BURIED STEEL PENSTOCKS

Steel Plate Engineering Data • Volume 4

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Steel Plate Engineering Data - Volume 4

BURIED STEEL PENSTOCKS
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Preface

It is the objective of this revised technical manual to assemble in one publication data and procedures that have been successfully used in the design of buried steel penstocks, incorporating the modern advances in steelmaking practices and pipe manufacturing methods to provide the most economical design for the wide range of conditions encountered in the industry.

Additional information is available from manual No. 79, Steel Penstocks from the American Society of Civil Engineers published in 1993.

Introduction

Traditionally, steel penstocks have been considered as very high pressure conduits, usually of large diameter, and operating with frequent surges during the normal condition. Penstocks may also be subject to pulsations of varying frequency and amplitudes transmitted from the turbine or pump. When penstocks are installed above ground, this can sometimes cause excessive vibration. These are the perceived differences between a penstock and an ordinary pipeline. Based on these conditions, penstocks have been designed to standards established in 1949 with minor revisions based on an allowable design stress at normal conditions of 2/3 of yield or 1/3 of tensile strength.

In the last 25 years there have been many changes in steel making practice, pipe manufacturing methods and welding procedures. During this time, thousands of miles of steel pipe manufactured to American Water Works Association Standard C-200 have been put into service for water transmission lines including: flow lines, inverted siphons and pump discharge lines. These lines have usually been buried lines operating at working pressure of 150 to 350 p.s.i. plus transient pressures. Many lines, however, have been installed since 1960 with working pressure as high as 640 p.s.i. using high strength low alloy steels with an allowable design stress of 50% of yield at working pressure and up to 75% of yield at transient pressures in accordance with AWWA M-11 "Manual of Water Supply Practices" for steel pipe. These lines have utilized O-ring joints to working pressures of 250 p.s.i. or more and bell and spigot welded joints to working pressures of 400 p.s.i. with butt welded joints used at working pressures over 400 p.s.i.

Today there are many penstocks installed utilizing thousands of feet of pipe with operating pressures varying from no pressure at the inlet structure, to low or moderate pressures, or to very high pressures at the power plants. Most of these penstocks are buried and many parallel the stream from which the water was diverted. They are usually in remote locations. With certain types of turbines and an adequate control valve system, sometimes involving a synchronous bypass system, transient pressures can be limited. A buried penstock will not be subject to the problem of harmonic vibrations sometimes associated with the traditional penstock.

For many of these installations, when carefully evaluated by the engineer, the normal quality of AWWA C-200 pipe and the design standards of AWWA M-11 should be considered adequate and will provide the most economical material for this service. As working pressure and pipe diameter increase, the use of high strength low alloy steel will become economical. At design stresses over 21,000 p.s.i. at normal conditions, additional testing, including 100% ultrasonic or radiographic inspection of welds, is appropriate.

On many long line penstocks, the AWWA Standard and the traditional standard can be combined for the appropriate portions of the line. This manual, therefore, will address the design, materials and fabrication for both types of installation.

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CHAPTER 1

Hydraulics

Hydraulic and economic studies are required in connection with each installation for the determination of proper conduit diameter, location and layout details, and pressure gradients for the design conditions. Head losses should be computed as accurately as possible, economic diameters should be ascertained on the basis of the best information available, and water hammer computations are required to determine the transient pressure gradients and desirability of installing surge tanks or surge suppressors.

Section 1.1 Hydraulic Losses

Hydraulic losses in a penstock reduce the effective head in proportion to the length and approximately as the square of the water velocity. The mechanism of resistance in the flow of fluids is complex and has not yet been fully evaluated. Several conventional flow formulas have been developed and used in the past. Of these, the most notable and widely used in the waterworks field are the exponential formulas of Hazen-Williams and Scobey. At present, these conventional flow formulas are more widely used in the waterworks field than the universal flow formulas. The latter are rationally founded formulas applicable to many fluids having different viscosity, density, and fluidity characteristics which change with temperature. These conventional and universal flow formulas are discussed in AWWA Manual M11. Until the universal flow formulas become fully established as being applicable to penstock service, and roughness values are more definitely fixed for such pipe and related coatings, the use of conventional formulas appears practical and adequate. The Hazen-Williams formula is widely used in the waterworks field. However, Scobey's formula is more widely used in the design of penstocks and is expressed as:

$$H_f = K_s \frac{V^{1.9}}{D^{1.1}}$$

where: H_f = head loss in feet per 1,000 feet of pipe
K_s = loss coefficient
V = mean velocity (fps)
D = pipe diameter (ft.)

Values of K_s vary depending on the interior surface condition and range from 0.32 to 0.40. The loss coefficient of 0.32 may be used for penstocks with a newly applied lining. To allow for some deterioration of the lining a value of K_s = 0.34 is usually assumed in the design of penstocks whose interior is readily accessible for periodic inspection and lining maintenance. For penstocks too small to permit entering for inspection and lining maintenance, a value of K_s = 0.40 is usually assumed. Friction losses for various flows and pipe diameters computed from Scobey's formula using K_s = 0.34 are shown in Figure 1.1.1.

In addition to straight pipe frictional losses, the various other losses which occur are as follows:

1. Trashrack losses
2. Entrance losses
3. Bend losses
4. Valve losses
5. Manifold or fitting losses

FRICTION LOSS - FEET PER 1000 FEET

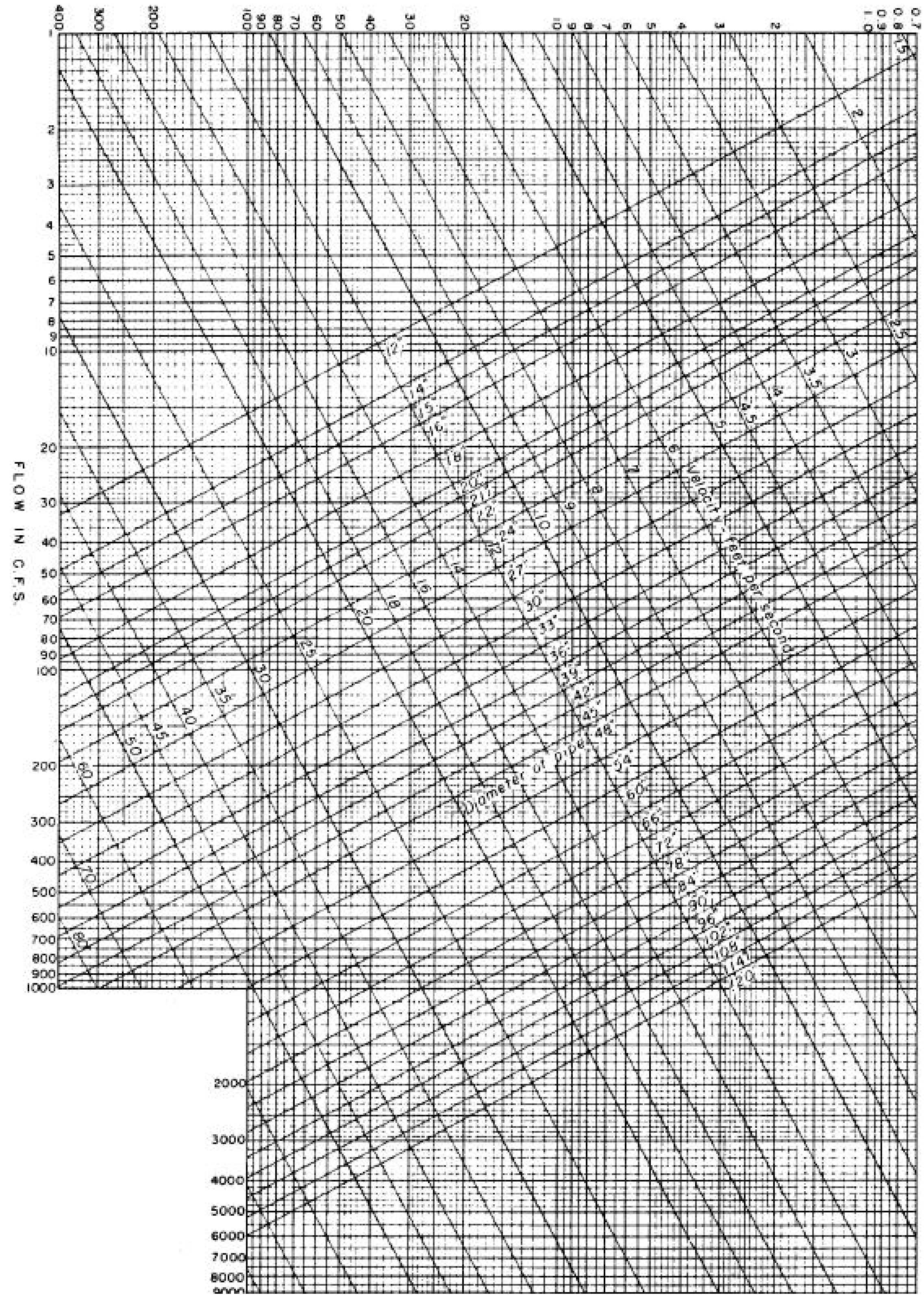


Figure 1.1.1 - Friction losses in welded steel pipe based on Scobey's formula, $K_s = 0.34$.

Losses through intake trashracks vary according to the velocity of flow through the trashrack and may be taken as 0.1, 0.3, and 0.5 foot, respectively, for velocities of 1.0, 1.5, and 2.0 feet per second.

Entrance losses depend on the shape of the intake opening. A circular bellmouth entrance is considered to be the most efficient form of intake if its shape is properly proportioned. It may be formed in the concrete with or without a metal lining at the entrance. The most desirable entrance curve was determined experimentally from the shape formed by the contraction of a jet (vena contracta) flowing through a sharp-edged orifice. For a circular orifice, maximum contraction downstream from the orifice occurs at a distance of approximately one-half its diameter. Losses in circular bellmouth entrances are estimated to be 0.05 to 0.1 of the velocity head. For square bellmouth entrances, the losses are estimated to be 0.2 of the velocity head.

Bend losses vary according to the shape of the bend and the condition of the inside surface. Mitered bends constructed from plate steel no doubt cause greater losses than smooth curvature bends formed in castings or concrete; however, there is no way to evaluate such effects since data on actual installations are very meager.

Thoma's formula (4) is based on experiments using 1.7-inch-diameter smooth brass bends having Reynolds numbers up to 225,000, and is expressed as:

$$Hg = C\frac{V^2}{2g}$$

where: H_b = bend loss in feet
C = experimental loss coefficient, for bend loss
V = velocity of flow in feet per second
g = acceleration due to gravity

The losses in figure 1.1.2 vary according to the R/D ratio and the deflection angle of the bend. An R/D ratio of six results in the lowest head loss, although only a slight decrease is indicated for R/D ratios greater than four. As the fabrication cost of a bend increases with increasing radius and length, there appears to be no economic advantage in using R/D ratios greater than five.

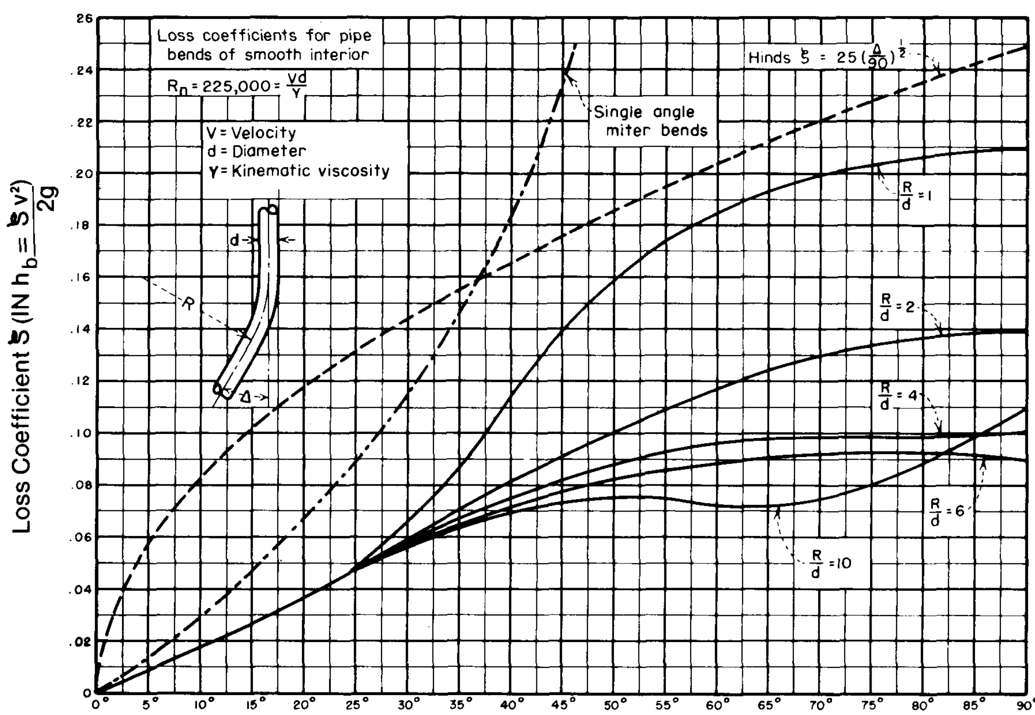


Figure 1.1.2 - Loss coefficients for pipe bends of smooth interior.

Head losses in gates and valves vary according to their design, being expressed as:

$$H_g = K\frac{V^2}{2g}$$

in which K is an experimental loss coefficient whose magnitude depends upon the type and size of gate or valve and upon the percentage of opening. Because gates or valves placed in penstocks are not throttled (this being accomplished by the wicket gates of the turbines), only the loss which occurs at the full open condition needs to be considered.

For gate valves, an average value of K = 0.10 is recommended; for needle valves, K = 0.20; and for butterfly valves with a ratio of leaf thickness to diameter of 0.2, a value of K = 0.26 may be used. For sphere valves having the same opening as the pipe, there is no reduction in area and the head loss is negligible.

Manifold or fittings should be designed with as smooth and streamlined interiors as practicable, since this results in the least loss in head. Data available on losses in large fittings are meager. For smaller fittings, as used in municipal water systems, the American Water Works Association recommends the following values for loss coefficients, K: for reducers, 0.25 (using velocity at smaller end); for increasers, 0.25 of the change in velocity head; for right angle tees, 1.25; and for wyes, 1.00. These coefficients are average values and are subject to wide variation for different ratios between flow in main line and branch outlet. They also vary with different tapers, deflection angles, and streamlining. Model tests made on small tees and branch outlets at the Munich Hydraulic Institute show that for fittings with tapered outlets and deflection angles smaller than 90° with rounded corners, losses are less than in fittings having cylindrical outlets, 90° deflections, and sharp corners. (See Figure 1.1.3.) These tests have served as a basis for the design of the branch connections for many penstock installations.

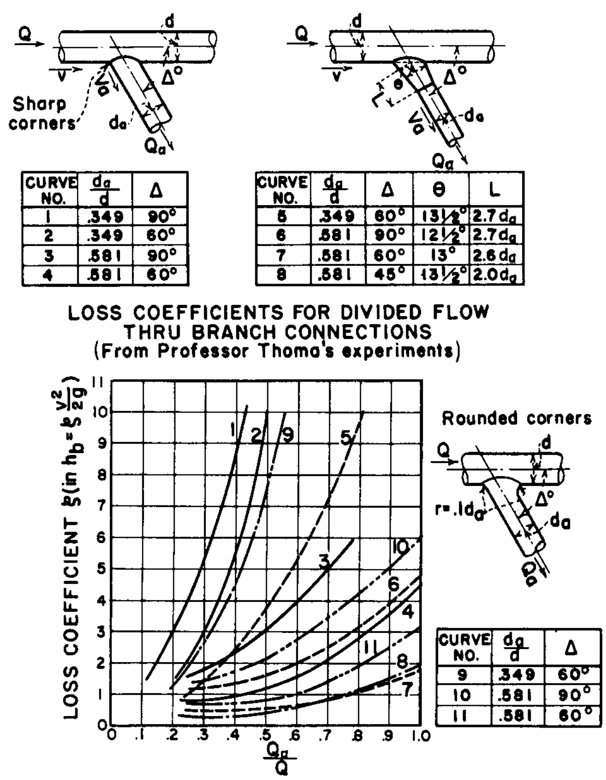


Figure 1.1.3 - Loss coefficients for divided flow through branch connections.

Section 1.2 Economic Diameter

A penstock is designed to carry water to a turbine with the least loss of head, consistent with the overall economy of the installation. An economic study to determine the penstock size generally requires that the annual penstock cost plus the value of power lost in friction be minimal. The annual penstock cost includes amortization of all related construction costs, operation and maintenance costs, and replacement reserve. A precise analytical evaluation, taking all factors into account, may be neither justified nor practical, since all variables entering into the problem are subject to varying degrees of uncertainty. Several formulas or procedures (1), (2), (7) have been found convenient and practical to use in estimating economic penstock diameters. After an economic diameter has been tentatively selected, based on monetary considerations, this diameter must be compatible with all existent design considerations. An example would be an installation where the economic diameter would require the installation of a surge tank for regulation, but an overall more economical installation would be to install a considerably larger penstock in lieu of the surge tank.

Figure 1.2.1 was derived from the method presented by Voetsch and Fresen (1) and Figure 1.2.2 is an example of its use. Special attention must be given to the "load factor," Figure 1.2.1, as this item materially affects the calculations.

The "step-by-step" method should be considered for the final design of long penstocks, in which case it is frequently economical to construct a penstock of varying diameters.

Section 1.3 Water Hammer

"Water Hammer" is a term applied to the phenomenon produced when the rate of flow in a conduit is rapidly changed. It consists of the development of a series of positive and negative pressure waves, the intensity of which is proportional to the spread of the propagation and the rate at which the velocity of flow is decelerated or accelerated. Joukovsky's fundamental equation, which is based on the elastic water column theory, gives the maximum increase in head for closure times of less than 2L/a seconds:

$$\Delta H = \frac{av}{g}$$

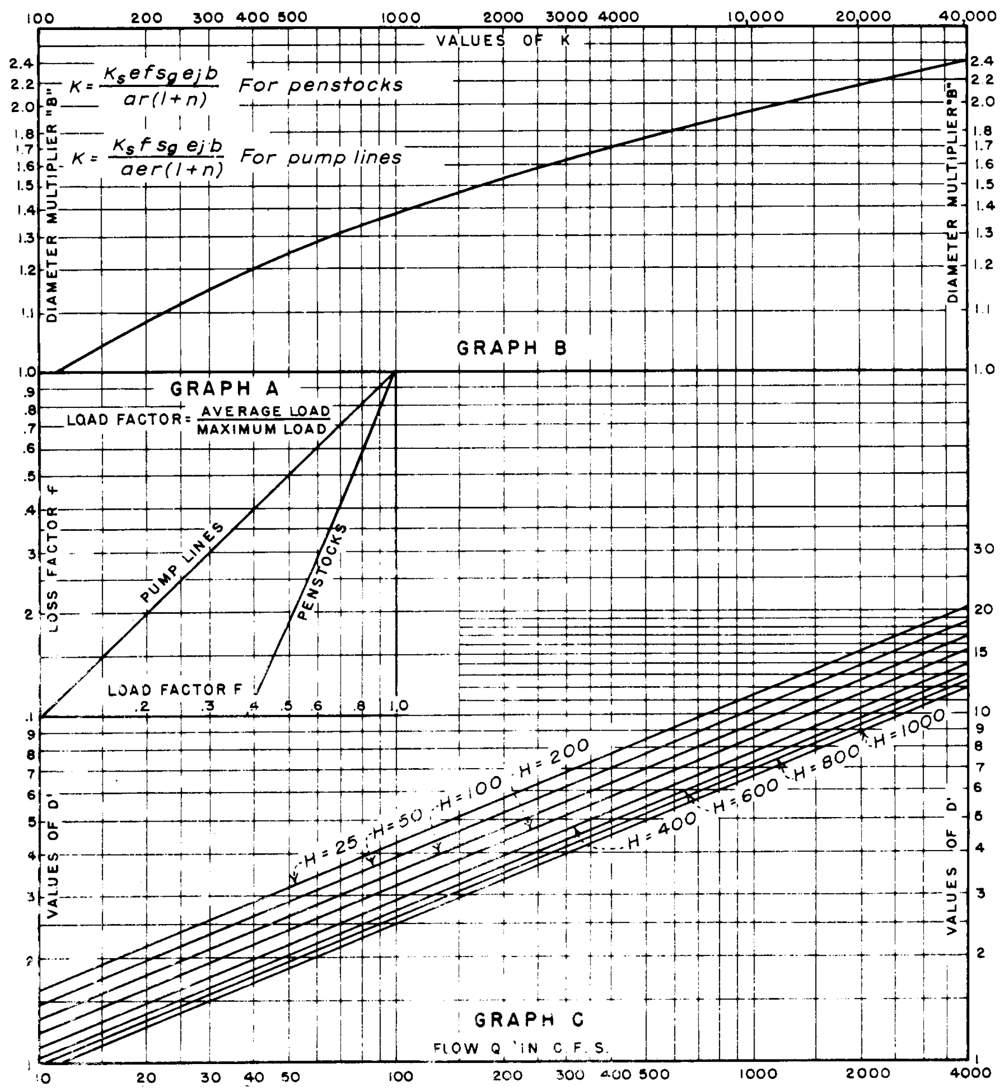
in which: ΔH = maximum increase in head - feet
a = velocity of pressure wave – ft/sec
v = velocity of flow destroyed – ft/sec
g = acceleration due to gravity – ft/sec^2
L = length of penstock from forebay to turbine gate – feet

From this formula, Allievi, Gibson, Durand, Quick, and others developed independent equations for the solution of water hammer problems (8).

A comprehensive account of methods used for the analysis of water hammer phenomena occurring in water conduits, including graphical methods, was published by Parmakian (8). In this reference, the graphical method of analysis provides a method for determining the head changes at various points in a pipeline for any type of gate movement. When the effective gate opening varies uniformly with respect to time, the gate motion is called "uniform gate closure." Figure 1.3.1 shows R. S. Quick's chart for uniform gate closure to zero gate and a chart showing velocity of pressure wave in elastic water column. To determine maximum pressure rise at gate it is necessary to calculate time constant, $N = aT/2L$, and pipeline constant, $K = aV_o/29H_o$. Then the value of pressure rise, P, as a proportion of h_{max} is read from the chart. From the relation, $2KP = h/H_o$, the pressure rise, h, is calculated.

NOTATION

a = Cost of pipe per lb, installed, dollars.
B = Diameter multiplier from Graph B.
b = Value of lost power in dollars per kwh.
D = Economic diameter in feet.
e = Overall plant efficiency.
ej = Joint efficiency.
f = Loss factor from Graph A.
H = Weighted overage head including water hammer, feet.
K_s = Friction coefficient in Scobey's formula.
n = Ratio of overweight to wt. of pipe shell.
Q - Flow in cubic feet per second.
r = Ratio of annual cost to a.
S_g: Allowable tension, p.s.i.



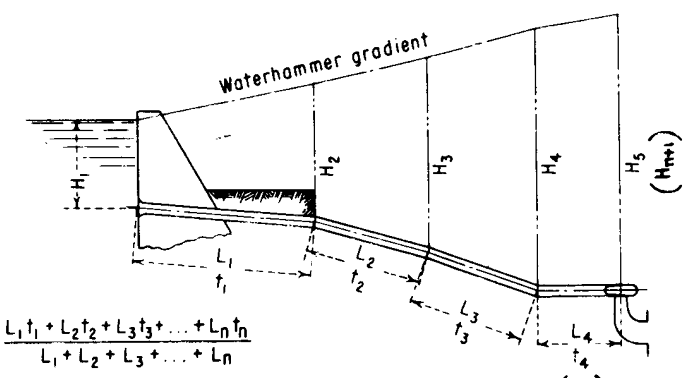
Use of chart: Obtain loss factor f from Graph A Compute K and obtain B from Graph B. Take D^1 from Graph C. The economic dia. is $D = B \times D^1$.

Figure 1.2.1 - Economic diameter for steel penstocks and pump lines

NOTATION

a = Cost of pipe per lb., installed, dollars.
B = Diameter multiplier from Graph B.
b = Value of lost power in dollars per kwh.
D = Economic diameter in feet.
e = Overall plant efficiency.
e_j = Joint efficiency.
f = Loss factor from Graph A.
H = Weighted average head including water hammer. (based on design head)
K_s = Friction coefficient in Scobey's formula (0.34).
n = Ratio of overweight to weight of pipe shell.
Q = Flow in cubic feet per second. (at design head of turbine)
r = Ratio of annual cost to "a" (see explanation)
s_g = Allowable tension, p.s.i.
t = Weighted average plate thickness (at design head) for total length.
t_n = Average plate thickness for length L_n.

EXPLANATION AND EXAMPLE



$$t = \frac{L_1 t_1 + L_2 t_2 + L_3 t_3 + \ldots + L_n t_n}{L_1 + L_2 + L_3 + \ldots + L_n}$$

$$H = \frac{\left(\frac{H_1+H_2}{2}\right)L_1 + \left(\frac{H_2+H_3}{2}\right)L_2 + \left(\frac{H_3+H_4}{2}\right)L_3 + \left(\frac{H_n+H_{n+1}}{2}\right)L_n}{L_1 + L_2 + L_3 + \ldots + L_n}$$

$$\%\ O.\&\ M. = \frac{O.\&\ M.\ \text{per foot}}{\text{Cost per foot of pipe}}$$

O. & M. = Cost of maintaining interior and exterior surface area of pipe (inside surface area for embedded pipe, inside and outside surface area for exposed pipe) per year.

Depreciation =

r = Interest + Depreciation + % O. & M.

Example for penstock **Q = 128 CFS**

ASSUME

Dia. = 5'-0" Avg. plate = $\frac{1}{4}$"
Value of power per kwh = $0.005 = b
Cost of steel pipe installed = $0.27/lb = a
load factor (see Graph A) = 0.75 (f = 0.510)
Interest = 3% n = 0.15
Depreciation = 0.005135
O. & M. = $0.02 per sq. ft.

$L_1 = 225'$; $L_2 = 150'$; $L_3 = 150'$; $L_4 = 100'$

$H_1 = 80'$; $H_2 = 120'$; $H_3 = 170$; $H_4 = 230'$; $H_5 = 240'$

$$H = \frac{\left(\frac{H_1+H_2}{2}\right)L_1 + \left(\frac{H_2+H_3}{2}\right)L_2 + \left(\frac{H_3+H_4}{2}\right)L_3 + \left(\frac{H_4+H_5}{2}\right)L_4}{L_1 + L_2 + L_3 + L_4} = 156.4'$$

$$O.\&M. = \frac{[(225 \times 15.71) + (400 \times 15.71 \times 2)]0.02}{625} = \$0.515$$

Weight / foot = 15.71 x 10.2 = 160.24 lbs.
Cost / foot = 160.24 x 0.27 = $43.26

$$\%\ O.\&\ M. = \frac{0.515}{43.26} = 0.0119$$

r = 0.03 + 0.005135 + 0.0119 = 0.0470

$$K = \frac{K_s e f s_g e_j b}{ar(1+n)} = \frac{0.34 \times 0.85 \times 0.510 \times 15{,}000 \times 0.90 \times 0.005}{0.27 \times 0.0470 \times 1.15} = 680$$

B = 1.285 (from Graph B); D' = 3.7 (from Graph C); Economic dia. = 1.285 x 3.7 = 4.75 (use 4'-10" dia.)

NOTE: Calculated economic diameter should be very close to assumed diameter as it is in this example. The problem should be reworked until this condition exists. Depreciation is based on the accumulation of an annual sinking fund earning 3% interest required to replace 50% of the pipe in 45 years. The annual payment required is equal to 0.005135 times the first cost.

Figure 1.3.1 – Sample calculation of economic diameter of penstocks

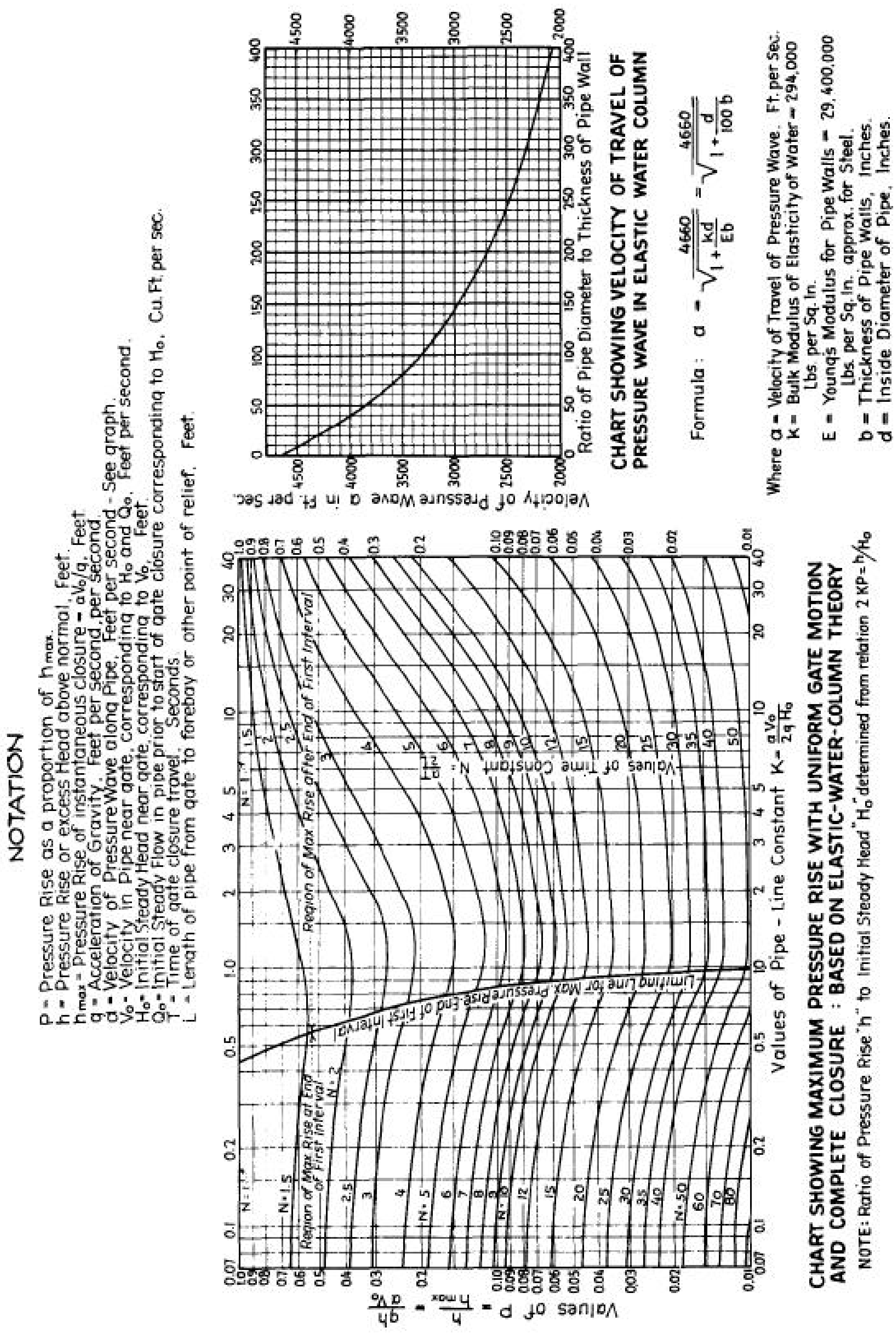


Figure 1.3.1 - Water hammer charts

CHAPTER 2

Materials

Section 2.1 General

GENERAL: All steel materials used in the fabrication of penstocks, including pressure-carrying components and non-pressure carrying attachments such as ring girders, stiffener rings, thrust rings, lugs, support systems, and other appurtenances shall be manufactured and tested in strict accordance with appropriate ASTM Specifications and as specified herein.

The properties of steels are governed by their chemical composition, by the processes used to transform the base metal into the shape, and by their heat treatment. The effects of these parameters on the properties of steels are discussed in the following sections.

CHEMICAL COMPOSITION: Constructional steels are a mixture of iron and carbon with varying amounts of other elements-primarily manganese, phosphorus, sulfur, and silicon. These and other elements are either unavoidably present or intentionally added in various combinations to achieve specific characteristics and properties of the finished steel products. The effects of the commonly used chemical elements on the properties of hot-rolled and heat-treated carbon and alloy steels are presented in Table 2.1. The effects of carbon, manganese, sulfur, silicon, and aluminum are of primary interest to the present discussion.

Carbon is the principal hardening element in steel where each additional increment increases the hardness and tensile strength of the steel. Carbon has a moderate tendency to segregate and increased amounts of carbon cause a decrease in ductility, toughness, and weldability.

Manganese increases the hardness and strength of steels but to a lesser degree than does carbon. Manganese combines with sulfur to form manganese sulfides thus decreasing the harmful effects of sulfur.

