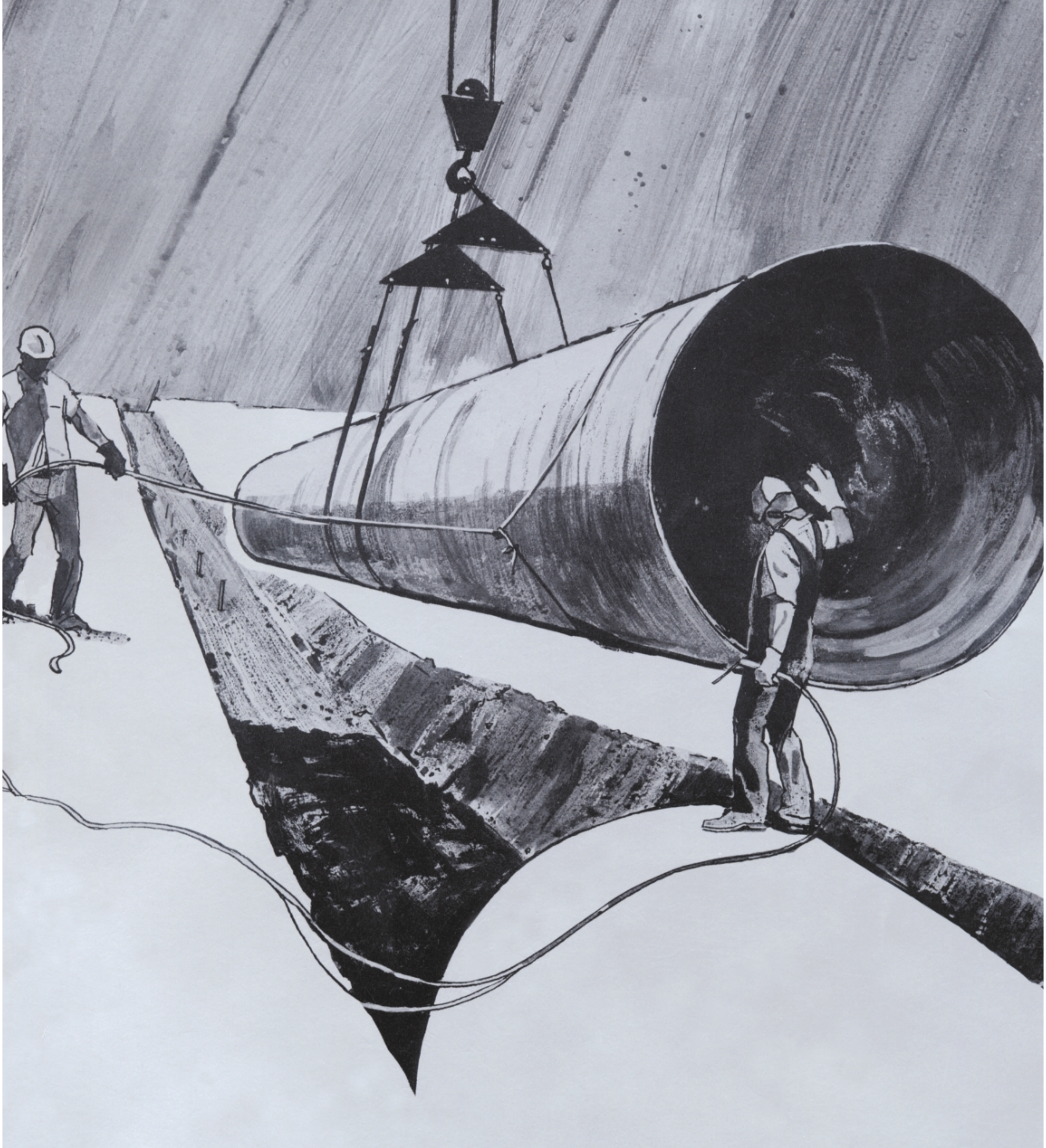
WELDED STEEL PIPE



Steel Plate Engineering Data • Volume 3

Acknowledgements

The Construction Marketing Committee wishes to acknowledge, with appreciation, the contributions made by the following members of the Steel Plate Fabricators Association Pipe Committee. Special thanks are given to Robert Card, Brico Industries, Dennis Dechant, Bruce VanderPloeg, Northwest Pipe Co. and George Tupac, Consultant, for their special assistance in producing this manual.

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Steel Plate Engineering Data - Volume 3

Welded Steel Pipe
Revised Edition – 1996

Merits, Design Standards, Technical Data and References

A compilation of useful information for the design of water transmission lines and distribution systems using Welded Steel Pipe.

Published by Construction Marketing Committee,
AMERICAN IRON AND STEEL INSTITUTE

In cooperation with and editorial collaboration by
STEEL PLATE FABRICATORS ASSOCIATION, INC.

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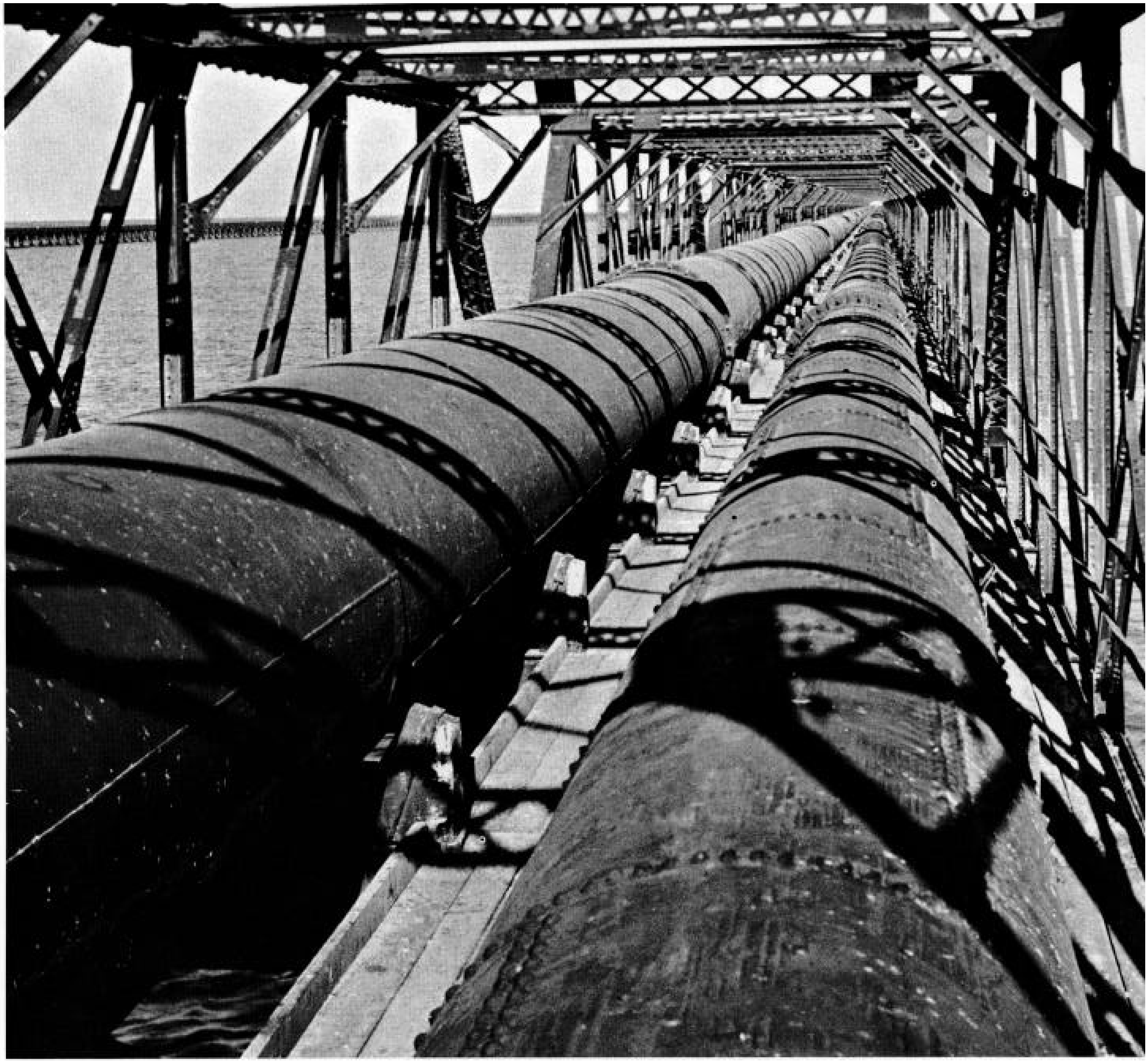


City of San Francisco, California -Bay crossing of the Hetch Hetchy Aqueduct. 66 inches diameter, 3/8-inch and l/2-inch steel plate.

"Wherever Water Flows
Steel Pipes
It Best"

Welded Steel Pipe

Noteworthy advancements have been made in the Twentieth Century in the fabrication of steel pipe. This is particularly true of pipe manufactured by the automatic welding processes. This pipe possesses many desirable qualities, including the seven chief requisites of any good conduit- durability, strength, economy, high carrying capacity, reliability, adaptability, and water-tightness.

Over the years, rigid specifications have been developed covering the chemical and tensile requirements of the steel from which the pipe is made. Great strides have been made in the fabrication, inspection, testing, joining, and coating of steel pipe. Welded steel pipe of high quality is available in the widest range of sizes, grades, wall thicknesses and lengths.

Included in this manual is the design criteria for steel pipe up to 180” in diameter, under conditions of internal pressure and external loads most commonly encountered. In addition to the text, useful technical charts and tables are included as well as a comprehensive bibliography. Applications of the principles and data shown should be based on responsible judgment and experience.

Research and Development

The leading steel producers, most of pipe fabricators and pipe protection material producers and designers in the United States are members of the American Iron and Steel Institute and the Steel Plate Fabricators Association. These firms maintain extensive facilities where metallurgical, welding and pipe lining and coating developments are researched. New product developments, as well as improvements in manufacturing techniques and processes, are continuously under study. In addition, representatives of these firms serve on committees engaged in the preparation of national standards, codes and specifications. Through these activities, members of the American Iron and Steel Institute and the Steel Plate Fabricators Association maintain leadership in modern manufacturing methods and product development, which assures the user that he is receiving the most modern, up-to-date product of the highest quality.

History of Steel Pipe

Search for Durability

Thousands of years ago men first learned the secret of conducting water through crude pipes. Long before the birth of Christ, the Chinese transported water through bamboo; a Babylonian king who reigned 4500 years ago had a bathroom with tile drain pipes; a municipal reservoir served Carthage about 800 B.C. and there is much evidence of the fine water supply systems of the Romans.

But as cities grew larger, and homes were built closer to each other, the problem of adequate water supply became acute and intensified efforts to construct more durable piping systems. This was especially so in the early days of this country when every means of enticing settlers was used to build up the new cities. Iron, used in Europe for pipe as early as 1685, was scarce in the United States and much more valuable as material for muskets. So our first pipe lines in such cities as New York, Boston and Philadelphia were constructed of bored logs as early as 1752.

American ingenuity was even then working hard to solve the problem of a pipe with real durability, and by 1825 a method of manufacturing pipe from long strips of hot metal was devised. This might be said to be the first basis for making strong pipe economically. Pipe mills, making wrought iron pipe, sprang up in several cities, and with the development of the Bessemer process in 1855 and the open hearth process in 1861, steel, the strongest and most versatile refinement of iron, became available for water pipe. The long years of steady development to combine the vitally necessary durability with strength had finally ended, and steel pipe was ready to play the truly dramatic role it has filled in the development of the country

Long Service Records

Available records disclose installations of steel pipe still in use which were laid as early as 1863 in a five mile line for supplying water to San Francisco. Beginning in 1870 with other riveted lines, and in 1887 with the installation of the first welded steel pipe, records show scores of examples of steel pipe which was laid more than 50 years ago. These records of long service attest to the basic durability of steel when it is remembered that a majority of the pipe was laid before the advent of modern protective linings, coatings and wrappings. Of particular interest is the considerable improvement in the quality of steel which is continually taking place. Modern steel pipe mains, properly lined, coated, wrapped and installed, can be expected to have a long, useful life.

Use Today

More than 200 of the major cities of the United States now have a total of more than 100 million feet of steel water pipe in use. This figure could be greatly increased by the inclusion of pipe in use by thousands of smaller municipalities, as well as the various district, state and national public projects which call for the use of water carrying pipe. Foreign cities and governments, too, have been heavy users of steel pipe for many years.

Long Time Users

The following list of installations illustrates the longevity of serviceable use as a major characteristic of steel water pipe:

PARTIAL TABULATION OF STEEL WATER PIPE INSTALLATIONS PRIOR TO 1916

YEAR INSTALLED	LOCATION	DIAMETER INCHES	FOOTAGE	THICKNESS
1858	Railroad Flat, Cal.	22”-11”	——	16 Ga.
1860	New York, N. Y.	90”	1,000’	l/2”
1863	San Francisco, Cal.	37”-30”	27,280’	l/4”
1870	San Francisco, Cal.	30”	42,240’	12 Ga.
1870	Magalia, Cal.	30”	——	10 Ga
1870	Pioche, Nev.	5	8,000’	14 Ga.
1871	San Francisco, Cal.	22”	2,105’	9 Ga.
1873	Santa Rosa, Cal.	11”-9”	10,000’	16 Ga.
1873	Virginia City, Nev.	12”	37,000’	5/16”-l/16”
1874	Carson City, Nev.	12”-7”	10,000’	16 Ga.
1874	Pittsburgh, Pa.	50”	2,900’	—
1875	San Francisco, Cal.	22”	2,226’	9 Ga.
1878	Texas Creek, Cal.	17”	4,000’	9-14 Ga.
1880	Los Angeles, Cal.	44”	——	—
1880	San Fernando, Cal.	8”	——	—
1881	Lawrence, Mass.	77”	——	3/8”
1882	San Francisco, Cal.	22”	6,800’	l/8”-3/16”
1882	San Francisco, Cal.	30”	3,480’	1/4”
1882	Longmont, Colo.	6”	23,000’	—
1883	Fort Collins, Colo.	10”	18,000’	3/16”
1884	San Francisco, Cal.	33”	2,409’	l/4”
1885	San Francisco, Cal.	30”	13,409’	l/4”
1885	San Francisco, Cal.	44”	90,000’	6 & 7 Ga.
1887	Riverside, Cal.	24”	45,000’	—
1887	Pasadena, Cal.	6”	——	—
1888	San Francisco, Cal.	22”	12,000’	—
1888	Pasadena, Cal.	22”	18,000’	14 Ga.
1888	Sierra Madre, Cal.	4” & 6”	15,000’	16 Ga.
1888	Altadena, Cal.	8”	1,200’	16 Ga.
1888	Redlands, Cal.	24”	2,200’	11 Ga.
1889	Nephi, Utah	3,,	1,500’	16 Ga.
1889	Alhambra, Cal.	7,,	900’	16 Ga.
1889	San Francisco, Cal.	44”	4,878’	3 Ga.
1889	Pasadena, Cal.	13”	6,000’	14 Ga.
1890	San Jose, Cal.	18”	31,000’	12 Ga.
1890	Santa Cruz, Cal.	14”	——	9 Ga.
1890	Detroit, Mich.	72”	——	—
1890	Redlands, Cal.	8”	6,000’	—
1890	Walla Walla, Wash.	20" - 6"	500,000’	7-14 Ga.
1891	Newark, N. J.	48”	111,800’	l/4”-3/8”
1891	Newark, N. J.	36”	23,980’	l/4”
1891	Pittsburgh, Pa.	50”	3,600’	5/8”
1891	The Dalles, Ore.	10”	8,000’	10 Ga.
1891	Pocatello, Ida.	12”	6,000’	16 Ga.
1892	Pasadena, Cal.	8” & 4”	3,000’	—
1892	Butte, Mont.	20”	3,114’	—
1893	Syracuse, N. Y.	54”	6,500’	3/8”
1893	Rochester, N. Y.	36”-38”	136,000’	—
1894	Portland, Ore.	42”-33”	132,000’	5/16”-6 Ga.
1894	Passaic Valley, N. J.	30”	12,300’	5/16”
1895	Pittsburgh, Pa.	60”	49,000’	l/2”
1895	Altadena, Cal.	12”	5,000’	14 Ga.
1895	Pasadena, Cal.	8”	1,200’	14 Ga.
1895	Vancouver, B. C.	22” & 16”	52,000’	12 Ga.
1895	San Francisco, Cal.	30”	4,090’	l/4”
1895	Kearney, N. J.	42”	8,800’	—
1896	Minneapolis, Minn.	48”	31,680’	—
1896	Newark, N. J.	48” & 42”	111,600’	l/4”
1896	Passaic Valley, N. J.	42”	8,700’	l/4”-3/8”
1896	New Bedford, Mass.	48”	42,000’	5/16”
1896	Bayonne, N. J.	30”	44,000’	—
1896	New Westminster, B. C.	14”	70,000’	—
1896	New York, N. Y.	72”	——	—
1897	Minneapolis, Minn.	50”	16,605’	7/16”
1897	Ogden, Utah	72”	4,600’	—
1897	Patterson, N. J.	42”	18,600’	5/16”
1897	Jersey City, N. J.	48”	——	l/2”
1898	Red Bluff, Cal.	7”	9,000’	14 Ga.

YEAR INSTALLED	LOCATION	DIAMETER INCHES	FOOTAGE	THICKNESS
1898	Duluth, Minn.	42”	30,500’	l/4"-1/2”
1898	Allegheny, Pa.	50”	——	—
1898	Albany, N. Y.	48”	8,000’	—
1899	Lawrence, Mass.	108”	154’	3/8”
1899	Passaic Valley, N. J.	51”	44,600’	l/4”-7/16”
1899	Seattle, Wash.	42”	32,000’	l/4” 12 Ga.-l/2”
1899	Kern, Cal.	60”-48”	5.000’	l/4”
1899	Newark, N. J.	51”-48”	17,000’	—
1899	Pittsburgh, Pa.	48”	4,400’	14 Ga.
1900	Redlands, Cal.	4”	3,000'	7 Ga.
1900	San Francisco, Cal.	36”	420’	3/16”
1900	Victor. Colo.	29”	2,500’	—
1900	Marquette, Mich.	42”	600’	—
1900	Butte, Mont.	26”	33,910’	5/16”
1900	Passaic Valley. N. J.	42”	18,600’	3/8”
1901	Pittsburgh, Pa.	42” & 50”	17,000’	l/4”
1901	Atlantic Citv. N. J.	30”	27,000’	—
1901	Seattle, Wash.	42”	61,000’	12 Ga.
1902	Glendora, Cal.	8"	1,120’	14 Ga.
1902	Altadena, Cal.	8”	3,600’	12 Ga.
1902	Montebello, Cal.	20"	——	5/16”
1902	Jersey City, N. J.	72"	93,000’	1/2”
1903	Pittsburgh, Pa.	48”	4,000’	l/4”
1903	Sacramento, Cal.	24"	9,000'	5/16”
1903	Newark, N. J.	60"-48”	40,000’	—
1903	Kansas City, MO.	36”	35,000’	3/8”
1903	Troy, N. Y.	33”	29,000’	14 & 16 Ga.
1903	Chino, Cal.	12”-4”	——	—
1903	Schenectady, N. Y.	36”	23,716’	l/4”-3/8”
1904	Bayonne, N. J.	30”	4,000’	—
1904	Astoria, L. I., N. Y.	60”	15,000’	—
1904	Erie, Pa.	60”	7,920’	—
1904	Toronto, Ont., Canada	72”	6,000’	12 Ga.
1904	Pasadena, Cal.	8”	1,688’	10 Ga.
1904	Red Bluff, Cal.	12”	1,600’	10 Ga.
1904	San Bernardino, Cal.	20”	16,800’	l/2”-3/ 16”
1905	Los Angeles, Cal.	98”-16”	1,108,000’	3/16”
1905	Tillamook. Ore.	10”	24,000’	l/2”
1905	St. Louis, MO.	84”	18,960’	l/4”
1905	Passaic Valley, N. J.	48”-42”	10,000’	3/8”-l/4”
1905	Pittsburgh. Pa.	50” & 30”	28,500’	12 Ga.
1905	Pasadena, Cal.	12”	1,500’	16 Ga.
1905	Altadena, Cal.	8” & 4”	5,000’	—
1905	Lynchburg, Va.	30”	11,500’	—
1905	Wilmington. Del.	48”-43”	20,000’	—
1905	Paterson, N: J.	48”-42”	11,500’	l/4”
1905	Cincinnati, Ohio	84”	1,521’	—
1905	Springfield, Mass.	42”-54”	63,500’	—
1906	Brooklyn, N. Y.	72”	42,300’	—
1906	Philadelphia. Pa.	48”-36”	86,980’	—
1906	Pittsburgh, Pa.	72”-30”	47,000’	3/8”-l/4”
1906	New York, N. Y.	72”	125,000’	7/16”
1906	Honolulu, T. H.	30”	8.000’	—
1907	Corona Heights, Cal.	9”	1,400’	14 Ga.
1907	Pasadena, Cal.	4”	1,235’	16 Ga.
1907	Trinidad. Colo.	15”	184,800’	l/4”
1907	Wilmington, Del.	43”-48”	20,340’	7/16”
1907	Trenton, N. J.	48”	7,000’	5/16”
1907	Lockport, N. Y,	30”	68,640’	l/4”
1907	Pittsburgh, Pa.	36”	3,700’	3/8”
1907	Vancouver, B. C.	30”-22”	65,000’	l/4”-5/16"
1907	St. Louis, MO.	84”	——	—
1907	Montreal, Canada	36"	11,000’	—
1907	Gary, Indiana	36”	4.000’	l/4”
1907	Philadelphia, Pa.	48”-36”	54,000’	—
1908	Canyon, Cal.	36”	1,500’	12 Ga.
1908	Springfield, Mass.	42”	75,000’	l/4”
1908	Missoula, Mont.	6”	20,000’	3/16”
1908	Passaic Valley, N. J.	30”	15,400’	l/4”
1908	Seattle, Wash.	52”-42”	15,000’	—
1908	Michigan City, Ind.	30”	4.000’	—

YEAR INSTALLED	LOCATION	DIAMETER INCHES	FOOTAGE	THICKNESS
1908	Montreal, Canada	36”	25,000’	—
1908	Philadelphia, Pa.	132”	1,590’	—
1909	Beaumont, Cal.	4”	16,000’	16 Ga.
1909	Springfield, Mass.	42”	24,200’	3/8”
1909	Seattle, Wash.	51”	7,660’	3/8”
1909	Portland, Ore.	48”-24”	17,600’	1/4”
1909	Boulder, Colo.	60”	2,640’	l/2”
1909	Erie, Pa.	56”	5,280’	—
1909	Vancouver, B. C.	24”	73,000’	l/4”-3/16"
1909	Brooklyn, N. Y.	72”	83,000’	—
1910	Ensley, Ala.	50”	8,840’	3/8”-5/16”
1910	Pasadena. Cal.	10”-6”	6,000’	14 Ga.
1910	Longmont, Colo.	16”	22,000’	l/4”
1910	New York, N. Y.	48”	16,000’	7/16”
1910	Pittsburgh, Pa.	24”	5,000’	—
1910	Portland, Ore.	52”-44”	128,000’	l/4”
1910	Seattle, Wash.	42”-24”	23,600’	l/4”
1910	New York, N. Y.	36”	11,000’	3/16”
1910	New York, N. Y.	135” & 117”	33,000’	7/16”-3/4”
1910	Montrose. Cal.	36” & 26”	5,200’	—
1910	Pittsburgh, Pa.	24”	5,000’	—
1910	Brooklyn, N. Y.	48”	16,200’	—
1910	Butte, Mont.	42”	1,200’	—
1910	Washington, D. C.	30”	1,220’	—
1910	Paterson, N. J.	42”	2,000’	5/16”
1911	Philadelphia, Pa.	20”	7,700’	—
1911	Glendora, Cal.	9”	5,000’	14 Ga.
1911	Los Angeles, Cal.	120”-90”	49,575’	l/4”-1-1/8”
1911	Pasadena, Cal.	30"	10,297’	10 Ga.
1911	Denver, Colo.	60"	1,111’	3/8”
1911	Portland, Ore.	52” & 44”	130,000’	l/4” & 5/16”
1911	Seattle, Wash.	42”-24”	16,000’	l/4”
1911	Tacoma, Wash.	46”-39”	7,300’	l/4”-l/2”
1911	Montreal, Canada	48”-30”	7,300’	—
1911	Lakeland, Fla.	20”	4,000’	—
1911	Massena, N. Y.	24”	1,323’	—
1911	Marquette, Mich.	66”	8,000’	—
1911	New York. N. Y.	66”	8,510’	—
1912	Chino, Cal.	12”	10,500’	14 Ga.
1912	Los Angeles. Cal.	68”-64”	28,940’	5/16”-3/8”
1912	Pittsburgh, Pa.	30”	5,300’	l/2”
1912	Seattle, Wash.	42”	13,243’	—
1912	Omaha, Neb.	48”	10,550’	—
1912	Ottawa. Canada	42”	2.400’	—
1912	Pittsburgh, Pa.	60”-72”	5,280’	3/8”-1/2”
1912	Union Bav. B. C.	50”	1,326’	—
1912	Rochester, N. Y.	66”	9,200’	—
1912	Winnipeg, Canada	36”	42,500’	—
1912	Akron, Ohio	36”	56,000’	—
1912	Altman, N. Y.	138”-96”	2,000’	—
1912	Belleville, Ohio	168”	2,920’	—
1912	Montclair, N. J.	24”	7,343’	—
1913	Los Angeles, Cal.	72”	——	—
1913	Baltimore, Md.	120”	2,465’	7/16”
1913	Minneapolis, Minn.	48”-54”	27,000’	5/16”-7/16”
1913	Montclair, N. J.	24”	7,325’	l/4”
1913	Utica, N. Y.	36”	1,000’	l/4”
1913	Murray City, Utah	26”-22”	3,882’	7 Ga.
1913	Vancouver, B. C.	36”-26”	46,250’	3/8”-1/4”
1913	Winnipeg, Canada	36”	42,000’	l/4”
1913	Schenectadv. N. Y.	24”	2,420’	l/4”
1913	Kansas City, MO.	48”	1,220’	—
1913	Massena, N. Y.	24”	1,200’	—
1913	Wilkes-Barre, Pa.	36”	1,335’	—
1913	Cleveland, Ohio	48”	2,265’	l/4”
1913	Falls Village, Conn.	108”	826’	5/16”-3/8”
1913	Lock Raven. Md.	120”	2,464’	7/16”
1913	Ocoe, Tenn	96”	1,320’	5/8”
1913	Crogham N. Y.	114”	2,555’	—
1913	Altman, N. Y.	138”	1,194’	5/8”
1914	Pittsburgh, Pa.	42”-48”	3,060’	—

YEAR INSTALLED	LOCATION	DIAMETER INCHES	FOOTAGE	THICKNESS
1914	Gardena, Cal.	12”-4”	——	16 Ga.
1914	Glendora, Cal.	8”	1,984’	14 Ga.
1914	Glendora, Cal.	12”	3,300’	12 Ga.
1914	Minneapolis, Minn.	48”	11,970’	l/4”-l/2”
1914	Butte, Mont.	24”	12,950’	—
1914	New York, N. Y.	66”	12,500’	7/16”-l/2”
1914	Schenectady, N. Y.	36”	10,500’	3/8”
1914	Tacoma, Wash.	30”	550’	l/4”
1914	Winnipeg, Canada	36”	21,569’	l/4”
1914	Springfield, Mass.	42”	——	—
1914	Essex Junction, Vt.	108” & 36”	2,440’	—
1914	Rutland, Vt.	54”	2,750’	—
1914	Rochester, N. Y.	66” & 48”	1,120’	—
1914	Cleveland, Ohio	48”	1,320’	—
1914	Massena, N. Y.	24”	22,000’	l/4”-3/8”
1914	Miami, Ariz.	152”	1,670’	—
1914	Riverside, Cal.	30”	35,000’	—
1915	Cleveland, Ohio	66”-72”	——	3,960’
1915	Oakdale, Cal.	12”	50,000’	14 Ga.
1915	Baltimore, Md.	84”	4,000’	7/16”
1915	Lewiston, Mont.	16”	30,000’	l/4”-3/16”
1915	Pittsburgh, Pa.	48”	3,900’	l/2”
1915	Greeley, Colo.	20”	5,280’	—
1915	Massena, N. Y.	24”	5,000’	5/16”3/8”
1915	Ogden, Utah	24”	17,250’	—
1915	Ottawa, Canada	51”	15,000’	—
1915	San Bernardino, Cal.	20”	3,500’	3/16”

Uses of Steel Pipe

Transmission Mains
Distribution Mains Intake
Lines Discharge Lines
Sewer Lines
Siphons
Pumping Plant Piping
Power Plant Piping
Penstocks Underwater Crossings
Crossings Under Railroads and Highways
Self-supporting Spans Over Marshes or Streams
On-Bridge Crossings
Dredge Pipe

1907. Elsie Janis was packing ‘em into the Opera House when a Philadelphia city photographer snapped a pipe-playing crew on Broad Street in 1907. The City was in the process of installing nearly 27 miles of steel pipe in 48-in. and 36-in. diameters. Some of the pipe was later removed to make way for the Broad Street Subway, but most of it is still in service - after more than 75 years on the job.

Advantages of Steel Pipe

Choosing The Material

When we say that steel pipe has "advantages," we mean, of course, that it has attributes which make it a better carrier of fluids than pipe made of other materials. Modern conditions, with their mounting demands on materials as a result of stresses, strains and emergency conditions to which they are subjected, make it essential for officials, engineers and contractors charged with the responsibility of designing, building and maintaining conveying systems to select the best material. And the material selected should qualify as "best" in every way.

Comparison with other commonly used materials reveals that steel pipe does qualify as the best in every essential respect for use in your system.

Essential Requirements

The essential requirements which material for a pipeline must meet are relatively simple.

They can be listed as:

1. Strength and toughness.
2. Durability and long service life.
3. Economy of installation and maintenance.
4. Permanent high-carrying capacity.
5. Ductility and adaptability.
6. Reliability and resiliency.
7. Watertight joints.

