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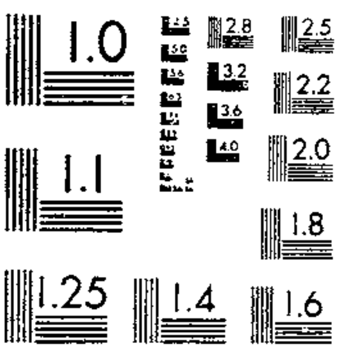

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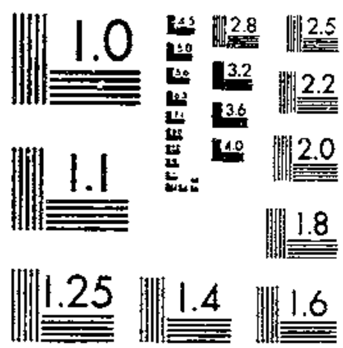

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TECHNICAL BULLETIN No. 150 JANUARY, 1930

UNITED STATES DEPARTMENT OF AGRICULTURE
WASHINGTON, D. C.

THE FLOW OF WATER IN RIVETED STEEL AND ANALOGOUS PIPES

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CONTENTS

INTRODUCTION

The carrying capacity of pipes made of sheet and plate steel or iron as used in general service for the conveyance of water under pressure is discussed in this bulletin. The discussion does not include pipes of cast iron, lead, brass, tin, or iron or steel pipes lined with various materials, such as cement, concrete, or wood, nor does it include pipes flowing partly full. Corrugated metal pipes flowing part full are covered in other publications (*182, 183*).[1] The majority of pipe lines discussed in this publication are of riveted steel.[2]

The research work, in conducting field experiments and in collecting all other known data on the subject, was for the primary purpose of determining the proper capacity of pipes for the conveyance of water for irrigation use. The laws thus developed of course apply equally well if the water is to be conveyed for power, domestic or other use.

[1] Italic figures in parentheses refer to "Literature cited," p. 128.

[2] Essentially the subject is that of the flow of water in riveted steel and analogous pipes; that is, in other sheet and plate metal pipes that are partially riveted or have been formed by methods that replace the riveting process. The discussion is based on field tests for the most part; these were made on pipes in commercial operation as distinguished from perfectly straight lines set up for laboratory tests. This bulletin is offered for use of engineers and other officials designing and operating metal pipe lines (except cast iron) for irrigation, power, municipal, mining, dredging, or other purposes, and for courts and attorneys at law interested in cases involving the carrying capacities of such metal pipes.

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Examination of practice in selecting pipe for conveying irrigation water shows that sheet steel and iron pipe divides the field with concrete and wood-stave pipe and, to a limited extent, with vitrified-clay pipe. Many irrigation systems in southern California have extensive mileage of both metal and concrete pipe operating under conditions essentially identical. In municipal use, main trunk lines from the source of supply to the city are built of many materials, with steel sheets or plates, concrete, and wood staves predominating. In the development of hydroelectric power the flow lines under moderate pressure are constructed of wood staves, concrete, and steel plates, but penstocks down the main power drops are almost invariably made of steel plates, either riveted or welded. As a rule large inverted siphons on irrigation, power, or municipal supply lines are made of steel plate, concrete, or wood-stave pipe.

This bulletin deals with pipes of nominal 4-inch size and larger. Several bulletins recently published by various institutions discuss flow of water in particular kinds of small metal pipes. To save space, relevant detailed data from these bulletins will not be repeated, but the net results of each experiment cited will be converted to a common basis for comparison. The bulletins referred to comprise the following citations: (*68, 72, 73, 31*).

Heretofore the designation "riveted pipe" or "steel pipe" has generally been considered specific enough as a basis for recommendations covering capacity, but such pipe will now be separated into three major classes: (1) Full-riveted pipe, (2) girth-riveted pipe, and (3) continuous-interior pipe. Different coefficients of retardation will be suggested for each class. It will be shown that thin-sheet pipe with flat-head rivets well buried in coating material—a type of pipe commonly used in irrigation practice—has a capacity appreciably above that of plate pipe with the usual prominent rivet heads. It has long been understood that metal pipe deteriorates in capacity with age, but enough significant data are now at hand to form the basis of a reasonable tentative law for deterioration.

It has been often suggested that the laws of fluid similarity and the viscosity of water as influenced by its temperature be considered in the derivation of capacity formulas. This has been done, but in making recommendations the author suggests coefficients for a temperature of 15° C. (about 60° F.), and then shows the percentage difference in capacity for any other temperature that may be used as a criterion—this difference, however, is so small that it may be neglected for most practical cases. This is true because the coefficient which probably would hold in any particular conduit at any particular time is much further from precise determination in advance than any difference temperature would produce. If it had not been true, such formulas as the Williams-Hazen could not have attained their high standing through a period of 25 years and still disregard all influence of viscosity, except as this influence existed in the base data used in the formulas' derivation.

NOTATION

Unless otherwise noted the various symbols used throughout this publication will have the following significance:

d = Mean inside diameter of the pipe in inches.
D = Mean inside diameter of the pipe in feet.
Q = Mean discharge of the pipe, during the test, in second-feet.
A = Mean area of the pipe bore, in square feet.
V = Mean velocity of the water, during the test, in feet per second.
L = Length of reach tested, in feet.

h_f = Head of elevation lost in overcoming internal resistance, in feet $=\frac{HL}{1,000}$.

H = Above loss (usually termed friction loss), per 1,000 linear feet of pipe $=\frac{1,000\ h_f}{L}$.

h_v = Head of elevation lost in creating the mean velocity, V, in feet; called velocity head.

P = Wetted perimeter; in a pipe under pressure, the inside circumference, $=\pi D$ or $2\pi r$, in feet.

R = Hydraulic radius $=\frac{A}{P}$; in a circular pipe, under pressure, $=\frac{D}{4}$, in feet.

s = Hydraulic grade or slope, in feet per foot of length of a pipe of uniform size $=\frac{h_f}{L}$.

C = So-called "coefficient of retardation" in the Chezy formula.
n = "Coefficient of roughness" (*67*) in Kutter's formula. As it appears in the formula, n is not a coefficient, mathematically considered.
C_w = Retardation coefficient in the Williams-Hazen formula, as named by its authors (*180*). Some present-day writers call it "Hazen-Williams."
f = Retardation coefficient in the Weisbach formula.
K = Retardation coefficient in the general exponential formula for flow of water in pipes.
K_s = General coefficient in the particular formulas offered in this bulletin on pages 10 and 79. K_s' is this value for new pipe.
M = Retardation coefficient in any one particular pipe equation (14, p. 79). It is the intercept on the line $V=1$ when the equation is platted on logarithmic paper as in Figure 3.
M' = Intercept M when projected from individual points at the accepted slope of 1.9. Used only where data are insufficient for development of M.

ν = Kinematic viscosity of water $=\frac{\text{absolute viscosity}}{\text{density}}$, given in the C. G. S. system and in English units in Table 6.
m = Retardation coefficient when viscosity is considered, as in formula 20, page 79, and in column 15, Table 1.
e = Base of Naperian logarithms; equal to 2.7183; found in exponential formulas involving the laws of organic growth, of organic decay, of compound interest, and others. In this bulletin such a law fits the data as to the increase in the retardation coefficient with the passage of time, t years. (See p. 89 and fig. 7.)

Sheet-metal thickness is referred to by gauge number for pipe use, ranging from one-fortieth inch for No. 24 to three-sixteenth inch for No. 7. Plate metal runs from three-sixteenths inch to 1¼ inches or more in thickness.

Each reach of pipe tested is given a number, carried consistently through Tables 1 to 4 (pages 23 and 63), Figure 3, and the description of the experiments given in the text or the Appendices. These should not be confused with the numerical references to the literature citations, which are in italics.

