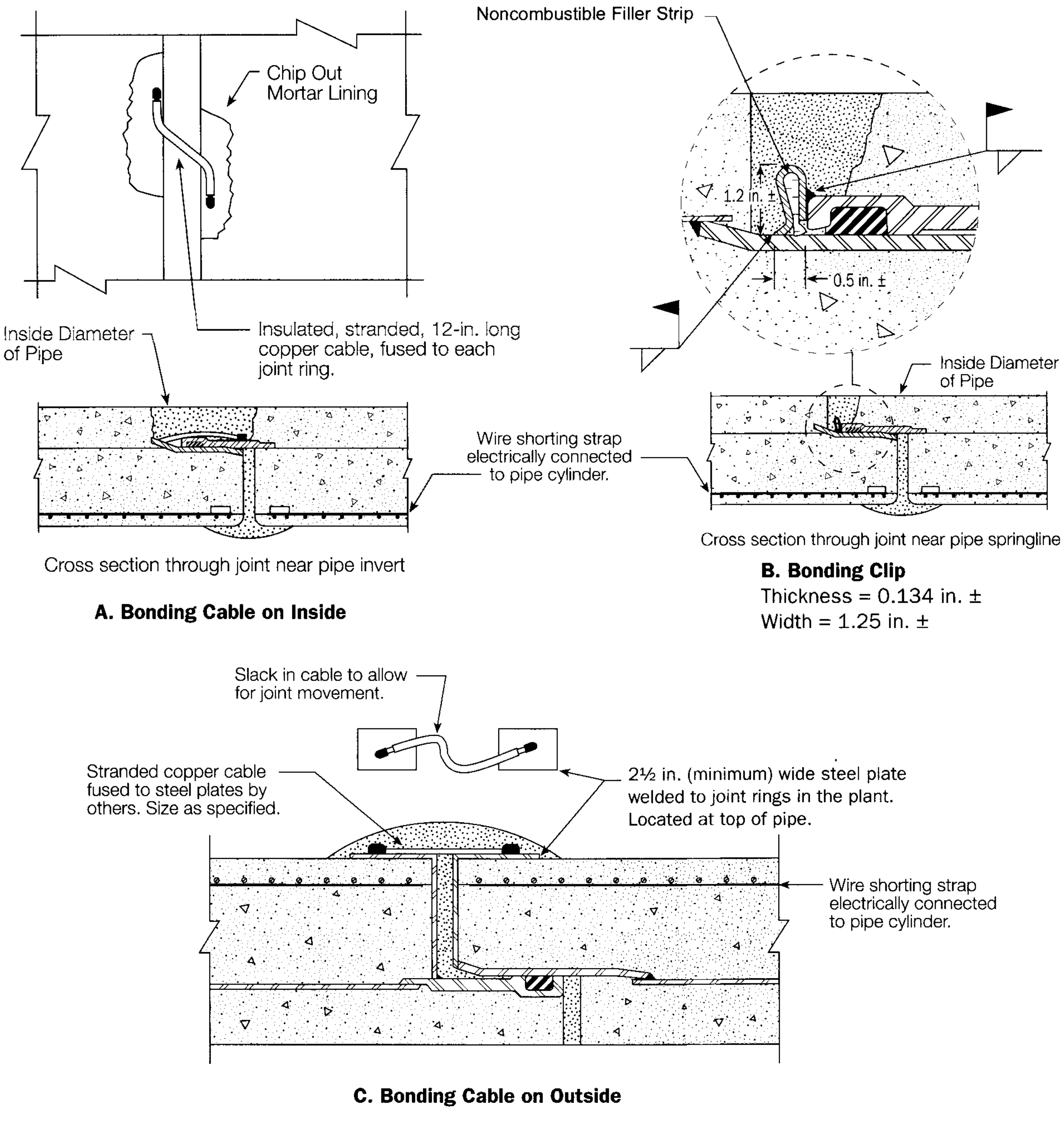
Electrical bonding of buried concrete pressure pipe joints is required if monitoring is planned in those lengths of pipeline or where possible application of cathodic protection is a possibility. Typical bonding details for ANSI/AWWA C301- and C303-type pipe are shown in Figures 12-1 and 12-2; similar procedures can be used on ANSI/AWWA C300- and C302-type pipe. A minimum of two bonds are sometimes specified for redundancy. Test leads and permanent test stations should be provided every 1,000 to 2,000 ft (300 to 600 m) along the pipelines. The spacing of test leads and test stations depends on pipe diameter, soil resistivity, pipeline alignment, location of appurtenances, and a number of other factors. The recommendations of an experienced corrosion engineer should be sought.

When bonding embedded cylinder (ANSI/AWWA C301-type) pipe, provisions for electrical continuity of steel elements within each pipe section must be provided during manufacture. A mild-steel shorting strap in electrical contact with each wrap of prestressing wire is sufficient to adequately reduce the voltage drop that would occur in the long, unshorted prestressing wire if cathodic protection were later applied. Typically, two shorting straps are installed approximately 180° apart on each pipe for redundancy.

To determine the number and size of bonds required per joint, the electrical resistance of the steel cylinder, joint bonds, and fringing effect must be calculated and related to the electrical current attenuation along the pipeline, taking into account soil resistivity and pipeline geometry. For more information on current attenuation, see Unz (1960).

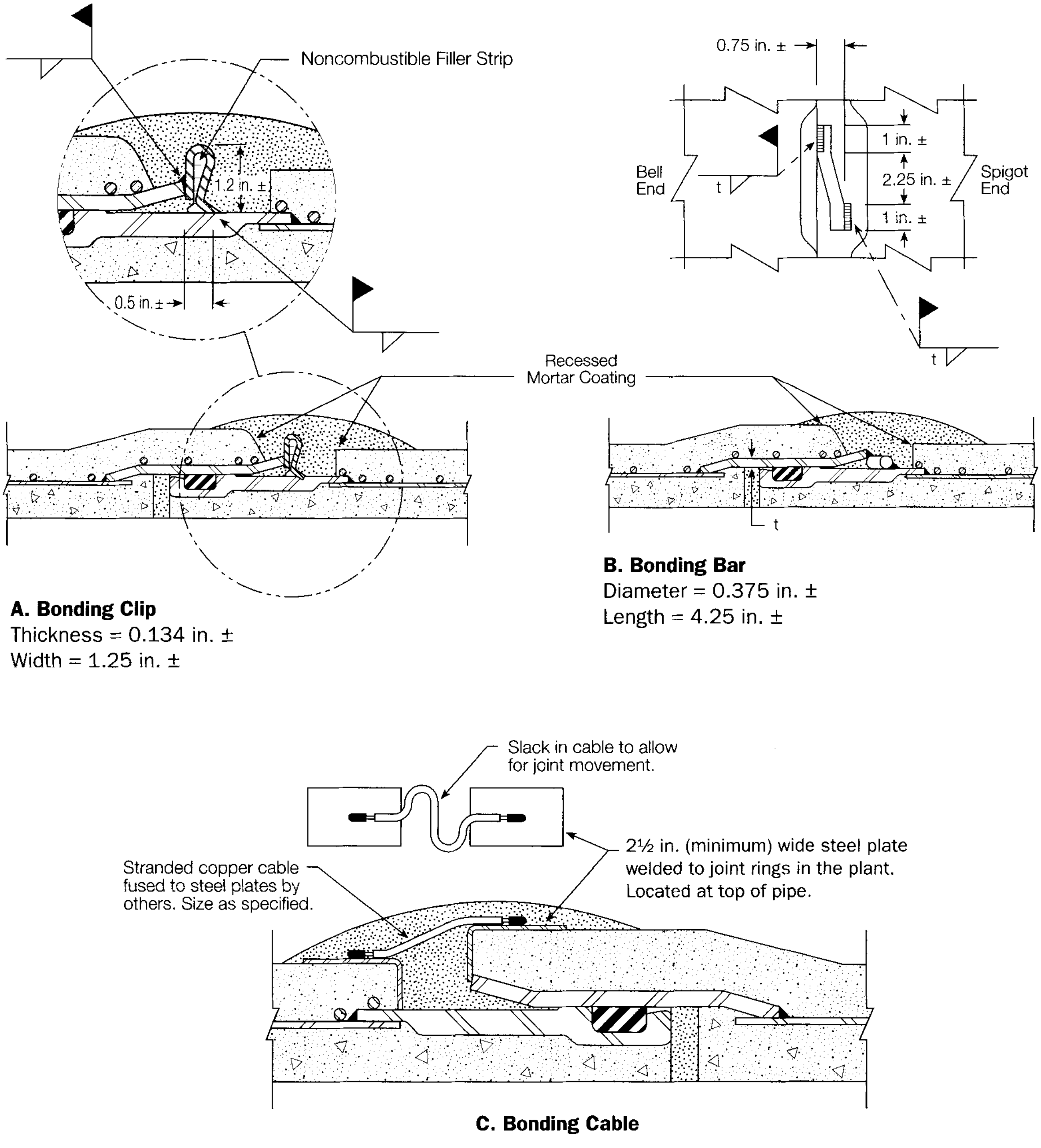
The maximum acceptable bond resistance for a particular design situation can be determined from Eq 12-1 after an acceptable current density attenuation factor $A_f$ is selected. An $A_f$ of 0.5 has been successfully used to provide adequate current density at locations remote from the electrical current drain point (i.e., the anode or rectifier connection).

The following procedures are suggested for determining a reasonable number of bonds of a particular size and shape to yield an acceptable attenuation. Some initial assumptions must be made and the equation solved on an iterative basis until a practical number of bonds $N$ of a particular size are determined.




NOTE:
Recesses in the lining of the concrete core are chipped as required after pipe field assembly or are blocked out during pipe manufacture. The bonding cable or bonding clip is welded to the joint rings as shown in diagrams A and B when bonding inside the pipe and all recesses are filled with cement mortar. When bonding from the outside as shown in diagram C, the bonding cable is welded to the angle-shaped clips and encased in the exterior joint grout.

Figure 12-1 Typical joint bonding details for embedded cylinder (AWWA C301-type) pipe




The bonding methods shown provide electrical conductivity across the joint and accommodate relative movement due to pipeline settlement. To provide access for welding the bonds, as shown in diagrams A and B, recesses are chipped in the mortar coating as required after field assembly. Separate bonding is not required when joints are field welded.

Figure 12-2 Typical joint bonding details for AWWA C303-type pipe or lined cylinder AWWA C301-type pipe

Step 1 Select an acceptable attenuation factor $Af$ and solve Eq 12-1 for $x$ with the terms for $R_b$ and $R_f$ set to zero. The quantity $x$ is one half the maximum number of pipe sections between drain points for the given attenuation factor when fringing resistance and bonding resistance are set to zero.

$$\underbrace{4.72\rho_s L_c / [\pi T_c (D - T_c)]}_{\text{(cylinder resistance } R_c)} + \underbrace{\rho_c L / (2.54 A_b N)}_{\text{(bond resistance } R_b)} + \underbrace{2D\rho_s \ln\{D/[2N(A_w/\pi)^{1/2}]\} / [2.54\pi T_c(D - T_c)]}_{\text{(fringing resistance } R_f)} = R_e[(-\ln A_f)/x]^2 \quad \text{(Eq 12-1)}$$

Where:

$R_c$ = cylinder resistance, $4.72\rho_s L_c / [\pi T_c(D - T_c)]$, ohms

$\rho_s$ = resistivity of cylinder steel, ohm-cm

$L_c$ = length of cylinder in a pipe section, taken as the laying length of pipe, ft

$T_c$ = thickness of cylinder, in.

$D$ = outside diameter of steel cylinder, in.

$R_b$ = bond resistance, $\rho_c L_b / (2.54 A_b N)$, ohms

$R_f$ = fringing resistance, $2D\rho_s \ln\{D/[2N(A_w/\pi)^{½}]\}/[2.54\pi T_c (D - T_c)]$, ohms

$\rho_c$ = bond material resistivity, ohm-cm

$L_b$ = length of bonding copper cable, steel bar, or clip, in.

$A_b$ = cross-sectional area of bonding copper cable, steel bar, or clip, in.$^2$

$N$ = number of copper cables, steel bars, or clips at each pipe joint

$A_w$ = surface area of bond cable, bar, or clip weldment to cylinder, in.$^2$

$A_f$ = attenuation factor

$x$ = one-half the number of pipe sections between current drain points

$R_e$ = the per unit earth resistance of a laying length of concrete pressure pipe, ohms. The quantity $R_e$ must be determined from Dwight's equation for a horizontal conductor taking into account soil resistivity and geometry. This is shown in the following equation.

$$R_e = [\rho_1 / (60.96\pi L_c)]\{[\ln(24L_T/r)] + [\ln(2L_T/s)] - 2 + (s / L_T) - [s^2/(4L_T^2)] + [s^4 / (32L_T^4)]\} \quad \text{(Eq 12–2)}$$

Where:

$\rho_1$ = soil resistivity, ohm-cm

$L_c$ = length of cylinder in a pipe section, taken as the laying length of pipe, ft

$L_T$ = total length of pipeline under study, ft

$r$ = radius of cylinder, in.

$s$ = twice the depth of soil cover over the pipe, ft

Step 2 Select values for $L_b$, $A_b$, and $A_w$.

Step 3 Reduce the value of $x$ determined from step 1 and substitute into Eq 12-1. Rearrange Eq 12-1, as shown in the following, and substitute integer values for $N$, the number of bonds at each pipe joint, until the inequality is satisfied. Evaluate the practicality of the resulting number. If the number of bonds at each joint is reasonable, then stop and design accordingly.

Rearranged Eq 12-1:

$$\rho_c L_b/(2.54A_b N) + 2D\rho_s \ln\{D/[2N(A_w/\pi)^{1/2}]\}/[2.54\pi T_c(D - T_c)] <$$
$$R_e[-\ln(A_f)/x]^2 - 4.72\rho_s L_c/[\pi T_c(D - T_c)]$$

Step 4 If the number of bonds is excessive, then one of the variables must be changed and step 3 repeated. The parameters that can be changed include $L_b$, $A_b$, $A_w$, and $x$. The number of bonds will be reduced more quickly by varying $x$, the number of pipe lengths between drain points. The other variables will reduce the number of bonds required; however, major changes will have little impact on $N$.

## Design Example

Determining the number of bonds to yield an acceptable attenuation.

48-in. lined cylinder pipe

$L_c$ = 20 ft

$T_c$ = 0.0598 in. (16 ga)

$D$ = 54 in.

$L_T$ = 10,000 ft

$s$ = 2 × 5 ft = 10 ft

$\rho_1$ = 15,000 ohm-cm

$r$ = $D/2$ = 27 in.

$\rho_c$ = 17 × $10^{-6}$ ohm-cm (Table 12-1)

Solution is as follows:

Step 1:

a. Let $A_f = 0.5$ $$x = \frac{-\ln(A_f)}{\sqrt{R_c/R_e}}$$

b. Solve Eq 12-1 for $x$ with $R_b$ and $R_f$ equal to zero.

$R_c + R_b + R_f = R_e[-\ln(A_f)/x]^2$

Determine $R_e$ by direct substitution into Eq 12-2.

$$R_e = [15{,}000/(60.96\pi 20)]\{\ln[24(10{,}000/27]\} = \{\ln[2(10{,}000/10)]$$
$$- 2 + 10/10{,}000 - 10^2/[4(10{,}000^2)] + 10^4/32(10{,}000^4)$$

$R_e$ = 57.55 ohms

c. Determine $R_c$

$R_c = 4.72\rho_s L_c/[\pi T_c(D - T_c)]$

$R_c = 4.72(17 \times 10^{-6})(20)/[\pi(0.0598)(54 - 0.0598)]$

$R_c = 1.5836 \times 10^{-4}$ ohms

d. Determine $x$

$$x = \frac{-\ln(0.5)}{[1.5836(10^{-4})/57.5466]^{1/2}} = 417.84$$

∴ Maximum distance between drain points 20 × 2 × 417.84 = 16,713 ft

Step 2: Select a 12-in. long, No. 6 copper wire bond. From Table 12-1,

$A_b = 0.0314$ in.$^2$, $A_w = 0.785$ in.$^2$, $\rho_c = 2.7 \times 10^{-6}$ ohm-cm

Step 3: Let $x = 125$ (distance between drain points = 125 × 20 × 2 = 5,000 ft)

Rearrange Eq 12-1 as follows:

$R_b + R_f < R_e[-\ln(A_f)/x]^2 - R_c$

$R_e = 57.55$ ohms (from step 1[b])

$R_c = 1.5836 \times 10^{-4}$ ohms (from step 1[c])

Try $N = 1$

$R_b = \rho_c L_b / (2.54 A_b N) = 2.7 \times 10^{-6}(12) / [(2.54)(0.0314)(1)]$

$R_b = 4.062 \times 10^{-4}$ ohms

$R_f = 2D\rho_s \ln\{D/[2N(A_w/\pi)^{0.5}]\} / [2.54\pi T_c(D - T_c)]$

$R_f = 2(54)(17 \times 10^{-6}) \ln\{54/[2(1)(0.785/\pi)^{0.5}]\} / [2.54\pi(0.0598)(54 - 0.0598)]$

$R_f = 2.8455 \times 10^{-4}$ ohms

$$R_b + R_f < R_e[(-\ln A_f / x]^2 - R_c$$

$$(4.062 \times 10^{-4}) + (2.8455 \times 10^{-4}) < 57.55\,[(-\ln 0.5) / 125]^2 - (1.5836 \times 10^{-4})$$

$$6.9074 \times 10^{-4} < 1.610 \times 10^{-3} \text{ (yes)}$$

Therefore, a single 12-in. long No. 6 copper bond will be sufficient.

