WELDED STEEL PIPE

Steel Plate Engineering Data • Volume 3
Acknowledgements

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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

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City of San Francisco, California - Bay crossing of the Hetch Hetchy Aqueduct. 66 inches diameter, 3/8-inch and 1/2-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:
<table>
<thead>
<tr>
<th>YEAR INSTALLED</th>
<th>LOCATION</th>
<th>DIAMETER INCHES</th>
<th>FOOTAGE</th>
<th>THICKNESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1858</td>
<td>Railroad Flat, Cal.</td>
<td>22&quot;-11&quot;</td>
<td>——</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1860</td>
<td>New York, N. Y.</td>
<td>90&quot;</td>
<td>1,000'</td>
<td>1/2&quot;</td>
</tr>
<tr>
<td>1863</td>
<td>San Francisco, Cal.</td>
<td>37&quot;-30&quot;</td>
<td>27,280'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1870</td>
<td>San Francisco, Cal.</td>
<td>30&quot;</td>
<td>42,240'</td>
<td>12 Ga.</td>
</tr>
<tr>
<td>1870</td>
<td>Magalia, Cal.</td>
<td>30&quot;</td>
<td>——</td>
<td>10 Ga</td>
</tr>
<tr>
<td>1870</td>
<td>Pioche, Nev.</td>
<td>5&quot;</td>
<td>8,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1871</td>
<td>San Francisco, Cal.</td>
<td>22&quot;</td>
<td>2,105'</td>
<td>9 Ga.</td>
</tr>
<tr>
<td>1873</td>
<td>Santa Rosa, Cal.</td>
<td>11&quot;-9&quot;</td>
<td>10,000'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1873</td>
<td>Virginia City, Nev.</td>
<td>12&quot;</td>
<td>37,000'</td>
<td>5/16&quot;-1/16&quot;</td>
</tr>
<tr>
<td>1874</td>
<td>Carson City, Nev.</td>
<td>12&quot;-7&quot;</td>
<td>10,000'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1874</td>
<td>Pittsburgh, Pa.</td>
<td>50&quot;</td>
<td>2,900'</td>
<td>—</td>
</tr>
<tr>
<td>1875</td>
<td>San Francisco, Cal.</td>
<td>22&quot;</td>
<td>2,226'</td>
<td>9 Ga.</td>
</tr>
<tr>
<td>1878</td>
<td>Texas Creek, Cal.</td>
<td>17&quot;</td>
<td>4,000'</td>
<td>9-14 Ga.</td>
</tr>
<tr>
<td>1880</td>
<td>Los Angeles, Cal.</td>
<td>44&quot;</td>
<td>——</td>
<td>—</td>
</tr>
<tr>
<td>1880</td>
<td>San Fernando, Cal.</td>
<td>8&quot;</td>
<td>——</td>
<td>—</td>
</tr>
<tr>
<td>1881</td>
<td>Lawrence, Mass.</td>
<td>77&quot;</td>
<td>——</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1882</td>
<td>San Francisco, Cal.</td>
<td>22&quot;</td>
<td>6,800'</td>
<td>1/8&quot;-3/16&quot;</td>
</tr>
<tr>
<td>1882</td>
<td>San Francisco, Cal.</td>
<td>30&quot;</td>
<td>3,480'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1882</td>
<td>Longmont, Colo.</td>
<td>6&quot;</td>
<td>23,000'</td>
<td>—</td>
</tr>
<tr>
<td>1883</td>
<td>Fort Collins, Colo.</td>
<td>10&quot;</td>
<td>18,000'</td>
<td>3/16&quot;</td>
</tr>
<tr>
<td>1884</td>
<td>San Francisco, Cal.</td>
<td>33&quot;</td>
<td>2,409'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1885</td>
<td>San Francisco, Cal.</td>
<td>30&quot;</td>
<td>13,409'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1885</td>
<td>San Francisco, Cal.</td>
<td>44&quot;</td>
<td>90,000'</td>
<td>6 &amp; 7 Ga.</td>
</tr>
<tr>
<td>1887</td>
<td>Riverside, Cal.</td>
<td>24&quot;</td>
<td>45,000'</td>
<td>—</td>
</tr>
<tr>
<td>1887</td>
<td>Pasadena, Cal.</td>
<td>6&quot;</td>
<td>——</td>
<td>—</td>
</tr>
<tr>
<td>1888</td>
<td>San Francisco, Cal.</td>
<td>22&quot;</td>
<td>12,000'</td>
<td>—</td>
</tr>
<tr>
<td>1888</td>
<td>Pasadena, Cal.</td>
<td>22&quot;</td>
<td>18,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1888</td>
<td>Sierra Madre, Cal.</td>
<td>4&quot; &amp; 6&quot;</td>
<td>15,000'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1888</td>
<td>Altadena, Cal.</td>
<td>8&quot;</td>
<td>1,200'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1889</td>
<td>Nephi, Utah</td>
<td>3&quot;</td>
<td>1,500'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1889</td>
<td>Alhambra, Cal.</td>
<td>7&quot;</td>
<td>900'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1889</td>
<td>San Francisco, Cal.</td>
<td>44&quot;</td>
<td>4,878'</td>
<td>3 Ga.</td>
</tr>
<tr>
<td>1889</td>
<td>Pasadena, Cal.</td>
<td>13&quot;</td>
<td>6,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1890</td>
<td>San Jose, Cal.</td>
<td>18&quot;</td>
<td>31,000'</td>
<td>12 Ga.</td>
</tr>
<tr>
<td>1890</td>
<td>Santa Cruz, Cal.</td>
<td>14&quot;</td>
<td>——</td>
<td>9 Ga.</td>
</tr>
<tr>
<td>1890</td>
<td>Detroit, Mich.</td>
<td>72&quot;</td>
<td>——</td>
<td>—</td>
</tr>
<tr>
<td>1890</td>
<td>Redlands, Cal.</td>
<td>8&quot;</td>
<td>6,000'</td>
<td>—</td>
</tr>
<tr>
<td>1890</td>
<td>Walla Walla, Wash.</td>
<td>20&quot;-6&quot;</td>
<td>500,000'</td>
<td>7-14 Ga.</td>
</tr>
<tr>
<td>1891</td>
<td>Newark, N. J.</td>
<td>48&quot;</td>
<td>111,800'</td>
<td>1/4&quot;-3/8&quot;</td>
</tr>
<tr>
<td>1891</td>
<td>Newark, N. J.</td>
<td>36&quot;</td>
<td>23,980'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1891</td>
<td>Pittsburgh, Pa.</td>
<td>50&quot;</td>
<td>3,600'</td>
<td>5/8&quot;</td>
</tr>
<tr>
<td>1891</td>
<td>The Dalles, Ore.</td>
<td>10&quot;</td>
<td>8,000'</td>
<td>10 Ga.</td>
</tr>
<tr>
<td>1891</td>
<td>Pocatello, Ida.</td>
<td>12&quot;</td>
<td>6,000'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1892</td>
<td>Pasadena, Cal.</td>
<td>8&quot; &amp; 4&quot;</td>
<td>3,000'</td>
<td>—</td>
</tr>
<tr>
<td>1892</td>
<td>Butte, Mont.</td>
<td>20&quot;</td>
<td>3,114'</td>
<td>—</td>
</tr>
<tr>
<td>1893</td>
<td>Syracuse, N. Y.</td>
<td>54&quot;</td>
<td>6,500'</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1893</td>
<td>Rochester, N. Y.</td>
<td>36&quot;-38&quot;</td>
<td>136,000'</td>
<td>—</td>
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<tr>
<td>1894</td>
<td>Portland, Ore.</td>
<td>42&quot;-33&quot;</td>
<td>132,000'</td>
<td>5/16&quot;-6 Ga.</td>
</tr>
<tr>
<td>1894</td>
<td>Passaic Valley, N. J.</td>
<td>30&quot;</td>
<td>12,300'</td>
<td>5/16&quot;</td>
</tr>
<tr>
<td>1895</td>
<td>Pittsburgh, Pa.</td>
<td>60&quot;</td>
<td>49,000'</td>
<td>1/2&quot;</td>
</tr>
<tr>
<td>1895</td>
<td>Altadena, Cal.</td>
<td>12&quot;</td>
<td>5,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1895</td>
<td>Pasadena, Cal.</td>
<td>8&quot;</td>
<td>1,200'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1895</td>
<td>Vancouver, B. C.</td>
<td>22&quot; &amp; 16&quot;</td>
<td>52,000'</td>
<td>12 Ga.</td>
</tr>
<tr>
<td>1895</td>
<td>San Francisco, Cal.</td>
<td>30&quot;</td>
<td>4,090'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1895</td>
<td>Kearney, N. J.</td>
<td>42&quot;</td>
<td>8,800'</td>
<td>—</td>
</tr>
<tr>
<td>1896</td>
<td>Minneapolis, Minn.</td>
<td>48&quot;</td>
<td>31,680'</td>
<td>—</td>
</tr>
<tr>
<td>1896</td>
<td>Newark, N. J.</td>
<td>48&quot; &amp; 42&quot;</td>
<td>111,600'</td>
<td>—</td>
</tr>
<tr>
<td>1896</td>
<td>Passaic Valley, N. J.</td>
<td>42&quot;</td>
<td>8,700'</td>
<td>1/4&quot;-3/8&quot;</td>
</tr>
<tr>
<td>1896</td>
<td>New Bedford, Mass.</td>
<td>48&quot;</td>
<td>42,000'</td>
<td>5/16&quot;</td>
</tr>
<tr>
<td>1896</td>
<td>Bayonne, N. J.</td>
<td>30&quot;</td>
<td>44,000'</td>
<td>—</td>
</tr>
<tr>
<td>1896</td>
<td>New Westminster, B. C.</td>
<td>14&quot;</td>
<td>70,000'</td>
<td>—</td>
</tr>
<tr>
<td>1896</td>
<td>New York, N. Y.</td>
<td>72&quot;</td>
<td>——</td>
<td>—</td>
</tr>
<tr>
<td>1897</td>
<td>Minneapolis, Minn.</td>
<td>50&quot;</td>
<td>16,605'</td>
<td>7/16&quot;</td>
</tr>
<tr>
<td>1897</td>
<td>Ogden, Utah</td>
<td>72&quot;</td>
<td>4,600'</td>
<td>—</td>
</tr>
<tr>
<td>1897</td>
<td>Patterson, N. J.</td>
<td>42&quot;</td>
<td>18,600'</td>
<td>5/16&quot;</td>
</tr>
<tr>
<td>1897</td>
<td>Jersey City, N. J.</td>
<td>48&quot;</td>
<td>——</td>
<td>—</td>
</tr>
<tr>
<td>1898</td>
<td>Red Bluff, Cal.</td>
<td>7&quot;</td>
<td>9,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>YEAR INSTALLED</td>
<td>LOCATION</td>
<td>DIAMETER INCHES</td>
<td>FOOTAGE</td>
<td>THICKNESS</td>
</tr>
<tr>
<td>----------------</td>
<td>------------------------------</td>
<td>-----------------</td>
<td>-------------</td>
<td>----------------</td>
</tr>
<tr>
<td>1898</td>
<td>Duluth, Minn.</td>
<td>42&quot;</td>
<td>30,500'</td>
<td>1/4&quot;-.1/2&quot;</td>
</tr>
<tr>
<td>1898</td>
<td>Allegheny, Pa.</td>
<td>50&quot;</td>
<td>8,000'</td>
<td>—</td>
</tr>
<tr>
<td>1898</td>
<td>Albany, N. Y.</td>
<td>48&quot;</td>
<td>154'</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1899</td>
<td>Lawrence, Mass.</td>
<td>108&quot;</td>
<td>44,600'</td>
<td>1/4&quot;-.7/16&quot;</td>
</tr>
<tr>
<td>1899</td>
<td>Passaic Valley, N. J.</td>
<td>51&quot;</td>
<td>32,000'</td>
<td>1/4&quot; 12 Ga.-1/2&quot;</td>
</tr>
<tr>
<td>1899</td>
<td>Seattle, Wash.</td>
<td>42&quot;</td>
<td>5,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1899</td>
<td>Kern, Cal.</td>