Carbon (C)
Principal hardening element in steel
Increases strength and hardness
Decreases ductility, toughness, and weldability
Moderate tendency to segregate
Manganese (Mn)
Increases strength
Controls harmful effects of sulfur
Phosphorus (P)
Increases strength and hardness
Decreases ductility and toughness
Considered an impurity, but sometimes added for atmospheric corrosion resistance
Strong tendency to segregate

Sulfur (S)
Considered undesirable except for machinability
Decreases ductility, toughness and weldability
Adversely affects surface quality
Strong tendency to segregate
Silicon (SI)
Used to deoxidize or “kill” molten steel
Aluminum (AI)
Used to deoxidize or “kill” molten steel
Refines grain size, thus increasing strength and toughness
Vanadium (V) and Columbium (Nb)
Small additions increase strength

Nickel (Ni)
Increases strength and toughness
Chromium (Cr)
Increases strength
Increases atmospheric corrosion resistance
Copper (CU)
Primary contributor to atmospheric corrosion resistance
Nitrogen (N)
Increases strength and toughness
Decreases ductility and toughness
Boron (B)
Small amounts (0.0005%) increase hardenability in quenched and tempered Steels
Used only in aluminum-killed steels
Most effective at low carbon levels

Sulfur is generally considered an undesirable element except where machinability is an important consideration. Sulfur adversely affects surface quality, has a strong tendency to segregate, and decrease ductility, toughness, and weldability.

Silicon and aluminum are the principal deoxidizers used in the manufacture of carbon and alloy steels. Aluminum is also used to control and refine grain size.

CASTING: The traditional steelmaking process is the one where molten steel is poured (teemed) into a series of molds to form castings known as ingots. The ingots are removed from the molds, reheated, then rolled into products with square or rectangular cross sections. This hot-rolling operation elongates the ingot and produces semi-finished products known as blooms, slabs, or billets. All ingots exhibit some degree of nonuniformity of chemical composition known as segregation, which is an inherent characteristic of the cooling and solidification of the molten steel in the mold.

The first liquid steel to contact the relatively cold walls and bottom of the mold solidifies very rapidly having the same chemical composition as the liquid steel entering the mold. However, as the rate of solidification decreases away from the mold sides, crystals of relatively pure iron solidify first. Thus, the first crystals to form contain less carbon, manganese, phosphorus, sulfur and other elements than the liquid steel from which they were formed. The remaining liquid is enriched by these elements that are continually being rejected by the advancing crystals. Consequently, the last liquid to solidify, which is located around the axis in the top half of the ingot, contains high levels of the rejected elements and has a lower melting point than the poured liquid steel. This segregation of the chemical elements is frequently expressed as a local departure from the average chemical composition. In general, the content of an element that has a tendency to segregate is greater than average at the center of the top half of an ingot and less than average at the bottom half of an ingot.

Certain elements tend to segregate more readily than others. Sulfur segregates to the greatest extent. The following elements also segregate, but to a lesser degree, and in descending order: phosphorus, carbon, silicon, and manganese. The degree of segregation is influenced by the composition of the liquid steel, the liquid temperature and ingot size. The most severely segregated areas of the ingot are removed by cropping, which is cutting discarding sufficient material during rolling.

CONTINUOUS CASTING: The direct casting of steel from the ladle into slabs. This steelmaking development bypasses the operations between molten steel and the semi-finished product which are inherent in making steel products from ingots. In continuous casting, molten steel is poured at a regulated rate into the top of an oscillating water-cooled mold with a cross-sectional size corresponding to the desired slab. As the molten metal begins to solidify along the mold walls, it forms a shell that permits the gradual withdrawal of the strand product from the bottom of the mold into a water-spray chamber where solidification is completed. The solidified strand is cut to length and then reheated and rolled into finished projects as in the conventional ingot process. The smaller size and higher cooling rates for the strand result in less segregation and greater uniformity in composition and properties for steel products made by the continuous casting process than for ingot products.

KILLED AND SEMI-KILLED STEELS: The primary reaction involved in most steelmaking processes is the combination of carbon and oxygen to form carbon monoxide gas. The solubility of this and other gases dissolved in the steel decreases as the molten metal cools to the solidification temperature range. Thus, excess gases are expelled from the metal and, unless controlled, continue to evolve during solidification. The oxygen available for the reaction can be eliminated and the gaseous evolution inhibited by deoxidizing the molten steel using additions of silicon or aluminum or both. Steels that are strongly deoxidized do not evolve any gases and are called killed steels because they lie quietly in the mold. Increasing amounts of gas evolution results in semi-killed, capped or rimmed steels.

In general, killed steel ingots are less segregated and contain negligible porosity when compared to semi-killed steel ingots. Consequently, killed steel products usually exhibit a higher degree of uniformity in composition and properties than semi-killed steel products.

HEAT TREATMENTS FOR STEELS: Steels respond to a variety of heat treatments that can be used to obtain certain desirable characteristics. These heat treatments can be divided into slow cooling treatments and rapid cooling treatments. The slow cooling treatments, such as annealing, normalizing and stress relieving, decrease hardness and promote uniformity of structure. Rapid cooling treatments, such as quenching and tempering, increase strength, hardness and toughness. Heat Treatments of base metal are generally mill options or ASTM requirements.

Annealing - Annealing consists of heating the steel to a given temperature followed by slow cooling. The temperature, the rate of heating and cooling, and the time the metal is held at temperature depend on the composition, shape and size of the steel product being treated and the desired properties. Usually steels are annealed to remove stresses; to induce softness; to increase ductility and toughness; to produce a given microstructure; to increase uniformity of microstructure; to improve machinability; or to facilitate cold forming.

Normalizing - Normalizing consists of heating the steel to between 1650ºF and 1700ºF followed by slow cooling in air. This heat treatment is commonly used to refine the grain size, improve uniformity of microstructure, and improve ductility and fracture toughness.

Stress Relieving - Stress relieving of carbon steels consists of heating the steel in the range 1000 to 1200ºF and holding for a proper time to equalize the temperature throughout the piece followed by slow cooling. The stress relieving temperature for quenched and tempered steels must be maintained below the tempering temperature for the product. Stress relieving is used to relieve internal stresses induced by welding, normalizing, cold working, cutting, quenching and machining. It is not intended to alter the microstructure of the mechanical properties significantly.

Quenching and *Tempering* - Quenching and tempering consists of heating and holding the steel at the proper austenitizing temperature (about 1650ºF) for a significant time to produce a desired change in microstructure, then quenching by immersion in a suitable medium (water for bridge steels). After quenching, the steel is tempered by reheating to an appropriate temperature usually between 800 and 1200ºF, holding for a specified time at temperature, then cooling under suitable conditions to obtain the desired properties. Quenching and tempering increases the strength and improves the toughness of the steel.

Controlled Rolling - Controlled rolling is a thermo-mechanical treatment at the rolling mill that tailors the time-temperature-deformation process by controlling the rolling parameters. The parameters of primary importance are (1) the temperature at start of controlled rolling in the finishing stand, (2) the percentage reduction from start of controlled-rolling to the final plate thickness, and (3) the plate finishing temperature.

Hot-rolled plates are deformed as quickly as possible at temperatures above about 1800ºF to take advantage of the hot workability of the steel at high temperatures. In contrast, controlled rolling incorporates a hold or delay time to allow the partially rolled slab to reach a desired temperature before start of final rolling. Controlled rolling involves deformations at temperatures in the range from 1500 and 1800ºF. Because rolling deformation at these low temperatures increases the mill loads significantly, controlled rolling is usually restricted to less than two-inch-thick plates. Controlled rolling increases the strength, refines the grain size, improves the toughness, and may eliminate the need for normalizing.

Controlled Finishing - Temperature Rolling - Controlled finishing-temperature rolling is a less severe practice than controlled rolling and is aimed primarily at improving notch toughness of plates up to 2-½inch thickness. The finishing temperatures in this practice (about 1600ºF) are higher than required for controlled rolling. However, because heavier plates are involved than in controlled rolling, mill delays are still required to reach the desired finishing temperatures. By controlling the finishing temperature, fine grain size and improved notch toughness can be obtained.

MECHANICAL PROPERTIES: Mechanical properties of a material are those properties that characterize its elastic and inelastic (plastic) behavior under stress or strain. These properties include parameters that are related to the material’s strength, ductility, and hardness. The strength and ductility parameters under tensile loading can be defined and explained best by analyzing the tensile stress-strain curve for the material.

Stress-Strain Curves - Figure 2.1 presents a schematic of an idealized tensile stress-strain curve for constructional steels. Such a curve is obtained by tensile loading to failure specimens, which have either rectangular or circular cross sections. A detailed discussion of definitions and details for tension test specimens and test methods are available in ASTM A370.

The curve shown in Figure 2.1 is an engineering tensile stress-strain curve, as opposed to a true tensile stress-strain curve, because the plotted stresses are calculated by dividing the instantaneous load on the specimen by its original, rather than reduced, cross-sectional area. Also, the strains are calculated by dividing the instantaneous elongation of a gage length of the specimen by the original gage length.

The initial straight line segment of the stress-strain curve represents the elastic behavior of the specimen where stress is linearly related to strain. In this region the strain is fully recoverable and the specimen returns to its original length when the load is removed. The slope of the line, which is the ratio of stress and strain in the elastic region, is the MODULUS OF ELASTICITY, or YOUNG’S MODULUS, and is approximately equal to 30×10^6 psi for constructional steels. As the load increases, the stresses and strains become nonlinear and the specimen experiences permanent plastic deformation. The stress corresponding to the initial deviation from linearity represents the YIELD STRENGTH of the material and the beginning of the plastic region. Usually, the stress required to produce additional plastic strain increases with increasing strain thus the steel strain hardens. The rate at which stress increases with plastic strain is the STRAIN-HARDENING MODULUS.

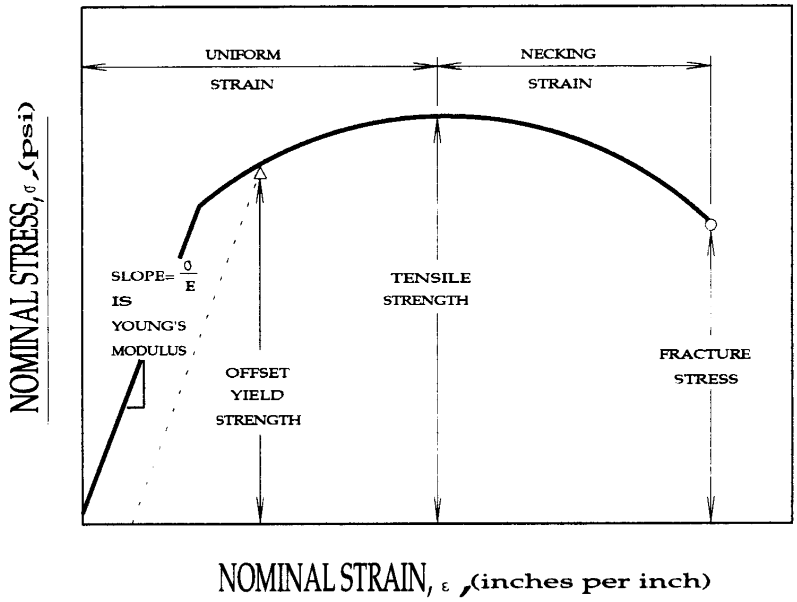


Figure 2.1 - Schematic stress-strain curves for steels

Yield Strength and Tensile Strength - The tensile stress-strain curves for constructional steels can be divided into two types which exhibit different behavior in the plastic region, Figure 2.2. One of these curves, Figure 2.2 (a), exhibits a smooth deviation from linearity and the stress continuously increases to a maximum value then decreases until the specimen fractures. On the other hand, the stress for the other curve, Figure 2.2 (b), reaches a peak immediately after the stress-strain curve deviates from linearity, dips slightly, and then remains at a constant value for a considerable amount of additional strain. Thereafter, the steel strain hardens and the stress increases with strain to a maximum then decreases until the specimen fractures. The stress corresponding to the peak value represents the YIELD STRENGTH is the stress at which the material exhibits a specific limiting deviation from linearity of stress and strain. The deviation may be expressed as a 0.2% offset or a 0.005 inch/inch total extension under load, Figure 2.2 (a).

The maximum stress exhibited by the engineering stress-strain curve corresponds to the TENSILE STRENGTH of the steel.

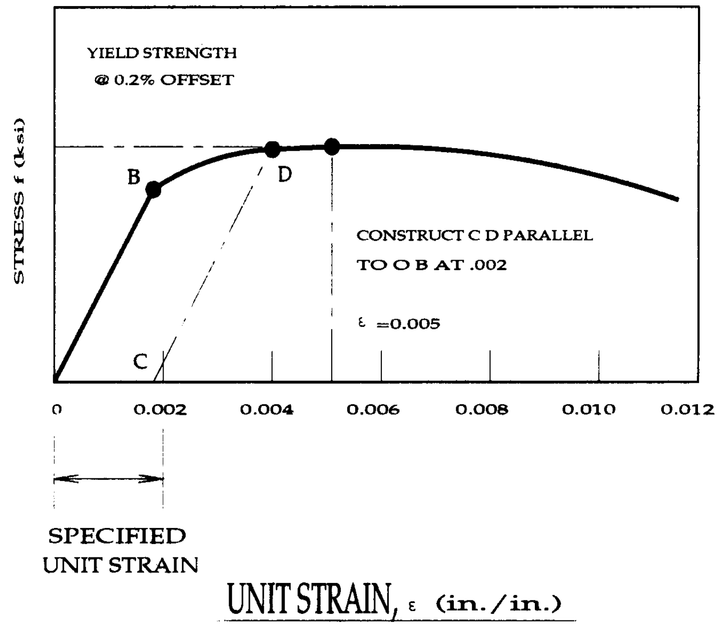
Ductility and Toughness - The tensile stress-strain curve can be divided into a uniform strain region and a non-uniform strain region which combine to give the total strain to fracture, Figure 2.1. In the uniform strain region, the cross-sectional area along the entire gage length of the specimen decreases uniformly as the specimen elongates under load. Initially, the strain hardening compensates for the decrease in cross-sectional area and the engineering stress continues to increase with increasing strain until the specimen reaches its ultimate tensile strength. Beyond this point the plastic strain becomes localized in a small region of the gage length and the specimen begins to neck locally with a corresponding decrease in total stress until the specimen fractures

The total percent ELONGATION and the total percent REDUCTION OF AREA at fracture are two measures of ductility that are obtained from the tension test. The percent elongation is calculated from the difference between the initial gage length and the gage length after fracture. Similarly, the percent reduction of area is calculated from the difference between the initial and the final cross-sectional area after fracture. Both elongation and reduction of area are influenced by gage length and specimen geometry. The conversion between elongation of 8-inch gage length strap specimen and 2-inch gage length round specimen can be found in ASTM A370.

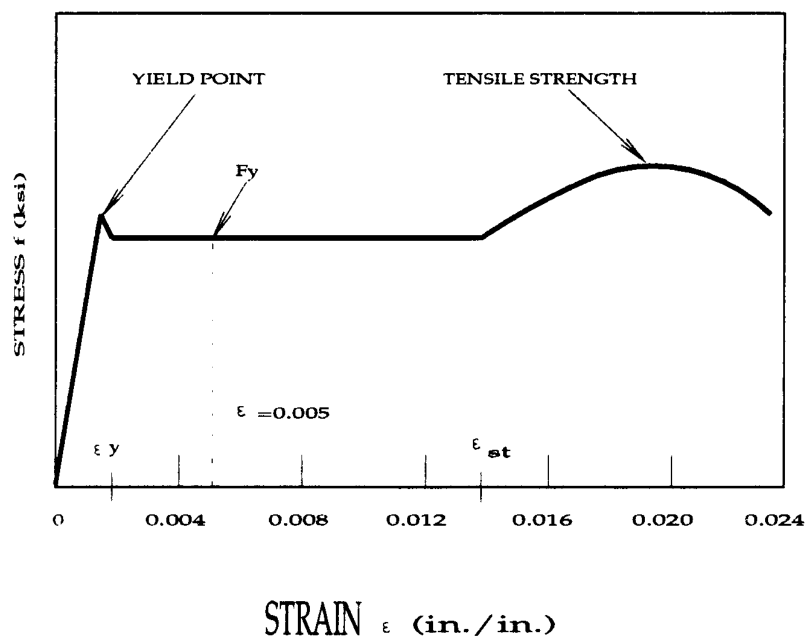
Ductility is an important material property because it allows the redistribution of high local stresses. Such stresses occur in welded connections and at regions of stress concentration such as holes and changes in geometry.

TOUGHNESS is the ability of a material to absorb energy prior to fracture and is related to the area under the stress-strain curve. The larger the area under the curve the tougher is the material.

Tensile Properties - The procedures and definitions for the tension test methods are presented in ASTM Standards A370. The general specifications and tolerances for acceptability of structural steel plates are presented in ASTM A6.



(a)



(b)

Figure 2.2 - Tensile stress-strain curve

Producers are aware of the chemical segregation and the variability in the properties of steel products. Consequently, based on their experience, they establish aim chemical compositions that would ensure meeting the requirements of the material specifications. Because of the recognized variations, the aim chemistries and processing practices are usually selected to result in properties that would exceed the minimum properties required by the material specifications.

FRACTURE TOUGHNESS: Most constructional steels can fracture either in a ductile or in a brittle manner. The mode of fracture is governed by the temperature at fracture, the rate at which the loads are applied and the magnitude of the constraints that would prevent plastic deformation. The effects of these parameters on the mode of fracture are reflected in the fracture-toughness behavior of the material. In general, the fracture toughness increases with increasing temperature, decreasing load rate and decreasing constraint. Furthermore, there is no single unique fracture-toughness value for a given steel even at a fixed temperature and loading rate.

Traditionally, the fracture toughness for low and intermediate-strength steels has been characterized, primarily by testing Charpy V-notch (CVN) specimens at different temperatures. However, the fracture toughness for materials can be established best by using fracture-mechanics test methods. The following presents a few aspects of fracture toughness of steels by using CVN and fracture-mechanics test results.

Charpy V-Notch Fracture Toughness - The Charpy V-notch impact specimen has been the most widely used specimen for characterizing the fracture-toughness behavior of steels. These specimens may be tested at different temperatures and the impact fracture toughness at each test temperature may be determined from the energy absorbed during fracture, the percent shear (fibrous) fracture on the fracture surface or the change in the width of the specimen (lateral expansion). At low temperatures, constructional steels exhibit a low value of absorbed energy (about 5 ft-lb), and zero fibrous fracture and lateral expansion. The values of these fracture toughness parameters increase as the test temperature increases until the specimens exhibit 100 percent fibrous fracture and reach a constant value of absorbed energy and of lateral expansion. This transition from brittle-to-ductile fracture behavior usually occurs at different temperatures for different steels and even for a given steel composition. Consequently, like other fracture-toughness tests, there is no single unique CVN value for a given steel, even at a fixed temperature and loading rate. Therefore, when fracture toughness is an important parameter, the design engineer must establish and specify the necessary level of fracture toughness for the material to be used in the particular structure or in a critical component within the structure.

Section 2.2 Material Specifications

PLATE: Steel plate is manufactured to two standards, ASTM-A6 for structural use and ASTM-A20 for pressure vessels. The physical and chemical properties of steel produced to these two standards can be very similar, particularly when “Supplemental Requirements” are specified. They differ, however, in the amount of testing required to assure uniform quality.

The individual ASTM Standards for plates gives the design engineer a wide range of properties from which to select the economical material for a particular application. Structural steel (A-6) is suitable for ring girders, stiffener rings, thrust rings, lugs, support systems and for the pipe shell in many of the less critical penstocks where design stress does not exceed 21,000 psi.

Pressure vessel quality plates (A-20) are normally used in the fabrication of the pipe shell for high pressure, large diameter penstocks. They may also be used for crotch plates, ring girders or other structural parts if desired.

COILS: Coils produced on today’s modern rolling mills from continuous cast slab, have proven to have the consistent chemical and mechanical properties throughout the coil required by ASTM A20 Standard. At this time the thickness of coil is limited to 1” and heat treatment of the coil (normalized or quenched and tempered) is not available.

When spiral weld pipe is produced directly from coils all applicable portions of ASTM A6 or ASTM A20 shall apply except that the testing provisions for chemical and physical properties shall be revised as follows:

The mill producing the coil shall furnish certified chemical analysis of each heat. All required physical tests may be taken by the pipe manufacturer from the coil or from the completed pipe. The spiral weld pipe manufacturer shall furnish certified reports of the physical tests. To qualify under ASTM A6, two sets of the required physical tests shall be taken for each heat or each 50 tons of each heat. To qualify under ASTM A20 one set of the required physical tests shall be taken from each coil used.

A set of physical tests shall consist of 3 tests on a coil; one from the outside wrap of the coil, one from the middle third of the coil and one from the inner wrap adjacent to the portion of the coil that is used. The set from the middle portion of the coil may be taken from the finished pipe. The orientation of tests shall be the longitudinal axis of the test specimens shall be transverse to the final rolling direction of the coil.

Steel coil, used in the manufacture of spiral welded steel pipe, may be used provided that it meets either PVQ (ASTM A-20) or SIR steel (ASTM A-6 as defined above).

TEMPERATURE CONSIDERATIONS: The toughness of steel at low temperatures must be considered in penstock design. This toughness is usually measured by charpy impact tests.

A buried penstock will have a lowest service temperature (LST) of about 30ºF. An above ground penstock will also normally have a lowest service temperature of close to 30ºF for the pipe shell. Structural support systems and ring girders and empty pipe may be subjected to lower temperatures.

"Fine Grain, Killed Steel, with a 55 k.s.i. minimum yield or less, in a thickness of 5/8" or less, will have adequate toughness when operating at a lowest service temperature of 20ºF or higher. Normalized and Quenched and Tempered Steels will have improved toughness at even lower temperatures.

Section 2.2.1 Attachment Materials and Criteria

NOZZLES: All openings in penstock shell shall be reinforced in accordance with ASME B&PV Code Section VIII, Division 1. Suitable materials are:

Forged steel weld couplets, weldolets, sweepolets, or other fittings conforming to ANSI Bl6.11 of material conforming to ASTM A- 105 Grade 2.

Steel pipe listed in Table 3.1.2 or fabricated in accordance with Section 7 MANUFACTURE.

BENDS, TEES, REDUCERS AND CAPS: Forged steel weld fittings shall be in accordance with ANSI B16.9 conforming to ASTM A234 of the schedule and grade required.

Fabricate miter type elbows, tees and reducers in accordance with Section 3.7 into pipe fabricated in accordance with Section 7 MANUFACTURE.

DISHED HEADS: Heads may be ellipsoidal, torispherical, hemispherical, conical or toriconical designed in accordance with ASME B&PV Code, Section VIII, Division 1. using materials listed in Table 3.1.2.

ATTACHMENTS: Non-pressure carrying attachments such as ring girders, stiffener rings, thrust rings, lugs, support systems and other appurtenances may be fabricated from any of the materials listed in Table 3.1.2.

MATERIAL SPECIFICATION: Table 3.1.2 summarizes some of the steels commonly used in the manufacture of steel penstocks. These materials have been specified with or without supplemental requirements.

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CHAPTER 3

Design

Section 3.1 Materials and Allowable Stresses

For the AWWA M-11 design, an allowable design stress of 50% of yield at (1) working pressure and 75% of yield at (2) transient pressure is suggested. These values are given in Table 3.1.2.

For the AWWA design (1) "working pressure" is defined as the vertical distance between the penstock centerline and the hydraulic grade line or the static head, whichever is greater, times .434 to convert feet of head to pressure in pounds per square inch. (2) "Transient pressure" is defined as the static head plus the water hammer and surge for a plant load rejection when all units are operating, with normal governor closure time. Field test pressures shall not exceed this pressure.

For the traditional design, an allowable design stress of 2/3 of yield or 1/3 of tensile at (1) normal operating condition is suggested. At (2) intermittent condition an allowable design stress of .8 of yield or .44 of tensile is suggested. These values are given in Table 3.1.2.

For the Traditional design (l), "normal operating condition" is defined as the maximum static head plus the water hammer and surge for a plant load rejection when all units are operating with normal governor closure time. (2) "Intermittent condition" is defined as condition during filling and draining and earthquake during normal operation.

Design is based on circumferential (hoop) stress or combined equivalent stress, circumferential and longitudinal, calculated in accordance with Hencky-Mises Theory, whichever is greater.

TABLE 3.1.1- BASIC CONDITIONS FOR INCLUDING THE EFFECTS OF WATER HAMMER IN THE DESIGN OF STEEL PENSTOCKS

The basic conditions for including the effects of water hammer in the design of turbine penstock installations are divided into normal and intermittent conditions with suitable factors of safety assigned to each type of operation.

Normal conditions of operation	**Intermittent conditions of operation**
1. Normal Condition - This condition includes maximum static head plus pres sure rise due to normal operation. The recommended allowable stress is equal to 2/3 the specified minimum yield stress or 1/3 the minimum specified tensile strength, whichever is smaller.	2. Intermittent Condition – This includes conditions during, filling and draining of the penstock and earthquake during normal operation. The recommended allowable stress is equal to 0.8 times the specified minimum yield stress or 0.44 times the specified minimum tensile strength, whichever is smaller.

Section 3.2 Internal Pressure

With pressure determined, the wall thickness is found using the equation:

$$t = \frac{pd}{2s}$$

Where: t = wall thickness (in.)
p = pressure (psi)
d = outside diameter of pipe (in.) steel cylinder (not including coatings)
s = allowable stress (psi), for design condition.

TABLE 3.1.2 - MATERIALS & ALLOWABLE STRESSES

ASTM Designation	Min. Yield	Min. Tensile	Allowable Design Stresses KSI AWWA		Traditional	
	KSI	KSI	1	2	1	2
A36	36	58	18	27	19.3	25.5
A53-B	35	60	17.5	26.2	20	26.4
*A139-B	35	60	17.5	26.2	20	26.4
*A139-C	42	60	21	31.5	20	26.4
*A139-D	46	60	23	34.5	20	26.4
*A139-E	52	66	26	39	22	29
*.25 Max. Carbon						
A516-55	30	55	15	22.5	18.3	24
A516-60	32	60	16	24	20	25.6
A516-65	35	65	17.5	26.2	21.7	28
A516-70	38	70	19	28.5	23.3	30.4
A537 C1. 1 thru 2 1/2"	50	70			23.3	30.8
A573 C1. 1 2 1/2 – 4"	45	65			21.7	28.6
A537 C1. 2 thru 2 1/2"	60	80			26.7	35.2
A537 C1. 2 2 1/2 – 4"	55	75			25	33
A537 C1. 2 4 – 6"	46	70			22.3	30.8
**A570-30	30	49	15	22.5		
A570-33	33	52	16.5	24.7		
A570-36	36	53	18	27		
A570-40	40	55	20	30		
A570-45	45	60	22.5	33.7		
A570-50	50	65	25	37.5		
A570-55	55	70	27.5	41.2		
**Sheet Spec. .229" max thickness						
A572-42	42	60	21	31.5	20	26.4
A572-50, S81, S91: 3/4" & under	50	70	25	37.5	23.3	30.8
over 3/4"	50	65	25	37.5	21.7	28.6
A907-30	30	49	15	22.5	16.3	21.6
A907-33	33	52	16.5	24.7	17.3	22.9
A907-36	36	53	18	27	17.6	23.3
A907-40	40	55	20	30	18.3	24.2
A907-45	45	60	22.5	33.7	20	26.4
A907-50	50	65	25	37.5	21.7	28.6
ASTM 935-45	45	60	22.5	33.7	20	26.4
ASTM 935-50	50	65	25	37.5	21.7	28.6

Section 3.3 External Loads

EARTH LOAD DETERMINATION

Determine the load on the pipe by the "Soil Prism" theory, as follows.

$$W_e = \frac{\omega HD}{12}$$

where: W_e = vertical soil load, pounds per inch
ω = weight of earth per unit volume, $lbs/(ft)^3$
H = height of fill over pipe (feet)
D = outside diameter of pipe (feet)

LIVE LOAD DETERMINATION

Determine the appropriate live load on the pipe using Table 3.3.0.1 or Section 3.3.1 EXTREME EXTERNAL LOADING CONDITIONS.

$$W_L = \frac{\text{Load from table (psf) x D}}{12} = \text{Vertical live load, pounds per inch}$$

Allowable deflection for various lining and coating systems that are often accepted are:

Mortar-lined and coated = 2 percent of pipe diameter
Mortar-lined and flexible coated = 3 percent of pipe diameter
Flexible lining and coated = 5 percent of pipe diameter

TABLE 3.3.0.1

Highway HS-20 Loading* Height of Cover (Ft)	Load, psf	Railroad E-80 Loading* Height of Cover (Ft)	Load, psf
1	1800	2	3800
2	800	5	2400
3	600	8	1600
4	400	10	1100
5	250	12	800
6	200	15	600
7	176	20	300
8	100	30	100

*Neglect live load when less than 100 psf; use dead load only.

Live load effect is generally based on AASHTO HS-20 truckloads, or Cooper E-80 railroad loads as indicated in Table 3.3.0.1. These values are given in pounds per square foot and include 50% impact factor. It is noted that there is no live load effect for HS-20 loads when the earth cover exceeds 8 feet, and for E-80 loads when the earth cover exceeds 30 feet.

DEFLECTION DETERMINATION

The Iowa deflection formula has been frequently rearranged. In one of its most common forms, deflection is calculated as follows:

$$\Delta_x = D_l \left(\frac{KW\,r^3}{EI + 0.061\,E'\,r^3} \right)$$

Δ_x = horizontal deflection of pipe in inches
D_l = deflection lag factor = 1.0
K = bedding constant = 0.10
W = load per unit of pipe length (lb./lin. inch of pipe) = $W_e + W_l$
r = radius of pipe in inches

E_s = 1 modulus of elasticity of steel = 30,000,000 lbs. per square inch
E_c = modulus of elasticity for cement mortar = 4,000,000 lbs. per square inch
I = moment of inertia of cross section of pipe wall = inches^4 per lin. inch of pipe. (t^3/ 12)
E ' = modulus of soil, lb./in.2 (See Table 3.3.0.2)

NOTE: Under load, the individual elements – i.e., mortar lining, steel shell, and mortar coating — work together as laminated rings ($E_L I_L + E_s I_s + E_c I_c$ — shell lining and coating). Structurally, the combined action of these elements increases the moment of inertia of the pipe section, above that of the shell alone, thus increasing its ability to resist loads. The pipe wall stiffness (EI) of these individual elements are additive.

TABLE 3.3.0.2

Type of soil	Depth of cover, ft	Standard AASHTO relative compaction			
		85%	90%	95%	100%
Fine-rained soils with less than 25% sand content (CL, ML, CL-ML)	0-5	500	700	1,000	1,500
	5-10	600	1,000	1,400	2,000
	10-15	700	1,200	1,600	2,300
	15-20	800	1,300	1,800	2,600
Coarse-grained soils with fines (SM, SC)	0-5	600	1,000	1,200	1,900
	5-10	900	1,400	1,800	2,700
	10-15	1,000	1,500	2,100	3,200
	15-20	1,100	1,600	2,400	3,700
Coarse-grained soils with little or no Fines (SP, SW, GP, GW)	0-5	700	1,000	1,600	2,500
	5-10	1,000	1,500	2,200	3,300

VALUES OF MODULUS OF SOIL REACTION, E' (PSI) BASED ON DEPTH OF COVER, TYPE OF SOIL AND RELATIVE COMPACTION.
Soil type symbols are from the Unified Classification System
Source: Hartley, James D. and Duncan, James M., "E' and its Variation with Depth," Journal of Transportation, Div of ASCE, Sept.1987.

Unified Soil Classification (ASTM D-2487) - Group Symbols

GW Well-graded gravels, gravel-sand mixtures, little or no fines
GP Poorly graded gravels, gravel-sand mixtures, little or no fines.
GM Silty gravels, poorly graded gravel-send-silt mixtures
GC Clayey gravels, poorly graded gravel-sand-clay mixtures
SW Well-graded sands, gravelly sands, little or no fines
SP Poorly graded sands, gravelly sands, little or no fines
SM Silty sands, poorly graded sand-silt mixtures
SC Clayey sands, poorly graded sand- clay mixtures

CL Inorganic clays of low to medium plasticity
MH Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH Inorganic clays of high plasticity, fat clays
OL Organic silts and organic silt-clays of low plasticity
OH Organic clays of medium to high plasticity
Pt Peat and other highly organic soils
ML Inorganic silts and very fine sand, silty or clayey fine sands

Section 3.3.1 Extreme External Loading Conditions

An occasional need to calculate extreme external loading conditions arises—for example, to determine off-highway loading from heavy construction equipment. A convenient method of solution for such load determination using modified Boussinesq equations is presented:

Assume:

Live load from a Euclid loader
Total weight = 127,000 lbs.
Weight on one set of dual wheels, P = 42,300 lbs.
Tire pattern is 44 in. x 24 in.

Calculation:

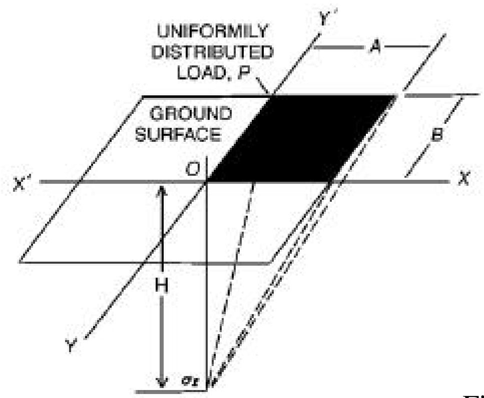


Figure 3.3.1

Using Figure 3.3.1 as reference

Tire pattern = $\frac{44}{12}$ x $\frac{24}{12}$ = 3.66 x 2.0 = 7.33 sq. ft.