Table 12-1 Assumed values for required joint bonds shown in Tables 12-2, 12-3, and 12-4

| | |
|---|---|
| Resistivity of steel, $\rho_c$ or $\rho_s$ | 17.0 microhm-cm |
| Resistivity of copper, $\rho_c$ | 2.7 microhm-cm |
| Thickness of steel cylinders, $T_c$ | 0.0598 in. (16 gauge) |
| Outer diameters of steel cylinders, $D$ | 48- to 50.75-in. pipe |
| | 84- to 88-in. pipe |
| | 144- to 150-in. pipe |
| Pipe laying length, $L_c$ | 20 ft for all pipe sizes |
| Bond wires: | |
| Length, $L_b$ | 12 in. |
| Cross-sectional area, $A_b$ | No. 6—0.0314 in.$^2$ |
| | No. 1/0—0.106 in.$^2$ |
| | No. 4/0—0.214 in.$^2$ |
| Weldment cross-sectional area, $A_w$ | 0.785 in.$^2$ |
| Bond clip: | |
| Length, $L_b$ | 2.125 in. |
| Cross-sectional area, $A_b$ | 0.168 in.$^2$ |
| Depth, $s/2$ | 5 ft |

Table 12-1 lists the basis for Tables 12-2 through 12-4, which present the number of bonds required by applying the previously described procedures to various pipe sizes and soil resistivities. The results are specific for a typical attenuation factor $A_f$ of 0.5 and are presented to illustrate the effects of soil resistivity on the number of bonds needed. Calculations similar to the ones presented should be performed with project-specific parameters.

Tables 12-2, 12-3, and 12-4 show the minimum number of bonds of a particular size versus pipeline length between drain points for various soil resistivities to give an attenuation factor of 0.5. The tables are based on establishing drain points along the pipeline in a symmetrical manner. As an example, using Table 12-2, a single No. 6 bond is adequate in 10,000 ohm-cm soil for 5,000 ft of 48-in. pipe between drain points. Therefore, for a 10,000-ft-long structure, drain points would be required at 2,500 ft and 7,500 ft from one end.

## MONITORING FOR PIPE CORROSION

It is good engineering practice to periodically monitor the electrical potential of all buried pipelines having metallic components to determine if electrochemical conditions exist that could detrimentally affect the performance of the pipeline. By periodically monitoring the pipeline, corrosion activity can be detected and remedial action taken before significant damage occurs.

Potentials reflect electrochemical reactions occurring on metal surfaces in the pipeline and are measured against a standard reference electrode. A copper–copper sulfate electrode (CSE) is commonly used for pipeline surveys. Potentials should be measured at close intervals equally spaced along the pipeline to avoid missing unusual conditions. Spacing of 3 to 10 ft is commonly used.

If a pipeline does not have applied direct current on it (no stray current and no cathodic protection), pipeline potentials less negative than –200 mV (CSE) indicate there is no damaging corrosion. Pipeline potentials more negative than –350 mV (CSE), large variations in potentials along the line, or long-term trends toward more negative potentials are indications of corrosion activity.

Table 12-2 Number of bonds for 48-in. prestressed concrete cylinder pipe

| | Number of Steel Bonding Clips for Indicated Soil Resistivity | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | | |
| 6,000 | 1 | 1 | | |
| 7,000 | 1 | | | |
| 8,000 | 1 | | | |
| 9,000 | 1 | | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 2 | | | |
| 13,000 | 2 | | | |
| 14,000 | 4 | | | |

| | Number of No. 6 Bond Wires for Indicated Soil Resistivity | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 3 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 2 | |
| 5,000 | 1 | 1 | 3 | |
| 6,000 | 1 | 2 | | |
| 7,000 | 1 | 3 | | |
| 8,000 | 1 | 5 | | |
| 9,000 | 1 | | | |
| 10,000 | 2 | | | |
| 11,000 | 2 | | | |
| 12,000 | 3 | | | |
| 13,000 | 4 | | | |

*Table continued next page*

Table 12-2 Number of bonds for 48-in. prestressed concrete cylinder pipe (continued)

| | Number of No. 1/0 Bond Wires for Indicated Soil Resistivity | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 1 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | 2 | |
| 6,000 | 1 | 1 | 4 | |
| 7,000 | 1 | 1 | | |
| 8,000 | 1 | 3 | | |
| 9,000 | 1 | 5 | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 2 | | | |
| 13,000 | 3 | | | |
| 14,000 | 4 | | | |

| | Number of No. 4/0 Bond Wires for Indicated Soil Resistivity | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 1 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | 1 | |
| 6,000 | 1 | 1 | 4 | |
| 7,000 | 1 | 1 | | |
| 8,000 | 1 | 2 | | |
| 9,000 | 1 | 5 | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 1 | | | |
| 13,000 | 2 | | | |
| 14,000 | 3 | | | |
| 15,000 | 5 | | | |

Table 12-3 Number of bonds for 84-in. prestressed concrete cylinder pipe

| | Number of Steel Bonding Clips for Indicated Soil Resistivity | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 1 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | 1 | |
| 6,000 | 1 | 1 | 3 | |
| 7,000 | 1 | 1 | | |
| 8,000 | 1 | 2 | | |
| 9,000 | 1 | 4 | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 1 | | | |
| 13,000 | 2 | | | |
| 14,000 | 3 | | | |
| 15,000 | 4 | | | |

| | Number of No. 6 Bond Wires for Indicated Soil Resistivity | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 3 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 2 | |
| 5,000 | 1 | 1 | 3 | |
| 6,000 | 1 | 2 | | |
| 7,000 | 1 | 3 | | |
| 8,000 | 1 | 4 | | |
| 9,000 | 1 | | | |
| 10,000 | 2 | | | |
| 11,000 | 2 | | | |
| 12,000 | 3 | | | |
| 13,000 | 4 | | | |

*Table continued next page*

Table 12-3 Number of bonds for 84-in. prestressed concrete cylinder pipe (continued)

| Number of No. 1/0 Bond Wires for Indicated Soil Resistivity | | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 1 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | 2 | |
| 6,000 | 1 | 1 | 4 | |
| 7,000 | 1 | 1 | | |
| 8,000 | 1 | 2 | | |
| 9,000 | 1 | 5 | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 2 | | | |
| 13,000 | 2 | | | |
| 14,000 | 3 | | | |
| 15,000 | 5 | | | |

| Number of No. 4/0 Bond Wires for Indicated Soil Resistivity | | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 1 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | 1 | |
| 6,000 | 1 | 1 | 3 | |
| 7,000 | 1 | 1 | | |
| 8,000 | 1 | 2 | | |
| 9,000 | 1 | 4 | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 1 | | | |
| 13,000 | 2 | | | |
| 14,000 | 3 | | | |
| 15,000 | 4 | | | |

## Table 12-4 Number of bonds for 144-in. prestressed concrete cylinder pipe

**Number of Steel Bonding Clips for Indicated Soil Resistivity**

| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
|---|---|---|---|---|
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 1 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | 2 | |
| 6,000 | 1 | 1 | 4 | |
| 7,000 | 1 | 1 | | |
| 8,000 | 1 | 2 | | |
| 9,000 | 1 | 5 | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 2 | | | |
| 13,000 | 2 | | | |
| 14,000 | 3 | | | |
| 15,000 | 5 | | | |

**Number of No. 6 Bond Wires for Indicated Soil Resistivity**

| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
|---|---|---|---|---|
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 3 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 2 | |
| 5,000 | 1 | 1 | 3 | |
| 6,000 | 1 | 2 | | |
| 7,000 | 1 | 3 | | |
| 8,000 | 1 | 5 | | |
| 9,000 | 1 | | | |
| 10,000 | 2 | | | |
| 11,000 | 2 | | | |
| 12,000 | 3 | | | |
| 13,000 | 4 | | | |

*Table continued next page*

Table 12-4 Number of bonds for 144-in. prestressed concrete cylinder pipe (continued)

| Number of No. 1/0 Bond Wires for Indicated Soil Resistivity | | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 2 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | 2 | |
| 6,000 | 1 | 1 | 4 | |
| 7,000 | 1 | 2 | | |
| 8,000 | 1 | 3 | | |
| 9,000 | 1 | 5 | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 2 | | | |
| 13,000 | 3 | | | |
| 14,000 | 4 | | | |

| Number of No. 4/0 Bond Wires for Indicated Soil Resistivity | | | | |
|---|---|---|---|---|
| Length Between Drain Points, *ft* | 25,000 ohm-cm | 10,000 ohm-cm | 5,000 ohm-cm | 1,000 ohm-cm |
| 1,000 | 1 | 1 | 1 | 1 |
| 2,000 | 1 | 1 | 1 | 1 |
| 3,000 | 1 | 1 | 1 | |
| 4,000 | 1 | 1 | 1 | |
| 5,000 | 1 | 1 | 1 | |
| 6,000 | 1 | 1 | 4 | |
| 7,000 | 1 | 1 | | |
| 8,000 | 1 | 2 | | |
| 9,000 | 1 | 4 | | |
| 10,000 | 1 | | | |
| 11,000 | 1 | | | |
| 12,000 | 1 | | | |
| 13,000 | 2 | | | |
| 14,000 | 3 | | | |
| 15,000 | 5 | | | |

Deciding to monitor a pipeline also requires commitment by the owner to establish a periodic monitoring program. When a line is not monitored, an owner is unlikely to realize the full benefits from the added initial cost of making the line electrically continuous, and there is a possibility that corrosion-related problems on a bonded line, if they occur, may progress faster than on an unbonded line.

There are two basic survey techniques that are employed during monitoring work; these are:

- Pipe to soil potential
- Side drains

Each is defined in the following sections with an additional discussion related to surface conditions.

**Pipe to soil potential.** A pipe to soil potential survey is performed on electrically continuous piping systems. Potentials are measured by connecting one pole of a voltmeter to the pipeline at a test station and connecting the other pole to a CSE. The CSE is moved along directly above the pipeline and voltage measurements are made approximately every 2.5 ft (0.75 m). Potentials are recorded and an "over-the-line" potential profile is later generated in graphic form.

**Side drains.** Side drain potentials refer to potentials measured laterally away from the pipeline in both directions. Physical constraints permitting, the side electrodes are placed away from the pipe a minimum distance of one half the pipe diameter plus the depth of cover. The voltmeter connection to the pipeline is the same as that described for a pipe-to-soil potential survey. Data are also collected every few feet along the pipeline. A potential profile is later generated in graphic form.

There are some limitations in monitoring for corrosion activity on any pipeline:

- The soil surface must be adequately moist to obtain meaningful readings.
- Monitoring potentials cannot be taken through either concrete or asphalt paving. When readings are to be taken in paved areas, it is necessary to drill small holes through the pavement to permit the reference electrode to be in contact with moist soil or to place the reference cell on unpaved soil near the pipeline.
- Stray currents or impressed direct currents affect potential measurements. Currents collecting on the pipeline make measured potentials more negative. Discharging currents make potentials more positive.
- Potentials of connected organically coated steel pipelines are more negative than potentials of concrete or mortar coated steel and will therefore appear as corrosion sites on a concrete pipeline.
- Knowledge of the existence and location of other underground structures (tanks, pipelines, etc.) in the vicinity and appurtenances (valves, branch outlets, etc.) on the pipeline being monitored are necessary for proper interpretation of the readings.

An effective monitoring program starts by taking an initial set of readings about one year after the pipeline is installed. Time intervals between subsequent surveys are usually established after evaluating the results of prior surveys. Changes in the vicinity of the pipeline, such as new construction, implementation of cathodic protection, or adjustments in existing cathodic protection systems on other structures, may necessitate surveys sooner than otherwise scheduled. Significant changes in the readings at a particular location over time or significant differences in the readings at one location compared with readings at adjacent or nearby locations may be indicative of corrosion or stray current interference. A qualified corrosion engineer, experienced in the protection of concrete pressure pipe, is important not only for conducting monitoring surveys but also for interpreting the readings.

## CATHODIC PROTECTION

Cathodic protection of concrete pressure pipe has rarely been used and should not be applied indiscriminately. The determination of whether to apply cathodic protection should include an analysis of potential for cathodic protection damage, possibly based on performance of other pipelines in the area, including both steel and concrete pressure pipe. For the unusual installation where protection is needed, a criterion for cathodic protection is a minimum of 100 mV shift of potential between the structure surface and a stable reference electrode contacting the electrolyte. The formation or decay of polarization can be measured to satisfy this criterion.

It is very important that maximum current interrupted (polarized or "instant off") potential not be more negative than –1,000 mV (CSE) on prestressed concrete pipe (ANSI/AWWA C301-type pipe) to avoid evolution of hydrogen and possible embrittlement of the prestressing wire. This maximum potential is based on the simplified Nernst equation (Eq 12-3), which states that the potential $E$ at which hydrogen is produced is dependent on the pH.

$$E = (-59.2\text{pH}) - 300 \qquad \text{(Eq 12-3)}$$

When cathodic protective current is applied to a steel surface at a pH of 12.5, which is typical for concrete and mortar, hydrogen will be produced at potentials more negative than –1,040 mV at 77°F (25°C) and 1 atmosphere pressure. If the environment around the steel has a lower pH, both hydrogen and hydroxyl ions will be produced until the hydroxyl ion concentration raises the pH immediately at the steel surface back to 12.5. Any installation of cathodic protection should be carefully evaluated by professionals experienced in the protection of concrete pressure pipe, particularly for prestressed concrete cylinder pipe. Each cathodic protection system must be carefully evaluated to ensure variations in local conditions do not cause overprotection or underprotection along a pipeline's length. Continued operation of an improperly designed or operated cathodic protection system could cause or accelerate deterioration of the pipe.

Even damaged and corroding concrete cylinder pipelines have retained passivation by mortar or concrete of the vast majority of their steel surfaces. Consequently, the current density requirements for arresting corrosion on a concrete pipeline are far less than would be needed if it were assumed all steel surfaces were bare. To achieve the minimum 100 mV shift, typical current density requirements for concrete pressure pipe range from 10 to 100 microamperes/ft$^2$. For close approximations, experience has shown that the surface area of the steel cylinder can be used to determine current requirements for ANSI/AWWA C301 embedded-cylinder pipe, as well as for ANSI/AWWA C300, C303, and C301 lined-cylinder pipe.

For the limited conditions when cathodic protection is required for concrete pipe, the sacrificial galvanic anode system is usually the most economical when considering the total life of systems. The sacrificial anode system has several advantages over an impressed current system, including the following:

- It is less likely to cause stray current interference problems on nearby structures.
- It is less likely to cause hydrogen evolution.
- It requires less frequent monitoring, maintenance, and adjustment of the system.

These advantages are more significant in developed or developing urban areas than in rural areas. For some pipeline projects, conditions may indicate that there are benefits from the application of an impressed current system. This type of system requires continuous surveillance by trained professionals to keep the system operating safely and efficiently throughout the life of the pipeline. In most cases, galvanic anode systems are safer, more economical, and more reliable than an impressed current system.

## REFERENCES


American Concrete Pressure Pipe Association. *External Protection of Concrete Cylinder Pipe*. Reston, Va.

Benedict, R.L. 1989. Corrosion Protection of Concrete Cylinder Pipe. CORROSION/89, paper No. 368. Houston, Texas, New Orleans, La.

———. 1985. Corrosion Protective Properties of Concrete Cylinder Pipe. In *Proceedings of the AWWA Seminar on Corrosion Control* (September 1985).

Hall, S.C. Cathodic Protection Criteria for Prestressed Concrete Pipe—An Update, Materials Performance, V. 37, No. 11, p. 14.

Hall, S.C., and I. Matthews. Cathodic Protection Requirements for Prestressed Concrete Cylinder Pipe. *Advances in Underground Pipeline Engineering.* From the Proceedings of the Second International Conference Pipeline Division and TCLEE/ASCE (June 1995).

Hausmann, D.A. 1967. Steel Corrosion in Concrete. *Materials Protection*, 6:11:19.

Lea, F.M. 1970. *The Chemistry of Cement and Concrete.* 3rd ed. Great Britain: Edward Arnold, Ltd.

Lewis, R.O. Improving Prestsressing Wire Reliability in Prestressed Concrete Cylinder Pipe. *NACE Materials Performance*, July 2002.

NACE International. 2004. Cathodic Protection of Prestressed Concrete Cylinder Pipelines, RPO 0100-2000. Houston, Texas.

Peabody, A.W. 2001. *Control of Pipeline Corrosion.* 2nd Edition. Houston, Texas: National Association of Corrosion Engineers.

Scott, G.N. 1965. Corrosion Protection Properties of Portland Cement Concrete. *Jour. AWWA*, 57(8):1038.

Speller, F. N. 1951. *Corrosion Causes and Prevention*. 3rd edition. New York: McGraw-Hill Book Company.

Turnipseed, S.P., and R.I. Mueller. Cathodic Protection Retrofit of a 35-Year-Old Concrete Pressure Pipeline. *Advances in Underground Pipeline Engineering.* From the Proceedings of the Second International Conference Pipeline Division and TCLEE/ASCE (June 1995).

Unz, M. 1960. Cathodic Protection of Prestressed Concrete Pipe. *Corrosion*, v. 16 (June 1960).

Verbeck, G.J. 1967. Field and Laboratory Studies of the Sulfate Resistance of Concrete. Bulletin 227. Portland Cement Association.

Willett, J.A. 1981. Concrete Pressure Pipe Performance in Acid Soils. *Jour. AWWA*, 73(9):503–504.

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*Chapter* 13

# Transportation of Pipe

Concrete pressure pipe is generally transported from the manufacturer's yard to the area of installation by flatbed trailer, low-bed trailer, or rail flatcar (Figure 13-1). Railroad piggyback and barge transportation are also used. The method of hauling depends on comparative shipping costs based primarily on the distance involved, weight and size of the pipe, availability of transportation equipment, and receiving capability at the jobsite.

Although concrete pressure pipe is a rugged product, the pipe must be properly supported and secured to prevent damage during transit. Reasonable care should also be exercised to prevent bumping and possible damage to the pipe ends.

## TRUCK TRANSPORTATION

For distances up to 300 mi (483 km), trucking is generally the most efficient way to haul pipe (Figure 13-2). Most manufacturers offer delivery along the trench when access roads are sufficient for highway trucks to operate under their own power. The hauling capacity per truckload is controlled by governmental highway regulations regarding the maximum size and weight of the loaded vehicle and the physical operating conditions over the route to be traveled, such as vertical clearance at overpasses, road width, degree of curvature, or bridge rating.