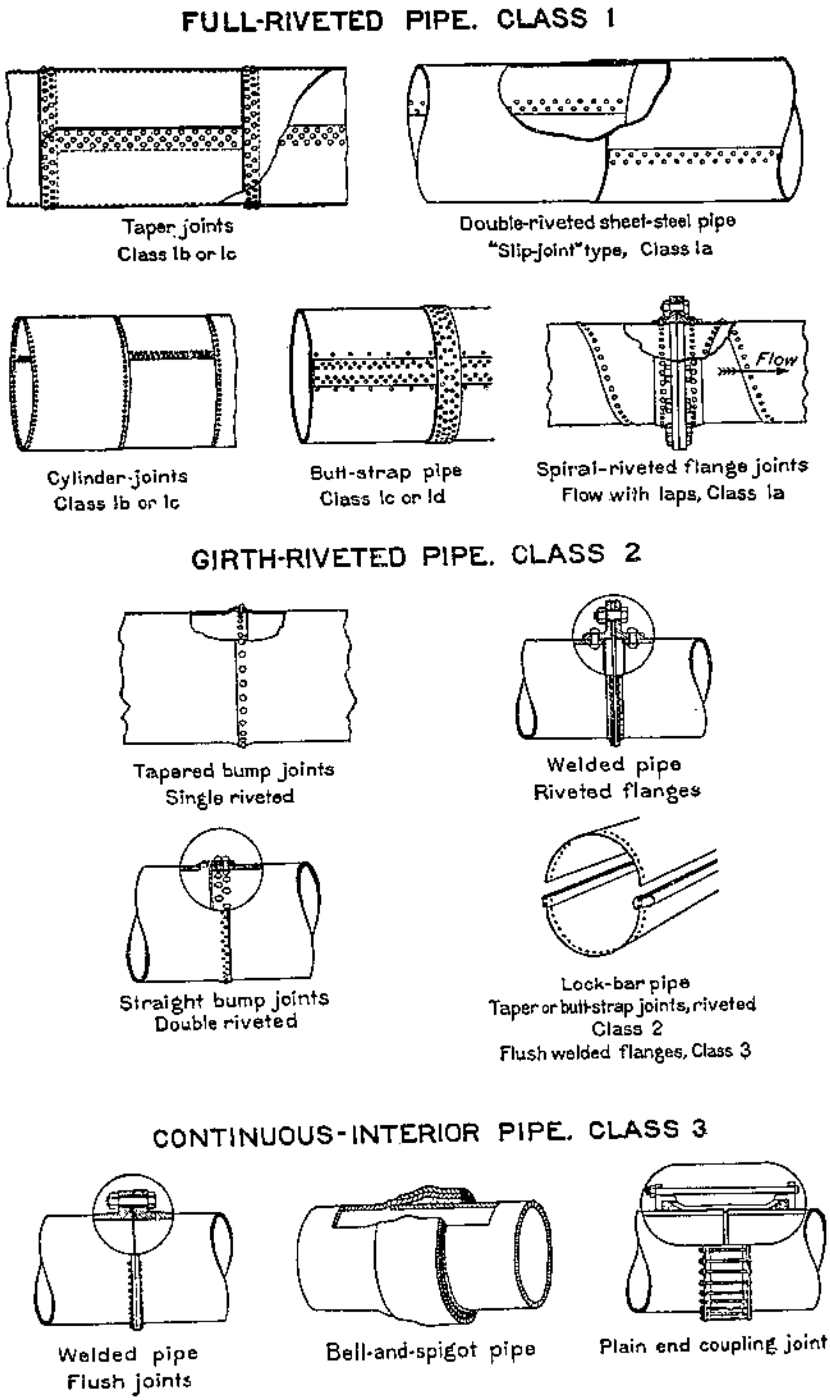
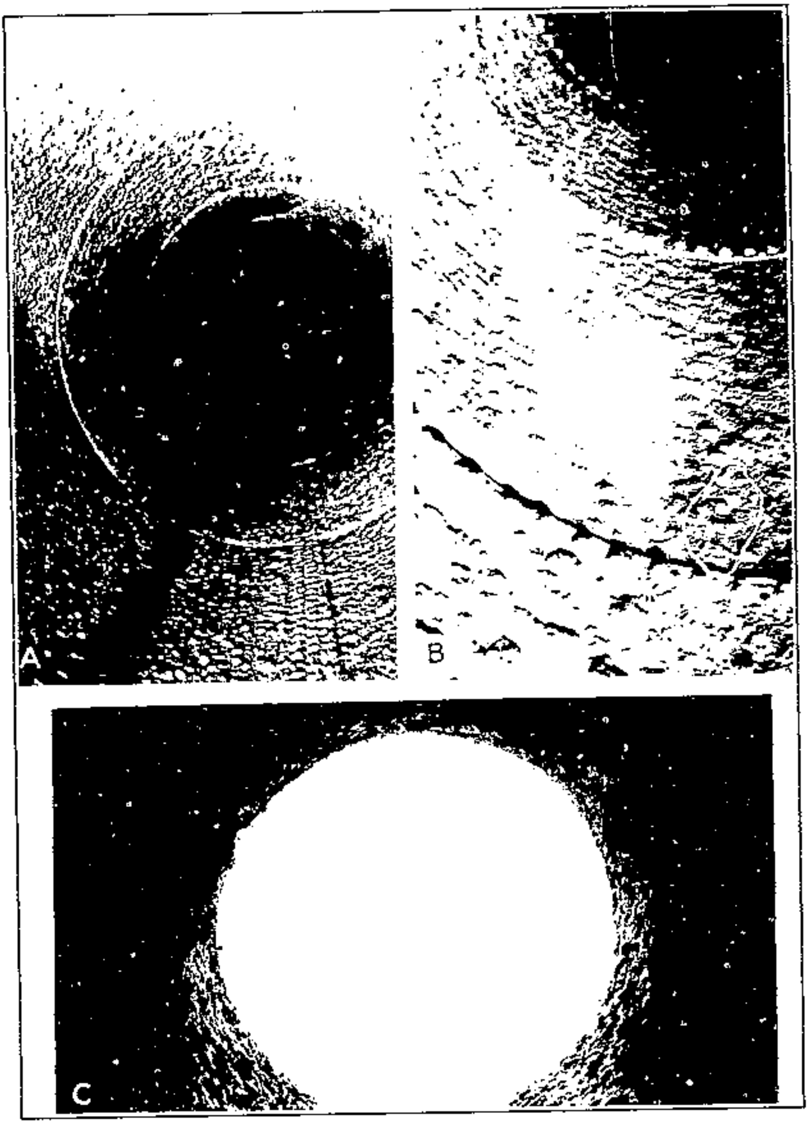


FIGURE 1.—Typical joints that distinguish the various types of riveted steel and analogous pipes. Full welded pipe is omitted as the interior is best shown in photographs as on Plate 1. (See p. 12 for explanation of subdivisions of class 1 pipes.)

A.—New [illegible] irrigation pipe. [illegible] joints, looking against direction of flow.
B.—New spiral [illegible] pipe. [illegible] welded, [illegible] and field joints. Continuous interior.
C.—New lock-bar pipe, San Francisco, Calif., 58 inches in diameter. Surface is smooth and glossy.
D.—New [illegible] pipe, East Bay municipal utility district, California. Note welded straight seams [illegible] for girth seams, [illegible] inches in diameter. Note glossy coat when first installed.



A. Sheet metal pipe, 9 years old. Conveys irrigation water pumped from delta cone of Kaweah River, Calif.
B. Wrought-iron, riveted pipe about 50 years old. Springfield, Mass. Taken up, cleaned of tubercles, recoated, and relaid for secondary service. View by courtesy of Allen Hazen.
C. Riveted steel pipe after about 8 years. East Bay region, California.

TYPES OF SHEET AND PLATE METAL PIPE

Types of pipe will be discussed only with reference to capacity. Figure 1 shows several methods of jointing pipes which have a direct influence on the interior surface of the pipe and hence on the carrying capacity. These surface differences are immediate in effect, and are in addition to corrugations of coating or later developments of chemical action, rust blisters, and tubercles. Sheet-metal pipe is made in thicknesses from one-fortieth inch (No. 24 gauge), up to three-sixteenths inch (No. 7 gauge). Plate-metal pipe is made from plates three-sixteenths of an inch up to 1¼ inch or more. Obviously the thickness of metal has a bearing on the retardation of flow. Where the metal is lapped, thin sheets present slight offsets but plates offer material obstruction. Thin sheets can be crimped into unobtrusive seams or riveted with flat-head rivets well buried in a coating, while riveted plate pipe is usually seamed with protruding cone-headed rivets. Occasionally rivet heads are countersunk more or less completely. (Pl. 5, A.)

For pipe up to 5½ feet in diameter a single longitudinal seam is necessary; from 5½ to 11 feet, two seams; from 11 to 16 feet, three seams; and from 16 to 21 feet, four seams (*32, p. 424*). Butt-joint pipe usually means "double butt-strap pipe" with straps both inside and outside. (Pl. 6, A.) For butt-joint pipe with the outside strap only (pl. 3, C), the plate thickness is not manifest on the pipe interior, but the difference in rivet heads still holds.