Surface pressure is: $\frac{42{,}300}{7.33}$ = 5768 psf

If Height of cover H is 2.0 ft., then:

$A = \frac{3.66}{2} = 1.83$ $B = \frac{2.0}{2} = 1.0$

$m = \frac{A}{H} = 0.915$ $n = \frac{B}{H} = 0.5$

Coefficient from Table 3.3.1 = 0.117
P = 0.117(4)(5768) = 2700 psf
If height of cover is 3.0 ft., then:
m = 0.610 n = 0.333
Coefficient = 0.07
P = 1615 psf.

Using the Iowa formula to calculate deflection for 54-in. pipe and 60-in. pipe, (CML-tape coated) wall thickness 1/4 in. for each size, E' = 1200, D_l = 1.0, and soil weight of 120 pcf, the results are:

Total load (dead and live load):

2 ft. cover: $W_c = [(120(2) + 2700)] \frac{2r}{144} = 40.8r$

3 ft. cover: $W_c = [(120(3) + 1615)] \frac{2r}{144} = 27.4r$

Using Spangler's formula, deflection =

60 in., 2 ft. cover:	= 1.61 in. = 2.7%	**54 in.**, 2 ft. cover:	= 1.43 in. = 2.6%
3 ft. cover:	= 1.08 in. = 1.8%	3 ft. cover:	= 0.96 in. = 1.8%

TABLE 3.3.1- INFLUENCE COEFFICIENTS FOR RECTANGULAR AREAS*

m = A/H or *n = B/H*	*n = B/H* or *m = A/H*								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0.1	0.005	0.009	0.013	0.017	0.020	0.022	0.024	0.026	0.027
0.2	0.009	0.018	0.026	0.033	0.039	0.043	0.047	0.050	0.053
0.3	0.013	0.026	0.037	0.047	0.056	0.063	0.069	0.073	0.077
0.4	0.017	0.033	0.047	0.060	0.071	0.080	0.087	0.093	0.098
0.5	0.020	0.039	0.056	0.071	0.084	0.095	0.103	0.110	0.116
0.6	0.022	0.043	0.063	0.080	0.095	0.107	0.117	0.125	0.131
0.7	0.024	0.047	0.069	0.087	0.103	0.117	0.128	0.137	0.144
0.8	0.026	0.050	0.073	0.093	0.110	0.125	0.137	0.146	0.154
0.9	0.027	0.053	0.077	0.098	0.116	0.131	0.144	0.154	0.162
1.0	0.028	0.055	0.079	0.101	0.120	0.136	0.149	0.160	0.168
1.2	0.029	0.057	0.083	0.106	0.126	0.143	0.157	0.168	0.178
1.5	0.030	0.059	0.086	0.110	0.131	0.149	0.164	0.176	0.186
2.0	0.031	0.061	0.089	0.113	0.135	0.153	0.169	0.181	0.192
2.5	0.031	0.062	0.090	0.115	0.137	0.155	0.170	0.183	0.194
3.0	0.032	0.062	0.090	0.115	0.137	0.156	0.171	0.184	0.195
5.0	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196
10.0	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196
	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196
m= A/H or *n = B/H*	*n = B/H* or *m = A/H*								
	1.0	1.2	1.5	2.0	2.5	3.0	5.0	10.0	
0.1	0.028	0.029	0.030	0.031	0.031	0.032	0.032	0.032	0.032
0.2	0.055	0.057	0.059	0.061	0.062	0.062	0.062	0.062	0.062
0.3	0.079	0.083	0.086	0.089	0.090	0.090	0.090	0.090	0.090
0.4	0.101	0.106	0.110	0.113	0.115	0.115	0.115	0.115	0.115
0.5	0.120	0.126	0.131	0.135	0.137	0.137	0.137	0.137	0.137
0.6	0.136	0.143	0.149	0.153	0.155	0.156	0.156	0.156	0.156
0.7	0.149	0.157	0.164	0.169	0.170	0.171	0.172	0.172	0.172
0.8	0.160	0.168	0.176	0.181	0.183	0.184	0.185	0.185	0.185
0.9	0.168	0.178	0.186	0.192	0.194	0.195	0.196	0.196	0.196
1.0	0.175	0.185	0.193	0.200	0.202	0.203	0.204	0.205	0.205
1.2	0.185	0.196	0.205	0.212	0.215	0.216	0.217	0.218	0.218
1.5	0.193	0.205	0.215	0.223	0.226	0.228	0.229	0.230	0.230
2.0	0.200	0.212	0.223	0.232	0.236	0.238	0.239	0.240	0.240
2.5	0.202	0.215	0.226	0.236	0.240	0.242	0.244	0.244	0.244
3.0	0.203	0.216	0.228	0.238	0.242	0.244	0.246	0.247	0.247
5.0	0.204	0.217	0.229	0.239	0.244	0.246	0.249	0.249	0.249
10.0	0.205	0.218	0.230	0.240	0.244	0.247	0.249	0.250	0.250
	0.205	0.218	0.230	0.240	0.244	0.247	0.249	0.250	0.250

*Source: Newmark, N.M. Simplified Computuation of Vertical Pressures in Elastic Foundations. Circ. 24. Engrg. Exp. Stn., univ. of Illinois (1935)

Section 3.4 Minimum Thickness for Handling

Two well-known formulas, in existence some 40 years, have been adopted by many specifying agencies. They are:

U.S. Bureau of Reclamation $t = \frac{D + 20}{400}$ Pacific Gas & Electric $t = \frac{D}{288}$

The Pacific Gas and Electric formula is more liberal in diameters below 54” and the Bureau of Reclamation formula more liberal in diameters above 54”. Assuming internal pressure or external loads do not control wall thickness, the formulas result in these wall thickness designs:

18” .074” (14 gage)
24” .104” (12 gage)
30” .134” (10 gage)
48” .188” (3/16”)
72” .250” (1/4”)
96” .3125”(5/16”)

Numerous pipelines with these minimum wall thicknesses have provided satisfactory service for many years.

Section 3.5 Vacuum Design

Vacuum tests were conducted by United Concrete Pipe Corporation and by Utah State University to find the vacuum at collapse of very thin-wall steel pipe. United Concrete Pipe tests were on 91-inch diameter with 1/4 inch wall thickness, for which D/t is equal to 364. Collapse did not occur at low values of ring deflection and high values of soil compaction. Collapse was achieved only when ring deflection was increased to 7.5 % and the soil density (dumped) was 73%, AASHTO T-99.

Based on this experience, Utah State University tests were performed on 64 inch with 0.104-inch wall thickness, for which D/t is equal to 615.

The results are shown in Figure 3.5.1.

Figure 3.5.2 shows values to be used when using minimum thickness criteria as shown on Sec. 3.4.

Figure 3.5.3 shows typical values of sands of relative densities. Note increase in friction angle as densities increase.

Figure 3.5.4 shows usual values of the friction angle for soil backfill. Information regarding the Utah State University tests may be directed to the companies shown on the inside cover of this publication.

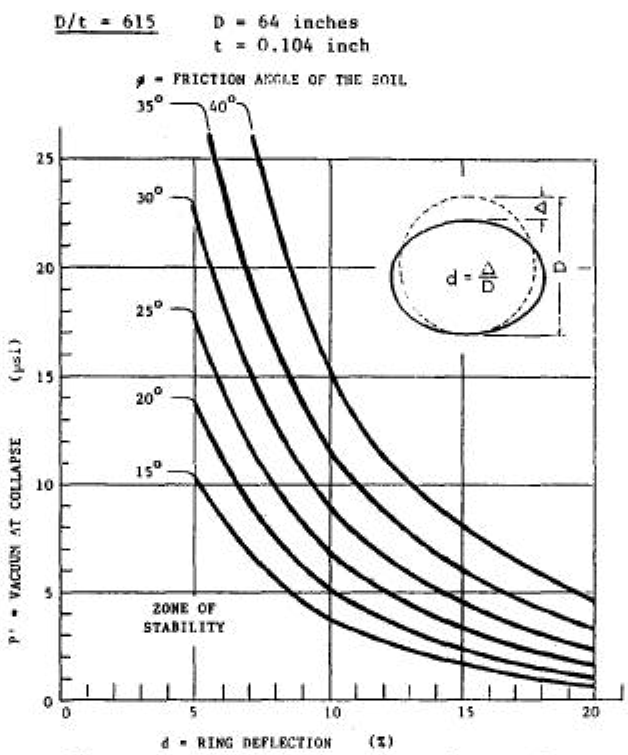


Figure 3.5.1 - Vacuum at collapse for 64 inch diameter very thin-wall steel pipes of 0.104 inch wall thickness and wall flexibility D/t = 615, buried in granule soil with two feet of soil cover.

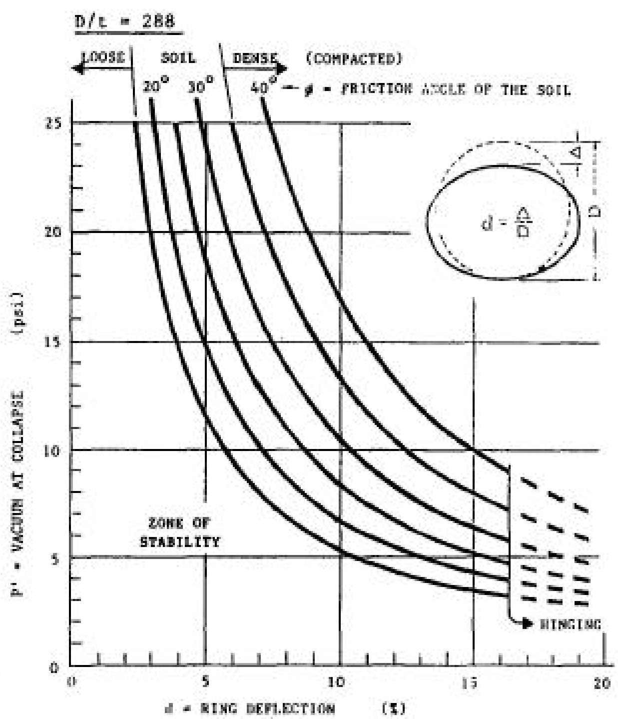


Figure 3.5.2 - Vacuum at collapse for 4 ft diameter thin-wall steel pipes of wall flexibility D/t = 288, buried in granular backfill with two feet of sod cover.

Backfill Material	Unit Weight ω Pounds per cu ft	Internal Friction Angle ϕ Degrees
Soft plastic clay	105-120	0-10
Wet, fine silty sand	110-120	15-30
Dry sand	90-110	25-40
Gravel	120-135	30-40
Loose loam	75- 90	30-45
Compact loam	90-100	30-45
Compact clay	90-110	25-45
Cinders	40	25-45
Compact sand-clay	115-125	40-50
Water	62.4	0

Source: Soil Engineering — Spangler — Second Edition.

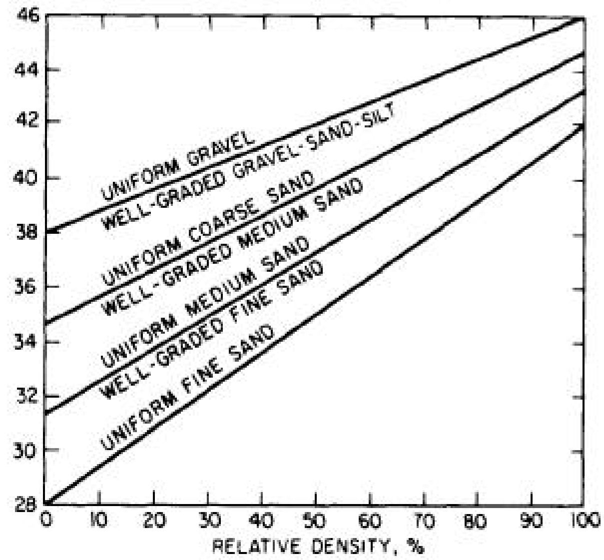


Figure 3.5.3 - Chart determines fiction for sands of various relative densities. (After J. H. Scbmertmann, 1978.)

Figure 3.5.4 – Usual values of unit weight and friction angle for sod backfill.

Section 3.6 Flotation Conditions

Buried penstock installations normally do not encompass the full spectrum of flotation (buoyancy) problems encountered by cross-country transmission pipelines carrying crude oil, natural gas, pulverized coal, etc. These pipelines are frequently laid across the bottoms of major rivers, naturally and artificially flooded areas, ocean bottoms, intermittently flooded areas, and areas permanently or periodically subjected to high groundwater tables.

The conditions involving potential flotation problems encountered by the penstock designer are as follows:

1. Crossing intermittent streams
2. Normal high groundwater areas
3. Areas subjected to intermittent high groundwater
4. Intermittently flooded areas

For the above conditions, buried penstocks normally are not additionally weighted to resist buoyant forces. Suitable backfill material alone results in an adequate anchoring force. It is important to note that unless the backfill material enters the liquid state and behaves as a viscous fluid, the penstock will not float. Most backfill materials do not act as fluids when wet. However, clay and silt are inherently unstable in the presence of water and have a tendency to become “quick” when saturated and to assume the character of a viscous fluid. Loose, poorly graded sand that is saturated and located in an earthquake region is highly susceptible to the phenomenon of liquefaction (quicksand) when disturbed by a shock loading. These materials in quantity should not be used under the conditions cited.

A common method of weighting is used when it is necessary to specifically design a penstock to resist flotation. This may be necessary when:

1. Unwatering the open trench during construction is not feasible or is uneconomical under existing conditions.
2. Excavated trench materials are unsuitable when they become saturated under design conditions and it is uneconomical to use suitable materials from borrow areas.

Additional weighting should be provided by a continuous, thick coating of reinforced concrete or by the addition of set on concrete river weights. The additional weight should be calculated to increase the bulk specific gravity of the penstock to 1.10 times the specific gravity of the anticipated viscous fluid soil-water backfill material.

Section 3.7 Fittings

Section 3.7.1 General

The wide range of design made possible by the welding and fabrication processes applicable to steel pipe provides the means of solving almost any problem involving fittings and specials.

AWWA C208 Standard for Dimensions for Fabricated Steel Water Pipe Fittings provides minimum dimensions for fabricated fittings with plain ends. In practice, however, one is not limited to these dimensions as fittings such as; miter cuts, elbows and outlets for access, pipe drains and air valves are normally fabricated into full pipe lengths by the pipe fabricator.

Normally the Engineer would provide a plan and profile of the penstock showing the station and elevation at critical points along the line. From this information, the pipe fabricator would provide a pipe laying schedule or assembly drawing showing the location of each standard pipe section and all fittings as required to fit the plan and profile. The pipe fabricator would also provide fabrication details of each fitting for the Engineer's approval.

Section 3.7.2 Elbows and Miter End Cuts

Small deflection angles can be taken at joints of O-rings, Mechanical Couplings and welded lap joints by pulling the joint asymmetrically. See the pipe or coupling manufacturer for the allowable deflections.

Deflection angles up to 5° can be taken in welded lap joints using miter cut bell ends. In this procedure, the pipe end is miter cut and then the bell is expanded square with the face of the miter cut. Deflection angles up to 5° can be taken in welded butt joints by miter end cuts of one or both pipe ends provided the difference in the circumference of the true circle and the ellipse formed by the miter end cut does not result in a joint fit up that would exceed the allowable plate edge offset.

Deflection angles greater than those allowed above shall be taken using fabricated elbows as shown in figures 2A - 2D except that the deflection per miter weld shall be limited to 22.5°. The radius of the elbow shall be at least 2.5 pipe diameters or the wall thickness of the elbow section shall be calculated using the following formula from AWWA C208, Sec. 2.

$$t = \frac{PD}{sf}\left[\frac{s}{2} + \frac{D}{3}\tan\frac{\theta}{2}\right]$$

Where: t = required elbow wall thickness (in.)
P = design pressure (psi)
f = allowable tensile stress at design pressure (psi)
D = outside diameter of pipe (in.)
s = segment length along inside of elbow as shown on Fig. 3.7.2.1
θ = segmented deflection angle as shown on Fig. 3.7.2.1

The above formula may be simplified as follows:

$$s = 2\left(R - \frac{D}{2}\right)\tan\frac{\theta}{2}$$

Substituting for s in the above formula

$$t = \frac{PD}{2f}\left[1 + \frac{D}{3R - 1.5D}\right]$$

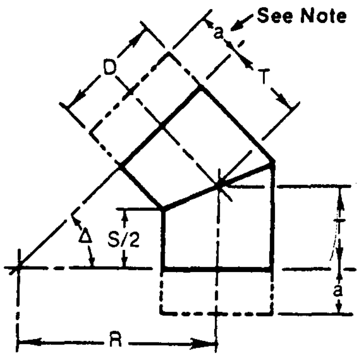


Fig. 2A: Two-Piece Elbow (0-22.5°)

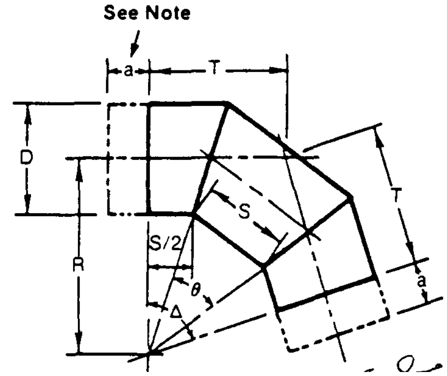


Fig. 2B: Three-Piece Elbow (22.5-45°)

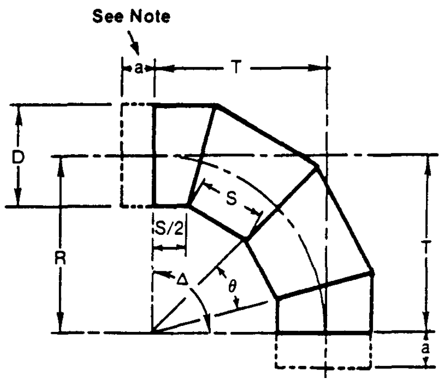


Fig. 2C: Four-Piece Elbow (45-67.5°)

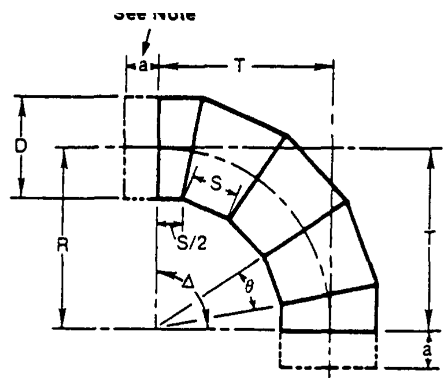


Fig. 2D: Five-Piece Elbow (67.5-90°)

NOTE: "a" may be extended beyond tangent line if necessary to suit joint requirements.

Figure 3.7.2.1 - Recommended dimensions for water pipe fittings (elbows)

An all welded line should be considered as fixed at each end for any segment along the line. Therefore, Poisson's effect, for longitudinal tensile stress equal to .3 of hoop stress must be considered. This longitudinal stress will be effective through the elbow, and, therefore, must be considered in calculating the equivalent combined stress of tension plus tension at right angles.

Using the Hencky-Mises Theory for this calculation, one gets the following:

$$f_e = (f_x^2 - f_x f_y + f_y^2)^{.5}$$

Where $f_y = .3 f_x$

$$f_e = [f_x^2 - f_x (.3f_x) + (.3f_x)^2]^{.5}$$

$$f_e = .889 f_x$$

Therefore, the factors for t, calculated above, may be reduced as follows for this application:

(1.167) x .889 = 1.037
(1.074) x .889 = .955
(1.035) x .889 = .920

Bends may be designed with a constant diameter or with a different diameter on each end (Figure 3.7.2.3) Compound or combined bends, in which the plain of the bend is neither horizontal nor vertical, requires certain trigonometric computations. Usually, the plan angle and profile angles are known and it is required to determine the true angle in the plane of the bend and the bend rotations. Computation method and applicable formulas for these bend properties are shown in Figure 3.7.2.2

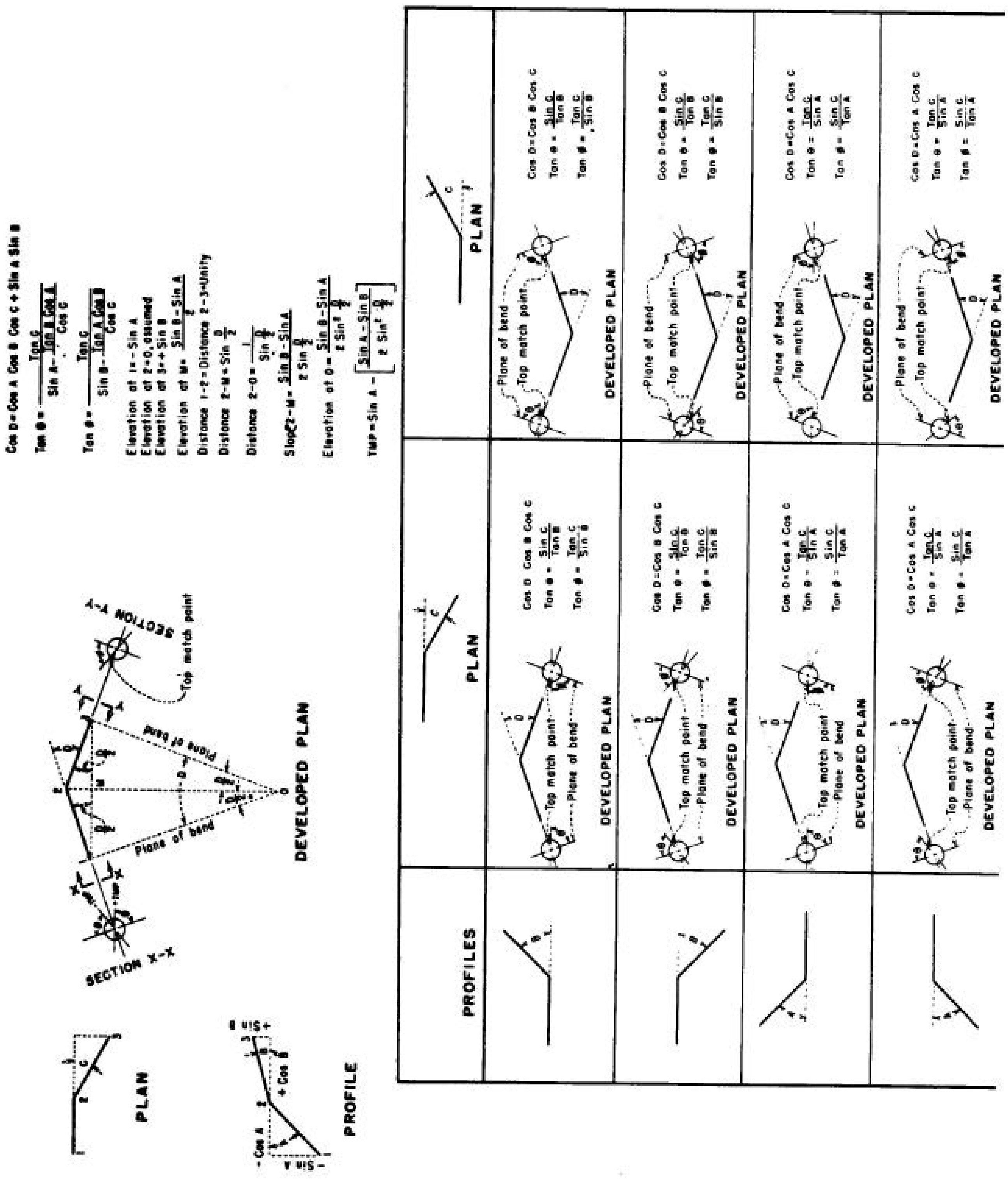
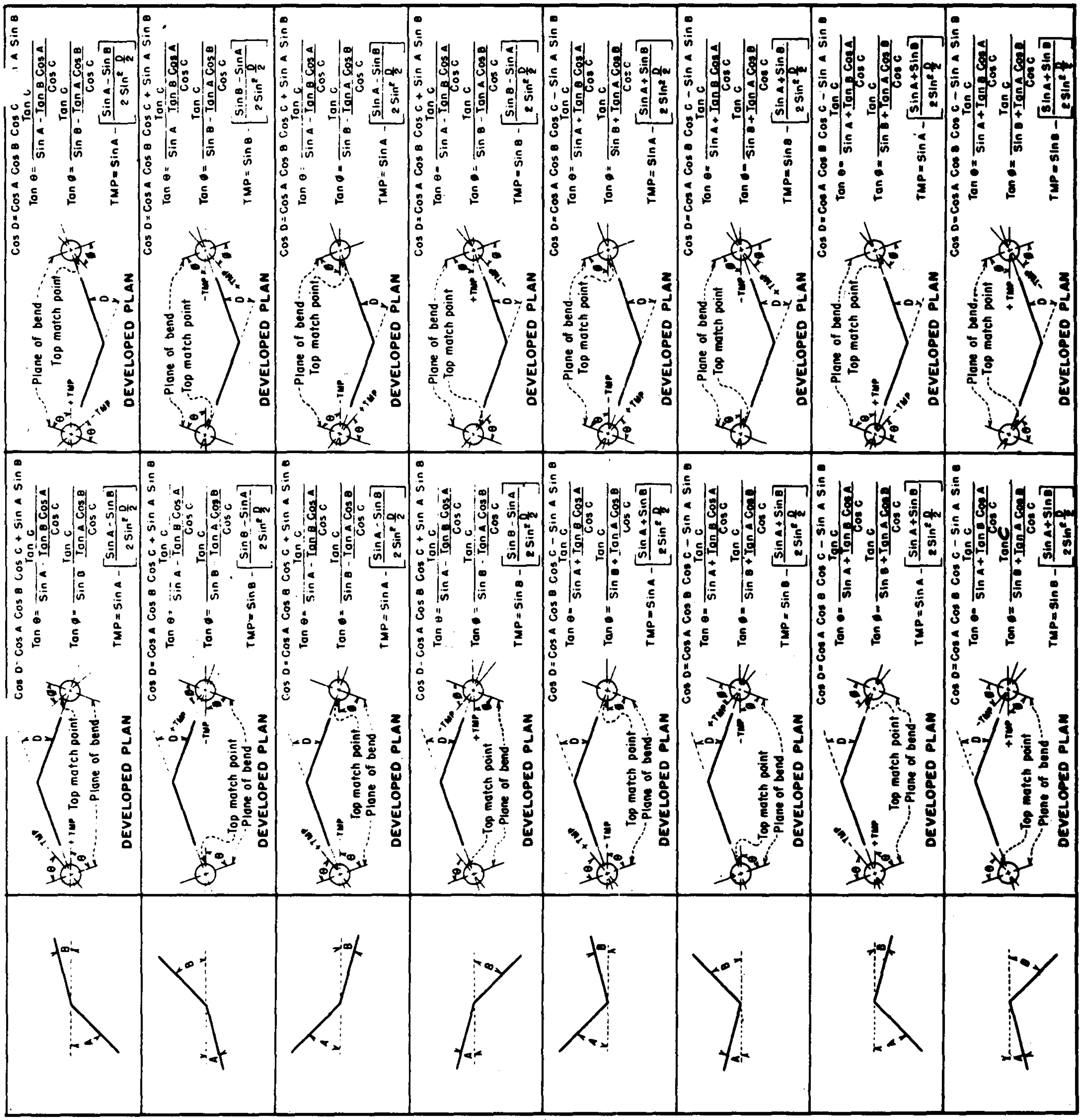
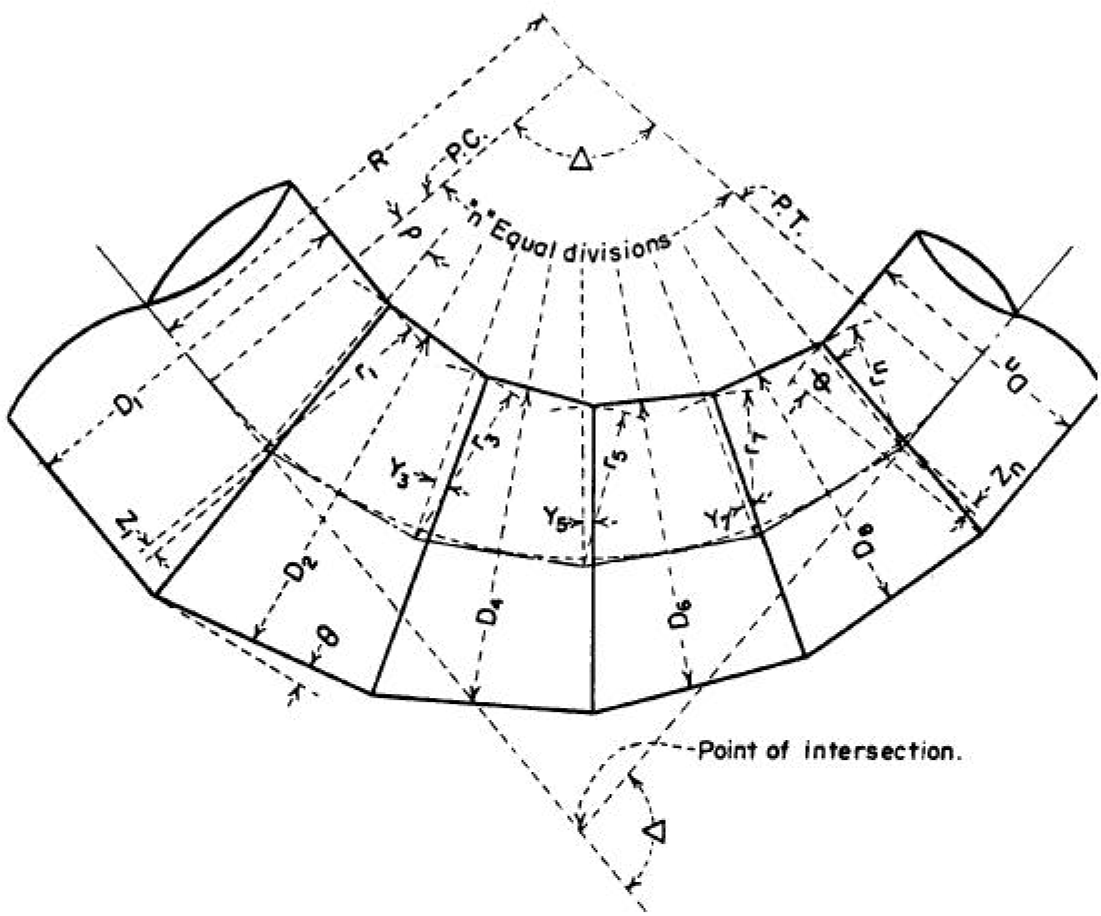


Figure 3.7.2.2 – Computative method and formulas for compound pipe bends





Δ = Angle of intersection.
R = Radius of bend.
n = 2(number of deflections).
D_1 = Inside diameter of large pipe.
D_n = Inside diameter of small pipe.

$$P = \frac{\Delta}{n}$$

$$\text{Sin}\ \theta = \frac{D_1 - D_2}{2(n-2)R \tan p}$$

$$r_1 = \frac{D_1}{2}$$

$$r_n = \frac{D_n}{2}$$

$$r_x = r_1 - (x-1)R \tan p \sin \theta$$

$$D_x = \frac{D_1 - 2(x-1)R \tan p \sin \theta}{\cos \theta}$$

Where X = number of divisions from P.C. to point under consideration.

$$2p = k + \Phi$$

$$\text{Tan}\Phi = \frac{\sin 2p}{\cos 2p + \cos \theta}$$

$$Z_1 = \frac{r_1 \sin \theta}{\cos 2p + \cos \theta}$$

$$Z_n = \frac{r_n \sin \theta}{\cos 2p + \cos \theta}$$

$$Y_x = \frac{r_x\ \text{sine}\ \theta}{\cos p}$$

Figure 3.7.2.3 - Reducing bend formulas

Section 3.7.3 Reducers

Changes in diameter are accomplished by use of concentric or eccentric cones or reducers placed in the straight section of a line or combined with a mitered elbow.

AWWA C-208 shows the common length of a reducer to be $4(D_2 – D_1)$ which gives a half-apex angle of 7° - 7.5°. For reducers of this half-apex angle or less, the wall thickness of the larger diameter pipe is adequate.

Reducers with half-apex angles over 8° should be designed in accordance with ASME B&PV Code Section VIII, Division I.

Section 3.7.4 Flanged Outlets

Flanged outlets can be assembled from a short piece of pipe using a steel ring flange, or a hub flange of the slip-on type can be used. Attachment of flanges should be in accordance with AWWA C207. The boltholes in flanges straddle the vertical and horizontal centerlines. If the main line slopes, the flange should be rotated with reference to this slope to bring the attachments vertical.

Outlet nozzles should be as short as possible to reduce the leverage of any bending force applied to the outlet. In general, every outlet should have a valve firmly attached to the mainline and a flexible connection to the pipe downstream from this valve.