A semitrailer truckload that does not exceed 80,000 lb (36,280 kg) in gross weight, 8½ ft (2.60 m) in width, 13½ ft (4.11 m) in height above the road, and 48 ft (14.6 m) in trailer length can be operated over most highways. The deck of a typical flatbed semitrailer is about 5 ft (1.5 m) above the road. Highway departments will usually issue special permits to allow hauling a load that exceeds published regulations if (1) the load is nondivisible (one piece), (2) the truck provides adequate weight distribution on the highway, and (3) the truckload can clear obstructions along the route of movement.

## RAIL TRANSPORTATION

Rail transportation is normally used when some of the following conditions exist: (1) shipping distances are over 300 mi (483 km), (2) load dimensions exceed normal highway allowance, (3) delivery is to a storage site served by railroad, or (4) delivery is to a trench site adjacent to a railroad line.

Transportation by railroad is also limited by weight and obstruction clearance restrictions. Flat cars are commonly used for railway delivery (Figure 13-3). The

Figure 13-1 Unloading a typical flatbed truck trailer loaded with 36-in. (910-mm) concrete pressure pipe

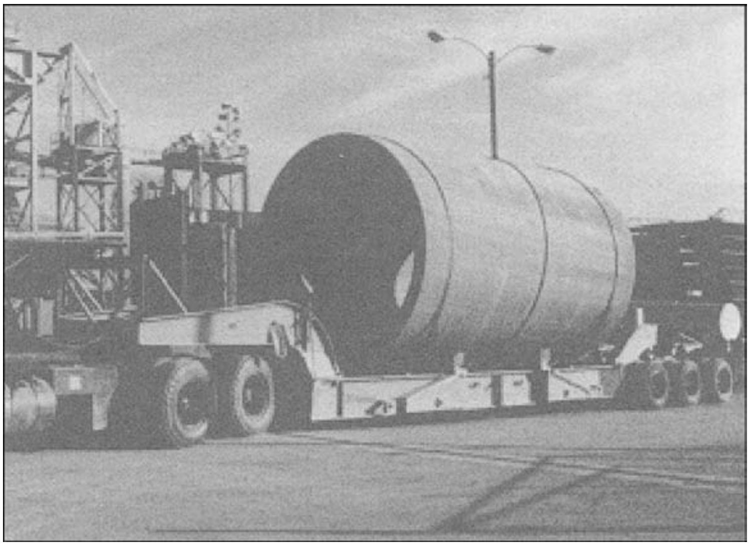


Figure 13-2 Special double-drop highway truck trailer loaded with a single large-diameter pipe

normal capacity of flat cars depends on the size of pipe and ranges from 20 or more pieces of 16-in. (410-mm) diameter pipe to one piece of 144-in. (3,660-mm) pipe (Figure 13-4). Dimensional clearances are variable, with usual maximum load widths of 11½ ft (3.50 m) and load heights of 17 ft (5.18 m) above the rail. Carloads that exceed normal limits must have approval of the originating railroad.

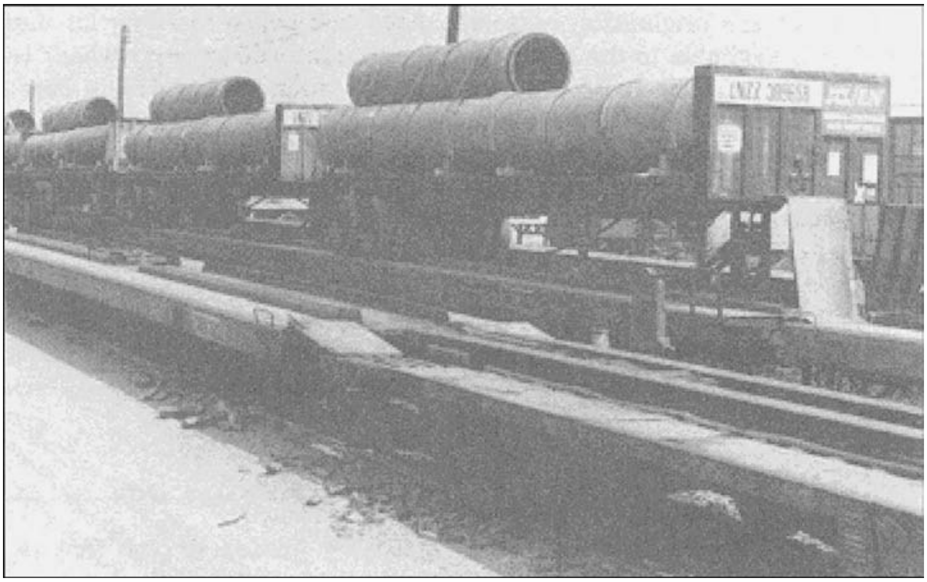


Figure 13-3 Flatbed piggyback truck trailers loaded with small-diameter pipe

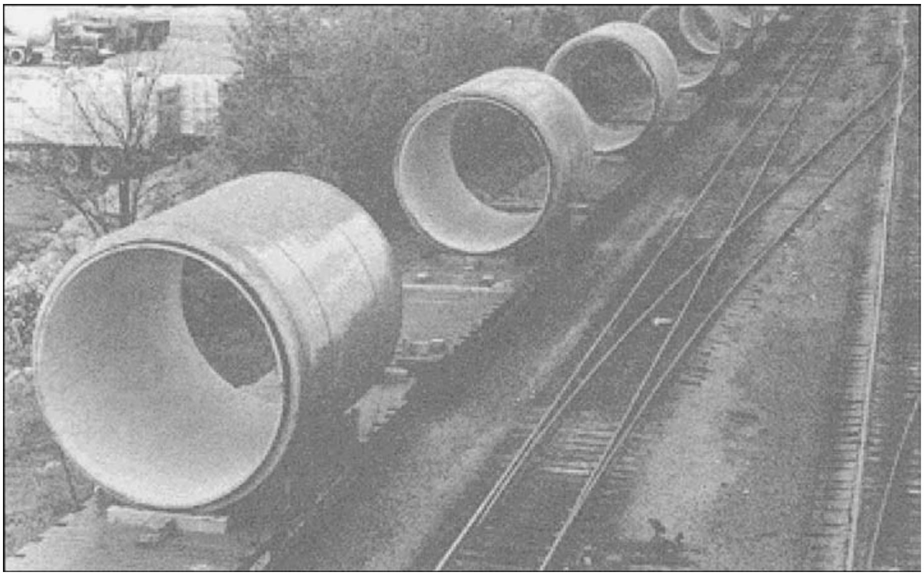


Figure 13-4 Rail flat cars loaded with 144-in. (3,660-mm) diameter concrete pressure pipe

## Railroad Piggyback

Railroad piggyback provides some of the long-distance economies of railroad transportation plus the convenience of truck delivery. The pipe is loaded on trailers at the manufacturer's plant and the trailers are then pulled by tractor to the nearest railroad piggyback ramp for loading onto special railroad flat cars.

The most common method of piggyback in the United States is "Plan 2½," under which the railroads are only responsible for the pipe from the time the loaded trailer arrives at the originating railroad's piggyback trailer parking lot until the loaded trailer is available to the hauler at the terminal railroad's piggyback trailer parking lot. In Canada the most common mode is "Plan 20," which is door to door. The railroad companies will only pay transportation damage claims if they are proven negligent. Arrangements for transporting the empty trailer from the railroad yard to the manufacturer's yard are usually made by the manufacturer. Transporting the loaded trailer from the terminal railroad yard to the delivery point, returning the empty trailer to the railroad yard, and any applicable detention charges on the trailer are usually the responsibility of the purchaser.

Piggyback transportation has about the same general weight and dimensional restrictions as trucks. However, low-bed and other special trailers are rarely available, and overdimension or overweight shipments are seldom accepted by the railroads.

## BARGE TRANSPORTATION

Barge transportation (Figure 13-5) is usually limited to pipe that is too large for feasible rail shipment, to jobsites that can only be reached by water, or to projects in which the volume and shipping distance produce favorable economies when compared to other modes of transportation. Barges range from harbor scows to oceangoing converted ships. Load capacity varies widely.

## LOADING PROCEDURES

Loading procedures vary according to the size of pipe involved, the handling method used, and the type of vehicle to be loaded. Most pipe is shipped on flatbed vehicles and is loaded with forklifts if the weight per piece is under 65,000 lb (29,484 kg). Larger pipe is generally loaded by cranes. Cranes are also used when loading barges.

In the United States, close attention must be paid to adequate blocking and bracing of pipe in conformity to rules prescribed by the US Department of Transportation and the American Association of Railroads. Timbers are generally used for pipe cradles and spacers. The pipe must be secured with chains, cable, or steel bands.

## DELIVERY AND UNLOADING

It is important that the manufacturer and purchaser agree on a definite delivery location prior to the shipment of pipe. The location may be by freight-on-board carrier at the manufacturer's plant or at points of delivery. Trench-side delivery may specify unloading and stringing of pipe by the supplier, the contractor, or the manufacturer. In any event, the manufacturer is responsible for providing pipe in satisfactory condition in accordance with the delivery terms.

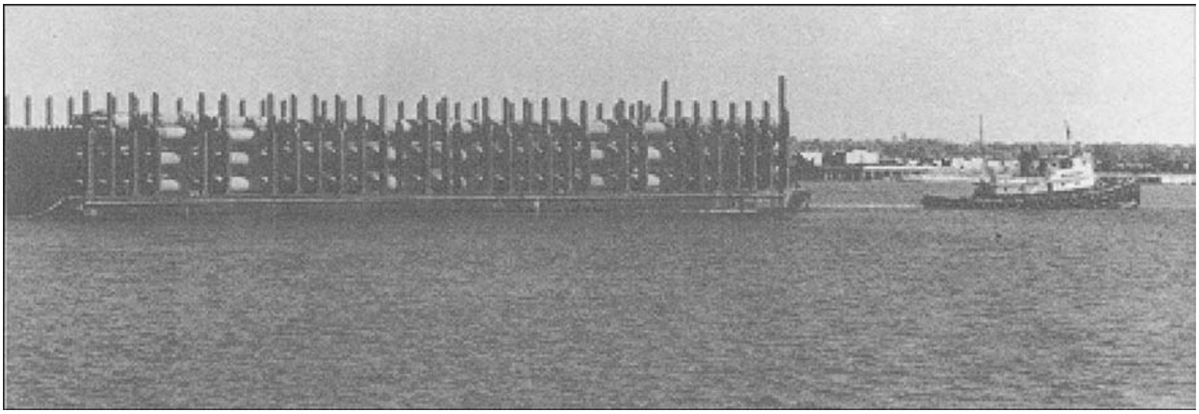


Figure 13-5 Oceangoing barge carrying approximately 18,000 ft (5,486 m) of concrete pressure pipe

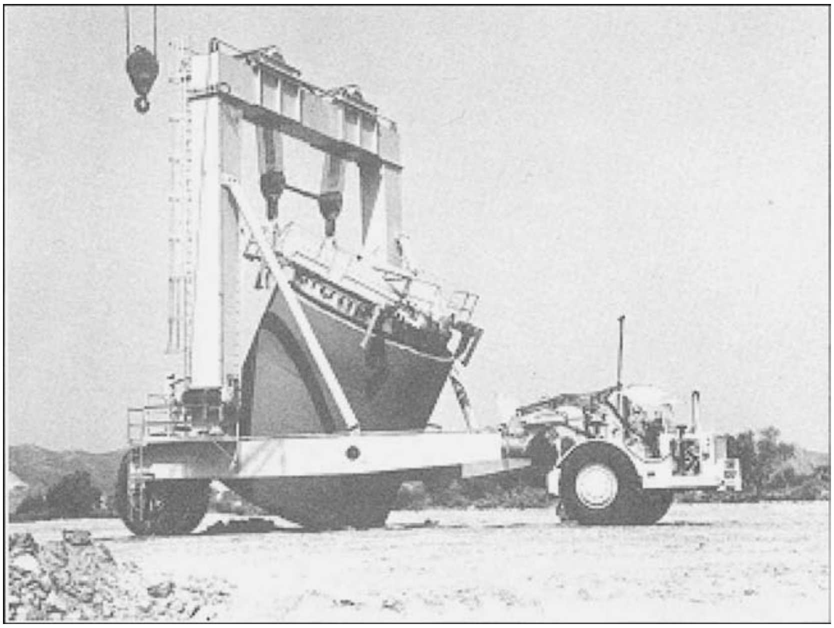


Figure 13-6 Specialized equipment for handling and transporting 252-in. (6,400-mm) (ID) pipe weighing 225 tons (204 metric tons) each

It is the purchaser's responsibility to inspect shipments at points of delivery. If the material is damaged, the purchaser must notify the carrier immediately and request a written damage-inspection report. The manufacturer must also be notified of the damage. According to interstate commerce law, the consignee cannot refuse to accept a damaged shipment, but, if the damage is extensive, the carrier will usually agree not to unload it. Handling of pipe beyond the agreed point of delivery is the purchaser's responsibility.

The equipment for unloading concrete pressure pipe varies with the method of transportation and the facilities at the destination. Whether or not the pipe has to be unloaded at a siding, strung along the ditch line, or stockpiled should also be considered.

With few exceptions, cranes of the proper capacity using steel cable, belt slings, or specially designed devices can load or unload any size of pipe regardless of the mode of transportation. Forklift trucks and front-end loaders that have sufficient capacity can unload pipe from most carriers if the delivery location is accessible and if the terrain is suitable for operating the equipment. Front-end loaders should have the bucket replaced with forks or have forks installed on the lip of the bucket. A choke cable attached to the bucket is also frequently used. The uprights of the forks and the sides of the bucket should be cushioned with suitable material to prevent damage to the pipe. Special equipment, such as the "pipemobile" shown in Figure 13-6, can be designed for handling and assembling large-diameter pipe.

When multiple-level stacking is necessary for jobsite storage, the pipe manufacturer should be contacted for recommendations that will prevent possible damage to the pipe. The contractor must also take steps to prevent handling damage during storage and installation.

*Chapter* 14

# Installation by Trenching or Tunneling—Methods and Equipment

This chapter covers the common practices and requirements for installing concrete pressure pipelines. Normal practices and standard equipment are discussed, and brief descriptions of special equipment and methods that have been used in the installation of very-large-diameter pipe and tunnel liners are also presented. The actual project requirements depend on several controlling factors that are discussed briefly in the following sections.

Specific details of trenching, laying, and backfilling can only be determined after considering the design and type of pipe to be used. The installation methods and equipment selected by the contractor should be based on the actual requirements and conditions of the project.

## TRENCHING—GENERAL CONSIDERATIONS

The trenching operation is a major item in the overall planning of a pipeline installation project. Many potential problems for both the contractor and the owner can be avoided if there is adequate planning prior to excavation. It is necessary to recognize that the engineering properties of the soils to be excavated and those to be used as backfill are very important. In the initial stages of planning, the right-of-way width should be determined by considering the size of pipe, depth of burial, type of material, safe slopes for the trench walls, and any special requirements, such as separation and stockpiling of top soil. Proper application of the principles of soil mechanics to both trenching and backfilling will lead to a safe, high quality, and economical pipeline installation.

### Planning of Trenching Procedures

Preconstruction conferences between the contractor and the project engineer should include in-depth discussions of the contractor's proposed trenching, pipe laying, and backfilling operations to be sure that they are compatible with the contract requirements and specifications. Differences can be settled more easily before construction begins. A meeting with the pipe manufacturer at this time is of equal importance so that manufacturing and delivery can be coordinated with the installation schedule. Special pipe or fittings generally require more lead time to fabricate, and delivery schedules of these items should be agreed on as soon as possible.

Prior to the start of any site work, the contractor should establish and lay out rights-of-way, work and storage areas, access roads, detours, protective fences or barricades, and pavement cuts. The contractor must also consider provisions for adequate access to businesses and residential properties.

## Occupational Health and Safety Act

Safety measures have always been important to protect both workers and the public, and the subject of safety has gained increasing attention in the United States since the advent of the Occupational Safety and Health Act. A project can be interrupted, or even shut down, for what may seem to be a minor infraction of the safety act. It is imperative that responsibility for safety be assigned to an authoritative person who has full knowledge of the rules, regulations, and requirements of federal, state, and local agencies.

## Construction Survey

A thorough reconnaissance survey of the entire right-of-way should be made to eliminate potential problems during construction. The size and operating space requirements of equipment, the conditions of streets, the intensity of traffic, and the location of trees and overhead lines should be noted and considered. Photographs and video of pertinent areas are useful and time saving, and they can be a valuable resource later in the project if questions are raised regarding conditions that existed prior to construction. Substructures and utilities must be located and planned for in advance. Public records and records of utility companies can supply this information.

Surveys to accurately establish the horizontal alignment and profile for the pipeline route, with adequate cross sections used where necessary, should be made on all portions of the route. When data from other sources are used, survey ties should be checked. Traverses should be closed for horizontal control, and benchmarks checked for vertical control. Concrete pressure pipelines are manufactured to fit the plan and profile. Variances between recorded and existing distances, either vertical or horizontal, can cause unnecessary difficulty, delay, and expense.

The transfer of line and grade to the excavation work from control points established by the engineer is the responsibility of the contractor. Care should be taken to preserve these references because, generally, a charge will be made for reestablishing grade stakes that are carelessly destroyed. The line and grade may be transferred to the bottom of the trench by using batter boards, tape and level, patented tape and plumb-bob units, or laser beams. Batter boards and batter-board supports must be suspended firmly across the trench and span the excavation without measurable deflection. Another method of setting grade is to set from offset stakes or offset batter boards and double string lines with the use of a grade rod that has a target near the top. When the pipe invert is on grade, a sighting between the grade rod and two or more consecutive offset bars or the double string line should show correct alignment. A third method, usually used on larger pipe and flatter grades, requires the line and grade for each length of pipe be set by means of a transit and level from either on top of or inside the completed part of the pipeline. Laser beams are also used.

### Site Preparation

The amount of site preparation required varies. Under some conditions, no preparation will be required; under other conditions, preparation can be a major item of the project cost. In some cases, site preparation can be considerable even in rural or undeveloped areas.