In order that the reader may have a definite understanding of the terms applying to the various pipe lines and an appreciation of the marked differences in methods used in assembling unit sheets or plates into completed pipes, the types most commonly used are described below.

Full-riveted pipe has all seams, longitudinal, girth, or spiral, held by projecting-head rivets.

Girth-riveted pipe has circular, "roundabout," or girth seams only, riveted, the longitudinal or straight seam being welded, crimped, or "locked" into a continuous, more-or-less smooth bead. Welded pipe with screw joints also comes under this class, the excess threads on the inside being taken as the equivalent of girth-rivet heads.

Continuous-interior pipe has longitudinal seams as in class 2, while girth seams are not evidenced by any material interior obstruction; includes full-riveted or girth-riveted pipe if all rivet heads are countersunk flush with the surface.

FULL-RIVETED PIPE

Full-riveted pipe is made with lap or butt joints. Types of joining are:

Cylinder joints, also called "in-and-out," or "parallel" joints made with alternate rings of inner (smaller), and outer (larger), "courses" of pipe, which produce an enlargement and contraction of the water prism to the extent of the shell thickness for each ring of pipe. The girth seam is generally single riveted, but sometimes double-riveted. Longitudinal seams are single, double, triple, or even quadruple, depending on the pressure head. The nominal diameter of the pipe is that of the inside of the smaller rings.

Taper, slip-joint, or stovepipe joints are similar to cylinder joints, except that each ring has a slight taper and is lapped outside the ring upstream and inside the ring downstream. (Fig. 1.) The term "taper" is usually applied to pipe made of plates three-sixteenths of an inch thick or thicker, and rarely less than 24 inches in diameter. Thin sheet pipe is now made of tapered units and generally termed slip joint pipe. As implied, the field joints are made without rivets, the end of one length merely being slipped into the larger end of the adjoining length. Tightness is secured by preheating the coatings of both pipes at the joint. Slip-joint pipe is made in sizes from 4 to 36 inches and of metal up to 10 gauge in thickness. In sizes over 20 inches of 10-gauge sheets and heavier, the field joints may be riveted if desirable.

Spiral-riveted pipe consists of a single ribbon of metal wound spirally with the edges continuously overlapping and riveted together. Lengths up to 20 feet for galvanized and 40 feet for asphalted pipe are united by bolted flanges, slip joints, or shrunk-and-peened joints. The standard practice is to lay the pipe so that the flow is "with the laps." (See arrow in fig. 1.) Sizes range from 3 to 42 inches.

Butt-strap joints consist of a band or strap of steel, girting the outside of the pipe over the abutting squared ends of the pipe rings, with a wider longitudinal strap on the inside of the pipe, forming the straight seams. Most pipe of this type (termed "double butt-strap" pipe), has, in addition, a narrower strap along the straight seams on the outside of the pipe. The outer rows of rivets pass through the wide inside band and the main plate only; the other rows extend through both straps and the ring plate. (Pl. 3, C.) In practice the inside straps sometimes form a continuous band down the top of the pipe (pl. 6, A) and sometimes alternate in position. The abutting pipe rings of course favor high carrying capacity for the line, but the large number of rivet heads—eight rows in the case of quadruple-riveted pipe—appears to have a marked retarding influence. Where the inside strap is not continuous, there is, of course, a series of strap ends to retard the flow.

GIRTH-RIVETED PIPE

In recent years several kinds of steel pipe have been placed on the market, which have more or less smooth longitudinal joints in contrast to the riveted joints of the pipe described above. These are usually sold under specific trade names. They subdivide into the following:

Locked-seam pipe is at present made in two general ways: (1) In units having straight longitudinal seams, and (2) in units with spiraled seams from end to end of the unit. The former type is made of both sheet and plate metal. Sheet-metal pipe is extensively used in some States for farm irrigation, the metal ranging from No. 26 up to No. 20 gauge. Pipe diameters range from 3 to 12 inches or larger. Lengths are the same as that of the sheets, usually being 10 feet. The crimped lock seam may be soldered and also tack-riveted.

The plate pipe is made in diameters of 20 to 72 inches with plate thickness from three-sixteenths to one-half inch. The interior of the assembled line is unbroken except for two longitudinal beads of the locking rods and the girth seams every 30 feet. These girth seams are roughened by a single or double row of rivet heads for the taper-joint pipe and at least two rows of rivet heads for either the riveted-flange joint or the outside butt-strap joint. However, the taper joint has an offset causing expansion of the jet to the extent of the plate thickness, while the flange or butt-strap type allows the ends of the pipe shell to be flush. The retardation caused by the double row of rivets would probably be approached by that of the single row plus the influence of the jet expansion. This type of pipe is factory-dipped vertically after being preheated. One of the first pipes of this kind was the famous Coolgardie line (No. 314) laid about 1902 in Australia. However, this particular pipe had abutting joints under leaded sleeves; hence the interior is classed as "continuous."

The spiraled lock-seam pipe is a new product made of sheet metal from 16 up to 10 gauge, in diameters from 4 to 30 inches. The interior is continuous except for a small spiral groove adjacent to a thickness of metal ribbon reinforcing the seam. It is probable that the influence of this ribbon is less than that of the flat rivet heads in the spiral-riveted pipe.

Seamless pipe has either been drawn through a die or has adjoining edges of the plates so fused that practically no joint is perceptible. This type of pipe has been used extensively in Europe and to a lesser extent in this country.

Welded pipes are of two kinds, hammer weld and electric weld. Hammer-weld pipe has the longitudinal joint made as the term implies; the edges of the plate are hammer forged into a tight joint. The pipe sections are made of plates one-fourth inch to 1½ inches thick, in sizes from 20 to 108 inches or more in diameter. The field (girth) joints are made in various ways, which are classified as girth riveted or continuous interior. (Fig. 1, p. 4.)

Electric-weld pipe ranges in diameter from a few inches to about 60 inches. The smaller sizes are shop welded on the longitudinal seams and field welded on the girth seams joining unit sections, placing them in the class of continuous-interior pipe. This type of pipe is also welded spirally with abutting edges, in diameters from 4 to 48 inches. In sizes above 60 inches the long seams are successfully welded in the shop, but field welding of girth seams, without the use of a butt strap, has not been successful as yet and the present practice is to rivet the girth seams. (Pl. 1, B (*38*).) These seams are usually made, for large penstocks, by means of the bump joint, shown in Figure 1. The unit rings are usually of the same size, without taper, but the ends are crimped or bumped to form a lap and are then single riveted or double riveted. The minimum size for this type of joint is 24 inches. Bump joints are also used for units having straight seams riveted instead of welded.

CONTINUOUS-INTERIOR PIPE

Where the longitudinal seam is formed in one of the ways mentioned under girth-riveted pipe and, in addition, the girth joint offers no obstruction to the flow of water, the interior surface of the pipe may be considered as continuous and relatively smooth when new. The girth joints are formed in various ways, as shown on Figure 1. The lighter pipes up to moderate diameters are now being welded in the field. As regards capacity the essential feature lies in the flush abutting ends of pipe units and absence of obstruction by rivet heads. Some trade-name pipes are made with both girth-riveted and continuous-interior joints. It is quite obvious that the latter will have slightly superior capacity qualities, other things being equal.

LENGTH OF FIELD UNITS

Metal pipe is usually delivered in the field in lengths of 20 to 40 feet for small, thin-sheet pipe; 20 to 24 feet for large riveted or welded pipe; and up to 30 feet for certain kinds of patent-joint pipe.

The large riveted pipes are usually shop assembled in three to five rings, or courses. The rings are made of plates from 5 to 8 feet wide.

Spiral-riveted pipe comes in lengths up to 40 feet for asphalt-coated pipe, and 20 feet for galvanized pipe.

It is obvious that the number of girth seams should be kept a minimum because of the effect on capacity.

NOMINAL DIAMETERS

In all computations of pipe capacity it is important that any appreciable differences between nominal and actual interior diameters be given full consideration. In manufacturers' catalogues pipe is known by its nominal inside diameter up to 15 inches. Beyond that the outside diameter is the nominal diameter. All differences in shell thickness affect the inside diameter only. The nominal diameter is appreciably less than the actual diameter up to about 1½-inch pipe. For larger sizes, up to 15 inches, the nominal and actual diameters are reasonably alike.