Section 3.8 Reinforcement of Fittings

Tees, crosses, laterals, wyes, headers, or other fittings that provide means of dividing or united flow in pipelines do not have as high a resistance to internal pressure as do similar sizes of straight pipe of the same wall thickness. This is because a portion of the side wall of the pipe in these fittings is removed to allow for the branching pipe. Also, there are longitudinal stresses in the throat of unrestrained elbows, owing to distortion or unbalanced hydrostatic pressure.

For ordinary waterworks installations, the wall thickness of the pipe commonly used is much greater than pressure conditions require. Consequently, the lowered safety factor of fittings having the same wall thickness as the straight pipe still leaves adequate strength in most cases, and reinforcing may be unnecessary. If the pipe is operating at or near maximum design pressure, however, the strength of the fittings should be investigated and the proper reinforcement or extra wall thickness provided.

Fittings may be reinforced in various ways for resistance to internal pressure. Typical fitting reinforcements are collars, wrappers, and crotch plates. The design stress in the reinforcement should not be greater than the hoop stress used in the design of the pipe.

The type of reinforcement* can be determined by the magniture of the pressure-diameter value PDV and the ratio of the branch diameter to the main pipe diameter *d/D.* The pressure-diameter value is calculated as:

$$\mathrm{PDV} = Pd^2 / D \sin^2 \Delta$$

where: P = design pressure (psi)
d = branch outside diameter (in.)
D = main pipe outside diameter (in.)
Δ = branch diameter angle of deflection.

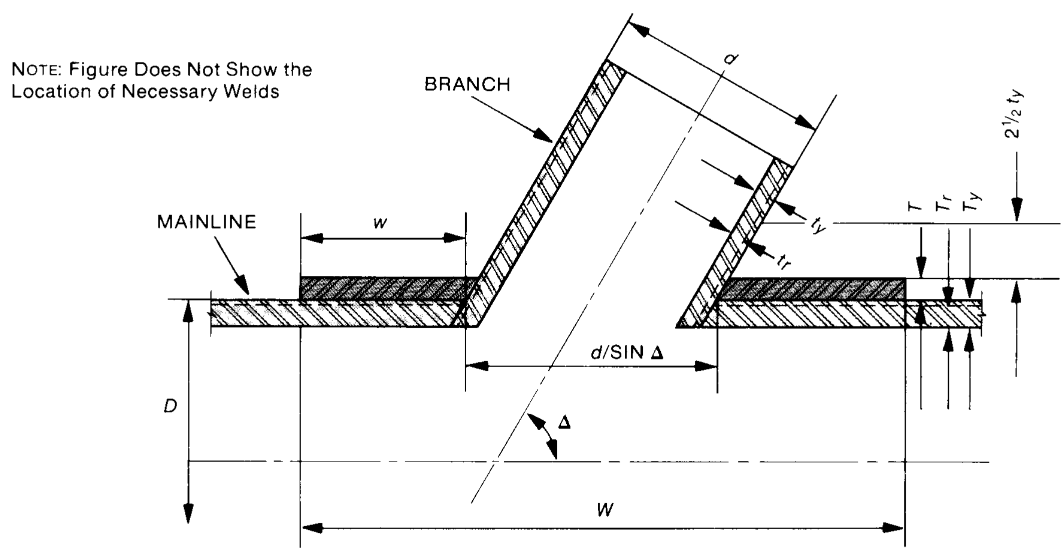
For PDV values greater than 9,000, the outlet reinforcement should consist of a crotch plate designed in accordance with the method described in Section 3.9. For PDV values less than 9,000, the outlet reinforcement may be either a wrapper or collar, depending on the ratio of the outlet diameter to the main pipe diameter *d/D*. For a *d/D* ratio greater than 0.7, a wrapper plate should be used; for a *d/D* ratio less than 0.7, either a collar or a wrapper plate may be used. The ratio *d/D* does not include the sin A as in the PDV determination because the controlling factor is the circumferential dimensions. Wrappers may be substituted for collars, and crotch plates may be substituted for wrappers or collars.

Wrappers and collars should be designed by the method described in Sec. VIII of the ASME Unfired Pressure Vessel Code. This code provides that the cross-sectional area of the removed steel at the branch is replaced in the form of a wrapper or collar. In addition to the ASME requirements when the PDV ranges between 6,000 and 9,000, the cross-sectional area of the replaced steel should be multiplied by an *M* factor of 0.000167 times the PDV. Figure 3.8.1 shows the reinforcement of wrapper and collar openings for welded steel pipe, and Table 3.8.1 lists a summary of recommended reinforcement types.

In determining the required steel replacement, credit should be given to any thickness of material in the main-line pipe in excess of that required for internal pressure, and to the area of the material in the wall of the branch outlet to the allowable distance from the collar or wrapper ($2.5t_y$). Weld areas should not be considered in the design. Overall width of the collar or wrapper should not be less than $1.67d/\sin\Delta$ and should not exceed $2.0d/\sin\Delta$. This width range produces a minimum edge width of $0.33d/\sin\Delta$. Collar edge widths in the circumferential direction should not be less than the longitudinal edge width.

Collars may be oval in shape, or they may be rectangular with rounded corners. The radii at corners should not be less than 4 in. or 20 times the collar thickness, whichever is greater (except for collars with a length or width less than 8 in.). Longitudinal seams should be placed at 90° or more from the center of the removed section.

On the branch outlet centerline, the limit line of the branch reinforcement occurs at a distance 2.5 times the thickness of the branch from the surface of the main pipe run or from the top of the collar or wrapper reinforcement. In Figure 3.8.1, the area $T_y(d\text{-}2t_y)/\sin\Delta$ represents the section of the mainline pipe cylinder removed by the opening for the branch. The hoop tension due to pressure within the pipe that would be taken by the removed section were it present must be carried by the total areas represented by $2wT$ and $5t_y\ (t_y - t_r)$, or $2.5t_y\ (t_y - t_r)$ on each side of outlet.



D = mainline pipe outside diameter (in.)
T_y = mainline cylinder thickness (in.)
T_r = required mainline cylinder thickness (in.)
D = branch pipe outside diameter (in.)
t_v = branch cylinder thickness (in.)

t_r = required branch cylinder thickness (in.)
Δ = branch deflection angle (degrees)
T = wrapper thickness (in.)
W = overall wrapper width (in.)
w = wrapper edge width (in.)

Figure 3.8.1 - Reinforcement of openings in welded steel pipe

*Reinforcement for large diameter, high-pressure wyes, or double laterals may require additional analyses beyond the criteria discussed herein.

TABLE 3.8.1 - RECOMMENDED REINFORCEMENT TYPE*

PDV	d/D	M Factor	Reinforcement Type
>9000	all	-	Crotch Plate
6000-9000	>0.7	0.000167 PDV	Wrapper
<6000	>0.7	1.0	Wrapper
6000-9000	$\leq$0.7	0.000167 PDV	Collar
<6000	$\leq$0.7	1.0	Collar

*These reinforcements are for resistance to internal pressure. They should be checked for ability to resist external loads.

COLLAR PLATE DESIGN

Criteria-data example - 48" x 20" manhole

Main-pipe size (nominal diameter)		48 in.
Main-pipe cylinder OD	D	49 5/8 in.
Main-pipe cylinder thickness	T_y	1/4 in.
Branch-outlet size (nominal diameter)		20 in.
Branch-outlet cylinder OD	d	21 1/2 in.
Branch-outlet thickness	t_y	1/4 in.
Deflection angle	Δ	90°
Design pressure	P	150 psi
Reinforcement steel allowable stress (The allowable stress is based on the design condition.)	f_s	18,000 psi

REINFORCEMENT TYPE

$$\text{PDV} = \frac{Pd^2}{D\sin^2\Delta} = \frac{(150)\,(21.5)^2}{49.625\ \sin^2 90°} = 1397$$

$$\frac{d}{D} = \frac{21.5}{49.625} = 0.433$$

Therefore, for PDV $\leq$ 9000 and $d/D \leq 0.7$, *use collar* unless wrapper is provided.

MULTIPLIER (M-factor)

For PDV <6000, $M = 1.0$

COLLAR DESIGN

Theoretical cylinder thicknesses.

Main pipe (T_r)

$$T_r = \frac{PD}{2f_s} = \frac{(150)\,(49.625)}{2\ (18{,}000)} = 0.207 \text{ in}$$

Branch outlet (t_r)

$$t_r = \frac{Pd}{2f_s} = \frac{(150)\,(21.5)}{2\ (18{,}000)} = 0.090 \text{ in}$$

Theoretical reinforcement area.

$$A_r = M\left[\,T_r\left(\frac{d - 2t_y}{\sin\Delta}\right)\right]$$

$$= 1.0\left[\,0.207\left(\frac{21.5 - 2(0.25)}{\sin 90°}\right)\right]$$

$$= 4.347 \text{ in.}^2$$

Area available as excess T_y and allowable outlet area.

Area available = A_a

$$A_a = \frac{(d - 2\,t_y)}{\sin\Delta}\;(T_y - T_r) + 5t_y\,(t_y - t_r)$$

$$= \frac{21.5 - 2(0.25)}{\sin 90^\circ}\;(0.25 - 0.207) + (5 \times 0.25)\,(0.25 - 0.090)$$

$$= 1.103 \text{ in.}^2$$

Reinforcement area.

Reinforcement area = A_w

$A_w = A_r - A_a$

$A_w = 4.347 - 1.103 = 3.244 \text{ in.}^2$

Minimum reinforcement thickness.

Minimum reinforcement thickness = T

$$w = \frac{d}{2\sin\Delta} = \frac{21.5}{2\sin 90^\circ} = 10.75 \text{ in.}$$

$$T = \frac{A_w}{2w} = \frac{3.244}{2(10.75)} = 0.151 \text{ in.}$$

Round up to the nearest standard thickness, but not less than 12-gauge.

$T = 0.164$ in. (8 ga.)

Reinforcement width.

$$w = \frac{A_w}{2T} = \frac{3.244}{2(0.164)} = 9.89 \text{ in.}$$

Minimum allowable width.

$$w(\text{min.}) = \frac{d}{3\sin\Delta} = \frac{21.25}{3\sin 90^\circ} = 7.17 \text{ in.}$$

7.17 in. < 9.89 in.

$\therefore w = 9.89$ in.

Overall reinforcement width.

$$W = 2w + \frac{d}{\sin\,\Delta} = 2(9.89) + \frac{21.5}{\sin\,90^\circ} = 41.28 \text{ in.}$$

Use: $T = 0.164$ in.
W = 41 $^5/_{16}$ in.

WRAPPER-PLATE DESIGN

Criteria-data example – 54 in. x 48 in. lateral

Main-pipe size (nominal diameter)		54 in.
Main-pipe cylinder OD	D	55 $^5/_8$ in.
Main-pipe cylinder thickness	T_y	¼ in.
Branch-outlet size		48 in.
Branch-outlet cylinder OD	d	49 $^5/_8$ in.
Branch-outlet thickness	t_y	¼ in.
Deflection angle	Δ	75º
Design pressure	P	150 psi
Reinforcement steel allowable stress (The allowable stress is based on the design conditions)	f_s	18,000 psi

REINFORCEMENT TYPE

$$\text{PDV} = \frac{Pd^2}{D \sin^2 \Delta} = \frac{(150)\,(49.625)^2}{(55.625)\,\sin^2 75°} = 7118$$

Therefore, for PDV $\leq$ 9000 and *d/D > 0.7, use wrapper.*

MULTIPLIER (*M*-factor)

For 6000 < PDV < 9000

$M = 0.000167$ PDV $= 0.000167(7118) = 1.19$

Therefore, use $M = 1.19$.

WRAPPER DESIGN

Theoretical cylinder thicknesses.

Main pipe (T_r)

$$T_r = \frac{PD}{2f_s} = \frac{(150)\,(55.625)}{2\,(18{,}000)} = 0.232 \text{ in.}$$

Branch outlet (t_r)

$$T_r = \frac{Pd}{2f_s} = \frac{(150)\,(49.625)}{2\,(18{,}000)} = 0.207 \text{ in}$$

Theoretical reinforcement area.

Theoretical reinforcement area = A_r

$$A_r = M\left[T_r\left(\frac{d - 2t_y}{\sin \Delta}\right)\right]$$

$$A_r = (1.19)\left[0.232\left(\frac{49.625 - 2(0.25)}{\sin 75°}\right)\right]$$

$$A_r = 14.041 \text{ in.}^2$$

Area available as excess T_y and allowable outlet area

Area available = A_a

$$A_a = \frac{(d - 2t_y)}{\sin \Delta} (T_y - T_r) + 5t_y (t_y - t_r)$$

$$A_a = \frac{49.625 - 2(0.25)}{\sin 75°} (0.25 - 0.232) + (5 \times 0.25)(0.25 - 0.207)$$

$A_a = 0.969$ in.^{2}

Reinforcement area.

Reinforcement area = A_w

$A_w = A_r - A_a$

$A_w = 14.041 - 0.969 = 13.072$ in.^{2}

Minimum reinforcement thickness.

Minimum reinforcement thickness = T

$$w = \frac{d}{2 \sin \Delta} = \frac{49.625}{2 \sin 75°} = 25.688 \text{ in.}$$

$$T = \frac{A_w}{2w} = \frac{13.072}{2(25.688)} = 0.254 \text{ in.}$$

Round up to the nearest standard thickness but not less than 12 gauge (0.105 in.).

$T = {}^5/_{16}$ in. (0.3125 in.)

Minimum reinforcement width.

$$w = \frac{A_w}{2T} = \frac{13.072}{2(0.3125)} = 20.915 \text{ in.}$$

Minimum allowable width.

$$w(\text{min.}) = \frac{D}{3 \sin \Delta} = \frac{49.625}{3 \sin 75°} = 17.125 \text{ in.}$$

17.125 in. < 20.915 in.

$\therefore w = 20.915$ in.

Overall reinforcement width.

$$W = 2w + \frac{d}{\sin \Delta} = 2(20.915) + \frac{49.625}{\sin 75°} = 93.206 \text{ in.}$$

Use: $T = {}^5/_{16}$ in.
W = 93 ¼ in.

Section 3.9 Nomograph Use in Wye-Branch Design

The nomograph design, based on design working pressure plus surge allowance, includes a safety factor that will keep stresses well below the yield point of steel. The minimum yield strength of the steel used in this report is 30,000 psi. The design pressure used in the nomograph was kept to 1.5 times the working pressure in order to approximate an allowable stress of 20,000 psi. For other allowable design stresses, use a modified factor determined by dividing 30,000 psi by the allowable design stress based on the design condition. Example: Traditional Design, Normal Operating Condition, A516-70 Steel, Factor is 30,000/23,300 = 1.29.

Section 3.9.1 Crotch-Plate (Wye-Branch) Design

When the PDV exceeds 9000, crotch-plate reinforcement should be used. Several types of plate reinforcement are illustrated in Figures 3.9.1 through 3.9.1.5. The following section on nomograph use was taken from a published study on crotch-plate (wye-branch) design at Los Angeles.

Step 1. Lay a straightedge across the nomograph (Figure 3.9.1.1) through the appropriate points on the pipe diameter (see step 2b) and internal-pressure scales; read off the depth of plate from its scale. This reading is the crotch depth for l-in. thick plate for a two-plate, 90º, wye-branch pipe.

Step 2a. If the wye branch deflection angle is other than 90º, use the N-factor curve (Figure 3.9.1.2) 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 diameter pipe will have been used in steps 1 and 2a, and these results should be multiplied by the Q factors found on the single-plate stiffener curves (Figure 3.9.1.3) to give d_w' and d_b'. These factors vary with the ratio of the radius of the small pipe to the radius of the large pipe.

Step 3. The depth of d_w should be limited to 30 times the thickness of the plate. The formula is based on 1" plate and can be converted to a lesser of greater thickness by use of the general equation. The minimum thickness should be at least 3/16".

$$d = d_1 \left(\frac{t_1}{t} \right)^{\left(0.917 - \frac{\Delta}{360}\right)}$$

where: d_1 = existing depth of plate
t_1 = existing thickness of plate
d = new depth of plate
t = new thickness of plate selected
Δ = deflection angle of the wye branch.

Step 4. To find the top depth d_t or d_t', use Figure 3.9.1.4, 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 90º from the crotch depths.

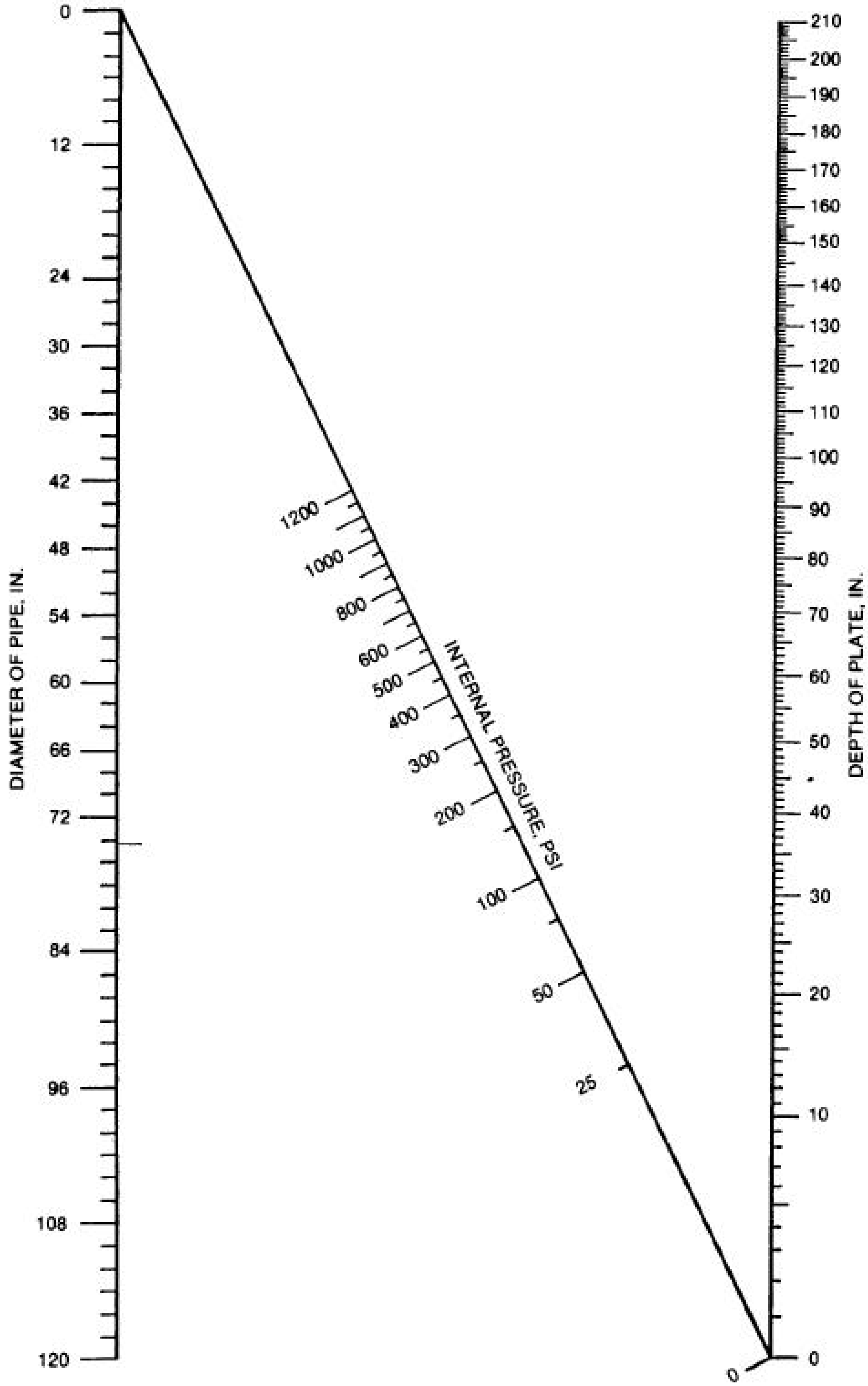
Step 5. The interior curves follow the cut of the pipe, but the outside crotch radius in both crotches should equal d_t plus the radius of the pipe, or in the single-plate design, d_t' plus the radius of the smaller pipe. Tangents connected between these curves complete the outer shape.

The important depths of the reinforcement plates, d_w, d_b, and d_t, (Figure 3.9.1.5), can be found from the nomograph. If a curved exterior is desired, a radius equal to the inside pipe radius plus d, can be used, both for the outside curve of the wye section and for the outside curve of the base section.

EXAMPLE 1 - ONE-PLATE DESIGN

R_B = 30 in.
R_s = 21 in.
Δ = 45º
Working pressure, 230 psi
Design pressure, 230 (1.5) = 350 psi

Step 1. With the larger pipe diameter 60 in. and the design pressure 350 psi, read the critical plate depth d from the nomograph (t = 1 in., Δ = 90º):
d = 50 in.

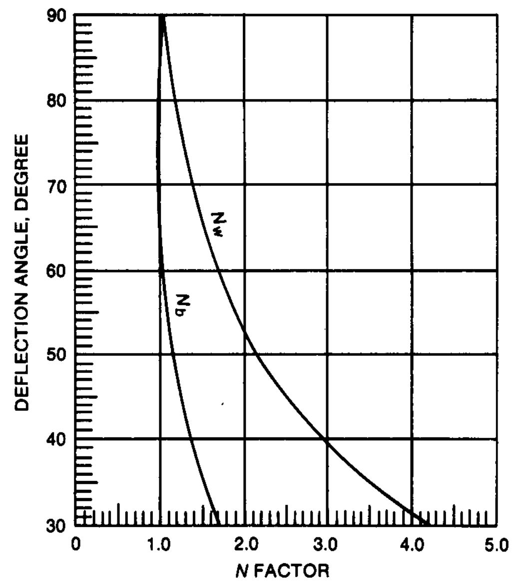


Source: Swanson, H.S. ET AL. Design of Wye Branches for Steel Pipe. Jour. AWWA. 47:6:581 (June 1955).

Plate thickness, 1 in.; deflection angle, 90°.

Figure 3.9.1 – Nomograph for selecting reinforcement plate depths of equal-diameter pipes

Figure 3.9.1.2 - N Factor curves

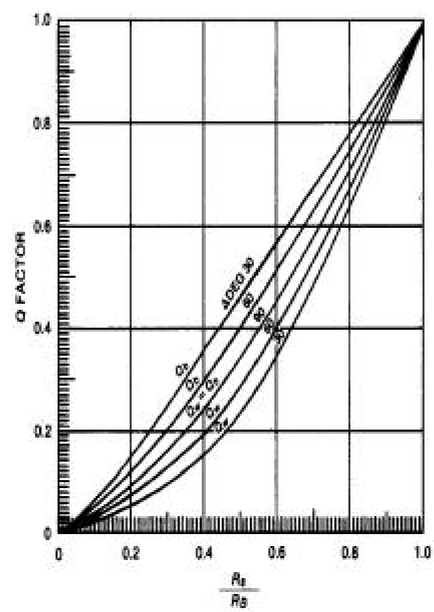


For wyes with deflection angles from 30º to 90º, the *N* factors obtained from the above curves are applied to the plate depth *d*, found from the nomograph (Figure 3.9.1), in accordance with the equations

$$d_w = N_w d;\ d_b = N_b d.$$

Source: Swanson, H.S. ET AL. Design of Wye Branches for Steel Pipe. Jour. AWWA, 47:6:581 (June 1955).

Figure 3.9.1.3 - Q Factor curves



For pipes of unequal diameter, find d_w and d_b for the larger-diameter pipe (from Figures 3.9.1 and 3.9.1.2); then: $Q_w d_w = d'_w$ crotch depth of single-plate stiffener; and $Q_b\ d_b = d'_b$, base depth of single-plate stiffener.

Source: Swanson, H.S. ET AL. Design of Wye Branches for Steel Pipe. Jour. AWWA. 47:6:561 (June 1955).

Step 2. 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.):

$d_w = N_w d = 2.45(50) = 122$ *in.*
$d_b = B_b d = 1.23(50) = 61.5$ in.

Step 3. With the ratio of the smaller pipe radius divided by the larger pipe radius (R_s/R_B) = (21/30) = 0.70 and the deflection angle (Δ = 45°), use Figure 3.9.1.3 to find the *Q* factors that give the crotch depths for a single-plate pipe wye stiffener (t = 1 in.):

Q_w = 0.52
Q_b = 0.66
d'_w = 0.52(122) = 63.4 in.
d'_b = 0.66(61.5) = 40.5 in.

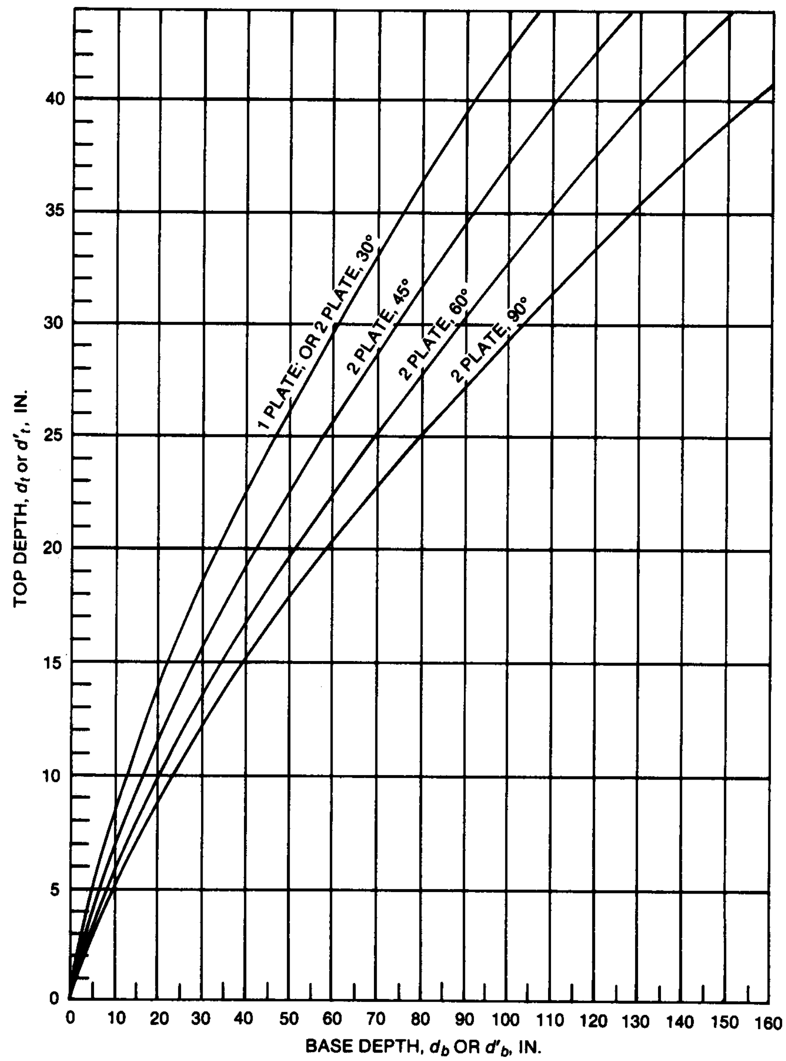
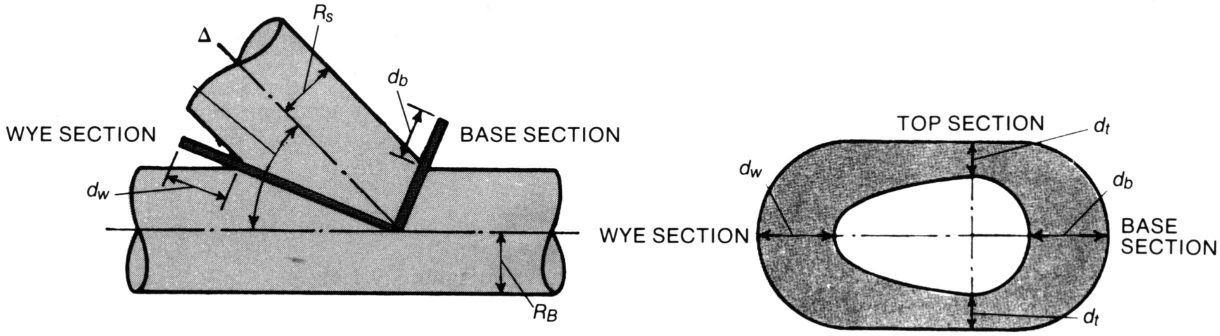


Figure 3.9.1.4 – Selection of top depth

Source: Swanson, H.S. ET AL. Design of Wye Branches for Steel Pipe. Jour. AWWA, 47:6:581 (June 1955).
d'_t and d'_b, are one-plate design dimensions; d_t and d_b are two-plate design dimensions.

Figure 3.9.1.5 - Wye branch plan and layout



Source: Swanson, H.S. ET AL. Design of Wye Branches for Steel Pipe. Jour. AWWA, 47:6:581 (June 1955).

Step 4. Because the depth d'_w is greater than 30 times the thickness *t,* the conversion equation should be used:

$$d = d_1 \left(\frac{t_1}{t} \right)^{(0.917 - \Delta/360)}$$

Try a thickness of 1½in.:

$$d = d_1 \left(\frac{1}{1.5} \right)^{(0.917 - 45/360)} \quad = d_1 \left(\frac{2}{3} \right)^{0.792}$$

$d = d_1\ (0.725)$
$d'_w = 63.4(0.725) = 46$ in.
$d'_b = 40.5(0.725) = 29$in.
Step 5. Find the top depth d'_t from the curve for one-plate design in Figure 3.9.1.4:
For $d'_b = 29$ in., $d'_t = 18$ in.

Final results:

Thickness of reinforcing plate, t $= 1^1/_2$ in.
Depth of plate at acute crotch, d'_w = 46 in.
Depth of plate at obtuse crotch, d'_b = 29 in.
Depth of plate at top and bottom, d'_t = 18 in.
Outside radius of plate at both crotches equals the top depth plus the inside radius of the small pipe $d'_t + R_s = 18 + 21 = 39$ in.

EXAMPLE 2 - TWO-PLATE DESIGN

$R_B = R_s = 36$ in.
$\Delta = 53°$
Working pressure, 150 psi
Design pressure, 150 (1.5) = 225 psi

Step 1. With a pipe diameter of 72 in. and a pressure of 225 psi, read the critical depth of plate from the nomograph (t = 1 in., A = 90º):
$d = 49$ in.

Step 2. From the N-factor curve, find the two factors at A = 53º; then, at $t = 1$ in.:
$d_w = 1.97(49) = 96.5$ in.
$d_b = 1.09(49) =$ *53.4 in.*

Step 3. Because d_w is greater than 30 times the thickness of the plate, try t = 2 in. in the conversion equation:

$$d = d_1 \left(\frac{t_1}{t} \right)^{(0.917 - \Delta/360)} = d_1 \left(\frac{1}{2} \right)^{0.770}$$

$d = d_1$ (0.586)

d_w = 96.5(0.586) = 57 in.
d_b = 53.4(0.586) = 31 in

Step 4. Read the top depth d_t from the two-plate design curve in Figure 3.9.1.4:
d_t = 15

Final results: Thickness of reinforcing plate, t = 2 in.
Depth of plate at acute crotch, d_w = 57 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, 51 in.

THREE-PLATE DESIGN

The preceding nomograph section has covered the design of one- and two-plate wye branches without touching on a three-plate design because of its similarity to the two-plate design. The function of the third plate is to act like 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 simply to 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 pressure is greater than 300 psi, a ring plate can be advantageous. If either of these factors is below the limit, the designer should be allowed 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 pipe, however, the shell plate thickness plus clearance should be subtracted from the top depth. This dimension should be constant through out, 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 the smaller 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 shell so that none of the shell stresses will be transferred to the ring plate.

CHAPTER 4

Thrust Restraint

Thrust forces are unbalanced forces which occur in pressure pipelines 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). These forces, if not adequately restrained, tend to disengage joints, as illustrated in Figure 4.0.1. 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. Since most water lines operate at relatively low velocities, the dynamic force is insignificant and is usually ignored when computing thrust. For example, the dynamic force created by water flowing at 8 fps is less than the static force created by 1 psi.

Section 4.1 Hydrostatic Thrust

Typical examples of hydrostatic thrust are shown in Figure 4.1.1. The thrust in dead ends, outlets, laterals, and reducers is a function of internal pressure, *P*, and cross-sectional area, *A,* at the pipe joint. The resultant thrust at a bend is also a function of the deflection angle, Δ, and is given by:

$$T = 2PA\sin(\Delta/2) \quad \text{(Eq. 4.1)}$$

where: T = hydrostatic thrust, lbs.
P = internal pressure, psi
Δ = $(\pi/4)D^2$ = cross-sectional area of pipe O.D., sq. in.
A = deflection angle of bend, deg.