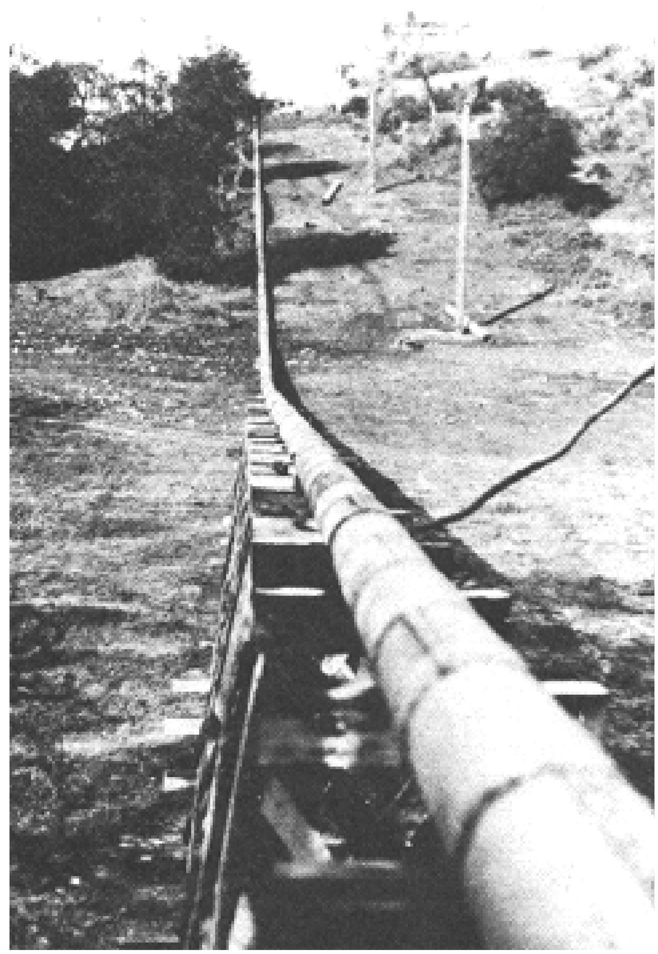
Steel pipe, when properly designed, installed and properly protected, answers each of these basic requirements better than any other material now used. Here is why:

1. An advantage of steel pipe over other materials is its great tensile strength. It stands alone in offering the greatest strength in proportion to wall thickness of any commercial piping material available for use in conveying systems.

There is no substitute for strength.

2. When it comes to durability and long life, steel pipe makes a superior showing among all types of fluid-carrying materials. Available records show many instances where steel pipe has been in service for more than 100 years . . . and is still doing a commendable job. With the great advances that have been made during the last few decades in the fabrication of steel and perfection of coatings, the useful life of properly designed steel pipe can now be conservatively estimated at 100 years or longer.

3. Steel pipe usually costs no more – and frequently costs less- to buy and install, and an all-important advantage is the economy of maintenance which characterizes a steel pipeline.



In 1858, steel sheets, shipped to San Francisco for building needs, were rolled into pipe 11 inches to 22 inches in diameter and installed in Calaveras County, Calif. The pipe has been in use practically continuously since that time.

4. Every system needs the maximum possible carrying capacity. Population increases can make a line obsolete quickly unless it can be depended upon not only to have the greatest possible capacity when installed, but to maintain that capacity in use. Properly protected steel pipe is resistant to corrosion and incrustation. It can be relied on to maintain its carrying capacity. An additional advantage is the wide margin of safety normally engineered into steel pipe. As a result, it is sometimes possible, in the event of greater future demands, to increase the carrying capacity by boosting the pressure, and still stay well within safety limits.

5. Of almost equal importance, the ductility of steel pipe is a unique advantage as compared with other materials. It is this unusual characteristic available only in steel pipe which makes possible its wide use in terrain situations where other materials either cannot be used, or can be installed only with difficulty or at additional expense.

6. Reliability is perhaps not so much an advantage as it is a definite necessity in any line. Once installed, engineers can depend upon steel pipe to do the job for which it was designed. This reliability extends not only to constant carrying capacity, but to its ability to withstand unexpected or emergency conditions. It means resistance to water hammer and to washouts. It includes the *resilience* to "give" under soil movement and surface vibrations.

7. A requirement of utmost importance which steel pipe fills completely is the necessity for "bottle tight" joints. No water line can operate successfully and economically without leak proofed joints. Wastage of fluids can be, as every engineer knows, the most expensive fault of any system. Here steel pipe excels. Joints in steel pipe, whether welded, mechanical, or rubber gasket, are completely water tight.

Steel . . .The Ideal Material

No other material offers all the requisites for a good conduit strength, long life, economy, permanent high carrying capacity, ductility, reliability, and "bottle-tight" joints.

Perhaps the major consideration in the selection of pipe material for conveyance of water should be strength without the handicap of rigidity. In this respect, steel surpasses all other materials because it has maximum strength with maximum ductility. The specified minimum tensile strength of steel normally used for water pipe is 60,000 pounds per square inch. Because of this strength and ductility, steel pipe resists suddenly applied emergency pressure, surge, water hammer, earthquakes, traffic vibrations, settlement, cave-ins, washouts, floods,

Tape coatings have been used in oil and gas pipe installations for 30 years, and are gaining popularity in the waterworks industry for their excellent protection against corrosion. Denver Water Board uses 91"I.D., 7/16" wall pipe on a major project shown here.

temperature changes, blastings, bombings, and other similar conditions which so easily destroy other types of pipe. This means that, unlike rigid materials, steel not only has great strength but also has the ability and toughness to withstand great shock without shattering.

Added to these major advantages of steel pipe are its many other complementary points of superiority. These include a wide selection of sizes, wall thicknesses and length which give you a pipe "custom tailored" for a particular job. Precision fabrication in conformity with A.W.W.A. standards, and careful testing, provide greater assurance of a satisfactory line. And its longer lengths, combined with considerably less weight than other materials, keep transportation and installation costs at a minimum.

Results Tell the Story

Whatever your basic interest, steel pipe will give you superior results:

1. *For the owner,* its nonporous structure and its leak-proof joints mean that profits won't seep away due to leakage. Rates won't need increasing to help pay for water wastage. Because of steel's ability to stand shock and vibration, costly damage claims resulting from sudden pipe failures will be minimized. These factor will improve public relations and customer goodwill with no increase in cost of installation.

2. *For the design engineer,* the many different diameters, wall thicknesses, joints and strength levels available minimize design problems. The fact that steel pipe will withstand a wide range of pressures again reduces design problems. Normal engineering practice provides for safety factors of 3 or more against bursting, whereas with pipe of rigid material it runs far less, sometimes as little as 1 1/2**.** This means that a steel line designed for a specific rate of flow can- should it become necessary - deliver much more water at higher pressures, yet still maintaining a margin of safety. And because each length of steel pipe is thoroughly tested for strength, the engineer can be sure he will get what he pays for.

3. *For the contractor,* the longer lengths available in steel pipe are especially important because they mean fewer field joints. For example, 40 foot lengths of 48" steel pipe, fabricated to withstand over 200 psi, require only 132 joints per mile as compared to 330 joints required by pipe made in 16 foot lengths. In addition, the 40 foot length of 48" steel pipe weighs approximately 7,000 pounds, whereas a 16 foot length of the same diameter in concrete weighs about 15,000 pounds. This means steel pipe not only requires fewer field joints, but permits using lighter field equipment to lay the line and a good chance of saving thousands of dollars in installation costs!

4. *For the operating engineer,* steel pipe's leakproof characteristic and its ductility mean fewer operating troubles. The sudden and complete failure of a properly designed steel pipe line is rare; thus, costly emergency calls to repair "breaks," flooded sub-surface structures, and cave-ins are virtually eliminated.

5. *For the "average citizen,* " a steel pipe water line reduces the likelihood of streets washing out and interrupting service - lower water rates, and a saving in his tax bill. And his investment is doubly protected because it is possible to design the line to provide for future increased requirements as well as present.

Therefore, regardless of where your interest lies in the conveyance of water, you will find that steel pipe is the ideal material.

San Diego, CA 96” I.D. Pipe

Conclusions

The contents of this manual may be summarized as follows:

1. Steel pipe for water service meets the highest requirements when properly designed, installed and permanently protected. Its useful service life, therefore, is assured for many decades. Its maintenance cost under these circumstances is at least as low and in most cases lower than any other type of water pipe.

2. Steel pipe possesses high strength (specified yield strength of 30,000 to 50,000 p.s.i and tensile strength of 50,000 to 70,000 p.s.i.); high sustained carrying capacity; durability, economy; and reliability.

3. Steel pipe, in general, is adaptable, versatile, efficient, uniform, flexible, watertight, resilient, secure, ductile, tough, easy to lay, highly resistant to sudden rupture, and strong as a beam. It is practically the only type of pipe that can be installed both above and below ground with confidence. Steel pipe is a modern necessity, and experience has proved its important contribution to the life of our nation and of the world.

Design Procedures

The most commonly used formula for computing the flow of water in a pipe line is the HazenWilliams formula:

$$V = 1.318\, C_W r^{.63} s^{.54}$$

Where
- V = velocity, feet per second
- r = hydraulic radius, feet (D/4)
- s = hydraulic slope, feet per foot
- C_W = Hazen-Williams coefficent = 140

From the standpoint of conservative design in determining the proper size of welded steel pipe lines, the following value of the coefficient of friction may be used with the above formula.

$$C_W = 140 \text{ for which } Q = 60.5\, D^{2.63} s^{.54}$$

Q = discharge, sec. ft.

D = diameter feet

This value is considered to be suitable for average conditions over a long period of time. New straight steel pipe will have greater than the indicated flow values. The chart on page 11 graphically solves the flow formula. If other coefficient values are desired, use multiplying factors shown on the flow diagram.

Diameter

Knowing the quantity of water to be carried by the pipe, the length of the pipe line, and the available static head, the proper diameter then can be determined.

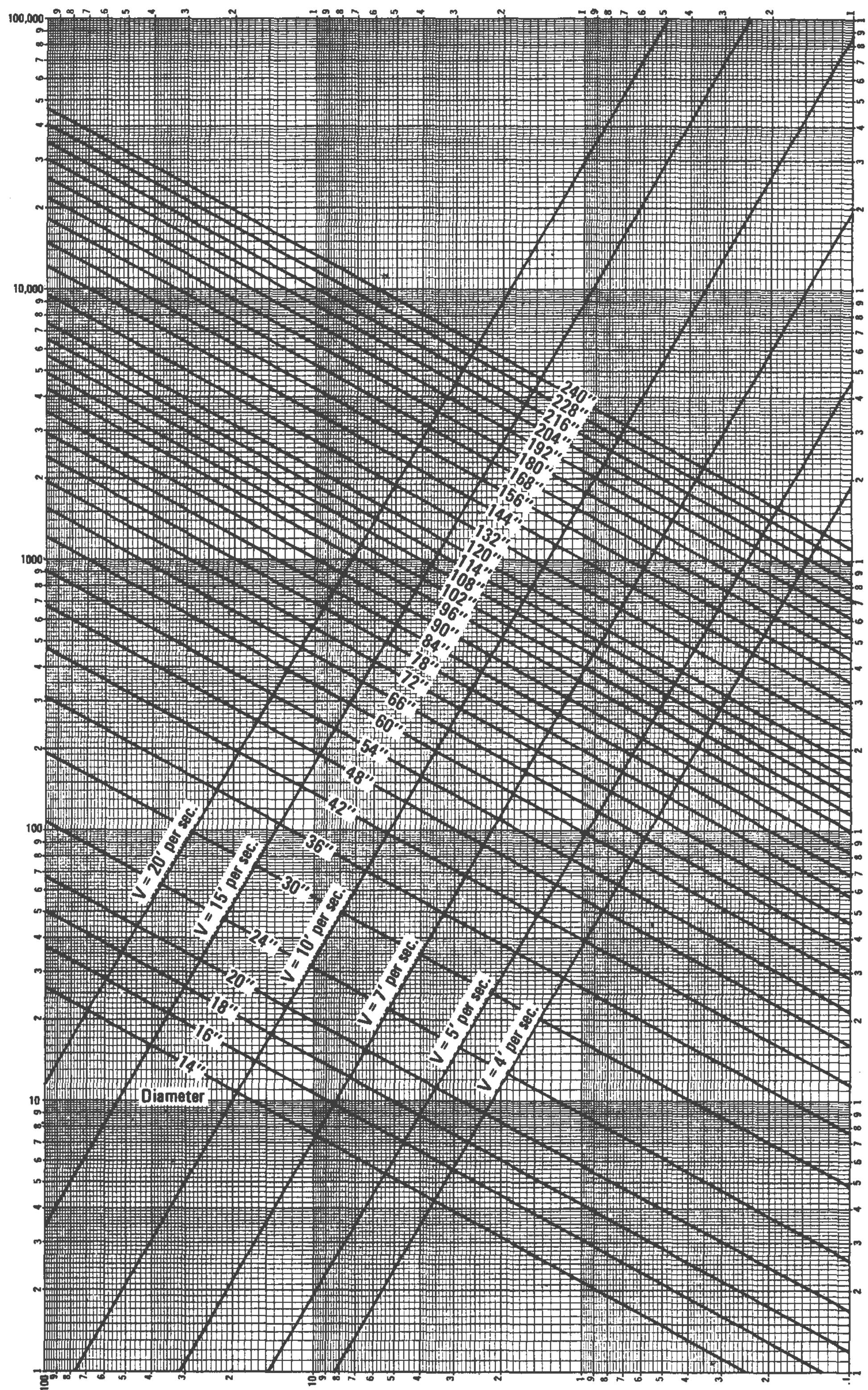
FLOW OF WATER

Solution of the Hazen- Williams formula,

based on $C_w = 140$ $\}$ $V = 1.318\, C_w r^{.63} s^{.54}$

BaseC_W=140

Other C_w Values	150	145	140	130
Relative Discharge	1.071	1.036	1.000	0.920



Discharge in Second Feet

s = Hydraulic Gradient in Feet Per Thousand Feet

Water Hammer

The basic fundamentals of water hammer in water pipelines are covered by the elastic wave theory, and include velocity of flow, length of the pipeline, time of closure, and the pressure wave velocity. For instantaneous closure, water hammer value can be determined from the formula:*

$$h = \frac{av}{g}$$

where
- h = head in feet above static head.
- a = velocity of pressure wave, ft. per sec.
- v = flow line velocity, ft. per sec.
- g = 32.2 ft. per sec. per sec.

Pressure wave velocity can be determined from the formula:

$$a = \frac{4660}{\sqrt{1 + \frac{k}{E} \times \frac{d}{t}}}$$

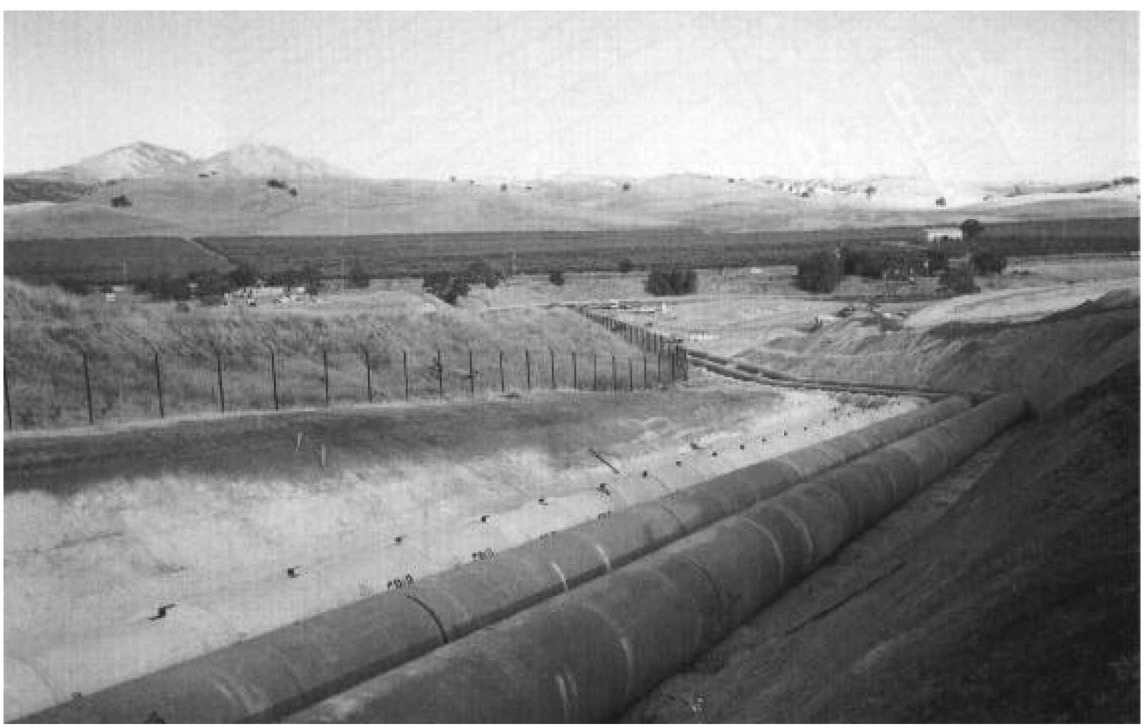
where
- k = modulus of compression of water = 300,000 psi
- E = modulus of elasticity of pipe material = 300,000 psi for steel
- d = diameter of pipe, inches.
- t = thickness of pipe wall, inches.
- L = length of line, feet.

For steel pipe, "a" varies from 4660 ft. per sec. for $\frac{d}{t}$ = 0 to 3400 ft. per sec. For $\frac{d}{t}$ = 90.

The critical time for valve closure is $\frac{2L}{a}$ which represents one wave cycle. For any cycle having less time than $\frac{2L}{a}$ the water hammer should be considered an instantaneous closure. Therefore, to keep surge pressures down to a minimum it is essential that valve closures be accomplished slowly, and that air chambers or other similar devices be used, if required.

As a general condition, steel pipe designed on a 50% specified yield strength basis can safely withstand an occasional water hammer surge when it does not cause the stress in the steel pipe to exceed 75% of the yield strength. As such, the pipe design does not require any allowance factor for water hammer.

* "Waterhammer Analysis," John Parmakian, Dover Publications, Inc., New York 1963.



Las Vegas, NV   108" I.D. Pipe

Materials

High-strength steel sheet, plates and coils having of miles of high pressure gas, oil and water lines, penstocks, pressure vessels and bridges all over the world. The tons of steel involved have run into the millions, and the benefits derived have been substantial.

The table below lists ASTM Plate and Sheet Steels for Pipe. The proper use of these steel grades can be made by considering all of the conditions involved for each case and comparing their strength to-cost ratios.

Generally, high-strength steels are advantageous only when a high internal pressure governs the pipe wall thickness. On long high-head penstocks, transmission lines or discharge lines, it may be more economical to use several types of steel as the pressure decreases along the line.

For low pressure installations where the wall thickness is governed by the external load, high strength steels have no advantage.

For the usual water transmission pipeline, AWWA has recommended a design stress at working pressure of at least 50% of the minimum yield point of the steel.

It is important that the designer be aware of the current developments in the technology of steel, so that the material may be used in a manner which effects the greatest economy without sacrificing safety, security and quality.

ASTM PLATE AND SHEET STEELS FOR PIPE

ASTM Designation	Grade	Minimum Yield ksi (mpa)	Minimum Tensile ksi (mpa)
Steel Plate			
ASTM A36/36M		36(348)	58(400)
ASTM A283/283M	C	30(205)	55(380)
	D	33(230)	60(415)
ASTM A572/572M	42	42(290)	60(415)
	50	50(345)	65(450)
Steel Sheet			
ASTM A139/139M	B	35(240)	60(415)
	C	42(290)	60(415)
	D	46(315)	60(415)
	E	52(360)	66(455)
ASTM A570/570M	30	30(205)	49(340)
	33	33(230)	52(360)
	36	36(248)	53(365)
	40	40(276)	55(380)
	45	45(310)	60(415)
	50	50(345)	65(450)
ASTM A607/607M	45	45(310)	60(415)
	50	50(345)	65(450)
ASTM A907/907M	30	30(205)	49(340)
	33	33(228)	52(360)
	36	36(248)	53(365)
ASTM A935/935M	40	40(276)	55(380)
	45	45(310)	60(415)
ASTM A936/936M	50	50(345)	65(450)
	50	50(345)	60(415)

Note: When toughness properties are necessary on any steel, the purchaser shall specify the test method, e.g., Charpy Impact test, test temperature and test values.

STEEL WATER PIPE MINIMUM WALL THICKNESS

Based on **internal** pressure only, using *Barlow's* formula $t = \frac{PD}{2S}$

Minimum walls are for 2 steel grades - S = 18,000 psi & S = 21,000 psi

Diameter Inches	Class 100psi		Class 150 psi		Class 200 psi		Class 250 psi		Class 300 psi		Class 350 psi		Class 400 psi	
	S = 18,000	S = 21,000	S = 18,000	S = 21,000	S = 18,000	S = 21,000	S = 18,000	S = 21,000	S = 18,000	S = 21,000	S = 18,000	S = 21,000	S = 18,000	S = 21,000
14	.039	.033	.058	.050	.078	.066	.097	.083	.117	.100	.136	.116	.156	.133
16	.044	.038	.067	.057	.089	.075	.111	.095	.133	.114	.156	.132	.178	.152
18	.050	.075	.064	.100	.085	.042	.125	.107	.150	.128	.175	.150	.200	.171
20	.056	.047	.083	.071	.111	.095	.139	.118	.167	.142	.194	.166	.222	.190
22	.061	.092	.078	.122	.104	.052	.153	.130	.183	.157	.214	.182	.244	.209
24	.067	.057	.100	.085	.133	.114	.167	.142	.200	.171	.233	.200	.267	.228
26	.072	.108	.092	.144	.124	.062	.181	.154	.217	.185	.253	.216	.289	.247
28	.078	.066	.117	.100	.156	.133	.194	.166	.233	.200	.272	.232	.311	.266
30	.083	.125	.107	.167	.142	.071	.208	.178	.250	.214	.292	.250	.333	.285
32	.089	.076	.133	.114	.178	.152	.222	.190	.267	.228	.311	.266	.356	.304
34	.094	.142	.121	.189	.162	.080	.236	.202	.283	.242	.331	.282	.378	.323
36	.100	.085	.150	.128	.200	.171	.250	.214	.300	.257	.350	.300	.400	.342
38	.106	.158	.135	.211	.181	.090	.264	.225	.317	.271	.369	.316	.422	.361
40	.111	.095	.167	.142	.222	.190	.278	.238	.333	.285	.389	.332	.444	.380
42	.117	.175	.150	.233	.200	.100	.292	.250	.350	.300	.408	.350	.467	.400
45	.125	.107	.188	.160	.250	.224	.313	.267	.375	.321	.438	.375	.500	.428
48	.133	.200	.171	.267	.228	.114	.333	.285	.400	.342	.467	.400	.533	.456
51	.142	.121	.213	.182	.283	.242	.354	.303	.425	.364	.496	.425	.567	.485
54	.150	.225	.192	.300	.257	.128	.375	.321	.450	.385	.525	.450	.600	.514
57	.158	.135	.238	.203	.317	.271	.396	.339	.475	.406	.554	.475	.633	.542
60	.167	.250	.214	.333	.285	.142	.417	.357	.500	.428	.583	.500	.667	.571
63	.175	.150	.263	.224	.350	.300	.438	.374	.525	.450	.613	.525	.700	.800
66	.183	.275	.235	.367	.314	.157	.458	.392	.550	.471	.642	.550	.733	.628
69	.192	.164	.288	.246	.383	.328	.479	.410	.575	.492	.671	.575	.767	.656
72	.200	.300	.257	.400	.342	.171	.500	.428	.600	.514	.700	.600	.800	.685
75	.208	.178	.313	.267	.417	.357	.521	.446	.625	.535	.729	.625	.833	.714
78	.217	.325	.278	.433	.371	.185	.542	.464	.650	.556	.758	.650	.867	.742
81	.225	.192	.338	.289	.450	.385	.563	.481	.675	.578	.788	.675	.900	.771
84	.233	.350	.300	.467	.400	.200	.583	.500	.700	.600	.817	.700	.933	.800
87	.242	.207	.383	.310	.483	.414	.604	.517	.725	.621	.846	.725	.967	.828
90	.250	.375	.321	.500	.428	.214	.625	.535	.750	.642	.875	.750	1.000	.857
93	.258	.221	.388	.332	.517	.442	.646	.553	.775	.664	.904	.775	1.033	.885
96	.267	.400	.342	.533	.456	.228	.667	.571	.800	.685	.933	.800	1.067	.914
102	.283	.242	.425	.364	.567	.485	.708	.606	.850	.728	.992	.850	1.133	.971
108	.300	.450	.385	.600	.514	.257	.750	.642	.900	.771	1.050	.900	1.200	1.028
114	.317	.271	.475	.406	.633	.542	.792	.678	.950	.813	1.108	.950	1.267	1.085
120	.333	.500	.428	.667	.571	.285	.833	.714	1.000	.856	1.167	1.000	1.333	1.142
126	.350	.300	.525	.450	.700	.600	.875	.750	1.050	.900	1.225	1.050	1.400	1.200
132	.367	.550	.471	.733	.628	.314	.917	.785	1.100	.942	1.283	1.100	1.467	1.257
138	.383	.328	.575	.492	.767	.656	.958	.821	1.150	.985	1.342	1.150	1.533	1.314
144	.400	.600	.514	.800	.685	.342	1.000	.856	1.200	1.028	1.400	1.200	1.600	1.371
150	.417	.357	.625	.535	.833	.714	1.042	.892	1.250	1.071	1.458	1.250	1.667	1.428
156	.433	.650	.556	.867	.742	.371	1.083	.928	1.300	1.113	1.517	1.300	1.733	1.485
162	.450	.385	.675	.578	.900	.771	1.125	.963	1.350	1.156	1.575	1.350	1.800	1.542
168	.467	.700	.600	.933	.800	.400	1.167	1.000	1.400	1.200	1.633	1.400	1.867	1.600
174	.483	.414	.725	.621	.828	.967	1.208	1.035	1.450	1.242	1.692	1.450	1.933	1.657
180	.500	.828	.750	.642	.856	1.000	1.250	1.071	1.500	1.285	1.750	1.500	2.000	1.714

Structural Analysis of Buried Welded Steel Pipes

Sructural analysis of a buried pipe is the analysis of interaction of the pipe and the soil in which it is embedded. The interaction is complex. Deviations are large - in loads, in geometry, and in properties of materials. Therefore, basic principles of mechanics provide the most realistic analyses. Although approximate, these analyses are kept conservative by assuming worst-case conditions. Safety factors should be ‘considered in any case.

This chapter comprises two sections: General Analyses and Specific Analyses. General Analyses are guides for preliminary design. Specific Analyses are useful in final design, special conditions, and in writing specifications. Examples are included.

General Analyses

General analyses are used for feasibility studies and engineers’ estimates. They are presented here in a typical order of procedure. See Figure 1 and Table 1 for nomenclature. Pertinent variables are:

D/t = ring flexibility of plain* pipe,

E = modulus of elasticity of steel pipe
= 30,000,000 lb/inch^2 (207 GPa),

S = allowable stress in the pipe
= 21,000 lb/inch^2(145 MPa), based on yield strength of 42,000 lb/inch^2 (290 MPa), with a safety factor of two.

*Plain pipe is monolithic (not composite, not lined or coated) and has smooth cylindrical surfaces (not corrugated or ribbed).

1. **Internal Pressure, P:**

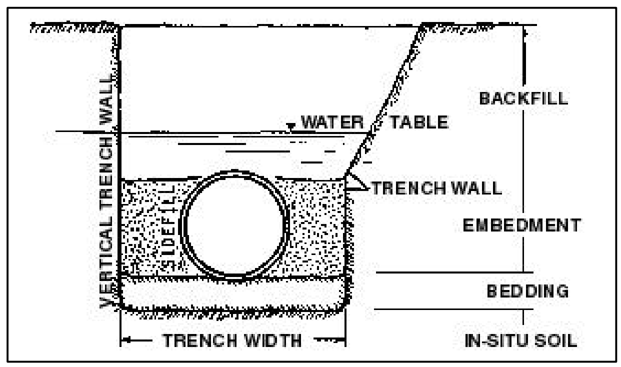
Internal pressure causes circumferential tension stresses in the pipe wall, = PD/2t. This is often referred to as the Barlow formula for hoop stress. Based on allowable stress of 21 ksi (145 MPa), minimum wall thickness is,

$$t = PD/42 \text{ ksi} \quad (t = PD/290 \text{ MPa}) \qquad (1)$$

Maximum allowable pressures in the pipe in terms of ring flexibility are shown in Figure 2. Units must be reconciled.

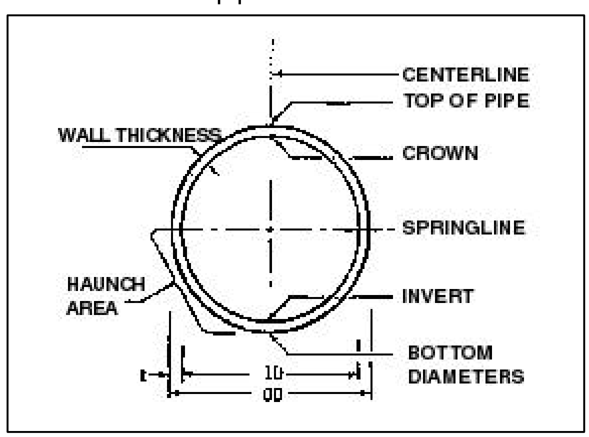
EXAMPLE –

What D/t is required if the internal pressure is 150 psi (1 MPa)? From Figure 2, ring flexibility must be less than D/t = 280.



Trench Cross Section

If the in-situ soil is of poor quality, it may be over excavated and replaced by better soil to provide a foundation for the pipe.



Pipe cross section (ring).

Figure 1. Terminology for cross sections of buried plain steel pipes. Plain pipes are not lined, or coated, or ribbed, etc.