<td>60&quot;-48&quot;</td>
<td>17,000'</td>
<td>—</td>
</tr>
<tr>
<td>1899</td>
<td>Newark, N. J.</td>
<td>51&quot;-48&quot;</td>
<td>4,400'</td>
<td>—</td>
</tr>
<tr>
<td>1899</td>
<td>Pittsburgh, Pa.</td>
<td>48&quot;</td>
<td>3,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1899</td>
<td>Redlands, Cal.</td>
<td>4&quot;</td>
<td>420'</td>
<td>7 Ga.</td>
</tr>
<tr>
<td>1900</td>
<td>San Francisco, Cal.</td>
<td>36&quot;</td>
<td>2,500'</td>
<td>3/16&quot;</td>
</tr>
<tr>
<td>1900</td>
<td>Victor, Colo.</td>
<td>29&quot;</td>
<td>600'</td>
<td>—</td>
</tr>
<tr>
<td>1900</td>
<td>Marquette, Mich.</td>
<td>42&quot;</td>
<td>61,000'</td>
<td>12 Ga.</td>
</tr>
<tr>
<td>1900</td>
<td>Butte, Mont.</td>
<td>26&quot;</td>
<td>33,910'</td>
<td>5/16&quot;</td>
</tr>
<tr>
<td>1900</td>
<td>Passaic Valley, N. J.</td>
<td>42&quot;</td>
<td>18,600'</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1901</td>
<td>Atlantic City, N. J.</td>
<td>30&quot;</td>
<td>27,000'</td>
<td>—</td>
</tr>
<tr>
<td>1901</td>
<td>Seattle, Wash.</td>
<td>42&quot;</td>
<td>1,120'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1901</td>
<td>Glendora, Cal.</td>
<td>8&quot;</td>
<td>3,600'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1902</td>
<td>Altadena, Cal.</td>
<td>8&quot;</td>
<td>12 Ga.</td>
<td></td>
</tr>
<tr>
<td>1902</td>
<td>Montebello, Cal.</td>
<td>20&quot;</td>
<td>93,000'</td>
<td>5/16&quot;</td>
</tr>
<tr>
<td>1902</td>
<td>Jersey City, N. J.</td>
<td>72&quot;</td>
<td>4,000'</td>
<td>1/2&quot;</td>
</tr>
<tr>
<td>1902</td>
<td>Pittsburgh, Pa.</td>
<td>48&quot;</td>
<td>9,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1902</td>
<td>Sacramento, Cal.</td>
<td>24&quot;</td>
<td>40,000'</td>
<td>5/16&quot;</td>
</tr>
<tr>
<td>1902</td>
<td>Newark, N. J.</td>
<td>60&quot;-48&quot;</td>
<td>35,000'</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1902</td>
<td>Kansas City, MO.</td>
<td>36&quot;</td>
<td>29,000'</td>
<td>14 &amp; 16 Ga.</td>
</tr>
<tr>
<td>1903</td>
<td>Troy, N. Y.</td>
<td>33&quot;</td>
<td>23,716'</td>
<td>1/4&quot;-.3/8&quot;</td>
</tr>
<tr>
<td>1903</td>
<td>Chino, Cal.</td>
<td>12&quot;-4&quot;</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>1903</td>
<td>Schenectady, N. Y.</td>
<td>36&quot;</td>
<td>4,000'</td>
<td>—</td>
</tr>
<tr>
<td>1904</td>
<td>Bayonne, N. J.</td>
<td>30&quot;</td>
<td>15,000'</td>
<td>—</td>
</tr>
<tr>
<td>1904</td>
<td>Astoria, L. I. N. Y.</td>
<td>60&quot;</td>
<td>7,920'</td>
<td>—</td>
</tr>
<tr>
<td>1904</td>
<td>Erie, Pa.</td>
<td>60&quot;</td>
<td>6,000'</td>
<td>12 Ga.</td>
</tr>
<tr>
<td>1904</td>
<td>Toronto, Ont., Canada</td>
<td>72&quot;</td>
<td>16,800'</td>
<td>10 Ga.</td>
</tr>
<tr>
<td>1904</td>
<td>Pasadena, Cal.</td>
<td>8&quot;</td>
<td>1,688'</td>
<td>10 Ga.</td>
</tr>
<tr>
<td>1904</td>
<td>Red Bluff, Cal.</td>
<td>12&quot;</td>
<td>1,600'</td>
<td>1/2&quot;-3/16&quot;</td>
</tr>
<tr>
<td>1904</td>
<td>San Bernardino, Cal.</td>
<td>20&quot;</td>
<td>16,000'</td>
<td>3/16&quot;</td>
</tr>
<tr>
<td>1905</td>
<td>Los Angeles, Cal.</td>
<td>98&quot;-16&quot;</td>
<td>1,080,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1905</td>
<td>Tillamook, Ore.</td>
<td>10&quot;</td>
<td>24,000'</td>
<td>—</td>
</tr>
<tr>
<td>1905</td>
<td>St. Louis, MO.</td>
<td>84&quot;</td>
<td>18,960'</td>
<td>—</td>
</tr>
<tr>
<td>1905</td>
<td>Passaic Valley, N. J.</td>
<td>48&quot;-42&quot;</td>
<td>35,000'</td>
<td>—</td>
</tr>
<tr>
<td>1905</td>
<td>Pittsburgh, Pa.</td>
<td>50&quot; &amp; 30&quot;</td>
<td>28,500'</td>
<td>—</td>
</tr>
<tr>
<td>1905</td>
<td>Pasadena, Cal.</td>
<td>12&quot;</td>
<td>1,500'</td>
<td>—</td>
</tr>
<tr>
<td>1905</td>
<td>Altadena, Cal.</td>
<td>8&quot; &amp; 4&quot;</td>
<td>5,000'</td>
<td>12 Ga.</td>
</tr>
<tr>
<td>1905</td>
<td>Lynchburg, Va.</td>
<td>30&quot;</td>
<td>11,500'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1905</td>
<td>Wilmington, Del.</td>
<td>48&quot;-43&quot;</td>
<td>20,000'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1905</td>
<td>Paterson, N. J.</td>
<td>48&quot;-42&quot;</td>
<td>11,300'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1905</td>
<td>Cincinnati, Ohio</td>
<td>84&quot;</td>
<td>1,521'</td>
<td>—</td>
</tr>
<tr>
<td>1905</td>
<td>Springfield, Mass.</td>
<td>42&quot;-54&quot;</td>
<td>63,500'</td>
<td>—</td>
</tr>
<tr>
<td>1905</td>
<td>Brooklyn, N. Y.</td>
<td>72&quot;</td>
<td>42,300'</td>
<td>—</td>
</tr>
<tr>
<td>1905</td>
<td>Philadelphia, Pa.</td>
<td>48&quot;-36&quot;</td>
<td>86,980'</td>
<td>—</td>
</tr>
<tr>
<td>1906</td>
<td>Pittsburgh, Pa.</td>
<td>72&quot;-30&quot;</td>
<td>47,000'</td>
<td>—</td>
</tr>
<tr>
<td>1906</td>
<td>New York, N. Y.</td>
<td>72&quot;</td>
<td>125,000'</td>
<td>—</td>
</tr>
<tr>
<td>1906</td>
<td>Honolulu, T. H.</td>
<td>9&quot;</td>
<td>8,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1906</td>
<td>Corona Heights, Cal.</td>
<td>30&quot;</td>
<td>1,400'</td>
<td>7/16&quot;</td>
</tr>
<tr>
<td>1907</td>
<td>Pasadena, Cal.</td>
<td>4&quot;</td>
<td>1,235'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1907</td>
<td>Trinidad, Colo.</td>
<td>15&quot;</td>
<td>184,800'</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1907</td>
<td>Wilmington, Del.</td>
<td>43&quot;-48&quot;</td>
<td>20,340'</td>
<td>7/16&quot;</td>
</tr>
<tr>
<td>1907</td>
<td>Trenton, N. J.</td>
<td>48&quot;</td>
<td>7,000'</td>
<td>5/16&quot;</td>
</tr>
<tr>
<td>1907</td>
<td>Lockport, N. Y.</td>
<td>30&quot;</td>
<td>68,640'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1907</td>
<td>Pittsburgh, Pa.</td>
<td>36&quot;</td>
<td>3,700'</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1907</td>
<td>Vancouver, B. C.</td>
<td>30&quot;-22&quot;</td>
<td>65,000'</td>
<td>1/4&quot;-5/16&quot;</td>
</tr>
<tr>
<td>1907</td>
<td>St. Louis, MO.</td>
<td>84&quot;</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>1907</td>
<td>Montreal, Canada</td>
<td>36&quot;</td>
<td>11,000'</td>
<td>—</td>
</tr>
<tr>
<td>1907</td>
<td>Gary, Indiana</td>
<td>36&quot;</td>
<td>4,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1907</td>
<td>Philadelphia, Pa.</td>
<td>48&quot;-36&quot;</td>
<td>54,000'</td>
<td>12 Ga.</td>
</tr>
<tr>
<td>1907</td>
<td>Canyon, Cal.</td>
<td>36&quot;</td>
<td>1,500'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1908</td>
<td>Springfield, Mass.</td>
<td>42&quot;</td>
<td>75,000'</td>
<td>3/16&quot;</td>
</tr>
<tr>
<td>1908</td>
<td>Missoula, Mont.</td>
<td>6&quot;</td>
<td>20,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1908</td>
<td>Passaic Valley, N. J.</td>
<td>30&quot;</td>
<td>15,400'</td>
<td>3/16&quot;</td>
</tr>
<tr>
<td>1908</td>
<td>Seattle, Wash.</td>
<td>52&quot;-42&quot;</td>
<td>15,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1908</td>
<td>Michigan City, Ind.</td>
<td>30&quot;</td>
<td>4,000'</td>
<td>—</td>
</tr>
<tr>
<td>YEAR INSTALLED</td>
<td>LOCATION</td>
<td>DIAMETER INCHES</td>
<td>FOOTAGE</td>
<td>THICKNESS</td>
</tr>
<tr>
<td>---------------</td>
<td>----------</td>
<td>-----------------</td>
<td>---------</td>
<td>-----------</td>
</tr>
<tr>
<td>1908</td>
<td>Montreal, Canada</td>
<td>36&quot;</td>
<td>25,000'</td>
<td>—</td>
</tr>
<tr>
<td>1908</td>
<td>Philadelphia, Pa.</td>
<td>132&quot;</td>
<td>1,590'</td>
<td>—</td>
</tr>
<tr>
<td>1908</td>
<td>Beaumont, Cal.</td>
<td>40&quot;</td>
<td>16,000'</td>
<td>—</td>
</tr>
<tr>
<td>1909</td>
<td>Seattle, Wash.</td>
<td>51&quot;</td>
<td>7,660'</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1909</td>
<td>Portland, Ore.</td>
<td>48&quot;-24&quot;</td>
<td>17,600'</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1909</td>
<td>Boulder, Colo.</td>
<td>60&quot;</td>
<td>2,640'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1909</td>
<td>Eric, Pa.</td>
<td>56&quot;</td>
<td>5,280'</td>
<td>1/4&quot;-3/16&quot;</td>
</tr>
<tr>
<td>1909</td>
<td>Vancouver, B. C.</td>
<td>24&quot;</td>
<td>73,000'</td>
<td>3/8&quot;-5/16&quot;</td>
</tr>
<tr>
<td>1909</td>
<td>Brooklyn, N. Y.</td>
<td>72&quot;</td>
<td>83,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1910</td>
<td>Ennslcy, Ala.</td>
<td>50&quot;-6&quot;</td>
<td>6,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1910</td>
<td>Pasadena, Cal.</td>
<td>16&quot;</td>
<td>22,000'</td>
<td>7/16&quot;</td>
</tr>
<tr>
<td>1910</td>
<td>Longmont, Colo.</td>
<td>24&quot;</td>
<td>5,000'</td>
<td>—</td>
</tr>
<tr>
<td>1910</td>
<td>New York, N. Y.</td>
<td>48&quot;</td>
<td>16,000'</td>
<td>—</td>
</tr>
<tr>
<td>1910</td>
<td>Pittsburgh, Pa.</td>
<td>36&quot; &amp; 44&quot;</td>
<td>128,000'</td>
<td>—</td>
</tr>
<tr>
<td>1910</td>
<td>Seattle, Wash.</td>
<td>42&quot;-24&quot;</td>
<td>29,600'</td>
<td>3/16&quot;</td>
</tr>
<tr>
<td>1910</td>
<td>New York, N. Y.</td>
<td>36&quot;</td>
<td>11,000'</td>