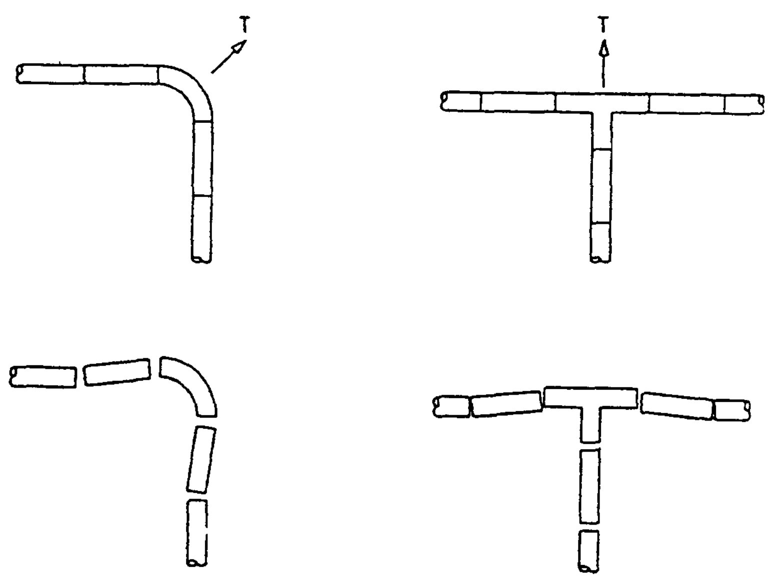


Figure 4.0.1 - Unbalanced thrusts and movements in pipeline

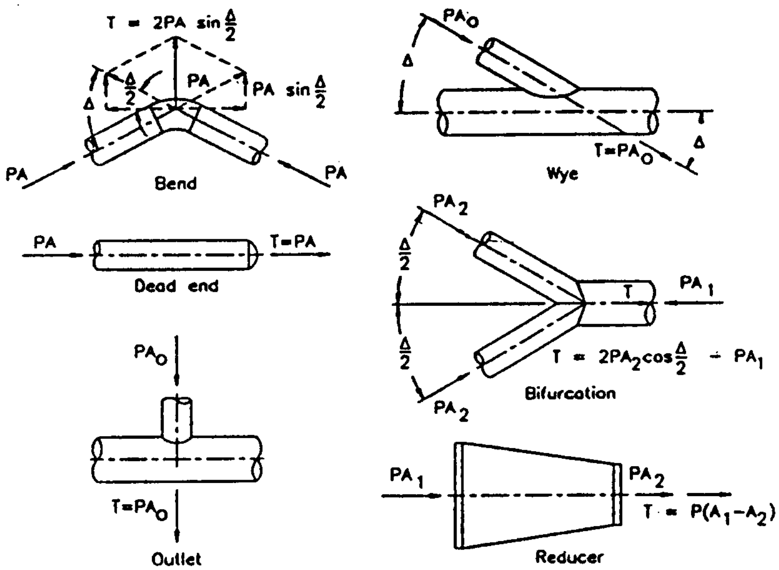


Figure 4.1.1 - Hydrostatic thrust, T, for typical fittings

Section 4.2 Thrust Resistance

For buried pipelines, thrust resulting from angular deflections at standard and beveled pipe with rubber gasket joints is resisted by dead weight or frictional drag of the pipe, and additional restraint is usually not needed. Other fittings subjected to unbalanced horizontal thrust have two inherent sources of resistance: (1) frictional drag from dead weight of the fitting, earth cover, and contained water, and (2) passive resistance of soil against the back of the fitting. If this type of resistance is not adequate to resist the thrust involved, then it must be supplemented either by increasing frictional drag of the line by "tying" adjacent pipe to the fitting or by increasing the supporting area on the bearing side of the fitting with a thrust block. Unbalanced uplift thrust at a vertical deflection is resisted by the dead weight of the fitting, earth, cover, and contained water. If this type of resistance is not adequate to resist the thrust involved, then it must be supplemented either by increasing the dead weight of the line by "tying" adjacent pipe to the fitting or by increasing the dead weight with a gravity type thrust block.

Section 4.3 Joints with Small Deflections

The thrust at beveled pipe or standard pipe installed with angular deflection is usually so small that supplemental restraint is not required.

Section 4.3.1 Small Horizontal Deflections

Thrust, *T,* at deflected joints on long-radius horizontal curves is resisted by friction on the top and bottom of the pipe as shown in Figure 4.3.1. The total friction developed is equal to the thrust and acts in the opposite direction. Additional restraint is not required when:

$T \leq fL\,(W_p + W_w + 2\,W_e)$ (Eq. 4.3)

where: T = $2\,P\,A\,\sin(\theta/2)$ = resultant thrust force, lbs.,

where θ is the deflection angle created by the deflected joint, deg.

f = coefficient of friction

L = length of standard or beveled pipe, ft.

W_p = weight of pipe, lbs/lin ft.

W_w = weight of water in pipe, lbs/lin ft.

W_e = earth cover load, lbs/lin ft.

The passive soil resistance of the trench backfill against the pipe was ignored in the above analysis. Depending on installation and field conditions, the passive soil resistance of the backfill may be included to resist thrust.

Tests and experience indicate that the value of f is not only a function of the type of soil, it is also greatly affected by the degree of compaction and moisture content of the backfill. Therefore, care must be exercised in the selection of f. Coefficients of friction are generally in the range of 0.30-0.40.

Determination of earth cover 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 soil density vary from 90 to 130 pounds per cubic foot, depending on the degree of compaction. Earth cover load should be taken as the prism of soil on top of the pipe:

$$W_e = wB_cH \qquad \text{(Eq. 4.3.1)}$$

where: W_e = earth cover load, lbs/lin ft.

w = unit weight of backfill, pcf

W_c = pipe outside diameter, ft.

H = height of earth cover, ft.

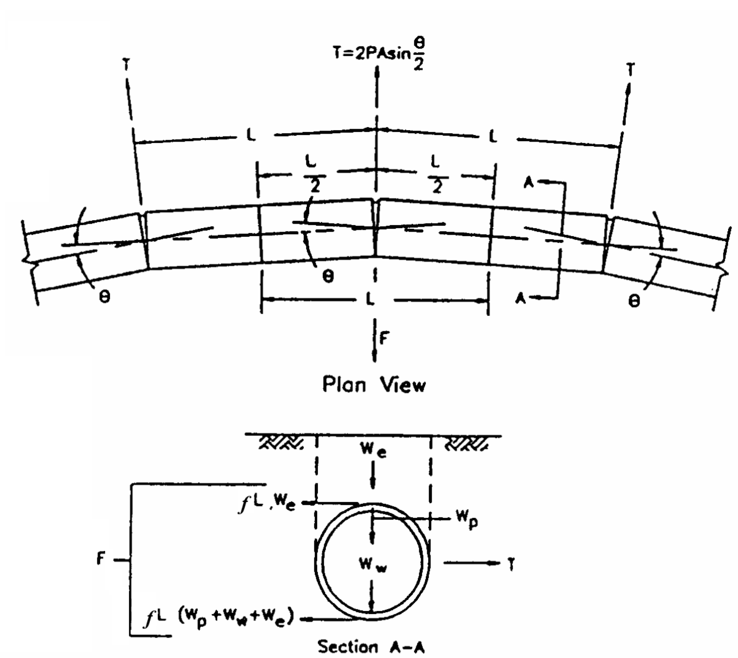


Figure 4.3.1 - Restraint of thrust at deflected joints on long radius horizontal curves

Section 4.4 Small Vertical Deflections

Uplift thrust at deflected joints on long-radius vertical curves is resisted by the combined dead weight, W_t, as shown in Figure 4.4.1. Additional restraint is not required when:

$$T \leqq fL\,(W_p + W_w + 2\,W_e)\cos(\alpha - \theta/2) \qquad \text{(Eq. 4.4)}$$

where: $T = 2\,P\,A\sin(\theta/2)$;

- L = length of standard or beveled pipe, ft.
- W_p = weight of pipe, lbs/lin ft.
- W_w = weight of water in pipe, lbs/lin ft.
- W_e = earth cover load, lbs/lin ft.
- α = slope angle, deg.
- θ = deflection angle, deg., created by angular deflection of joint

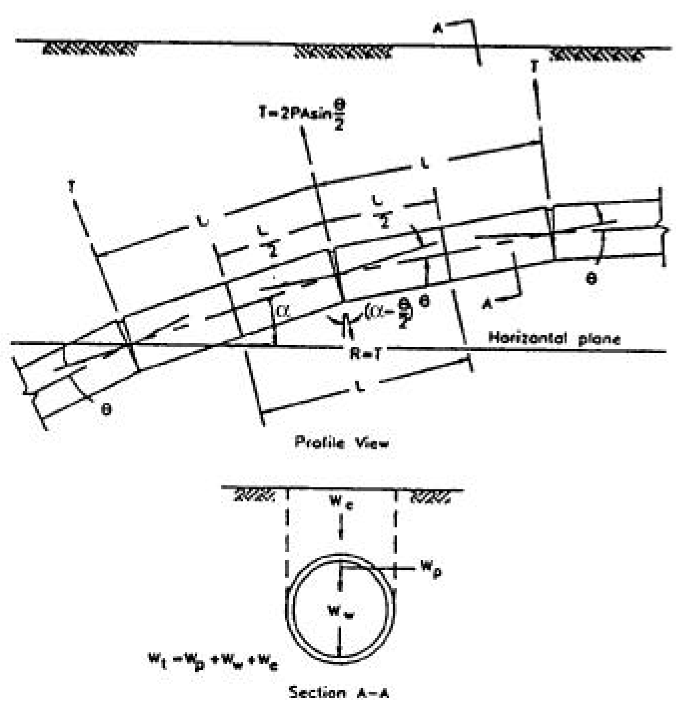


Figure 4.4.1 - Restraint of uplift thrust at deflected joints on long radius vertical curves

Downward thrust at deflected joints on long-radius vertical curves is resisted by bearing of the trench against the bottom of the pipe. Additional restraint is not required when:

$$T \leqq LBw \qquad \text{(Eq. 4.4.1)}$$

where: *T & L from above*

B = soil bearing value, lbs/sq ft.

w = bearing width, ft.

There is seldom a need to investigate thrust in this direction for properly bedded pipe.

Section 4.5 Tied Joints

Many engineers choose to restrain thrust from fittings by tying adjacent pipe joints. This method fastens a number of pipe on each side of the fitting to increase the frictional drag of the connected pipe to resist the fitting thrust. Frictional resistance on the tied pipe is assumed to act in the opposite direction of resultant thrust, *T.* Section A-A in Figure 4.4.1 shows a diagram of the external vertical forces acting on a buried pipe and the corresponding frictional resistance.

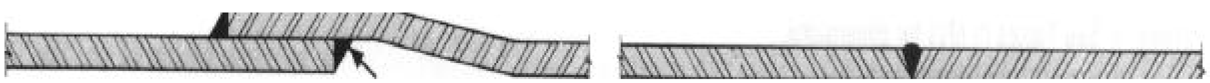
4.5.1 TYPES OF TIED JOINTS

Generally, there are two types of tied joints: (1) welded and (2) harnessed.

4.5.2 WELDED JOINTS

Figure 4.5.2.1 shows two typical details of a welded joint. Figure 4.5.2.1 (A) shows a lap welded joint. Normally, for pipe sizes larger than 30 inches, this joint is welded on the inside of the pipe. Figure 4.5.2.1 (B) shows a butt welded joint. This joint is usually specified for areas of high internal working pressures (above 400 psi).

Figure 4.5.2.1 - (A) Welded Lap Joint *(B) Welded Butt Joint*

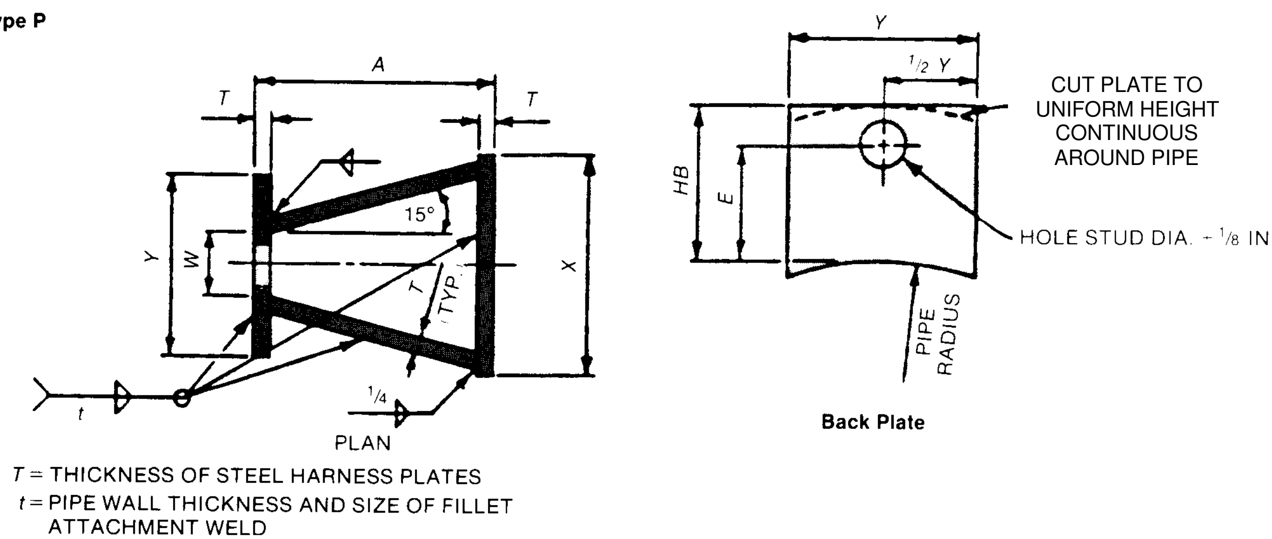


May be welded inside or outside, or both inside and outside when required.

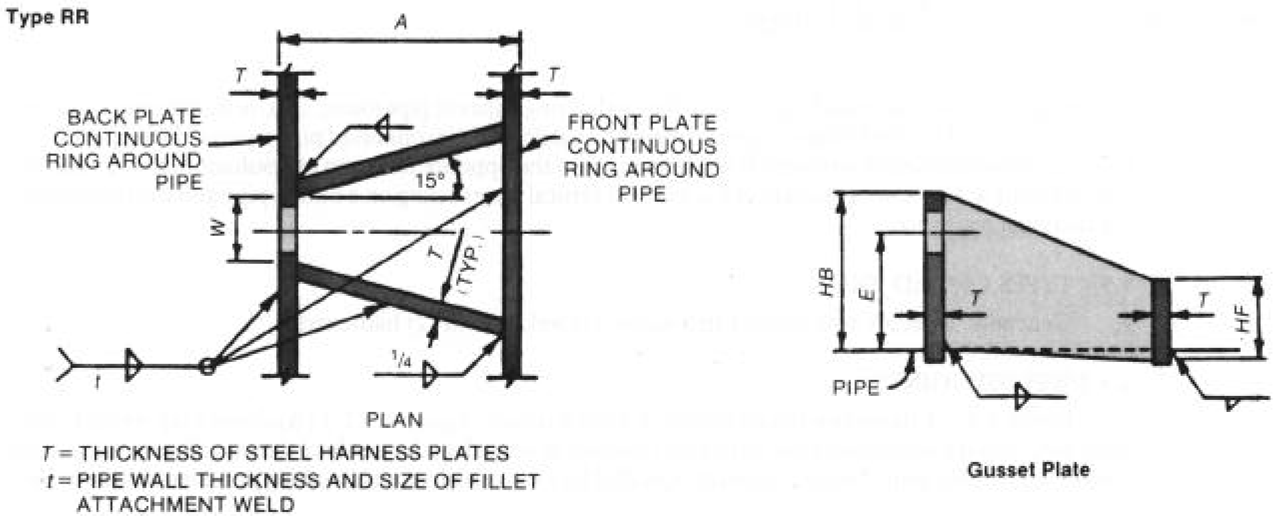
4.5.3 HARNESSED JOINTS

An alternate approach is to use harnessed joints, which provide a mechanical means of transmitting longitudinal thrust across the joints. Additional information and details on types of harnessed joints available and size and pressure limitations can be obtained from AWWA Manual of Practice M-11.

Type P



Notes: 1. See Tables in M11 for dimensions.
2. See M11 for design conditions.
3. For harness lug type P, the gusset plates between the back plate and the front plate may be perpendicular to the front and back plates with a minimum clear distance between each pair of gusset plates of dimension W.
4. The minimum weld thickness *t* shall be 3/16 in. for cylinder or wrapper thicknesses through 3/8 in.; and 1/4 in. minimum thickness for all other cylinder or wrapper thicknesses.



Notes:
1. See Tables in Ml 1 for dimensions.
2. See M11 for design conditions.
3. For harness lug type RR, the gusset plates between the back plate and the front plate may be perpendicular to the front and back plates with a minimum clear distance between each pair of gusset plates of dimension W.
4. The minimum weld thickness *t* shall be 3/16 in. for cylinder or wrapper thicknesses through 3/8 in.; and 1/4 in. minimum thickness for all other cylinder or wrapper thicknesses.

Figure 4.5.3.1 - Harness lug detail

Section 4.6 Other Uses for Restraints

Tied joints have other uses that are not related to thrust caused by internal pressure. If it is necessary to install a pipeline on a steep slope, it may be desirable to use anchor blocks, harnessed joints, or welded joints to keep the pipe from separating due to downhill sliding. Although the pipe may be capable of resisting downhill movement because of its own frictional resistance with the soil, the backfilling operation can sometimes provide enough additional downhill force to open the joint.

Section 4.7 Horizontal Bends and Bulkheads

As illustrated in Figure 4.7.1, the frictional resistance, F, needed along each leg of a horizontal bend is $P A \sin(\Delta/2)$. Frictional resistance per linear foot of pipe against soil is equal to:

$$F = f(2\, W_e + W_p + W_w) \qquad \text{(Eq. 4.7.1)}$$

where: F = frictional resistance, lbs/lin ft.
f = coefficient of friction between pipe and soil
W_e = earth cover load, lbs/lin ft.
W_p = weight of pipe, lbs/lin ft.
W_w = weight of water in pipe, lbs/lin ft.

Determination of the earth load is similar to the previous section. Tests conducted on buried pipe have shown that resistance to pipe movement approximately doubles after rainfall or if jetting of the trench consolidates the soil around the pipe. The length of pipe L to be tied to each leg of a bend is calculated as:

$$L = \frac{PA\sin(\Delta/2)}{f\,(2\,W_e + W_p + W_w)} \quad \text{(Eq. 4.7.2)}$$

where: L = length of pipe tied to each bend leg, ft.
P = internal pressure, psi
A = cross-sectional area of pipe joint, sq. in.
Δ = deflection angle of bend, deg.
f = coefficient of friction between pipe and soil
W_e = earth cover load, lbs/lin ft.
W_p = weight of pipe, lbs/lin ft.
W_w = weight of water in pipe, lbs/lin ft.

The length of pipe to be tied to a bulkhead is:

$$L = \frac{PA}{f\,(2\,W_e + W_p + W_w)} \quad \text{(Eq. 4.7.3)}$$

where: L = length of pipe tied to bulkhead, ft.
and all other variables are as defined above.

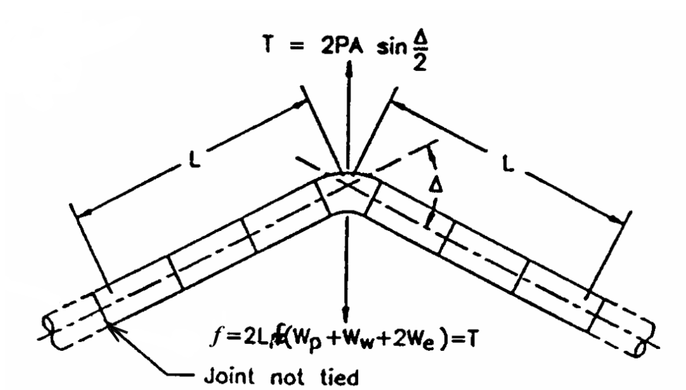


Figure 4.7.1 - Thrust restraint with tied joints at bends

DESIGN EXAMPLE 1: HORIZONTAL ELBOW

Given:

Pipe size	36 in.
Outside diameter	37 $^5/_8$ in.
Test pressure	150 psi
Bend angle	75 deg.
Earth cover	6 ft.

Design assumptions:

Soil density	100 pcf
Coefficient of friction	0.3
Pipe weight	135 lbs/lin ft.

Calculations:

Water weight $3.1416(36/24)^2(62.4) = 441$ lbs/lin ft.
Earth load $(37.625/12)(6)(100) = 1881$ lbs/lin ft.
Cross-sectional area at joint $3.1416(37.625/2)^2 = 1112$ sq. in.

$$L = \frac{PA\sin(\Delta/2)}{f\,(2W_e + W_p + W_w)}$$

$$= \frac{(150)(1112)[\sin(75/2)]}{0.3[2(1881) + 135 + 441]}$$

= 78 ft. on each side of the bend P.I.

DESIGN EXAMPLE 2 Dead End

Given: Same as Example 1, except for bulkheaded condition:

$$L = \frac{PA}{f\,(2\,W_e + W_p + W_w)} \qquad \text{(Eq. 4.7.3)}$$

$$= \frac{(150)(1112)}{0.3[2(1881) + 135 + 441]}$$

= 128 ft. out from the bulkhead

Section 4.8 Vertical Bends

The dead weight resistance needed along each leg of a vertical bend is $PA\ \sin(\Delta/2)$. Dead weight resistance per linear foot of pipe in a direction opposite to thrust is:

$$D = (W_e + W_p + W_w)\cos(\alpha - \Delta/2) \qquad \text{(Eq. 4.8.1)}$$

where: D = dead weight resistance, lbs/lin ft.
W_e = earth cover load, lbs/lin ft.
W_p = weight of pipe, lbs/lin ft.
W_w = weight of water in pipe, lbs/lin ft.
α = slope angle, deg.
Δ = deflection angle of bend, deg.

Length of pipe L to be tied to each leg of a vertical (uplift) bend is calculated as:

$$L = \frac{PA\ \sin(\Delta/2)}{(W_e + W_p + W_w)\cos(\alpha - \Delta/2)} \qquad \text{(Eq. 4.8.2)}$$

with variables as defined above. Slopes above 20 degrees should have all the joints restrained. Use eq. 4.8.2 only when solving for lengths of restrained joints on slopes less than 20 degrees.

Section 4.9 Thrust Blocks

Thrust blocks increase the ability of fittings to resist movement by increasing the bearing area. Typical thrust blocking of a horizontal bend is shown in Figure 4.9.1.

4.9.1 CALCULATION OF SIZE

Thrust block size can be calculated based on the bearing capacity of the soil:

Area of Block $= L_b \times H_b = (T/\sigma)$ (Eq. 4.9.1)

where: $L_b \times H_b$ = area of bearing surface of thrust block, sq. ft.

T = thrust force, lbs.

σ = safe bearing value for soil, psf

If it is impractical to design the block for the thrust force to pass through the geometric center of the soil bearing area, then the design should be evaluated for stability.

After calculating the thrust block size based on the safe bearing capacity of soil, the shear resistance of the passive soil wedge behind the thrust block should be checked since it may govern the design. For a thrust block having its height, H_b less than one-half the distance from the ground surface to base of block, h, the design of the block is generally governed by the safe bearing capacity of the soil. However, if the height of the block exceeds one-half h, then the design of the block is generally governed by shear resistance of the soil wedge behind the thrust block. Determining the value of the safe bearing and shear resistance of the soil is beyond the scope of this manual. Consulting a qualified geotechnical consultant is recommended.

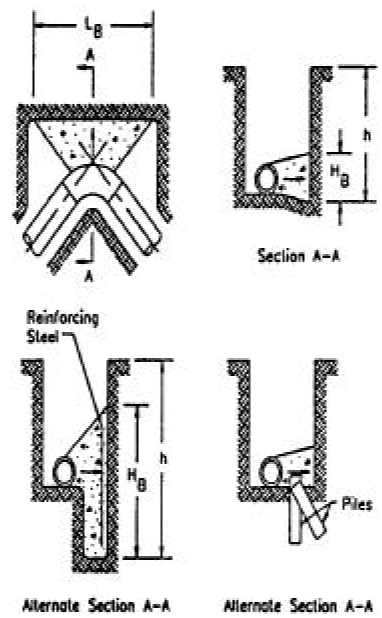


Figure 4.9.1 - Typical thrust blocking of a horizontal bend

Section 4.10 Typical Configurations

Determining the safe bearing value, σ, is the key to "sizing" a thrust block. Values can vary from less than 1000 pounds per square foot for very soft soils to several tons per square foot for solid rock. Knowledge of local soil conditions is necessary for proper sizing of thrust blocks. Figure 4.9.1 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 4.10.1 shows typical thrust blocking of vertical bends. Design of the block for a bottom bend is the same as for a horizontal bend, but the block for a top bend must be sized to adequately resist the vertical component of thrust with dead weight of the block, bend, water in the bend, and overburden.

Uplift thrust restraint provided by gravity type thrust blocks shown for the top bend in Figure 4.10.1 may also be provided by the alternate method of increasing the dead weight of the line by tying adjacent pipe to the vertical bend. Section A-A in Figure 4.4.1 shows a diagram of the vertical forces acting on a buried vertical (uplift) bend used in determining the thrust resistance by dead weight.

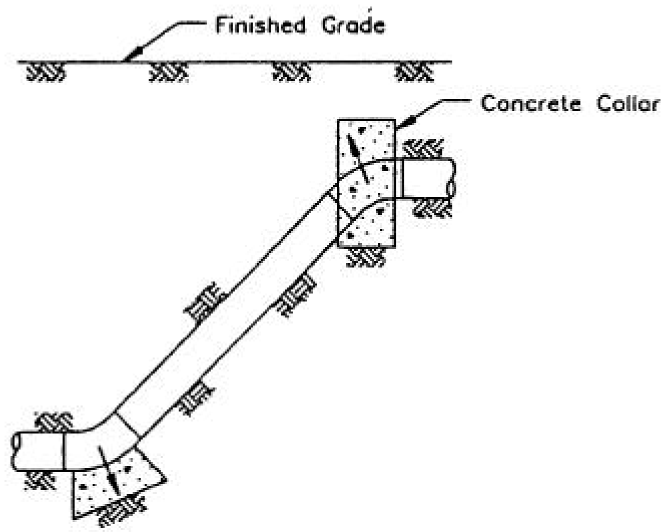


Figure 4.10.1 - Typical profile of vertical bend thrust blocking

Section 4.11 Proper Construction

Most thrust block failures can be attributed to 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 perpendicular to the direction of and centered on the line of action of the thrust. A surprising number of thrust blocks fail due to inadequate design or improper construction. Many people involved in construction and design do not realize the magnitude of the thrusts involved. As an example, a thrust block behind a 36-inch, 90-degree bend operating at 100 psi must resist a thrust force in excess of 100,000 pounds. Another factor frequently overlooked is that thrust increases in proportion to the square of pipe diameter. A 36-inch pipe produces approximately four times the thrust produced by an l8-inch pipe.

4.11.1 ADJACENT EXCAVATION

Even a properly designed and constructed thrust block can fail if the soil behind it is disturbed. Thrust blocks of proper size have been poured against undisturbed soil only to fail because another utility or an excavation immediately behind the block collapsed when the line was pressurized. The problem of later excavation behind thrust blocks has led many engineers to use tied joints.

4.11.2 ANCHOR RINGS

Anchor Rings for use in concrete anchor blocks or concrete walls are often required. These are usually specified when the pipe goes through a structure (such as a valve vault) or to anchor the pipe at the top and bottom of a steep slope. More detailed design information on anchor rings can be found in AWWA M-11.

CHAPTER 5

Pipe Joints

Section 5.1 Types

For a buried steel penstock, three types of field joints are commonly used depending on pipe diameter, wall thickness, pressures and the terrain or other considerations:

BELL AND SPIGOT JOINTS WITH RUBBER GASKETS are used for pipe diameters up to 72” and wall thickness through 3/8” for working pressures up to 250 psi. This type of joint allows for some angular deflection at the joint and is the least costly to install. When using this type joint, restraint or anchoring must be considered at all points of longitudinal thrust (see Section 2.0). AWWA C-200 Standard covers specifications for this type joint. Common types are shown in Figure 5.1.1, A-l, A-2 and A-3.

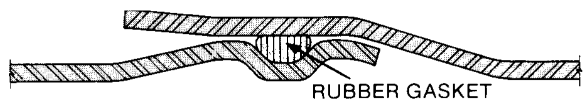


Figure 5.1.1, A-l - Rolled-groove rubber gasket joint

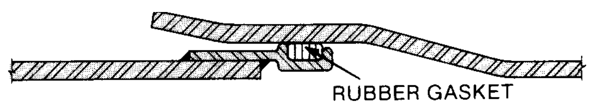


Figure 5. 1.1, A-2 - Carnegie-shape rubber gasket joint

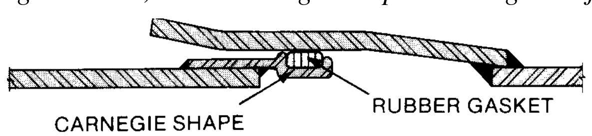
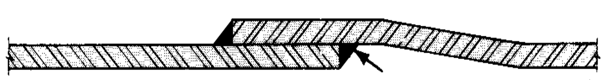


Figure 5. 1.1, A-3 - Carnegie-shape rubber gasket joint with weld-on bell ring

WELDED LAP JOINTS are used for all pipe diameters for working pressures up to 400 psi. This type of joint allows for some angular deflection at the joint and the joint can be assembled very quickly in the field, resulting in a considerable savings over butt welded joints. The lap welded joint may be welded on the outside only, or for sizes over 24” diameter, it may be welded on the inside only. In certain special conditions, it may be desirable to weld on both the inside and outside and to perform an air test on the joint. AWWA C-200 Standard covers specification for this type of joint and AYUWA C-206 covers the welding of this type of joint. Figure 5.1.1, B shows this joint.



May be welded inside or outside, or both inside and outside when required.

Figure 5. 1. 1, B - Welded Lap Joints

WELDED BUTT JOINTS are suggested for working pressures over 400 psi. This type of joint does not allow angular deflection at the joint except by miter cut of the pipe ends. Deflections of up to 5° can be taken by mitering one end of a pipe and deflections of up to 10% can be taken by equal miter cuts of both pipe ends. Fit up of this type of joint is more difficult and is usually accomplished by the use of line-up clamps or by the use of a back-up bar welded to one end of the pipe. A full penetration groove weld is required for this joint. AWWA C-200 Standard covers specifications for this type of joint and AWWA C-206 covers the welding of this type of joint. ANSI/AWS D.1.1 Fig. 5.1.2 below shows several details for this joint.

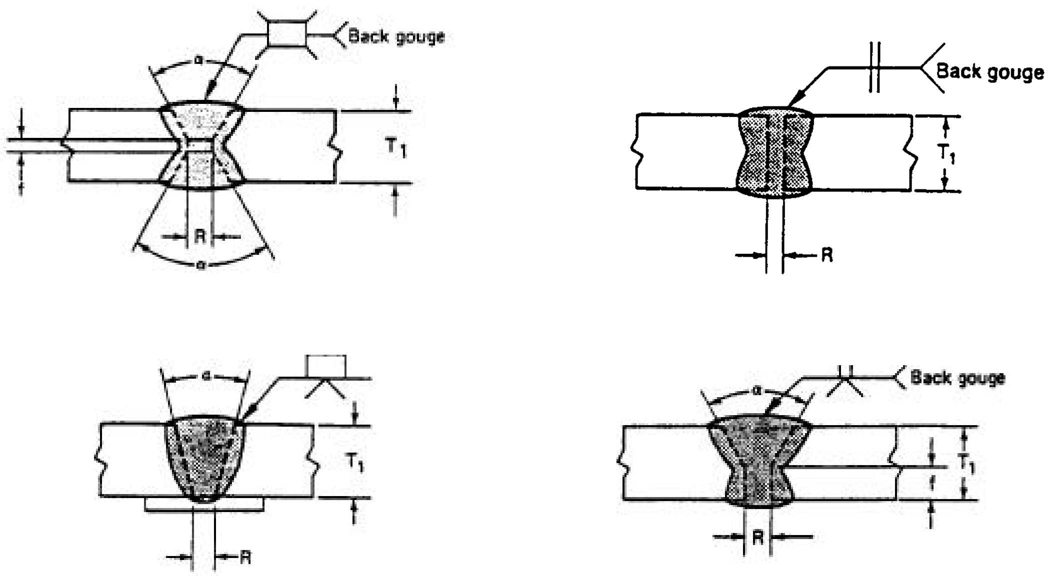


Figure 5.1.1, C-1 – Prequalified complete joint penetration groove welded joints

BUTT STRAP JOINTS are used for closures or for temperature stress control joints on butt welded lines. The butt strap may be shipped loose in one or two sections requiring a longitudinal field weld or it may be shop welded to the end of one pipe. In certain conditions, it should be welded inside and outside, in which case, tapped holes for an air test as shown in AWWA C-206 standard can be provided. See Figure 5.1.1, C.

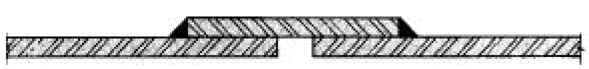


Figure 5.1.1, C-2 - Butt strap joint

MECHANICAL COUPLINGS are used on pipelines of all diameters and especially on lined pipe too small for a person to enter. Very complete technical data have been published. A typical solid sleeve coupling is shown in Figure 5.1.1, D. AWWA C219 Standard covers this type of coupling.