Abbreviations:

kps = kilo-pound [k = kilo = 10^3, M = mega = 10^6, G = Giga = 10^9]

psi = pounds per square inch [1 psi = 6.9 kPa]

ksi = kips per square inch [1 ksi = 6.90 MPa]

ksf = kips per square foot [1 ksf = 47.9 kPa]

pcf = pounds per cubic foot [1 pcf = 157 N/m^3] (m = metre)

kPa = kilo-Pascals of pressure [Pa = $N/metre^2$]

N = Newton of force [1 lb = 4.4482 N]

in = inch [1 inch = 25 millimetres (mm)]

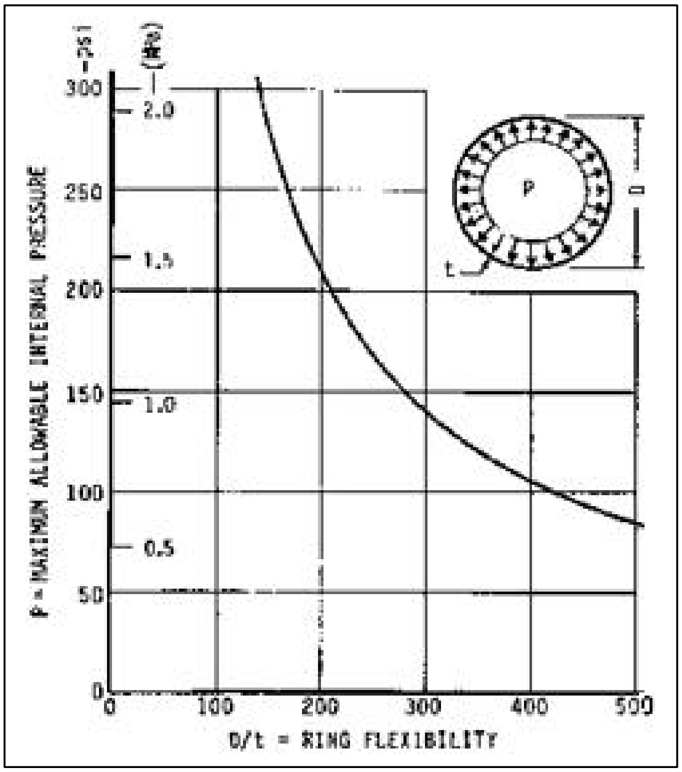


Figure 2. Maximum allowable pressure in steel pipes based on hoop stress of 21 ksi (145 kPa). The safety factor is two based on yield stress of 42 ksi (290 kPa).

Table 1. Notation and Nomenclature Geometry:

D = mean diameter (ID = inside diameter and OD = outside diameter

t = wall thickness

r = mean radius of curvature of the pipe

r_i = radius of curvature (Subscripts $_{i,x,y,A}$ indicate locations)

I = $t^3/12$ = moment of inertia of the pipe wall cross section

L = length of a pipe section or length of a cord of the ring

H = height of soil cover over the pipe

h = height of ground water (or flood level)

Loads, Pressures, and Stresses:

F = diametral line load on the pipe

P′ = vertical external pressure on top of the pipe

P = internal pressure or vacuum (or the equivalent external hydrostatic pressure)

γ = unit weights of materials (Subscripts, s,c,w, etc., refer to saturated soil, concrete, water, etc.)

W = wheel load on ground surface

σ = normal stress (Subscripts, $_x$ and $_y$, refer to directions x and y. Subscript, $_r$, refers to ratio σ_x/σ_y. Subscripts and $_1$, and $_3$ refer to max and min principal stresses.)

σ_f = normal stress in steel at elastic limit

τ_f = shearing stress in steel at elastic limit

= vertical strain (compression) of sidefill soil

Properties of Materials:

S = allowable stress in steel pipes = one-half of yield strength

S_Y = yield stress = 42 ksi (290 MPa) for common pipe steels

E = modulus of elasticity = 30,000,000 psi (207 GPa)

= Poisson ratio = 0.3

ν = coefficient of thermal expansion

α = $6.5(10^{-6})/°F[11.7(10^{-6})/°C]$

E″ = soil stiffness = modulus of the soil (slope of a secant on the stress-strain diagram)

ϕ = soil friction angle

C = cohesion of the soil

Pertinent Parameters (Dimensionless):

D/t = ring flexibility

$E''D^3/EI$ = stiffness ratio = ratio of soil stiffness, E″, to ring stiffness, EI/D^3 or its equivalent, $E/12(D/t)^3$

d/∈ = ring deflection term

d = ring deflection = Δ/D, where

Δ = decrease in vertical pipe diameter

K = $(1+\sin\phi)/(1-\sin\phi)$ = ratio of maximum to minimum stresses, σ_1/σ_3 at soil slip

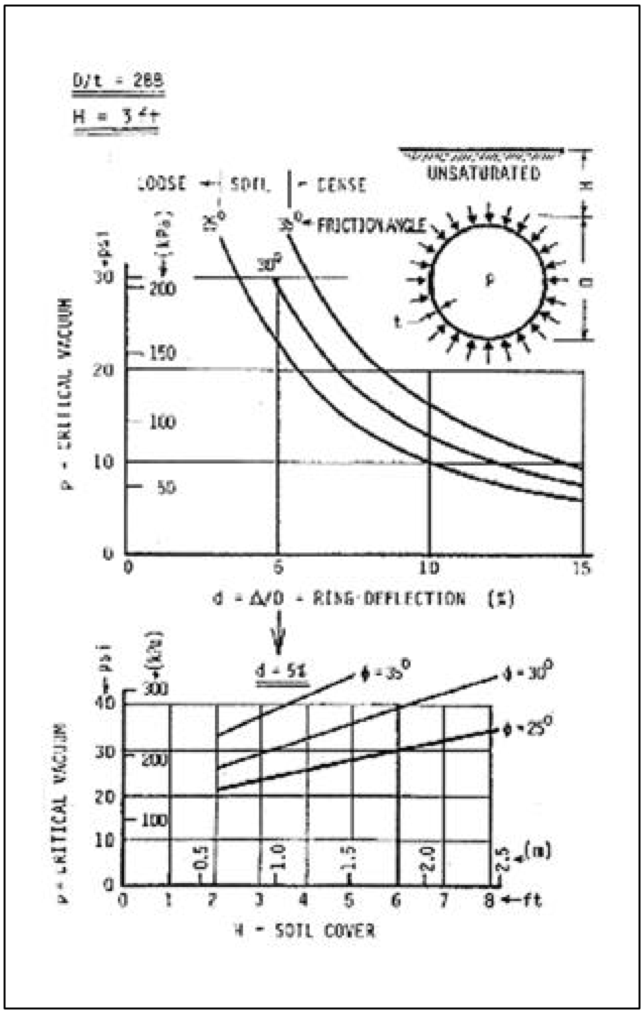


Figure 3. Example of internal critical vacuum in a plain pipe in *unsaturated* soil for which:

D = 72 inch (1800 mm)	H = 3 ft (0.9 m) [top]
t = l/4 inch (6.35 mm)	d = 5% [bottom]
D/t = 288 γ = soil unit weight =	100 pcf (15.7 kN/m^3)

2. Handling and Installing:

The load during handling and installing of pipes is usually a diametral line load which could dent the pipe or crack the mortar lining or coating. See Specific Analyses. Handling may require the use of slings and straps. Pipes must not be stacked higher than their ring deflection limit.

Design and placement of soil is less precise than the design and installation of steel. Because soil embedment supports the pipe and protects the pipe, specifications for soil quality may be appropriate. The soil should be placed carefully, including under the haunches - not just “dozed-in” over the pipe.
The embedment is part of the conduit, and should be installed as a structural component. Below the water table, embedment should be dense enough to resist soil liquefaction. Embedment is the basis for control of ring deflection. Under some circumstances, mechanical compaction may be required.

Based on experience in handling and installing pipes, the following two specifications for minimum wall thickness are in common use:

t = (D + 20in)/400
= (D+500mm)/10.16(10^3)
U.S.Bureau of Reclamation (2)

t = D/288 Pacific Gas & Electric *(3)*

Equation 2 is more liberal in diameters greater than 54 inches (1370mm), and Equation 3 is more liberal in diameters less than 54 inches. From the P.G. & E. formula, ring flexibility is D/t = 288 which is a typical, but conservative, upper limit for buried pipes. If D/t is greater than 288, refer to Specific Analyses.

3. Ring Stability:

Stability is resistance to collapse. Resistance to collapse is a function of ring flexibility, D/t. Generalized recommendations for upper limits of ring flexibility for buried pipes, safety factors included, are as follows:

Upper Limits
D/t = 158 for full atmospheric vacuum,
D/t = 240 for cement mortar lined pipes with flexible Coating (4)
D/t = 288 for less-than-well controlled installations,

D/t = 325 for well-controlled installations.

These recommendations are based on nearly circular pipes (ring deflection less than 5%). These recommendations are conservative. They do not take into account the contribution to ring stiffness of the linings and coatings.

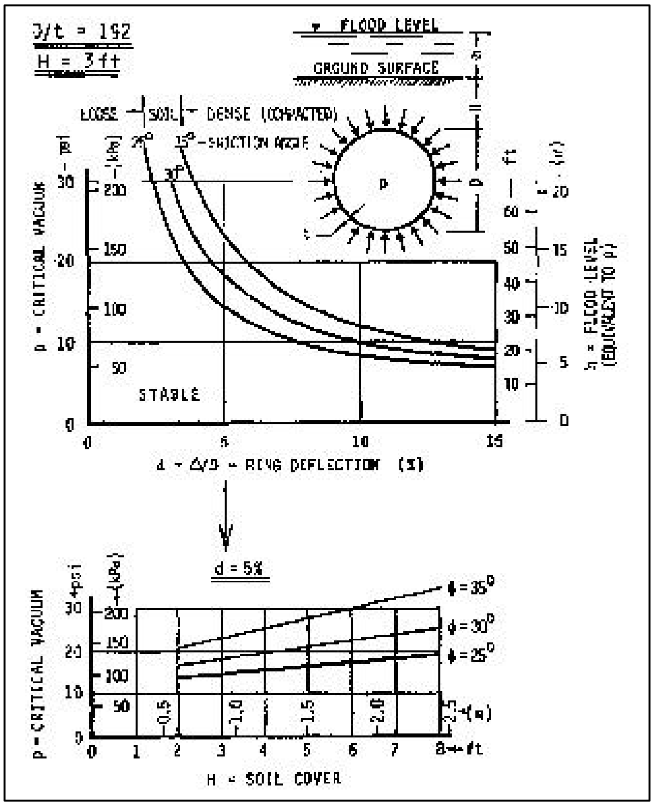


Figure 4. Example of internal vacuum (equivalent flood level) at collapse of an empty, buried, plain pipe in *saturated* soil for which:

D = 72 inches (1830 mm) H = 3 ft (0.9 m) [top]
t = 0.375 inch (9.5 mm) d = 5% [bottom]
D/t = 192 Saturated soil unit wt. = 125 pcf (19.6 kN/m^3)

Unburied

An unburied circular pipe with D/t = 158, or less, can withstand internal vacuum of atmospheric pressure, p = 14.7 psi (100 kPa). The critical vacuum (at collapse) includes external pressure such as the pressure on an empty pipe immersed in water or grout.

Buried

A buried circular pipe is supported by the embedment. Soil support increases critical vacuum depending on the density of the soil and the ring deflection.
However, a water table above the pipe decreases the soil support. Therefore, two cases of soil support must be analyzed unsaturated and saturated.

EXAMPLE —

Unsaturated Soil (water table below the pipe)

A 72 inch plain pipe is buried in *unsaturated* granular soil weighing 100 pcf (15.7 kN/m^3). Height of soil cover is 3 ft (0.9 m). D/t = 288. This is a flexible pipe with t=0.25 inch (6.35 mm). What is the critical vacuum? Figure 3 shows plots of critical vacuum from Specific Analyses. As long as ring deflection is less than 5%, the pipe can withstand vacuum greater than atmospheric - even in less than-well compacted embedment. **If ring deflection is less than 5%, and the embedment is loose but not saturated, the ring is stable under full atmospheric vacuum.**

From the diagram at the bottom of Figure 3, for which ring deflection is d = 5%, critical vacuum, P, increases as the height of soil cover, H, increases. Confinement of the ring is improved with depth of soil cover. In this example [72 inch (1800 mm) pipe, D/t = 288 and H = 3 ft (0.9 mm)], if the pipe diameter is greater than 72 inches (1800 mm), all else unchanged, the critical vacuum is increased. But if the diameter is reduced, all else unchanged, the critical vacuum is reduced. For example, if the diameter is D = 36 inches (900 mm), [D/t = 288, d = 5%, H = 3 ft (0.9 m) and = 25º], critical vacuum is reduced to p = 16 psi (110 kPa) from p = 23 psi when D = 72 inches (1800 mm). It is for this reason that manufacturers reduce D/t for smaller diameter pipes. If D/t were 288 for D = 36 inches (900 mm), the wall thickness would be only 0.125 inch (3.1 mm). 36 inch diameter pipes this flexible are usually lined (and/or coated) to stiffen them.

EXAMPLE —

Saturated Soil (water table above the pipe)

A 72 inch plain pipe is buried in *saturated* granular soil weighing 125 pcf (19.6 kN/m^3). D/t = 192. Wall thickness is 0.375 inch (9.4 mm). Because of the groundwater problem, this is a more conservative wall thickness than the pipe in the unsaturated example above. What is the critical vacuum? Figure 4 shows plots of critical vacuum. On the right is shown flood levels that are equivalent to critical vacuum.

See Specific Analyses for further discussion. It is noteworthy that: **If ring deflection is less than 5%, and the embedment soil is saturated, but compacted, the ring is stable under full atmospheric pressure.** Safety factors should apply to allowable pressure - not D/t.

EXAMPLE—

Suppose the pipe of Figure 4 is buried in well-compacted granular embedment for which ϕ= 35°. If ring deflection is found to bed = 5%, what is the flood level at collapse? From Figure 4, h = 54 ft (16.5 m), which is equivalent to critical vacuum of 23.4 psi (160 kPa). The effect of H is shown at the bottom of Figure 4.

4. Maximum Height of Cover:

If buried pipes are nearly circular (ring deflection less than 5%), maximum height of soil cover is found by equating ring compression stress to allowable strength. The result is Figure 5 which shows maximum allowable soil cover, H, as a function of D/t.

For example, if soil weighs 100 pcf (16 kN/m^3), what is the maximum allowable soil cover over a

pipe for which D/t = 288? From Figure 5, H = 175 ft (53 m). But this is based on soil unit weight of 120 pcf (18.8 kN/m3). With a weight correction factor of 120/100, H = 175(120/100) ft = 210 ft (64 m).

5. Minimum Height of Cover:

Minimum height of soil cover over a buried pipe may be of concern because of frost, flotation of the pipe in saturated soil, and ring deformation due to surface wheel loads. Frost depth is usually not critical unless slow flow in the pipe allows water in the pipe to freeze, or unless the pipe is on a slope within the soil creep zone wherein soil slips incrementally down the slope due to cycles of freezing and thawing, wetting and drying, earth
tremors, etc.

Flotation is caused by liquefaction of saturated embedment. Flotation is prevented by soil cover of at least half a pipe diameter of compacted soil above the pipe. The pipe is assumed to be empty (worst case). In order to avoid liquefaction, soil density should be greater than 85% standard (ASTM D698 or AASHTO T-99). Design engineers specify a minimum of 90% density.

Minimum cover under surface live loads is based on the wheel load, W, and D/t. Performance limit is localized inversion of the top of the pipe. Figure 6 shows minimum cover for two common wheel loads.

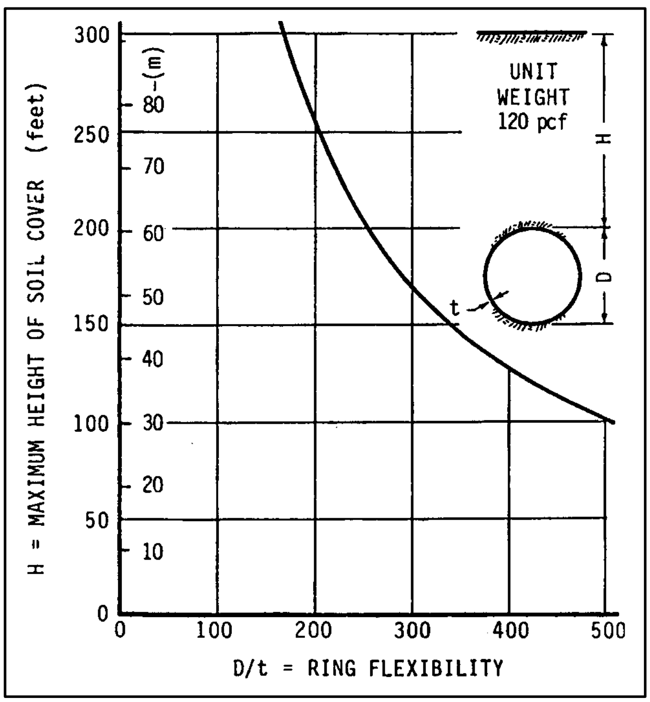


Figure 5. Maximum allowable height of soil cover over a plain pipe, based on ring compression stress of 21 ksi (145 kPa) and soil unit weight of 120 pcf (18.8 kN/m^3). Height of cover varies inversely as unit weight.

Unit weight can be adjusted for unit weights other than 120 pcf (18.8 kn/m^3) by inverse proportionality. For example, if unit weight is only 100 pcf (15.7 kN/m^3), the maximum height of soil cover is increased by a factor of 120/100.

It is assumed that ring deflection is less than d = 5%.

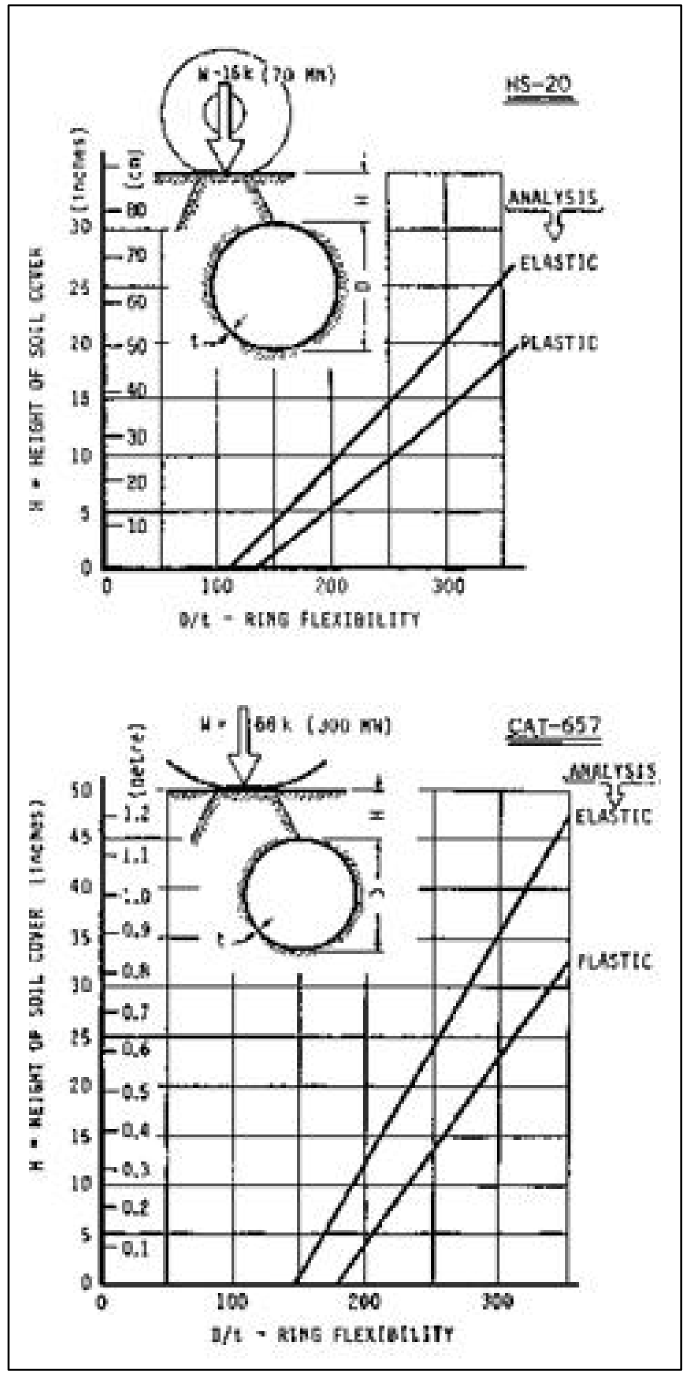


Figure 6. Minimum soil cover (with built-in safety factor) for pipes buried in compacted granular soil with wheel loads: W = 16 kips (70 MN) - HS-20 truck on unpaved surface [top]

W = 68 kips (300 MN) - Caterpillar 657 scraper [bottom]

EXAMPLE —

What is the minimum soil cover over a pipe for which D/t = 288 if a wheel load of 68 kips (300 kN) must pass over? The soil is compacted. No ruts are left by the wheel. From the plastic analysis of Figure 6, H = 21 inches (525 mm).

The ground surface is unpaved. As the load approaches (shown left of pipe centerline), the moment in the ring is maximum on the right. The required D/t is based on that moment. The elastic analysis graphs show soil cover at yield stress. But elastic theory is not inversion. The plastic analysis graphs, on the verge of plastic hinging, are more reasonable performance limits. For design, the upper graphs are sometimes used because of built-in safety factors. The contributions of longitudinal soil arching and pipe beam strength are disregarded. Some engineers consider a soil cover of three feet (0.9 m) to be minimum. From Figure 6 such a margin of safety for HS-20 truck loads is larger than necessary. However, the three ft (0.9 m) minimum may be desirable for other reasons - dynamic and repetitive loads, bedding settlement, etc.

6. Longitudinal Stress Analysis:

Longitudinal stresses in a buried pipe can be caused by: changes in temperature and pressure in the pipe, longitudinal beam action of the pipe, and "thrusters" -- valves, wyes, elbows, and reducers, that resist internal pressure or change direction of flow. A pipe descending a steep slope feels thrust due to gravity. Thrust restraints must be provided. For welded joints, the pipe itself provides thrust restraint. For pipes with gasketed joints, or slip couplings, external thrust restraints must be provided. External thrust restraints include thrust blocks, thrust pins, cross ties, and embedment soil. Longitudinal restraint is also developed by soil pressure on the pipe and the coefficient of friction of soil against pipe.

One worst case of longitudinal stress is a fixed-end pipe (external thrust restraints) subjected to large decrease in temperature and large internal pressure. In general, steel pipes are well suited to resisting longitudinal stress. However, slip couplings or gasketed joints at proper spacing along the pipeline can relieve the pipe of longitudinal stresses which must then be resisted by external thrust restraints. See Specific Analyses.

7. Ring Deflection:

Steel pipes are generally flexible enough that the ring conforms with the soil. Therefore,

Ring deflection of unpressurized pipes is less than, or equal to, the vertical compression of the sidefill soil.

Vertical compression of the sidefill soil is a conservative upper limit for ring deflection. Soil compression can be predicted from compression diagrams provided by soils laboratories.

8. Allowable Ring Deflection:

Because ring deflection can affect all of the analyses described above, it is prudent to control ring deflection by specification - either directly by a performance specification, or indirectly by procedural specifications that assure compaction of sidefill soil. Ring performance is not affected significantly by ring deflection less than 5%. In the 1940's Professor Spangler recommended maximum allowable ring deflection of five percent based on a safety factor of four and potential inversion at 20% ring deflection. Spangler's five percent recommendation is still considered to be maximum allowable, but for reasons other than ring inversion.

For plain pipes, design engineers limit maximum ring deflection to 5%. For pressure pipes, ring deflection greater than 5% may be mitigated because the pipe tends to reround when pressurized.

For typical mortar lined pipes, the recommended maximum allowable ring deflection is 3% based on incipient disbonding. Safety factor is 2. For mortar lined and mortar coated pipes, maximum allowable ring deflection is 2% based on 0.01 inch cracks in the coating. Safety factor is 1.5.

9. Backfill and Embedment Specifications:

The soil protects the pipe, supports the pipe, and maintains pipe alignment. The soil must not move. It should be in full contact with the pipe and dense enough to assure stability. Embedment should be placed in lifts and, if necessary, compacted to achieve required density.

Specific Analyses

The interactions of a buried pipe and its soil embedment are mutually complementary. The pipe is the liner for a soil conduit. The soil holds the pipe in shape and supports and stiffens it. The pipe is the form that retains a soil arch over the pipe. The soil arch protects the pipe and supports much of the load.

Pertinent Variables

There are many pertinent variables in the complex interaction of pipe and soil. See Figure 1 for a generalized cross section, and Table 1 for notation and nomenclature. One widely recognized variable is ring flexibility, D/t. It is an inverse form of ring stiffness, EI/D^3, and may be used in place of EI/D^3 for analyses involving ring stiffness of plain pipes (no linings, coatings, or stiffening rings). Another pervasive variable is ring deflection, $d = \Delta/D$, which depends primarily on vertical compression of the embedment soil. Ring deflection, d, is usually limited by specification in order to eliminate its effect on ring performance. Ring stiffness is EI/D^3 where I is the centroidal moment of inertia of the wall cross section per unit length of pipe. The procedure for evaluating I for pipes with mortar linings and coatings is discussed under, *The Effect of Mortar Linings and/or Coatings on Ring Stiffness.*

EXAMPLE —

Table 2 provides values of I and EI/D^3 for a mortar lined and coated steel pipe. It is assumed, conservatively, that there is no bond between mortar and steel. In fact, there is bond, but it is undependable. Disbonding may occur due to unknown extreme temperature changes and unpredictable loads. Modulus of elasticity of mortar is $4(10^6)$ psi (27.6 GPa). Subscripts $_S$, $_L$, and $_C$ represent respectively, steel, lining, and coating.

The most pertinent soil variables are soil strength at slip (friction angle, ϕ) and compressibility (soil modulus, E").

In Table 2, it is noteworthy that the contribution of steel to the ring stiffness is only seven percent for the standard thicknesses in AWWA C-205, and three percent for the increased mortar thickness. Clearly, little is gained by increasing steel thickness. On the other hand, by increasing mortar thicknesses by l/4 inch, ring stiffness is increased by a factor of 2.5. Increasing mortar thickness is a cost effective means of increasing ring stiffness. Including mortarsteel bond, ring stiffness is actually greater than the values shown in Table 2. The AWWA recommendations for mortar thicknesses are listed in Table 3.

Table 2. Ring Stiffness, EI/D^3 for a Mortar Lined and Coated Pipe
(based on a section transformed to its equivalent in steel)

D = 42 inches (1050 mm)
t_S = 0.175 inch (4.445 mm)
D/t = 240

AWWA C-205		**INCREASED MORTAR THICKNESS**
t_L = 0.50 inch (13 mm)		t_L = 0.75 inch (19 mm)
t_C = 0.75 inch (19 mm)		t_C = 1.00 inch (25 mm)
I_S = 0.0054 in^4/ft (7.32 mm^3)		0.0054 in^4/ft (7.32mm^3)
I_L = 0.0167 in^4/ft (22.76 mm^3)		0.05625 in^4/ft (76.81 mm^3)
I_C = 0.05625 in^4/ft (76.81 mm^3)		0.1333 in^4/ft (182.08 mm^3)
I = 0.078 in^4/ft (107 mm^3)		0.195 in^4/ft (266 mm^3)
EI/D^3 = 2.6 psi (18 kPa)		EI/D^3 = 6.6 psi (45 kPa)

Table 3. Recommended Thickness of Mortar Linings by AWWA C-205

D = NOMINAL PIPE DIAMETER		t_L = LINING THICKNESS	
inches	(mm)	inches	(mm)
4 - 10	(100 - 250)	1/4	(6)
11 - 23	(275 - 575)	5/16	(8)
24 - 36	(600 - 900)	3/8	(10)
> 36	(900)	1/2	(13)

Table 4. Ring Stiffness, EI/D^3, for a Range of Common Diameters — Based on AWWA C-205 Thicknesses of Mortar Linings

Steel Thickness = 0.175 inch (4.4 mm)

D = NOMINAL PIPE DIAMETER		D/t	t_L = LINING THICKNESS		EI/D^3 = RING STIFFNESS	
inch	(mm)		inch	(mm)	psi	(kPa)
42	(1050)	240	1/2	(12.7)	0.74	(5.1)
36	(900)	206	1/2	(12.7)	1.18	(8.1)
36	(900)	206	3/8	(9.5)	0.66	(4.6)
23	(575)	131	5/16	(7.9)	1.95	(13.4)
10	(250)	57	1/4	(6.4)	18.60	(128)

The effect of pipe diameter on ring stiffness is shown in Table 4 which lists values of ring stiffness for a range of pipe diameters. For lined pipes smaller than 24 inch (600 mm), ring stiffness is great enough that instability and damage during handling and installing are not critical. The pipes are comparatively rigid. For pipes larger than 36 inches (900 mm), ring stiffness is less than 1.0 psi (7 kPa). The pipes are comparatively flexible. Under adverse circumstances, it may be advisable to either increase mortar thicknesses, or increase care in handling. Stulls are often placed in large pipes with low ring stiffness to prevent handling damage and to hold circular shape during handling and installing.

Performance Limits

Performance limits are excessive deformations of the pipe and soil movement. Movement of the soil includes soil settlement, washout of soil from around the pipe, liquefaction, and soil slip next to the pipe. Excessive pipe deformation includes fracture. Excessive deformations occur beyond yield where stress loses relevancy. The value of stress analysis is a universal familiarity with the stress theory. The pipe must be able to resist loads due to handling/installing. The pipe-soil conduit must resist internal pressure and external pressure.

Design for Internal Pressure

When designing pipes for internal pressure, it is conservative to neglect external soil restraint. Circumferential stress (hoop stress) in the pipe wall is found by the Barlow formula, = PD/2t, from which minimum pipe wall thickness is: t = PD/2S. Diameter, D, is inside diameter, but for thin-wall steel pipes, the differences in mean diameter, nominal diameter, and inside diameter are usually negligible. Design strength, S, is specified to be one-half of yield strength - typically S = 21 ksi (145 MPa). For penstocks, S is one-half yield or one-third ultimate. High strength steels are available. Longitudinal stress due to internal pressure is less than half the circumferential stress. See *Longitudinal* Analysis.