<td>7/16&quot;-3/4&quot;</td>
</tr>
<tr>
<td>1910</td>
<td>New York, N. Y.</td>
<td>135&quot; &amp; 117&quot;</td>
<td>33,000'</td>
<td>—</td>
</tr>
<tr>
<td>1910</td>
<td>Montrose, Cal.</td>
<td>36&quot; &amp; 26&quot;</td>
<td>5200'</td>
<td>—</td>
</tr>
<tr>
<td>1910</td>
<td>Pittsburgh, Pa.</td>
<td>48&quot;-36&quot;</td>
<td>5,000'</td>
<td>—</td>
</tr>
<tr>
<td>1910</td>
<td>Brooklyn, N. Y.</td>
<td>42&quot;</td>
<td>12,000'</td>
<td>—</td>
</tr>
<tr>
<td>1910</td>
<td>Butte, Mont.</td>
<td>30&quot;</td>
<td>2,000'</td>
<td>5/16&quot;</td>
</tr>
<tr>
<td>1910</td>
<td>Washington, D. C.</td>
<td>42&quot;</td>
<td>2,000'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>Paterson, N. J.</td>
<td>42&quot;-30&quot;</td>
<td>12,000'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>Philadelphia, Pa.</td>
<td>20&quot;</td>
<td>7,700'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>Glendora, Cal.</td>
<td>9&quot;</td>
<td>5,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1911</td>
<td>Los Angeles, Cal.</td>
<td>120&quot;-90&quot;</td>
<td>49,575'</td>
<td>1/4&quot;-1/8&quot;</td>
</tr>
<tr>
<td>1911</td>
<td>Pasadena, Cal.</td>
<td>30&quot;</td>
<td>10,297'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>Denver, Colo.</td>
<td>60&quot;</td>
<td>11,111'</td>
<td>10 Ga.</td>
</tr>
<tr>
<td>1911</td>
<td>Portland, Ore.</td>
<td>36&quot;-44&quot;</td>
<td>130,000'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>Seattle, Wash.</td>
<td>42&quot;-24&quot;</td>
<td>16,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1911</td>
<td>Tacoma, Wash.</td>
<td>48&quot;-39&quot;</td>
<td>7,300'</td>
<td>1/4&quot;-1/2&quot;</td>
</tr>
<tr>
<td>1911</td>
<td>Montreal, Canada</td>
<td>48&quot;-30&quot;</td>
<td>7,300'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>Lakeland, Fla.</td>
<td>20&quot;</td>
<td>4,000'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>Massena, N. Y.</td>
<td>24&quot;</td>
<td>1,323'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>Marquette, Mich.</td>
<td>66&quot;</td>
<td>8,000'</td>
<td>—</td>
</tr>
<tr>
<td>1911</td>
<td>New York, N. Y.</td>
<td>66&quot;</td>
<td>8,510'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Chino, Cal.</td>
<td>12&quot;</td>
<td>10,500'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1912</td>
<td>Los Angeles, Cal.</td>
<td>68&quot;-64&quot;</td>
<td>28,940'</td>
<td>5/16&quot;-3/8&quot;</td>
</tr>
<tr>
<td>1912</td>
<td>Pittsburgh, Pa.</td>
<td>30&quot;</td>
<td>5,300'</td>
<td>1/2&quot;</td>
</tr>
<tr>
<td>1912</td>
<td>Sealte, Wash.</td>
<td>42&quot;</td>
<td>13,243'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Omaha, Neb.</td>
<td>48&quot;</td>
<td>10,850'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Ottawa, Canada</td>
<td>42&quot;</td>
<td>2,400'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Pittsburgh, Pa.</td>
<td>60&quot;-72&quot;</td>
<td>5,280'</td>
<td>3/8&quot;-1/2&quot;</td>
</tr>
<tr>
<td>1912</td>
<td>Union Bv, B. C.</td>
<td>50&quot;</td>
<td>1,326'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Rochester, N. Y.</td>
<td>66&quot;</td>
<td>9,200'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Winnipeg, Canada</td>
<td>36&quot;</td>
<td>42,500'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Akron, Ohio</td>
<td>36&quot;</td>
<td>56,000'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Altman, N. Y.</td>
<td>138&quot;-96&quot;</td>
<td>2,000'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Belleville, Ohio</td>
<td>168&quot;</td>
<td>5,920'</td>
<td>—</td>
</tr>
<tr>
<td>1912</td>
<td>Montclair, N. J.</td>
<td>24&quot;</td>
<td>7,343'</td>
<td>—</td>
</tr>
<tr>
<td>1913</td>
<td>Los Angeles, Cal.</td>
<td>72&quot;</td>
<td>2,465'</td>
<td>7/16&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Baltimore, Md.</td>
<td>120&quot;</td>
<td>2,465'</td>
<td>—</td>
</tr>
<tr>
<td>1913</td>
<td>Minneapolis, Minn.</td>
<td>48&quot;-54&quot;</td>
<td>27,000'</td>
<td>5/16&quot;-7/16&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Montclair, N. J.</td>
<td>24&quot;</td>
<td>7,325'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Utica, N. Y.</td>
<td>36&quot;</td>
<td>1,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Murray City, Utah</td>
<td>26&quot;-22&quot;</td>
<td>3,882'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Vancouver, B. C.</td>
<td>36&quot;-26&quot;</td>
<td>46,250'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Winnipeg, Canada</td>
<td>36&quot;</td>
<td>42,000'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Schenectady, N. Y.</td>
<td>24&quot;</td>
<td>2,420'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Kansas City, MO.</td>
<td>48&quot;</td>
<td>1,220'</td>
<td>—</td>
</tr>
<tr>
<td>1913</td>
<td>Massena, N. Y.</td>
<td>24&quot;</td>
<td>1,200'</td>
<td>—</td>
</tr>
<tr>
<td>1913</td>
<td>Wilkes-Barre, Pa.</td>
<td>36&quot;</td>
<td>1,335'</td>
<td>—</td>
</tr>
<tr>
<td>1913</td>
<td>Cleveland, Ohio</td>
<td>48&quot;</td>
<td>2,265'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Falls Village, Conn.</td>
<td>108&quot;</td>
<td>826'</td>
<td>5/16&quot;-3/8&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Lock Raven, Md.</td>
<td>120&quot;</td>
<td>2,464'</td>
<td>7/16&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Ocoe, Tenn.</td>
<td>96&quot;</td>
<td>1,320'</td>
<td>5/8&quot;</td>
</tr>
<tr>
<td>1913</td>
<td>Crogham N. Y.</td>
<td>138&quot;</td>
<td>2,555'</td>
<td>—</td>
</tr>
<tr>
<td>1913</td>
<td>Altman, N. Y.</td>
<td>138&quot;</td>
<td>1,194'</td>
<td>5/8&quot;</td>
</tr>
<tr>
<td>1914</td>
<td>Pittsburgh, Pa.</td>
<td>42&quot;-48&quot;</td>
<td>3,060'</td>
<td>—</td>
</tr>
<tr>
<td>YEAR INSTALLED</td>
<td>LOCATION</td>
<td>DIAMETER INCHES</td>
<td>FOOTAGE</td>
<td>THICKNESS</td>
</tr>
<tr>
<td>---------------</td>
<td>---------------------------</td>
<td>-----------------</td>
<td>-------------</td>
<td>-----------</td>
</tr>
<tr>
<td>1914</td>
<td>Gardena, Cal.</td>
<td>12&quot; - 4&quot;</td>
<td>——</td>
<td>16 Ga.</td>
</tr>
<tr>
<td>1914</td>
<td>Glendora, Cal.</td>
<td>8&quot;</td>
<td>1,984'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1914</td>
<td>Glendora, Cal.</td>
<td>12&quot;</td>
<td>3,300'</td>
<td>12 Ga.</td>
</tr>
<tr>
<td>1914</td>
<td>Minneapolis, Minn.</td>
<td>48&quot;</td>
<td>11,970'</td>
<td>1/4&quot;-1/2&quot;</td>
</tr>
<tr>
<td>1914</td>
<td>Butte, Mont.</td>
<td>24&quot;</td>
<td>12,950'</td>
<td>—</td>
</tr>
<tr>
<td>1914</td>
<td>New York, N. Y.</td>
<td>66&quot;</td>
<td>12,500'</td>
<td>7/16&quot;-1/2&quot;</td>
</tr>
<tr>
<td>1914</td>
<td>Schenectady, N. Y.</td>
<td>36&quot;</td>
<td>10,500'</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1914</td>
<td>Tacoma, Wash.</td>
<td>30&quot;</td>
<td>550'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1914</td>
<td>Winnipeg, Canada</td>
<td>36&quot;</td>
<td>21,569'</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>1914</td>
<td>Springfield, Mass.</td>
<td>42&quot;</td>
<td>——</td>
<td>—</td>
</tr>
<tr>
<td>1914</td>
<td>Essex Junction, Vt.</td>
<td>108&quot; &amp; 36&quot;</td>
<td>2,440'</td>
<td>—</td>
</tr>
<tr>
<td>1914</td>
<td>Rutland, Vt.</td>
<td>54&quot;</td>
<td>2,750'</td>
<td>—</td>
</tr>
<tr>
<td>1914</td>
<td>Rochester, N. Y.</td>
<td>66&quot; &amp; 48&quot;</td>
<td>1,120'</td>
<td>—</td>
</tr>
<tr>
<td>1914</td>
<td>Cleveland, Ohio</td>
<td>48&quot;</td>
<td>1,320'</td>
<td>—</td>
</tr>
<tr>
<td>1914</td>
<td>Massena, N. Y.</td>
<td>24&quot;</td>
<td>22,000'</td>
<td>1/4&quot;-3/8&quot;</td>
</tr>
<tr>
<td>1914</td>
<td>Miami, Ariz.</td>
<td>152&quot;</td>
<td>1,670'</td>
<td>—</td>
</tr>
<tr>
<td>1914</td>
<td>Riverside, Cal.</td>
<td>30&quot;</td>
<td>35,000'</td>
<td>—</td>
</tr>
<tr>
<td>1915</td>
<td>Cleveland, Ohio</td>
<td>66&quot; - 72&quot;</td>
<td>——</td>
<td>3,960'</td>
</tr>
<tr>
<td>1915</td>
<td>Oakdale, Cal.</td>
<td>12&quot;</td>
<td>50,000'</td>
<td>14 Ga.</td>
</tr>
<tr>
<td>1915</td>
<td>Baltimore, Md.</td>
<td>84&quot;</td>
<td>4,000'</td>
<td>7/16&quot;</td>
</tr>
<tr>
<td>1915</td>
<td>Lewiston, Mont.</td>
<td>16&quot;</td>
<td>30,000'</td>
<td>1/4&quot;-3/16&quot;</td>
</tr>
<tr>
<td>1915</td>
<td>Pittsburgh, Pa.</td>
<td>48&quot;</td>
<td>3,900'</td>
<td>1/2&quot;</td>
</tr>
<tr>
<td>1915</td>
<td>Greeley, Colo.</td>
<td>20&quot;</td>
<td>5,280'</td>
<td>—</td>
</tr>
<tr>
<td>1915</td>
<td>Massena, N. Y.</td>
<td>24&quot;</td>
<td>5,000'</td>
<td>5/16&quot;-3/8&quot;</td>
</tr>
<tr>
<td>1915</td>
<td>Ogden, Utah</td>
<td>24&quot;</td>
<td>17,250'</td>
<td>—</td>
</tr>
<tr>
<td>1915</td>
<td>Ottawa, Canada</td>
<td>51&quot;</td>
<td>15,000'</td>
<td>—</td>
</tr>
<tr>
<td>1915</td>
<td>San Bernardino, Cal.</td>
<td>20&quot;</td>
<td>3,500'</td>
<td>3/16&quot;</td>
</tr>
</tbody>
</table>