Mechanical couplings provide tightness and strength with flexibility. They relieve expansion and contraction forces in a pipeline and provide sufficient flexibility so that pipe may be laid on long radius curves and grades without the use of specials. The rubber gaskets are firmly held between the coupling parts and the pipe, and they join the lengths securely against high pressure, low pressure, or vacuum. The completely enclosed rubber gaskets are protected from damage and decay. These joints have been used successfully since 1891.

Acceptable axial movement in flexible solid sleeve couplings results of a maximum of 3/8” from shear displacement of the rubber gaskets rather than from sliding of the gaskets on the mating surface of the pipe. If greater displacement is needed, true expansion joints should be provided rather than mechanical couplings.

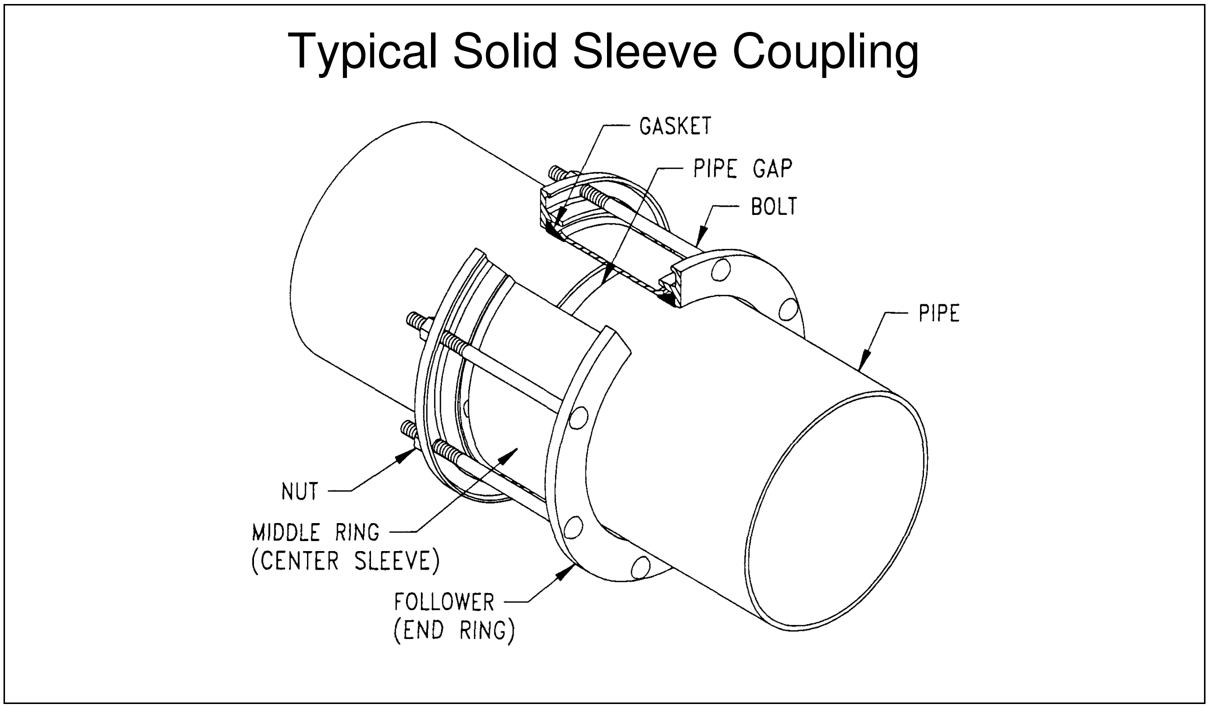


Figure 5.1.1, D – Mechanical joint

Unrestrained mechanical couplings transmit only minor tension or shear stresses across pipe joints, and they will not permit differential settlement at the joints when used alone. However, a degree of flexibility is possible when used in conjunction with another adjacent flexible joint. Mechanical couplings are suitable for joining buried or exposed anchored pipes that are laid on curves established using deflections up to the maximum permitted at the coupling.

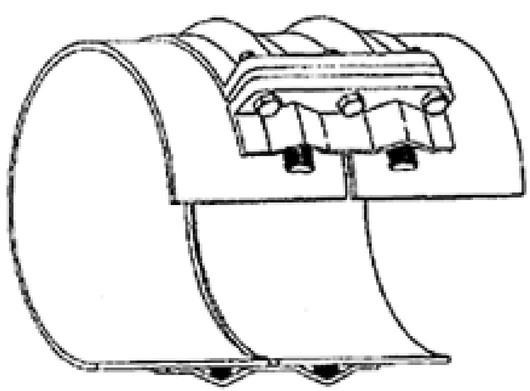


Figure 5.1.1, D-2 – Typical Split Sleeve Coupling

Split sleeve couplings are often utilized as an alternate to solid sleeve couplings. Split sleeve couplings can be designed to allow for greater pipe out-of-roundness (barrel deflection), complete joint restraint, expansion and contraction greater than 3/8 in. and many other common (or uncommon) field conditions. A typical split sleeve coupling is shown in Figure 5.1.1, D-2. The materials, quality and performance of split sleeve type couplings shall conform to the requirements of AWWA C219.

FLANGED JOINTS are used for connecting to valves, turbines and other appurtenances. The standard flanges with pressure ratings for cold water service (-20° to 100°F) are shown below. Other flanges designed to ASME Boiler and Pressure Vessel Code, Section VIII, Division I Standard may also be used.

AWWA C207 Class B 86 psi
AWWA C207 Class D 175 psi 4” to 12”
AWWA C207 Class D 150 psi over 12”
AWWA C207 Class E 275 psi
AWWA C207 Class F 300 psi
ANSI B16.5 Class 150 285 psi
ANSI B16.5 Class 300 740 psi
ANSI B16.5 Class 400 990 psi

CHAPTER 6

Linings and Coatings

Introduction: It is the purpose of this section to assist in the proper selection of protective linings and coatings. The selection of proper protective linings and coatings is critical in order to insure long term protection. The linings and coatings referred to herein are believed to be the most reliable as proved in practice. Some of the following standards apply to both linings and coatings and some to either as follows:

Section 6.1 Exterior and/or interior systems

SYSTEM I

AWWA C-203: Coal-Tar Protective Coatings and Linings for Steel Water Pipelines-Enamel and Tape Hot Applied This standard describes the material and application requirements for shop-applied coal-tar protective coatings and linings for steel water pipelines intended for use under normal conditions when the temperature of the water in the pipe will not exceed 90º F (32º C). The standard covers coal tar enamel applied to the interior and exterior of pipe, special sections, connections and fittings; and hot-applied coal tar tape applied to the exterior of special sections, connections and fittings.

Coal-tar enamel is applied over a coal tar or synthetic primer. External coal tar enamel coatings use bonded felt and fibrous glass mat to reinforce and shield the coal tar enamel. The applied external coating is usually finished with either a coat of whitewash or a single wrap of Kraft paper.

Internally, the coal tar enamel is used without reinforcement or shielding. The hot enamel is spun into the pipe and provides a smooth internal lining which has low hydraulic frictional resistance.

The standard provides a rigid yet reasonable manufacturer's guide for the production of the coating, calls for tests of material and its behavior which assure the purchaser that the product has the desired qualities, and furnishes directions for the effective application of the coating.

SYSTEM II

AWWA C-2O5: Cement-Mortar Protective Lining and Coating for Steel Water Pipe - 4" and larger - Shop Applied. C205 describes the material and application requirements to provide protective linings and coatings for steel water pipe by shop application of cement mortar.

Cement mortar is composed of Portland cement, sand and water well mixed and of the proper consistency to obtain a dense, homogenous lining or coating. Cement mortar linings are limited to services with flow velocity of 20 ft./sec. or less. Externally, the coating is a reinforced cement mortar pneumatically or mechanically applied to the pipe surface. Reinforcement consists of spiral wire, wire fabric or ribbon mesh. The standard provides a complete guide for application and curing of the mortar lining and mortar coating.

SYSTEM III

*Coal-Tar Epoxy for the Interior or Exterior of Steel Water Pipe***.** This describes the material and application requirements for shop-applied coating system, suitable for potable and non-potable water service which will provide corrosion protection to the interior and exterior of steel water pipe, fittings and special sections installed underground or underwater. The system consists of one or more coats of a two-component coal tar epoxy yielding a minimum of 16 mils. D.F.T. The surface preparation and application parameters are to be per the paint manufacturer's recommendation. The paint shall be a minimum of 70% solids by volume. Adhesion shall be not less than 800 psi, average of 3 trials per ASTM D 4541 method. Abrasion resistance per ASTM D 4060 method, G-17 wheel, 1000 gm load shall be no more than 170 mg. loss after 1000 cycles.

SYSTEM IV

AWWA C210, Standard for Liquid Epoxy Coating Systems for the Interior and Exterior of Steel Water Pipelines. AWWA C210 describes a liquid epoxy coating system, suitable for potable water service, which will provide corrosion protection to the interior and exterior of steel water pipe, fittings, and special sections installed underground or underwater. The coating system consists of one coat of a two-part chemically cured inhibitive epoxy primer, and one or more coats of a two-part chemically cured epoxy finish coat. The finish coat may be a coal-tar epoxy coating, or it may be an epoxy coating containing no coal tar. The coating system may alternately consist of two or more coats of the same epoxy coating without the use of a separate primer, provided the coating system meets the performance requirements of AWWA C210.

These coatings are suitable when used for corrosion prevention in water service systems at temperatures up to 140º F (60º C). The products are applied by spray application, preferably airless.

The liquid epoxy system described in the standard differs from the customary product commercially available in that it has a very high flexibility, elongation, and impact resistance. Any liquid epoxy offered for water utility purposes must meet the requirements of AWWA C210.

Section 6.2 Exterior Systems

SYSTEM V

AWWA C-209: Cold-Applied Tape Coatings for Special Sections, Connections and Fittings for Steel Water Pipelines. AWWA C2O9 has been issued to incorporate the use of a cold primer and cold-applied tape on the exterior of special sections, connections and fittings for steel water pipelines installed underground in any soil under normal or average conditions. Tapes with both polyvinyl chloride and polyethylene backing are listed in AWWA c209. The thicknesses of the tapes vary; however, all tapes may be sufficiently overlapped to meet changing performance requirements. Cold-applied tapes provide ease of application without the use of special equipment and can be applied over a broad application temperature range. If severe construction or soil conditions exist where mechanical damage may occur, a suitable overwrap of an extra thickness of tape or other wrapping may be required.

SYSTEM VI

AWWA C-214: Tape Coating Systems for the Exterior of Steel Water Pipelines. This standard covers the materials, the systems and the application requirements for prefabricated cold-applied tapes for the exterior of all diameters of steel water pipe placed by mechanical means. The fabricated cold-applied tape coating shall be at least three layers consisting of (a) primer (b) corrosion protection and (c) an outer-layer for mechanical protection for a total of 50 mil thickness for normal conditions and diameters to 54”. For larger diameters or unusually severe conditions, an additional outer-layer of 30 mils is recommended. This standard covers application at coating plants.

The primer is supplied in the form of a liquid consisting of solid ingredients carried in a solvent. The corrosion preventive tape and the mechanical protective tape are supplied in suitable thicknesses and in roll form.

Section 6.3 Interior Systems

SYSTEM VII

AWWA C-602: Cement-Mortar Lining of Water Pipelines- 4” and larger - In Place. This standard describes the materials and application processes for the cement-mortar lining of pipelines in place including both newly installed pipes and older pipelines. Detailed procedures are included for surface preparation and application, surface finishing, and curing of the cement mortar. Cement mortar linings are limited to services with flow velocity of 20 ft./sec. or less.

SYSTEM VIII

Single Component High Build Coal Tar Solution System for the Interior of Steel Water Pipe. This describes the material and application requirements for shop-applied coating system, suitable for potable and non-potable water service which will provide corrosion protection to interior surfaces of steel water pipes, fittings and special sections installed underground or underwater. This system consists of one or more coats of a single component ambient cured high build coal tar solution yielding a minimum of 8 mils. D.F.T. Surface preparation and application parameters to be per paint manufacturer's recommendations. This paint volume by solids shall be a minimum of 61%.

Section 6.4 Coating Application

This manual does not furnish details on methods of coating and paint application, but the importance of obtaining proper application cannot be overemphasized. Effective results cannot be secured with any coating material unless adequate care is taken in preparing the surfaces for coating, in applying the coating, and in handling the pipe after coating. AWWA standards provide the requirements for obtaining good coating work. The coating manufacturer, the applicator, and the engineer should all cooperate to see that the work is of the prescribed quality. Many excellent sources of information have been published dealing with the protection of steel pipe, the pitfalls of coating work, and the means of avoiding these problems.

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CHAPTER 7

Pipe Manufacturing

Section 7.1 Manufacturing for the AWWA Type Penstock

7.1.1

ANSI/AWWA C200 Standard covers electrically butt-welded straight-seam or spiral-seam pipe 6" in diameter and larger intended for use in conveyance of water. Section 3 and 4 of AWWA C200 covers the fabrication of pipe and fittings from plates or sheets with butt welded straight seams including: welder qualifications, production weld tests, shop hydrostatic testing, end preparation and allowable tolerances. This standard is intended for shop fabricated pipe which limits the diameter to about 14 feet. The wall thickness for pipe fabricated to this standard should not exceed 1½ and design stress at normal condition should not exceed 21,000 psi, except that this may be exceeded with additional testing.

7.1.2

Section 3 of AWWA C200 also includes pipe manufactured to ASTM A139 Standard. This standard covers double submerged arc welded pipe manufactured from steel coils on a spiral weld machine where the diameter of the pipe is controlled by the angle of the coil feed to the axis of the pipe, in relation to the width of the coil. In this process of manufacturing pipe, the steel coil is unrolled, the coil edges are prepared as required, and the coil is formed continuously on forming rolls. The first spiral weld is made on the inside of the pipe at the invert and the second weld is made on the outside of the pipe when the spiral seam reaches the top of the pipe. In this method of manufacturing pipe, the forming and welding process are continuous until the coil is used up and/or the mill is stopped for a coil splice or other reasons. Pipe lengths are cut off to length as required by an automated torch as the pipe comes off the mill. Pipe ends are then prepared for the required field joints including expansion to meet the required joint tolerance. This pipe is available in diameters from 16" to 156" and in wall thickness from $^1/_8$" to $^7/_8$". This standard covers the steel specification and the manufacturing and testing of the pipe. Pipe manufactured to ASTM A-139 Standard requires qualified welding procedures, production weld tests, and shop hydrostatic test at 60 to 80% of yield as proof of quality.

7.1.3

Section 3 of AWWA C200 also includes pipe manufactured to ASTM A-53 Type E - Electric-resistance welded or Type S - Seamless in diameters through 26" in Grades A&B.

7.1.4

Pipe manufactured to these standards has been used successfully on water works projects for many years and should be considered acceptable for use in the AWWA M- 11 designed Penstock.

Section 7.2 Fabrication or Manufacturing of Pipe for the "Traditional Penstock"

Shop fabrication of pipe and fittings for this type of penstock shall conform to either: (1) "ASME Boiler and Pressure Vessel Code-Section VIII, Division 1, covering fabrication, inspection and tests plus 7.2.1 Specific Additional Requirements, or: (2) Section 7.2.2 "Supplemental Requirements to ANSI AWWA C-200, ASTM A-139 Standards-for the Manufacture of Spiral Weld Pipe for the Traditional Penstock."

Section 7.2.1 Specific Additional Requirements to (1) ASME B&PV Code Section VIII, Division 1.

a. Forming -The plates shall be cut accurately to size and shape with edges prepared (beveled) to suit the welding procedure. Plates shall be bent or rolled to true circular shape with curvature continuous from the edge of the plates. Any out of roundness shall be in the form of a smooth oval that can be jacked back to a circular shape. Pipe with a *D/t* ratio over 120 shall be stulled, prior to shipment, with internal temporary braces as required to maintain roundness.

The measured difference between the maximum and minimum diameters at the stulls shall not exceed 1% of the nominal pipe diameter. The radial offset of plate edges at the welds shall not exceed those shown in Table UW33.

There shall not be more than 2 longitudinal seams in a pipe and girth seams shall be spaced at least 8’ apart except for specials. Longitudinal seams shall be staggered at least 30º at girth welds.

b. Welding - all longitudinal and girth welds shall be double-welded butt joints with complete penetration. All welding shall be done by a process that protects the molten metal from the atmosphere, and where practical, automatic machines shall be used. All welding shall be in accordance with the requirements of the ASME Code. Where weld metal is deposited in successive layers, each layer shall be cleaned thoroughly before the subsequent layer is deposited. Particular care shall be taken in aligning and separating the edges of plates to be joined by butt welding so that complete penetration and fusion of the welds will be assured. After the welding is completed, all weld spatter shall be removed. The Contractor shall protect the work and the operator from the wind, rain and snow during welding operations. No welding of any kind shall be done on wet surfaces or when the temperature of the steel is lower than 0º F. At temperatures between 0 and 32º F, the surface of all areas within 3 inches of the point where a weld is to be started shall be heated to at least 60º F or as required by the approved welding procedure. All postweld heat treatment and preheat requirements for wye branch or bifurcation reinforcements shall be in accordance with the requirements of the ASME Code: (1) Material over 1½” nominal thickness and (2) Materials over 1¼” nominal thickness through 1½” nominal thickness unless preheat is applied at a minimum temperature of 200º F during welding.

c. Qualification of welders and welding procedures - All welders and welding procedures shall be qualified in accordance with Section IX of the ASME Code. If any procedure has been previously qualified under the ASME Code, requalification will not be required. If in the opinion of the Contracting Officer, the work of any welder any time appears questionable, such welder shall be required to pass another qualification test. The test plates from which the specimens are to be prepared for qualification tests of welders shall be of such dimensions that all of the specimens required for the tests can be obtained with a discard at each end of test plates of not less than 2 inches. The test plates shall be made of identical material used in the fabrication of the steel penstock, and the technique including the rate, position, and welding electrodes used shall be identical to that actually required for fabrication. The preparation of all test plates shall be witnessed by the Contract Officer. All qualification tests of welders and tests of welding procedures including the physical test of the specimens welded in the qualifications tests shall be made by the Contractor who shall if required supply to the Contracting Officer certified copies of reports of physical tests of specimens welded in the qualification tests.

d. Radiographic/radioscopic inspection of welds - The entire length of all girth and longitudinal welds and all butt welds in wye branch, or bifurcation joints shall be radiographed, except that rod to plate and plate segment butt welds of the wye branch or bifurcation reinforcing plates may be ultrasonically tested at the Contractor’s option. The radiographic/radioscopic and ultrasonic examination shall be in accordance with the requirements of the ASME Code. Before making radiographs of the welds, the Contractor shall place suitable identification markers adjacent to the welds. The markers shall be painted, stamped, or fastened to the shells as directed by the Contracting Officer and shall not be removed until all of the welds in one joint have been accepted. In addition, corresponding markers shall be temporarily provided at each film location so that the images of these markers will appear on the radiographs. All radiographs and records of ultrasonic testing shall be delivered to the Contracting Officer or his representative who shall judge the acceptability of all welded joints. Defects in welds shall be repaired in accordance with the requirements of the ASME Code. Portions of welds that have been repaired shall be radiographed.

The method of radiographing the welds and the equipment and the technique used shall be subject to the approval of the Contracting Officer. The Contractor shall prepare and supply a marking diagram of the steel penstock, showing the location of each radiograph in each welded joint. All radiographs shall become the property of the Owner and shall be forwarded, properly identified, to the Owner.

e. Magnetic particle and liquid penetrant inspection of welds -The entire length of all fillet welds and groove welds joining wye branch or bifurcation reinforcing plate to pipe shell, shall be inspected by the magnetic particle or liquid penetrant method in accordance with the requirements of ASME Code.

f. Hydrostatic test - All straight pipe sections shall be shop hydrostatically tested to a pressure of 1½ times the working pressure (for penstocks the working pressure would be the normal operating condition #l) or to a pressure that will produce a stress not exceeding 80% of the minimum yield strength of the steel, whichever is less, as determined by the formula:

$P = 2St/D$

where: P = hydrostatic test pressure (psi)
S = .8 times the minimum yield of the steel (psi)
t = wall thickness (inches)
D = specified outside diameter of the pipe (inches)

Any pipe sections with nozzles, reducers or wye branches shall be hydrostatically tested after welding is completed. Miter cuts and elbows may be fabricated from previously hydrostatically tested pipe provided all welding done after the shop hydrostatic test is 100% inspected by radiographic or ultrasonic examination.

The hydrostatic test pressure shall be held for a minimum of 10 seconds or as long as required to visually observe the welds on the pipe section. There shall be no leaks. The hydrostatic tester shall be equipped with a recording gage that will record the test pressure and duration of time it is applied to each length of pipe. Records or charts shall be available for the purchaser's examination at the plant.

g. Charpy Impact Tests - When specified, Charpy Impact Tests shall be taken conforming to the methods described in ASTM A-370. Tests shall be transverse to the direction of final rolling.

h. Tolerances - The pipe shall be round with a maximum difference between maximum and minimum diameters at any one point of 1% of the nominal O.D. of the pipe. The out of roundness shall be in the form of a smooth oval that may be jacked back to a circular shape. Pipe with a *D/t* ratio greater than 120 shall be stulled prior to shipment with internal temporary wooden braces as required to maintain this roundness.

Tolerance on straightness shall be: Finished pipe shall not deviate by more than " from a 10' long straight edge held against the pipe in a longitudinal direction.

Specified pipe lengths shall not vary more than ± 2" unless other tolerances are agreed to by the customer and the fabricator.

Specified wall thickness shall not vary more than the allowable mill tolerance for the specified ASTM steel.

i. Preparation of Pipe Ends - Ends of pipe sections shall be of the type specified by the purchaser. All pipe ends shall be smooth and free of notches, weld splatter, and burrs.

(1) Ends for mechanically coupled field joints shall be as specified by the purchaser and shall be plain, grooved, or banded. The outside of ends of plain-end pipe shall be free from surface defects and shall have the longitudinal or spiral welds ground to surface for a sufficient distance from the ends to permit the pipe to make a watertight joint with the coupling. Grooved or banded ends shall be prepared to fit the type of mechanical coupling to be used.

(2) Ends for lap joints for field welding. The bell ends shall be formed by expanding with segmental dies on a hydraulic expander, pressing on a plug die, or by rolling. After forming, the minimum radius of the curvature of the bell end at any point shall not be less than 15 times the nominal thickness of the steel shell. Bell ends shall be formed in a manner to avoid impairment of the physical properties of the steel shell. Joints shall permit a lap, when the joint is assembled, of 1½" or 3 times the thickness of the pipe shell, whichever is greater. Bell depth shall be such that no part of the field weld shall be closer than 1" to the nearest point of tangency to a bell radius. The longitudinal or spiral weld on the inside of the bell end and the outside of the spigot end on each section of pipe shall be ground flush with the plate surface. The inside edge of the bell and the outside edge of the spigot shall be scarfed or lightly ground to remove sharp edges or burrs.

(3) Plain-end pipe. Pipe shall be furnished with a plain right-angle cut. All burrs at the ends of the pipe shall be removed.

(4) Beveled ends for field-butt welding. When pipe is specified to have the ends beveled for field-butt welding of circumferential joints, the ends shall be beveled to an angle of 30º, measured from a line drawn at right angles to the axis of the pipe, with a tolerance of + 5º, - 0º, and with a width of root face (or flat at the end of the pipe) of 1/16º ± 1/32º unless otherwise ordered. For large diameter pipe with wall thickness ¾" or less, an alternative butt weld detail is recommended using an outside backup bar with inside bevel in accordance with approved field welding procedure. (Example ANSI/AWS D1.1 Fig. 2.9.1). For wall thickness over ¾" a double bevel should be considered.

(5) Ends fitted with butt straps for field welding. Ends of the pipe to be fitted with butt straps for field welding shall comply with details shown on the drawings supplied by the purchaser. Butt straps may be made in halves or as complete cylinders. They shall be welded to the pipe by the manufacturer or shipped separately, as required by the purchaser. The weld at the pipe ends and inside the butt straps shall be ground flush with the plate surfaces for a distance sufficient to facilitate installing the butt strap.

(6) Bell-and-spigot ends with rubber gasket. Bell-and-spigot ends shall be so designed that when the joint is assembled, it will be self-centered and the gasket will be restrained or confined to an annular space in such a manner that movement of the pipe or hydrostatic pressure cannot displace it. When the joint is completed, compression of the gasket shall not be dependent on water pressure in the pipe and shall be adequate to ensure a watertight seal when subjected to the specified conditions of service. The bell-and-spigot ends shall conform to the drawings submitted by the manufacturer and approved by the purchaser.

NOTE: AWWA Manual M 11 shows several types of bell-and-spigot joints with rubber gasket. Other types are available from various pipe manufacturers.

(a) Fabrication. Bell-and-spigot ends may be formed integrally with the steel cylinder or may be fabricated from separate plates, sheets, or special sections for attachment to pipe ends. Bell ends formed integrally with the cylinder shall be shaped either by pressing over a machined swage or die, or by sizing with an internal expander. Spigot ends may be formed integrally with the steel cylinder by rolling with suitable equipment, or by welding a preformed shape to the spigot end of the pipe to form a groove of the proper configuration. All welds on the inside of the bell and outside of the spigot shall be ground even with the plate surface for a distance not less than the depth of insertion.

(b) Rubber gaskets. The pipe manufacturer shall supply a continuous rubber gasket of such cross section so that continuous contact is attained with the bell and the spigot with sufficient pressure to maintain the seal under all permissible assembly conditions. Gaskets shall be of sufficient volume to substantially fill the recess provided when the pipe joint is assembled. The gasket shall be the sole element depended on to make the joint watertight. Gaskets shall have smooth surfaces free from pitting, blisters, porosity, and other imperfections. The rubber compound shall contain not less than 50 percent of volume of first-grade synthetic rubber or synthetic-rubber blends. The remainder of the compound shall consist of pulverized fillers free from rubber substitutes, reclaimed rubber, and deleterious substances. The compound shall contain 10 to 20 parts per hundred of type SBR-1500 additive (styrene buthene rubber) to reduce the effects of hysteresis. The compound shall meet the following physical requirements when tested in accordance with appropriate ASTM standards:

Tensile strength: 2300 psi minimum (ASTM D412).

Elongation at rupture: 350 percent minimum (ASTM D412).

Specific gravity: Consistent within ± 0.05 and in the range of 0.95-1.45 (ASTM D297).

Compression set: 20 percent maximum. The compression set determination shall be made in accordance with ASTM D395, with the exception that the disc shall be a ½ thick section of the rubber gasket.

Tensile strength after aging: After being subjected to an accelerated aging test for 96h in air at 158ºF (70ºC) in accordance with ASTM D57 3, reduction in tensile strength shall not exceed 20 percent of its original value.

Shore durometer: The shore-durometer hardness shall be in the range of 50-65 and shall be within ±5 points of the value specified by the joint manufacturer. Values shall be determined in accordance with ASTM D2440, with the exception of Sec. 4 thereof. The determination shall be taken directly on the gasket.

Testing and certification: Rubber gaskets shall be tested to ensure that the material is fully cured and homogeneous, and that the gasket cross section contains no voids or physical defects that will impair its ability to maintain compression strength and provide the necessary volume, as designed. The supplier shall provide test results showing that the material meets the requirements of Sec. (b) and gaskets have been tested in accordance with this section.

(7) Plain ends fitted with flanges. Ends to be fitted with flanges shall have the longitudinal or spiral welds on the pipe ground to plate or sheet surface for sufficient distance from the ends to accommodate the flange.

(8) Manufacturing tolerances at ends. Tolerances for pipe ends shall be in accordance with the following, as applicable. The length of pipe subject to the stated tolerance shall be that distance that comes in direct contact with the mating pipe or external appurtenances.

(a) Out-of-roundness. The out-of-roundness of pipe ends shall be consistent with the diameter and wall thickness of the pipe supplied and the type of joint specified. Any out-of-roundness will be limited to a smooth oval that may be jacked back to a circular shape.

(b) Diameter. For purposes of this section, the diameter of the pipe ends shall be as determined by accurate circumferential measurement with a steel tape.
(c) The circumference of the following types of pipe ends shall not vary by more than 0.196" (1/16" on diameter) under, or 0.393") (1/8" on diameter) over, the specified outside diameter:
    plain-end pipe;
    beveled ends for field butt welding;
    plain ends fitted with flanges; and
    ends fitted with butt straps for field welding
(d) Ends for mechanical couplings shall have tolerances within the limits required by the manufacturer of the type of coupling to be used.
(e) For lap-joint pipe prepared for field welding, the inside circumference of the bell end shall not exceed the outside circumference of the spigot end by more than 0.400".
(f)For bell-and-spigot ends with rubber gaskets, the clearance between the bells and spigots shall be such that when combined with the gasket groove configuration and the gasket itself, watertight joints will be provided under all operating conditions when properly installed. The pipe manufacturer shall submit details complete with significant dimensions and tolerances, and shall also submit performance data indicating the proposed joint has performed satisfactorily under similar conditions. In the absence of a history of field performance, the results of a test program shall be submitted.
9. Squareness of ends for welding. For pipe that is to be butt-welded in the field, the ends of pipe sections shall not vary by more than $^1/_8$" at any point from a true plain perpendicular to the axis of the pipe and passing through the center of the pipe at the end.

Section 7.2.2 Supplemental requirements to ANSI/AWWA C200, ASTM A139 Standards for the Manufacture of Spiral Weld Pipe for the Traditional Penstock

7.2.2.0 QUALIFICATION OF STEEL COIL

Steel coil shall conform to the physical and chemical properties of any of the Structural or Pressure Vessel Quality, ASTM Standards listed in Section 3.1 that are available in coils. ASTM A-6 "General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use" or ASTM A20 "General Requirements for Steel Plates for Pressure Vessels" shall apply as applicable to coils. The steel shall be killed steel, with fine austenitic grain size.

The mill producing the coil shall furnish certified chemical analysis of each heat. All required physical tests may be taken by the pipe manufacturer from the coil or from the completed pipe. One set of the required physical tests shall be taken for each heat or each 50 tons of each heat. A set of physical tests shall consist of 3 tests on a coil; one from the outside wrap of the coil, one from the middle third of the coil and one from the inner wrap adjacent to the portion of the coil that is used. The set from the middle portion of the coil may be taken from the finished pipe. The orientation of tests shall be-the longitudinal axis of the test specimens shall be transverse to the final rolling direction of the coil.

Care shall be taken in production and shipping of coils to obtain uniform width, acceptable edges and to avoid excessive telescoping of the coil. Lead ends and tail ends of coils shall be scrapped to the point that acceptable uniformity of coil width, thickness and flatness is obtained.

7.2.2.1 FORMING

The spiral mill shall be set up to the correct feed angle for the coil width used and the pipe O.D. required. The pipe shall be formed to the required O.D. by forming rolls in a continuous process. The initial start-up section of pipe shall be scrapped to the point that diameter control has been obtained and the automatic welding procedure is operating.

The tolerance on pipe barrel outside circumference shall be ± .35% of the nominal outside circumference based on the diameter specified, not to exceed ± $^1/_2$". Pipe ends shall be expanded if necessary, to meet field joint tolerances.

For pipe wall thicknesses of $^3/_8$" or less, the maximum radial offset (misalignment) of plate edges in the weld seam shall be .1875 times the pipe wall thickness or $^1/_8$", whichever is larger. For pipe wall thickness greater than $^3/_8$", the maximum radial offset shall be .1875 times the pipe wall thickness or $^1/_8$", whichever is smaller. The offset shall be measured with commercially available equipment such as a Cambridge type gauge or a V wack gauge and shall be measured from both sides of the weld and the average of the two values should be used as the offset value.

Repair on out-of-tolerance pipe may be made on localized areas with offset up to .3*t*, with $^3/_8$" maximum for a length of 8 inches, with fill metal added to provide a 4 to 1 transition.

7.2.2.2 WELDING

All welds shall be complete joint penetration double groove welded butt joints with at least one pass on the outside and at least one pass on the inside.

The spiral weld shall be an automatic submerged arc. The coil splice weld shall be automatic submerged arc on the inside of the pipe and manual shielded metal arc weld or semi-automatic flux cored arc weld on the outside. Weld repair where required may be manual shielded metal arc weld or semi-automatic flux cored arc weld.

All welds shall be made using Welding Procedure Specifications with Procedure Qualification Records and Welders and Welding Operators Qualified by the manufacturer in accordance with ASME BPV Code Section IX and/or AWA D1.1.