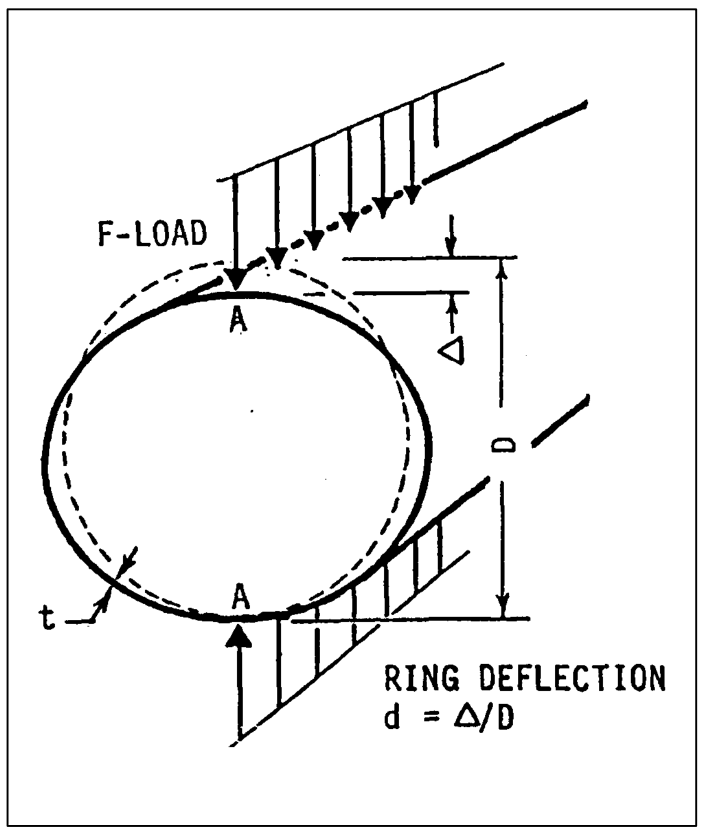


Figure 7. F-load on a pipe during handling and installation.

Handling

Handling forces on a pipe are generally diametral F-loads. See Figure 7. Point loads are analyzed, conservatively, as line loads. From Castigliano's equation for deflection, and the equations of equilibrium, the moment in the ring at A, is,

$$M_A = Fr/\pi \qquad (6)$$

Castigliano's equation is found in texts on mechanics of solids. Circumferential stress on the surface of plain pipe is $\sigma = 6M/t^2 = 6Fr/\pi t^2$ at point A. Strain (below yield) is $^a = \sigma/E$.

$$\in = 6Fr/\pi Et^2 \qquad (7)$$

A very conservative performance limit is yield stress. Beyond yield, surface strain in the steel pipe might cause disbonding of mortar lining and coating. The strain at yield is, $\in_y = 1.4(10^{-3})$ where yield stress is S_y = 42 000 psi (290 MPa) and E=30,000,000 psi (207GPa). Yield stress in plain pipes during handling is not necessarily a performance limit.

F-load at Yield Stress:
See Figure 7. From Equation 6, the moment at A is $M_A = Fr/\pi$. Moment resistance at yield stress, from theory of elasticity, is $M_A = 2S_y l/t$. Equating and solving, $FD = 4\pi S_y l/t$. For plain pipe,

$$F = \pi S_y D/3(D/t)^2 \qquad (8)$$

EXAMPLE —
What is the F-load on plain pipe at yield stress? D = 72 inches (1800 mm), t = 0.300 inch (7.6 mm), (D/t) = 240, and S_y = 42 ksi (290 MPa). Yield stress on the surface of the steel is a conservative performance limit because permanent deformation has not started, and, even if it had, yielding does not decrease strength or reduce service life. Substituting values into Equation 8, the F-load at yield stress is *F_e = 376 lb/ft (5.49 kN/m).* At the start of plastic hinging, *F_p= 564lb/ft (7.97 kN/m)*. The plastic moment is 1.5 times the elastic moment at yield stress. The F-load is avoided by using cradles, stulls, etc., during handling and transporting.

Ring Deflection, d, at Yield Stress Due to F-load:
One way to find out if yield stress is exceeded by an F-load is to measure the ring deflection between points A. See Figure 7. From mechanics of solids, $d = 0.0186FD^2/EI$. At points A, $FD = 4\pi S_y l/t$ Substituting,

$$d = 0.234(S_y/E)(D/t) = 0.234\in_y (D/t) \qquad (9)$$

EXAMPLE —
A plain pipe is F-loaded. See Figure 7. What is the ring deflection when stresses at points A reach yield? D = 72 inches (1800 mm), t = 0.300 inch (7.62 mm), (D/t)= 240, and Sy= 42 ksi (290 MPa) at $\in_y = 1.4(10^{-3})$. Substituting into Equation 9, *d = 7.9%.* F-load analyses apply to handling. F-load analyses do not apply to pipes after they are buried.

Soil Mechanics

The soil in which a pipe is buried is a major component of the conduit. Soil applies pressure on the pipe, but it also supports the pipe and supports much of the load. The pertinent variables of soil are: stress, strength, unit weight, and vertical soil compressibility. Compressibility affects ring deflection.

Soil Stresses:

The basic soil stress is P´ acting vertically on top of the pipe. It includes weight of soil, external hydrostatic pressure, and the effect of live surface loads. Soil support is based on soil strength which is affected by groundwater. Vertical compression of the sidefill soil causes ring deflection. Soil compression is caused by intergranular (effective) soil pressure, P' which is total pressure, P´, minus hydrostatic pressure, u;

$$\bar{P}' = P' - u = P' - h\gamma_w \qquad (10)$$

Pressure on the pipe is,

$$P' = P_d + P_l \qquad (11)$$

Where

P_d = dead load pressure due to weight of the soil

γ = unit weight of the soil,

γ_w = unit weight of water,

H = height of the soil cover,

h = height of the water table above the pipe,

W = live load on the ground surface over the pipe (wheel load),

u = hydrostatic pressure = $h\gamma_w$,

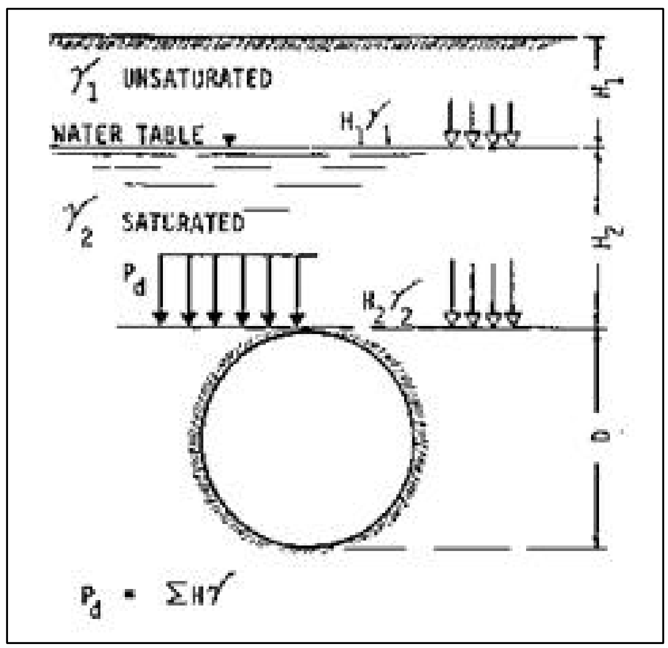


Figure 8. Dead load pressure, P_d, at the top of the pipe.

P_d = dead load pressure = pressure of soil above the pipe (See Figure 8),

P_l = live load pressure = the effect of surface loads on the pipe (See Figure 9)

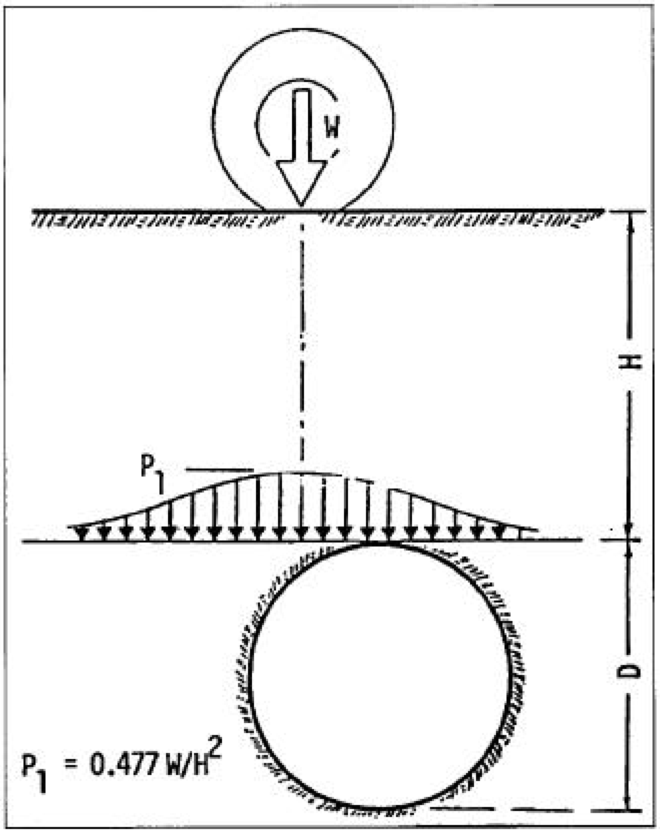


Figure 9. Live load pressure, P_l, at the top of the pipe due to a concentrated surface load, W (after Boussinesq).

According to Boussinesq, pressure, P_l, directly below surface load, W, is,

$$P_l = 0.477W/H^2 \qquad (12)$$

Graphs are available for evaluating P′ See the AISI graphs, Figure 10.

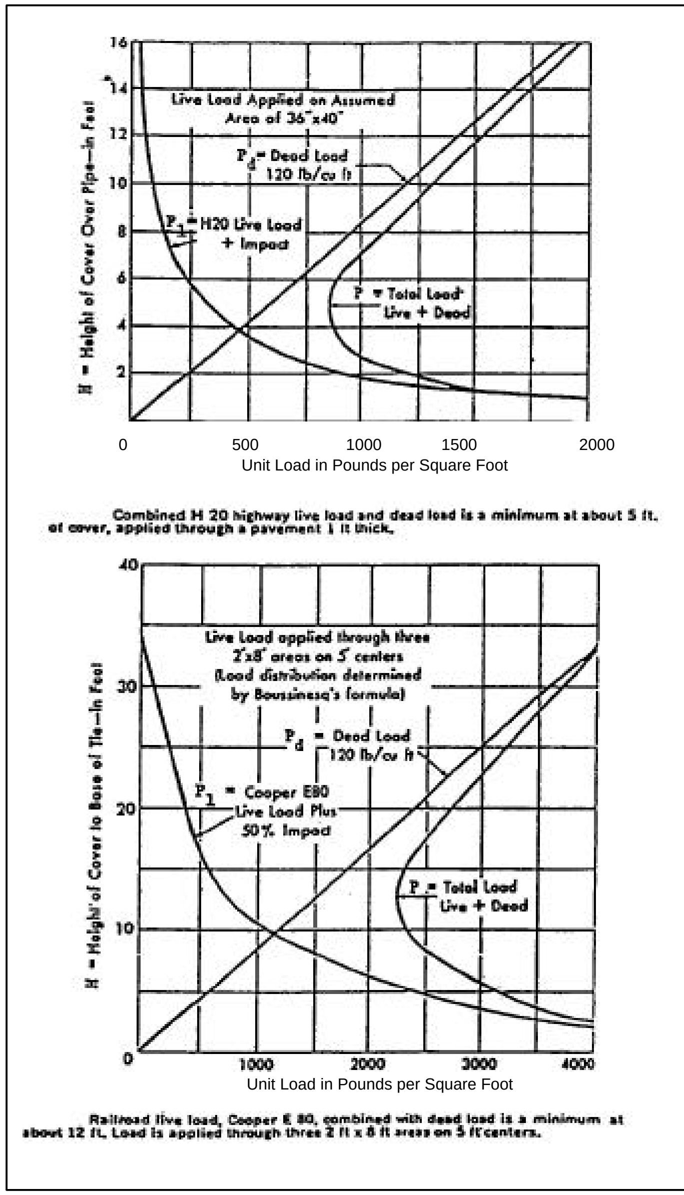


Figure 10. Graphs for vertical soil pressure at the tops of buried pipes, as published by the American Iron and Steel Institute.

EXAMPLE –

What is the pressure on top of a pipe if a 68 kip (300 kN) wheel load passes over? Height of cover is H = 8 ft (2.44 m). The water table is h = 5 ft (1.52 m) above the pipe. Unit weights are: 100 pcf (15.7 kN/ms^3) above the water table, and 125 pcf (19.6 kN/m^3) below the water table. Dead load pressure at the top of the pipe is Pd = (3ft)(l00pcf) + (5ft)(l25pcf) =925 lb/ft^2 (44.3 kPa). Live load pressure due to the 68 kip wheel load is, from Equation 12, P_l = 507 lb/ft^2 (24.3 kPa). The sum is P′ = 1.4 *kips/ft*^{2} (69 *kPa).* Effective pressure is P′ =P′ - P_u =1,432 - (62.4)5 = 1.1 $kips/ft^2$ (54 kPa).

Soil Strength: The primary soil performance limit is soil slip. Resistance to slip is soil strength which comprises cohesion and friction. Rarely is there any significant cohesion in the embedment. Under adverse circumstances, soil cement may be used to fill voids under pipe haunches. But most pipes are embedded in granular soil for which cohesion is negligible. Soil strength is the ratio of maximum to minimum principal stresses at soil slip. See the soil cube in Figure 11. If minimum vertical stress is y, maximum horizontal stress at slip is $\sigma_x = K\sigma_y$, where K = (1 +sinϕ)/(l-sinϕ). Friction angle, ϕ, is a function of soil compaction. For granular soil with some compaction, the friction angle is often assumed to be ϕ = 30° from which K = 3. The soil slips if the ratio of maximum to minimum principal stresses exceeds *3.* If ϕ = 35º, K = 3.69. If ϕ = 250, K = 2.46.

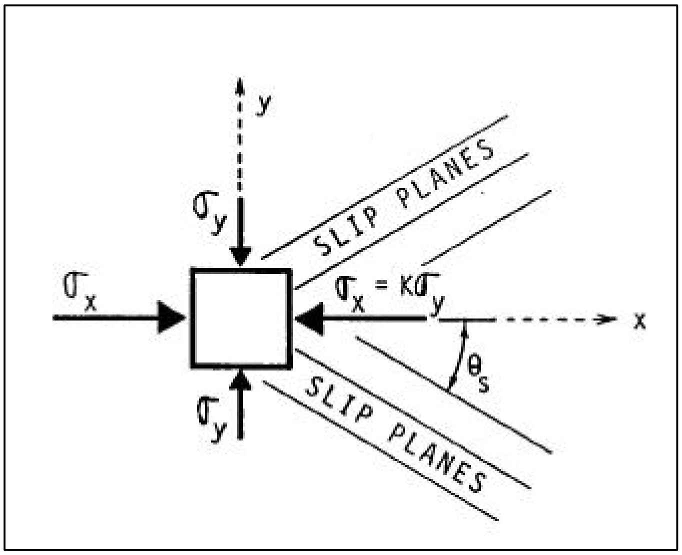


Figure 11. Infinitesimal soil cube showing the ratio of the maximum stress σ_X (passive) to minimum stress σ_Y (active) at soil slip. Slip planes form at angles: è$_S$ = 45°-ϕ/2.

Table 5. Approximate Friction Angles for Granular Soil

COMPACTION	ϕ = FRICTION ANGLE
Heavy (dense)	35º to 40º
Light	25º to 35º
None (loose)	15º to 25º

Soil Friction Angle: Precise values of soil friction angle, ϕ, can be obtained from soils testing laboratories. However, for granular sidefill soil, Table 5 is adequate on the conservative side for most buried pipe design and analysis, and is used in the examples in this text. Figure 12 shows approximate values of the soil friction angle as a function of density for the basic granular soil types. The values are low, but reasonable, working values. Conservatism is justified because the soil around buried pipes is typically non-uniform.

Slip planes develop at angle θ_s = 45º - ϕ/2 as shown in Figure 11. *Passive* soil resistance, σ_x, is the maximum support that the soil can provide at the sides of a buried pipe. If the soil cube is rotated 90º, the horizontal stress becomes the minimum (active) soil stress. If the sides of a pipe are unable to resist active soil stress, the pipe will collapse inwardly from the sides. Soil slip is the basis for analysis due to heavy surface wheel loads over buried flexible pipes with minimum soil cover.

Soil Compression: A secondary performance limit is excessive soil compression. Vertical compression of sidefill soil results in ring deflection of flexible pipes. Soil compression can be predicted from stress-strain diagrams from soils laboratories.

Soil Density: Soil density is a common expression for unit weights of soil. It is the basis for specifying degree of compaction of embedment. Because of its broad use, Standard Proctor density is the basis for density in this text. Soil density is the dry unit weight of soil as a percent of the unit weight of the same soil compacted to 100% density according to test method ASTM 0698 or AASHTO T-99.

Critical Density: Critical soil density is that density greater than which the soil skeleton increases in

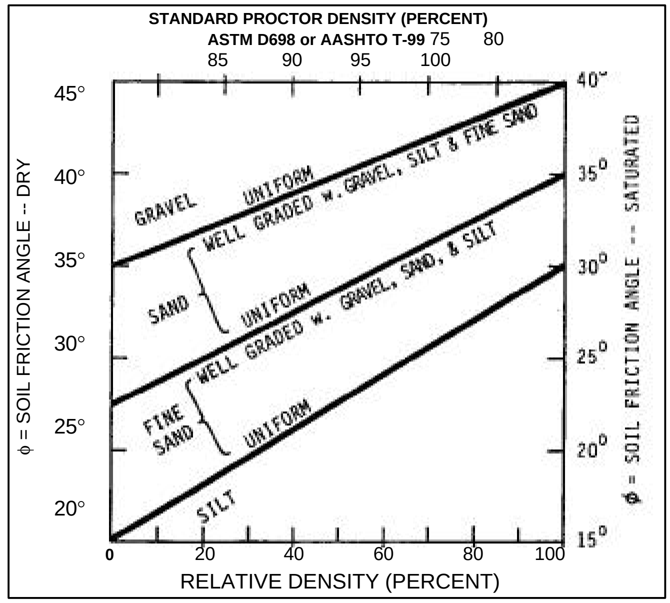


Figure 12. Conservative working values (lower limits) of soil friction angle, ϕ, for granular embedment — natural (not crushed).

The values on the graphs are reduced from peak lab values to more typical field conditions that include nonuniform

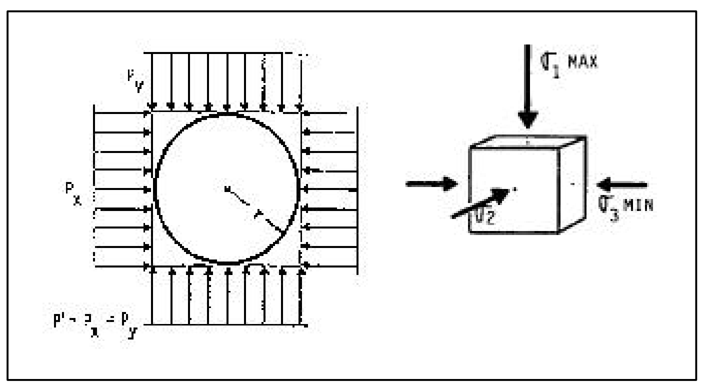


Figure 13. Flexible ring in equilibrium, subjected to uniform, external, radial pressure, P′, (left), and showing principal stresses on an infinitesimal cube at spring line (right).

volume when disturbed (shaken up) by sudden earth movements such as earth tremors. At less than critical density, soil decreases in volume (shaken down) when disturbed; and, if saturated, non-compressible water must support the load. The mass liquefies (becomes mud) and the soil particles in suspension increase the unit weight of the liquid.

As long as granular soil density is greater than about 85% Standard Proctor, it is above critical density and does not liquefy. Because critical density varies depending upon soil types, permeability, degree of confinement of the soil mass, and earth tremors (period and amplitude); 90% Standard Proctor density (ASTM D898 or AASHTO T-99) is usually considered to be the specified minimum.

Pipe Mechanics

A flexible circular ring is in equilibrium when subjected to uniform external pressure. See Figure 13. It is seldom necessary to combine or compound stresses. Shearing stresses are avoided by using principal stresses, σ_1 and σ_3 which are usually determinable. Intermediate stress, σ_2, at right angles to σ_1 and σ_3, is not critical. Principal stresses are analyzed for each load. Stresses in the same direction are not always combined. For example, σ_1 is the sum of ring compression stress and ring flexure stress. But, performance limits are different for ring compression and ring flexure. Therefore each is analyzed separately and compared to its own performance limit. For ring compression, performance limit is yield stress. For ring flexure, performance limit is excessive out-of-roundness.

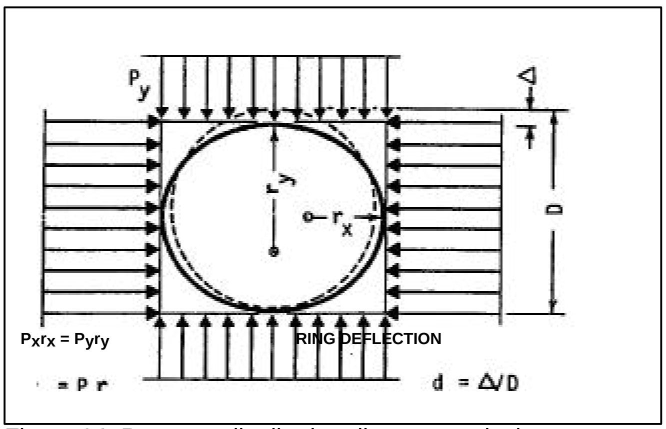


Figure 14. Pressure distribution diagram equivalent to external radial pressures at equilibrium on an elliptically deflected flexible ring.

When buried, the basic out-of-round shape of the ring is an ellipse. See Figure 14. Other shapes may be superimposed on the ellipse. Loose soil under the haunches causes jowls — a Δ-shape superimposed on the ellipse. Heavy compaction on the sides tends to square the ellipse. But for most analyses, basic deformation is an ellipse, for which:

1. $r_y = r(1+d)^2/(l-d)$ (13)
2. $r_x = r(1-d)^2/(1+d)$ (14)
3. The ratio of radii is $r_r = r_y/r_x = (1+d)^3/(1-d)^3$ (15)
4. The cross sectional area of the ellipse is $A_e = \pi r^2(1-d^2)$. If ring deflection were d = 5%, the reduction in cross sectional area of the pipe would be only 0.25%.
5. Soil pressures against the pipe are perpendicular to the pipe surface. It is conservative to neglect shearing stresses between soil and pipe. Shearing stresses break down due to changes in temperature, rise and fall of the water table, earth tremors, etc.

A flexible ring is in equilibrium if circumferential force in the ring is constant; i.e. P'r is constant all around the ring. If the ring is deflected out-of-round,

$$P_x r_x = P_y r_y \quad (16)$$

EXAMPLE –

A buried flexible pipe is deflected into an ellipse during backfilling. The measured ring deflection is 3%. The diameter is D = 72 inches (1800 mm), and soil cover is H = 4 ft (1.2 m) at 120 pcf (19 kN/m^3). What is the pressure of pipe against sidefill at the spring lines? Pressure on top of the pipe is $P' = P_y$ = (4ft)(l20 pcf) = 480 psf (23.0 kPa). From Equation 15, the ratio of radii is r_y/r_x =1.2. From Equation 16, P_x = 576 *psf* (27.6 Pa). The sidefill soil only needs to resist 576 psf (27.6 kPa). If ring deflection were 10%, the sidefill would have to resist P_x = 876 psf (41.9 kPa) — easily resisted by granular sidefill. Because the required K is 876/480, the minimum soil friction angle is $\phi = 17°$.

Pipe mechanics and soil mechanics are combined into pipe-soil interaction in the following.

External Pressures and Loads

1. Ring Compression Stress

Ring compression stress, σ, is circumferential stress in the ring; i.e., $\sigma = P'r(1+d)/t$. P' is pressure on top of the pipe. Performance limit is yield. If ring deflection, d, is not greater than five percent, it is negligible. For allowable stress, S, the minimum wall thickness is,

$$t = P'r/S \quad (17)$$

S includes a safety factor. When yield stress is exceeded, the ring is on the verge of localized wall crushing. The pipe does not collapse, but yield stress is considered to be performance limit because the pipe is stressed to its limit. The soil must provide the margin of safety in the event of an unanticipated added load. Internal pressure reduces ring compression, but is neglected because there may be occasions when there is no internal pressure.

EXAMPLE —

A 72 inch (1800 mm) pipe with 0.245 inch (6.22 mm) wall thickness is buried under H = 4 ft (1.2 m) of soil. A highway fill is to be placed over the pipe. Allowable stress in the pipe is S = 21 ksi (145 MPa). What is the maximum allowable height of fill over the pipe at unit weight γ = 110 pcf (15.7 kN/m^3)? Substituting $P' = \gamma H$ into Equation 17, and solving for maximum height of cover, *H = 157 feet (48m)*

2. Ring Deflection

See Figure 15. Because the ring is flexible, ring

deflection is nearly equal to the vertical compression of the sidefill soil; i.e.

$$d = \in \tag{18}$$

where $\in$ is the vertical soil strain predicted from laboratory compression tests. See Figure 15. The flexible ring is not deflected more than the sidefill soil is compressed.

A stiff ring resists compression of the soil. Figure 16 shows the ring deflection term, $d/\in$, as a function of stiffness ratio, $E''/(EI/r^3)$; where E'' is the soil stiffness, and (EI/D^3) is the ring stiffness. The stiffer the ring, the smaller the ring deflection term.

EXAMPLE —

A 72 inch (1800 mm) steel pipe with 0.245 inch (6.23 mm) wall is embedded in granular soil at 80% density (ASTM D898 or AASHTO T-99), with soil cover H = 4 ft (1.22 m). This case may require mitigation because soil density less than 90% is not usually specified. However, in this case, during installation, ring deflection was controlled — essentially zero — so no problem. But now it is proposed to increase soil cover to a height of H′= 30 ft (9.14 m). Soil unit weight is 120 pcf (19 kN/m^3). What is the predicted ring deflection of the pipe? Initially, P′ = 480 psf (23 kPa). The final P′ = 3600 psf (172 kPa). From Figure 15, soil strain from 480 psf to 3600 psf on the 80% density graph is $\in$ = (3.15%-0.4%) = 2.75%.

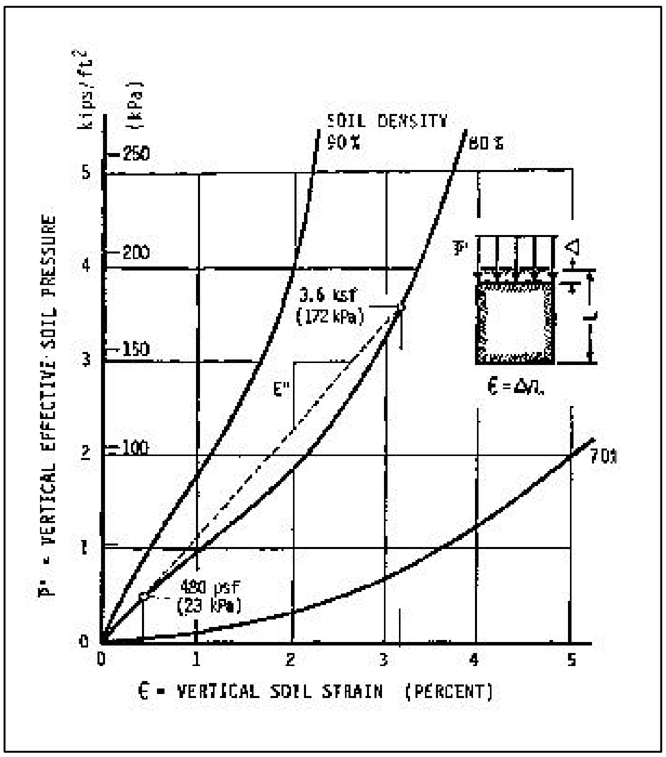


Figure 15. Examples of stress-strain graphs from compression tests on granular soil at three densities (ASTM D698 or AASHTO T-99) showing the soil modulus E″ = 788psi (543MPa) for a soil pressure increase from

E″ is the slope of the secant from 480 to 3600. E″ = 788 psi (5.43 MPa). Ring stiffness is EI/r^3 = 0.79 psi (5.4 kPa). Stiffness ratio is $E''/(EI/r^3)$ = 1,000. The ring deflection term, $d/\in$, is found by entering Figure 16 with stiffness ratio, 1.000, which falls off the chart. Even with a standard AWWA lining (ring stiffness = 1.68 psi) the stiffness ratio, EI/r^3 = 469, falls off the chart. The lined pipe is flexible, $d/\in$ = 1, and $d = \in$ = 2.75%. Allowable ring deflection for lined pipe is 3% based on a safety factor of two against disbonding. Is disbonding a performance limit? Probably not.

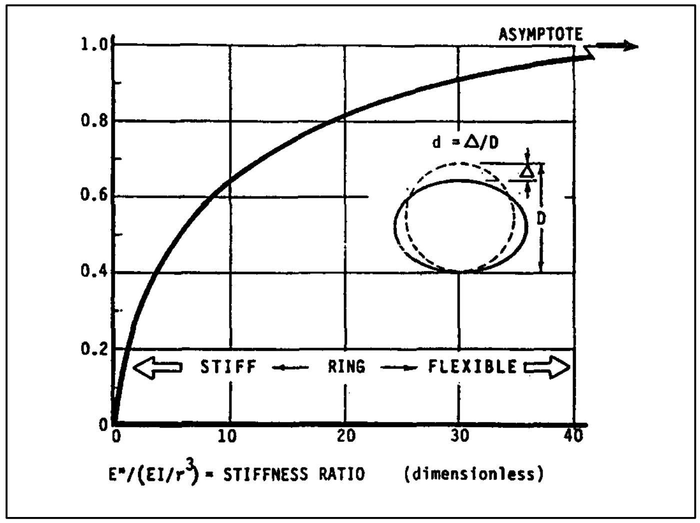


Figure 18. Graph of ring deflection term as a function of stiffness ratio from tests. Ninety percent of all data falls below the graph.

What would happen if ring deflection should encroach slightly into the margin of safety of the 3% allowable ring deflection? The answer is, “nothing.”

Ring deflection of buried steel pipes is less than or equal to the vertical compression of the sidefill soil. d = $\in$

This rule is generally true for steel pipes. In order for the stiffness ratio of a 72 inch (1800 mm) plain pipe to fall on the chart of Figure 15 - say $E''/(EI/r^3)$ = 40 the wall thickness would have to be 0.75 inch (19 mm), for which ring flexibility is D/t = 96. So, if D/t is less than 100, a ring might be considered less than flexible. In very poor embedment, soil stiffness might be so low that the stiffness ratio is less than 40. Figure 18 is available for such conditions.

Because ring deflection affects so many aspects of performance, maximum ring deflection is specified — typically, 5% for plain pipes (no lining or coating or ring stiffeners), 3% for mortar lined pipes and 2% for mortar lined and coated pipes. Constructors comply by compacting the sidefill soil or by using select embedment which, without compaction, compresses less than the allowable ring deflection. Under adverse circumstances, stulls may be used to hold the pipe circular during backfilling. Soil cement

is sometimes used to assure support under the haunches so that the pipe does not tend to flatten on the bottom.