Operations included in site preparation are clearing trees, brush, or other plant growth; removing rocks or unsuitable soils; constructing access roads and detours; relocating existing sanitary and drainage facilities; and protecting or relocating existing utilities and services. The extent and diversity of this list is obvious; however, it should be stressed that successfully keeping the project on schedule depends on the thoroughness of planning and timely execution of the site preparation work.

## OPEN TRENCH CONSTRUCTION

The specific details of trenching, laying, backfilling, and testing any pipeline depend on several factors, including the type and purpose of the line; its size; operating pressure; whether it is an urban, suburban, or rural location; and the type of terrain in which it is to be laid.

The details are also affected by the depth of the trench and, as previously mentioned, the type and classification of the native soil and its suitability for use as backfill material.

### Trench Dimensions

The trench at and below the top of the pipe should be only as wide as necessary for the proper installation and backfill of the pipe. Because of load considerations (discussed in chapter 5), the depth and width of the trench should be as shown on the plan and profile or as specified by the project engineer. If these dimensions are exceeded, improved bedding may be required. The effect of the trench width from the top of the pipe to the ground surface on safety, adjoining facilities, and surface conditions must be considered.

In open country, economic considerations often justify sloping the sides of the trench above the top of the pipe to the ground surface. This may eliminate substantial amounts of sheeting and bracing, unless safety regulations require it. In improved or paved streets, it may be desirable to restrict the trench width at the ground surface to reduce cutting of pavement and restoration costs.

### Excavation

The trench should be dug to approximate grade with sufficient width to permit proper placement of backfill material around the pipe.

With favorable ground conditions, excavation can be accomplished in one operation; under more adverse conditions it may require several steps. The trench bottom should receive careful attention and adequate provisions for maintaining grade. Pipe foundations are discussed in more detail later in this chapter.

### Excavating Machines and Methods

The trench excavating equipment used varies with soil conditions, trench depth, terrain, and the contractor's preference. The more common types are backhoes, draglines, clamshells, power shovels, front-end loaders, and rotary trenching machines.

Backhoes (Figure 14-1) are used extensively in excavating trenches with widths exceeding 2 ft (0.6 m) and depths up to 25 ft (7.6 m). They can be used in relatively restricted areas and in nearly all types of soil. The backhoe is also frequently used with a cable sling for lowering pipe into the trench.

Figure 14-1 Backhoe laying pipe

In open country or in a wide right-of-way, a dragline can be used. In cases of very deep trench excavation (30 to 50 ft [9.1 to 15.2 m]), a dragline can be used for the upper part of the excavation, and a backhoe operating at an intermediate level can relay material to the dragline for removal to the spoil bank or trucks at the surface.

A clamshell is best employed when soil conditions or underground structures restrict the use of other types of machines (Figure 14-2). In wet or unstable soil that requires sheet piling, the use of such vertical-lift equipment as the clamshell allows the bracing and sheet piling to be installed as required.

In wide, deep trenches, a front-end loader can be used as an auxiliary to a backhoe or dragline. The backhoe or dragline can be used to excavate the upper part of the trench, leaving a bottom bench for a front-end loader to place its spoil within reach of the backhoe or dragline.

Trenching machines are used to excavate trenches of moderate depth and width in light, cohesive soils. Under such conditions, a trenching machine can make rapid progress at relatively low cost. Trenching machines can also be used to excavate moderately hard rock, but progress will be much slower. In harder rock, trenching machines or rock saws with carbide pointed teeth can be used to cut or saw the trench.

## Sheeting and Bracing

The primary function of trench sheeting and bracing is to prevent a cave-in of the trench walls or areas adjacent to the trench. Responsibility for the adequacy of any required sheeting and bracing is usually accepted by the contractor. If sheeting is to be removed, consideration must be given to additional loads that may be transferred to the pipe (see chapter 5). The design of the system of supports should be based on sound engineering principles of soil mechanics and the materials to be used, and the design must comply with applicable safety requirements.

In trenches with relatively stable soil, skeleton sheeting consisting of pairs of planks set vertically with spreaders or trench jacks may be used. Horizontal spacing will vary with soil conditions.

Continuous sheeting involves the use of panels prefabricated with plank stiffeners. The panels are set vertically, with spreaders or trench jacks spaced as required. For wider and deeper trenches or for more unstable soil, a system of wales and cross struts of heavy timber is often used. The horizontal wales are used to distribute the pressure from the vertical sheeting to the cross struts.

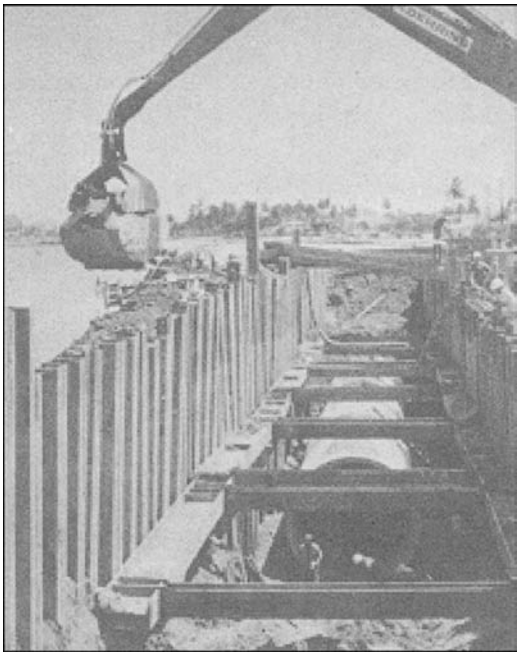


Figure 14-2 Clamshell excavating inside sheeting

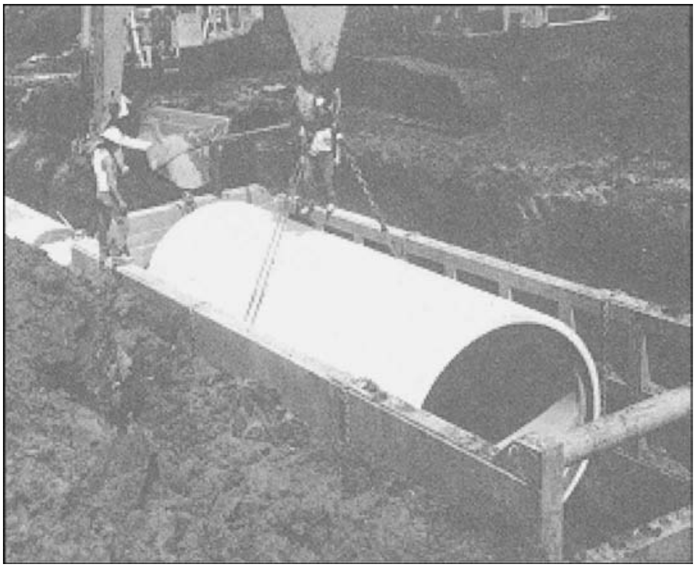


Figure 14-3 Lowering 84-in. (2,140-mm) pipe to final bedding position in laying shield

In some soil conditions it is economical and practical to use a prefabricated unit that is at least as long as one section of pipe. The units are called *laying shields* or *trench shields* (Figure 14-3) and are pulled forward as the trenching and pipelaying progresses. Although they do not support the trench walls, they do protect workers from sloughs and cave-ins. Trench shields should be used only when the trench width is not critical and the shield is high enough and of sufficient strength to protect the workers from sloughing soil.

In noncohesive soils combined with groundwater, it may be necessary to use continuous steel sheet piling to prevent soil movement. Steel sheet piling can be installed so that it is relatively watertight, and, if necessary, dewatering with trench-bottom sump pumps can be undertaken.

In limited reaches of pipelines, it is often necessary to make special provisions for supporting or bracing a trench. Excavations next to existing structures may be subject to surcharge loads. In such cases, the trench bracing or supports must be adequate to withstand these loads, not only to prevent cave-ins, but also to prevent any movement or possible damage to the structure itself.

### Dewatering

If not planned for, groundwater can be a serious problem during excavation and pipe laying. It is a condition that the contractor should be aware of prior to bidding the project.

Where feasible, the trench should be dewatered until the pipe has been installed with the prescribed bedding and backfill has been placed to a height at least above the groundwater level. Dewatering a large amount of groundwater, particularly where the water creates an unstable soil condition, may require the use of a well-point system consisting of a series of perforated pipes driven into the water-bearing strata and connected to a header pipe and pump. If trenching is necessary in coarse water-bearing material, turbine well pumps can be used to lower the water table during construction. For dewatering lesser volumes of water, the trench may be overexcavated and backfilled to grade with crushed stone or gravel (Figure 14-4) to facilitate drainage of water to the point of removal.

Storm drains or adjacent water courses, when present, can be used to dispose of the water. In some cases, it may be possible to drain the water through the completed portion of the pipeline to a point of satisfactory discharge. However, if the completed pipeline is to be used for potable water, care should be taken to minimize the size and volume of material that enters the pipeline, as residual materials that are not easily flushed from the pipeline may complicate disinfection (see chapter 15).

## BEDDING

The bottom of the trench should be accurately graded and bell holes dug to accommodate pipe joints with projecting bells. The pipe barrel should be in contact with the trench bottom for its full length to ensure uniform bearing of the pipe. Although sometimes performed, careful trimming and shaping of the bottom to fit the pipe barrel is costly. Unless a special shaping machine is used, it can be very difficult to accomplish the desired degree of accuracy. Because it is both practical and economical, overexcavation followed by backfilling to pipeline grade with granular bedding material is increasingly being used to provide uniform bedding. In many soils, this cushion under the invert can also be achieved by raking the trench bottom with the backhoe teeth. When a soft trench bottom is encountered, overexcavating and backfilling with granular material may be necessary for stabilization with the material being placed thick enough to prevent upward movement of the soft subgrade into the bedding material. The required depth of stabilizing material should be determined by tests and observation.

In cases where a trench bottom cannot be stabilized with the bedding material and where intermittent areas of unequal settlement are anticipated, special foundations for the pipe may be necessary.

If the trench bottom is solid rock, it must be overexcavated to make room for bedding material that will support the pipe uniformly (Figure 14-5). The trench bottom should be cleared of all loose or projecting rocks prior to placement of the bedding material.


Figure 14-4 Pipe being laid on granular foundation

Figure 14-5 Pipe bedding

The pipe design and its required supporting strength are based on specific bedding and backfill conditions. Therefore, it is imperative that the entire pipeline be installed with the exact bedding specified in the contract documents. A detailed definition of the types of pipe beddings for both rigid and semirigid pipe is presented in chapter 6.

## PIPE INSTALLATION—GENERAL

Proper and careful installation of concrete pressure pipe is of benefit to both the owner of the pipeline and the installing contractor. Acceptance of pressure pipelines should be based on a performance test.

Installation procedures for the several types of concrete pressure pipe, although basically similar, may have different requirements in order to attain successful installation. The pipe manufacturer should provide the installer with this information and instructions for the proper method of laying pipe.

### Pipe Delivery

Concrete pressure pipe is normally not a stock item, so it is common for fabrication and installation to be carried on simultaneously. Therefore, delivery schedules should be resolved by the manufacturer and the contractor. When changes are required, the pipe manufacturer should be notified as soon as possible so that the production schedule can be modified.

### Jobsite Storage

Pipe can be stored or stocked at the jobsite prior to the start of laying. Pipe delivery can be scheduled to coincide with the installation, and normally the pipe is strung along the right-of-way in the proper sequence for laying. Whether it is delivered directly to the jobsite or placed in temporary storage areas, care should be taken to place the pipe so it can be reached for movement to the trench with as little extra handling as possible. Also, every precaution should be taken to prevent damage to the pipe. Pipe ends are particularly vulnerable to damage from impact or point loading that may result from contact with construction equipment, rocks, or other obstacles on the ground.

### Pipe Handling

At all times, pipe should be handled with equipment designed to prevent damage to the inside and outside surface of the pipe.

## LAYING THE PIPE

If a special pipe bedding is required in the bottom of the trench before the pipe is laid, it will be described in the job specifications. Normal installation is made on a bedding prepared to line and grade (such as those shown in Figure 6-1), which is free of rocks or other potentially damaging materials. Uniform bearing should be provided for the full length of the pipe barrel.

### Equipment

The equipment used to place the pipe in the trench depends on the size, weight, and length of the pipe sections. The most versatile and the most frequently used machines are backhoes. The size of the backhoe used depends on the size of the pipe to be laid and the depth of the excavation. Backhoes come in a variety of sizes, and the larger ones are capable of handling a wide range of pipe sizes and weights. Other types of equipment used for installing pipe include cranes and, for small to intermediate size pipe, sideboom tractors. For very large and heavy pipe sections, equipment specially designed for laying and jointing the pipe may be required.

### Making the Joint

Pipe should be supported free of the bedding or foundation during the jointing process. A suitable excavation should be made in the trench bottom to receive pipe with raised bells or to provide the necessary clearance required for grouting the exterior joint space. Any adjustments required to maintain grade should be made by scraping away or adding bedding material. Pipe must never be struck with the excavating bucket or other equipment to drive the pipe to grade. Such impact loads will damage the exterior mortar coating and may result in eventual pipe failure.

Prior to jointing, both the bell and the spigot should be clean and free of any dirt or mud, and a thin layer of an approved commercial vegetable-type lubricant should be applied to the face of the bell, the gasket groove of the spigot, and the gasket

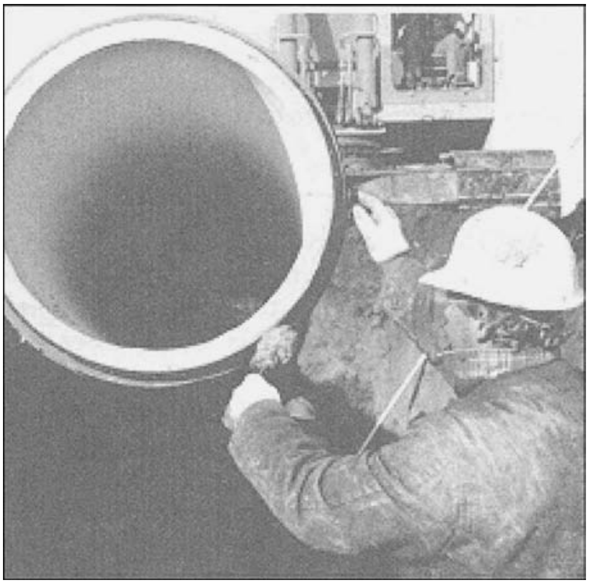


Figure 14-6 Applying lubricant to the gasket prior to installation

(Figure 14-6). Petroleum-based lubricants should never be used as they may damage the gasket and are generally not acceptable for use in potable water systems. Rubber gaskets are designed and sized to provide the correct volume of rubber required to fill the groove and the annular space between the bell and spigot of the completed joint. This volume is based on a predetermined, stretched diameter of the gasket. For this reason, it is important that the stretch of the gasket be equalized around the entire circumference just prior to making the joint. This can be accomplished by inserting a round rod under the gasket after it has been placed in the spigot groove and moving the rod around the full circumference.

## Pulling the Pipe Home

Because of the close tolerances in concrete pressure pipe joints and the compression of the rubber gasket, considerable force is required to engage the joint. Come-alongs and power winches may be rigged in larger pipe to provide this force. However, for all but very large pipe, a backhoe or other laying equipment is generally used to force the pipe together by applying pressure to the wire rope choker used to lay the pipe. It is important to note that the joint should be made using a straight axial force. Raising the far end of the pipe so the top of the spigot is inserted first, and then lowering the pipe to insert the bottom half, may result in a displaced gasket and a leaking joint.

## Checking the Completed Joint

The joint should be checked as soon as it is completed by inserting a steel feeler gauge approximately 0.5 in. (12.7 mm) wide and 0.010 in. (0.25 mm) thick into the joint to determine, by feel, if the gasket is properly seated in the groove. Temporarily placing properly sized spacer blocks between the joint ends during laying allows access so that the feeler gauge can easily be inserted for the gasket check. To ensure a leak-free joint, the gasket should be checked around the full circumference of the joint. If the gasket is found to be out of place, the joint must be remade. On large pipe, this type of checking is more easily done from the inside of the pipe; on smaller sizes, it can be accomplished from the outside. After each gasket is checked for a proper seal, these temporary spacer blocks are removed and the pipe is pushed home to the proper line and grade. The final joint separation should then be checked to ensure it is within specified limits. Joint openings exceeding the maximum must be adjusted or relayed.

## Interior Joint Protection

The exposed surfaces of steel joint rings may be protected by metalizing, by other approved coatings, or by pointing with portland cement mortar. In some areas, pipelines carrying potable water are not pointed with cement mortar if the joint rings are protected with zinc metalizing or other protective coatings. For pipe large enough to accommodate a worker, pointing the joint space with cement mortar is normally performed with a hand pointing trowel. Before mortar is applied, the joint space should be clean and the concrete surface should be wetted. The cement-to-sand ratio for the mortar mix should be one part cement to no more than three parts sand, and the mortar consistency should be dry enough so that it will not fall out when placed in the top of the joint. Pointing of the inside joint space in small-diameter pipe can be accomplished by "buttering" the back face of the bell with the mortar just before the spigot is inserted.

## Grouting the Exterior Joint Space

Grouting the exterior joint space is accomplished by using a grout band placed around the pipe and held in place on either side of the joint with steel straps (Figure 14-7). The ends of the grout band are pulled together near the top of the pipe so the access hole for pouring allows the portland cement grout to be poured down one side and rise on the other. The grout should be one part cement to not more than three parts sand and should be wet enough to flow all the way around the joint when it is poured into the opening in the grout band near the top of the pipe.

The grout should be placed by filling the band from one side only until the grout rises on the opposite side, and the grout should be rodded or agitated so that all voids are filled. The grout band should be filled completely and then rodded or agitated on both sides of the pipe alternately to settle the grout. After the band is filled and rodded or agitated, the grout should then be left undisturbed for at least 15 minutes to allow it to mechanically stiffen and, when permeable grout bands are used, to allow excess water to seep through the band. After this period, more grout should be added if necessary to refill the joint completely. The gap at the top of the grout band should be protected from penetration of backfill into the grout by allowing the grout to stiffen, by capping with a stiff mortar mix, or by covering with a structurally protective material. The band should not be removed from the joint.