7.2.2.3 INSPECTION AND TESTING

(1) All welds in pipe wall of .203" or heavier shall be 100% ultrasonically inspected in accordance with API 5L Section 9.
(2) Production weld tests conforming to methods described in ASTM A-370 shall be made as follows:
    (a) The weld-test specimens shall be taken perpendicularly across the weld and from the end of the pipe. The test specimen shall have the weld approximately in the middle of the specimen. The specimens shall be straightened and tested at room temperature.
    (b) Two reduced-section tension specimens made in accordance with Figure 7.2.2.3.1 shall show a tensile strength not less than 100 percent of the minimum specified tensile strength of the base material used.
    (c) Two bend-test specimens shall be prepared in accordance with Figure 7.2.2.3.2 and shall withstand a 180º bend in a jig in accordance with Figures 7.2.2.3.3, 4, or 5 & Table 7.2.2.3.1. When making the guided-bend tests, one specimen shall be bent so that the face representing the inside of the pipe is on the inside of the test bend. The second bend test shall be made so that the face of the specimen representing the inside of the pipe is on the outside of the test bend. A guided-bend test specimen shall be considered as having passed if no crack or other open defect exceeding $^1/_8$" measured in any direction is present in the weld metal or between the weld and base material after the bending.
    (d) If any test specimen shows defective machining or develops flaws not associated with the welding, it may be discarded and another specimen substituted.
    (e) There shall be at least one set of weld-test specimens taken from the spiral weld and one set of weld-test specimens taken from the coil splice weld for each lot of 50 lengths or less of each size, wall thickness, or grade of steel.
    (f) Retests - if the tensile-elongation specimen, or one or both of the guided-bend specimens, fail to conform to the specified requirements, the manufacturer may elect to repeat the tests on specimens cut from two additional lengths of pipe from the same lot. If such specimens conform to the specified requirements, all lengths in the lot shall be accepted, except the length initially selected for test. If any of the retest specimens fail to pass the specified requirements, the manufacturer may elect to test specimens cut from the individual lengths remaining in the lot. The manufacturer may also elect to retest any length which has failed to pass the test by cropping back and cutting two additional specimens from the same end. If the requirements of the original test are met by both of these additional tests, that length shall be acceptable. No further cropping and retesting is permitted. Specimens for retests shall be taken in the same manner as specified above.
(3) Reduced-Section Tension Tests shall be made conforming to methods described in ASTM A-370 in accordance with the procedure.

7.2.2.4

The following sections of 7.2.1 shall also apply to section 7.2.2:

e. Magnetic particle & liquid penetrant inspection of welds
f. Hydrostatic test
g. Charpy Impact Tests (for coil follow procedure of 7.2.1).
h. Tolerances
i. Preparation of pipe ends

TABLE 7.2.2.3.1 - GUIDED-BEND TEST JIG DIMENSIONS*

	Specified Minimum Yield Strength psi			
	Up to 42,000	**42,000**	**45,000**	**50,000-55,000**
Radius of male member, R_A	2*t*†	3*t*	3 1/2*t*	4*t*
Radius of female member, R_B	3*t* + 1/16 in.	4*t* + 1/16 in	4 1/2*t* + 1/16 in	5*t* + 1/16 in.
Width of male member, *A*	4*t*	6*t*	7*t*	8*t*
Width of groove in female member, *B*	6*t* + 1/8 in.	8*t* + 1/8 in.	9*t* + 1/8 in.	10*t* + 1/8 in.

*For intermediate grades of pipe, the above dimensions of the bending jig shall conform to those shown for the next lower grade or shall be proportional thereto, † t = specified all thickness of the pipe.

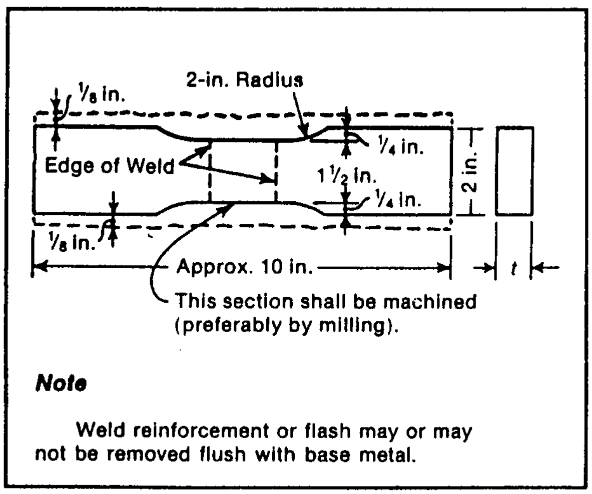


Figure 7.2.2.3.1 –
Reduced-section tension test specimen

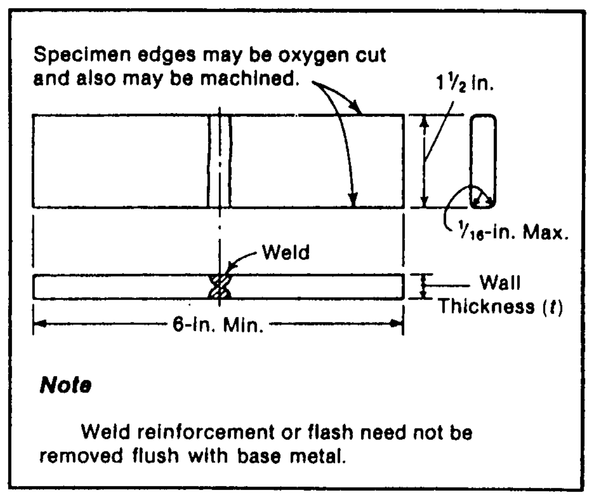
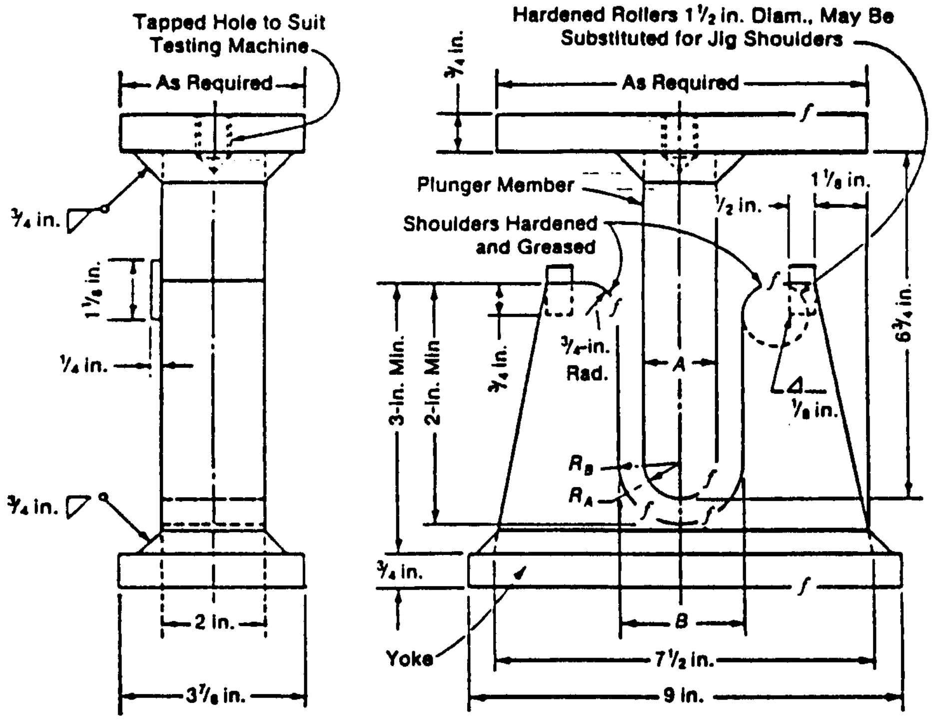


Figure 7.2.2.3.2 –
Guided-bend test specimen

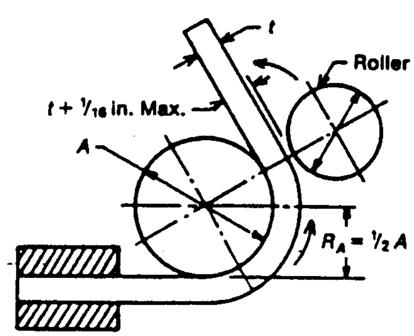


Art reprinted by permission from ASTM A370, Methods and Definitions for Mechanical Testing of Steel Products.

Notes

1. See Table 2 for jig dimensions.
2. The symbol *f* indicates a light finish cut; *t* is the specified wall thickness of pipe.
3. See Figures 4 and 5 for alternative guided-bend test fixtures.

Figure 7.2.2.3 - Jig for guided-bend test



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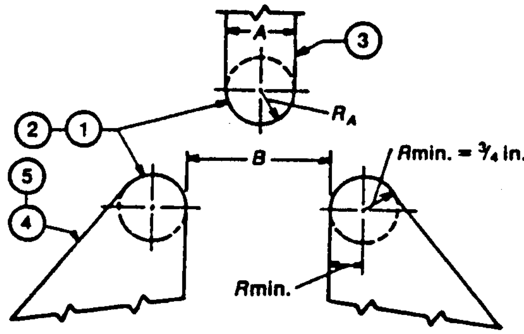
Notes

1. Dimensions not shown are the option of the designer. The essential consideration is to have adequate rigidity so that the jig parts will not spring.
2. The specimen shall be firmly clamped on one end so that there can be no sliding of the specimen during the bending operation.
3. Test specimens shall be removed from the jig when the outer roll has been removed 180° from the starting point.

Figure 7.2.2.3.4 - Alternative guide-bend roller jig



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Notes

1. Either hardened and greased shoulders or hardened rollers free to rotate shall be used.
2. The shoulders or rollers shall have *a* minimum bearing surface of 2 In. for placement of the specimen. The rollers shall be high enough above the bottom of the jig so that the Specimens will clear the rollers when the ram is in the low position.
3. The ram shall be fitted with an appropriate base and provision made for attachment to the testing machine, and shall be designed to minimize deflection and misalignment. The ram to be used with the roller jig shall be of identical dimensions to the ram shown In Figure 3.
4. If desired, either the rollers or the roller supports may be made adjustable in the horizontal direction so that specimens of *t* thickness may be tested on the same jig.
5. The roller supports shall be fitted with an appropriate base designed to safeguard against deflection or misalignment and equipped with means for maintaining the rollers centered, midpoint. and aligned with respect to the ram.
6. The weld and heat-effected zone in the case of a transverse-weld band specimen shall be completely within the band portion of the specimen after tasting.

Figure 7.2.2.3.5 - Alternative guided-bend wraparound jig

CHAPTER 8

Principles of Corrosion and Corrosion Control

Corrosion is the deterioration of a substance (usually a metal) or its properties because of a reaction with its environment. Even though the process of corrosion is complex and the detailed explanations even more so, relatively nontechnical publications on the subject are available.

An understanding of the basic principles of corrosion leads to an understanding of the means and methods of corrosion control. Methods of corrosion control are discussed in this chapter.

Section 8.1 General Theory

All materials exposed to the elements eventually change to the state that is most stable under prevailing conditions. Most structural metals, having been converted from an ore, tend to revert to it. This reversion is an electrochemical process—that is, both a chemical reaction and the flow of a direct electric current occur. Such a combination is termed an electrochemical cell. Electrochemical cells fall into three general classes:

- galvanic cells, with electrodes of dissimilar metals in a homogenous electrolyte,
- concentration cells, with electrodes of similar material, but with a nonhomogeneous electrolyte,
- electrolytic cells, which are similar to galvanic cells, but which have, in addition, a conductor plus an outside source of electrical energy.

Three general types of corrosion are recognized: galvanic, electrolytic, and biochemical.

GALVANIC CORROSION

Galvanic corrosion occurs when two electrodes of dissimilar materials are electrically connected and exposed in an electrolyte. An example is the common flashlight cell (Figure 8.1.1). When the cell is connected in a circuit, current flows from the zinc case (the anode) into the electrolyte, carrying ionized atoms of zinc with it. As soon as the zinc ions are dissolved in the electrolyte, they lose their ionic charge, passing it on by ionizing atoms of hydrogen. The ionic charge (the electric current) flows through the electrolyte to the carbon rod (the cathode). There, the hydrogen ions are reduced to atoms of hydrogen, which combine to form hydrogen gas. The current flow through the circuit, therefore, is from the zinc anode to the electrolyte, to the carbon rod cathode, and back to the zinc anode through the electrical conductor connecting the anode to the cathode. As the current flows, the zinc is destroyed but the carbon is unharmed. In other words, the anode is destroyed but the cathode is protected.

If the hydrogen gas formed in the galvanic cell collects on the cathode, it will insulate the cathode from the electrolyte and stop the flow of current. As long as the hydrogen film is maintained, corrosion will be prevented. Removal or destruction of the hydrogen film will allow corrosion to start again at the original rate. Formation of the film is called polarization; its removal, depolarization. Corrosion cells normally formed in highly corrosive soils or waters are such that the hydrogen formed on the cathode escapes as a gas and combines with dissolved oxygen in the electrolyte, thus depolarizing the cathode and allowing corrosion to proceed.

In the flashlight battery, the zinc case is attacked and the carbon is not. However, zinc or any other metal may be attacked when in circuit with one metal, but not attacked when in circuit with another. A metal listed in Table 8.1.1 will be attacked if connected in a circuit with one listed beneath it in the table, if they are placed in a common electrolytic environment such as water or moist soil.

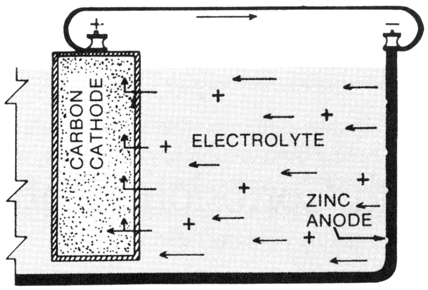


Figure 8.1.1 - Galvanic cell-dissimilar metals

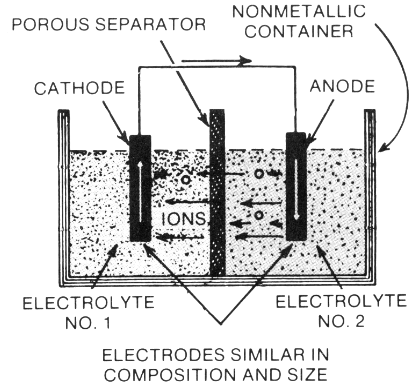


Figure 8.1.2 - Galvanic cell-dissimilar electrolytes

TABLE 8.1.1 - GALVANIC SERIES OF METALS AND ALLOYS*

Magnesium and magnesium alloys	
Zinc	
Aluminum 2S	
Cadmium	
Aluminum 17ST †	
Steel or iron	
Cast iron	
Chromium-iron (active)	
Ni-Resist †	
18-6 Stainless steel (active) †	
16-6-3 Stainless steel (active) †	
Hastelloy C †	Anodic or Corroded End ↑
Lead-tin solders	
Lead	
Tin	
Nickel (active)	
Inconel (active) †	
Hastelloy A †	
Hastelloy B †	
Brass	
Copper	
Bronzes	
Copper-nickel alloy	Cathodic or Protected End ↓
Monel †	
Silver solder	
Nickel (passive)	
Inconel (passive)	
Chromium-iron (passive)	
16-8 stainless steel (passive)	
18-8-3 Stainless steel (passive)	
Silver	
Graphite	
Gold	
Platinum	

*A "passive" metal has a surface film of absorbed oxygen or hydrogen. A metal may be initially "active" and become "passive" to the other metal when the protective film is formed.

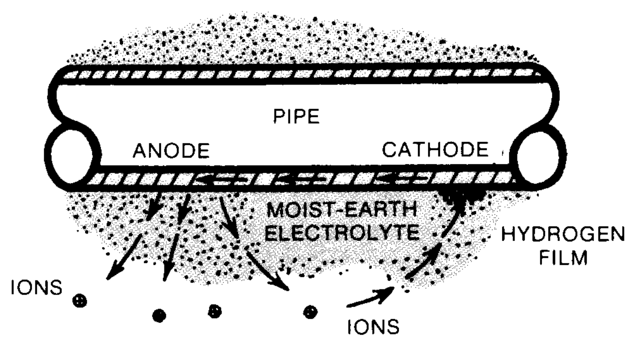
†Composition of Items IS as follows: Aluminum 17ST-95% Al, 4% Cu. 0 5% Mn, 0 5% Mg, N!-Resist, International Nickel Co, New York, NY – austenitic nickel and cast iron; 18-8 stainless steel-18% Cr, 8% NI, 18-E-3 stainless steel-18% Cr, 8% NI, 3% MO, Hastelloy C, Union Carbide Carbon Co., Niagara Falls, N.Y -59% NI, 17% MO, 14% Cr. 5% Fe, 5% W, Inconel International Nickel Co, New York, N Y -59.80% NI, 10.20% 0, O-23% Fe, Hastelloy A-60% NI, 20% MO, 20% Fe, Hastelloy B-65% NI. 30% MO, 5% Fe, Mom?-63.67% Ni. 29-30% Cu, l-2% Fe, 0 4-11% Mn

Source Hertzberg. LB Suggested Non-technical Manual on Corrosion for Water Works Operators *Jour. AWWA.* 48:7:9 (June 1956).

The order in Table 8.1.1 is known as the galvanic series; it generally holds true for neutral electrolytes. Changes in the composition or temperature of the electrolyte, however, may cause certain metals listed to shift positions or actually reverse positions in the table. For example, zinc is listed above iron in the table, and zinc will corrode when connected to iron in fresh water at normal temperature. But when the temperature of the water is above about 150º F (66º C), the iron will corrode and protect the zinc. Thus, the table cannot be used to predict the performance of all metal combinations under all conditions.

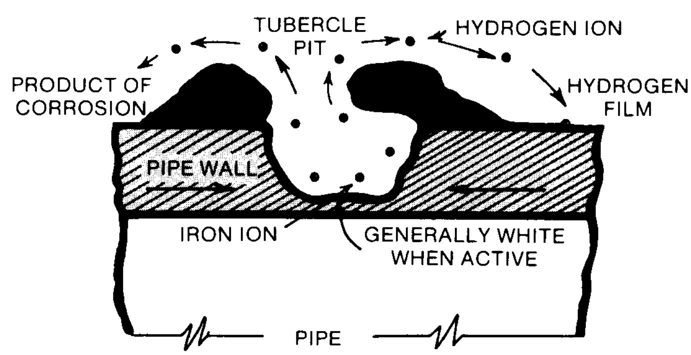
In the flashlight battery, dissimilar metals and a single electrolyte cause the electric current to flow. Similar metals in dissimilar electrolytes can also produce a current, as illustrated in Figure 8.1.2. In corrosion underground, differential oxygen concentration in soils is one of the chief reasons for dissimilarity in the electrolyte. Differential oxygen concentration (or differential aeration) may be caused by unequal compactness of backfill, unequal porosity of different soils or of one soil at different points, uneven distribution of moisture, or restriction of air and moisture movement in the soil caused by the presence of buildings, roadways, pavements, and vegetation.

The electrochemical cells described in the preceding paragraphs demonstrate the fundamental principles of the many kinds of electrochemical cells found in practice. The common forms of corrosion encountered on unprotected buried pipelines are shown in Figures 8.1.3 through 8.1.1.1.



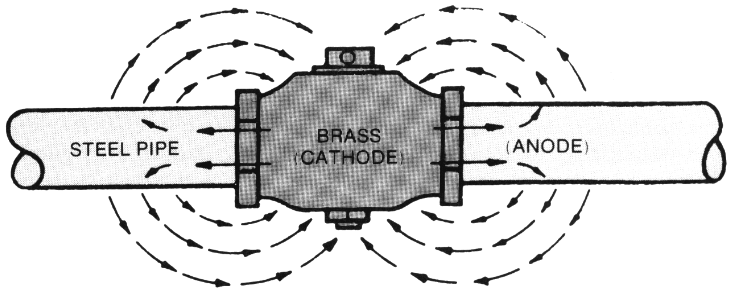
Moist earth is electrolyte; two areas on the pipe are anode and cathode; pipe wall takes place of wire in Figures 7-1 and 7-2. Pipe wall at anode will corrode like zinc battery case; pipe wall at cathode will not corrode but will tend to be coated with hydrogen gas, which if not removed, will tend to build resistance to current flow and thereby check corrosion of pipe wall at anode.

Figure 8.1.3 - Galvanic cell on embedded pipe without protective coating

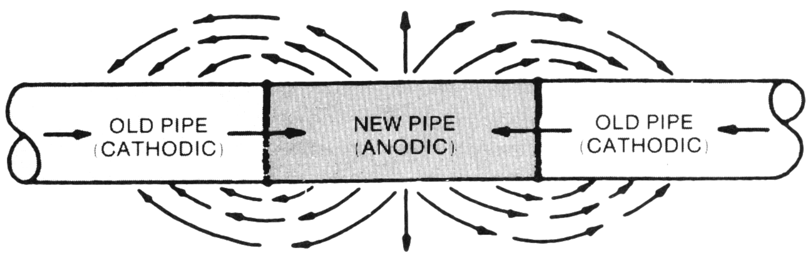


Detail of pope wall at anode in Figure 7-3 is shown. As current leaves surface of anode, It carves with It small particles of metal (ions). These ions go into solution in soil (electrolyte) and are immediately exchanged for hydrogen ions, leaving metal behind as rusty scale or tubercle around pit area. In many soils especially comparatively dry ones, this barnacle-like scab will “seal off” pit so that ion (electric current) cannot get through and cell becomes Inactive as long as tubercle is not disturbed.

Figure 8.1.4 - Galvanic cell - pitting action

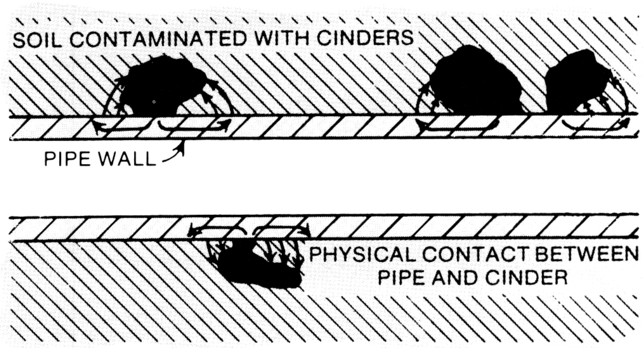


Brass valve is cathode (protected area), steel pipe is anode (corroding area), and surrounding earth is electrolyte. As long as cathode is small in area relative to anode, corrosion is not ordinarily severe or rapid. If these area proportions are reversed, corrosion may be much more rapid.



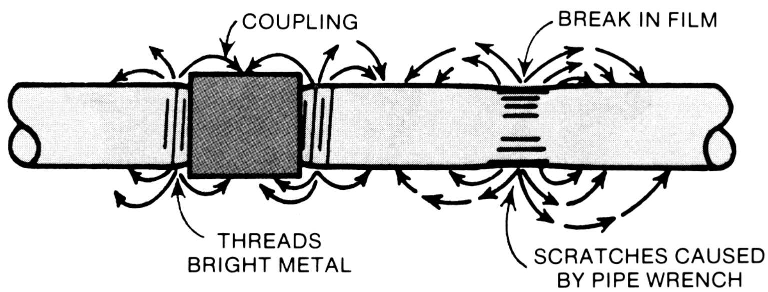
Although seldom considered, galvanic cell is created by installing piece of new pipe in old line. New pipe always becomes anode and its rate of corrosion will depend on type of soil and relative areas of anode and cathode. Therefore, careful protective measures are essential.

Figure 8.1.6 - Corrosion due to dissimilar metals



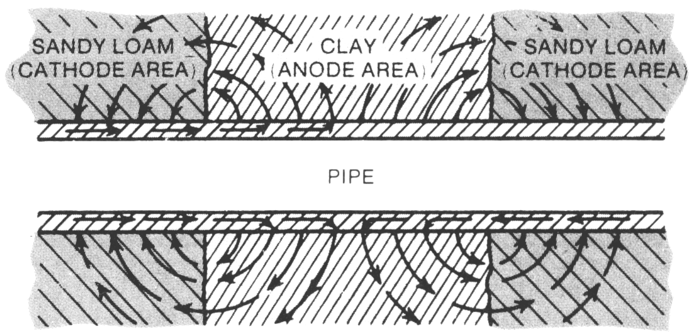
When metal pipe is laid in cinders. Corrosive action is that of dissimilar metals. Cinder is one metal (cathode) and pipe the other (anode). Acid leached from cinders contaminates soil and increases its activity. No hydrogen collects on cinder cathode, cell remains active, and corrosion is rapid.

Figure 8.1.7 - Corrosion due to cinders



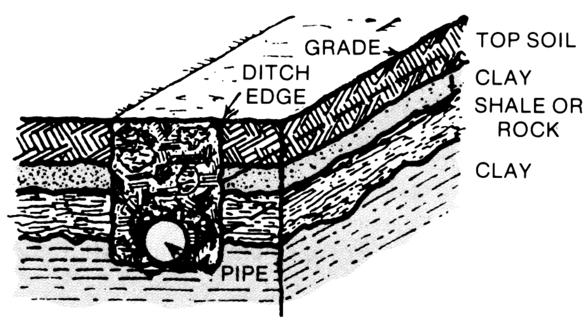
Bright scars or scratches of threads become anode areas in buried pipe, and rest of the pipe is cathode area. In some soils, these bright areas can be very active and destructive because the small anode area and large cathode area produce the most unfavorable ratios possible.

Figure 8.1.8 - Corrosion caused by dissimilarity of surface conditions



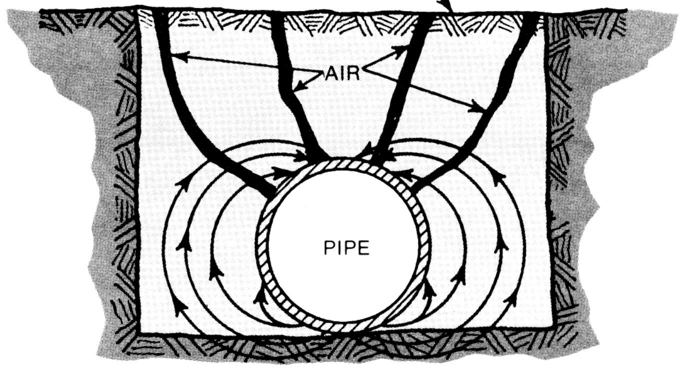
In this galvanic cell of dissimilar electrolytes (compare Figure 7-3, sections of pipe in sandy loam are cathodes (protected areas), sections in clay are anodes (corroding areas) and soil is electrolyte. If resistance to electric current flow is high in electrolyte, corrosion rate will be slow. If resistance to current flow is low, corrosion rate will be high. Thus, knowledge of soil resistance to electric-current flow becomes important in corrosion control studies.

Figure 8.1.9 - Corrosion caused by dissimilar soils



Dissimilarity of electrolytes, due to mixture of soils, causes formation of galvanic cell. If large clods of dirt, originally from different depths in ditch, rest directly against unprotected pipe wall, contact area tends to become anode (corroding area), and adjacent pipe cathode. Small, well-dispersed clods, such as result in trenching by machine, reduce cell-forming tendency. Galvanic cells having anode and cathode area distributed around circumference of pipe are often called short-path cells.

Figure 8.1.10 - Corrosion caused by mixture of different soils



This is another galvanic cell of dissimilar-electrolyte type. Soil throughout depth of ditch is of uniform kind, but pipe rests on heavy, moist, undisturbed ground at bottom of ditch while remainder of circumference is in contact with drier and more aerated soil backfill. Greatest dissimilarity - and most dangerous condition - occurs along narrow strip at bottom of pipe, which is anode of cell.

Figure 8.1.11 - Corrosion caused by differential aeration of soil

ELECTROLYTIC CORROSION

The transportation industry and other industries use direct current (DC) electricity for various purposes in their operations. It is common practice with DC circuits to use the ground as a return path for the current. In such cases, the path of the current may stray some distance from a straight line between two points in a system in order to follow the path of least resistance. Even where metallic circuits are provided for handling the direct currents, some of the return current may stray from the intended path and return to the generator either through parallel circuits in the ground or through some metallic structure. Because these currents stray from the desired path, they are commonly referred to as stray earth currents or stray currents.

The diagrammatic sketch of an electric street-railway system shown in Figure 8.1.12 is an example of a system that can create stray DC currents. Many modern subway systems operate on the same principle. In Figure 8.1.12, the direct current flows from the generator into the trolley wire, along this wire to the streetcar, and through the trolley of the car to the motors driving it. To complete the circuit, the return path of the current is intended to be from the motors to the wheels of the car, then through the rails to the generator at the substation. But because of the many mechanical joints along these tracks, all of which offer resistance to the flow of the electricity, what usually happens is that a portion of the current, seeking an easier path to the substation, leaves the rails, passes into the ground, and returns to the substation through the moist earth. If, in its journey through the ground, the current passes near buried metal pipe-which offers an easier path for return than does the ground around it-the current will flow along the metal walls of the pipe to some point near the substation; there it will leave the pipe to flow through the ground back to the rail, and finally return to the substation generator.

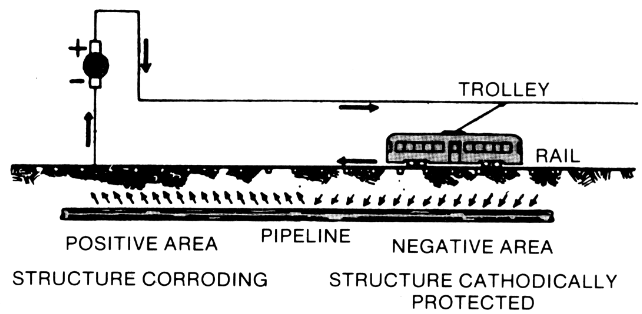
Areas of the pipe where the current is entering are not corroded. Where the current is leaving the pipe, however, steel is destroyed at the rate of about 20 lb per ampere-year of current discharged. To combat electrolysis, an insulated metal conductor must be attached to the pipe where it will remove and return the current to the source, rather than allowing the current to escape from the pipe wall. Figure 8.1.13 diagrammatically shows this method.

BIOCHEMICAL CORROSION

Certain soil bacteria create chemicals that may result in corrosion. Bacterial corrosion, or anaerobic-bacterial corrosion, is not so much a distinct type of corrosion as it is another cause of electrochemical corrosion. The bacteria cause changes in the physical and chemical properties of the soil to produce active pseudogalvanic cells. The bacterial action may be one of removing the protective hydrogen film. Differential aeration plays a major role in this activity.

The only certain way of determining the presence of anaerobic bacteria, the particular kind of microorganism responsible for this type of corrosion, is to secure a sample of the soil in the immediate vicinity of the pipe and develop a bacterial culture from that sample. Inspection under a microscope will determine definitely whether harmful bacteria are present.

Figure 8.1.12 - Stray-current corrosion caused by electrified railway systems



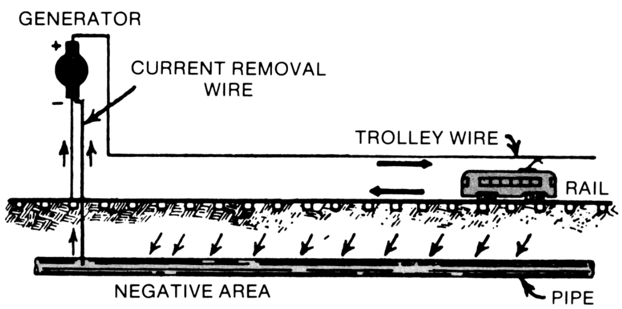


Figure 8.1.13 - Control of stray-current corrosion

STRESS AND FATIGUE CORROSION

Stress corrosion is caused from tensile stresses that slowly build up in a corrosive atmosphere. With a static loading, tensile stresses are developed at the metal surfaces. At highly stressed points, accelerated corrosion occurs, causing increased tensile stress and failure when the metal's safe yield is exceeded.

Corrosion fatigue occurs from cyclic loading. In a corrosive atmosphere, alternate loadings cause corrosion fatigue substantially below the metal's failure in noncorrosive conditions.

CREVICE CORROSION

Crevice corrosion in a steel pipeline is caused by a concentration cell formed where the dissolved oxygen of the water varies from one segment of the pipe metal to another. In a crevice area, the dissolved oxygen is hindered from diffusion, creating an anodic condition that causes metal to go into solution.

SEVERITY OF CORROSION

Severity of corrosion in any given case will depend on many different factors, some of which may be more important than others. The factors most likely to affect the rate of corrosion are

- relative positions of metals in the galvanic series,
- size of anode area with respect to cathode area,
- location of anode area with respect to cathode,
- resistance of metallic circuit,
- type and composition of electrolyte,
- conductivity or resistivity of electrolyte,
- uniformity of electrolyte,
- depolarizing conditions.

SOIL-CORROSION INVESTIGATIONS

The first organized soil-corrosion investigation was begun by the National Bureau of Standards (NBS) in 1911. The purpose at that time was to study the effect of stray current from street-railway lines on buried metallic structures. In its initial investigation, the bureau found that in many instances where rather severe corrosion was anticipated, little damage was observed, whereas in others, more corrosion was found than seemed to be indicated by the electrical data associated with the corroded structure. These observations led to a second investigation, undertaken in 1921. Originally about 14,000 specimens were buried at 47 test sites, but the number was subsequently increased to 36,500 specimens at 128 test sites. The American Petroleum Institute and the American Gas Association collaborated in analyzing the results of the latter tests.

Burial sites were selected in typical soils representing a sampling of areas in which pipe was or might be buried. The purpose of the investigation was to determine whether corrosion would occur in pipelines in the absence of stray currents under conditions representative of those encountered by working pipelines.