For sheet or plate pipe of cylinder-joint type the nominal diameter should be that of the interior of the smaller rings or courses. For similar pipe with taper joints, the nominal diameter should equal that of the interior of the small ends of the courses. Thus for cylinder or taper pipe the average size is larger than the nominal size; hence the average velocity is less than that computed for the nominal size, and the actual retardation is consequently less than that necessary for the nominal-diameter velocity. The extent of these differences is measured by the plate thicknesses, and may be immaterial or may be of some moment, depending on the relationship between the pipe diameter and the plate thickness.

FORMULAS FOR FLOW OF WATER IN METAL PIPE

Water is caused to flow and velocity is created by the force of gravity. The flow follows the general law of falling bodies, and the velocity tends to become constantly accelerated. In a pipe this tendency toward constant acceleration is balanced by the influences retarding the flow, and a uniform velocity results.

For a pipe carrying flowing water under pressure, the difference in elevation H_E (fig. 2), between the surfaces of the water at the intake and outlet, is the effective head through which the force of gravity acts. The lost head of elevation is made up of several individual losses, as follows (fig. 2):

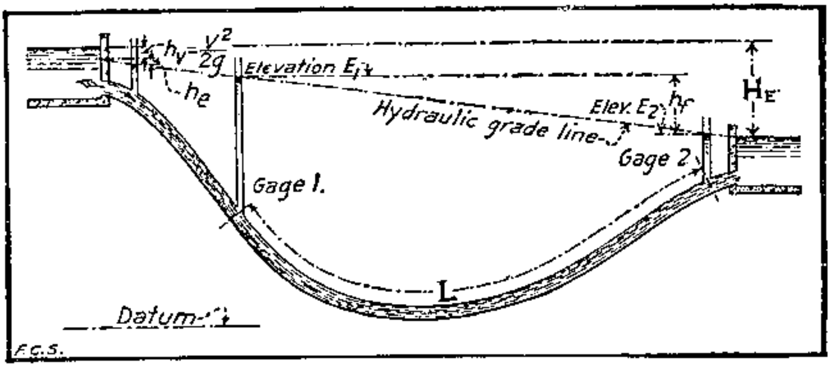


FIGURE 2.—Hydraulic elements for loss of head in pipe

$$\text{Velocity head} = h_v = \frac{V^2}{2g} \tag{1}$$

This is the head absorbed in creating the mean velocity V, at which the water is conveyed through the pipe. This loss occurs at the intake. As a rule, some of this velocity head is recovered at the outlet of the pipe, although often it is not taken into account.

$$\text{Entry head} = h_e \tag{2}$$

The amount of loss at the entry, due to the effect of contraction eddies and other retarding influences, is variable and uncertain, but most authorities agree that it should be taken as half the velocity head, unless the inlet structure is especially designed to minimize it.

Friction head, h_f, is that lost in overcoming the retarding influences. In pipe lines of great length, the amount of this loss so far exceeds the two losses first mentioned that they often may be disregarded, especially if diameters are small. This is the loss upon which the experiments described in this bulletin were concentrated.

In addition to the above losses there may be others, such as those due to bends and valves or other obstructions; but in general, such losses are not considered in the design of pipe for irrigation purposes.

For this use the pipe is generally laid on curves both horizontal and vertical, so gentle that such losses may be disregarded.

If 1775, Chezy, a French engineer, offered his now well-known formula for the flow of water in both open channels and closed conduits:

$$V=C\sqrt{Rs} \tag{3}$$

Here C is a coefficient, originally thought to be constant, but now known to vary as a function of the slope, the hydraulic radius, the velocity, and with some factor representing the retarding influences in the channel. Some of the formulas formerly used in this country for the design of pipes have accepted the Chezy formula as a basis and made only such modifications as experience showed to be necessary.

Since the hydraulic elements secured in the field experiments furnish the necessary data for the determination of the factor representing the retarding influence in all the formulas most used in this country, this bulletin will show the retardation factor as developed by field tests for several formulas, as follows:

(*a*) The Chezy formula (*27, 159, 61, 86, 126*)

$$V=C\sqrt{Rs}=CR^{0.5}s^{0.5} \tag{4}$$

(*b*) The Kutter modification of the Chezy formula, (*103, 67*)

$$V=\left\{\frac{\frac{1.811}{n}+41.66+\frac{0.00281}{s}}{1+\left(41.66+\frac{0.00281}{s}\right)\frac{n}{\sqrt{R}}}\right\}\sqrt{Rs} \tag{5}$$

in which C is elaborated so that it takes into consideration the influences of the hydraulic grade and the mean hydraulic radius, and introduces a new variable, n, which was supposed to represent all the retarding influences. For open channels the value of n has been found to be quite constant, but for pipe of the types discussed in this bulletin it is found to vary with size and velocities over a range of from two to three units in the third decimal place. (See Table 13, p. 98.)

(*c*) The Weisbach formula,[3] which has been used in textbooks as a general formula for flow of water in clean pipes. It is best adapted to use on short lines with many special features so that the several individual losses can be summed up as so many "velocity heads." The following basic equation can be established by usual hydraulic reasoning:

$$h_f=\frac{fLV^2}{2\,Dg} \tag{6}$$

(*d*) The Williams-Hazen general formula (*180*), for many kinds of pipes:

$$V=C_wR^{0.63}s^{0.54}\,0.001^{-0.04} \tag{7}$$

[3] This basic formula, in various forms, can be identified as that of Chezy, Darcy, Weisbach, or Fanning.

which may be arranged in the form—

$$H=k\frac{V^{1.852}}{D^{1.167}} \tag{8}$$

The authors of the formula suggest 110 as the proper value of C_w for new riveted steel pipe, but state that it decreases in the course of about 10 years to 100. (See pp. 85 to 88.)

Some of the above formulas will be taken up again after an analysis of the data, and specific equivalents given so that the engineer familiar with one type of formula and not desiring to change to a new one may have the best suggestions offered by the available data and in terms familiar to him.

(*e*) For the reasons given on pages 77 to 88 the writer offers the following formulas, which include viscosity influence for a temperature of 15° C. and differentiate between various types and classes of sheet and plate metal pipe by means of coefficients, which are themselves constant throughout the whole range of sizes and velocities for a given class of pipe:

$$H=K_s\frac{V^{1.900}}{D^{1.100}}=K_s\frac{V^{1.9}}{D^{1.1}} \quad \text{(Compare with formula 8)} \tag{9}$$

$$V=\frac{H^{0.526}D^{0.58}}{K_s^{0.526}}\text{, for working formula, say } V=\frac{H^{0.53}D^{0.58}}{K_s^{0.53}} \tag{10}$$

$$Q=\frac{0.785\ H^{0.526}D^{2.58}}{K_s^{0.526}}\text{, for working formula, say } Q=\frac{0.78\ H^{0.53}D^{2.58}}{K_s^{0.53}} \tag{11}$$

It should be noted that the above formulas are for new pipe, when used with the values of K_s' given on page 12. It has long been appreciated that all metal pipe deteriorates in carrying capacity; the rate of decrease being dependent on the efficiency of the pipe coating, the composition and treatment of the pipe material and the activity of the water conveyed in the pipe. Filtered and treated water causes less deterioration than raw water. Long lines deteriorate more rapidly at the upper ends than at the lower ends.

So far as available data indicate, the increase in the value of K_s with age (see p. 88 and fig. 6), is expressed by the formula—

$$K_s=K_s'\ e^{0.015t} \tag{12}$$

where t is the age, in years. When t is 0 than K_s becomes K_s'.

In order to study the probable performance of a proposed pipe line throughout a long period of years any one of formulas 9, 10, or 11 may be made progressive by substituting for K_s various values of $K_s'\,e^{0.015t}$ as taken from Table 7, page 89. Thus formula 9 becomes—

$$H=(K_s'\ e^{0.015t})\ \frac{V^{1.9}}{D^{1.1}} \tag{13}$$

Where it is required that the capacity of a line be determined as closely as possible at a certain age to meet the requirements of a given painting, building, or extension program then the values of K_s for that age can be taken from the diagram in Figure 7, or from Table 7. Initial value of K_s' as given in classes 1 to 3 below are for new pipe with water at a temperature of 15° C. A variation of 10°

either way affects the computed capacity within a range of only 1.5 per cent. If the flow of a line must be a maximum in very cold weather, then the value of V should be modified by the percentage factor given in the last column of Table 6, page 81. The argument relating to the influence of temperature upon the viscosity of water is given on page 78.