The exterior joint for raised-bell (lined cylinder ANSI/AWWA C301-type or ANSI/AWWA C303-type) pipe should be completed and the grout allowed to mechanically stiffen before any bedding or backfill material is placed above the bottom of the pipe. This prevents the backfill soil pressure from pressing against the empty grout band, which would result in reduced grout cover for the joint-ring steel.

The deeper exterior joint recess for all other (flush bell) pipe may be filled in one or more lifts after the initial grout is poured from one side only and rises some height above the bottom of the pipe on the opposite side. A temporary reinforcement may be placed around the band before the grout is placed, or bedding and backfill may be placed and consolidated below the top of each lift after the lift is rodded and agitated, to provide support for the band to contain the weight of the grout.

# BACKFILLING

Backfilling usually begins a few lengths behind pipe installation. Good backfill procedures are important in the installation of all kinds of pipe, but are particularly

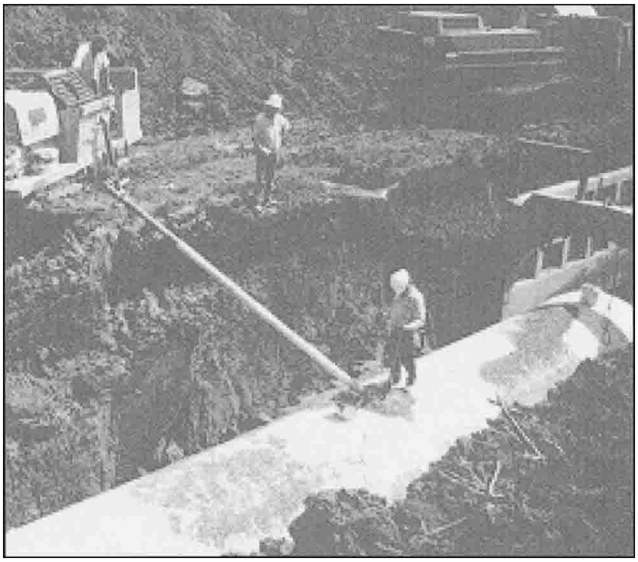


Figure 14-7 Grouting a pipe joint

important for semirigid pipe. This is because the load-carrying capabilities of the pipe can only be realized if the pipe is uniformly bedded and supported and the load is uniformly distributed along the sides and invert of the pipe for the entire length. Rigid or heavy-walled pipe, although not as dependent on good backfill consolidation for its load-carrying capabilities, must be uniformly supported along its bottom and under the haunches to prevent settling or movement.

## Backfill Material

Most project specifications indicate backfill material requirements. When soil excavated from the trench can be used for backfill, the economic advantages are obvious. But debris, large rocks, or organic material should be removed. Solid objects or rocks that exceed 3 in. (76 mm) in diameter should not be allowed in backfill placed within 6 in. (152 mm) of the pipe. In general, all backfill material must be free of rocks, tree stumps, broken pavement, or other unyielding solid objects. Unsuitable backfill material can result in voids around the pipe and subject it to irregular settling and the concentration of unusual forces for which it was not designed. If clean material is not available from the excavation, it should be imported.

A minimum cover of fill should be specified to protect pipe from the effects of heavy trucks and construction equipment. Where pipe will be located below pavement, the soil should be compacted to the top of the trench to prevent settlement.

Sometimes flowable fill is used in place of compacted soil. Flowable fill is a mixture of soil, water, and cement and sometimes fly ash or other additives. In pipeline construction, it can be used as the embedment, the backfill, or both. For both rigid and flexible pipe, flowable fill is placed to the same level as for compacted soil. Care is needed to prevent flotation or misalignment of the pipe (Howard and McGrath 1998).

## Placing the Backfill

Backfill can be placed with front-end loaders, bulldozers, or other equipment (Figure 14-8). To prevent shifting or side movement of the pipe, the initial backfill should be placed uniformly on both sides of the pipe.

Figure 14-8 Backfilling

## Backfill Densification

When densification is required, it is commonly accomplished by (1) using water in a flooding or jetting operation, or (2) compaction using air tools, vibrating compactors, or other equipment. The method to be used depends on the type of soil encountered and the requirements of the specifications.

For porous, self-draining soils, such as sand or gravel, the best and fastest method is consolidation with water if available. In such consolidation, it is important for all of the material to be wetted, but care should be taken not to float the pipe. If flooding is used, the fill should either be flooded in successive layers or be hosed continually as it is placed in the trench. If jetting is used, jet pipes should be inserted and worked up and down all the way to the trench bottom. Removal of jet pipes should be done slowly to prevent voids.

For tight, nondraining soils, such as clay or other impervious soils, consolidation of the backfill with water should not be attempted. Water will not drain off and the backfill could take weeks to dry out sufficiently to support the pipe properly. In these types of soils, densification should be accomplished with mechanical equipment by compacting backfill in 6- to 12-in. (150- to 305-mm) layers. Care should be taken to ensure that the power tamping equipment does not damage the pipe.

# TUNNEL INSTALLATIONS

Various construction methods are used for placing pipe in tunnels, depending on the pressure, diameter, length of line, and soils encountered. A pipeline can be installed inside a primary liner or as a solitary pipe forced through the earth by large hydraulic jacks. Each of these methods will be briefly described and their major features presented in the following sections.

Gravity flow pipelines are often jacked into place through a tunnel. The process involves excavating a tunnel with a tunnel boring machine (TBM) or other earth mining machinery. Then, as the excavation proceeds, the pipe is pushed into, and through, the tunnel with jacks. Internal earth augers are frequently used to excavate in front of smaller-diameter pipe. But all pipe and pipe materials, even concrete pressure pipe, are subject to jacking operation hazards, including scouring of the exterior of the pipe while being forced through the earth and damaged joints from eccentric or excessive jacking force.

For pressure pipelines, it is good practice to place pipelines of any material within a primary liner or casing. The primary liner may be a concrete pipe, concrete liner segments, or other materials installed by jacking or tunneling methods.

Each project requires individual evaluation by experienced engineers to determine the appropriate installation method to use. It is always important to find out as much as possible about the soil along the pipe route from boring samples. Large boulders or other unexpected obstructions are typical hazards when jacking pipe or tunneling and plans for overcoming hazards should be made before the operation begins. The following sections present general information about jacking and tunneling methods, followed by a description of the procedures that should be followed when grouting tunnels.

## JACKING METHODS

Jacking installations incorporate the use of large hydraulic jacks to push pipe as the heading excavation progresses (Figure 14-9). An approach trench or jacking pit is excavated at the starting point to accommodate the backstop, jacks, and pushing frame, as well as to introduce the jack pipe. A circular steel cutting shoe slightly larger than the outside diameter of the pipe is fitted to the end of the lead pipe, and this shoe cuts a hole for the jacked pipe. This process is generally used to place the pipe liner in which the pressure pipe will be placed.

### Pipe Joints

The joint design and pipe configuration must be capable of transmitting jacking forces from pipe to pipe. It is very important that protective joint cushions be used to prevent damage to the joints and to provide sufficient uniform bearing around the joint circumference. The pipe manufacturer should be consulted if the pipe is to be installed by jacking; the recommendations of the pipe manufacturer should be closely followed.

### Line and Grade

In all jacking operations it is important to carefully establish the direction of jacking prior to starting work. Guide rails for the pipe, set accurately to grade and alignment, must be installed in the approach trench or jacking pit. For large pipe, it is desirable to have the guide rails set in concrete. Backstops must be strong enough and large enough to distribute the large jacking forces to the soil and to provide uniform support for the jacks so that the pipe is not pushed out of alignment. The backstops and pushing frame are usually fabricated from steel; however, concrete thrust pads are often used for large-diameter pipe. A system for continuously checking line and grade during the jacking operation is needed, with laser-beam alignment control being the most reliable.

### Jacking the Pipe

The pipe should be jacked upgrade if possible to facilitate drainage in the event that groundwater is encountered at the heading. Jacks should be designed to operate in the horizontal position and should have capacities well in excess of the anticipated force required. The lead pipe should be fitted with a steel cutting edge, and it may be desirable to surround the outside of the pipe with a lubricant, such as bentonite slurry, to reduce the frictional resistance.

Once started, it is best if the jacking operation is continuous, with interruptions only to reset the jacks and to add additional pipe lengths. Extended delays, including overnight, may result in subsidence of the earth and a large increase in restarting

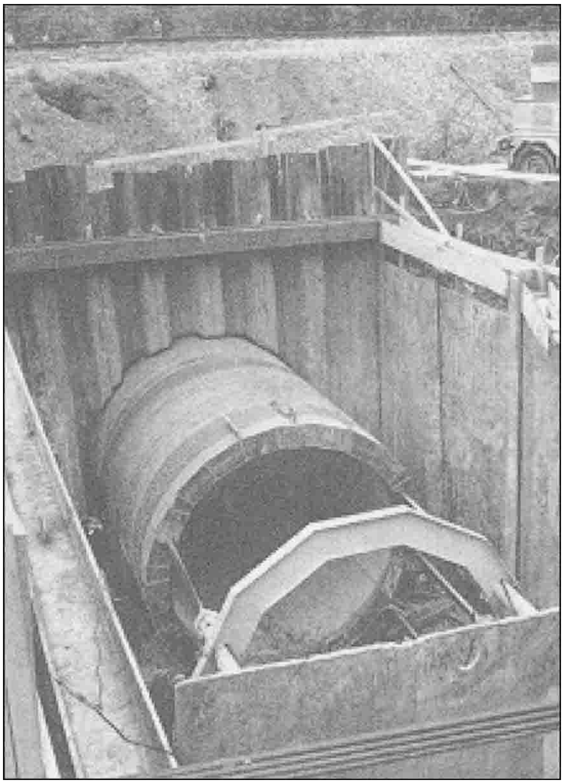


Figure 14-9 Tunnel casing pipe (132 in. [3,350 mm] in diameter) being jacked under a railroad using a metal push ring with a plywood cushion to protect the face of the pipe

resistance that must be overcome when jacking resumes. These large forces can cause severe damage to the pipe joints, which may require extensive repairs and delays.

## Excavation

The excavation at the top and sides of the heading should be just slightly larger than the outside diameter of the pipe. The bottom should be cut accurately to line and grade. For lines less than 36 in. (910 mm) in diameter, a mechanical auger is generally used at the heading, and the earth is removed through the pipeline by the auger or a conveyor. For larger sizes with more working room, excavation can be accomplished by mechanical means. Hand excavation and removal is seldom used today because it is expensive and results in a less uniform excavation. The excavation material should be trimmed with care and, to prevent cave-ins, the earth heading should minimally precede the lead pipe.

## Installation

When using concrete pressure pipe for the carrier pipe, several methods have been used to prevent damage to the pipe exterior while sliding the carrier pipe through the liner. One method is to strap timbers to the exterior of each pipe to act as runners. Another method, which is applicable only to mortar-coated pipe (ANSI/AWWA C301- and C303-type pipe), is to apply thick mortar coating rings in the pipe plant as wearing surfaces. For smaller-diameter pipe, a third method is to use an exterior segmented band with plastic-tipped steel ribs that act as runners. The segmented bands, which are generally called *casing spacers*, are supplied separately and bolted together around the pipe at the jobsite. These three methods of installing concrete pressure pipe as a carrier pipe inside a liner are illustrated in Figure 14-10.

# MINING METHODS

Because there is a wide variety of mining methods, machinery, and safety measures that must be used in tunnel excavations, the discussion in this section will be primarily limited to the installation of concrete pressure pipe as a finish or secondary liner. Federal and state regulations generally provide detailed construction and safety requirements, and the contractor should be aware of and make every effort to comply with those requirements.

Large tunnels (5 ft [1.5 m] in diameter or larger) in clay or granular material will normally require the use of a tunnel shield at the face and a primary lining of sufficient strength to support the surrounding earth. The primary liner may be jacked concrete pipe or a segmented liner system installed behind or within the tail of the shield. As the tunneling and installation of the primary lining progresses, the annular space behind segmented liners is filled with a cement grout pumped through connections that are provided in the primary liner. Segmented primary liners are usually made of precast segmented blocks of reinforced concrete, steel liner plate, or steel ribs and timber lagging, as shown in Figure 14-11.

## Casing Liners

For pressure lines through short tunnels, such as those under a highway or railroad, it is common practice to jack reinforced concrete pipe or install a steel liner first and then to slide the pressure pipe (the carrier pipe) through the liner.

## Secondary or Finish Liners

For tunnels required to be watertight or operate under pressure, concrete pressure pipe has proven to be an excellent and economical finish liner. In rock tunnels that do not require a primary liner, the pipe may be installed as the finish liner. Grout connections can be built in the pipe wall for easy filling of the annular space between the pipe and the tunnel excavation. Concrete pressure pipe is also used as the secondary or finish liner in mined tunnels, with the primary liner placed as the excavation progresses, as shown in Figure 14-11. Grouting the space between the pipe and the primary liner is accomplished in the same way as for rock tunnels and is discussed at the end of this chapter.

## Installation

A variety of techniques have been used for transporting or moving large concrete pipe into, or through, the excavated tunnel for placement. The methods vary from a simple skidding arrangement, presented previously, to special pipe-carrying machines that not only transport the pipe but also are capable of positioning and joining a section of pipe to the previously placed section (Figure 14-12).

# GROUTING

If grout is to be placed in the annular space between the pipe and the primary liner or the tunnel surface, it can best be accomplished by using pumping or pneumatic placing equipment suitable for handling the mixture to be placed. The grout mix must meet project specifications, but if it is to be pumped, the mix should include not less than four sacks (376 lbs [170 kg]) of cement per cubic yard. To enhance its flow characteristics and facilitate placement, the mixture should also contain fly ash or another suitable plasticizer at its recommended dosage.

A bulkhead for retaining the grout must be placed in the annular space at each end of the section that is to be grouted, and positive provisions must be made to prevent

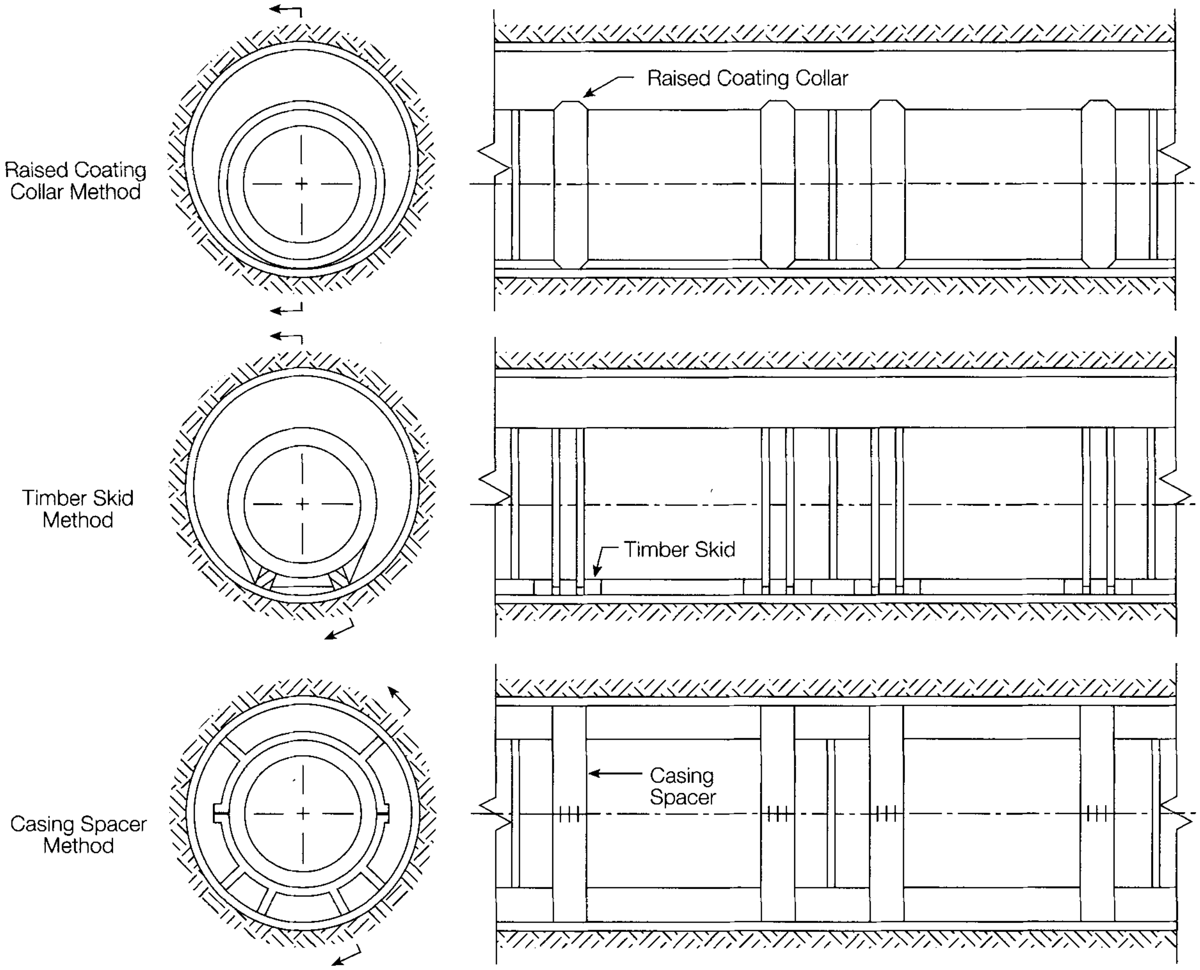



**Figure 14-10** Concrete pressure pipe as a carrier pipe inside a casing

flotation of the pipe and to vent air. At the start of the placing operation, the grout discharge pipeline should extend from the placing equipment to the bulkhead at the remote end. During placement, the grout discharge pipe must be positioned so that its discharge end is kept well buried in the grout at all times. After the grout is built up over the crown of the pipe, the grout placement line can be withdrawn as the grouting progresses. The placing of grout should continue under pressure until grout overflow reaches the required height in the riser pipes. An alternate method of grouting is through ports that were installed in the pipe walls during manufacture. Also, sand or gravel is sometimes used in place of grout to fill the annular space between the pipe and the liner or the tunnel surface.