Ring Stability

1. Without Soil Support:

The classical equation for collapse of a ring subjected to uniform external pressure, p, is $pr^3(1-\nu^2)/EI = 3$. For steel, Poisson ratio is $\nu = 0.3$. When squared, it is small. Its effect is less for a buried pipe than for a ring. It is conservative to neglect ν; i.e.,

$$p/(EI/r^3) = 3 \quad (18)$$

For a circular plain pipe (no lining, coating, or stiffener rings), $I = t^3/12$, and Equation 18 reduces to,

$$(p/E)(D/t)^3 = 2 \quad (19)$$

where p is external collapse pressure. This occurs only when an empty pipe is immersed in water or grout, or when a vacuum occurs inside an unburied pipe, or when the pipe is buried in soil that liquefies. In fact, from tests, liquefied embedment (mud) still provides enough soil strength that collapse pressure, P′, is about twice as great as the predicted values from theoretical Equations 18 and 19.

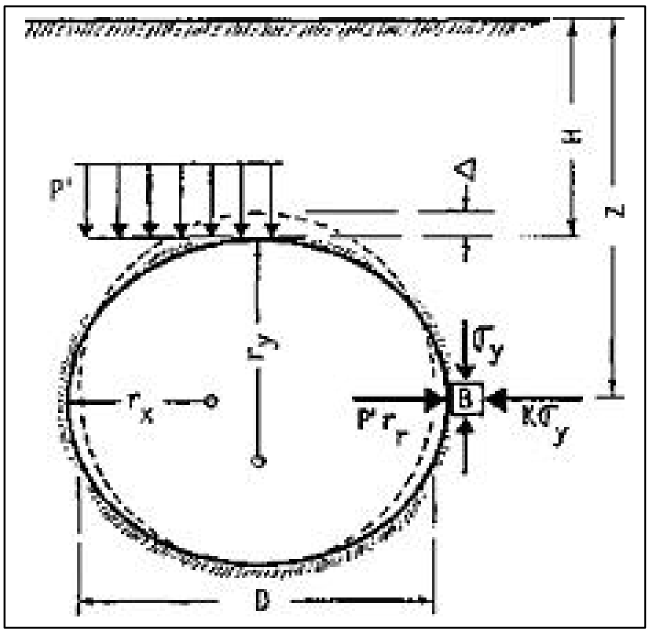


Figure 17. Buried flexible ring showing stresses on an infinitesimal cube B at soil slip. $r_r = r_x/r_y$.

EXAMPLE —

What is the maximum D/t for a plain pipe subjected to atmospheric pressure (vacuum) of P′= 15 psi (103 kPa)? E= *30,000,000* psi (207 GPa).
From Equation 19, *D/t = 758.*

The contribution of mortar lining and coating to increased critical pressure is significant. Table 2 shows ring stiffness as a result of mortar linings and coatings.

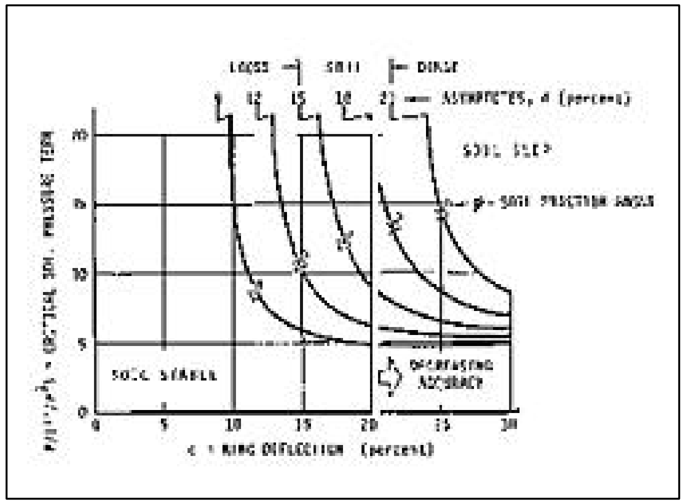


Figure 18. Soil pressure term at soil slip as a function of ring deflection, d, in sidefill soil compacted to soil friction angle ϕ. Sidefill is granular. Water table is below the pipe.

2. With Soil Support and No Water Table or internal Vacuum:

Performance limit is soil slip at the spring lines. Figure 17 shows an infinitesimal cube of soil, B, at the spring line. At soil slip, the pressure of the pipe against the soil is equal to passive soil resistance;
i.e.

$$P'r_r = K\sigma_y \quad (20)$$

P′ is soil pressure at the top of the pipe and $r_r = (1+d)^3/(1-d)^3$. From Equation 20, ring deflection, d, can be found at soil slip.

EXAMPLE —

What is the ring deflection at sidefill soil slip if D= 72 inches (1800 mm) and H = 4 ft (1.2 m)? Embedment is poor, granular, loose soil. Unit weight is γ = 100 pcf (15.7 kN/m^3). The soil friction angle is assumed to be $\phi = 15^\circ$ from which, $K = (1+\sin\phi)/(1-\sin\phi) = 1.7$. $P' = \gamma H$ = 400 psf (19.15 kPa). $\sigma_y = \gamma Z$ = (100 pcf)(4ft+3ft) = 700 psf (33.5 kPa). For the first trial, let Z = 7 ft (2.13 m). Substituting these values into Equation 20, $P'r_r = (400 \text{lb/ft}^2)(1+d)^3/(1-d)^3 = 1189$ lb/ft^2 (56.93 kPa). Solving, d = 18%. For a second trial, let Z = 4ft+2.5ft = 6.5 ft (2.0 m) to account for the 18% reduction in vertical diameter of the deflected ring. This solution yields d =17%. From an old, dubious, assumption, the ring inverts at d = 20%.

The above analysis is conservative because ring stiffness is ignored. Ring stiffness is included in Figure 18 which shows pressure at soil slip as a function of ring deflection and sidefill soil friction angle. It is assumed that soil cover is high enough that H is essentially equal to Z. Two important conclusions follow. Compaction of the embedment has a significant effect on pressure, P′, at soil slip.

Soil does not slip if ring deflection is less than five percent. Therefore, maximum allowable ring deflection is limited by specification, to five percent - or less if other performance limits prevail.

EXAMPLE —

If height of cover is H = 12 ft (3.7 m), what is the ring deflection at soil slip? D = 72 inches (1800 mm) and t = 0.245 in (6.23 mm). The embedment is loose soil for which γ = 100 pcf (16 kN/m^3). The friction angle is assumed to be $\phi = 15°$ because the soil is of poor quality. EI/r^3 = 0.78 psi (5.4 kPa). P = 1200 psf = 8.3 psi (57 kPa). The soil pressure term is $P/(EI/r^3)$ = 10.7. From Figure 18, *d = 11%.* No problem is anticipated if ring deflection is less than five percent — even in poor soil. The safety factor is greater than two.

3. With Soil Support and Internal Vacuum - Unsaturated Soil:

The performance limit for internal vacuum and/or external soil pressure is inversion. Critical vacuum, p, is sensitive to radius of curvature. Ring deflection reduces critical vacuum because the vertical radius, r_y, is greater than r. Therefore, ring stiffness, EI/r_y^3, is less than EI/r^3, and the vacuum at collapse is less for a deflected ring than for a circular ring.

The stability analysis can include internal vacuum, p, and the resistance of ring stiffness which, for plain pipes, is Ed/m^3. The horizontal stresses on the

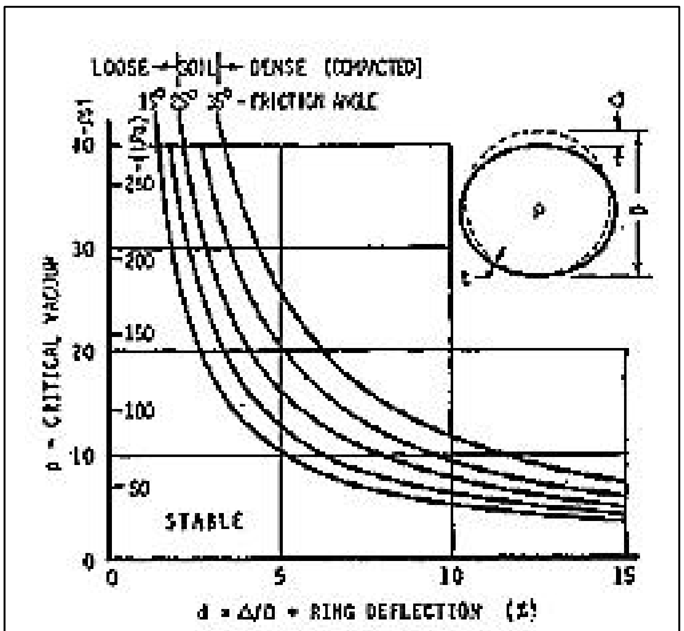


Figure 19. Example of critical vacuum for a 48 inch (1.22m), thin-wall steel pipe for which D/t = 288, buried in granular embedment with two feet of cover and no water table — soil is ***UNSATURATED,*** and has unit weight of 100 pcf (15.7 kN/m^3).

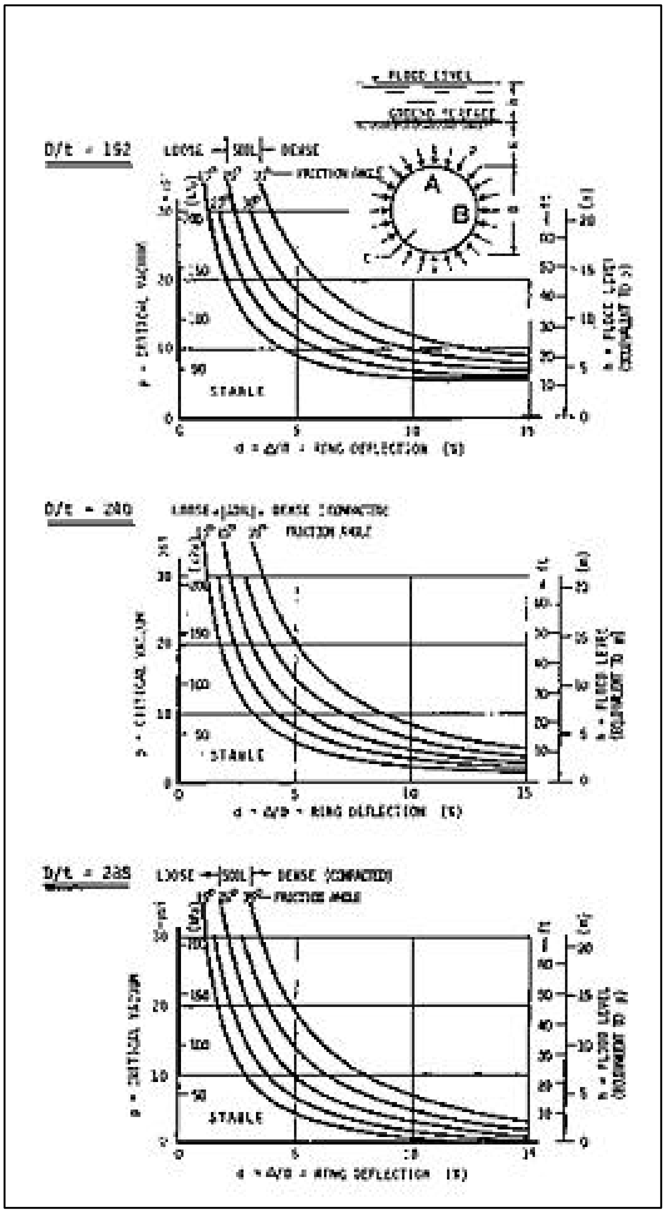


Figure 20. Examples of critical vacuum as a function of ring deflection and soil friction angle — in ***SATURATED*** soil at 125 pcf (19.8 kN/m^3).
D = 51 inches (1275 mm), H = 3 ft (0.9 m)

infinitesimal cube, 6, of Figure 17 can be equated at soil slip. Solving for vacuum, p, at soil slip,

$$p(r_r - 1) = K\sigma_y - (P_A - Ed/m^3)r_r$$

UNSATURATED SOIL (21)

For notation, see the more general form, Equation 22. Figure 19 is an example showing graphs of Equation 21 for a plain pipe with D/t = 288 in granular embedment. It is noteworthy that critical vacuum increases significantly by limiting ring deflection and by compacting the embedment (increased soil friction angle, ϕ). The effect of soil unit weight on critical

vacuum is small. The effect on critical vacuum of decreasing D/t is small.

EXAMPLE —

A plain pipe is 51 inches (1275 mm) in diameter with 0.187 inch (4.75 mm) wall thickness. D/t = 273. The soil cover is 2 ft (0.61 m). The soil is silty sand (SM) with soil friction angle $\phi = 25^o$ (light compaction), and unit weight of about 100 pcf (16 kN/m^3). If the buried ring deflection is discovered to be d = 5%, what is the internal vacuum at soil slip? Because D/t is close to 288, try using Figure 19. p = 16 *psi (110 kPa).* The pipe can resist a vacuum of full atmospheric pressure. Using Equation 21 with the correct value of D/t = 273, critical vacuum is still p = 16 psi (110 kPa). Clearly the effect of changes in D/t is small for flexible pipes. Embedment is the primary structural element. Had the soil been compacted such that $\phi = 35^o$, all else unchanged, the pipe could have withstood a vacuum of *p = 26 psi (189 kPa).* It is noteworthy that:

a) The most significant variables are ring deflection and soil density.

b) The effect of D/t on critical vacuum is minor for values of D/t greater than about 240.

c) Critical vacuum is increased slightly by increasing soil cover H. See Figure 3.

For the design of pipes to withstand internal vacuum, a safety factor of 1.5 is recommended. It is prudent to require that embedment soil be denser than critical. Even without a water table, percolating water and earth tremors tend to shake the soil down such that ring deflection could increase and reduce critical vacuum.

4. With Soil Support and With Water Table Above the Pipe Saturated Soil:

If the water table is above the top of the pipe, the soil will not liquefy if density of the embedment is 90% Standard Proctor (ASTM D698 or AASHTO T-99). The height of water table, h, above ground surface, must be added to the internal vacuum. The worst case is an empty pipe with the water table above ground surface (flood level). See Figure 20. Using the stability analysis of Figure 17, but including ring stiffness and vacuum and water table, the equation of stability is,

$$p(r_r - 1) = K\sigma_y + u_B - (P_A + \pi r\gamma_w/2 - Ed/m^3)r_r$$

SATURATED SOIL (22)

where:

P = vacuum and/or pressure of flood level h,

σ_y = effective vertical soil stress at B,

P_A = total vertical pressure at the top of the pipe at A,

K = $(1+\sin\phi)/(1-\sin\phi)$,

ϕ = soil friction angle,

U_B = water pressure at B = $(h + H + r)\,\gamma_w$,

h = height of water table above the ground surface,

γ_w = unit weight of water = 62.4 pcf (9.8 kN/m^3),

E = modulus of elasticity of steel = 30,000,000 psi (207 GPa),

d = ring deflection (ellipse assumed) = Δ/D,

D = circular diameter of the pipe,

m = r/t = ring flexibility,

r = D/2 = radius of the circular pipe,

t = wall thickness,

r_r = r_y/r_x

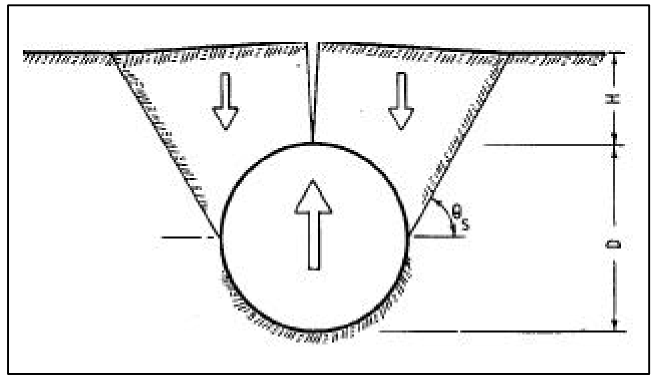


Figure 21. Flotation of a pipe, showing the reaction of soil wedges to the buoyant uplift force of an empty pipe buried below the water table under less than minimum cover.

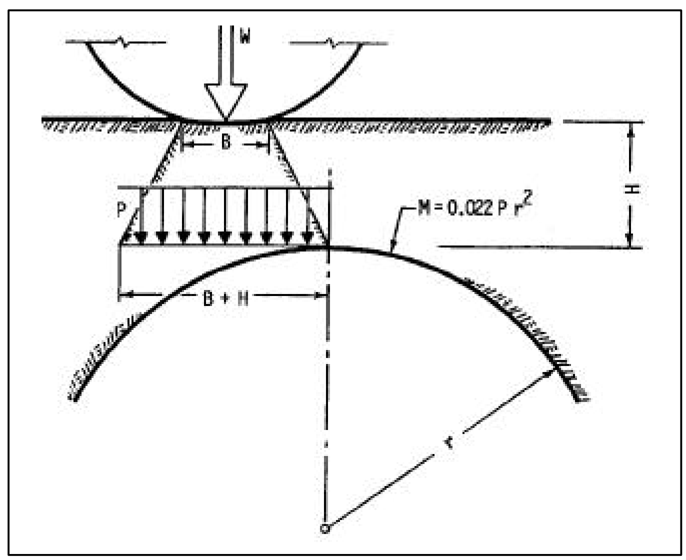


Figure 22. Conditions for inversion of the top of a pipe due to a heavy wheel load and minimum cover, showing the truncated pyramid punched through to the pipe, and showing the maximum moment.

The term, $(\pi r\gamma_w/2)$, is uplift pressure equivalent to buoyancy of the empty pipe. If the pipe is full of water, this term is eliminated from Equation 22.

San Diego, CA 96" I.D. Pipe

Noteworthy from Figure 20:

a) A water table reduces the critical vacuum.

b) The most significant variables are ring deflection and soil density.

c) The effect of D/t on critical vacuum is minor for values of D/t greater than about 240.

d) Increase in soil cover H increases the critical vacuum slightly. See Figure 4.

EXAMPLE —

A plain pipe of diameter D = 51 inches (1275 mm) and wall thickness t = 0.187 inch (4.75 mm) is buried under a soil cover of H = 4 ft (1.22 m). The embedment is poor, loose granular soil with saturated unit weight of 125 pcf (19.6 kN/m^3). Assume ϕ = 15º. The water table is at ground surface. Ring deflection is 5%. What is the internal vacuum at ring collapse? Substituting values into Equation 22, the critical vacuum is *p = 3.8 psi (22 kPa).* If the vacuum were due to a flood, the pipe could sustain a flood level of 8.8 ft (2.7 m). If embedment had been compacted such that saturated unit weight were 130 pcf (20.4 kN/m^3) and ϕ = 35º, critical vacuum would have been *p = 4.6 psi* (31.7 kPa) and the pipe could sustain a flood level up to 10.6 feet (3.2 m) above the ground surface. A safety factor is recommended if floods are anticipated — maybe an allowable flood limit of 5 ft (1.5 m).

Flotation

An empty pipe will float if the water table is above the pipe and the soil cover is inadequate. Worst case is a water table at, or above, ground surface. For a unit thick slice of cross section, the buoyant uplift force is $\pi D^2 \gamma_w/4$ which must be resisted by the buoyant weight of the soil wedges. See Figure 21. The wedge slip planes are at θ_s= 45º + ϕ/2. Buoyant unit weight is γ_b = γ_{sat} - γ_w (saturated unit weight of soil minus unit weight of water). The resistance to flotation is the area of the soil wedge above the pipe times its buoyant unit weight. In general, the height of cover should be greater than H = D/2. The soil should be denser than critical density. The minimum specified density is generally 90% (ASTM D698 or AASHTO T-99). If the soil is looser than critical density, it has the potential to liquefy.

Minimum Soil Cover

H is the height of soil cover from the top of pipe to the bottom of the ruts of a wheel load passing over. See Figure 22. Soil cover is minimum if less cover allows soil slip planes to reach the pipe with the potential to invert the pipe under a truncated soil pyramid punched through by wheel load, W. The loaded surface area is the tire print. For an HS-20 dual wheel, the tire print is roughly a rectangle 7x22 inches (178x559 mm) at 105 psi (724 kPa) tire pressure. In granular soil, the angle of the slip

planes is approximately 1 h:2v. For a dual wheel, pressure at the top of the pipe is P = W/(B + H)(L + H) where B and L are dimensions of a dual tire print. Dead load is neglected because it is small at minimum cover, and is balanced. Dead load left of centerline, is balanced by dead load right of centerline.

The critical location of pressure P for ring inversion is on one side of centerline as shown. Maximum moment occurs on the opposite side, and, from analysis by Castigliano's equation, is located about 12° from centerline. The maximum moment is,

$$M = 0.022Pr^2 \tag{23}$$

From theory of elasticity, M = σI/c where I/c is the wall section modulus. Substituting into Equation 23, with allowable stress, σ = S, the *minimum* required I/c is,

$$\text{Min } (I/c) = Pr^2/45S$$
$$\text{ELASTIC THEORY} \tag{24}$$

For plain pipes, stress due to the moment is $\sigma = 6M/t^2$ from which the required pipe wall section modulus is $t^2/6 = M/S$. Solving for M and substituting into Equation 23, $t^2S = 0.132Pr^2$. From this the *maximum* allowable ring flexibility, D/t is,

$$\text{Max } (D/t)^2 = 30S/P$$
$$\text{ELASTIC THEORY - Plain Pipe} \tag{25}$$

where

P = W/(B+ H)(L+ H) =punch-through live load pressure on the pipe

S = allowable stress in the steel.

At formation of a plastic hinge, the moment is 1.5 times the elastic moment. Equations 24 and 25 become,

$$\text{Min } (I/c) = Pr^2/68S$$
$$\text{PLASTIC THEORY} \tag{26}$$

$$\text{Max } (D/t)^2 = 45S/P$$
$$\text{PLASTIC THEORY - Plain Pipe} \tag{27}$$

The plastic theory, implies that as the wheel passes over the pipe, the top of the pipe could be deformed even though the pipe does not collapse. In fact, the pipe may not even be deformed because of its longitudinal beam

EXAMPLE —

What is the minimum granular soil cover over a plain pipe if D/t = 274, D = 51 inches (1275 mm), t = 0.187 inch (4.75 mm), and S, = 42 ksi (290 MPa)? The live load is W= 16 kips (71 kN) on a dual wheel with a rectangular tire print, B = 7 inches (178 mm) and L = 22 inches (559 mm). The soil is compacted, no ruts are left by the passing wheel. Using plastic theory, Equation 27, critical P = 25.2 psi (174 kPa). The actual P under the truncated pyramid is P = 16,000lb/(7 + H)(22 + H). Equating the two values of P and solving, *H = 11.8 inches (300 mm).* Safety factors are ample because every step in the analysis is conservative. Nevertheless, pipeline engineers usually call for a minimum cover of 3 ft (0.9 m). If the ground surface is paved, the effective loaded surface area, LxB, may be increased depending upon the type of pavement and the thickness. See HS-20 load in Figure 10.

Trench Conditions

The predominant pipe-soil interaction is. between the pipe and the embedment, not between pipe and trench walls. Properties of the native (in-situ) trench walls and dimensions of the trench are subdominant — with the following two caveats:

1. Buried pipes can be affected by: differential subgrade settlement, landslides, sidehill soil creep, soil liquefaction, and slip on seismic faults. The native soil must maintain alignment and must protect the pipe.
2. OSHA guidelines must be complied with in trench configurations.

Trench Shield:

When trench walls are retained by sheet piling or trench shield (trench box), voids could be left when the retainer is pulled. The pipe is not affected if the bottom edge of the retainer is above the spring line. Even if embedment slips into the voids, the slip planes are on a slope that does not intersect the pipe. In a narrow trench, if the bottom of the retainer is below spring tine, the voids below spring line should be filled as the retainer is pulled (by jetting, injecting grout, vibrating, etc.). The pipe is not affected if the trench wall soil slips in against the embedment; but if the embedment slips out against the trench wall, ring deflection of the pipe could increase.

Trench Width:

In general, the trench should be narrow — just wide enough for alignment of the pipe and for placement of soil against the pipe. Because $P_r = P_x r_x = P_y r_y$, as long as the ring is nearly

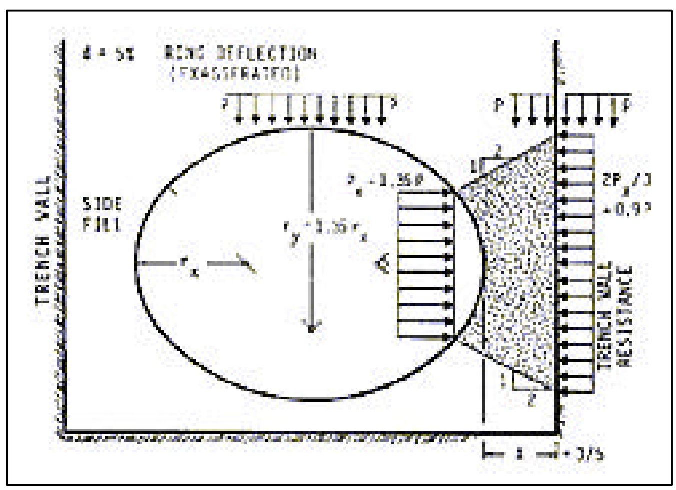


Figure 23. Trench wall support required for flexible ring stability, showing the sidefill wedge at soil slip. In this case, ring deflection is d = 5% and width of sidefill is X = D/5. In this worst case example, and in general, the required trench wall resistance is less than pressure P on the pipe.

circular, sidefill support only needs to be equal to the pressure P on top of the pipe. Theoretically, the soil can be liquid (no strength) if the ring is circular. Practically, if the trench is excavated in poor soil, a good practice is to place good embedment to a width of half a pipe diameter on each side of the pipe. It is presumed that ring deflection is less than 5%. Trench wall support of the embedment is active pressure only. Minimum trench width is sometimes specified to be two pipe diameters in poor native soil with low bearing capacity. Two special cases follow.

In the case of an elliptical ring, the required side support is, from Equation 16, $P_x = Pr_r = K\sigma_y$. For good sidefill, K equals 3 or more. At spring line, σ_y is greater than P on top of the pipe. However, assuming, conservatively, that $\sigma_y = P$, $r_r = 3$ at soil slip, and critical ring deflection is 18%. Clearly, 18% is unacceptable for other reasons than trench width.

EXAMPLE —

In Figure 23, ring deflection is d = 5% for which $r_r = (1+d)^3/(l-d)^3 = 1.35$. Pressure on top is P. Therefore $P_x = Pr_r = 1.35P$. In this case, the sidefill cover is a narrow X = D/5. If the granular sidefill soil slips, shear planes slope at 1v:2h. From geometry of the sidefill wedge, the area of contact of wedge on trench wall is roughly 1.5 times the contact area of pipe on wedge. Consequently, pressure on the trench walls is 1.35(2P/3) = 0.9P. High bearing capacity is not a prerequisite of the native soil in the trench walls.

Of greater concern is surface load. Figure 24 shows a dual wheel passing over a pipe with minimum soil cover. At minimum cover, a truncated pyramid is punched through. The pipe feels pressure,

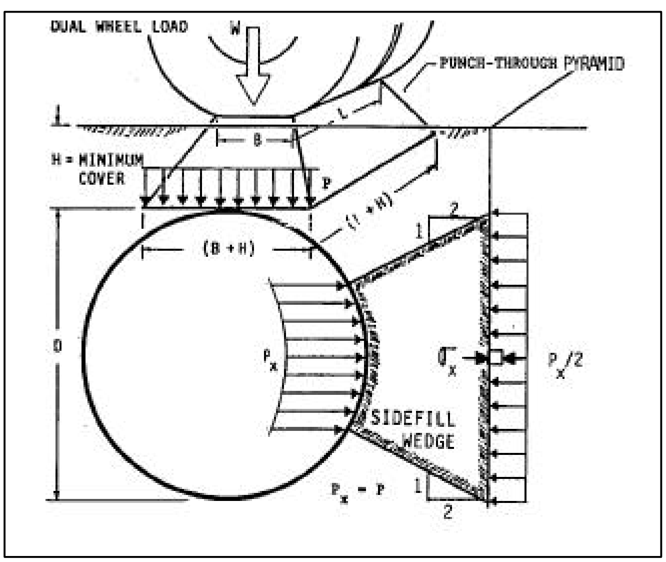


Figure 24. Trench wall support required for ring stability when a wheel load passes over a circular, flexible pipe with less-than-minimum soil cover. Active support for the wedge is negligible.

$P = P_d + P_l$, where $P_l = W/(B + H)(L + H)$. If the ring is nearly circular, $P_x = P$. In Figure 24 it is assumed that P, is great enough to punch out a sidefill wedge against the trench wall. The sidefill wedge forms within 90° of pipe arch along slip plane slopes of lv:2h. If width of sidefill is X = D/2, the contact area of wedge on trench wall is roughly twice the contact area of pipe on wedge. Therefore, pressure on the trench wall is $\sigma_x = P_x/2 = P/2$, which must be resisted by the trench wall. This is worst case. However, if a pyramid punches through, soil cover is less than minimum, and is unacceptable regardless of the trench width. This analysis is conservative because longitudinal beam resistance of the pipe is neglected.

EXAMPLE —

In Figure 24, the pipe diameter is D = 51 inches (1275 mm), t =0.187 inch (4.75 mm), and ring deflection is negligible. The soil unit weight is γ = 100 pcf (15.7 kN/m^3). There is no water table. Sidefill width is X = D/2. The soil cover is H = 2 ft (0.6 m). If an HS-20 dual wheel load of 16 kips (71 kN) passes over, what soil friction angle is required in the trench wall to prevent soil slip at the spring line level? Because H is greater than minimum soil cover, a pyramid does not punch-through. However, the pipe feels the live load pressure. From Boussinesq, $P_l = 0.447W/H^2$ = 1908 psf (91 kPa). This is upper limit because coefficient 0.477 yields maximum pressure, whereas the live load pressure at spring lines is an average. P_d = 200 psf. P = 2108 psf. For a circular flexible ring, $P_x = P$. See Figure 24, $\sigma_x = P_x/2$ = 1054 psf. At spring line level, σ_x = 413 psf. $K = \sigma_x/\sigma_y$ = 1054/413 = 1.555. But $K = (1 + \sin\phi)/(l-\sin\phi)$. Solving, $\phi = 26^o$ required in the trench walls. An additional safety factor is not needed.