CAPACITY CLASSIFICATION

The sole purpose of any conduit is to convey water from one place to another. Hence relative capacity is a true measure for conduit comparison. Obviously the larger the pipe, the more water it will carry. However, a similar result may be attained, to a marked degree, by improving the character of the interior surface of the line. A quarter century ago, in economic studies of water lines, relative values were assigned to unit costs of pipe of iron, steel, wood staves, concrete, or other material. Consideration was also given to the strength and probable life of each material. There, with a few exceptions, the comparison stopped. To-day, practically no line of magnitude is considered without comparison of the relative capacities of lines of various materials and types of fabrication at various ages. The final results of such studies usually take the form of a request for bids on pipe of a given size for certain material and type of construction, and on larger sizes for other materials and other types of construction that are known to yield pipes of inferior capacity.

A glance at Figure 1 (p. 4), and pages 5 to 9 shows that a great many classes might be considered in a study of sheet-metal and plate-metal pipe. It is here considered feasible to use but three general classes, first assuming that the following premises hold true:

All iron or steel pipe interiors should be chemically protected with a coating that forms the true interior surface, at least during the first years of the life of a conduit. Coatings are many and varied. Some use for a base such materials as asphalt, coal and water-gas tar, and other hydrocarbon compounds. There are a number of compounds and trade-name products. Paint and galvanizing are also used. The life and efficiency of a given coating with a given water is a material factor in the progressive capacity of a pipe line. The method of application has much to do with the life of a coating. No particular coating is associated with a particular mechanical type of pipe. Innumerable combinations are possible.

Since most coatings are of appreciable body and tend to submerge minor differences in original surfaces, many special trade-name pipes merge into the same categories, their interior surfaces being essentially identical after being coated, although the methods of jointing may be quite different.

Conversely, some pipe makers show several types of joints in their catalogues but do not differentiate between capacity classifications. Categories used in this bulletin separate such types.

Obstructions—rivet heads, plate offsets, blisters, tubercles, etc.—have much greater influence than appears possible to the eye. It has long been known that an occasional obstruction has greater effect than might be expected from the combined effect of a number of obstructions.

A pipe with one or two continuous longitudinal projections such as are found on lock-bar pipe or the smaller but rougher beads in most electric-welded pipe will be considered the same as a pipe without such projections. Water flowing at "commercial velocities" does not follow straight lines, and there must be some retardation due to these beads, especially in a sinuous line where considerable "roping" of the water prism takes place. The uncertainty as to the amount of the retardation and the fact that it is overshadowed by the probable condition of the pipe coating seems to justify ignoring these projections.

With the above premises in mind, sheet and plate metal pipe have been classified, and proper coefficients determined for use in formula 9

(p. 10), for reasonably new pipe of each class. The change in coefficient is given under "the effect of age" on page 88.

The three major classes are:

Class 1, *full-riveted* pipe, having both longitudinal and girth seams held by one or more lines of rivets with projecting heads. From a capacity standpoint, pipe with counter-sunk rivet heads on the interior belongs in class 3.

Class 2, *girth-riveted* pipe, having no retarding rivet heads in the longitudinal seams, but having the same girth seams as full-riveted pipe.

Class 3, *continuous-interior* pipe, having the interior surface unmarred by plate offsets or by projecting rivet heads in either longitudinal or girth seams. Not necessarily described as "smooth."

The following are the suggested coefficients:

Class 1–a, $K_s'=0.38$ for new sheet metal up to three-sixteenths inch thick.

Class 1–b, $K_s'=0.44$ for new plate metal from three-sixteenths to seven-sixteenths inch thick, with either taper or cylinder joints.

Class 1–c, $K_s'=0.48$ for new plate metal from one-half inch up, with either taper or cylinder joints, and for plate from one-fourth to seven-sixteenths inch thick when butt jointed.

Class 1–d, $K_s'=0.52$ for new butt-strap pipe of plate from one-half inch up.

It will be noticed that no difference is made, in class 1, between lap-riveted pipe of either cylinder or taper joint. Although there are appreciable differences in these joints, they seemingly reduce to approximate equality in carrying capacity. Lap-riveted pipe with taper joints "shingled" downstream, have enlargements for the water prism at the lower end of each ring, but no contractions for the prism. Cylinder joints, on the other hand, have alternate enlargements and contractions of the water prism. The influence of the contracting offset is compensated for to a certain extent by the fact that the size of pipe and nominal velocity are based on the smaller rings, and that there are somewhat lower velocities and lower friction losses in the larger rings.

No difference is made between single and double or triple riveted pipe. As a rule the number of rows of rivets increases with the thickness of the plates and inherent differences in capacity are included in the classification on the basis of plate thickness.

Class 2. $K_s'=0.34$ for new girth-riveted pipe. This class covers all sheet and plate pipe with continuous-seamed longitudinal joints, but with the girth joints, particularly the field joints, made with the usual rivet heads inside the pipe. It includes lock-bar and hammer-weld pipe with lap or flange-riveted field (girth) joints; electric-weld, hammer-weld, and drawn pipe with riveted bump joints; and all other types with surface continuous except as broken by a girth belt of rivet heads between field units. These girth seams are usually single-riveted when joints are bump or taper type, but of course require at least two rings of rivets for flange or butt-strap joints with exterior strap only.

Class 3. $K_s'=0.32$ for new continuous-interior [4] pipe. This class comprises all types of sheet and plate metal pipe that offer a practically uniform interior surface of relative smoothness. Full-welded crimped slip joint, lock-bar with welded flange or leaded sleeve connections, bell-and-spigot, bolted-coupling pipes, all belong to this class.

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[4] "Continuous-interior" is offered as descriptive of an interior surface unbroken by rivet heads or appreciable shell-thickness offsets. Use of the apparently obvious synonym "smooth" has purposely been avoided, as this description might hold only while the pipe is new.

TREND OF ENGINEERING THOUGHT REGARDING THE CAPACITY OF RIVETED STEEL AND ANALOGOUS PIPES

In the bulletins on the capacity of wood-stave (*152*) and concrete (*153*) pipes a definite trend of thought can be followed, all concentrated in the period of the present generation, as these two materials have come into general use during this time. Riveted pipe, on the other hand was used during the 40 years prior to 1900, and was classed with all other pipes, one formula being used for clean pipe and, sometimes, one for tuberculated pipe. Metal pipe—lead, tin, brass, or cast iron—has been used since the days of the Caesars (*88*). From Brahms (1757) and Chezy (1775) through nearly 100 years, formulas appeared from time to time for flow of water in pipes. (See Table 5, p. 80.)

Darcy and Bazin, after their pioneer experiments at the middle of the nineteenth century, on various kinds of small pipes and conduits, undoubtedly realized that all surfaces were not alike when it came to the conveyance of water. From their investigations Darcy offered his formula, in the binomial form, for clean cast iron and tuberculated cast-iron pipe. Francis (*63*) in 1872 converted this formula into English measures and it has since been used more or less for cast-iron pipe.

By the time Hamilton Smith made his experiments (1873–1876) it was definitely known that the carrying capacity of cast-iron pipe decreased, both by the throttling of the conduit as a result of chemical growths and the excess retardation of velocity within the remaining water prism. However, it was not until 1890 that it was realized that riveted-steel and iron pipe are also subject to this time deterioration, though perhaps in lesser degree than cast iron. Neither Fanning (*53*) nor Flynn (*61*) mentions riveted pipe as being in a category separate from cast-iron pipes, which they discuss at length. It appears quite certain that riveted pipe was first given definite individuality by Herschel (*86*) in 1896.

For at least 30 years riveted pipe has been placed in a distinct category and definite ideas have been expressed in regard to the capacity as distinguished from that of cast-iron pipe. Development of pipes now classed as girth-riveted or continuous-interior commenced about 1900. These have grown up as analogies of riveted pipe and it is but natural that their capacities should have been considered along with that of riveted pipe. It has been known for some time that their capacities exceeded that of full-riveted plate pipe, but the degree of this excess has remained a matter of opinion only.

So far as it has been possible to ascertain, the first large riveted pipes in this country were built by the city of San Francisco. Pracy (*136*), describes the line as a riveted wrought-iron pipe 30 inches in diameter and 15 miles long, laid in 1868. Numerous smaller riveted wrought-iron lines had been laid throughout the hydraulic-mining sections of California from 1852 on. These lines were of slightly tapering sections joined in the field by slipping the small end of one section into the larger end of the next, giving the name stovepipe which is used to this day for this type of joint.

In the early seventies Hamilton Smith, then engineer for mining properties in California, started the classic observations and studies that resulted in his Hydraulics (*159*). He made careful tests of the loss of head for various velocities in pipes from 11 to 30 inches in diameter, and computed the resulting values of C in the Chezy formula, which apparently was the only formula used to any extent in this country until about 1900. (Pipes Nos. 9, 12, 15, 18, 25, 26.)