The exposed interior surfaces of steel joint rings must be protected, as was explained earlier in this chapter in the section on laying the pipe. If the annular space between the pipe and the primary liner or tunnel surface is not filled with grout, the exposed exterior surfaces of steel joint rings must also be protected. Also, if the annular space between the pipe and the liner or tunnel surface is not to be filled with grout, sand, or gravel, then blocks or hold-down jacks should be used to prevent flotation and side drifting of the pipe.

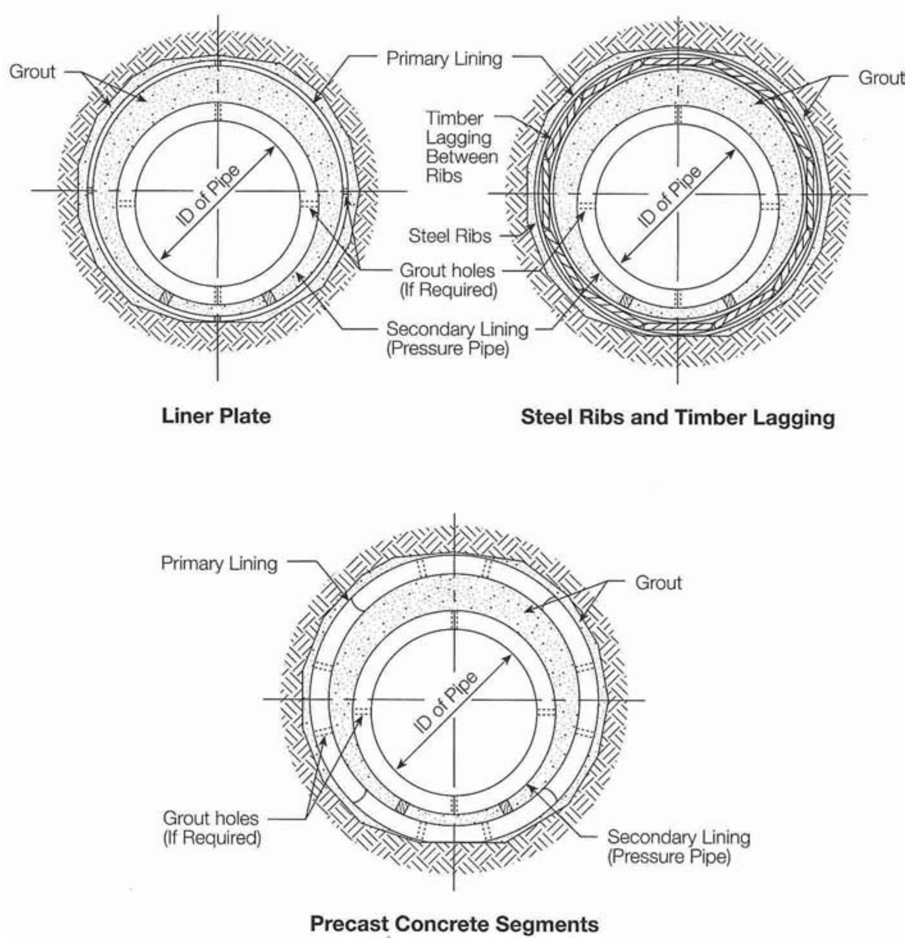



Figure 14-11 Typical tunnel sections

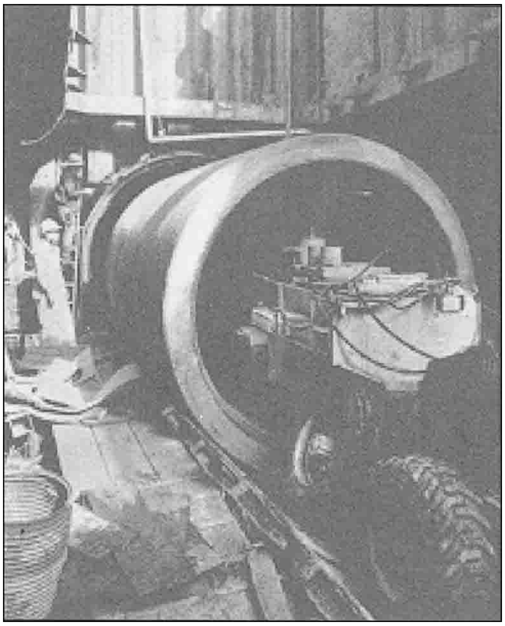


Figure 14-12 Special equipment, called a "tunnelmobile," being used to install large-diameter pipe in a tunnel

## REFERENCES

American Concrete Pipe Association. 1985. *Concrete Pipe Design Manual.* Vienna, Va.: American Concrete Pipe Association.

———. 1980. *Concrete Pipe Handbook.* Vienna, Va.: American Concrete Pipe Association.

Howard, A., and T. J. McGrath. *Design and Installation of Buried Pipes,* ASCE 1998.

Water Pollution Control Federation. 1969. *Design and Construction of Sanitary and Storm Sewers,* WPCF Manual of Practice No. 9, ASCE Manuals and Reports on Engineering Practice No. 37. Washington, D.C.: Water Pollution Control Federation.

Chapter 15

# Hydrostatic Testing and Disinfection of Mains

## HYDROSTATIC TESTING

Hydrostatic testing, which provides proof of pipe system integrity, is typically required by the owner prior to final acceptance of a pressure pipeline. The contractor is typically required to perform these tests, which are witnessed by the owner or his representative. For very long lines, it is sometimes more convenient to test shorter sections as they are completed, rather than wait and test the entire project at one time. Air testing of a pipeline is dangerous and should *never* be attempted.

### Makeup Water Allowance

During the pressure test, the contractor is generally required to meter the amount of water that is added to the line to maintain test pressure. If the quantity added is less than a predetermined value, the line is considered acceptable. The addition of water maintains the pressure, which might drop due to trapped air, absorption of water by the pipe walls, take-up of restraints, and temperature variations during testing. Specifications should require that all observed leaks be repaired regardless of the results of leakage measurements by metering equipment during the test.

### Test Pressures

To test the system, the hydrostatic pressure should be brought to at least the maximum working pressure of the line, and this pressure should be maintained for a prescribed period of time. Usually, specifications require that the tested section be subjected to 120 percent of the working pressure at the lowest point in the pipeline. The 20 percent increase could cause some increase in cost because the size of thrust blocks, the number of harnessed joints, and the pressure rating of appurtenances, such as flanges and valves, may need to be increased. The costs for a 20 percent increase in pressure are generally acceptable; however, increasing test pressures to a greater pressure may not be cost effective.

## Preparation for Testing

Pipelines are generally backfilled prior to testing because the amount of open trench is usually limited for safety reasons, and, when tied joints are the method of thrust restraint, the tied sections must be backfilled prior to the test in order to develop necessary soil friction. When concrete thrust blocks are used, the concrete must have been in place long enough to achieve sufficient strength to transmit the thrust to the undisturbed soil at the time of the testing.

Proper preparation prior to hydrostatic testing can help keep makeup water to a minimum. All outlets should be closed or plugged, and air valves should be located and checked to be sure they are operational. Lines with several high and low points and without air valves should have small outlets at all high points to bleed air while the line is being filled.

A newly installed line is generally filled by connecting it to an existing line. When this is not feasible, water can be pumped from a nearby source with a pump adequate to fill the line in a reasonable time. This pump is usually different than the pump used to conduct the pressure test. Both test pumps and pumps used to fill the line should always be monitored to avoid accidental overpressuring of the pipeline. Positive displacement pumps should always be used with pressure-relief valves, and centrifugal pumps should have shutoff heads less than the pipeline's pressure limitations.

The line should be filled at a slow rate to minimize air entrapment and potential surge pressures. After the line is filled, much of the remaining trapped air can be removed by flushing the line for a sufficient period to move the air to air valves or outlets. After filling, the line should be left pressurized (generally at the pressure of the filling source) for a minimum of 48 hours prior to testing. This will saturate the concrete lining, reduce the apparent leakage due to absorption by the pipe walls, and provide adequate time for small leaks to saturate the ditch backfill so that they can be located. In granular soils, leak location may be more difficult and may require leak-detection equipment.

Before beginning the test, all test equipment valves should be checked to see if they are open, connections and valves should be checked to make sure they do not leak, and equipment should be checked to see that it is operational. During the test, two gauges are desirable at the pump to provide a means of ensuring there is not an error in the pressure reading that could allow excess pressures that might damage the pipeline. Valves should be located at air-bleed outlets, between the pump and bulkhead, and at the low-point pressure gauge in the line. Meters for measuring makeup water and the pressure gauge at the low point are generally provided and calibrated by the owner.

## Makeup Water Allowances and Test Period

Generally, leakage allowances specified for pipe with a steel cylinder (ANSI/AWWA C300-, C301-, and C303-type pipe) differ from those for noncylinder pipe (ANSI/AWWA C302-type pipe), as follows:

| Type of Pipe | Makeup Allowance, *gal/in. dia./mi pipe/24 hr* *(L/mm dia./km pipe/24 hr)* |
|---|---|
| ANSI/AWWA C300, C301, and C303 | 10 (1) |
| ANSI/AWWA C302 | 50 (4.6) |

These assigned values are intended only to give the contractor some allowance for makeup water, because any observed leaks must be repaired. A 2- to 4-hour test is generally ample.

### Bulkheads and Thrust Restraint

Bulkheads for use in conducting hydrostatic tests are available from pipe manufacturers. Generally they have two outlets, one for filling and draining the pipeline and one for bleeding air from the line. A system to restrain thrust at the bulkhead must be provided. Most manufacturers provide either a dished head, designed to be braced against a thrust block or tied to the end of the pipeline, or a flat bulkhead, designed to be braced against a thrust block. Thrust blocks must be of adequate size and must be cast against undisturbed soil (see chapter 9). For a tied bulkhead, there must be enough pipe to restrain the thrust (see chapter 9).

For large-diameter pipelines, internal bulkheads are sometimes favored by contractors wishing to make periodic hydrostatic tests without interrupting pipe laying (Figure 15-1). These bulkheads are generally of a dished-head type or fabricated out of steel plate and reinforced with ribs. In the shop, the bulkhead is internally welded to the steel joint ring or cylinder of a special pipe or fitting. Thrust at the bulkhead may be resisted by compression of downstream piping. An added advantage of the internal bulkhead is that it can be designed for hydrostatic testing from either side of the bulkhead. After both adjacent sections of the pipeline have passed the test, the bulkhead is carefully removed and the recess in the pipe lining is filled with concrete or mortar.

## DISINFECTION OF MAINS

After a pipeline has passed the hydrostatic test, it must be disinfected and flushed if it is to carry potable water. Acceptable methods for disinfecting water mains are described in ANSI/AWWA C651. Flushing can be reduced if, during construction of a water main, care has been taken to avoid unnecessary contamination of the pipeline's interior. The use of uncontaminated water will reduce the time and the number of repetitions needed to disinfect the newly laid line to an acceptable level. Also, water

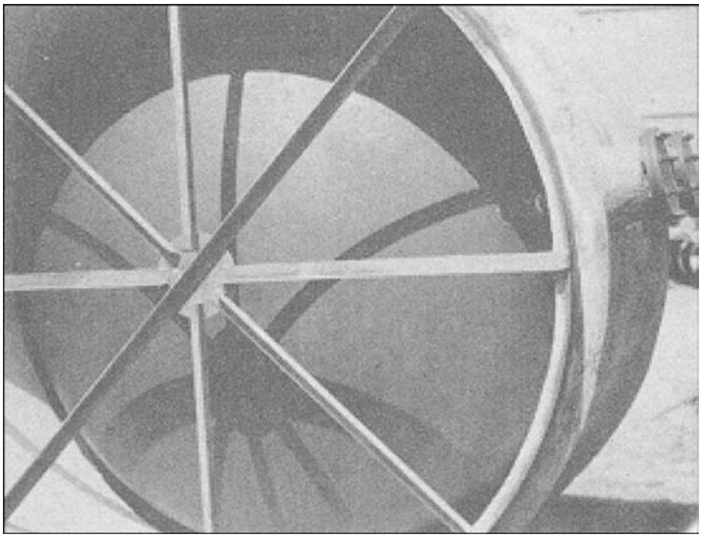


Figure 15-1 Temporary internal test plug

used for disinfection purposes may not be released to streams, sewers, or other bodies of water without neutralization of the chlorine content prior to its release.

## REFERENCE

American Water Works Association (AWWA). 2005. AWWA C651, *Standard For Disinfecting Water Mains*. Denver, Colo.: AWWA.

*Chapter* 16

# Tapping Concrete Pressure Pipe

Tapping into concrete pressure pipe is a routine procedure. According to the American Concrete Pressure Pipe Association (ACPPA), more than 60,000 taps have been made on concrete pressure pipe in the United States and Canada. Some pipe manufacturers have their own tapping services, and there are also specialized service companies available throughout the country. Many water utilities routinely make their own taps for pipe diameters up to 12 in. (300 mm). The procedures for tapping concrete pressure pipe under pressure are basically the same as for other kinds of pipe. It is important for the designer to be familiar with the basic features of a pressure tap as well as the particular requirements for concrete pressure pipe.

Weld-on tapping assemblies should not be used for pressure-tapping concrete pressure pipe because of structural damage problems that may occur to the pressurized pipe cylinder during welding. The tapping-saddle configuration should allow for the saddle to be securely bolted to the pipe prior to the removal of prestressing wire or reinforcing rod from the tap opening.

Tapping-saddle design is beyond the scope of this manual. However, the saddle length parallel to the centerline of the pipe must be sufficient to provide adequate reinforcement for the tapped opening. The waterways of tapping assemblies larger than 2 in. (51 mm) should be lined with cement mortar or an epoxy lining suitable for immersion service.

## THREADED PRESSURE TAPS UP TO 2 IN. (51 MM) IN DIAMETER

Mueller Company,* T.D. Williamson Inc.,† and similar type drilling machines are normally used for making small taps in concrete pressure pipe. Carbide-tipped masonry drills are required for satisfactory cutting of concrete.

### Two Types of Saddles

Two general types of tapping saddles are used in tapping lined cylinder pipe. These differ basically in the manner in which they are secured to the pipe. The first type uses straps around the pipe (Figure 16-1), while the second type of saddle is attached

* Mueller Company, Decatur, Ill.
† T.D. Williamson Inc., Tulsa, Okla.

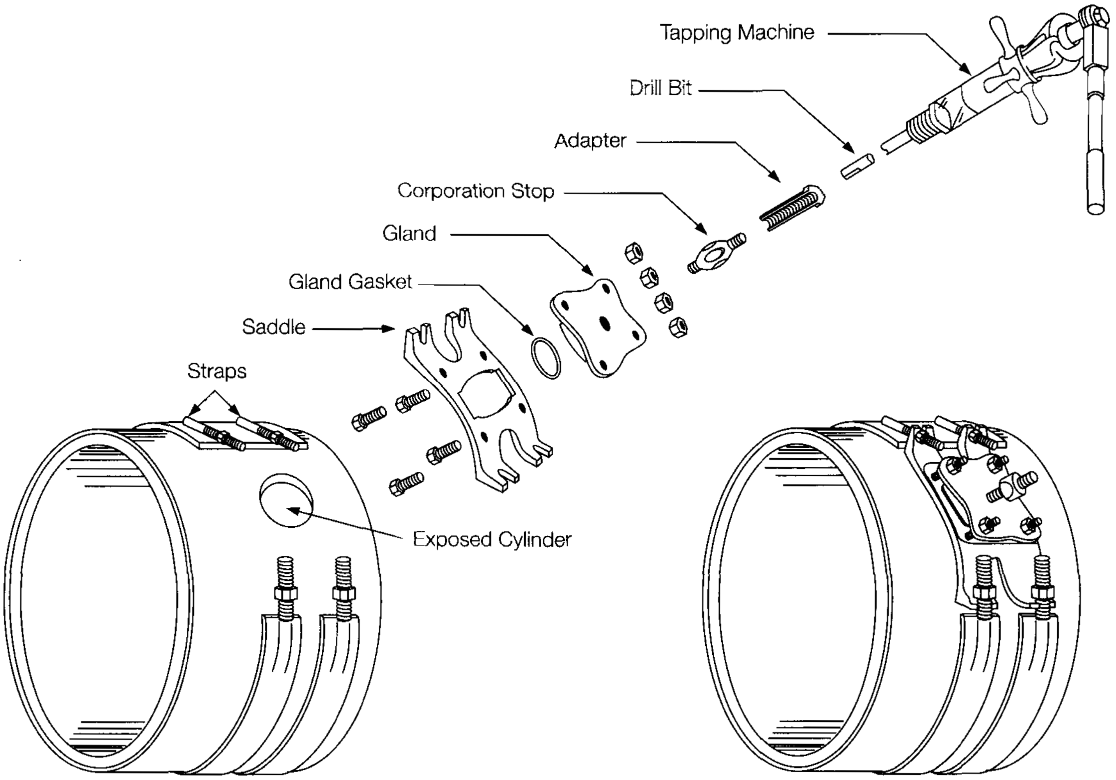



Figure 16-1 Small-diameter pressure tap using saddle with straps

to the prestressing wires (Figure 16-2). The second type is limited to a maximum of 1¼ in. (32 mm) taps, and no excavation around the pipe is required for installation.

## Securing Gland to Cylinder

In general, the procedure is as follows:

Type 1 (shown in Figure 16-1)

1. Chip away the mortar coating in an area slightly larger than the gland to expose the reinforcing wires and cylinder. Be sure there is no cylinder weld seam in the area where the saddle gasket will seat.
2. Position the saddle with inserted outlet bolts over the hole in the coating. Secure the saddle in place with straps.
3. Cut away the wires carefully in a manner that does not damage the pipe cylinder. (For pressure taps in this size range, other types of tapping saddles are available that require cutting the reinforcing wires prior to installation of the saddle.)
4. Install the gland gasket and gland, and outlet nuts. Tighten the outlet nuts to attain a seal between the cylinder and gland gasket, taking care not to dent the cylinder or crack the concrete core by overtightening.