If H is increased to 2.5 ft (0.75 m), what soil friction angle is required in the trench walls to prevent soil slip Following the same procedure, $\phi = 13^o$. With soil cover of 2.5 ft, if the trench wall can support the HS-20 truck, surely its friction angle is high enough to prevent soil slip. If H is increased to 3 ft (0.9 m), $\phi = 3^o$.

The above analyses apply to soil cover greater than minimum. At minimum cover, H = 11.8 inches (300 mm), a truncated pyramid is punched through by the wheel load, and P_l = 3626 psf. P_d = 98 psf. σ_x = 1862 psf, σ_y = 311 psf, and K = 5.987 = $(1 + \sin\phi)/(1 - \sin\phi)$. Solving, $\phi = 45.5^o$. Excellent soil is required in the trench wall. However, the analysis is moot, because performance limit is now the minimum cover H — not the trench wall.

What must be the soil cover, H, if the trench wall is of such poor quality soil that ϕ = 0? Solving the cubic equation, H = 3.2 ft (1 m).

All of the above analyses are based on cohesionless soil. Many trenches are excavated in soil with enough cohesion to stand in vertical cut. In such cases, the above analyses are conservative. Cohesion increases soil strength.

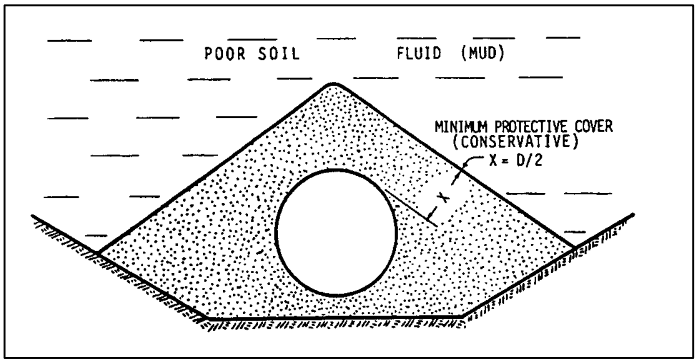


Figure 25. Minimum cover of embedment over a pipe in either an embankment or a trench with sloping walls in poor native soil.

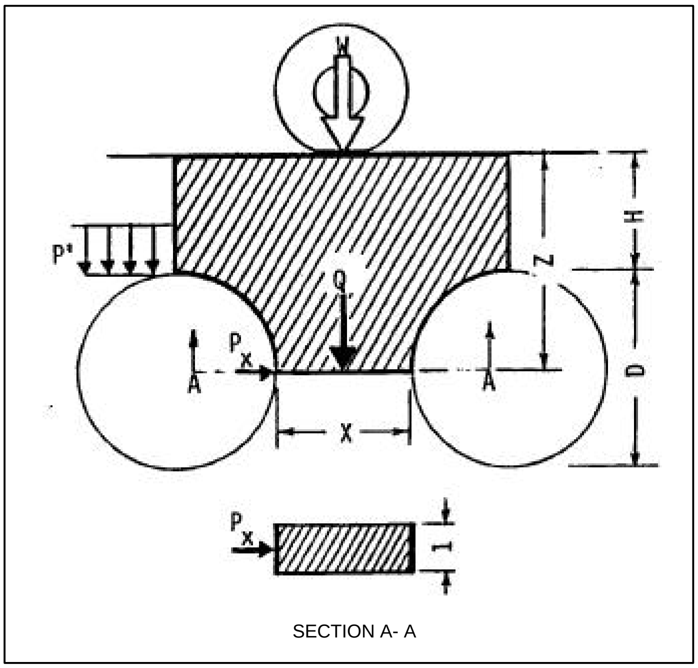


Figure 26. Load, Q, on steel-clad soil column Section A-A between parallel pipes.

Parallel Trench

When a trench is excavated parallel to a buried pipe, the question arises, how close can an open trench come to the buried pipe? At less than minimum side cover, X, side support is lost and the prism of soil on the pipe must be supported by the pipe. If ring stiffness is inadequate, the pipe collapses. Critical conditions are a flexible ring and a parallel trench with vertical walls. Soil strength can be measured by excavating a test trench. The depth, Z, at which the trench walls start to slough in, is a measure of soil strength. Z can be found on site by a backhoe, or it can be calculated from equations in texts on soil mechanics.

From tests, (X/D) = 1.4(H/Z). With ample safety factor for design, minimum X is found from the

$$(X/D) = 3(H/Z) \qquad (28)$$

If the soil is non-cohesive, slopes are nearly equal to the angle of repose. Soil cover of D/2 provides adequate protection as shown in Figure 25.

These parallel trench analyses are conservative because ring stiffness is neglected. Moreover, longitudinal beam action helps to resist collapse if the parallel trench is short.

Parallel Pipes

One performance limit for buried parallel pipes is soil slip between the pipes. See Figure 26. Section A-A is critical. It is the cross section of a slice of steel-clad soil column which supports dead weight, Q_d, of the soil shown crosshatched, plus the effect of live load, $Q_l = P_l X$, where P_l is the pressure at depth H + r according to Boussinesq. Total load on Section A-A is $Q = Q_d + Q_l$ of which steel cladding carries P'D where P' is dead load pressure at the top of the pipes. The remainder, Q - P'D, is supported by the soil. Horizontal stress in the soil is the horizontal pressure of the pipes against the soil, $\sigma_x = P_x = P'r_r$. The vertical stress is $\sigma_y = (Q - P'D)/X$. Knowing that $\sigma_x/\sigma_y = K = (1+\sin\phi)/(1-\sin\phi)$, the required soil friction angle, ϕ, can be calculated.

EXAMPLE –

Two parallel 51 inch (1275 mm) pipes are separated by X = 3 ft (0.91 m) under soil cover of H = 2.75 ft (0.84 m). What is the maximum dual wheel load, W, that can pass over the pipes? The soil is granular (cohesionless), well compacted with unit weight of 120 pcf (18.8 kN/m^3). Soil friction angle is $\phi = 37°$. Depth of Section A-A is Z = 4.875 ft (1.486 m). See Figure 26. The load on Section A-A includes dead load and live load. Dead load weight of soil (crosshatched) is Q_d = 3390 lb/ft (4.6 kN/m). Live load pressure at depth Z is $P_l = 0.477W/Z^2 = W/49.82ft^2$. Live load on column Section A-A is $Q_l = XP_l$ = W/16.6 ft. Total load on the column is $Q = Q_d + Q_l$. Load carried by the steel cladding is P'D = 510 lb/ft^2. Vertical stress on the soil column is σ_y = (Q-P'D)/X. Horizontal stress is the pressure of the pipe against the soil column; i.e., $\sigma_x = P_x = P$ = 330lb/ft^2 (15.8 kPa). At soil slip, $s_x/ s_y = K = 3$. Solving, W = 75 *kips* (66.7 *kN).*

With no live load, soil slips at the spring line if X is less than

$$X = 2rH/[H(K-1)-r]. \qquad (29)$$

For the example above, minimum separation is X = 1.9 ft (0.58 m). If ϕ is reduced from 37° to 30°, then X = 3.6 ft (1.1 m). Clearly, stability is sensitive to soil density. The analysis is conservative because ring stiffness and longitudinal beam action are both neglected.

Longitudinal Analysis

Following are three longitudinal stress (strain) analyses: thrust restraint, longitudinal contraction (expansion), and beam action.

1. Thrust Restraint:

Longitudinal thrust is caused by internal pressure and change in direction of flow. Thrust also occurs at “thrusters” (valves, elbows, wyes, tees, reducers, etc.). A large thrust occurs at 90° elbows. With pressure in the pipe, a cap or closed valve causes thrust with longitudinal stress of σ_z = Pr/2t in the pipe. Even so, the longitudinal stress is, at most, only half as great as the circumferential hoop stress. Thrust must either be resisted by the pipe itself (welded joints) or by external restraints such as thrust blocks, anchors, or the embedment soil through friction of soil on pipe. Welded joints between the thruster and the restraint must be able to resist the longitudinal thrust.

EXAMPLE —

A 51 inch (1275 mm) water pipe with internal pressure, P = 120 psi (827 kPa), flows at about v = 15 ft per second (4.57 m/second). What is the longitudinal thrust caused by a 90° elbow (q =90°)? 8 is the angle offset of the elbow. Thrust, $Q = Q_p + Q_i$, where: due to pressure, $Q_p = P\pi D^2(1-\cos q)/4$ = 245 kips (1.09 MN); and due to change in direction of flow (impulse), $Q_i = \gamma_w \pi D^2(1-\cos q)v^2/4g$ = 6 kips (26.7 kN) where g is acceleration of gravity. Adding Q_p and Q_i, *Q* = 251 kips (1.12 MN). The thrust due to impulse, Q_i, accounts for only 2.4% of the total thrust. Impulse thrust is usually neglible.

The thrust due to thrusters is:

$$Q = (\pi D^2/4)(1-\cos q)(P + \gamma_w v^2/g) \qquad (30)$$

2. Longitudinal Contraction:

Longitudinal stresses are caused by temperature change and internal pressure *if the ends of the pipe are fixed;* i.e., if the pipe cannot change in length. Longitudinal tension stresses are: $E\alpha(\Delta T)$, caused by temperature decrease; and $\nu PD/2t$, caused by internal pressure increase (Poisson effect). Longitudinal tension stress, σ_z, in a straight, *fixed-ended* pipe, due to temperature decrease and internal pressure increase, is,

$$\sigma_z = E\alpha(\Delta T) + \nu PD/2t \qquad (31)$$

where

E = modulus of elasticity of steel = 30,000,000 psi (207 GPa),

α = coefficient of thermal expansion of steel = $6.5(10^{-6})$/°F

ΔT = decrease in temperature in degrees Fahrenheit, °F, [°F = 1.8(°C) + 32 where °C is in degrees Celsius.],

ν = Poisson ratio = 0.3 for steel,

P = internal pressure in the pipe,

D = diameter,

t = wall thickness,

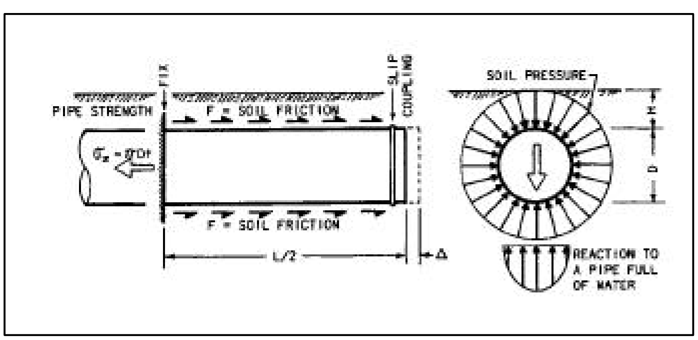


Figure 27. Friction, F, of soil on pipe due to contraction of the pipe. L/2 is found by equating friction force, FL/2 to longitudinal pipe strength, $\sigma_z \pi Dt$.

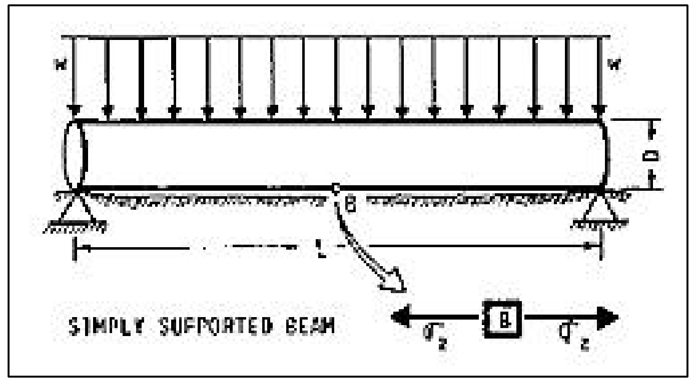


Figure 28. Worst-case beam loading for longitudinal stress analysis of a simply-supported buried pipe section.

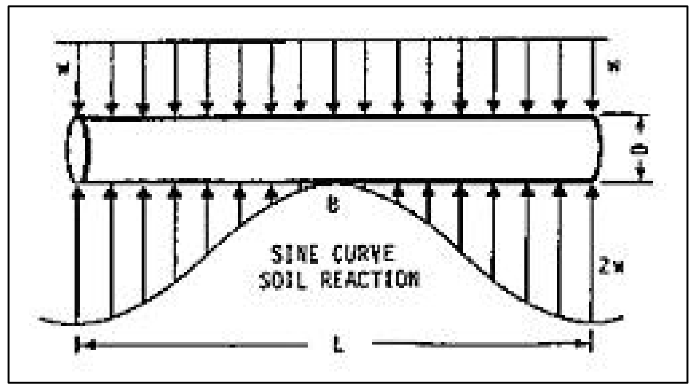


Figure 29. Reasonable soil reaction for longitudinal analysis.

EXAMPLE —

A straight plain steel pipe, D = 51 inches (1275 mm) and t = 0.187 inch (4.75 mm) is fixed-ended. It is positioned and welded up at day-time temperature of 90°F (32°C). When in service with water in the line, temperature is 40°F (4°C) and internal pressure is 150 psi (1.03 MPa). If D/t = 274, what is the longitudinal stress? From Equation 31, σ_z = 9.75 ksi + 6.14 ksi. σ_z *15.9 ksi (110 MPa).*

3. Slip Couplings:

It is possible to reduce longitudinal stress in the pipe by installing slip couplings or gasketed joints. See Figure 27. If the only restraint is soil friction, the friction force, FL/2 equals longitudinal pipe strength, $\sigma_z \pi Dt$. Solving for maximum spacing, L, of slip couplings in a welded pipe,

$$L = 2\sigma_z t/\mu(\gamma H + \gamma_w D/4 + w_p \pi D) \quad (32)$$

The three terms in parentheses are: external soil pressure, weight of water in the pipe and weight of the pipe.

EXAMPLE –

What is the minimum spacing of slip couplings in a buried pipe? The joints are single lap welds. Therefore, longitudinal stress in the pipe is not to exceed 75% of allowable stress of S = 21 ksi (145 MPa). Data are as follows.

L = spacing of slip couplings along the pipeline,

T = wall thickness,	= 0.250 (6.35 mm)
D = pipe diameter,	= 51 inches (1275 mm)
H = height of soil cover,	= 3 ft (0.91 m)
γ= unit weight of soil,	= 110 pcf (17.3 kN/m^3)
γ_w = unit weight of water,	= 62.4 pcf (9.8 kN/m^3)
w_p = weight of pipe per unit length,	= 102.33 lb/ft
σ_z = allowable longitudinal stress,	= 15.75 ksi (109 MPa)
μ= coefficient of friction of soil on pipe,	= 0.32 for tape coated
μ= coefficient of friction of soil on pipe,	= 0.4 for mortar coated

If the pipe is tape coated, μ = 0.32. Substituting data into Equation 32, L = 731 ft. This is based on the assumption that the pipe will contract due to internal pressure and decrease in temperature. In practice, temperature changes are minimized by welding some of the joints (say every 500 feet) after the pipe is buried and the pipe temperature is less than the temperature of the pipes when first placed in the trench. Contraction is significantly reduced. Therefore, slip couplings are often spaced at L = 1000 to 1500 ft.

4. Beam Action:

In order to vertically align a pipeline, sections are sometimes supported on mounds at the ends of each section. In the case of gasketed pipe sections, a worst case is poor, or no, soil under the haunches. See Figure 28. Beam strength is required. For the simply supported span, the maximum longitudinal stress, σ_z occurs at B where, by classical analysis,

$$\sigma_z = (w/2\pi t)(L/D)^2$$

SIMPLY SUPPORTED (33)

where

σ_z = longitudinal stress in the pipe wall,

w = P'D + weight of pipe and contents per unit length,

t = pipe wall thickness,

L = length of the pipe section,

D = diameter,

P' = vertical soil pressure at the top of the pipe.

If joints are welded (not gasketed), the beam is fixed ended. Maximum longitudinal stresses are at the ends of the supported sections – not at midspan and are two – thirds as great as the simply supported beam of Equation 33; i.e.,

$$\sigma_z = (w/3\pi t)(L/D)^2$$

FIXED-ENDED (34)

The above stresses are analyzed with concentrated reactions. Some soil support is inevitable under the haunches, even when the bedding does not contact the pipe. Therefore, a more reasonable distribution of soil support is a sine curve shown in Figure 29. For this case, the maximum longitudinal stresses are forty percent of the stresses calculated for concentrated reactions. A safety factor of 2.5 is built in to the idealized Equations 33 and 34.

5. Buried Pipe on Piles:

A worst case of beam action is a buried pipe on piles in a zone where soil settles. The purpose of the piles is to maintain vertical alignment. When the soil settles, the pipe lifts soil wedges as shown in Figure 21. The wedge soil load is greater than the prismatic P'D load of Equation 20.

EXAMPLE —

A pipe is to be buried under four feet of saturated soil in a tidal zone where soil heaves and settles. In order to maintain vertical alignment, the pipe is to be positioned on saddles supported by piles spaced at L = 40 ft (12.2 m). The unit weight of the saturated soil is 125 pcf (19.6 kN/m^3). Assume the soil friction angle is 30°. D = 51 inches (1275 mm), t = 0.187 inch (4.75 mm). What is the maximum longitudinal stress, σ_z, when the soil settles and the water table is below the pipe at low tide?

w_p = 102 lblft = weight of the pipe,
w_w = 885 lb/ft = weight of the water,
w_s = 3448 lb/ft = weight of the soil,
w = 4435 lb/ft (6 kN/m).

Substituting into Equation 31, σ_z = 18.6 *ksi (128 kPa).* If the spacing of the piles were 60 feet instead of 40 feet, longitudinal stress would be σ_z = 41.8 ksi (288 kPa) too much for welded joints. Check concentrated stresses due to the pipe bearing on the saddle.

Backfilling

To prevent soil settlement and liquefaction, embedment should be placed at a density greater than critical void ratio density. With a margin of safety, 90% Standard Proctor density (ASTM D698 or AASHTO T-99) is usually the density specified. Other performance limits, such as ring deflection, may require greater soil density. For most installations, pit-run gravel or dry coarse sand falls into place at adequate density without additional compactive effort.

1.Water Compaction:

Various methods of “settling” the soil with water include flushing, ponding and jetting. The density achieved by water compaction is not as great as can be achieved by mechanical compaction; but is often adequate.

a) Flushing – Sand can be flushed under the pipe from a windrow by a high pressure hose. The saturated sand flows into place under the pipe. Good drainage is essential. Flotation must be avoided.

b) Ponding – If granular soil is placed to the spring lines, it can be partially settled by flooding the surface with water and leaving it for a few hours to a few days as the soil settles and shrinks in volume. Good drainage is essential. Flotation can be a problem.

c) Jetting – With soil up to the spring lines, water jets can be used to settle the soil. The jets are high pressure stingers - three-quarter inch (19 mm) pipes five ft (1500 mm) long - which can be thrust vertically into the soil to a depth near the pipe bedding. High pressure water flushes granular soil laterally under the haunches. As the stinger is withdrawn, it may be vibrated to shake soil down into the hole left by the jet.

Mechanical Compaction:

Soil placed in lifts of 8 to 12 inches on the sides of the pipe can be mechanically compacted. Soil moisture content should be at or near optimum. The pipe should be monitored to prevent vertical elongation of the pipe caused by heavy compaction at the sides of the pipe. Compactors should not hit the pipe. For efficient compaction techniques, geotechnical engineers should be consulted.

a) Light compaction zone – Only hand operated compactors should be permitted within three feet from the pipe and closer than 45º planes tangent to the haunches. Outside of the light compaction zone, heavy compactors can be used, and heavy equipment can pass over.

b) The top of the pipe is sensitive to compaction. With only one lift of backfill over the pipe, it is well to avoid compacting directly above the pipe. The result could be ring deflection and disbonding of mortar lining. With the second lift over the pipe, it is prudent to compact over the sidefill before compacting over the pipe - compacting from the trench walls in toward the center to form a soil arch over the pipe.

3. Soil Cement:

Under adverse circumstances, soil cement slurry or grout can be used to form a bedding and to fill voids under the haunches. The soil cement is placed from one side to make sure that voids are filled under the pipe as soil cement rises on the other side. High strength is not warranted because the bedding is confined. Compressive strength of 40 psi (280 kPa) is generally adequate. Higher strength may assure greater flowability, but at increased cost. In fact, the basic purpose of the cement is “flowability”. A slump of ten inches is generally about right for flowability without excessive shrinkage when the soil cement sets.

Compound Stress Analysis

Performance limit is tensile strength, σ_f. The standard test is a tensile test of a circular rod. For steel, stress is critical on a 45º plane where shearing stress is maximum. Shearing strength is, $\tau_f = \sigma_f/2$

For compound stress analysis, the free-body-diagram is an infinitesimal cube subjected to three principal stresses. Shearing stresses are zero if the cube is oriented to principal stresses. See Figure 30. Strength is a function of principal stresses.

In steel pipe, the minimum principal stress, σ_x, is usually internal pressure which is of opposite sign from the maximum principal stress. Compared to σ_y, the minimum principal stress, σ_x, is usually small enough to be neglected. The strength envelopes of Figure 30 are based on $\sigma_x = 0$. If values for σ_y and σ_z are known, the corresponding strength can be found from Figure 30. Based on shearing strength theory, the strength envelope is shown dotted in Figure 30. Tests show that the strength envelope for steel is more nearly an ellipse as shown in solid line. Strain-energy analyses produce elliptical strength envelopes.

Huber-Hencky-vonMises Equation

One elastic strain-energy model for steel is the Huber-Hencky-vonMises equation which subtracts out that part of strain energy that only results in volume change. Assuming that $\sigma_x = 0$, the equation for the strength envelope is,

$$\sigma_y + \sigma_z^2 - \sigma_y\sigma_z = \sigma_f^2 \quad \text{HUBER-HENCKY-vonMISES} \qquad (35)$$

a plot of which is the ellipse shown in Figure 30. The stresses are all principal stresses. For most buried pipes, the Huber-Hencky-vonMises analysis is not justified. Equation 35 is based on elastic analysis. But elastic limit (yield stress) is not necessarily the performance limit for buried steel pipes. If a section of pipe is capped such that $\sigma_z = \sigma_y/2$ (both σ_z and

σ_z are in tension), from Figure 30, the hoop strength is $\sigma_y = 1.155\sigma_f$. The increase in hoop strength is only 15.5%. It is conservative to design by uniaxial stress analysis; i.e. critical stress is $\sigma_y = \sigma_f$. If, perchance, longitudinal stress, σ_y, is of opposite sign from the hoop stress, σ_y, Equation 35 should be applied. The strength envelope is shown in the upper left and lower right quadrants of Figure 30. The probability that σ_z and σ_y are of opposite signs is remote.

Stresses at Mitred Joints

Figure 31 shows a mitred joint — exaggerated. Due to pressure, P, inside the pipe, force, Q, on the elbow is $Q = 2P\pi r^2 \sin\theta$. The impulse force due to change in direction of flow is neglected because it is usually relatively small. With no external thrust restraints, Q must be resisted by the pipe wall for which shear and thrust forces are:

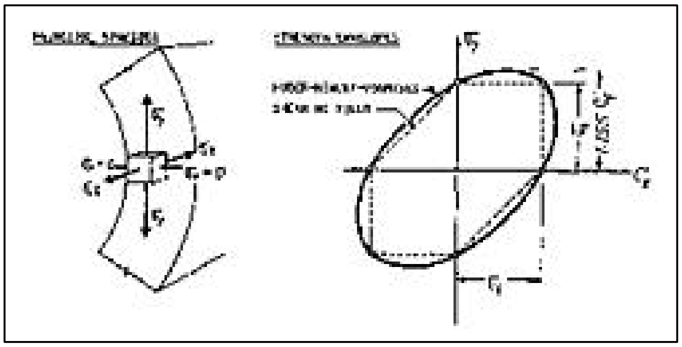


Figure 30 Strength envelopes at elastic limit, σ_f, by compound stress analyses when $\sigma_x = 0$.

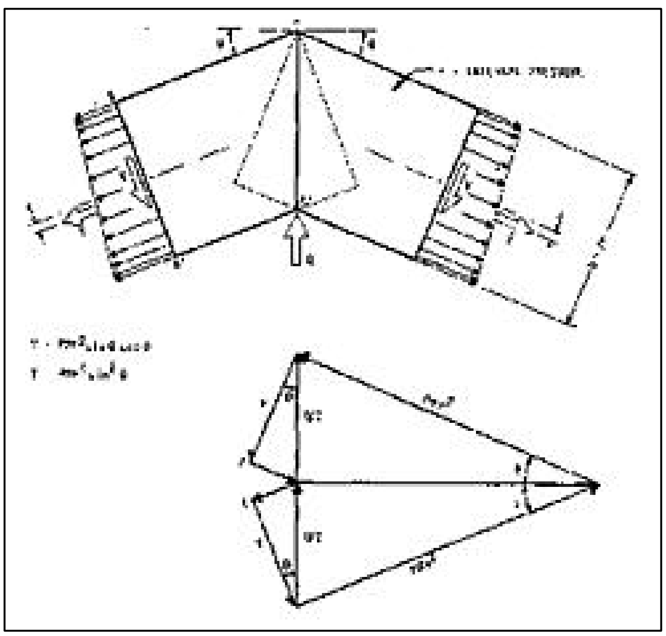


Figure 31 Free-body-diagram of a mitred joint (elbow) in a pipe (top), and the free-vector-diagram of forces acting on it (bottom) due to internal pressure, P.

$$V = P\pi r^2 \sin\theta\cos\theta \qquad (36)$$

$$T = P\pi r^2 \sin^2\theta \qquad (37)$$

If the maximum angle offset for a bend is 15º, then $q = 7.5°$, $V = 0.13\ P\pi r^2$, and $T = 0.017\ P\pi r^2$. Hydraulics usually limit the offset to 15°. Shear, V, is easily resisted by the pipe wall, and is reduced by soil support. Thrust, T, is small enough to be neglected. The rationale is as follows.

If pipe alignment is held (by soil embedment), and if cut BB′ is moved up to the seam so that it passes through A', stress distribution is a triangle rather than a trapezoid. The maximum longitudinal stress (at A') is twice the average longitudinal stress in the pipe. Because this is about the same as hoop stress in the pipe, yield stress is not exceeded. In fact, allowable stress is only half of yield stress, so a safety factor of two is still in effect. A good weld is essential if the safety factor is to be maintained.

Hoop strength is lost at the mitred seam because of the skew cuts. The hoop stress triangles to be resisted are shown in Figure 32. The seam must resist hoop stresses from both sides of the seam so,

$$w = 2Pr \sin\alpha \cos\theta \qquad (38)$$

For exceptionally large pipes and high pressures, a stiffener ring may be required. But for most pipes, an adequate stiffener ring is formed by the V-intersection of the mating pipes and a good weld. There is no need to increase the wall thickness of the mitred joints for most pipes. Moreover, pipes are not membranes, but have significant stiffness — especially at welded V-intersections. Experience confirms the sturdiness of mitred joints. Cost is reduced if wall thickness is not increased. Mitres are usually shop-fabricated on the end of a section of pipe of the same wall thickness.

Strength of Welded Joints

If the weld is a full-penetration butt weld, longitudinal strength is no less than the longitudinal strength of the pipe. The wall usually yields before the weld yields. For conservative design, to allow for welding flaws, the longitudinal strength of a butt weld is assumed to be 100% of pipe strength. But allowable stress is only half of yield strength. The margin of safety is still substantial.

Figure 33 shows a single lap weld and a double lap weld. For the strength of lap joints, see “Strength of Bell and Spigot Joints”, by Roger L. Brockenbrough, Journal of Structural Engineering, Vol. 116, July, 1990. A question is sometimes raised concerning flexural stress in the weld due to moment caused by the offset (similar to Figure 34). This analysis is flawed. The force on the weld is basically a shearing force spigot pipe slips in a bell sleeve. In fact, the moment caused by the offset is resisted by the pipe - not the weld. Both the bell and the spigot resist moment. The curved surfaces of the cylinders also resist moment.

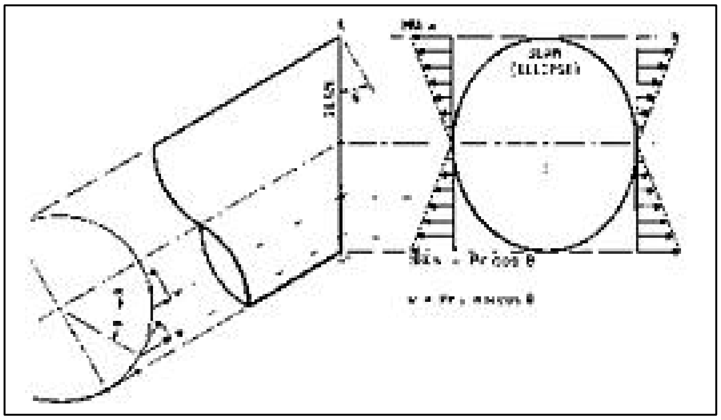


Figure 32 Forces, w, on the mitre seam due to loss of resistance to internal pressure of the hoops that are cut. This is a conservative analysis based on membrane theory.

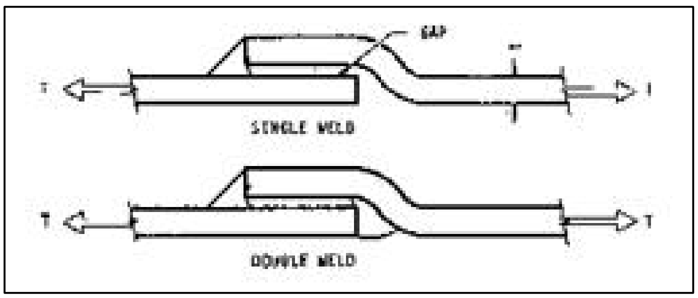


Figure 33 Lap welds - single and double.

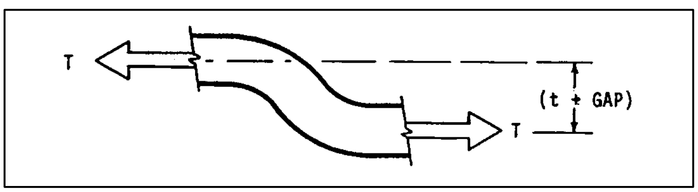


Figure 34 Transition in the pipe wall from pipe to bell.