**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.
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.

Stein... 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, etc.

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 factors 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.
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);

San Diego, CA 96" I.D. Pipe

Design Procedures

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

\[ V = 1.318 \times C_w \times r^{-63} \times s^{54} \]

Where  
\[ V \] = velocity, feet per second  
\[ r \] = hydraulic radius, feet (D/4)  
\[ s \] = hydraulic slope, feet per foot  
\[ C_w \] = Hazen-Williams coefficient = 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.

\[ C_w = 140 \text{ for which } Q = 60.5 \times D^{2.63} \times s^{.54} \]

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,

\[ V = 1.318 \times C_w^{0.63} \times s^{0.54} \]

Base \( C_w = 140 \)

Other \( C_w \) Values | 150 | 145 | 140 | 130
---|---|---|---|---
Relative Discharge | 1.071 | 1.036 | 1.000 | 0.920

\( s = \) Hydraulic Gradient in Feet Per Thousand Feet
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:

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

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.

igh-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

### ASTM Plate and Sheet Steels for Pipe

<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>Grade</th>
<th>Minimum Yield ksi (mpa)</th>
<th>Minimum Tensile ksi (mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Steel Plate</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM A36/36M</td>
<td>C</td>
<td>36(348)</td>
<td>58(400)</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>30(205)</td>
<td>55(380)</td>
</tr>
<tr>
<td></td>
<td>42</td>
<td>33(230)</td>
<td>60(415)</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>42(290)</td>
<td>60(415)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50(345)</td>
<td>65(450)</td>
</tr>
<tr>
<td><strong>Steel Sheet</strong></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>ASTM A139/139M</td>
<td>B</td>
<td>35(240)</td>
<td>60(415)</td>
</tr>
<tr>
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<td>C</td>
<td>42(290)</td>
<td>60(415)</td>
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<td>D</td>
<td>46(315)</td>
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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 $t = \frac{P_D}{2S}$

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

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14
Structural Analysis of Buried Welded Steel Pipes

Structural 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:

- \( \frac{D}{t} \) = 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, \( t = \frac{PD}{2t} \). 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,

\[
\frac{t}{D} = \frac{PD}{42} \text{ ksi} \quad (t = \frac{PD}{290} \text{ MPa})
\]

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

EXAMPLE –

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

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 \( [k = \text{kilo} = 10^3, M = \text{mega} = 10^6, G = \text{Giga} = 10^9] \)
psi  = pounds per square inch \([1 \text{ psi} = 6.9 \text{kPa}]\)
ksi  = kips per square inch \([1 \text{ ksi} = 6.90 \text{ MPa}]\)
ksf  = kips per square foot \([1 \text{ ksf} = 47.9 \text{kPa}]\)
pcf  = pounds per cubic foot \([1 \text{ pcf} = 157 \text{N/m}^3]\) \(\text{m} = \text{metre}\)
kPa  = kilo-Pascals of pressure \([\text{Pa} = \text{N/metre}^2]\)
N   = Newton of force \([1 \text{ lb} = 4.4482 \text{N}]\)
in  = inch \([1 \text{ inch} = 25 \text{millimetres (mm)}]\)

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)
\(\gamma\) = unit weights of materials (Subscripts, s,c,w, etc., refer to saturated soil, concrete, water, etc.)
W  = wheel load on ground surface
\(\sigma\) = normal stress (Subscripts, \(x\) and \(y\), refer to directions \(x\) and \(y\). Subscript, \(r\), refers to ratio \(\sigma_x/\sigma_y\). Subscripts and \(1\), and \(3\), refer to max and min principal stresses.)
\(\sigma_i\) = normal stress in steel at elastic limit
\(\tau_i\) = shearing stress in steel at elastic limit
\(\Delta\) = 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)
\(\nu\) = Poisson ratio = 0.3
\(\alpha\) = coefficient of thermal expansion
\(E^*\) = soil stiffness = modulus of the soil (slope of a secant on the stress-strain diagram)
\(\phi\) = soil friction angle
C  = cohesion of the soil

Pertinent Parameters (Dimensionless):
\(D/t\) = ring flexibility
\(E^*/D^*/EI\) = stiffness ratio = ratio of soil stiffness, \(E^*\), to ring stiffness, \(EI/D^3\) or its equivalent, \(E/12(D/t)^3\)
\(d/\in\) = ring deflection term
\(d\) = ring deflection = \(\Delta/D\), where
\(\Delta\) = decrease in vertical pipe diameter
\(K\) = \((1+\sin\phi)/(1-\sin\phi)\) = ratio of maximum to minimum stresses, \(\sigma_1/\sigma_3\) at soil slip

Table 1. Notation and Nomenclature

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<tr>
<td>D  = mean diameter ((\text{ID} = \text{inside diameter and OD = outside diameter}))</td>
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<td>t  = wall thickness</td>
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<td>r  = mean radius of curvature of the pipe</td>
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<td>(r_i) = radius of curvature ((\text{Subscripts}, i,x,y,A \text{ indicate locations}))</td>
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<td>I  = (t^4/12) = moment of inertia of the pipe wall cross section</td>
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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).
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:

\[ t = \frac{(D + 20\text{in})}{400} = \frac{(D + 500\text{mm})}{10.16 \times 10^3} \quad \text{U.S. Bureau of Reclamation} \quad (2) \]

\[ t = \frac{D}{288} \quad \text{Pacific Gas & Electric} \quad (3) \]

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 \( \frac{D}{t} = 288 \) which is a typical, but conservative, upper limit for buried pipes. If \( \frac{D}{t} \) 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, \( \frac{D}{t} \). Generalized recommendations for upper limits of ring flexibility for buried pipes, safety factors included, are as follows:

**Upper Limits**

\( \frac{D}{t} = 158 \) for full atmospheric vacuum,

\( \frac{D}{t} = 240 \) for cement mortar lined pipes with flexible Coating

\( \frac{D}{t} = 288 \) for less-than-well controlled installations,

\( \frac{D}{t} = 325 \) 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 3. Example of internal critical vacuum in a plain pipe in unsaturated soil for which:

\( D = 72 \text{ inch (1800 mm)} \)

\( t = \frac{1}{4} \text{ inch (6.35 mm)} \)

\( H = 3 \text{ ft (0.9 m)} \) [top]

\( d = 5\% \) [bottom]

\( \frac{D}{t} = 288 \)

\( \gamma = \text{soil unit weight} = 100 \text{ pcf (15.7 kN/m}^3) \)
Figure 4. Example of internal vacuum (equivalent flood level) at collapse of an empty, buried, plain pipe in saturated soil for which:

\[ D = 72 \text{ inches (1830 mm)} \quad H = 3 \text{ ft (0.9 m)} \quad t = 0.375 \text{ inch (9.5 mm)} \quad d = 5\% \quad \text{[bottom]} \]

\[ D/t = 192 \]

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%, 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 m)], 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 \( \phi = 25\% \), 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 (9.5 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.

**EXAMPLE —**

Suppose the pipe of Figure 4 is buried in well-compacted granular embedment for which \( \phi = 35\% \). If ring deflection is found to be \( d = 5\% \), 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.

**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, \( \phi \)) 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 1/4 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)

<table>
<thead>
<tr>
<th></th>
<th>AWWA C-205</th>
<th>INCREASED MORTAR THICKNESS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( t_s = 0.50 ) inch (13 mm)</td>
<td>( t_s = 0.75 ) inch (19 mm)</td>
</tr>
<tr>
<td></td>
<td>( t_c = 0.75 ) inch (19 mm)</td>
<td>( t_c = 1.00 ) inch (25 mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( I_s = 0.0054 ) in(^4)/ft (7.32 mm(^3))</td>
<td>( I_s = 0.0054 ) in(^4)/ft (7.32 mm(^3))</td>
</tr>
<tr>
<td></td>
<td>( I_L = 0.0167 ) in(^4)/ft (22.76 mm(^3))</td>
<td>( I_L = 0.05625 ) in(^4)/ft (76.81 mm(^3))</td>
</tr>
<tr>
<td></td>
<td>( I_c = 0.05625 ) in(^4)/ft (76.81 mm(^3))</td>
<td>( I_c = 0.1333 ) in(^4)/ft (182.06 mm(^3))</td>
</tr>
<tr>
<td></td>
<td>( I = 0.078 ) in(^4)/ft (107 mm(^3))</td>
<td>( I = 0.195 ) in(^4)/ft (268 mm(^3))</td>
</tr>
<tr>
<td></td>
<td>( EI/D^3 = 2.6 ) psi (18 kPa)</td>
<td>( EI/D^3 = 6.6 ) psi (45 kPa)</td>
</tr>
</tbody>
</table>
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, \( \sigma = \frac{PD}{2t} \), from which minimum pipe wall thickness is: \( t = \frac{PD}{2S} \). 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.](image)
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,

\[ M_A = \frac{Fr}{\pi} \]  

(6)

Castigliano's equation is found in texts on mechanics of solids. Circumferential stress on the surface of plain pipe is

\[ \sigma = \frac{6M}{t^2} = \frac{6Fr}{\pi t^2} \]  

at point A. Strain (below yield) is \( \varepsilon = \frac{\sigma}{E} \).

\[ \varepsilon = \frac{6Fr}{\pi Et^2} \]  

(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, \( \varepsilon_y = 1.4(10^{-3}) \) where yield stress is \( S_y = 42 \text{ ksi} (290 \text{ MPa}) \) and \( E = 30,000,000 \text{ psi} (207 \text{ GPa}) \). 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 \( M_A = \frac{Fr}{\pi} \). From theory of elasticity, is \( M_y = 2S_yl/t \). Equating and solving, \( FD = \frac{4}{\pi} S_yl/t \) (8)

EXAMPLE —
What is the F-load on plain pipe at yield stress? \( D = 72 \text{ inches} (1800 \text{ mm}), t = 0.300 \text{ inch} (7.6 \text{ mm}), (D/t) = 240, \) and \( S_y = 42 \text{ ksi} (290 \text{ MPa}) \). 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 \( F = 376 \text{ lb/ft} (5.49 \text{ kN/m}) \).

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, \( FD = 4\pi S_yl/t \) Substituting,

\[ d = 0.234(S_y/E)(D/t) = 0.234\varepsilon_y(D/t) \]  

(9)

EXAMPLE —
A plain pipe is F-loaded. See Figure 7. What is the ring deflection when stresses at points A reach yield? \( D = 72 \text{ inches} (1800 \text{ mm}), t = 0.300 \text{ inch} (7.62 \text{ mm}), (D/t) = 240, \) and \( S_y = 42 \text{ ksi} (290 \text{ MPa}) \) at \( \varepsilon_y = 1.4(10^{-3}) \). 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, \( u \);

\[ P' = P'' - u = P'' - h\gamma_w \]  

(10)

Pressure on the pipe is,

\[ P = P_d + P_l \]  

(11)

Where
\( P_d = \) dead load pressure due to weight of the soil, \( \gamma = \) unit weight of the soil, \( \gamma_w = \) 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.
\[ P_d = \text{dead load pressure} = \text{pressure of soil above the pipe (See Figure 8)}, \]
\[ P_l = \text{live load pressure} = \text{the effect of surface loads on the pipe (See Figure 9)} \]

According to Boussinesq, pressure, \( P_l \), directly below surface load, \( W \), is,
\[ P_l = 0.477 \frac{W}{H^2} \quad (12) \]

Graphs are available for evaluating \( P' \). See the AISI graphs, Figure 10.

**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/m³) above the water table, and 125 pcf (19.6 kN/m³) below the water table. Dead load pressure at the top of the pipe is \( P_d = (3\text{ft})(100\text{pcf}) + (5\text{ft})(125\text{pcf}) = 925 \text{ Ib/ft}^2 \) (44.3 kPa). Live load pressure due to the 68 kip wheel load is, from Equation 12, \( P_l = 507 \text{ lb/ft}^2 \) (24.3 kPa). The sum is \( P' = 1.4 \text{ kips/ft}^2 \) (69 kPa). Effective pressure is \( P' = P' - P_d = 1,432 - (62.4)5 = 1.1 \text{ kips/ft}^2 \) (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 \( \gamma \), maximum horizontal stress at slip is \( \sigma_x = K \gamma \), where \( K = \frac{(1 + \sin \phi)}{(1 - \sin \phi)} \). Friction angle, \( \phi \), is a function of soil compaction. For granular soil with some compaction, the friction angle is often assumed to be \( \phi = 30^\circ \) from which \( K = 3 \). The soil slips if the ratio of maximum to minimum principal stresses exceeds 3. If \( \phi = 35^\circ \), \( K = 3.69 \). If \( \phi = 250^\circ \), \( K = 2.46 \).
Figure 11. Infinitesimal soil cube showing the ratio of the maximum stress $\sigma_x$ (passive) to minimum stress $\sigma_y$ (active) at soil slip. Slip planes form at angles: $\theta_s = 45^\circ - \phi/2$.