In 1873 to 1875 the city of Rochester, N. Y., laid what is now known as conduit No. 1. This was a compound line of 36-inch and 24-inch wrought-iron pipe, and 24-inch cast-iron pipe. In 1876, L. L. Nichols, under the direction of J. N. Tubbs (*170*), made measurements that were offered as gaugings, showing the capacity of the line to be above 9,000,000 gallons per day, although it is stated that the original design, by "standard formula," called for a flow of but 7,000,000 gallons. These measurements were apparently given full weight by Hiram Mills and Hamilton Smith, both contemporaries, in making similar gaugings. Both were competent to judge such measurements. Mills (*122, p. 203*) who was at the height of his long engineering career at the time of the 1876 tests, accepted them 40 years or more later with these words:

> It was at first thought that there must be some radical error in the early measurements but we find the change is one to be expected and not inconsistent with the decrease in discharge found in other like conduits and with the decrease that has continued through the following 14 years. [From 1876 to 1890.]

About 1890 two occurrences started an active discussion among hydraulic engineers regarding the capacity of riveted-pipe lines. These were the construction of a 48-inch trunk line to serve Newark, N. J., and additional tests made on the Rochester line, then 14 years old. (See quotation above.) According to Herschel (*86, p. 10*), the size of the Newark line was computed originally by the Lampe formula (No. 3, Table 5), and checked by tables offered by Smith (*159, p. 271*). The figures given out for the capacity of the Rochester line in 1876 justified the resulting size of pipe, which was slightly less than 48 inches in diameter. All the data then available had been used, these being Darcy's tests on a 11¼-inch plate-iron pipe and Smith's tests on riveted pipe in California, the Rochester test, and one by Herschel on a very short reach of 103-inch pipe in the Holyoke trunk line. (Pipe No. 77.)

The Newark line was designed for a capacity of 50,000,000 gallons per day. When it was put in commission in 1892, the draft was only about 20,000,000 gallons per day, and it was not until 1896 that full capacity was required. At this time the maximum capacity was determined as about 35,000,000 gallons per day, from which Herschel estimated the original capacity to have been about 43,000,000 gallons per day.

The second attack on the hydraulic calculations of the Newark line came in the form of Rafter's tests (1890) and Kuichling's tests (1891) of the Rochester line (*140*). Instead of carrying the 9,000,000 gallons per day, reported for 1876, the line carried slightly more than 7,000,000 gallons per day in 1890, less than 7,000,000 gallons per day in 1891, and progressively less in subsequent years. Even if it were conceded that the 1876 tests were erroneous, here was evidence that riveted pipe decreased in capacity from year to year; but, to quote Herschel regarding the computations for the Newark line:

> No allowance was made in the computation for deterioration of carrying capacity by the formation of tubercles. This was omitted because it was then supposed that steel pipes would not deteriorate in this way, like cast-iron pipes, or, as stated in Hamilton Smith's Hydraulics, "would remain free from rust and tubercles."

Thus the years of the early nineties may be said to mark the beginning of a true understanding of the capacity of riveted pipe. Interest in this subject was heightened by the discussion in engineering literature, centering around the measurements at Rochester in 1876 and the subsequent tests of the Rochester line in the years from 1890 to 1895. In dismissing the 1876 figures for the Rochester pipe line, a statement may be made on the basis of comparison with several hundred tests listed in this bulletin. The retardation factors are given for historical reasons, even though they are, perhaps, without value as evidence of capacity. (See pipe No. 150, Table 2, p. 42). Since the Rochester pipe is a compound line, a loss must be assumed for the cast-iron pipe and the remaining loss used in computing retardation factors for the riveted line. For the average flows of three tests the value of C_w in the Williams-Hazen formula is 126 if their recommendation of a value of 130 for new cast-iron pipe be accepted. If a value for the cast-iron portion of the Rochester main be taken as 135 for new pipe the corresponding value for the riveted wrought-iron portion becomes 119.6. Both values for the riveted line are more favorable than should be expected.

In 1897, Herschel published his "115 Experiments" (*86*), reporting the results of measurements on the lines of the East Jersey Water Co. by his assistants, J. Waldo Smith and W. H. Herschel (*86, p. 47*). These figures were supplemented by all other tests on "large, riveted, metal conduits" which were known at that time. The only retardation factor considered was Chezy's C. The only graphic study was made on ordinary coordinate paper. Herschel "declined to evolve a formula" from the 115 experiments. The writer has included the experiments listed by Herschel and made them comparable with all other similar data by computing additional retardation factors for other and later formulas than that of Chezy. (See Tables 1 to 4, inclusive.)

During the late nineties the Kutter formula (see p. 9) was much used for pipe lines, and finally a rather definite value of $n=0.015$ for riveted pipe became accepted.

The next important development was the Williams-Hazen formula. About the beginning of the present century technical literature began to indicate a decided reaction against the use of the basic simple assumption that $V=C\sqrt{Rs}$ or $V=CR^{0.5}\,s^{0.5}$, which may be converted into the general formula

$$H=k\frac{V^z}{D^x} \quad \text{(See Table 5, p. 80.)}$$

where

$$H=1000s,\ z=2,\ \text{and}\ x=1$$

By this time there were available several series of tests on pipe lines of various materials. When plotted on logarithmic paper, with loss of head as ordinates and velocity as abscissas, the observations resulted in a straight-line relation. These lines with rare exceptions were not at a slope of 2.00, but were at various slopes between 1.70 and 2.00 an indication that the loss of head does not vary as the square of the velocity for all kinds of pipe.

While Hagen, Saint-Venant, Lampe, Tutton, and others had offered formulas for flow of water in pipe that made use of fractional exponents it remained for Williams and Hazen to choose one set of exponents and varying coefficients from a consideration of some 1,100 experiments, which undoubtedly agree within reason with the performance of pipes in commercial service. Their experimental data covered pipes of many materials. Under their classification of riveted pipe—the nearest approach to the types covered in this bulletin—the data available included the experiments of Darcy, Hamilton Smith, Herschel, Kuichling, and Marx, Wing, and Hoskins. They had no experiments on pipes now classed as girth-riveted and continuous-interior.

In the Scobey studies of wood-stave pipe (*152*), concrete pipe (*153*), and in the present study, it was found necessary to differ slightly from the formula of Williams and Hazen in offering formulas for the flow in specific kinds of pipe, but the writer has been continually impressed with the value of the Williams-Hazen formula as a framework for use on all kinds of pipes, as opposed to any other one formula. However, no reason is apparent why specific formulas, when supported by sufficient evidence, should not be used in the detail studies for specified types of pipe.

In order to give the opinions of practicing hydraulic engineers on the formulas and suggestions of investigators before them the following items have been collected:

Smith (*159, p 265*), who made experiments on thin-shell pipe in the early seventies, sensed the deduction which the writer makes in establishing a separate category for thin-shell pipe. He states that, in his experiments, riveted pipe, when coated with asphalt, carried as much water as well-made cast-iron pipes similarly coated. This is now known to be incorrect as a general statement for riveted pipe, but it is approximately true for class 1–a pipe, which includes the kinds of pipe tested by Smith.

E. Kuichling says that a value of C (Chezy) of 118 was used in the design of conduit No. 2 at Rochester. (Pipe No. 40.)

In 1897, Goldmark (*70*), used the Weisbach formula (No. 6, p. 9), in the design of the 6-foot riveted-steel, (No. 69), and wood-stave pipe line at Ogden, Utah, which was then credited with being the largest riveted steel pipe of its type. For the stave line he used a value of $f=0.01$ which may be converted to a value of Chezy's $C=160$, and for the riveted steel portion he used a value of $C=120$ "taken at three-fourths the above value." Actual tests made soon after completion of the line indicated that both values of C were too optimistic—by about 30 per cent for the stave line and by at least 10 per cent for the steel pipe. (See pipe Nos. 69 and 69a.)

John R. Freeman, in reporting on a New York water supply line, recommends a value of $n=0.011$ for new riveted pipe and 0.016 for foul riveted pipe, with smooth interior, countersunk rivet heads, and butt joints on girth seams. (Class 3.)

The Coolgardie lock-bar pipe line (No. 314) was designed, using a value of C (Chezy) of 98.

In 1911, Hazen (*82*), wrote that double-riveted pipe should be 4 per cent larger in diameter than cast-iron or lock-bar steel pipe.