More critical than the weld is the radius of transition of the pipe wall from pipe to bell. See Figure 34. When fracture occurs, it is generally in the transition. The radius of transition should be great enough that the pipe wall is not cracked when it is expanded into a bell. In general, cracks do not form if strain is less than 21%. From fundamentals of mechanics, $l/R = 2\epsilon/t$. Minimum radius R is,

$$R = t/2\epsilon \qquad (39)$$

where

R= radius of the neutral surface of the pipe wall,

t= wall thickness,

ϵ = strain in the wall surface at cracking.

If allowable strain is about twenty percent, R = 2.5t. With a safety factor of two, the radius of transition should be at least *R= 5t*. Some conservative pipe manufacturers use R = 7t.

If tests for longitudinal strength are required, full pipe sections — not just coupons — are recommended. Longitudinal strength is the strength of a three dimensional cylinder weld — not just a two dimensional slice

Width of the gap should be controlled. “Slugged” welds should be avoided. If the gap is large enough to insert a bar of reinforcing steel, it is too large.

The Effect of Mortar Linings and/or Coatings on Ring Stiffness

Discounting bond between mortar and steel, the moment of inertia of the lined and/or coated wall section is the sum of the separate moments of inertia of steel, lining, and coating. If the moment of inertia is found in terms of the mortar (equivalent cross section in mortar — not steel), the steel shell must be transformed into its equivalent width, n, in mortar. Consider a unit slice of wall, Figure 35, showing the transformed section in mortar.

X_s = n = 7.5 = unit width for steel transformed into its mortar equivalent,

x_m = 1 = width of mortar coating and lining (unit slice),

E_m = $4(10^6)$ psi (28.6 GPa) for mortar,

E_m = $30(10^6)$ psi (207 GPa) for steel,

n = E_s/E_m = 7.5 for transforming the unit width of steel into its mortar equivalent.

Find I:

For each mortar layer, $I = t^3/12$. For the steel equivalent in mortar, $I = nt^3/12$. The moment of inertia of the wall section is the sum of the I’s for the layers. See Table 2 for typical values.

Find stress, σ:

Critical stress occurs in the thicker of the two mortar layers. Assuming the coating to be thicker

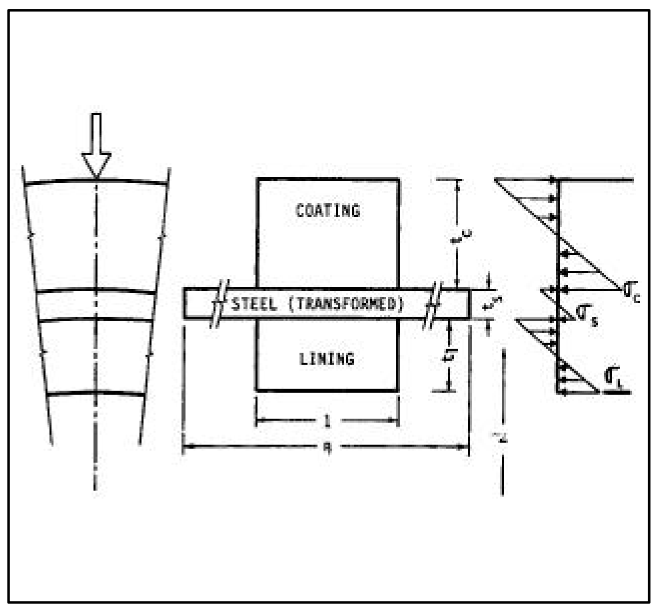


Figure 35. Transformed section of a unit slice of mortar lined and coated wall, transformed into its equivalent in mortar, for evaluating the moment of inertia. Shown on the right is the elastic stress distribution of the three layers.

than the lining, the maximum stress in the mortar is $\sigma_c = M_c t_c/2I_c$. The moment M_c, is that fraction of total moment, M, resisted by the coating. For a transformed section, resistance in each layer is provided by its moment of inertia. If I is the sum of the moments of inertia of all three layers; for the coating $M_c/M = I_c/I$. Total moment M is a function of loads on the ring.

EXAMPLE –

What is maximum stress in the coating of the AWWA mortar lined and coated steel pipe of Table 2? The pipe is loaded by an F-load for which the maximum moment is $M = Fr/\pi$. From Table 2, $t_c = 0.75$ inch (19 mm), and $I_c/I = 0.05625/0.07835 = 0.718$. Maximum moment is M_c = 71.8 percent of M. Substituting into the stress equation, maximum stress in the coating is $\sigma_c = 6M_c/t_c^2 = 2.4378$ Fr/in^2. *$\sigma_c = 2.44\ Fr/in^2$ [0.0038 $Fr/(mm^2)$].*

Plastic Analysis

Figure 36 is a section of plain pipe wall (no coating, lining or stiffeners) subjected to a moment due to ring deflection, showing the elastic stress distribution on the right and the plastic stress distribution on the left. Because yield stress, σ_y, is the same for both analyses, the moment can be calculated for each stress distribution. It is easily demonstrated that $M_p = 3M_e/2$.

Plastic analysis is the more reasonable performance limit for plain pipe because it represents the moment at plastic hinging - the maximum moment that the wall can resist.

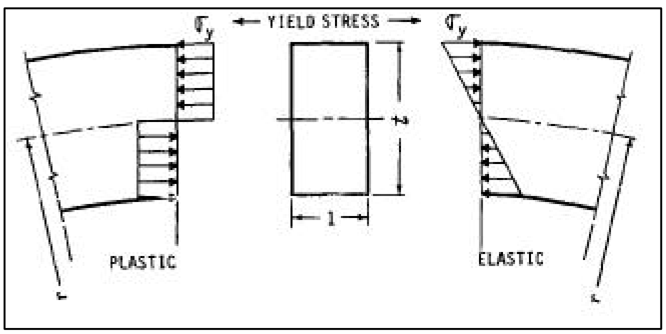


Figure 36. Unit width of plain pipe wall at a location where the moment causes elastic stresses shown on the right and plastic stresses shown on the left.
$M_p = \sigma_y t^2/4$ and $M_e = \sigma_y t^2/6$.

Measurement of Radius of Curvature

Elliptical ring deflection is not the only ring deformation that affects collapse. The critical element of deformation is excessive radius of curvature which can occur at flattened areas of pipe. Small dings and dents are not the issue because longitudinal beam action bridges over then. An approximate analysis of flattened areas that extend more than one diameter along the pipe, is the elliptical ring deflection analysis, except that measured values of maximum and minimum radii are substituted for elliptical r_y and r_x.

Radius of curvature can be measured by laying a short straight edge of known length tangent to the pipe on the outside, or a cord on the inside. See Figure 37. Offset, e, is measured from pipe to straight edge as shown. If L is the length of the straight edge, the radius of curvature is $r = L^2/8e + e/2$ plus or minus a t/2 correction to the neutral surface. Accuracy is usually adequate for,

$$r = L^2/8e \quad (40)$$

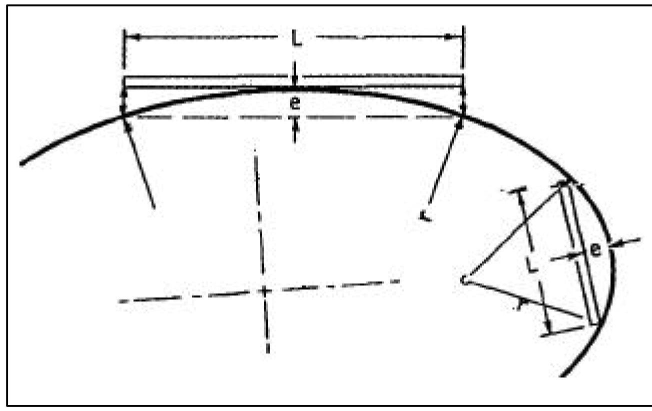


Figure 37. Technique for finding the radius of curvature, r, of a pipe ring by measuring the offset, e, from a cord of known length, L, to the pipe surface.

Soil Modulus

Soil modulus is the slope of a cord secant on the stress-strain diagram for the embedment soil. The terminal points of the cord are the initial and final soil stresses between which the modulus is defined. See Figure 15. From this modulus, the soil strain due to the increase in soil stress can be found. In order to apply this modulus, questions have been posed.

1. Should the stress-strain diagram (compression test) be based on a confined soil test, or on a triaxial soil test which allows radial (lateral) expansion?

At the sides of a buried pipe, soil is essentially confined. Longitudinally, the soil is completely confined. Vertically, the soil is *compressed* — not expanded. Laterally, the soil is compressed and forms the basis for the oft-cited horizontal soil modulus, E′. For both horizontal and vertical stress-strain, the argument is the same: *A confined soil test is the most relevant stress-strain rest for pipe embedments..*

2. Is the horizontal soil modulus, E′, at the sides of the pipe the primary soil property for predicting ring deflection of a flexible pipe?

No. A horizontal E′ is neither constant nor precisely measurable. E′ varies with depth of burial and with ring deflection which is based on soil stiffness and ring stiffness. Analysis of E′ is of minor value anyway. Horizontal E′ is less relevant than vertical modulus E″ in predicting ring deflection. *The vertical ring deflection of a flexible pipe is primarily a function of vertical soil*

modulus E". Vertical soil modulus E" is easily found in the soils laboratory by a confined compression test at whatever soil density is specified, and at whatever vertical soil stresses are anticipated.

3. Moot, but often asked, is the question, "Because a thin blanket of loose soil occurs next to the pipe, does the pipe have to deflect some before passive resistance E' is developed?"

Any ring deflection causes passive resistance because soil at the sides of the pipe slips. A soil slip phenomenon is not an elastic soil compression (horizontal E') phenomenon. The zone of soil slip increases in volume as ring deflection increases. If the loose soil blanket is the same on top as on the sides of the pipe, the compression of the soil is constant all around a flexible pipe. This can be altered by ring stiffness. A stiff ring will increase compression of the soil on top, and reduce compression of the soil at the sides. The net result is a lower value of the ring deflection term, d/E in Figure 16. For most flexible steel pipes, d/E is essentially unity - independent of horizontal E'.

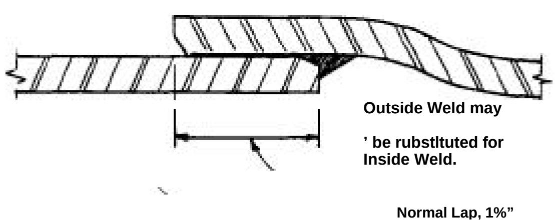
San Diego, CA 96" I.D. Pipe

Joints

Steel pipe lengths can be joined together in the field by many different methods to effect rigid or flexible connections, as follows:

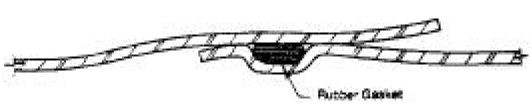
Bell & Spigot Lap Welded Joint
Bell & Spigot Rubber Gasket Joint
Harness Joint — Bell & Spigot
Carnegie Shape Rubber Gasket Joint
Butt Welded Joints
Butt Strap Joint for Welding
Mechanically Coupled Joints
Flanged Joint for Bolting

1. Bell & Spigot Lap Welded Joint



The Bell and Spigot lap welded joint is widely used because of its flexibility, ease in forming and joining, water-tightness and simplicity. Small angle changes can be made in this joint. The joint may be welded on either the inside or outside with a small fillet weld.

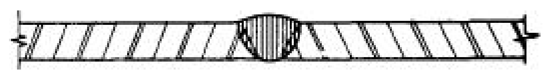
2. Bell & Spigot Rubber Gasket Joints



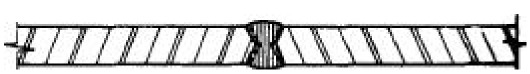
Rolled-groove rubber gasket joint, usually applied to small diameter water pipe.

Bell and Spigot Rubber Gasket Joints simplify laying the pipe and require no field welding. They permit flexibility, water-tightness, lower installation costs, elimination of bell-holes, etc. Gaskets conform to AWWA Standards.

3. Butt-Welded Joints



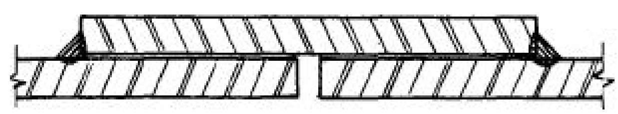
Single-V Butt-Welded



Double-V Butt-Welded

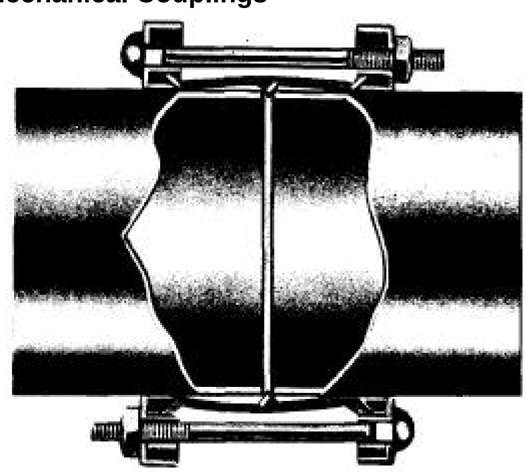
Butt-welded joints will develop full strength, but will require more care in cutting and fitting up in the field if changes in alignment or profile occur frequently. This joint is not commonly used.

4. Butt Strap Joint for Welding

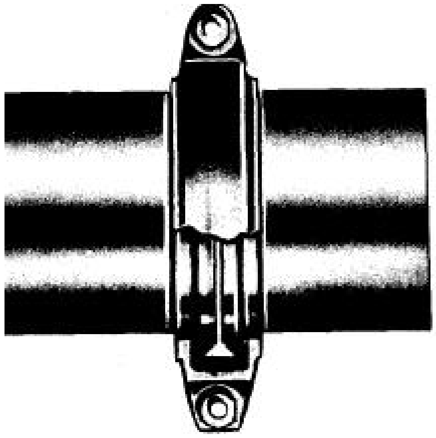


The butt strap is a closure joint used for joining ends of pipe when adjustments are required in the field.

5. Mechanical Couplings

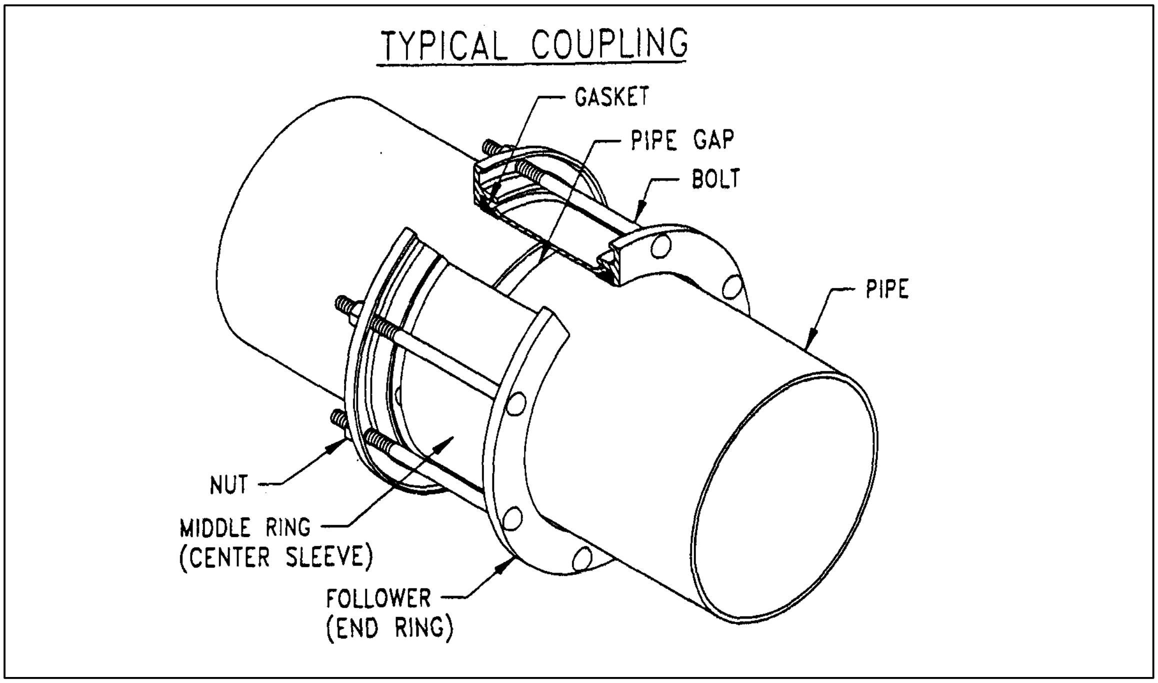


a. Sleeve type



b. Grooved and shouldered type

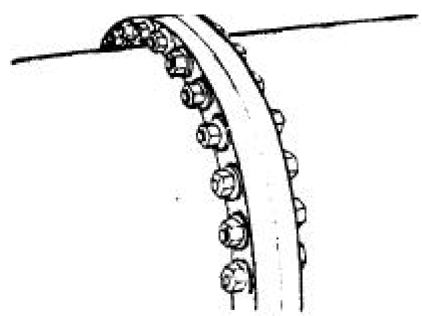
Mechanical couplings provide ease of installation and flexibility and are represented by the sleeve and clamp type of coupling.



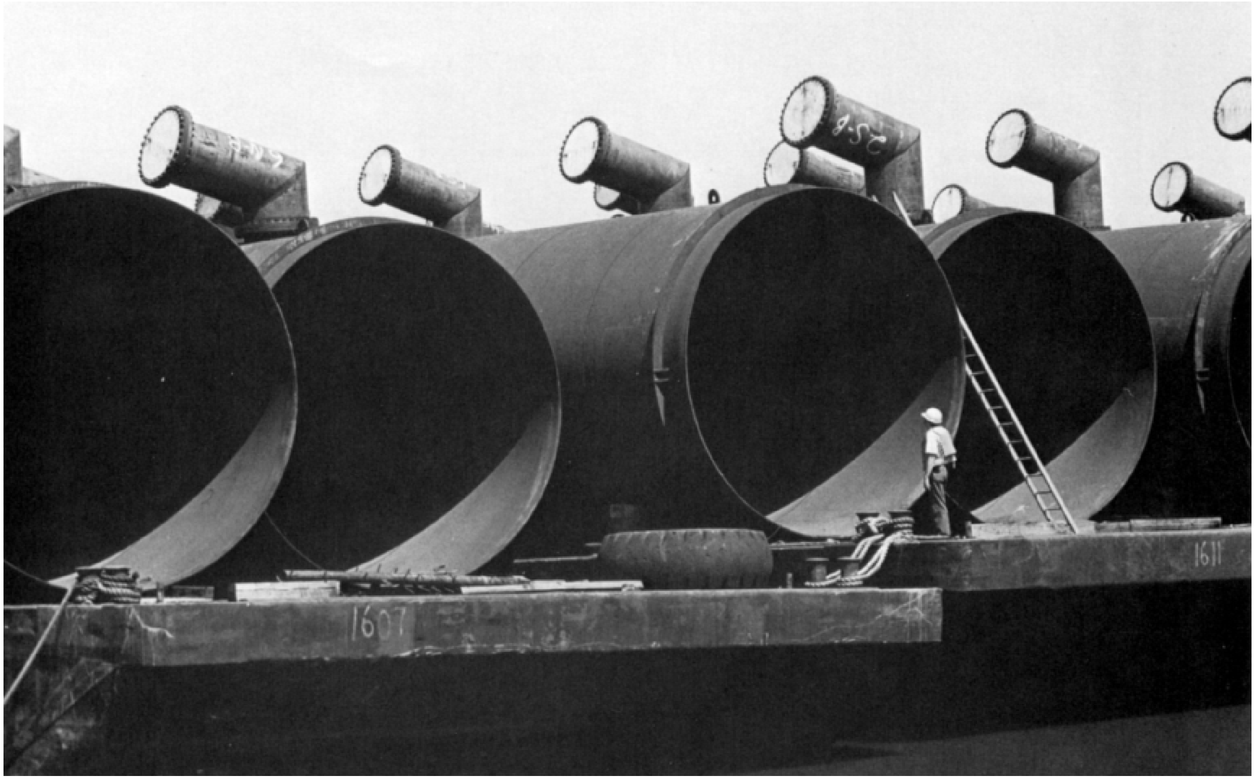
Mechanical joint

6. Flanged Joints

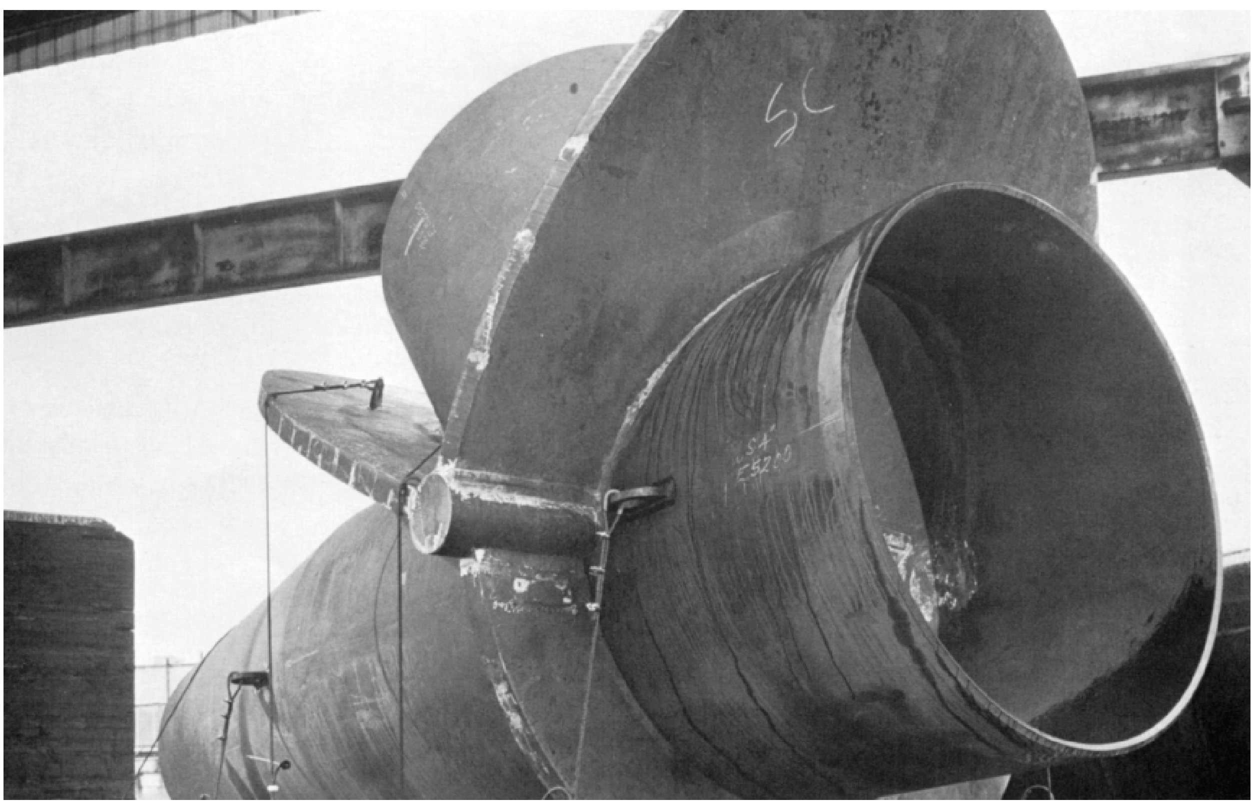
Flanged joints are not generally used for field joints on large diameter steel pipe because of their high cost and lack of flexibility. They are advantageous, however, for special conditions, such as connections to flanged gate valves, bridge crossings, meters, and for field connections by unskilled labor.



Flanged joint



40′ long, 11′ diameter pipe ready for shipment for power plant inlet pipeline in Lake Michigan.



84′ Bifurcation, Hydroelectric Project, 5″ thick reinforcing plates with 12″ diameter rod.

Above-Ground Installations

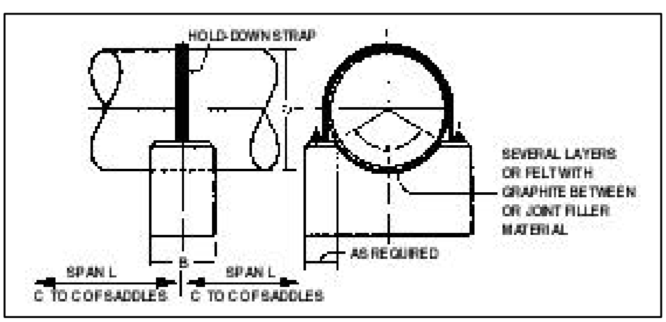
Welded steel pipe, laid above-ground or in tunnels, is generally supported on saddles or by ring girders supported by piers. Ring girders are usually spaced at greater intervals than saddles.

Saddle Supports

There has been very little uniformity in the design or spacing of saddle supports. The spans have been gradually increased, however, as experience has shown that such increases were safe and practical. In general, the ordinary theory of flexure applies when a circular pipe is supported at intervals, is held circular at and between the supports, and is completely filled. If the pipe is only partially filled and the cross section at points between supports becomes out-of-round, the maximum fiber stress is considerably greater than indicated by the ordinary flexure formula, being highest for the half-filled condition.

In the case of a pipe carrying internal pressure where the ends are fully restrained, the Poisson-ratio effect of the hoop stress, which produces lateral tension must be added to the flexural stress to obtain the total beam stress.

Excessive deflection should be avoided when the pipe acts as a beam. A maximum deflection of $^1/_{360}$ of the span is suggested as good practice. This is the same recommendation used for beams carrying plastered ceilings.



Saddle supports cause local stresses both longitudinally and circumferentially in unstiffened, comparatively thin-wall pipe at the tips and edges of the supports. The highest local stresses are the circumferential bending stresses at the saddle tips. Stresses vary with the load, the diameter-wall thickness ratio, and the angle of contact with the pipe. In practice, the contact angle varies from 90° to 120°. The difficulty encountered with 180° contact angles has been eliminated by reducing the angles to 120°. For equal load, the stresses are less for a large contact angle than for a small one, and interestingly, their intensity is practically independent of the width of the saddle (Dimension B, per preceding illustration). The width of the saddle may therefore be that which is most desirable from the standpoint of good pier design.

Because saddle supports cause critical points of stress in the metal adjacent to the saddle edges, it is frequently more economical to increase the wall thickness of the pipe when it is overstressed than to provide stiffening rings. This is especially true where pipe sizes are 36 in. in diameter and smaller. Even a small increase in wall thickness has a great stiffening effect. The whole length of the span may be thickened, or only a length at the saddle support - equal to about two pipe diameters plus saddle width - need be thickened.

When pipe lengths resting on saddles are joined by flanges or mechanical couplings, the strength and position of the joints must be such that they will safely resist the bending and shear forces while remaining tight. Ordinarily it is advisable to place joints at, or as near as practicable to, the point of zero bending moment in the span or spans. Manufacturers of mechanical joints should be consulted regarding the use of their joints on self-supporting pipe spans.

The pipe should be held in each saddle by a steel hold-down strap bolted to the concrete. Secure anchorages must be provided at intervals in multiple-span installations.

Research* has shown that, for pipelines supported by saddles, secondary stresses at the supports are large enough to create critical conditions only near the saddle tips. The highest stress is the circumferential bending stress, which tends to decrease as the internal pressure increases. Therefore, the critical condition is usually with the pipe full but at zero pressure. This stress can be calculated from:

$$S_{cs} = k\, P/t^2 \cdot \log_e (R/t) \qquad (1)$$

where:
- S_{cs} = local bending stress at saddle (psi)
- k = 0.02 - 0.00012 *(A - 90)* (contact angle factor)
- A = contact angle (degrees) See Preceding Illustration
- P = total saddle reaction (lbs)
- *R* = pipe radius (in.)
- *t* = pipe wall thickness (in.)

If there is a longitudinal stress present near the saddle tips, such as a thermal stress and/or the beam bending stress **at that depth on the pipe**, designate its calculated value as S_{ls}. Then calculate the effective stress, S_e:

$$S_e = (S_{cs}^{\,2} + S_{ls}^{\,2} - S_{cs}S_{ls})^{1/2} \qquad (2)$$

This stress (S_e) must not exceed the yield point. It is not necessary to apply a safety factor because tests have shown that, since this is a very localized condition, the resulting design will have a safety factor of approximately two.

The bending stress when the pipe is under pressure can be found by multiplying S_{cs} by a reduction factor (RF) calculated from :

$$RF = (\tanh A)/A \qquad (3)$$

where:
- *A* = 1.1 $(R/t)\,(S_h/E)^{1/2}$
- S_h = hoop stress (psi)
- *E* = Modulus of elasticity (psi) (30,000,000 for steel)

tanh denotes hyperbolic tangent

The hoop stress should be the sum of the membrane stress caused by pressure (usually tension) and the membrane stress at the tip of the cradle caused by the supported load (usually compression). It must be added to the reduced bending stress to get the total circumferential stress. But it is usually not necessary to make this calculation because the zero pressure condition controls. The constant of 1.1 in the reduction factor was experimentally calibrated for a 150 degree saddle and is considered reasonable for a 120 degree saddle.

Of course, as with all support systems, the maximum beam bending stress for the pipe span must be calculated and limited to a suitable allowable stress. It is usually not necessary to add the beam bending stress at the bottom of the pipe at the support (e.g., at an intermediate support in a continuous span arrangement) to a secondary saddle stress, as was sometimes done in past procedures, because Stokes has shown that these stresses are much smaller than those given by Equation 1. As mentioned previously, if the pipe is under pressure and the ends are restrained, the Poisson-ratio effect of the hoop stress ($0.30S_h$) must be added to the beam flexure stress (S_f). The total longitudinal stress (S_t) is taken as:

$$S_t = S_f + 0.30S_h \qquad (4)$$

EXAMPLE —

42-in. dia. by $^5/_{16}$ in. wall pipe, A283 Grade C steel (FY =30,000 psi), 54’ span, wt. of pipe & water = 40,000 lb, total radial reaction on 120 degree saddle, longitudinal stress of 3,000 psi compression (thermal plus bending at saddle tips)

k = .02 - .0012(120-90) = .0164

S_{cs} = $.0164 \times \dfrac{40000}{(.3125)^2} \log_e(21/.3125) = 28300$ psi

S_{ls} = - 3000 psi

S_e = $1000\,[28.3^2 + 3.0^2 - (-3.0 \times 28.3)]^{1/2} = 29900$ psi

29900 < 30000 O.K.

Beam stresses must still be checked by Equation 4.

The flexure stress S_f should be calculated in the usual manner. In single spans, this stress is maximum at the center between supports and may be quite small over the support if flexible joints are used at the pipe ends. In multiple-span cases, the flexure stress in rigidly joined pipe will be that indicated by the theory of continuous beams.