Table 5. Approximate Friction Angles for Granular Soil

<table>
<thead>
<tr>
<th>COMPACTION</th>
<th>$\phi = \text{FRICTION ANGLE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy (dense)</td>
<td>$35^\circ$ to $40^\circ$</td>
</tr>
<tr>
<td>Light</td>
<td>$25^\circ$ to $35^\circ$</td>
</tr>
<tr>
<td>None (loose)</td>
<td>$15^\circ$ to $25^\circ$</td>
</tr>
</tbody>
</table>

Soil Friction Angle: Precise values of soil friction angle, $\phi$, 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 $\theta_s = 45^\circ - \phi/2$ as shown in Figure 11. Passive soil resistance, $\sigma_x$, is the maximum support that the soil can provide at the sides of a buried pipe. If the soil cube is rotated $90^\circ$, 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 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.

Figure 12. Conservative working values (lower limits) of soil friction angle, $\phi$, 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 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.

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).
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, \( \sigma_1 \) and \( \sigma_2 \), which are usually determinable. Intermediate stress, \( \sigma_i \), at right angles to \( \sigma_1 \) and \( \sigma_2 \), is not critical. Principal stresses are analyzed for each load. Stresses in the same direction are not always combined. For example, \( \sigma_1 \) 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.

**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\(^2\)). What is the pressure of pipe against sidefill at the spring lines? Pressure on top of the pipe is \( P' = P = (4 ft)(120 \) pcf\) = 480 psf \((23.0 \) kPa\). From Equation 15, the ratio of radii is \( r/r_s = 1.2 \). From Equation 16, \( P_s = 576 \) psf \((27.6 \) Pa\).

The deflection \( \Delta \) is the pressure of pipe against sidefill at the spring lines? Pressure on top of the pipe is \( P' = P = (4 ft)(120 \) pcf\) = 480 psf \((23.0 \) kPa\). From Equation 15, the ratio of radii is \( r/r_s = 1.2 \). From Equation 16, \( P_s = 576 \) 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 \( P_s = 876 \) psf \((41.9 \) kPa\) — easily resisted by granular sidefill. Because the required \( K \) is \( 876/480 \), the minimum soil friction angle is \( \phi = 17^\circ \).

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, \( \sigma \), is circumferential stress in the ring; i.e., \( \sigma = P'(1 +d)/t \). \( P' \) 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, \( t = P'r/S \) (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 \( \gamma = 110 \) pcf \((15.7 \) kN/m\(^2\))?

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.

\[ d = \varepsilon \]  \hspace{1cm} (18)

where \( \varepsilon \) 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, \( d/\varepsilon \), as a function of stiffness ratio, \( E''/(E/l^3) \); 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 \( \varepsilon = (3.15\% - 0.4\%) = 2.75\% \).

\[ E'' = \text{the slope of the secant from 480 to 3600.} \]
\[ E'' = 788 \text{ psi (5.43 MPa).} \]
\[ \text{Ring stiffness is } E/l^3 = 0.79 \text{ psi (5.4 kPa).} \]
\[ \text{Stiffness ratio is } E''/(E/l^3) = 1000. \]
\[ \text{The ring deflection term, } d/\varepsilon, \text{ is found by entering Figure 16 with stiffness ratio, 1.000, which falls off the chart. Even with a standard AWWA lining (ring stiffness = 1.68 psi) the stiffness ratio, } E/l^3 = 469, \text{ falls off the chart. The lined pipe is flexible, } d/\varepsilon = 1, \text{ and } d = \varepsilon = 2.75\%. \]  
Allowable ring deflection for lined pipe is 3% based on a safety factor of two against disbonding. Is disbonding a performance limit? Probably not.

**Ring deflection of buried steel pipes is less than or equal to the vertical compression of the sidefill soil.** \( d = \varepsilon \)

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 \( E''/(E/l^3) = 40 \) 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

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**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 \( E'' = 788 \text{ psi (543 MPa)} \) for a soil pressure increase from...**

**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."

---

**EXAMPLE** —

**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. \( d = \varepsilon \)

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 \( E''/(E/l^3) = 40 \) 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 \( p r^3/(1 - \nu^2)/EI = 3 \). 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 \( \nu \); i.e.,
   \[
   \frac{p}{(EI/r^3)} = 3 \tag{18}
   \]
   For a circular plain pipe (no lining, coating, or stiffener rings), \( I = t^3/12 \), and Equation 18 reduces to,
   \[
   \frac{(p/E)(D/t)^3}{3} = 2 \tag{19}
   \]
   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. \( r_r = r_r/r_0 \).](image)

**EXAMPLE**

What is the maximum \( D/t \) for a plain pipe subjected to atmospheric pressure (vacuum) of \( P = 15 \text{ psi} (103 \text{ kPa}) \)? \( E = 30,000,000 \text{ psi} (207 \text{ GPa}) \). 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 \( \phi \). Sidefill is granular. Water table is below the pipe.](image)

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.
   \[
   P' r_r = K \sigma_y \tag{20}
   \]
   \( P' \) is soil pressure at the top of the pipe and \( r_r = (1 + d)/(1-d)^3 \). 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 \((1800 \text{ mm}) \) and \( H = 4 \text{ ft} (1.2 \text{ m}) \)? Embedment is poor, granular, loose soil. Unit weight is \( \gamma = 100 \text{ pcf} (15.7 \text{ kN/m}^3) \). The soil friction angle is assumed to be \( \phi = 15^\circ \) from which, \( K = (1+\sin\phi)/(1-\sin\phi) = 1.7 \). \( P' = \gamma H = 400 \text{ psf} (19.15 \text{ kPa}) \). \( \sigma_y = \gamma Z = (100 \text{ pcf})(4\text{ft}+3\text{ft}) = 700 \text{ psf} (33.5 \text{ kPa}) \). For the first trial, let \( Z = 7 \text{ ft} (2.13 \text{ m}) \). Substituting these values into Equation 20, \( P'r_r = (400\text{lb/ft}^2)(1+d)/(1-d)^3 = 1189 \text{ lb/ft}^2 (56.93 \text{ kPa}) \). Solving, \( d = 18\% \). For a second trial, let \( Z = 4\text{ft}+2.5\text{ft} = 6.5 \text{ ft} (2.0 \text{ m}) \) 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 \( \gamma = 100 \) pcf (16 kN/m\(^3\)). The friction angle is assumed to be \( \phi = 15^\circ \) because the soil is of poor quality. \( Ei/r^2 = 0.78 \) psi (5.4 kPa). \( P = 1200 \) psf = 8.3 psi (57 kPa). The soil pressure term is \( P/(El/r^2) = 10.7 \).
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, \( r_v \), is greater than \( r \). Therefore, ring stiffness, \( Ei/r_v^3 \), 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 infinitesimal cube, 6, of Figure 17 can be equated at soil slip. Solving for vacuum, \( p \), at soil slip, \( p(r-1) = K\sigma_y - (P - Ed/m^3)\).

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)

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, \( \phi \)). 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 \( \phi = 25^\circ \) (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 \( \phi = 35^\circ \), 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,

\[
p(r-1) = K\sigma_y + U_B - \left( P_A + \frac{\pi r y_w}{2} - Ed/m^3 \right)r
\]

SATURATED SOIL

where:

- \( P = \) vacuum and/or pressure of flood level \( h \),
- \( \sigma_y = \) effective vertical soil stress at B,
- \( P_A = \) total vertical pressure at the top of the pipe at A,
- \( K = \frac{(1+\sin\phi)}{(1-\sin\phi)} \),
- \( \phi = \) soil friction angle,
- \( U_B = \) water pressure at B = \( (h + H + r) \gamma_w \),

\( \gamma_w = \) 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) = \( \Delta/D \),

\( D = \) circular diameter of the pipe,

\( m = r/t = \) ring flexibility,

\( r = D/2 = \) radius of the circular pipe,

\( t = \) wall thickness,

\( r_r = r/r \).

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, \( (\pi r y_w/2) \), is uplift pressure equivalent to buoyancy of the empty pipe. If the pipe is full of water, this term is eliminated from Equation 22.
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 $\phi = 15^\circ$. 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 $\phi = 35^\circ$, 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 $\pi D^2 \gamma_b/4$ which must be resisted by the buoyant weight of the soil wedges. See Figure 21. The wedge slip planes are at $\theta_s = 45^\circ + \phi/2$. Buoyant unit weight is $\gamma_b = \gamma_{sat} - \gamma_w$ (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,

\[
M = 0.022Pr^2
\]  
(23)

From theory of elasticity, \( M = \sigma l/c \) where \( l/c \) is the wall section modulus. Substituting into Equation 23, with allowable stress, \( \sigma = S \), the minimum required \( l/c \) is,

\[
\text{Min} \ (l/c) = Pr^2/45S
\]  
ELASTIC THEORY
(24)

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

\[
\text{Max} \ (D/t)^2 = 30S/P
\]  
ELASTIC THEORY - Plain Pipe
(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,

\[
\text{Min} \ (l/c) = Pr^2/68S
\]  
ELASTIC THEORY - Plain Pipe
(26)

\[
\text{Max} \ (D/t)^2 = 45S/P
\]  
PLASTIC THEORY - Plain Pipe
(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 \( S_1 = 42 \) 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 line, 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 \( P_r = P \), 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.](image-url)
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, \( P_s = Pr = Ka \). For good sidefill, \( K \) equals 3 or more. At spring line, \( \sigma_y \) is greater than \( P \) on top of the pipe. However, assuming, conservatively, that \( \sigma_y = P \), \( r_f = 3 \) 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 \( r_f = (1 + d)/(1 - d) = 1.35 \). Pressure on top is \( P \). Therefore \( P_s = Pr = 1.35P \). 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, \( P = P_x + P_y \), where \( P_y = W/(B + H)(L + H) \). If the ring is nearly circular, \( P_y = P \). 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 1v: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 \( \sigma_y = P_x/2 = P/2 \), 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 \( \gamma = 100 \) psf (15.7 kN/m²). 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, \( P_s = 0.447W/H^2 = 1908 \) 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. \( P_y = 200 \) psf. \( P = 2108 \) psf. For a circular flexible ring, \( P_x = P \). See Figure 24, \( \sigma_y = P_x/2 = 5054 \) psf. At spring line level, \( \sigma_y = 413 \) psf. \( K = \sigma_y/\sigma_o = 1054/413 = 1.555 \). But \( K = (1 + \sin \phi)/(1 - \sin \phi) \). Solving, \( \phi = 26^\circ \) 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, \( \phi = 13^\circ \). 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), \( \phi = 3^\circ \).

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 \( P_x = 3626 \) psf. \( P_y = 98 \) psf. \( \sigma_y = 1862 \) psf. \( \sigma_o = 311 \) psf. Therefore, in Figure 24, \( \phi = 5.987 \) (1 + sin \( \phi)/(1 - \sin \phi) \). Solving, \( \phi = 45.5^\circ \). 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 \( \phi = 0^\circ \)? 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.
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

\[ (X/D) = 3(H/Z) \]  

(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, \(Q_l = P X\), where \(P\) is the pressure at depth \(H + r\) according to Boussinesq. Total load on Section A-A is \(Q = Q_d + Q\), 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, \(\sigma_x = P_x = P'r\). 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, \(\phi\), 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 \(\phi = 37^\circ\). 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 \(P = 0.477W/Z^2 = W/49.82\) ft. Live load on column Section A-A is \(Q = XP = W/16.6\) ft. Total load on column is \(Q = Q_d + Q\). Load carried by the steel cladding is \(P'D = 510\) lb/ft\(^2\). Vertical stress on the soil column is \(\sigma_y = (Q - P'D)/X\). Horizontal stress is the pressure of the pipe against the soil column; i.e., \(\sigma_x = P_x = P = 330\) lb/ft\(^2\) (15.8 kPa). At soil slip, \(\sigma_x/\sigma_y = K = 3\). Solving, \(W = 75\) kips (66.7 kN).