For riveted pipe, after years of use, Williams-Hazen suggested a value of $C_w=95$ for design purposes (*180, p. 8*).

The board of consulting engineers of the Los Angeles Aqueduct determined for the riveted-steel siphon pipes a value of Chezy's C of 90, equivalent to Kutter's $n=0.019$ (*112, p. 81*). These relatively low capacity factors resulted from the board's study of the time-deterioration data then available, which included pipes Nos. 21, 60, and 64 and the Newark lines reported by Herschel. Before obtaining the deterioration data this board suggested as coefficient for steel pipe with rivet heads and seams projecting on the interior a value of $n=0.016$.

Parker (*132*) suggests the use of Tutton's formula $V=C_1R^{0.66}S^{.51}$ (*171*) with a value of $C_1=130$ as a working value for new lap-riveted pipe, tarred or asphalted with rivets projecting, and a value of $C_1=112$ for the same type of pipe when "old."

For the Ontario Power Co.'s lines a value of C_w of 110 was used for butt girth joints.

In the study by the Columbia Basin Survey Commission for the proposed Columbia Basin irrigation project, in Washington, a value of $n=0.015$ was used for the design of riveted-steel siphon pipe up to 23 feet in diameter.

The hydraulic power committee of the National Electric Light Association (*127*) after assembling the data on pipes Nos. 65, 66, 67, 68, 72, 73, 74, 79, 152, 156, 156a, 160, 228, and 230, concluded:

> From the experimental data available it is believed that the following values for n (Kutter) are conservative and can be used with safety for purposes of design:

Lapwelded pipe with bump joints	0.013
Thin riveted pipe with lap joints	.014
Pipe of moderate thickness with butt joints	.016
Heavy pipe with triple-riveted butt joints	.018

As a result of a questionnaire submitted by the subcommitee on artificial waterways of the power committee mentioned above, it is reported that the Southern Sierras Power Co. used $n=0.016$ for lap-riveted pipe, $n=0.018$ for butt strap joint pipe, and $n=0.012$ for lap-welded penstocks from 28 to 66 inches in diameter; the Southern California Edison Co. used $n=0.012$ for lap welded pipe and $n=0.017$ for riveted pipe for Kern No. 3 plant (pipes Nos. 156 and 156a); the California-Oregon Power Co. used Williams-Hazen $C_w=100$ for riveted steel pipe. The same value was used by San Joaquin Light & Power Co. for at least three of its plants, and by the Pacific Gas & Electric Co. for many of its riveted penstocks while it used Kutter's $n=0.013$ for the Hat Creek penstocks of riveted steel from 96 to 120 inches in diameter.

In the specifications for a pipe line to increase the water supply of the District of Columbia, prepared in 1925, the War Department states that friction losses were calculated on the basis of the Williams-Hazen formula with $C=100$ for cast-iron and steel pipe and 130 for concrete pipe. The specifications state that, "The riveted steel pipe shall be made 2 inches larger in diameter than other kinds of pipe to make their carrying capacity approximately the same as cast-iron or welded steel."

To sum up: The design of riveted pipe, from a capacity standpoint, was at first based on a comparison with cast-iron pipe without empirical data as support. With the continued use of the Kutter formula in this country, a general value of $n=0.015$ for riveted pipe was accepted. During the past 25 years the Kutter formula has been more or less superceded by the Williams-Hazen formula, especially for the design of power penstocks, a round value of $C_w=100$

being used. However, it is noticeable that the hydraulic power committee of the National Electric Light Association makes suggestions in terms of Kutter's n, the value varying from 0.013 for welded pipe to 0.018 for heavy butt-joint pipe. As for the girth-riveted and continuous-interior types of pipe it has been appreciated, of course, that their capacities exceed that of ordinary riveted pipe, but there have been but few actual field data offered to establish definite relative capacities.

In this bulletin the writer will endeavor to establish:

That the available data warrant an exponential formula satisfying both empirical data and the theory of similarity of fluid flow.

That the suggested formula contains exponents sufficiently removed from those of the Williams-Hazen formula to warrant a change, especially for high velocities and large-diameter pipes.

That there are now available sufficient experiments on girth-riveted and continuous-interior pipes to establish reasonable coefficients as separate categories.

That sufficient progressive data are now available to warrant a tentative formula for time-deterioration in the capacity of riveted steel and analogous pipes.

That the value of Kutter's n varies at least 0.002—for instance from 0.011 to 0.013—for identical surfaces when followed through a wide range of sizes and velocities, thus rendering the Kutter formula unreliable when considered with a constant value of n.

That engineers not wishing to adopt the new formula offered in this bulletin, although it most nearly conforms to the experimental data, should not digress farther than to use the Williams-Hazen formula, which conforms with reasonable closeness to that of the writer for usual sizes and velocities.

NECESSARY FIELD DATA FOR DETERMINING THE RETARDATION ELEMENTS OF VARIOUS FORMULAS

A glance at pages 9 and 10 shows that in a study of the various formulas, all of which contain the same hydraulic elements, the following data must be determined by field tests:

V = the mean velocity of water in the pipe.

h_f = the loss of head due to retardation in a section of pipe within a known distance.

D or d = the internal size of pipe.

The above data having been secured, the coefficients of retardation may be computed for each of the various formulas.

MEAN VELOCITY OF WATER

The velocity of the water flowing in a reach of pipe may be measured in two general ways:

(1) Directly by timing a given volume of water through a known distance.

(2) Indirectly by measuring the discharge of the pipe, thus determining the quantity, Q, and solving the equation $V=\frac{Q}{A}$.

Where the velocity is determined by the direct method the error is probably smaller than where the indirect method is used, unless exceptional facilities for complete measurements, including interior diameters, are at hand.

LOSS OF HEAD DUE TO RETARDATION

Most of the recent experiments on the flow of water in pipes of uniform size have been made with piezometer columns. This was the method used by the writer. If a piezometer (fig. 2, p. 8) be properly attached to the pipe, the pressure in the latter will support a column of water the surface of which is at elevation E_1, on the hydraulic grade line. In the same way the pressure at gauge No. 2 will lift a column to elevation E_2. For a pipe of uniform size the difference between these elevations is the head lost, h_f, due to the retarding influences.

INTERNAL SIZE OF PIPE

The method used in ascertaining the inside cross-sectional area of the pipe is described for each test, where possible. In some cases several sections of pipe, remaining after construction, were measured and their mean inside cross-sectional areas accepted as the internal sizes of the operated pipes. For some tests the external circumferences were measured and the known plate thickness deducted from the external diameter. In other cases the nominal diameter of the pipe was accepted.

SCOPE OF THE EXPERIMENTS

Experiments by the Division of Agricultural Engineering were made for the most part on thin-sheet and plate pipe in irrigation service. It was appreciated that there already existed a great number of tests on riveted and analogous pipes largely used in connection with municipal installations and power plants. It merely remained to discover possible differences due to irrigation practice, and to analyze all the available data with a view to proving any one of the accepted formulas or to the development of a new formula if it should be required. The division made 98 tests on 29 pipes ranging from 4 to 168 inches in diameter. From other sources there have been collected 1,080 observations on 169 reaches of pipe ranging in age from new to 47 years old. Many of these tests cover progressive gaugings on the same reach of pipe, extending over a long period of years, thus furnishing data on the time deterioration of capacity long known to exist. Many of the data assembled have never been made available to the public. The total observations number 1,178 on 198 reaches of pipe segregated as follows:

	Observations	Reaches of pipe
Full-riveted pipes; thin sheet metal	125	38
Full-riveted pipe; plate metal	375	72
Girth-riveted pipe	123	31
Continuous-interior pipe	91	13
Compound pipe	50	9
Dredge pipe	68	11
Corrugated pipe	7	3
Spiral-riveted pipe [1]	339	21
Total	1,178	198

[1] Of these, 201 observations on 14 reaches of pipe are found in reference 73 only. The pipe equations are given in Table 4.

EQUIPMENT AND METHODS EMPLOYED FOR COLLECTING AND INTERPRETING FIELD DATA

With the exceptions given below, the equipment and methods used were those employed in the experiments on wood-stave and concrete pipe, described in Bulletins 376 and 852 (*152, 153*). For the sake of brevity the descriptions will not be repeated.