Type 2 (shown in Figure 16-2)

1. Chip away the mortar in an area larger than the saddle plate to expose reinforcing wires or rods and the cylinder.

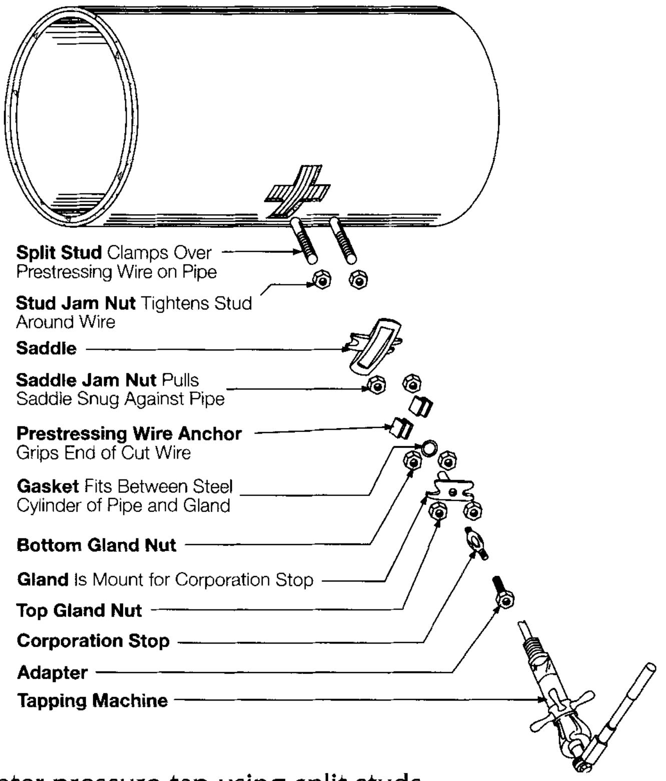



Figure 16-2 Small-diameter pressure tap using split studs

2. Attach the saddle plate to the reinforcing wires with split studs, and then anchor the wires that are to be cut to the saddle plate with anchor blocks.
3. Cut away the wires carefully so that there is no damage to the pipe cylinder.
4. Install the gland gasket and gland. Tighten against the cylinder using split studs.

## Completion of Tap

The following steps are common to both types of saddles described:

1. Tighten the corporation stop into the gland and perform a pressure test to ensure the seal is watertight. Open fully. Attach the tapping machine to the corporation stop and drill through the cylinder and concrete core (Figure 16-3). Retract the bit completely, close the corporation stop, and remove the tapping machine. Open the corporation stop and flush away the cuttings.
2. Coat all metal parts with a thin cement grout mixture, then build up a minimum of 1 in. (25 mm) thickness over the metal parts using a stiff mortar. As an alternative, protect the straps by using a joint diaper and poured grout. This completes the tap.

# PRESSURE TAPS FOR FLANGED OUTLETS 3 IN. (76 MM) AND LARGER

The following tapping procedure describes a general method only. More specific information is available from pipe manufacturers. Flanged pressure taps are limited to the next smaller pipe diameter of the pipe being tapped. Tap size may also be limited by availability of gate or tapping valves. Carbide-tipped shell cutters, pilot drills, and power-operated, automatic-feed tapping machines are recommended.

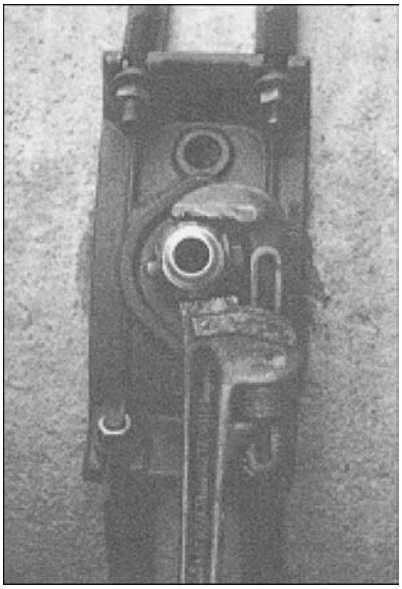


Figure 16-3 Installation of a small-diameter threaded pressure tap using a saddle with straps

## Tapping Procedure

Numbered references in the following procedure correspond to the numbered details shown in Figure 16-4. Figures 16-5 and 16-6 show pressure tapping of large-diameter and 24-in. (600-mm) diameter pipe, respectively.

1. Chip away the mortar coating from the area (1) where the tap is to be made.
2. Position the saddle in place, tightening all the straps (8) around the pipe. Pour grout into the prepared openings (4) in the saddle, filling the space between the saddle and pipe. Fast-setting, early-strength portland cement grouts are available to reduce the time required for the grout to set. Allow the grout to set. For large-diameter taps, be sure the pipeline pressure does not exceed the safe tapping pressure for the cylinder. Cut the reinforcing wires (5) to provide clearance for the gland to seal against the cylinder. For embedded cylinder pipe, remove the outer portion of the concrete core to expose the cylinder. Pneumatic chipping hammers are used in the removal of the mortar and outer core.
3. Position the gland (7) in the saddle (2) with its rubber gasket (6) against the steel cylinder. Evenly tighten the outer circle of bolts connecting the two flanges to compress the gasket against the steel cylinder to achieve a permanent watertight seal. Engage the stabilizing set screws through the gland flange against the face of the saddle flange. Seal the gland flange and test the cavity and gland gasket seal for watertightness. The fluid pressure must not exceed the pressure inside the pipe being tapped.
4. Remove the gland flange seal. If required for larger taps, secure the concrete core to be cut to the steel cylinder coupon using special toggle equipment.
5. Attach a tapping or gate valve to the gland flange with the inner circle of bolts. Pressure test to ensure a watertight flange seal.
6. Connect the tapping machine to the valve. Be certain that the saddle and gland does not have to carry the weight of the valve and tapping machine by providing support for both pieces throughout and after the tapping procedure.

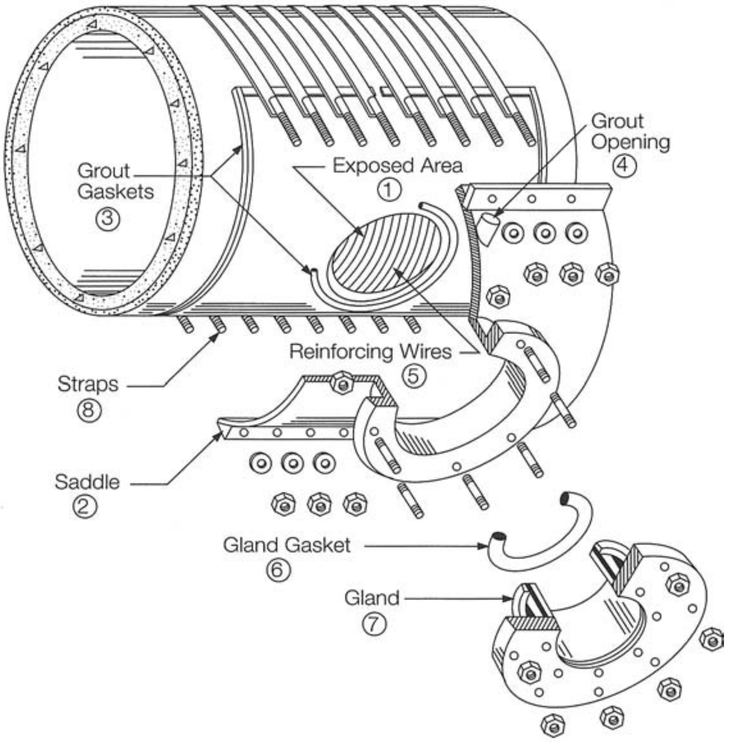



Figure 16-4 Large-diameter tapping assembly

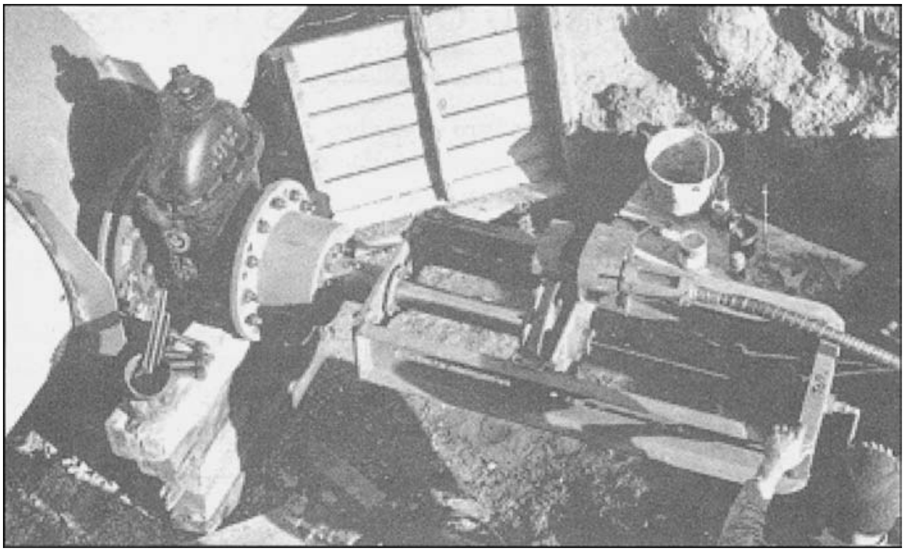


Figure 16-5 Pressure tapping a 12-in. (300-mm) diameter flanged outlet on a larger-diameter pipe

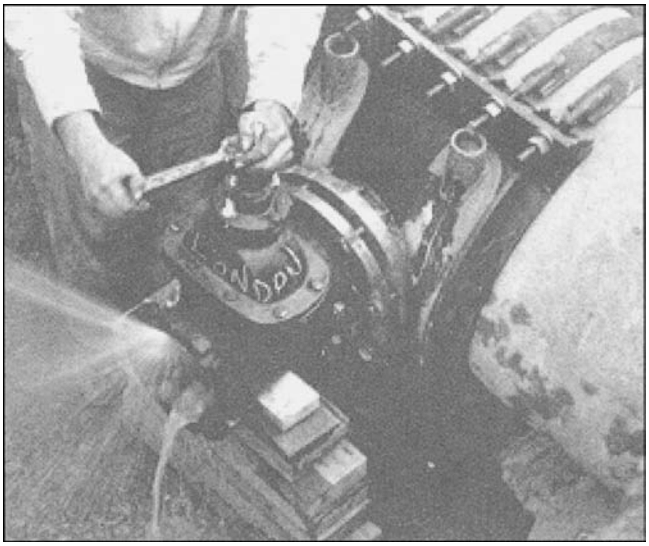


Figure 16-6 Pressure tap of a 12-in. (300-mm) diameter flanged outlet on a 24-in. (600-mm) diameter pipe

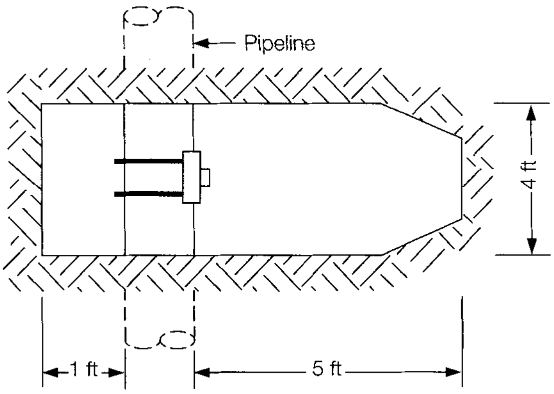
7. Open the valve completely. Advance the cutter using the handscrew through the opened valve to the steel cylinder of the pipe. When power is applied, the pilot drill will begin to cut the cylinder. Resistance to feed will suddenly increase as the shell cutter contacts the pipe cylinder and begins its circular cut. When the automatic feed screw has advanced the proper predetermined distance, the cut is complete.
8. Withdraw the cutting head past the gate and close the valve. Disconnect the tapping machine. Open the valve slightly to flush out any small cuttings that remain. Fill the space between the saddle and gland with grout, and apply a protective coating of cement mortar over the entire assembly.
9. Provide thrust blocking behind the tap and permanent support beneath the valve.

## TAPPING NONCYLINDER REINFORCED CONCRETE PRESSURE PIPE

Noncylinder reinforced concrete pressure pipe (ANSI/AWWA C302-type pipe) can also be tapped with slight modifications to the previously described procedure.

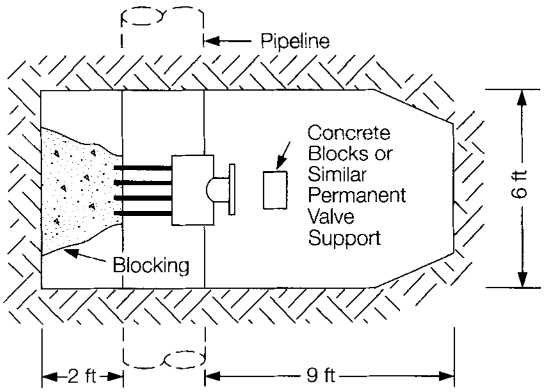
## SUPPLEMENTAL DATA

Information concerning minimum excavation dimensions and the range of outlet diameters for pressure taps are given in Figure 16-7.



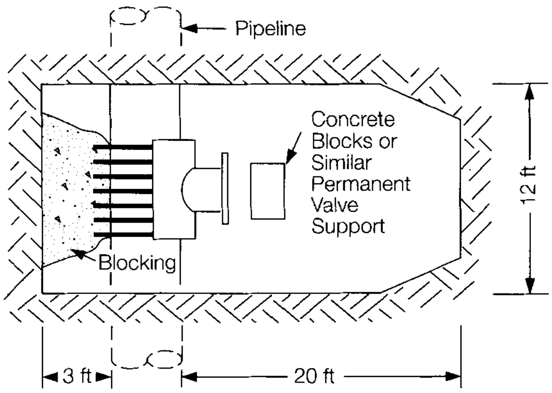

**Threaded Tap Sizes ¾ in. to 2 in.**

Air — 90 $ft^3$/min at 90 psi
Approximate machine weight 40 lb




**Flanged Tap Sizes 4 in. to 18 in.**

Air — 90 $ft^3$/min at 90 psi
Approximate machine weight 1,000 lb




**Flanged Tap Size 20 in. to 48 in.**

Air — 125 $ft^3$/min at 125 psi
Approximate machine weight 7,000 lb
(Lifting equipment must be able to lift 8 ft above ground level.)

NOTES:
1. Excavate 2 ft under pipeline for all taps.
2. Tapping personnel are not permitted to enter excavations that do not meet Occupational Safety and Health Administration (OSHA) safety standards.
3. All dimensions shown are approximate; contact the local tapping representative for actual dimensions.

Figure 16-7 Typical pressure-tapping site requirements

*Chapter* 17

# Guide Specifications for the Purchase and Installation of Concrete Pressure Pipe

Specifications for purchasing concrete pressure pipe can be simplified by referring to the applicable AWWA standards and providing the supplemental details and information required by each standard. Typically, these provisions are found in the applicable standard and titled "Drawings (or Plans) and Data to Be Provided by the Purchaser." To avoid errors in interpretation, it is recommended that information supplied by the purchaser be given in terms identical to those used in the standards. The AWWA standards include the controls on component materials, structural dimensions, and manufacturing procedures necessary to ensure high-quality pipe. Project specifications must include sufficient information for proper pipe design and sufficient controls for pipe installation to make sure the installed pipeline will provide the desired service with minimum maintenance.

The following guide specifications for the purchase and installation of concrete pressure pipe are intended to assist the purchaser and engineer in controlling the variables of pipe installation. The purchaser and/or engineer may need to provide additional specifications to cover critical characteristics of individual projects. In addition to these guide specifications, additional design information should be provided as follows:

| Design Information | Where Provided |
|---|---|
| Sizes, types (ANSI/AWWA C300-, C301-, C302-, or C303-type pipe), and design requirements for each class of pipe. | To be listed on project drawings and, if applicable, bid schedule. |
| Station limits for each size, type, and pressure class of pipe. | To be shown on project drawings. |
| Project-specific soil characteristics. | To be listed in specifications paragraph on design. |
| Typical trench cross sections* showing maximum trench widths when applicable, bedding and backfill materials, soil characteristics, and consolidation requirements. | To be provided on project drawings. |
| Tied-joint thrust restraint locations if applicable. | To be shown on project drawings. |
| Thrust blocking designs and locations if applicable. | To be shown on project drawings. |
| Groundwater elevations if applicable. | To be shown on proposed pipeline profile on project drawings or soil-boring logs. |

*Suggested pipe beddings are shown in chapter 6, Bedding and Backfilling.

The project drawings must also include the plan and necessary profile views of the pipeline, alignments and grades, outlet locations and descriptions, connections, appurtenances, and any other special details or information as are necessary for the design and manufacture of pipe and fittings in accordance with the applicable AWWA standard and the project specification.

In these guide specifications, the term *engineer* is defined as the purchaser's representative responsible for the project design, inspection, and recommending approval of the project installation.

# GUIDE SPECIFICATIONS FOR PURCHASING CONCRETE PRESSURE PIPE

## General

The contractor shall provide all of the pipe, including all fittings and special pipe sections, in the types, sizes, and design classes shown on the project drawings and/or bid schedule. The pipe shall conform to the requirements of ANSI/AWWA C300, ANSI/AWWA C301, ANSI/AWWA C302, or ANSI/AWWA C303 as applicable, and to the applicable sections of the contract documents and this manual.