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

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

(29)

For the example above, minimum separation is \(X = 1.9\) ft (0.58 m). If \(\phi\) is reduced from \(37^\circ\) to \(30^\circ\), 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 \( \sigma_z = \frac{P_r}{2t} \) 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 (\( \theta = 90^\circ \))?

8 is the angle offset of the elbow. Thrust, \( Q = Q_p + Q_i \), where: due to pressure, \( Q_p = \frac{P\pi D^2(1-\cos \theta)}{4} = 245 \) kips (1.09 MN); and due to change in direction of flow (impulse), \( Q_i = \gamma w \frac{\pi D^2(1-\cos \theta)}{4g} = 6 \) 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 negligible.

The thrust due to thrusters is:

\[
Q = \left( \frac{\pi D^2}{4} \right)(1-\cos \theta)(P + \gamma w v^2/g)
\]

(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

![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, \( \sigma_z \pi D t \).](image)

![Figure 28. Worst-case beam loading for longitudinal stress analysis of a simply-supported buried pipe section.](image)

![Figure 29. Reasonable soil reaction for longitudinal analysis.](image)

tension stress, \( \sigma_z \), in a straight, *fixed-ended* pipe, due to temperature decrease and internal pressure increase, is,

\[
\sigma_z = E\alpha\Delta T + \nu PD/2t
\]

(31)

where

- \( E \) = modulus of elasticity of steel = 30,000,000 psi (207 GPa),
- \( \alpha \) = coefficient of thermal expansion of steel = 6.5\( \times 10^{-6}^\circ \text{F} \),
- \( \Delta T \) = decrease in temperature in degrees Fahrenheit, \( ^\circ \text{F} = 1.8( ^\circ \text{C}) + 32 \) where \( ^\circ \text{C} \) is in degrees Celsius,
- \( \nu \) = Poisson ratio = 0.3 for steel,
- \( P \) = internal pressure in the pipe,
- \( D \) = diameter,
- \( t \) = wall thickness.

**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, \( \sigma_z = 9.75 \) ksi + 6.14 ksi. \( \sigma_z = 15.9 \) 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, \( \sigma_i \pi D t \). Solving for maximum spacing, \( L \), of slip couplings in a welded pipe,

\[
L = 2\sigma_i t/\mu (\gamma H + \gamma_p D/4 + w_p \pi D) \tag{32}
\]

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

**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 \( S = 21 \text{ ksi} \) (145 MPa). Data are as follows.

\[
L = \text{spacing of slip couplings along the pipeline},
T = \text{wall thickness}, \quad = 0.250 \text{ (6.35 mm)}
D = \text{pipe diameter}, \quad = 51 \text{ inches (1275 mm)}
H = \text{height of soil cover}, \quad = 3 \text{ ft (0.91 m)}
\gamma = \text{unit weight of soil}, \quad = 110 \text{ pcf (17.3 kN/m}^3\text{)}
\gamma_p = \text{unit weight of water}, \quad = 62.4 \text{ pcf (9.8 kN/m}^3\text{)}
w_p = \text{weight of pipe per unit length}, \quad = 102.33 \text{ lb/ft}
\sigma_i = \text{allowable longitudinal stress}, \quad = 15.75 \text{ ksi (109 MPa)}
\mu = \text{coefficient of friction of soil on pipe}, \quad = 0.32 \text{ for tape coated}
\mu' = \text{coefficient of friction of soil on pipe}, \quad = 0.4 \text{ for mortar coated}
\]

If the pipe is tape coated, \( \mu = 0.32 \). Substituting data into Equation 32, \( L = 731 \text{ ft} \). 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, \( \sigma_z \), occurs at \( B \) where, by classical analysis,

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

SIMPLY SUPPORTED \tag{33}

where

\( \sigma_z = \text{longitudinal stress in the pipe wall}, \)
\( w = P'D + \text{weight of pipe and contents per unit length}, \)
\( t = \text{pipe wall thickness}, \)
\( L = \text{length of the pipe section}, \)
\( D = \text{diameter}, \)
\( P' = \text{vertical soil pressure at the top of the pipe}. \)

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 \tag{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 \( L = 40 \text{ ft (12.2 m)} \). The unit weight of the saturated soil is \( 125 \text{ pcf (19.6 kN/m}^3\text{)} \). Assume the soil friction angle is 30°. \( D = 51 \text{ inches (1275 mm), t = 0.187 inch (4.75 mm)} \). What is the maximum longitudinal stress, \( \sigma_z \), when the soil settles and the water table is below the pipe at low tide?

\[
w_p = 102 \text{ lb/ft} \quad = \text{weight of the pipe},
w_w = 885 \text{ lb/ft} \quad = \text{weight of the water},
w_s = 3448 \text{ lb/ft} \quad = \text{weight of the soil},
w = 4435 \text{ lb/ft (6 kN/m)}. \]

Substituting into Equation 31, \( \sigma_z = 18.6 \text{ ksi (128 kPa)} \). If the spacing of the piles were 60 feet instead of 40 feet, longitudinal stress would be \( \sigma_z = 41.8 \text{ ksi (288 kPa)} \) 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, \( \sigma_t \). 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 stress is, \( \tau = \sigma_t/2 \).

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, \( \sigma_b \), is usually internal pressure which is of opposite sign from the maximum principal stress. Compared to \( \sigma_t \), the minimum principal stress, \( \sigma_b \), is usually small enough to be neglected. The strength envelopes of Figure 30 are based on \( \sigma_b = 0 \). If values for \( \sigma_t \) and \( \sigma_y \) 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 \( \sigma_y = 0 \), the equation for the strength envelope is,

\[
\sigma_t + \sigma_y^2 - \sigma_v\sigma_z = \sigma_r^2
\]

Huber-Hencky-vonMises (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 \( \sigma_b = \sigma_t/2 \) (both \( \sigma_1 \) and
σ, are in tension), from Figure 30, the hoop strength is \( \sigma_z = 1.155 \sigma_f \). 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, \( \sigma_y \), is of opposite sign from the hoop stress, \( \sigma_z \), Equation 35 should be applied. The strength envelope is shown in the upper left and lower right quadrants of Figure 30. The probability that \( \sigma_z \) and \( \sigma_y \) 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:

\[
V = P \pi r^2 \sin \theta \cos \theta \tag{36}
\]
\[
T = P \pi r^2 \sin^2 \theta \tag{37}
\]

If the maximum angle offset for a bend is 15º, then \( \theta = 7.5^\circ \), \( V = 0.13 P \pi r^2 \), and \( T = 0.017 P \pi r^2 \). Hydraulics usually limit the offset to 15º. 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,

\[
w = 2Pr \sin \alpha \cos \theta \tag{38}
\]

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, $l/R = 2\varepsilon/t$. Minimum radius $R$ is,

$$R = \frac{t}{2\varepsilon}$$

(39)

where

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

$t$ = wall thickness,

$\varepsilon$ = 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.

$$X_s = n = 7.5 = \text{unit width for steel transformed into its mortar equivalent},$$

$$x_m = 1 = \text{width of mortar coating and lining (unit slice)},$$

$$E_m = 4\times10^6 \text{ psi (28.6 GPa) for mortar},$$

$$E_s = 30\times10^6 \text{ psi (207 GPa) for steel},$$

$$n = E_s/E_m = 7.5 \text{ for transforming the unit width of steel into its mortar equivalent}.$$

Find $I$:

For each mortar layer, $I = t/12$. For the steel equivalent in mortar, $I = nt/12$. 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, $\sigma$:

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
\[ \sigma_c = \frac{M_c}{t_c^2/2}. \]
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 \( M_c / M = \frac{l_c}{l} \). 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 \( l/l = 0.05625/0.07835 = 0.718 \). Maximum moment is \( M_c = 71.8 \) percent of \( M \). Substituting into the stress equation, maximum stress in the coating is
\[ \sigma_c = 2.4378 \frac{Fr}{t_c^2} = 2.44 \frac{Fr}{\text{in}^2}. \]

**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, \( \sigma_y \), 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 \( M_e = \sigma_y t^2/6 \).](image)

**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 them. 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 \( r_x \) and \( r_y \).

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.](image)

**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 test 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 $E'$. 
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

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

b. Grooved and shouldered type

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.
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.
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 \( \frac{1}{360} \) 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:

\[ S_{cs} = k \frac{P}{t^2} \cdot \log_e \left( \frac{R}{t} \right) \] (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 (Ibs)
- \( R \) = 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, \( S_e \):

\[ S_e = (S_{cs}^2 + S_{ls}^2 - S_{cs}S_{ls})^{1/2} \] (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:

\[ RF = \frac{(\tanh A)}{A} \] (3)

where:
- \( A = 1.1 \left( \frac{R}{t} \right) (S/E)^{1/2} \)
- \( S_i \) = hoop stress (psi)
- \( E \) = Modulus of elasticity (psi)
  \( (30,000,000 \text{ for steel}) \)
- \( \tanh \) 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 \( (0.30S_i) \) must be added to the beam flexure stress \( (S) \). The total longitudinal stress \( (S) \) is taken as:

\[ S = S_i + 0.30S_e \] (4)

**EXAMPLE** —

42-in. dia. by \( \frac{7}{16} \) 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)

\[ k = 0.02 - 0.0012(120 - 90) = 0.0164 \]

\[ S_{cs} = 0.0164 \times \log_e (21/0.3125) = 28300 \text{ psi} \]

\[ S_{ls} = -3000 \text{ psi} \]

\[ S_e = 1000 \left[ 28.3^2 + 3.0^2 - 3.0 \times 28.3 \right]^{1/2} = 29900 < 30000 \text{ O.K.} \]

Beam stresses must still be checked by Equation 4. The flexure stress \( S \) 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).

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 = 2\( r \), 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.
There are numerous other installations with spans below 60 feet.

### Stress in Pipe Shell

Maximum combined ring stress

\[ f_r = \frac{D(w+qh)}{2t} \]

Combined maximum longitudinal stress

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

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

\[ F_{bo} = 1.82 \left( \frac{A-ct}{t} \right) \left[ \frac{pr}{t} \right] \]

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 \( f_L \) as found by equation above in place of \( F_r \).

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

\[ a = 0.04r \] outside the neutral axis.

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

\[ M = 0.01 \cdot Qr \]

Maximum bending stress

\[ f_1 \]

(General bending formula)

Maximum ring stress due to shear forces

\[ f_2 = \]