The NBS soil corrosion tests are probably the most extensive, well coordinated, and best analyzed of any test made for the same purpose. A final report on the studies made between 1910 and 1955, including over 400 references, has been published. An important finding was that in most soils, the corrosion rate decreased with time. This is largely due to the fact that corrosion products, unless removed, tend to protect the metal.

Figure 8.1.14, taken from the NBS reports, clearly shows the decrease in corrosion rate with time in all but the worst soil group. Only a very small percentage of pipe is ever buried in soil belonging to that group. Modern methods of corrosion prevention generally make it unnecessary to allow extra wall thickness as a safeguard against corrosion. Tables 8.1.2 and 8.1.3 give summary data on the corrosivity of soils and the relationship of soil corrosion to soil resistivity.

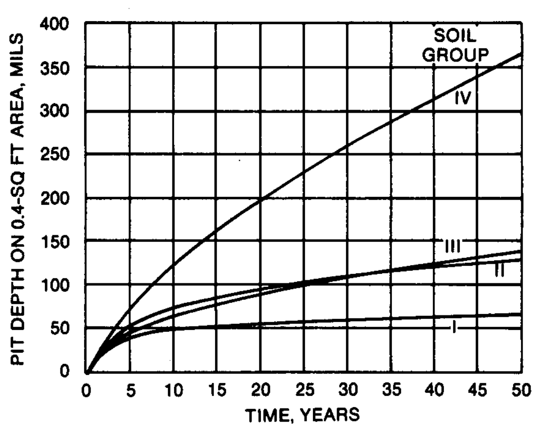


Figure 8.1.14 - Corrosion rate in various soils

Source: Bernard, R.E. A Method of Determining Wall Thickness of Steel Pipe for Underground Service. Jour. AWVA, 29:6:791 (June 1937)

Soil groups are defined in Table 8.13.

TABLE 8.1.2 - SOILS GROUPED IN ORDER OF CORROSIVE ACTION ON STEEL

Group I - Lightly Corrosive

Aeration and drainage good. Characterized by uniform color and no mottling anywhere in soil profile and by very low water table. Includes:

1. Sands or sandy loams
2. Light, textured silt loams
3. Porous loams or clay loams thoroughly oxidized to great depths.

Group II - Moderately Corrosive

Aeration and drainage fair. Characterized by slight mottling (yellowish brown and yellowish gray) in lower part of profile (depth 16-24 in.1 and by low water table. Soils would be considered well drained in an agricultural sense, as no artificial drainage is necessary for crop raising. Includes:

1. Sandy loams
2. Silt loams
3. Clay loams

Group III - Badly Corrosive

Aeration and drainage poor. Characterized by heavy texture and moderate mottling close to surface (depth 6-8 in.) and with water table 2-3 ft. below surface. Soils usually occupy flat areas and would require artificial drainage for crop raising. Includes:

1. Muck
2. Peat
3. Tidal marsh
4. Clays and organic soils

Group IV - Badly Corrosive

Aeration and drainage very poor. Characterized by bluish-gray mottling at depths of 6-8 in. with water table at surface, or by extreme impermeability because of colloidal material contained. Includes:

1. Muck
2. Peat
3. Tidal marsh
4. Clays and organic soils
5. Adobe clay

TABLE 8.1.3 - RELATIONSHIP OF SOIL CORROSION TO SOIL RESISTIVITY

Soil Class	Description	Resistance ohm/cc
1	excellent	10,000-6000
2	good	6000-4500
3	fair	4500-2000
4	bad	2000-0

Section 8.2 Internal Corrosion of Steel Pipe

Corrosion of the internal surfaces of a pipe is principally caused by galvanic cells. The extent of corrosion of the interior of an unlined pipe depends on the corrosivity of the water carried. Langelier has developed a method for determining the corrosive effect of different kinds of water on bare pipe interiors, and Wier has extensively investigated and reported the effect of water contact on various kinds of pipe linings. Although some unlined pipes have been pitted through by some waters, the principal result of interior corrosion is a reduction in flow capacity. This reduction is caused by a formation of tubercles of ferric hydroxide, a condition known as tuberculation. It is primarily to maintain flow capacity that pipe linings have been developed. Where internal corrosion is allowed to persist, quality of water deteriorates, pumping and transmission capacity decreases, efficiency diminishes, and costly replacement becomes inevitable. Serious accidents and loss of revenues from system shutdowns are also possible. The occurrence of these problems can be reduced by the use of quality protective linings.

Section 8.3 Methods of Corrosion Control

The electrochemical nature of corrosion suggests three basic methods of controlling it on underground and underwater pipelines. First, pipe and appurtenances can be isolated and electrically insulated from the surrounding soil and water by means of a protective coating. Second, electric currents can be imposed to counteract the currents associated with corrosion. Third, an inhibitive environment can be created to prevent or reduce corrosion.

To implement the frost method, satisfactory and effective protective coatings have been developed. Cathodic protection, implementing the second method, is being more and more widely used in corrosion control. Inhibitive coatings implement the third method by providing an environment in which oxidation or corrosion of steel is inhibited. By judicious use of all of these methods, any required degree of corrosion control can be economically achieved.

Coatings and linings are covered in Chapter 6. The remainder of this chapter deals with corrosion control by cathodic protection.

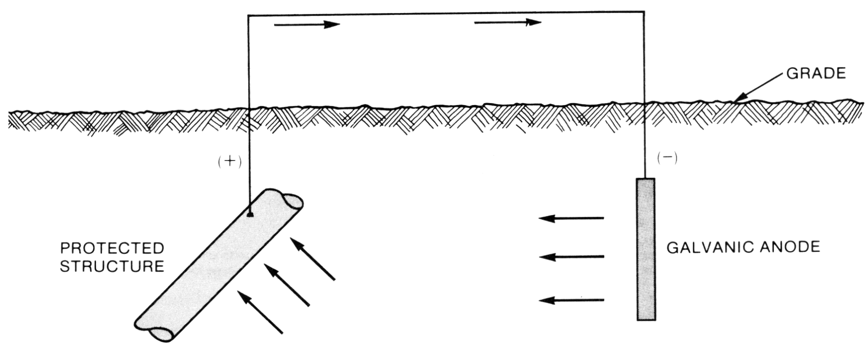


Figure 8.4.1 - Cathodic protection-galvanic anode type

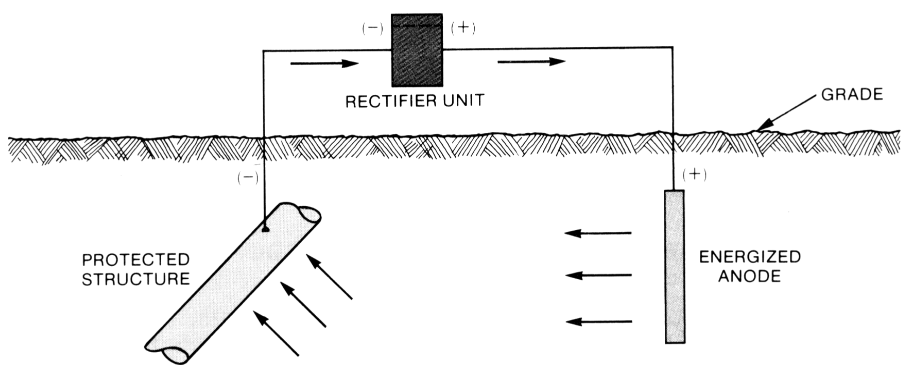


Figure 8.4.2 - Cathodic protection-rectifier type

Section 8.4 Cathodic Protection

Cathodic protection systems reverse the electrochemical corrosive force by creating an external circuit between the pipeline to be protected and an auxiliary anode (sacrificial metal) immersed in water or buried in the ground at a predetermined distance from the pipe. Direct current applied to the circuit is discharged from the anode surface and travels through the surrounding electrolyte to the pipe (cathode) surface.

Two methods are available for generating a current of sufficient magnitude to guarantee protection. In the first method, sacrificial-anode material such as magnesium or zinc is used to create a galvanic cell. The electrical potential generated by the cell causes current to flow from the anode to the pipe, returning to the anode through a simple connecting wire (Figure 8.4.1). This system is generally used where it is desirable to apply small amounts of current at a number of locations, most often on coated pipelines in lightly or moderately corrosive soils.

The second method of current generation is to energize the circuit with an external DC power supply, such as a rectifier. This technique, commonly referred to as the impressed current method, uses relatively inert anodes (usually graphite or silicon cast iron) connected to the positive terminal of a DC power supply, with the pipe connected to the negative terminal (Figure 8.4.2). This system is generally used where large amounts of currents are required at relatively few locations, and in many cases it is more economical than sacrificial anodes.

BONDING OF JOINTS

Where a pipeline is to be cathodically protected, or where a pipeline is to be installed with the possibility of future cathodic protection, the bonding of joints is required to make the line electrically continuous (Figures 8.4.3 and 8.4.4). It is usually desirable to bond all joints at the time of installation, because the cost later will be many times greater. In addition to bonding, the pipeline should have test leads connected to it at appropriate intervals to permit monitoring of the activity of electrical currents within the pipeline, whether under cathodic protection or not. Field-welded lines require no additional bonding.

CURRENT REQUIRED

For impressed-current cathodic protection to be effective, sufficient current must flow from the soil to the pipe to maintain a constant voltage difference at the soil-metal interface, amounting to 0.25 V or more (approximately 0.80-0.85 V between pipe and copper sulfate electrode in contact with soil). This minimum voltage requirement has been determined by experience, but it may be subject to variations at specific sites.

DESIGN OF CATHODIC PROTECTION SYSTEMS

In many situations, cathodic protection for steel pipelines will not be installed until proven necessary. However, all joints in steel pipe should be electrically bonded and electrical test stations provided along the pipeline as necessary.

CORROSION SURVEY

A corrosion survey, including chemical-physical analyses of the soil, must be performed along the pipeline right-of-way. Some of the measurements taken include soil resistivity, soil pH, and tests for stray currents.

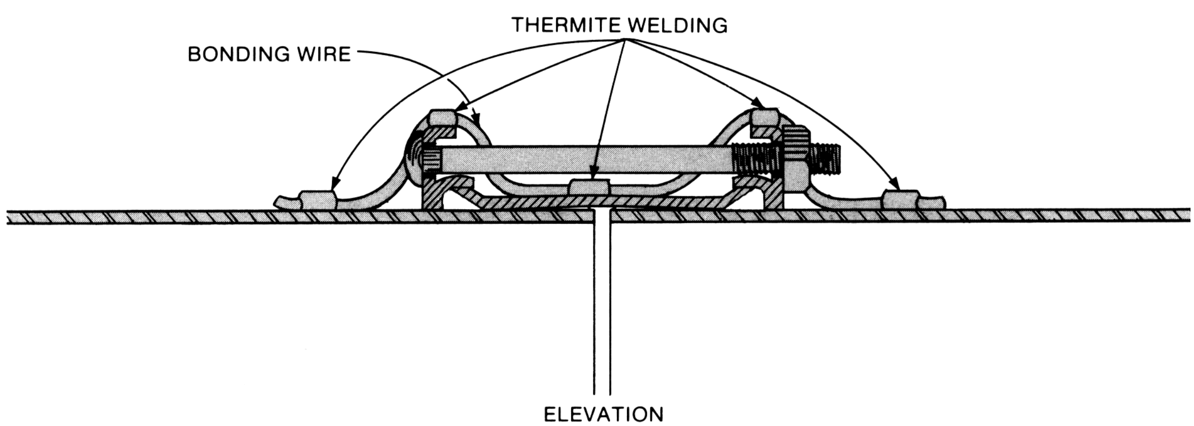


Figure 8.4.3 - Bonding jumpers installed on sleeve-type coupling

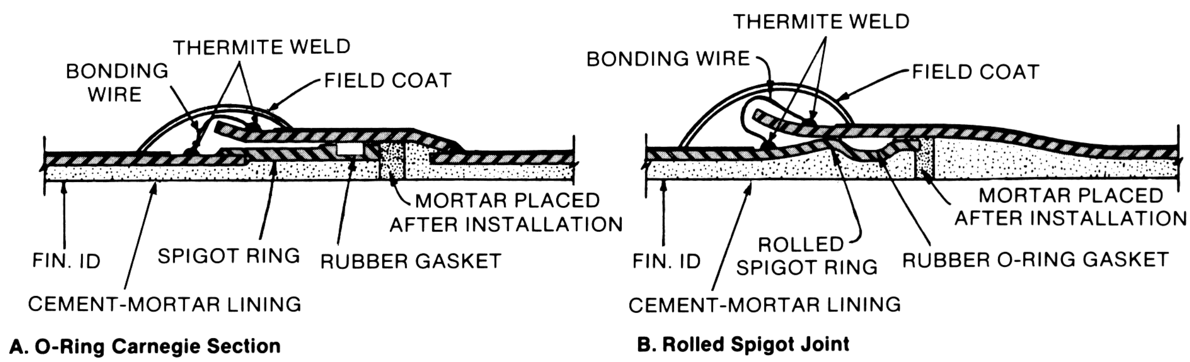


Figure 8.4.4 - Bonding wire for bell and spigot rubber-gasketed joint

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CHAPTER 9

Transportation, Installation, and Testing

Section 9.1 Transportation

Penstock pipe will usually be shipped directly to the jobsite on flatbed trucks. This one-time handling between shipper and customer avoids damage sometimes encountered by multiple handling and is therefore the first choice. In some cases, economics will require that the pipe be shipped by rail or barge and re-loaded onto trucks for final delivery.

In all cases, the flexible steel pipe shall be adequately stulled to maintain roundness during shipping and handling. Coated pipe shall be handled with nylon slings or padded forklifts that will not damage the coating. Coated pipe shall be shipped on padded bunks or contoured blocks to provide uniform bearing and shall be tied down with nylon belts or padded steel banding. In no case shall the pipe be handled with unpadded chains or cables or other equipment that might damage the coating. Rail cars shall be loaded in accordance with current association of America Railroads rules with pipe blocked to prevent lateral movement that will cause damage to the pipe ends or to the coating.

Section 9.2 Unloading & Storage

Upon arrival at the jobsite, the pipe shall be unloaded and strung along the right of way or placed in storage areas. Coated pipe shall be laid on padded wood blocks, earth berms, sand bags, or other suitable supports to protect the pipe coating.

Section 9.3 Installation

9.3.1 GENERAL

The successful installation of flexible steel pipe is dependent on proper trenching, bedding, and backfill. Buried penstocks will normally be laid with 4’ to 6’ of cover over the top of the pipe, with a minimum cover of 2’ where live loads are not anticipated. In some cases, it may be desirable to lay a penstock in a half trench and cover the penstock with a minimum of 2’ of earth and rock. This option will be most effective for large diameters and where excavation is difficult. See typical trench conditions, Figures 9.3.1, 9.3.2, and 9.3.3.

9.3.2 ALIGNMENT AND GRADE

The Engineer will usually furnish a plan and profile of the pipe line indicating critical vertical and horizontal points. From this plan and profile, the pipe manufacturer will produce a pipe laying schedule with stations and elevations at the pipe invert shown for each change of grade or alignment. The trench should be dug to this grade and alignment.

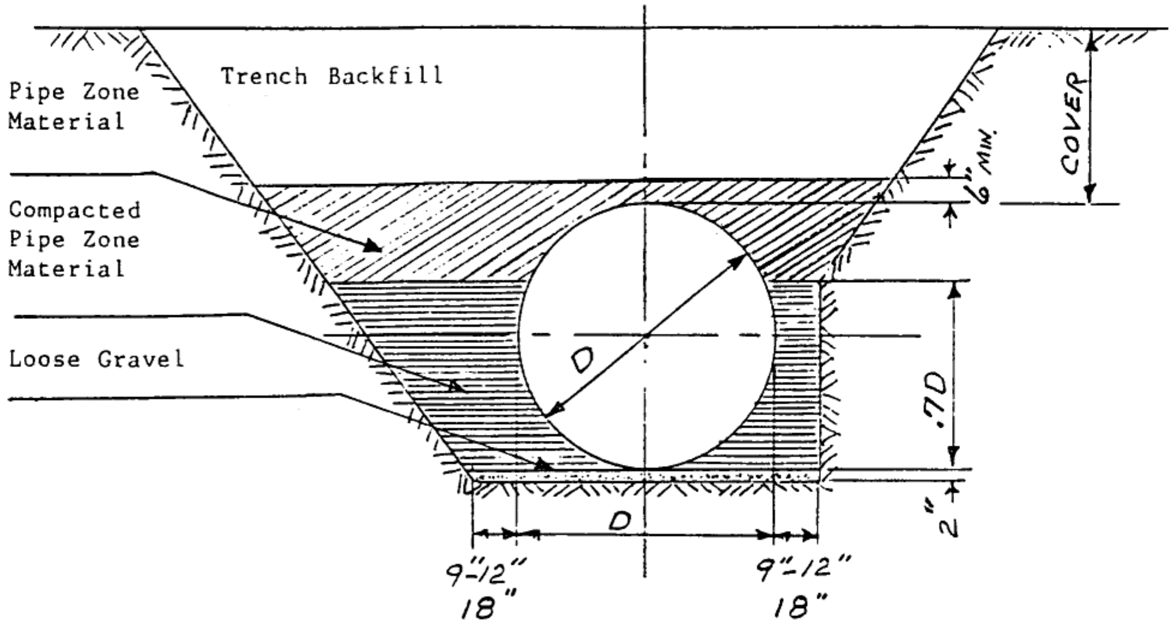

Figure 9.3.1 - Normal Trench

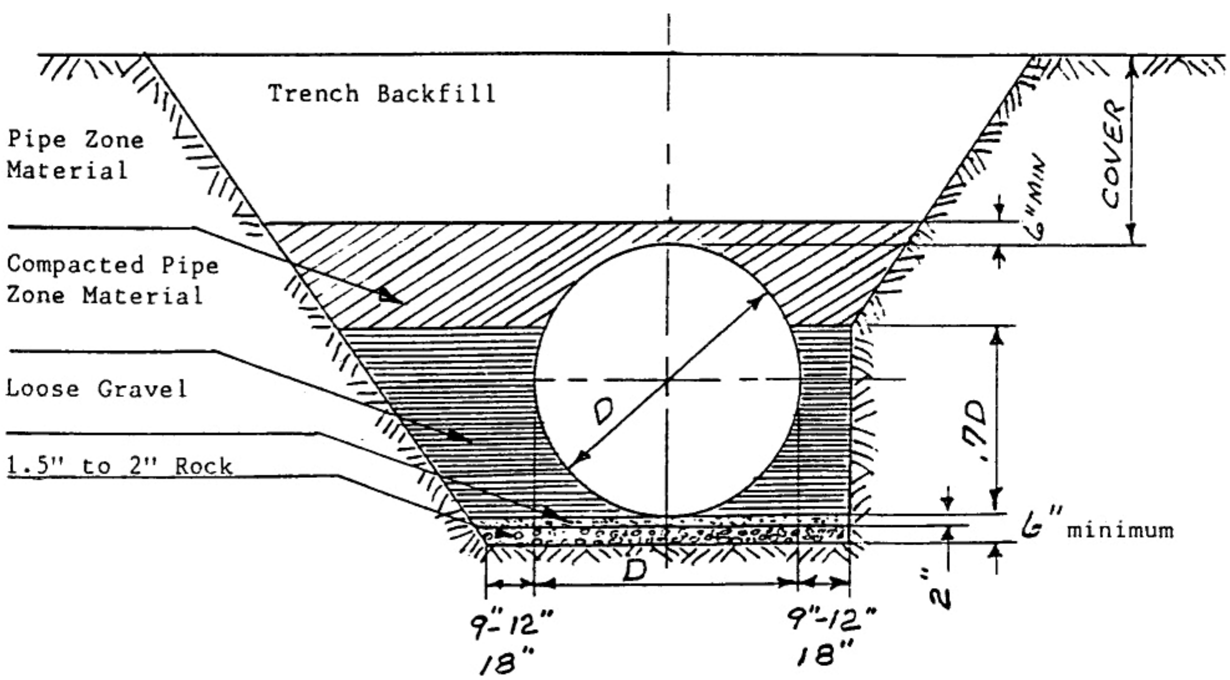

Figure 9.3.2 - Special Grade

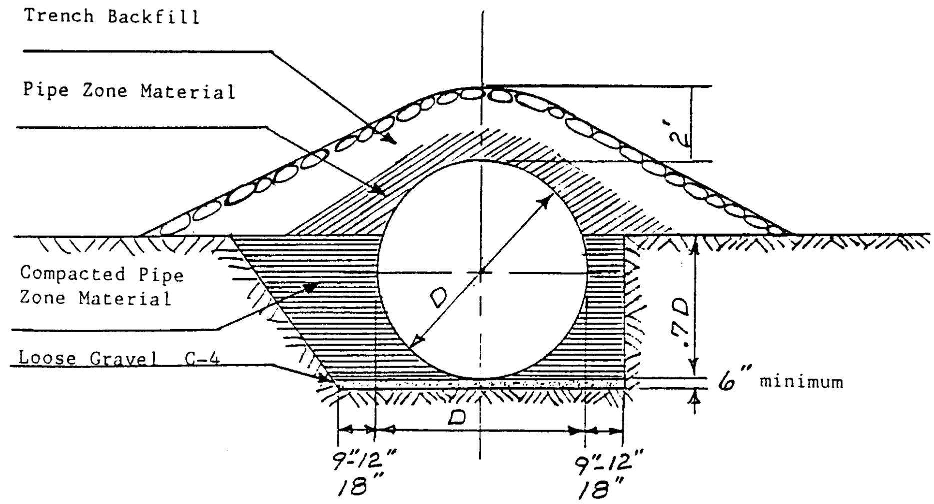

Figure 9.3.3 - Half trench

9.3.3 TRENCH EXCAVATION

The trench should be dug to the minimum width that will permit placement of the pipe and compaction of the pipe zone material, consistent with OSHA regulations for a safe trench. If a coarse sand material is used with compaction by jetting and vibrating, a minimum of 9" to 12" each side of the pipe is required. If materials requiring mechanical compaction are used, a minimum of about 18" each side of the pipe is required. Bell holes will also be required at the field joints. See Figures 9.3.1, 9.3.2, and 9.3.3 for typical trench conditions.

9.3.4 FOUNDATION AND BEDDING

For the normal trench, excavation should extend to a depth a minimum of 2 f_l below the established grade line of the outside bottom of the pipe. This excavation should then be filled with loose material such as pea gravel which can easily be graded to the established grade line to provide uniform support for the entire length of the pipe. Steel pipe should not be set on rigid blocks on the trench bottom that would cause concentration of the load on small areas of pipe coating or that would cause deformation of the pipe wall.

Where the trench bottom is unstable, or where it includes organic materials, or where the subgrade is composed of rock or other hard and unyielding materials, the trench bottom should be excavated to a depth of at least 6" below the bottom of the pipe or as required to establish a firm foundation for the pipe.

This over excavation should be filled with clean washed rock uniformly graded between 1½" and 2" and covered with a minimum of 2" of loose material same as for the normal trench. Where wet silty foundation conditions exist, the use of geotextile fabric can also be used to advantage.

9.3.5 ASSEMBLY OF PIPE

When the bedding has been prepared and graded, the penstock pipe shall be laid to the established grade and alignment. On slopes of 10% or greater, the pipe should be laid uphill. A close visual inspection of the exterior coating should be made prior to placing the pipe in the trench and any damage to the coating should be repaired. Bell holes should be prepared as the trench is dug and a space in the 2" layer of loose material should be provided for the removal of slings. Bell holes should be of sufficient depth and width to allow field coating of the assembled joint. If pipe is to be welded from the outside, they must be of sufficient depth, width and length to safely accommodate the welding operation. The pipe joints should be assembled in the trench following the pipe manufacturer's recommended procedure for the type of joint specified.

(a) *Bell-and-Spigot Rubber-Gasket Joints*

Under normal laying conditions, work should proceed with the bell end of the pipe facing the direction of laying. Before setting the spigot in place, the bell should be thoroughly cleaned and then lubricated in accordance with the pipe manufacturer's recommendations. After the O-ring rubber gasket has been placed in the spigot groove, it should be adjusted so the tension on the rubber is uniform around the circumference of the joint. Following assembly, the pipe joint should be checked with a thin metal feeler gauge to ensure that proper gasket placement exists in the spigot groove and that the proper amount of joint lap has been achieved. Sufficient pipe zone material should be placed near the middle of each section to hold it in position before the next section is assembled.

(b) *Field- Welded Joints*

Technical requirements for good field welding are contained in AWWA C206, Standard for Field Welding of Steel Water Pipe. If pipe that has been lined and coated is to be field welded, a short length of the pipe barrel at either end must be left bare so that the heat of the welding operation will not adversely affect the protective coating. The length of the unprotected section may vary depending on the kind of protective coating and pipe wall thickness. Care must be exercised when cutting and welding on pipes with combustible linings and coatings to avoid the risks of fire. The use of welded joints results in a rigid pipeline which is advantageous in restraining elbows in soils of low bearing capacity. Thermal stress must be considered when field joints are welded and the difference in temperature of the pipe at the time it is laid and the temperature in service is expected to be great enough to cause excessive stress.

Good practice is to keep the backfill up within 300' of the joint welding or to leave a joint unwelded at approximately 500' intervals and at all closures until the backfill has been brought up to the joint. In hot weather welding of the joint should be done during the coolest part of the day. When pipe over 24" diameter is welded from the inside, it is helpful to have pass holes at approximately 320' to 360' spacing to permit the entrance of welding cable without interference to the assembly of additional pipe sections. When welding or performing other work inside of the pipe, provisions for adequate ventilation must be considered.

Inspection of the field welded joints should be comparable to that specified for the pipe. That is, magnetic particle test of fillet welds, and ultrasonic or radiographic inspection.

9.3.6 FIELD COATING OF JOINTS

Before backfilling, the field joint should be coated with material compatible with the shop applied coating. The interior lining at the field joint may be completed at any time prior to the field hydrostatic test.

9.3.7 PIPE-ZONE BEDDING AND BACKFILL

Upon completion of the field joints, the remainder of the material in the pipe zone should be placed in uniform lifts on each side of the penstock and compacted so that the total pipe deflection of the vertical diameter does not exceed the limits given in Section 3.3.

The pipe zone material and compaction shall conform to one of the following alternatives:

(a) The bedding and pipe zone material shall be clean, well-graded, free-draining sand with no clay fines and shall conform to the following limits when tested by means of laboratory sieves:

Sieve Size	Total Passing by Sites (Percentage by Weight)
$^3/_8$ inch	100
No. 4	70 - 100
No. 8	36 - 93
No. 16	20 - 80
No. 30	8 - 65
No. 50	2 - 30
No. 100	1 - 10
No. 200	0 - 3

For this material, the backfill may be placed in lifts not exceeding 4' in thickness or to .7D before compaction. All bedding and pipe zone material to a height of. 7D shall be compacted by jetting and vibrating to obtain 70% relative density as determined by ASTM Designations D 4253 and D 4254. Material over .7D shall be compacted as required for surface conditions. Special precautions shall be taken to prevent flotation of the pipe.

(b) The bedding and pipe zone material shall be coarse-grained soils with fines (SC, SM) or better with ¼ maximum rock size. The material shall be placed in lifts not exceeding 12" in thickness before compaction to a height of .7D. Material to this height shall be compacted by mechanical tamping to a Std. AASHTO relative compaction T99 equal or greater than that used in the design for external load Section 3.3. Material over .7D shall be compacted as required for surface conditions.

9.3.8 TRENCH BACKFILL OVER PIPE ZONE

After confirmation that compaction of pipe zone material complies with the specified compaction, the remaining trench backfill can be placed. Compaction of trench backfill above the pipe zone material in locations other than those in roadways or locations specified by the owner will not be required except to the extent necessary to prevent future settlement or erosion.

Where compaction is required, mechanical compaction or jetting and vibrating shall be completed to a safe height over the top of the pipe before power-operated compaction equipment or construction loads are allowed over the pipe.

Section 9.4 Hydrostatic Field Test

The purpose of the hydrostatic field test is primarily to determine if the field joints are watertight. The hydrostatic test is usually conducted after backfilling is complete. It is performed at a fixed pressure above the design working pressure of the line. If thrust resistance is provided by concrete thrust blocks, a reasonable time for the curing of the blocking must be allowed before the test is made. If thrust restraint is provided by tied joints and pipe to soil friction, backfill and compaction must be completed before the test is made.

9.4.1 FIELD TESTING CEMENT-MORTAR-LINED PIPE

Cement-mortar-lined pipe to be tested should be filled with water of approved quality and allowed to stand for at least 24 hours to permit maximum absorption of water by the lining. Additional water should be added to replace water absorbed by the cement-mortar lining. (Pipe with other types of lining may be tested without this waiting period.) Pipe to be cement-mortar lined in place may be hydrostatically tested before or after the lining has been placed.

9.4.2 BULKHEADS

If the pipeline is to be tested in segments and valves are not provided *to* isolate the ends, the ends must be provided with bulkheads for testing. A conventional bulkhead usually consists of a section of pipe 2-3 ft long, on the end of which a flat plate or dished plate bulkhead has been welded containing the necessary outlets for accommodating incoming water and outgoing air.

9.4.3 AIR VENTING

The pipeline should be filled slowly to prevent possible water hammer, and care should be exercised to allow all of the air to escape during the filling operation. After filling the line, it may be necessary to use a pump to raise and maintain the desired pressure.

9.4.4 ALLOWABLE LEAKAGE

The hydrostatic test pressure is usually applied for a period of 24 hours before the test is assumed to begin, principally to allow for the lining material to absorb as much water as is possible. After that, the pipeline should be carefully inspected for evidence of leakage. The amount of leakage that should be permitted depends on the kind of joints used in the pipeline.

In making the test, the water pressure should be raised (based on the elevation at the lowest point in the section of the line under test) to a level such that the test section is subjected to not more than 125 percent of the normal operating pressure or to a pressure that will produce a hoop stress not to exceed 80 % of the minimum yield strength of the steel, whichever is less. The test pressure should be maintained for at least 2 hours. There should be no significant leakage in an all-welded pipeline or one that has been joined with properly installed mechanical couplings. On pipe joined with O-ring rubber gaskets, a small tolerance for leakage should be allowed. A leakage of 10 gal per in. of diameter per mile per 24 hours is usually permitted. Pinhole leaks that develop in welded joints should not be stopped by peening; instead, they should be marked for proper repair by welding. Such welding frequently can be accomplished without emptying the pipeline, providing pressure can be relieved.

If a section fails to pass the hydrostatic field test, it will be necessary to locate, uncover, and repair or replace any defective pipe, valve, joint, or fitting. The pipeline must then be retested.

References

STANDARDS AND SPECIFICATIONS

1. AWWA C200 Standard for Steel Water Pipe 6 In. and Larger
2. AWWA C203 Standard for Coal-Tar Protective Coatings and Linings for Steel Water Pipelines-Enamel and Tape-Hot-Applied
3. AWWA C205 Standard for Cement-Mortar Protective Lining and Coating for Steel Water Pipe-4 In. and Larger-Shop-Applied
4. AWWA C206 Standard for Field Welding of Steel Water Pipe
5. AWWA C207 Standard for Steel Pipe Flanges for Waterworks Service-Sites 4 In. Through 144 In.
6. AWWA C208 Standard for Dimensions for Steel Water Pipe Fittings
7. AWWA C2O9 Standard for Cold-Applied Tape Coatings for Special Sections, Connections and Fittings for Steel Water Pipelines
8. AWWA C210 Standard for Coal-Tar Epoxy Coating System for the Interior and Exterior of Steel Water Pipe
9. AWWA C214 Standard for Tape Coating Systems for the Exterior of Steel Water Pipeline
10. AWWA C219 Standard for Bolted Sleeve Type Couplings for Plain End Pipe
11. AWWA C602 Standard for Cement-Mortar Lining of Water Pipelines in Place
12. AWWA Ml 1 Steel Pipe-A Guide for Design and Installation
13. ASTM A6 Standard Specification for General Requirements for Rolled Steel Plate, Shapes, Sheet Piling and Bars for Structural Use
14. ASTM A20 Standard Specification for General Requirements for Steel Plate for Pressure Vessels
15. ASTM A36 Standard Specification for Structural Steel
16. ASTM A53 Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
17. ASTM Al39 Standard Specification for Electric-Fusion (Arc)-Welded Steel Pipe (Sizes 4 In. and Over)
18. ASTM A516 Standard Specification, Pressure Vessel Plate-Heat-Treated, Carbon-Manganese-Silicon Steel
19. ASTM A537 Standard Specification, High Strength, Low-Alloy, Columbium-Vanadium Steels of Structural Quality
20. ASTM A570 Standard Specification for Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality
21. ASTM A572 Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
22. ASTM A907 Standard Specification for Steel, Sheet and Strip, Heavy Thickness Coils, Carbon, Hot-Rolled, Structural Quality.
23. ASTM A935 Standard Specification for Steel, Sheet and Strip, Heavy Thickness Coils, High Strength, Low-Alloy, Columbium or Vanadium, or Both, Hot-Rolled.
24. ASME Boiler and Pressure Vessel Code, Section VIII, Pressure Vessels

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