Ring Girders

When large diameter steel pipe is laid above-ground, or across ravines or streams, the use of rigid ring girders spaced at relatively long spans for supporting the pipe has been found to be very effective. These girders prevent the distortion of the pipe at the points of support, and thus maintain its ability to act as a beam. Practical considerations generally limit spans from 40’ to 100’.

A satisfactory rational design of this type of construction, based on the elastic theory, was published in Proceedings of the American Society of Civil Engineers, September, 1931, titled “Design of Large Pipe Lines” by Herman Schorer. This reference will also be found in the ASCE “Transactions” 98:101 (1933).

* Ft. D. Stokes, “Stresses in Steel Pipelines at Saddle Supports,” Civil Engineering Transactions, October 1965, The Institution of Engineers, Australia.

Lake Wallenpaupack, Pennsylvania - 8500' of 176" diameter welded steel pipe supported by ring girders. Installed in 1956.

The basic analysis presented therein includes the following stresses:

1. Transverse
   (a) Shear at the point of support.
   (b) Ring tension due to internal pressure.
2. Longitudinal
   (a) Rim bending at point of support.
   (b) Stresses due to pipe acting as a continuous beam.
   (c) Longitudinal shear due to beam action.
   (d) Elongation due to ring tension and temperature stresses.

The basic formulas for ring girder pipe supports are as follows:

Where a = eccentricity of reaction Q/2 from a tangent to center of pipe shell, inches.

c = width of circular girder ring, inches.

f_r = maximum combined ring stress in shell, psi.

f_L = combined maximum longitudinal beam stress, psi.

f_{bo} = maximum longitudinal rim bending stress in shell, psi.

h = head above bottom of pipe, feet.

p = variable pressure on inside of pipe circumference, psi.

q = unit weight of fluid flowing in pipe, lbs. per cu. ft.

r = mean radius of pipe, inches.

t = thickness of shell, inches.

w = weight of pipe shell, lbs. per sq. ft.

y = distance from neutral axis to extreme fiber, inches.

A_r = area of supporting ring, in a plane along the axis of pipe, sq. in.

D = diameter of pipe = 2r, inches.

I = moment of inertia, inches^4.

L = length of span from center to center of ring girder supports, feet.

M = moment, inch pounds.

Q = total load of pipe shell transmitted by shear to one supporting ring, pounds.

PARTIAL TABULATION OF RING GIRDER SUPPORTED STEEL PIPELINES

Project	Location	Year	Diameter Ins.	Thickness Ins.	D/t	Length Feet	Max. Head Feet	Span Feet
U.S. Bureau of Reclamation	Wyoming	1948	123	–1 ½	197	856	455	150
State of Montana	Toston	1941	84	– ¾	224	450	Low	126
U.S. Bureau of Reclamation	Wash.	1959	186	9/16 – 1 1/16	330	2,500	Low	120
City of Denver	Colo.	1937	78	–1	125	220	Low	105
So. Calif. Edison	B.C. #4	1951	180	– 1	205	650	Low	100
U.S. Bureau of Reclamation	Okla.	1961	61	¾-1	81	2,960	545	100
City of Los Angeles	2nd Aque.	1969	85-78	½-1	170	7,800	1,050	100
Montrose	Colo.	1952	96	5/16 – ½	306	510	Low	96
Crowheart	Wyoming	1938	36	¼– ½	144	220	Low	88
Los Animas	Colo.	1938	123	5/16 – 7/16	393	400	Low	84
City of Los Angeles	1st Aque.	1948	114	½	228	200	175	80
Calgary Power	Canada	1958	96-84	5/16 – ¾	306	1,500	900	80
Penn. Power & Light	Penn.	1955	168	½	336	18,500	Low	80
City of Denver	Colo.	1964	60	½	120	567	Low	80
Natahala	N.C.	1941	120-96	.41 – 2 3/16	294	2,750	1,000	80
State of California	Pastoria	1969	192	9/16	341	1,700	115	80
So. Vietnam	Danhim	1960	60	.36-1.57	167	11,000	High	75
Switzerland	—	—	84	¾	112	—	—	73 ½
City of Denver	Colo.	1936	52	¼	208	1,781	146	77
Omaha	Neb.	1957	42	7/16 – 5/8	96	194	Low	72
Shiprock	N.M.	1959	60	¼– 5/16	240	3,500	Low	72
Port Angeles	Wash.	1961	72	¼– 3/8	288	158	Low	78
City of Los Angeles	Soledad	1944	120	½	240	4,500	260	70
Arizona Public Service	—	1955	60	¼	240	800	Low	69
Lander	Wyoming	1961	36	¼	144	297	Low	68
U.S. Bureau of Reclamation	Imp. Vall.	1948	186	½	372	720	Low	66
Reisseck	Europe	1961	53	3/8	141	6,000	High	65
Worland	Wyoming	1938	72	¼	288	136	Low	64
City of Denver	Colo.	1965	36	5/16	115	368	Low	62 ½
Montrose	Colo.	1947	60	5/16	191	150	Low	60
State of Colorado	—	1936	60	¼– 5/16	240	120	Low	60
So. Calif. Edison	Mammoth	1958	156-90	¾- 2 1/8	208	2,000	1,125	60
Mariposa County	Ariz.	1935	120	¼	480	1,200	Low	60
City of Los Angeles	Gorge	1949	106-92	3/8 – 1 1/16	283	9,000	790	60
U.S. Bureau of Reclamation	Oregon	1936	80	¼– 9/16	320	23,178	268	60
U.S. Bureau of Reclamation	Shasta	1943	180	¾- 2 ½	240	4,000	475	60

There are numerous other installations with spans below 60 feet.

Stress in Pipe Shell

Maximum combined ring stress

$$f_r = \frac{D(w+qh)}{2t\ 144}$$

Combined maximum longitudinal stress

$$f_L = \frac{L^2}{4t}\left[\frac{2w}{D} + \frac{q}{24}\right]$$

This latter equation was developed through the analysis of a pipe supported at the ends acting as a simple beam. In the case of a continuous pipe line supported at intervals and acting as a continuous beam, the values obtained for f_L must be multiplied by 2/3.

Maximum rim bending stress of the shell

$$F_{bo} = \frac{1.82\ (A_r - ct)}{(A_r = 1.56t\sqrt{rt}}\left[\frac{pr}{t}\right]$$

This equation was developed on the assumption that the rim load is symmetrical. Since the rim load is not symmetrical, due to the weight of the water, a good approximation of the true value of f_{bo}, is obtained by substituting the value of f_r as found by equation above in place of $\frac{pr}{t}$.

Total combined longitudinal shell stress

$$f = f_L + f_{bo}$$

Stresses in Supporting Ring Girder

The minimum possible value of the maximum bending moment in the ring girder occurs when

$a = .04r$ outside the neutral axis.

When this is true, the maximum bending moment in the girder is

$$M = .01\ Qr$$

Maximum bending stress

$f_1 =$ (General bending formula)

Maximum ring stress due to shear forces

$f_2 =$

Ring stress due to radical forces

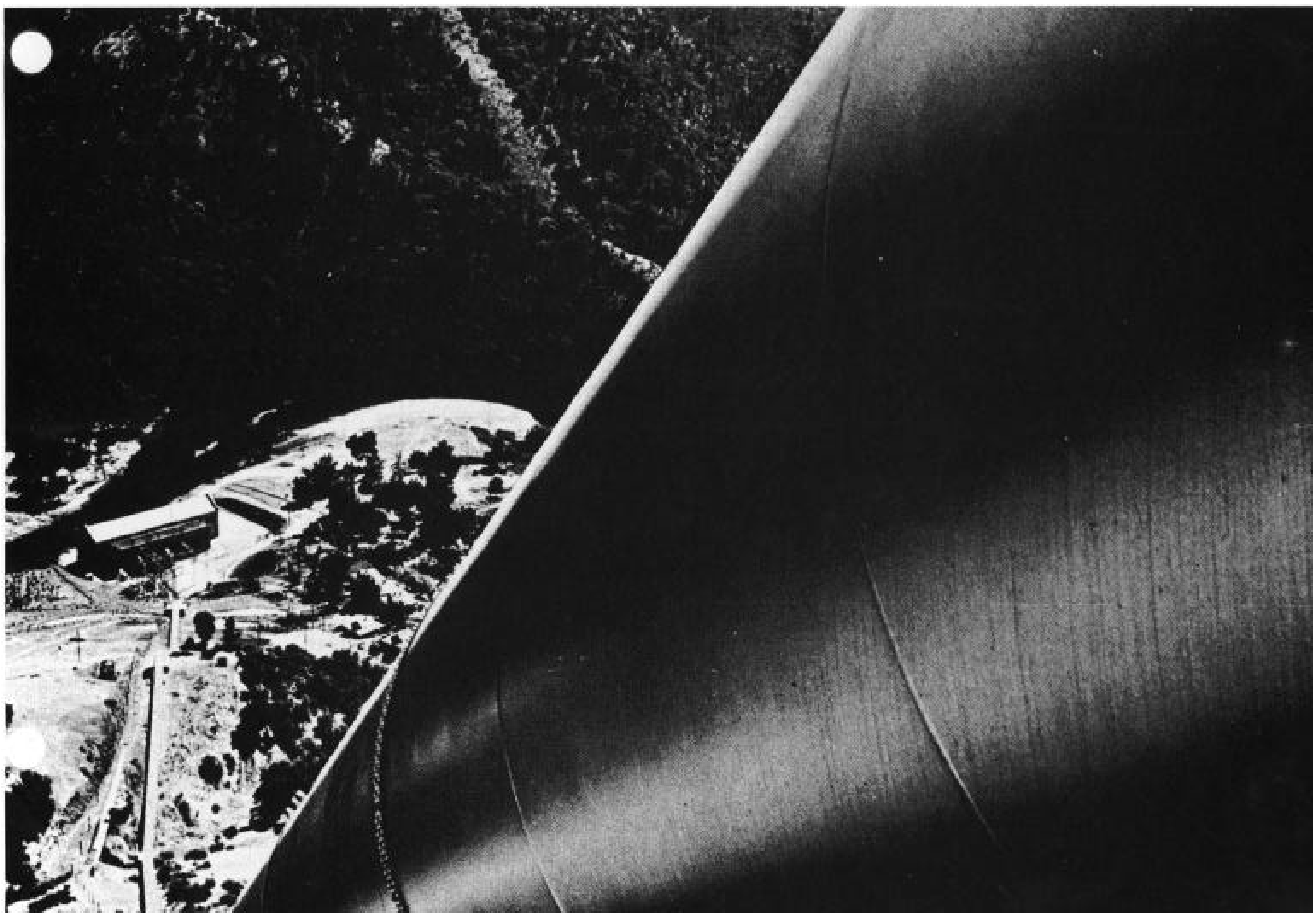
$$f_3 = \frac{pr}{A_r}\left[c + 1.56 \sqrt{rt}\ \frac{(A_r - ct)}{(A_r + 1.56t\sqrt{rt})} \right]$$

Since all of these stresses are combined at the horizontal diameter the total maximum stress in the ring girder is

$$f = f_1 + f_2 + f_3$$

To support the ring girder, a short column on each side of the pipe is attached to the girder and supported on a pier either by direct bearing, roller device, rocker assembly or pin connection. In any event, the design must permit longitudinal movement of the pipe as well as afford adequate support. In addition to proper design of long span, ring-girder-supported steel pipe lines, careful field erection is essential, particularly in regard to alignment and camber, avoidance of movement caused by temperature differences on opposite sides of the pipe, and correct welding procedure. The maximum allowable stress in the ring girder or the pipe shell when the pipe is fully loaded is usually 10,000 p.s.i.

Electra Penstock Pacific Gas & Electric Co., Electra, California. 90 inches through 120 inches diameter, 7/l 6-inch through 3/4-inch steel plate, 1200-foot head.



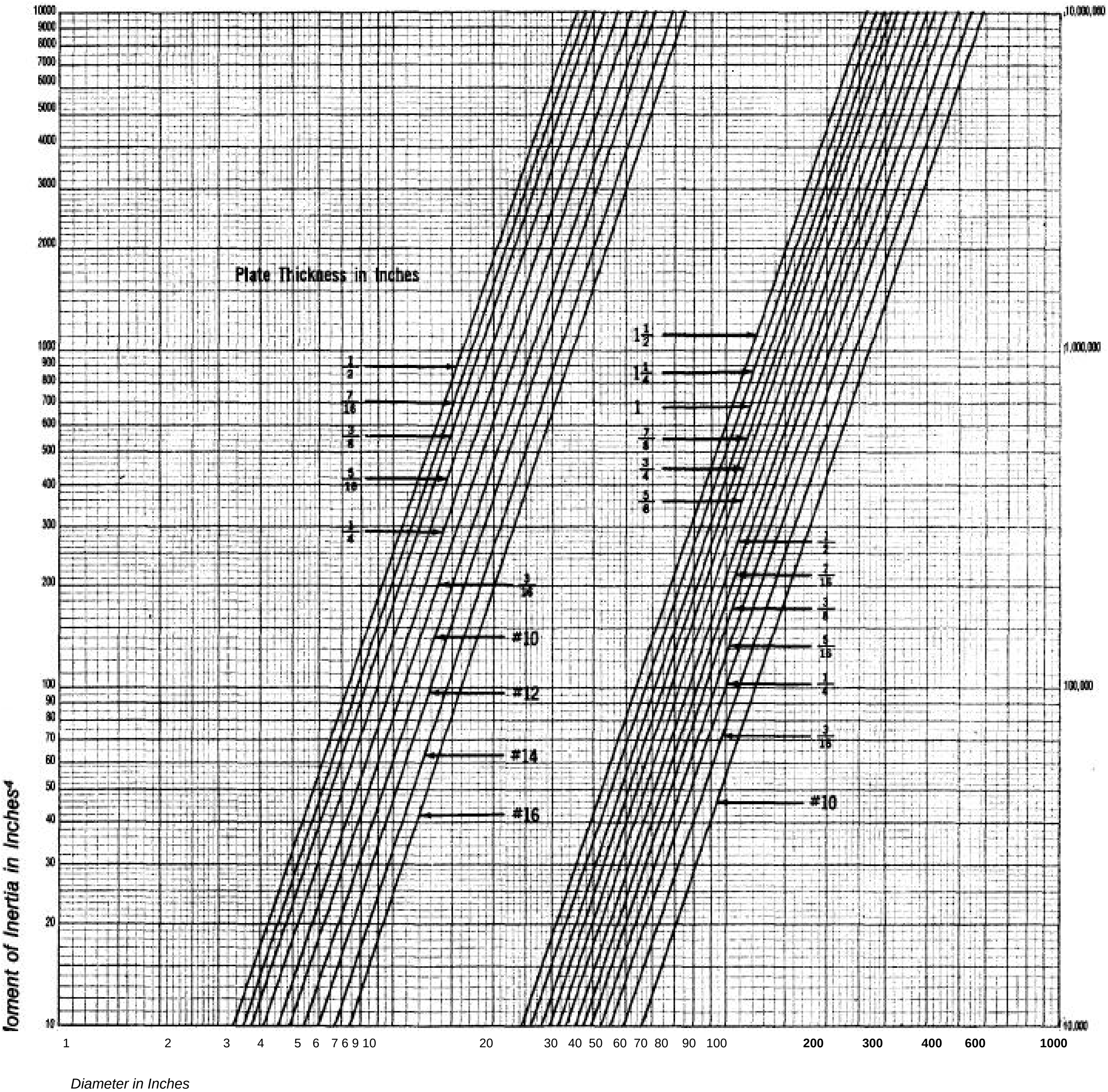
PARTIAL TABULATION OF ABOVE-GROUND STEEL WATER PIPE INSTALLATIONS

Owner & Location	Length	Diameter	Head	Year Installed
City of Los Angeles Aqueduct				
Nine Mile Siphon	1415’	9’6”	175’	1912
No name Siphon	2016’	9’3”	365’	1912
Sand Canyon Siphon	890’	8’6”	455’	1912
Grapevine Siphon	2339’	9’3”	355’	1912
Jawbone Siphon	8095’	10’0” to 7’6”	850’	1912
Pine Tree Siphon	3841’	9’0”	480’	1912
Antelope Siphon	15,597’	10’0”	200’	1912
Deadman Siphon	3430’	11’0”	245’	1913
Little Lake Siphon	3800’	78”	800’	1969
Soledad Siphon	8941’	11’0” to10’0”	260’	1913
Quigley Siphon	612’	11’0”	67’	1913
Placerita Siphon	1572’	11’0”	105’	1913
Haiwee Penstock	1600’	108” to 102”	—	1917
Power House #1 Penstock	8657’	84” to 54”	938’	1916 & 1928
Power House #2 Penstock	4290’	84”to 72”	540’	1921 & 1932
San Fernando Penstock	4600’	8’3” to 7’5”	240’	1922
Bouquet Canyon Line	18,000’	94” to 80”	840’	1933
Owens River Gorge Penstocks	9000’	106” to 92”	790’	1949
Bureau of Reclamation Projects—U.S.				
Owyhee & Snively Siphons,Ore.	2500’	126” to 108”	350’	1934
Bully Creek & Fairman Coulee Siphons, Vale, Ore.	7500’	101”	160’	1931
Yakima Project, Wash.	1000’	145”	—	1931
Grand Coulee Discharge Lines	9373’	144”	363’	1947
Numerous Penstocks		—		
Deer Creek	1300’	72”	300’	1940
Shasta Dam	4000’	180”	475’	1943
Imperial Valley	—	186”	—	1948
Cody	—	123”	—	1948
Pole Hill	2040’	96”	1040’	1954
Flat Iron	10,600’	84” — 72”	1392’	1954
Owyhee	1630’	80”	268’	1936
So. Canadian River	2960’	61”	545’	1961
Wahluke	2500’	186”	—	1959
Malheur River	23,178’	80”	268’	1936
City of San Francisco Hetch Hetchy Aqueduct				
Moccasin Creek Power House	6000’	104” — 54”	—	1924
Red Mountain Bar Siphon	1700’	114”	—	1922
East Bay Municipal Utility District				
Mokelumne Aqueduct	30 Miles	54”—68”—88”	500’	1924,1947& 1963
Southern California Edison Co.				
Big Creek 2-A Penstock	6678’	108”—66”	2418’	1928
Big Creek #8 Penstock	2740’	96”—72”	720’	1921
Big Creek #3 Penstock	4200’	78”—72”	820’	1921
Kern River #3 Penstock	5000’	84”—60”	821’	1920
Little Brush Creek Siphon	1170’	114”—96”	300’	1920
Big Creek #4	650’	180”	—	1951
Mammoth Pool	2000’	156”—90”	1125’	1958
Pacific Gas & Electric Co.				
Hat Creek Power House #1	1600’	120”—96”	217’	1924
Drum Power House	6272’	72”	1375’	1920
Kerckhoff Power House	3000’	96”—84”	—	1920
Pit 5	5600’	102”—90”	630’	1943
Electra	3400’	114”—90”	1268’	1948
Drum	3754’	72”	250’	1928
Salt Springs	1004’	129”—72”	700’	1930
Cresta	1500’	144”	400’	1948
El Dorado	2038’	60”	400’	1946
Colgate	1500’	96”—66”	1350’	1948
Rock Creek	1725’	144”—126”	650’	1948
Bear River	4685’	72”—46”	2000’	1950
Pit 4	1684’	144”	450’	1953

PARTIAL TABULATION OF ABOVE-GROUND STEEL WATER PIPE INSTALLATIONS

Owner & Location	Length	Diameter	Head	Year Installed
Maricopa County Municipal Water Conservation District (Arizona)				
Steel Pipe Flumes	1200′	120″	Gravity	1935
Utah Power & Light Co				
Olmsted Plant	14,500′	102″	Gravity	1948
Northwestern Power & Light Co				
Glines Canyon Penstock	2000′	120″	120’	1923
Portland Railway, Light & Power Co				
Oak Grove Ore	32,000′	108″	925’	1923
California-Oregon Power				
Lemolo #2	3847′	126″—88″	—	1954
California-Electric Power				
Bishop Creek	3300′	60″	60’	1960
Metropolitan Water District				
5 Pumping Plants	18,000′	120″—72″	440’	1936, 1954 & 1962
Calgary Power				
Spray River	1500′	96″—84″	900’	1958
Cerro De Pasco Corp				
Padcarthonbo (Peru)	4600′	80″—74″	1730’	1957
Reisseck-Kreuzeck				
Europe	13,740′	53″—37″	5800’	1961
Northern New York Utilities				
South Edwards #2	—	120″	—	1920
(laid continuously directly on top of ground)				
Platte Valley Public Power	1200′	160″	228’	1935
Penn. Power & Light	18,500′	168″	—	1955
State of Montana	450′	84″	—	1941
City of Denver	220′	78″	—	1937
City of Denver	9197′	111″—52″	—	1936
City of Denver	935′	96″—36″	—	1964
State of Colorado	120′	60″	—	1936
City of Tacoma	1356′	63″	400’	1932
Crowheart Wyom	220′	36″	—	1938
L.A. Co. Flood Control District	1200′	123″	248’	1938
So. Vietman	14,844′	60″	3017’	—
Montrose Colo	660′	96″—60″	—	1947-52
Los Animas Colo	400′	123″	—	1938
Shiprock NM	3500′	60″	—	1959
City of Pasadena	800′	38″	500’	1948
Arizona Public Service	800′	60″	—	1955
Polson, Mont	1890′	48″	330’	1939
Adelaide Australia	150,000′	58″	520’	1951
Nantahala, NC	2750′	120″—96″	1000’	1941
Labrador	2052′	162″	290’	1960
Ontario Paper	700′	216″	117’	1938
B. C. Electric	6300′	75″	1200’	1948
B. C. Electric	1760′	70″	1992’	—
Niagara Mohawk Power	10,235′	84″	218’	1927
Creede, Colo	700′	84″	—	1934
Lander, Wyo	297′	36″	—	1961
U.S.B.R.— San Luis	9300′	210″—84″		1964
U.S.B.R. —Delta	2800′	180″		1965
State of California—Buena Vista	4673′	108″	350’	1967
State of California—Wind Gap	10,000′	109″	285’	1967
State of California—Oso	9000′	109″	230’	1967
State of California—Wheeler Ridge	10,000′	109″	350’	1967
State of California—Pastoria	1642′	192″	230’	1967
City of Los Angeles—Little Lake	104,897′	78″	600’	1968
Penn. Power Company	18,500′	168″		1955
Lake Waulenpaupack Pa	8500′	176″		1956

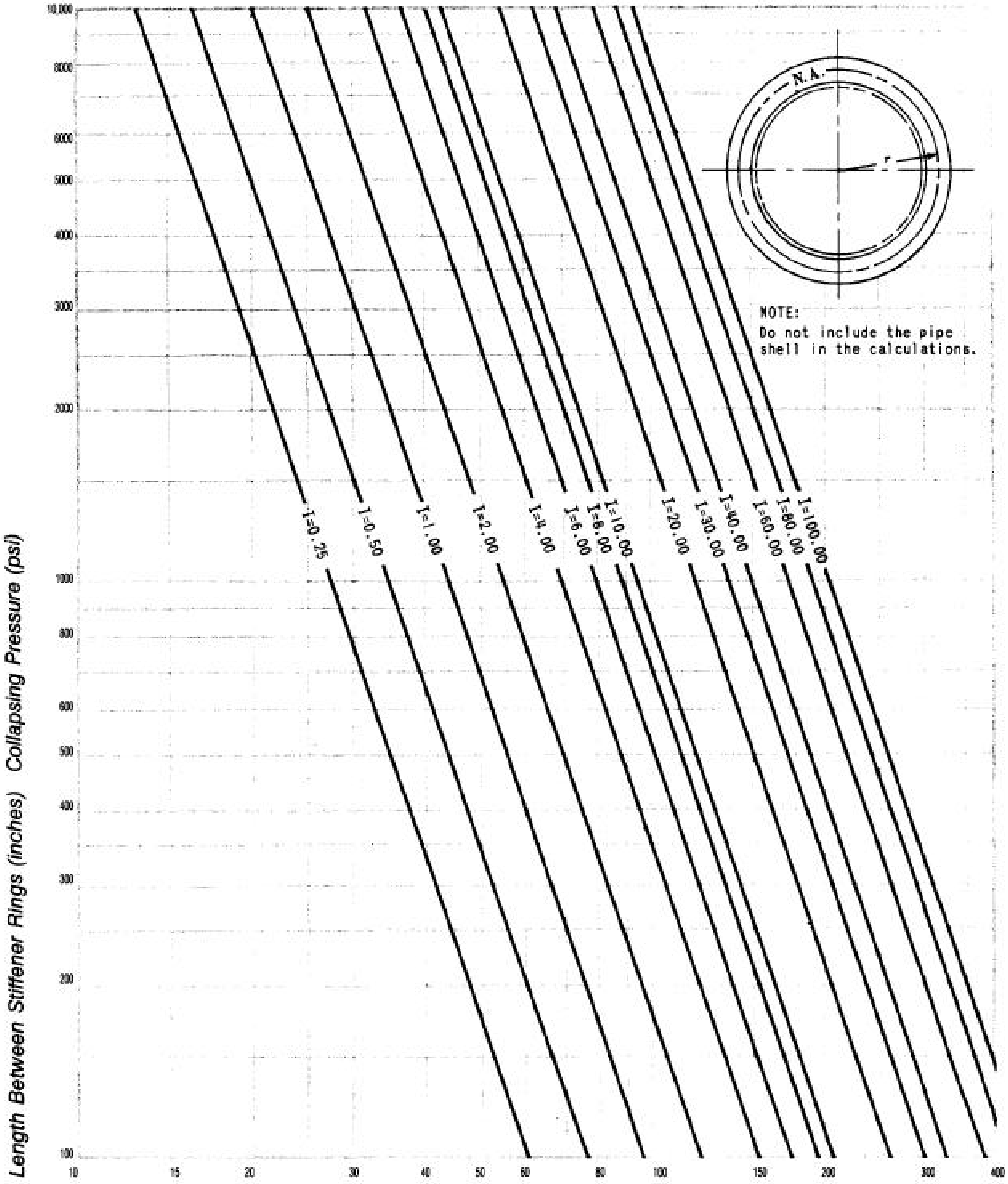
MOMENT OF INERTIA OF PIPE, TAKEN ABOUT DIAMETER AS AXIS



$I = .04909\ (d_1^4 - d_2^4)$
d_1 = Outside diameter in Inches
d_2 = Inside diameter in Inches

Approximate formula
$I = .4\ t\ d^3$
t = Thickness in Inches
d = Average diameter in Inches

REQUIRED MOMENT OF INERTIA STIFFENER RINGS TO PREVENT PIPE COLLAPSE FROM EXTERNAL PRESSURE



Radius "r" (inches)

$$I = \frac{0.37 r^3 L P}{E}$$

Where: I = Moment of inertia of the cross section of the ring ($inches^4$)

P = Collapsing pressure (psi)

r = Radius of ring to N.A. (inches

L = Distance between rings (inches)

E = 29,000,000 psi

The above formula will give ring about 10% more strength than pipe shell.

Ref.: Strength of thin cylindrical shells under external pressure by Saunders & Windenburg — Trans. A.S.M.E. — A.P.M. 53-17a Vol. 53, 1931.

Las Vegas, NV 108″ I.D. Pipe

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Arranged by subject chronologically

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Standards and Specifications

ANSIIAWWA C200	Steel Water Pipe 6 in. and Larger.
ANSIIAWWA C203	Coal-Tar Protective Coatings and Linings - Enamel and Tape - Hot Applied
ANSVAWWA C205	Cement-Mortar Protective Lining and Coating for Steel Water Pipe - 4 in. and larger - Shop Applied.
ANSVAWWA C206	Field Welding of Steel Water Pipe
ANSVAWWA C207	Steel Pipe Flanges for Waterworks Service - Sizes 4 in. through 144 in.
ANSIIAWWA C208	Dimensions for Fabricated Steel Water Pipe Fittings.
ANSIIAWWA C209	Cold-Applied Tape Coatings for the Exterior of Special Sections, Connections, and Fittings for Steel Water Pipelines.
ANSVAWWA C210	Liquid-Epoxy Coating Systems for the Interior and Exterior of Steel Water Pipelines.
ANSUAWWA C213	Fusion-Bonded Epoxy Coating for the Interior and Exterior of Steel Water Pipelines. Tape Coating
ANSVAWWA C214	for the Exterior of Steel Water Pipelines. Extruded Polyolefin Coatings for the Exterior of Steel
ANSVAWWA C215	Water Pipelines. Heat Shrinkable Cross-Linked Polyolefin Coatings for the Exterior of Special Sections
ANSVAWWA C216	Connections and Fittings for Steel Water Pipelines.
	Cold-Applied Petrolatum Tape and Petrolatum Wax Tape Coatings for the Exterior of Special Sections, Connections, and Fittings for Buried Steel Water Pipelines.
ANSVAWWA C218	Coating for Exterior of Above-ground Steel Water Pipelines and Fittings.
ANSUAWWA C219	Bolted, Sleeve-Type Couplings for Plain-End Pipe.
ANSVAWWA C220	Stainless-Steel Pipe, 4 in. and Larger. Standard Specification for Pipe, Steel, Black and Hot-
ASTM AS3	Dipped, Zinc-Coated Welded and Seamless.
ASTM AI34	Standard Specification for Pipe, Steel, Electric-Fusion (Arc)-Welded (Sizes NPS 16 and over)
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ASTM AI35	Standard Specification for Electric-Fusion (ARC)-Welded Steel Pipe (NPS 4 and over) Standard
ASTM AI 39/I 39M	Specification for Low and Intermediate Tensile Strength Carbon Steel Plates. Standard
ASTM A2831283M	Specification for Steel Sheet and Strip, Carbon, Hot-Rolled of Structural Quality. Standard
ASTM A5701570M	Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality.
ASTM A5721572M	
ASTM A607/607M	Standard Specification for Steel, Sheet and Strip, High Strength, Low-Alloy, Columbium or Vanadium, or Both, Hot-Rolled and Cold-Rolled.
ASTM A635/635M	Standard Specification for Steel, Sheet and Strip, Heavy-Thickness Coils, Carbon, Hot-Rolled.
	Standard Specification for Steel, Sheet and Strip, Heavy Thickness Coils, Carbon, Hot-Rolled, Structural Quality.
ASTM A9071907M	
ASTM A936/936M	Standard Specification for Steel, Sheet and Strip, Heavy Thickness Coils, High Strength, Low-Alloy, Hot-Rolled with Improved Formability.