Ring stress due to radial forces

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

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, 7/16-inch through 3/4-inch steel plate, 1200-foot head.
<table>
<thead>
<tr>
<th>Owner &amp; Location</th>
<th>Length</th>
<th>Diameter</th>
<th>Head</th>
<th>Year Installed</th>
</tr>
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<tbody>
<tr>
<td>City of Los Angeles Aqueduct</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nine Mile Siphon</td>
<td>1415'</td>
<td>9'6&quot;</td>
<td>175'</td>
<td>1912</td>
</tr>
<tr>
<td>No name Siphon</td>
<td>2016'</td>
<td>9'3&quot;</td>
<td>365'</td>
<td>1912</td>
</tr>
<tr>
<td>Sand Canyon Siphon</td>
<td>890'</td>
<td>8'6&quot;</td>
<td>455'</td>
<td>1912</td>
</tr>
<tr>
<td>Grapevine Siphon</td>
<td>2339'</td>
<td>9'3&quot;</td>
<td>355'</td>
<td>1912</td>
</tr>
<tr>
<td>Jawbone Siphon</td>
<td>8095'</td>
<td>10'0&quot; to 7'6&quot;</td>
<td>850'</td>
<td>1912</td>
</tr>
<tr>
<td>Pine Tree Siphon</td>
<td>3841'</td>
<td>9'0&quot;</td>
<td>480'</td>
<td>1912</td>
</tr>
<tr>
<td>Antelope Siphon</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15,597'</td>
<td>10'0&quot;</td>
<td>200'</td>
<td>1912</td>
<td></td>
</tr>
<tr>
<td>Deadman Siphon</td>
<td>3430'</td>
<td>11'0&quot;</td>
<td>245'</td>
<td>1913</td>
</tr>
<tr>
<td>Little Lake Siphon</td>
<td>3800'</td>
<td>7&quot;</td>
<td>800'</td>
<td>1969</td>
</tr>
<tr>
<td>Soledad Siphon</td>
<td>8941'</td>
<td>11'0&quot; to 10'0&quot;</td>
<td>260'</td>
<td>1913</td>
</tr>
<tr>
<td>Quigley Siphon</td>
<td>612'</td>
<td>11'0&quot;</td>
<td>67'</td>
<td>1913</td>
</tr>
<tr>
<td>Placerita Siphon</td>
<td>1572'</td>
<td>11'0&quot;</td>
<td>105'</td>
<td>1913</td>
</tr>
<tr>
<td>Hainee Penstock</td>
<td>1170'</td>
<td>108&quot; to 102&quot;</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Power House #1 Penstock</td>
<td>8657'</td>
<td>84&quot; to 54&quot;</td>
<td>938'</td>
<td>1916 &amp; 1928</td>
</tr>
<tr>
<td>Power House #2 Penstock</td>
<td>4290'</td>
<td>84&quot; to 72&quot;</td>
<td>540'</td>
<td>1921 &amp; 1932</td>
</tr>
<tr>
<td>San Fernando Penstock</td>
<td>4600'</td>
<td>8'3&quot; to 7'5&quot;</td>
<td>240'</td>
<td>1922</td>
</tr>
<tr>
<td>Bouquet Canyon Line</td>
<td>18,000'</td>
<td>94&quot; to 80&quot;</td>
<td>840'</td>
<td>1933</td>
</tr>
<tr>
<td>Owens River Gorge Penstocks</td>
<td>9000'</td>
<td>106&quot; to 92&quot;</td>
<td>790'</td>
<td>1949</td>
</tr>
<tr>
<td>Bureau of Reclamation Projects—U.S.</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Owyhee &amp; Snively Siphons, Ore.</td>
<td>2500'</td>
<td>126&quot; to 108&quot;</td>
<td>350'</td>
<td>1934</td>
</tr>
<tr>
<td>Bully Creek &amp; Fairman Coulee Siphons, Vale, Ore.</td>
<td>7500'</td>
<td>101&quot;</td>
<td>160'</td>
<td>1931</td>
</tr>
<tr>
<td>Yakima Project, Wash.</td>
<td>1000'</td>
<td>145&quot;</td>
<td>—</td>
<td>1931</td>
</tr>
<tr>
<td>Grand Coulee Discharge Lines</td>
<td>9373'</td>
<td>144&quot;</td>
<td>363'</td>
<td>1947</td>
</tr>
<tr>
<td>Numerous Penstocks</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deer Creek</td>
<td>1300'</td>
<td>72&quot;</td>
<td>300'</td>
<td>1940</td>
</tr>
<tr>
<td>Shasta Dam</td>
<td>4000'</td>
<td>180&quot;</td>
<td>475'</td>
<td>1943</td>
</tr>
<tr>
<td>Imperial Valley</td>
<td></td>
<td>186&quot;</td>
<td>—</td>
<td>1948</td>
</tr>
<tr>
<td>Cody</td>
<td></td>
<td>123&quot;</td>
<td>—</td>
<td>1948</td>
</tr>
<tr>
<td>Pole Hill</td>
<td>2040'</td>
<td>96&quot;</td>
<td>1040'</td>
<td>1954</td>
</tr>
<tr>
<td>Flat Iron</td>
<td>10,600'</td>
<td>84&quot; to 72&quot;</td>
<td>1392'</td>
<td>1954</td>
</tr>
<tr>
<td>Owyhee</td>
<td>1630'</td>
<td>80&quot;</td>
<td>268'</td>
<td>1936</td>
</tr>
<tr>
<td>So. Canadian River</td>
<td>2960'</td>
<td>61&quot;</td>
<td>545'</td>
<td>1961</td>
</tr>
<tr>
<td>Wahluke</td>
<td>2500'</td>
<td>180&quot;</td>
<td>—</td>
<td>1959</td>
</tr>
<tr>
<td>Malheur River</td>
<td>23,178'</td>
<td>80&quot;</td>
<td>268'</td>
<td>1936</td>
</tr>
<tr>
<td>City of San Francisco Hetch Hetchy Aqueduct</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mocassin Creek Power House</td>
<td>6000'</td>
<td>104&quot; to 54&quot;</td>
<td>—</td>
<td>1924</td>
</tr>
<tr>
<td>Red Mountain Bar Siphon</td>
<td>1700'</td>
<td>114&quot;</td>
<td>—</td>
<td>1922</td>
</tr>
<tr>
<td>East Bay Municipal Utility District</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mokelumne Aqueduct</td>
<td>30 Miles</td>
<td>54&quot; to 68&quot; to 88&quot;</td>
<td>500'</td>
<td>1924, 1947 &amp; 1963</td>
</tr>
<tr>
<td>Southern California Edison Co.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Big Creek 2-A Penstock</td>
<td>6678'</td>
<td>108&quot; to 66&quot;</td>
<td>2418'</td>
<td>1928</td>
</tr>
<tr>
<td>Big Creek #8 Penstock</td>
<td>2740'</td>
<td>96&quot; to 72&quot;</td>
<td>720'</td>
<td>1921</td>
</tr>
<tr>
<td>Big Creek #3 Penstock</td>
<td>4200'</td>
<td>78&quot; to 72&quot;</td>
<td>820'</td>
<td>1921</td>
</tr>
<tr>
<td>Kern River #3 Penstock</td>
<td>5000'</td>
<td>84&quot; to 60&quot;</td>
<td>821'</td>
<td>1920</td>
</tr>
<tr>
<td>Little Brush Creek Siphon</td>
<td>1170'</td>
<td>114&quot; to 96&quot;</td>
<td>300'</td>
<td>1920</td>
</tr>
<tr>
<td>Big Creek #4</td>
<td>650'</td>
<td>180&quot;</td>
<td>—</td>
<td>1951</td>
</tr>
<tr>
<td>Mammoth Pool</td>
<td>2000'</td>
<td>156&quot; to 90&quot;</td>
<td>1125'</td>
<td>1958</td>
</tr>
<tr>
<td>Pacific Gas &amp; Electric Co.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hat Creek Power House #1</td>
<td>1600'</td>
<td>120&quot; to 96&quot;</td>
<td>217'</td>
<td>1924</td>
</tr>
<tr>
<td>Drum Power House</td>
<td>6272'</td>
<td>72&quot;</td>
<td>1375'</td>
<td>1920</td>
</tr>
<tr>
<td>Kerkhoff Power House</td>
<td>3000'</td>
<td>96&quot; to 84&quot;</td>
<td>—</td>
<td>1920</td>
</tr>
<tr>
<td>Pit 5</td>
<td>5600'</td>
<td>102&quot; to 90&quot;</td>
<td>630'</td>
<td>1943</td>
</tr>
<tr>
<td>Electra</td>
<td>3400'</td>
<td>114&quot; to 90&quot;</td>
<td>1268'</td>
<td>1948</td>
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<tr>
<td>Drum</td>
<td>3754'</td>
<td>72&quot;</td>
<td>250'</td>
<td>1928</td>
</tr>
<tr>
<td>Salt Springs</td>
<td>1004'</td>
<td>129&quot; to 72&quot;</td>
<td>700'</td>
<td>1930</td>
</tr>
<tr>
<td>Cresta</td>
<td>1500'</td>
<td>144&quot;</td>
<td>400'</td>
<td>1948</td>
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<td>El Dorado</td>
<td>2038'</td>
<td>60&quot;</td>
<td>400'</td>
<td>1946</td>
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<tr>
<td>Colgate</td>
<td>1500'</td>
<td>96&quot; to 66&quot;</td>
<td>1350'</td>
<td>1948</td>
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<tr>
<td>Rock Creek</td>
<td>1725'</td>
<td>144&quot; to 126&quot;</td>
<td>650'</td>
<td>1948</td>
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<tr>
<td>Bear River</td>
<td>4685'</td>
<td>72&quot; to 46&quot;</td>
<td>2000'</td>
<td>1950</td>
</tr>
<tr>
<td>Pit 4</td>
<td>1684'</td>
<td>144&quot;</td>
<td>450'</td>
<td>1953</td>
</tr>
<tr>
<td>Owner &amp; Location</td>
<td>Length</td>
<td>Diameter</td>
<td>Head</td>
<td>Year Installed</td>
</tr>
<tr>
<td>-----------------------------------------</td>
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<td>----------------</td>
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<tr>
<td>Maricopa County Municipal Water Conservation District (Arizona)</td>
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<tr>
<td>Steel Pipe Flumes</td>
<td>1200’</td>
<td>120”</td>
<td>Gravity</td>
<td>1935</td>
</tr>
<tr>
<td>Utah Power &amp; Light Co</td>
<td>14,500’</td>
<td>102”</td>
<td>Gravity</td>
<td>1948</td>
</tr>
<tr>
<td>Olmsted Plant</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Northwestern Power &amp; Light Co</td>
<td>2000’</td>
<td>120”</td>
<td>120’</td>
<td>1923</td>
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<tr>
<td>Glines Canyon Penstock</td>
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<tr>
<td>Portland Railway, Light &amp; Power Co</td>
<td>32,000’</td>
<td>108”</td>
<td>925’</td>
<td>1923</td>
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<tr>
<td>Oak Grove Ore</td>
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<tr>
<td>California-Oregon Power</td>
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<tr>
<td>Lemolo #2</td>
<td>3847’</td>
<td>126”—88”</td>
<td>—</td>
<td>1954</td>
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<tr>
<td>California-Electric Power</td>
<td></td>
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</tr>
<tr>
<td>Bishop Creek</td>
<td>3300’</td>
<td>60”</td>
<td>60’</td>
<td>1960</td>
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<td>Metropolitan Water District</td>
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<td></td>
</tr>
<tr>
<td>5 Pumping Plants</td>
<td>18,000’</td>
<td>120”—72”</td>
<td>440’</td>
<td>1936, 1954 &amp; 1962</td>
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<td>Calgary Power</td>
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<tr>
<td>Spray River</td>
<td>1500’</td>
<td>96”—84”</td>
<td>900’</td>
<td>1958</td>
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<tr>
<td>Cerro De Pasco Corp</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Padcarthonbo (Peru)</td>
<td>4600’</td>
<td>80”—74”</td>
<td>1730’</td>
<td>1957</td>
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<tr>
<td>Reissneck-Kreuzeck</td>
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<tr>
<td>Europe</td>
<td>13,740’</td>
<td>53”—37”</td>
<td>5800’</td>
<td>1961</td>
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<tr>
<td>Northern New York Utilities</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>South Edwards #2</td>
<td></td>
<td>120”</td>
<td>—</td>
<td>1920</td>
</tr>
<tr>
<td>(laid continuously directly on top of ground)</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Platte Valley Public Power</td>
<td>1200’</td>
<td>160”</td>
<td>228’</td>
<td>1935</td>
</tr>
<tr>
<td>Penn. Power &amp; Light</td>
<td>18,500’</td>
<td>168”</td>
<td>—</td>
<td>1955</td>
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<tr>
<td>State of Montana</td>
<td>450’</td>
<td>84”</td>
<td>—</td>
<td>1941</td>
</tr>
<tr>
<td>City of Denver</td>
<td>220’</td>
<td>78”</td>
<td>—</td>
<td>1937</td>
</tr>
<tr>
<td>City of Denver</td>
<td>9197’</td>
<td>111”—52”</td>
<td>—</td>
<td>1936</td>
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<td>City of Denver</td>
<td>935’</td>
<td>96”—36”</td>
<td>—</td>
<td>1936</td>
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<tr>
<td>State of Colorado</td>
<td>120’</td>
<td>60”</td>
<td>—</td>
<td>1936</td>
</tr>
<tr>
<td>City of Tacoma</td>
<td>1356’</td>
<td>62”</td>
<td>400’</td>
<td>1932</td>
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<tr>
<td>Crowheart Wyom</td>
<td>220’</td>
<td>36”</td>
<td>—</td>
<td>1938</td>
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<tr>
<td>L.A. Co. Flood Control District</td>
<td>1200’</td>
<td>123”</td>
<td>248’</td>
<td>1938</td>
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<tr>
<td>So. Vietnam</td>
<td></td>
<td></td>
<td>3017’</td>
<td>—</td>
</tr>
<tr>
<td>Montrose Colo</td>
<td>660’</td>
<td>96”—60”</td>
<td>—</td>
<td>1947-52</td>
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<tr>
<td>Los Animas Colo</td>
<td>400’</td>
<td>123”</td>
<td>—</td>
<td>1938</td>
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<tr>
<td>Shiprock NM</td>
<td>3500’</td>
<td>60”</td>
<td>—</td>
<td>1959</td>
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<tr>
<td>City of Pasadena</td>
<td>800’</td>
<td>38”</td>
<td>500’</td>
<td>1948</td>
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<tr>
<td>Arizona Public Service</td>
<td>800’</td>
<td>60”</td>
<td>—</td>
<td>1955</td>
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<tr>
<td>Polson, Mont</td>
<td>1890’</td>
<td>48”</td>
<td>330’</td>
<td>1939</td>
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<tr>
<td>Adelaide Australia</td>
<td>150,000’</td>
<td>58”</td>
<td>520’</td>
<td>1951</td>
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<tr>
<td>Nantahala, NC</td>
<td>2750’</td>
<td>120”—96”</td>
<td>1000’</td>
<td>1941</td>
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<tr>
<td>Labrador</td>
<td>2052’</td>
<td>162”</td>
<td>290’</td>
<td>1960</td>
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<tr>
<td>Ontario Paper</td>
<td>700’</td>
<td>216”</td>
<td>117’</td>
<td>1938</td>
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<tr>
<td>B. C. Electric</td>
<td>6300’</td>
<td>75”</td>
<td>1200’</td>
<td>1948</td>
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<tr>
<td>B. C. Electric</td>
<td>1760’</td>
<td>70”</td>
<td>1992’</td>
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<tr>
<td>Niagra Mohawk Power</td>
<td>10,235’</td>
<td>84”</td>
<td>218’</td>
<td>1927</td>
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<tr>
<td>Creede, Colo</td>
<td>700’</td>
<td>84”</td>
<td>—</td>
<td>1934</td>
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<tr>
<td>Lander, Wyo</td>
<td>297’</td>
<td>36”</td>
<td>—</td>
<td>1961</td>
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<tr>
<td>U.S.B.R.—San Luis</td>
<td>9300’</td>
<td>210”—84”</td>
<td>—</td>
<td>1964</td>
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<tr>
<td>U.S.B.R.—Delta</td>
<td>2800’</td>
<td>180”</td>
<td>—</td>
<td>1965</td>
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<tr>
<td>State of California—Buena Vista</td>
<td>4673’</td>
<td>108”</td>
<td>350’</td>
<td>1967</td>
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<tr>
<td>State of California—Wind Gap</td>
<td>10,000’</td>
<td>109”</td>
<td>285’</td>
<td>1967</td>
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<tr>
<td>State of California—Oso</td>
<td>9000’</td>
<td>109”</td>
<td>230’</td>
<td>1967</td>
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<tr>
<td>State of California—Wheeler Ridge</td>
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<td>109”</td>
<td>350’</td>
<td>1967</td>
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<tr>
<td>State of California—Pastoria</td>
<td>1642’</td>
<td>192”</td>
<td>230’</td>
<td>1967</td>
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<tr>
<td>City of Los Angeles—Little Lake</td>
<td>104,897’</td>
<td>78”</td>
<td>600’</td>
<td>1968</td>
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<tr>
<td>Penn. Power Company</td>
<td>18,500’</td>
<td>168”</td>
<td>—</td>
<td>1955</td>
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<tr>
<td>Lake Waulenpaupack Pa</td>
<td>8500’</td>
<td>176”</td>
<td>—</td>
<td>1956</td>
</tr>
</tbody>
</table>
MOMENT OF INERTIA OF PIPE, TAKEN ABOUT DIAMETER AS AXIS