## Pipe Structural Design

ANSI/AWWA C301-type pipe shall be designed in accordance with the design standard ANSI/AWWA C304. ANSI/AWWA C300-, ANSI/AWWA C302-, and ANSI/AWWA C303-type pipe shall be designed in accordance with the procedures given in chapter 7 of this manual. The pipe design shall be based on the working pressures, surge pressures, earth cover, and live load criteria provided in the contract documents. The external load design shall be suitable for the installation conditions shown in the project plans. The following soil characteristics shall be used when calculating earth loads in accordance with Marston's procedure:

Trench Installations

$w$ = 120 lb/ft³ (1,922 kg/m³)

$K\mu'$ = 0.150

Embankment Installations

$w$ = 120 lb/ft³ (1,922 kg/m³)

$K\mu$ = 0.190

$r_{sd}p$ = 0.50

Where:

$w$ = soil unit weight, lb/ft³

$K$ = ratio of active lateral unit pressure to vertical unit pressure

$\mu'$ = coefficient of friction between fill material and sides of trench

$\mu$ = coefficient of internal soil friction

$r_{sd}$ = settlement ratio

$p$ = projection ratio

Soil unit weight shall be modified, if applicable, in accordance with the groundwater conditions as shown on the project drawings.

## Fittings and Special Sections

Fittings and special pipe sections shall be designed and fabricated to the requirements of the appropriate AWWA standard and chapter 8 of this manual, as applicable, and as shown in the details of the contract documents. All required flanges shall conform to ANSI/AWWA C207, Standard for Steel Pipe Flanges for Waterworks Service—Sizes 4 In. Through 144 In. (100 mm Through 3,600 mm), requirements for standard steel flanges corresponding to the pipe working pressures.

## Submittals by the Contractor

The pipe design details and layout schedule shall be prepared by the manufacturer for the contractor's submittal to the purchaser. This submittal package shall be prepared in accordance with the applicable portions of the appropriate AWWA standard, AWWA Manual M9, and the contract documents.

## Marking of Pipe

The inside of each pipe section, fitting, or special pipe section shall be plainly marked with the pipe diameter and pressure class for which the section or fitting is designed. In addition, all fittings and special pipe sections shall be marked with an identifying number or station corresponding to that shown on the layout schedule. All fittings or special sections requiring special field orientation during installation shall be properly marked.

## Pipe From Inventory

Pipe sections may be provided from inventory if they meet the requirements of the contract documents with the appropriate certification.

## Inspection

The purchaser reserves the right to witness the manufacturing progress for its project. At the purchaser's option and expense, all certification testing may be witnessed by the purchaser and/or third-party inspectors. All testing results will be recorded and maintained as certification records at the manufacturer's facility for a minimum period of one year from the date of pipe delivery.

### Rate of Delivery

This section is applicable to "purchase" or "supply bid" agreements only. When the contract documents include both purchasing and the installation of pipe, the pipe delivery requirements should not be a part of the contract document considerations. Only the project completion criteria should be specified. This will allow the pipe manufacturer and the contractor to establish the responsibilities for the schedule for pipe deliveries. An example of a pipe delivery schedule is as follows:

> Delivery of the pipe, with associated specials and fittings, shall begin at station (  ) on or before (  ), but not sooner than (  ) weeks after receipt of owner-approved engineering details, and shall proceed at the rate of (  ) feet per week.

## GUIDE SPECIFICATIONS FOR THE INSTALLATION OF CONCRETE PRESSURE PIPE

### General

This section of the specifications covers the excavation for, and the handling, laying, bedding, backfilling, chlorination, and field testing of all pipe, fittings, and special pipe sections in the sizes and classes shown in the contract documents.

### Excavation and Subgrade Preparation

Trenches shall be excavated to a depth sufficient to provide the required bedding of the pipe shown on the project drawings. The trench shall be deep enough to provide at least 3 ft (900 mm) of cover over the pipe. The pipeline profile shall be selected so as to minimize high points where air may be trapped.

Trench width shall not be less than the pipe outside diameter plus 20 in. (510 mm) so that sufficient access is provided on each side of the pipe for bedding and backfill consolidation. Sufficient width shall also be provided for any necessary trench safety equipment. When applicable, the maximum allowable trench width is shown on the trench detail. Should the actual trench width at the top of the pipe exceed the maximum allowed, approved bedding and backfill materials and other measures to increase the pipe support to resist the resulting additional external load shall be provided at the contractor's expense.

If unstable trench conditions are encountered, the contractor shall employ appropriate remedial measures to stabilize the condition.

If the trench is inadvertently excavated deeper than necessary, it shall be backfilled to the proper grade with approved compacted granular material at the contractor's expense.

If the trench bottom is composed of rock or other unyielding material, the contractor will be required to excavate to a depth of at least 6 in. (150 mm) below the bottom of the pipe. The overexcavation shall be filled with compacted granular material up to the elevation of the bottom of the pipe bedding.

The bottom of the trench shall be prepared to receive the pipe as shown on the trench detail in the project drawings. The trench bottom shall be graded to the established line to provide uniform support for the full length of the pipe.

The contractor shall excavate any trenches required to route the pipeline around existing pipelines or other obstructions. The contractor shall give notice to the owners of any such lines or obstructions in order that they may have time to take the necessary precautions for protecting their property. The contractor shall be responsible for protecting the purchaser from any damage that may be caused by the contractor's operations in such work.

All applicable local, state, and federal laws and regulations shall be carefully observed, including those relating to the protection of the excavations, the safety of the workers, and provisions for the required barriers, signs, and lights.

## Handling and Laying

Pipe, fittings, valves, and other accessories shall be hauled to and distributed at the site of the project by the contractor (Figure 17-1). These materials shall be handled with care at all times to avoid damage. In loading and unloading, the materials shall be lifted or moved with appropriate equipment in a manner that avoids sudden impact or damage. Under no circumstances shall pipe be dropped. Pipe must not be skidded or rolled against pipe already on the ground. Pipe shall be placed along the trench alignment at the work site, with bell ends facing the direction in which the work will proceed (Figure 17-2).

Proper implements, tools, equipment, and facilities shall be provided and used by the contractor to allow for the safe and professional execution of the work. All pipe, fittings, specials, valves, and so on, shall be lowered into the trench by means of suitable pipelaying equipment and shall not be rolled or dumped into the trench. Pipe shall be hoisted from the trench side into the trench using a suitable sling or other device that will not damage the pipe. The laying equipment shall have sufficient capacity and stability to handle and maneuver the pipe throughout the laying operation. The method of construction shall be subject to the engineer's approval. Before being lowered into the trench, each joint of pipe shall be inspected and any unsound or damaged pipe shall be repaired or rejected.

The pipe shall be cleaned of all sticks, dirt, and trash during the laying operation. At the close of each operating day, the open end of the pipe shall be sealed to prevent the entrance of foreign objects and water. No pipe shall be laid in water or when trench conditions or weather are unsuitable for such work, except in an emergency with permission by the engineer.

Each pipe shall be supported by a sling while the joint is being positioned for engagement (Figure 17-3). All pipe shall be laid accurately to established lines and grades with valves and fittings at the required locations and with joints centered and spigots pushed home. Pipe shall not be installed on rigid blocks or similar supports.

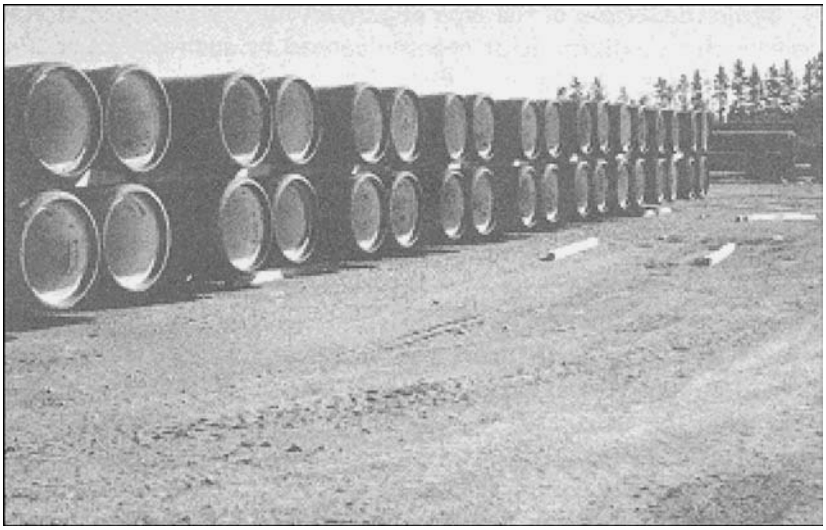


Figure 17-1 30-in. (750-mm) pipe ready for delivery

Figure 17-2 24-in. (600-mm) lined cylinder pipe strung along a right-of-way

Minor deflection of the pipe alignment may be obtained at standard pipe joints; however, the maximum joint opening caused by such deflection shall not exceed the recommendations of the pipe manufacturer. When it becomes necessary to make larger deflections, sections of pipe with beveled ends or fabricated fittings shall be used. Random-length pipe and/or bevel adaptors may be used to make unforeseen changes in the field.

## Pipe Jointing

Before laying each joint of pipe, the bell and spigot shall be thoroughly cleaned by wire brushing and wiping until clean and dry. The bell, spigot, and joint gasket shall be lubricated with a nontoxic, water-soluble lubricant suitable for use in potable water lines. The gasket shall be placed in the spigot groove and the tension in the gasket shall be equalized by inserting a smooth, round rod under the gasket and moving it completely around the circumference of the joint. The spigot end of the pipe shall be centered with the bell of the last length of previously laid pipe and pushed into position. Following assembly, the entire circumference of the pipe joint shall be checked with a feeler gauge to ensure the gasket remains seated in the spigot groove. If the gasket is found to be displaced, the joint shall be removed, the gasket inspected, the joint relaid, and the gasket position rechecked. The joint shall then be checked to make certain that the proper amount of joint engagement has been achieved. On completion of pipe jointing, the interior of the pipeline shall be cleaned of all debris.

Except for subaqueous installations, the interior of steel joints in pipelines transporting brackish water, brine, sanitary sewage, or similar aggressive water shall be pointed with portland cement mortar. The interior of steel joints for pipelines transporting nonaggressive water shall be protected with portland cement mortar, metallized zinc, or other protective coating, as recommended by the pipe manufacturer. Except for subaqueous installations, the exterior joint recess will typically be filled with portland cement mortar. Proposed protection for steel joint rings in subaqueous installations shall be submitted to and approved by the engineer prior to the manufacture of the pipe.

When interior joints are protected with mortar, the procedure for mortar placement shall be as follows. Prior to the placing of mortar, any dirt or trash that has collected in the joint shall be cleaned out, and the surfaces of the joint space shall

Figure 17-3 Prestressed concrete cylinder pipe being lowered into position. Note depth of embankment cut in background.

be moistened with water by spraying or brushing with a wet brush. The inside joint recess at the bell end of 36-in. (910-mm) and smaller pipe shall be filled immediately prior to placing the pipe together by buttering the bell recess with mortar. After the joint is engaged, the joint mortar of pipe 18 in. (460 mm) in diameter and smaller shall be smoothed and cleaned with a swab, and the joint mortar of pipe larger than 18 in. (460 mm) shall be finished off smooth by hand trowel. The inside joint recess of pipe larger than 36 in. (910 mm) shall be filled with mortar and finished smooth after the joint is engaged. Careful inspection shall be made of each joint to ensure a smooth, continuous interior surface.

When the exterior joints are to be protected with mortar, the procedure for mortar placement shall be as follows. A grout band shall be placed around the pipe and positioned to straddle the joint recess. The band shall be of sufficient length to essentially encircle the pipe and shall be secured so that joint mortar will be contained with little or no leakage. The band shall be completely filled with grout in one operation by filling from one side only until the grout rises on the opposite side, and then the grout shall be rodded or agitated on both sides of the pipe alternately to settle the grout and fill all voids.

After the grout band is filled and rodded or agitated, the grout should then be left undisturbed for at least 15 min to allow it to mechanically stiffen and, when permeable grout bands are used, to allow excess water to seep through the band. After this period, more grout shall be added if necessary to refill the joint completely. The gap at the top of the grout band must be protected from penetration of backfill into the grout by (1) allowing the grout to stiffen, (2) capping with a stiff mortar mix, or (3) covering with a structurally protective material. The grout band shall not be removed from the joint.

For flush-bell pipe (ANSI/AWWA C300-, ANSI/AWWA C301- [embedded cylinder], or ANSI/AWWA C302-type pipe), bedding and backfilling up to the level just below the top of the pipe may proceed sooner than 15 min, and if backfilling is necessary to support the grout, before the grout band is completely filled. As an alternative to backfilling to support the weight of the grout, temporary reinforcement may be placed around the band. For raised-bell pipe (i.e., lined cylinder ANSI/AWWA C301- or ANSI/

AWWA C303-type pipe), additional bedding or backfill shall not be placed on either side of the pipe until after the grout band has been filled and the grout has mechanically stiffened.

The mortar used at joints shall consist of one part portland cement to no more than three parts clean sand mixed with water. Interior joint mortar shall be mixed with as little water as possible so that the mortar will be very stiff, yet workable. Exterior joint mortar shall be mixed with water until it has the consistency of thick cream. During periods of cold weather, the joint mortar shall be adequately protected from freezing.

## Tunnel Construction

Concrete pressure pipe installed as carrier pipe in a tunnel or encasement shall have uniform alignment. The pipe shall slide on sacrificial mortar bands, timber skids, or similar supports that will protect the pipe exterior from damage during installation. The contractor shall submit procedures for the installation of pipe and any necessary joint protection to the engineer for approval prior to the installation of pipe. Pipe shall be installed in such a manner that it cannot float when empty.

## Protective Coatings Applied in the Field

The contractor shall provide a 1-in. (25-mm) minimum thickness concrete or cement–mortar coating in the field for the protection of all exposed steel, such as flanges, threaded outlets, and closures. The cement mortar used shall consist of one part portland cement to no more than three parts of clean sand. Mortar coating for areas larger than 100 $in.^2$ (0.06 $m^2$) that are not formed with a grout band shall be reinforced with galvanized wire mesh.

Any surface receiving a cement–mortar coating shall be thoroughly cleaned and wetted with water just prior to placing the cement mortar. After placement, care shall be taken to prevent the mortar from drying out too rapidly by covering with a curing compound, damp earth, or burlap. Cement–mortar coating applied during periods of cold weather shall be adequately protected from freezing.

## Thrust Restraint

Prior to backfilling, thrust restraints shall be installed at dead-end plugs, bends, tees, and similar sources of thrust. Thrust restraints shall be either tied joints or thrust blocking as required on the project drawings. When tied joints are required, either welded or harnessed joints may be used unless a particular method is specified on the drawings. The contractor shall submit the proposed design of any required tied joints to the engineer for approval prior to installation.

When thrust blocking is required, concrete for blocking shall develop a minimum compressive strength of 2,500 psi (17.2 MPa). All materials, including aggregates, cement, and water, as well as the mixing and placing of the concrete, shall be approved by the engineer. Blocking shall be placed between the pipeline and solid undisturbed ground and centered on the direction of force from the fitting. The area of bearing on pipe and on ground in each instance shall be that required by the engineer. The blocking shall, unless otherwise directed, be placed so the pipe and fitting joints will be accessible for repair.

## Bedding and Backfilling

The materials and soil densifications required for the placement of pipe bedding and backfill are shown on the trench detail in the project drawings. Materials within 6 in. (152 mm) of the pipe shall be free from rocks or clods larger than 3 in. (76 mm) in diameter. All bedding and backfill materials shall be placed free of metallic debris to avoid electrical interference with future corrosion monitoring that may be performed.

To avoid pipe displacement, bedding and backfill shall be placed and consolidated at essentially the same rate on both sides of the pipe. The depth of layers of fill shall be consistent with the method of consolidation used.

Cohesive soils shall be densified by compaction using mechanical or hand tamping. Care shall be taken not to damage the pipe exterior during soil compaction. Moisture content of cohesive soils shall be adjusted to facilitate the degree of compaction required on the detail drawings for bedding and backfill.

Granular bedding and backfill may be densified by tamping or internal vibration. Where both native and fill materials are free-draining, bedding and backfill may also be densified by consolidation with water. Materials to be hydraulically consolidated shall pass a 1½-in. (38-mm) screen and have not more than 10 percent pass a 200-mesh sieve. When water is used for fill consolidation, provisions shall be taken to prevent the pipe from floating.

## Chlorination and Field Testing

During the construction operations, workers shall be required to use the utmost care to ensure all interior surfaces of structures, pipe, fittings, jointing materials, valves, and so on, are maintained in a sanitary condition. Every effort shall be made to keep the inside of the pipe, fittings, and valves free of all foreign matter, including sticks, dirt, rocks, and similar material.

After the entire pipeline or certain selected sections thereof have been laid and backfilled, but prior to replacement of any pavement, the line or sections of line shall be filled and chlorinated as specified in ANSI/AWWA C651, Standard for Disinfecting Water Mains. Filling and chlorination shall not begin until the engineer's approval of the method, apparatus, chlorinating agent, and sections of line has been obtained. All connections, gauges, and meters shall be provided by the contractor at its expense. Water for filling and initial testing of the line shall be provided by the owner at no expense to the contractor. Water for any retest shall be provided at the contractor's expense.

The line shall be carefully filled at a slow rate to minimize air entrapment and water hammer. In no case shall the water velocity in the pipeline being filled exceed 2 ft/s (0.61 m/s). After the line is filled, chlorinated, and entrapped air removed, the line shall be left lightly pressurized for a minimum of 48 hr prior to the field hydrotest.

The field hydrostatic test pressure shall be 120 percent of the working pressure for which the line is designed, measured at the lowest point in each test section. At intervals during the test, the entire route of the section under test shall be inspected by the contractor and the engineer for evidence of any leaks. Regardless of the amount of leakage, all identified leaks in the material supplied for this contract shall be repaired at the expense of the contractor.

For the purpose of field testing, leakage is defined as the quantity of water supplied into the newly laid pipe (or any valved section under test) necessary to maintain the specified pressure for the duration of the test. The maximum allowable leakage for ANSI/AWWA C300-, ANSI/AWWA C301-, or ANSI/AWWA C303-type pipe is 10 gal per inch of diameter per mile of pipe per 24 hr (1 L/mm/km/24 hr) at test pressure. The maximum allowable leakage for AWWA C302-type pipe is 25 gal per inch of diameter per mile of pipe per 24 hr (2.3 L/mm/km/24 hr). The test pressure shall be applied for a minimum of 2 hr. All pressure tests shall be continued or repeated until the engineer is satisfied that the pipeline meets the requirements of this specification.

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