\[ I = 0.04909 \left( d_1^4 - d_2^4 \right) \]

- \( d_1 \): Outside diameter in Inches
- \( d_2 \): Inside diameter in Inches

Approximate formula:

\[ I = 0.4 \ t \ d^3 \]

- \( t \): Thickness in Inches
- \( d \): Average diameter in Inches
REQUIRED MOMENT OF INERTIA STIFFENER RINGS TO PREVENT PIPE COLLAPSE FROM EXTERNAL PRESSURE

Radius “r” (inches)

\[ I = \frac{0.37r^4L}{E} \]

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

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

\( L \) = Distance between rings (inches)

\( P \) = Collapsing pressure (psi)

\( E = 29,000,000 \) psi

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

Las Vegas, NV 108" I.D. Pipe
Useful Publications

Arranged by subject chronologically

COATINGS

Useful Publications (continued)


CORROSION AND CATHODIC PROTECTION

Useful Publications (continued)


Useful Publications (continued)

44. “Pipeline Corrosion Survey Techniques” by A.W. Peabody, Materials Protection (NACE), April 1962.
49. “Understanding and Controlling Corrosion” by Harry Neugold, Jr., Civil Engineering, March 1965.
Useful Publications (continued)


DIAMETER SIZE


EXTERNAL LOADS

13. “A Study of the Collapsing Pressure of Thin-Walled Cylinders” by Rolland G. Sturm - University of Illinois Engineering Experiment Station, Bulletin Series No. 329, November 11, 1941.
Useful Publications (continued)

Useful Publications (continued)


FITTINGS AND APPURTENANCES
13. “Fatigue Strength of Gusseted Pipe Bends” by D.S. MacFarlane, British Welding Journal, Vol. 9 No. 12,

GENERAL ARTICLES
15. “Welding 78” Steel Supply Line for Seattle Water Department” by G.W. Desellem, Engineering News Record, August 16, 1934.
31. “Steel Pipe Feeder for Orange County Cities Completed”, Southwest Builder and Contractor, May 1941.
Useful Publications (continued)

44. “25-Mile Conduit Add 100 m.g.d. for Portland” by G.B. Arthur, Public Works, August 1953.
Useful Publications (continued)

64. “Construction Problems and Welding Controls of Large Steel Pipelines” by Peter J. Bier - Water Power, March 1964.
71. “Use of Pipelines in the California Water Project” by Alfred R. Golze, Journal, Pipeline Division, American Society of Civil Engineers, November 1967. (Discussion Journal, Pipeline Division, ASCE, October 1968.)
74. “Selection of Materials for Large Diameter Water Pipe” by D.P. Proudfit, Preprint 606, Transportation Engineering Conference at San Diego, California, American Society of Civil Engineers, February 1968.
81. Steel Plate Fabricators Association Bulletins:
   a. Wherever Water Flows Steel Pipes It Best.
   d. Steel Water Pipe - Economical to Install - Economical to Maintain.
   e. The High Carrying Capacity of Steel Water Pipe.
   f. Ductility and Adaptability of Steel Pipe.
   g. The Great Reliability and Resiliency of Steel Pipe.
   h. Bottle-Tight Joints.
   “The Steel Pipe Line” Nos. 1-P, 2-P, 3-P, 4-P, 5-P and 6-P.
Useful Publications (continued)


87. “Suspension Bridge Carries Pipeline Across Canyon” by Stephen Chen and John McMullan, Civil Engineering, May 1974.


HYDRAULICS


Useful Publications (continued)


INSTALLATION

PENSTOCKS
2. “Steel Penstock Design by a Graphical Method” by Peter Bier, Engineering News-Record, October 20, 1927.
6. “Penstocks for Boulder Dam” by C.M. Day and Peter Bier, Mechanical Engineering, August 1934.
Useful Publications (continued)

40. ASCE Proceedings Symposium Series No. 4, March 1961 - “Symposium on Penstocks”.
   d. “Penstock Experience and Design Practice” by Gordon V. Richards (with discussion) - Proc. Paper 1397.
   e. “Large Spiral Casings of T-I Steel” by E.L. Seeland (with discussion) - Proc. Paper 1398.
Useful Publications (continued)

44. ASCE Power Division Specialty Conference - Denver, Colorado - August 1965.
   a. “Penstock Codes - United States and Foreign Practice” by Andrew Eberhardt, (P. 725)
   b. “Service Record Experience of Steel Penstocks” by Walter H. Cates. (P. 771)
   c. “Use of High-Strength Steels in Penstocks” by Russell G. Hornberger. (P. 807)
   d. “A Review of Penstock Branch Connections” by Frederick O. Ruud. (P. 829)
   e. “Planning and Design of Cabin Creek Pumped-Storage Hydro-Electric Project” by Lawrence M. Robertson. (P. 211)
   f. “Operating Experience at Taum Sauk” by George J. Vencill. (P. 228)

SUBMERGED PIPE LINES

7. “Floating a Pipeline into Place” by Harry U. Fuller, Engineering News-Record, September 11, 1941.
Useful Publications (continued)

13. “Pulling a 1,040-Foot Outfall into the Ocean in Two Hours”, Western Construction, February 1952.
22. “Pipeline River Crossings” by Leo M. Odom, Journal, Pipeline Division, American Society of Civil Engineers, June 1957.
23. “Outfall Pipeline Pulled 7 Miles to Sea” by D.R. Miller, Western Construction, July 1957.
Useful Publications (continued)


SUPPORTS AND RING GIRDERS

5. “Siphon Self-Supporting in Long Spans” by P.J. Bier, Engineering News-Record, June 20, 1940.
18. “Power Penstocks” by P.J. Bier, Water Power, Parts 1, 2 and 3, June, July and August 1958.
Useful Publications (continued)


WATER HAMMER


Useful Publications (continued)


Other Useful Publications

Steel Plate Engineering Data Volume 1 — Steel Tanks for Liquid Storage.
Steel Plate Engineering Data Volume 2 — Useful Information on the Design of Plate Structures. Note Vol. 1 and 2 are in one document published by AISI/SPFA.
Steel Plate Engineering Data Volume 4 — Buried Steel Penstocks, Published by AISI/SPFA.
Steel Design Manual, Brockenbrough & Johnston, Published by U.S. Steel Corp.
Publications by SPFA:
Critical Vacuum in Buried Thin-Wall Steel Pipes, Buried Structures Laboratory, Utah State University, AISI, Dec. 1989.
Trench Widths for Buried Pipes, Reynold K. Watkins, Advances in Underground Pipeline Engineering, ASCE
## Standards and Specifications

<table>
<thead>
<tr>
<th>Standard</th>
<th>Description</th>
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<tbody>
<tr>
<td>ANSI/AWWA C200</td>
<td>Steel Water Pipe 6 in. and Larger.</td>
</tr>
<tr>
<td>ANSI/AWWA C203</td>
<td>Coal-Tar Protective Coatings and Linings - Enamel and Tape - Hot Applied</td>
</tr>
<tr>
<td>ANSI/AWWA C205</td>
<td>Cement-Mortar Protective Lining and Coating for Steel Water Pipe - 4 in. and larger - Shop Applied</td>
</tr>
<tr>
<td>ANSI/AWWA C206</td>
<td>Field Welding of Steel Water Pipe</td>
</tr>
<tr>
<td>ANSI/AWWA C207</td>
<td>Steel Pipe Flanges for Waterworks Service - Sizes 4 in. through 144 in.</td>
</tr>
<tr>
<td>ANSI/AWWA C208</td>
<td>Dimensions for Fabricated Steel Water Pipe Fittings.</td>
</tr>
<tr>
<td>ANSI/AWWA C209</td>
<td>Cold- Applied Tape Coatings for the Exterior of Special Sections, Connections, and Fittings for Steel Water Pipelines.</td>
</tr>
<tr>
<td>ANSI/AWWA C210</td>
<td>Liquid-Epoxy Coating Systems for the Interior and Exterior of Steel Water Pipelines.</td>
</tr>
<tr>
<td>ANSI/AWWA C215</td>
<td>Cold- Applied Petrolatum Tape and Petrolatum Wax Tape Coatings for the Exterior of Special Sections, Connections, and Fittings for Buried Steel Water Pipelines.</td>
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<tr>
<td>ANSI/AWWA C216</td>
<td>Bolted, Sleeve-Type Couplings for Plain-End Pipe.</td>
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**ASTM Standards**

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<tr>
<th>Standard</th>
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<tr>
<td>ASTM A134</td>
<td>Standard Specification for Pipe, Steel, Electric-Fusion (Arc)-Welded (Sizes NPS 16 and over)</td>
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<tr>
<td>ASTM A2831283M</td>
<td>Specification for Low and Intermediate Tensile Strength Carbon Steel Plates. Standard</td>
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<tr>
<td>ASTM A5701570M</td>
<td>Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality. Standard</td>
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