PIEZOMETERS

For nearly all the experiments the same type of piezometer connection was used. In essentials this was a piece of brass tube three-sixteenth inch in external diameter and 8 inches long. One end was sealed and two 1/32-inch holes were drilled 1 inch apart on the same longitudinal element, the hole nearest the end being 1 inch back from the rounded seal. The tube was ground to a slight taper so that midway of its length it fitted tightly into a 3/16-inch hole in the shell of the steel pipe to be tested. The piezometer holes, being drilled completely through both sides of the tube, gave four pressure orifices. Care was taken to insure that these orifices were neutral to the current of the water in the pipe. For small pipes of relatively great length only one tube was placed at each end of the reach.

On the larger pipes two or three piezometer tubes were used at each end. Any positive or negative pressure influence, due to velocity in the pipe as the water slipped past both sides of the piezometer tubes, was existent at both ends of the reach and need not be considered. The piezometer tubes at both ends of the reach were under the same dynamic conditions so that they reflected only the fall in the hydraulic gradient, h_f, usually termed the "friction" loss. This type of connection was considered as much superior to connections to holes drilled through the pipe shell, which have more or less burr on the inside. It would not have been feasible with most of the pipes tested to turn out the water, drill holes, and smooth off any interior burrs. For the smaller pipes this could not have been done except when the pipe was laid.

When the piezometer tubes were first thrust into the pipe there was usually a slight leakage around the tube, but this quickly stopped. No difficulty was experienced in holding these tubes by friction alone in 3/16-inch plate pipe under a pressure head of 160 feet.

Upon completion of the test each hole was repaired by driving into it a copper rivet, well smeared with asphalt. A 1/16-inch hole was then drilled nearly through the center of the rivet. A small wire nail was inserted in this hole and gently tapped, the bevel cuts at the end of the nail expanding the copper rivet on the inside of the pipe. Care was exercised not to drive the nail through the interior end of the rivet. Some of these rivets were examined 10 years after insertion and were found to be tight and apparently sound.

This type of piezometer connection and pipe repair offered a simple solution to a problem that at first appeared quite complicated, costly, and laborious.

OFFICE EQUIPMENT AND METHODS

Where feasible, original multiplication, division, and addition were performed on mechanical devices or with logarithms, and checking was done by an alternative process. Tables containing uniformly progressive figures were checked by noting differences and recomputing every fifth or tenth item. The discarding of excess figures may result in apparent errors in the last figure, if it be recomputed with only the base figures as listed. Where the difference is slight the listed answer is probably more nearly correct than the recomputed answer.

ELEMENTS OF EXPERIMENTS FOR THE DETERMINATION OF FRICTION OF LOSSES IN SHEET AND PLATE METAL PIPES

Tables 1 to 4 give the elements of nearly all known observations on riveted and analogous pipes under pressure. They do not include published results of laboratory tests on spiral riveted pipes (*73*). However, the individual pipe equations for these pipes and comparisons of observed capacities to those computed by the formula recommended in this bulletin are given in Table 4. The various series are arranged in ascending sizes of pipe and within each series the observations are arranged in ascending order of velocities. The observations for simple pipes are given in Table 1. A slightly different arrangement is necessary for compound pipes, which are listed in Tables 2 and 4. In Table 4 the various series are summarized and data common to all the observations within that series are given, as well as average values of C_w, K_s, and m. This arrangement results in economy of space and places data in a convenient form for certain studies of capacity.

EXPLANATORY NOTES, TABLES 1, 2, AND 3

Column 1 gives the numbers assigned to the pipes. The same order of presentation is followed in Table 4 and in the discussions in the following pages and in the appendices. Experiments conducted by the writer are listed FCS in column 2 and are discussed in the text, while the essential data secured from other sources are abstracted in appendix 1. The initials of the hydraulician making the experiments, where known, are given in column 2 and his full name is given in the discussion. Assisting the writer at various times in the experiments and computations were P. A. Ewing, T. H. McCarthy, E. C. Fortier, G. H. Henderson, W. J. Manetta, and J. M. Brockway.

Column 14 of Table 1 gives the retardation coefficient in the accepted exponential formula (9 and 21) disregarding the small amount of influence due to differences in viscosity of the water. This column is therefore on a par with columns 10 to 13, inclusive. Column 15 gives the values of m as determined by dividing K_s, in column 14, by the value of $\nu^{0.1}$. The symbol ν is the kinematic viscosity (in English units) for the actual temperature of the water (Table 6, col. 7, on p. 81), where this item is given, or for an assumed temperature of 15° C. where it is not given. For most studies the values of K_s in column 14 are sufficient. The other columns are considered as self explanatory. Complete data were not available for a few of the pipes; they may be produced in future.

For the compound pipes, described in Table 2, the base data must be treated in a manner slightly different from that for a simple pipe. For a given quantity, Q, the separate velocities in the various sizes of pipe can be computed. If the pipe be of uniform type then a single retardation factor can be found, in any one of the formulas considered, that will represent all the friction losses (columns 12 and 14), in a given pipe equalling the observed total loss, including enlargement and contraction losses. Where a pipe line has two types of pipe it is necessary to assume a reasonable factor for the type of least influence and compute the corresponding factor for the sizes of the predominating type. As Kutter's n is not a constant throughout the ranges of size and velocities for a given surface of pipe, it is proper only to compute an approximate value for the various parts of a compound pipe.

EXPLANATORY NOTES, TABLE 4

In Table 4, column 6 gives the capacity class according to the specifications on pages 11 and 12. Where the plate thickness varies so as to place parts of the line in different classes the range is listed. Column 11 presents the individual pipe equations of the exponential type (formula 14, p. 79), for all series that covered a range of velocities sufficient to give a definite trend. Where only individual observations or limited series are available the column shows the values of M' as projected at the accepted slope of 1.9. All these values of M and M', when plotted as in Figure 5 (p. 83), justify the acceptance of the exponent 1.1 for D in formula 21, page 79. Columns 12 to 14, inclusive, give the average values of C_w, K_s, and m, after exclusion of all observations given a D rating. Column 15 shows the variation of the carrying capacity of the pipe, as determined from the observed hydraulic elements, from the capacity of the same pipe had it been computed according to the suggestions in this bulletin, giving consideration to type and age of pipe and also the small corrections for temperature of water, where this was given. Where the observed capacity is much less (indicated by minus sign), than the computed capacity, it will generally be found that the pipe is supplied from an open canal and the capacity has been reduced by silting or algæ growths. Where the observed capacity is much greater and the pipes are old the rate of decrease in capacity is much less than usual.

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*

FULL-RIVETED PIPE

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	*°C*						
1	FCS, 1917, 2 years.	C	3.88	L. J. 3-2 Lat. Okanogan project Washington. U. S. Bureau of Reclamation	--------	0.648	0.653	20.0	89.2	126.2	0.0098	0.032	0.426	1.34
		C				.906	1.308	20.0	88.1	121.1	.0100	.033	.453	1.42
		B				1.337	2.76	20.0	89.5	119.2	.0100	.032	.455	1.43
2	HD, 1851, new	C	7.71	New pipe; sheet iron, covered with bitumen; straight; longitudinal seams riveted; screw joints; slightly inclined upward; loss of head by water and mercury manometers; Q by calibrated tanks.	--------	.591	.20	21.0	104.2	141.0	.0096	.024	.335	1.04
		B				.912	.48	21.0	103.8	135.6	.0101	.024	.204	.91
		B				1.529	1.29	21.0	106.2	133.3	.0100	.023	.354	1.10
		A				2.559	3.30	21.0	111.1	134.3	.0098	.021	.341	1.06
		A				3.530	5.80	21.0	115.6	136.6	.0096	.019	.325	1.01
		A				5.436	11.9	21.0	124.3	143.0	.0091	.017	.293	.91
		A				5.509	12.0	21.0	125.4	144.0	.0090	.016	.288	.89
		A				7.411	21.0	21.0	127.5	143.2	.0089	.016	.288	.89
		A				9.000	29.7	21.0	130.2	144.2	.0088	.015	.281	.87
		A				10.013	36.4	21.0	130.9	143.1	.0087	.015	.282	.87
		A				10.718	121.6	21.0	141.1	147.6	.0082	.013	.260	.81
3	F C S, 1917, new.	A	8.0	Ben Dougal private line, Omak, Wash. New galvanized straight pipe; asphaltum coat applied hot.	--------	1.157	.730	18.0	104.8	134.1	.0102	.023	.354	1.10
		A				1.172	.741	18.0	105.5	134.7	.0101	.023	.351	1.10
		A				1.356	1.047	18.0	102.7	129.3	.0105	.024	.373	1.16
		A				1.478	1.210	18.0	104.2	130.4	.0103	.024	.367	1.15
4	HH, 1915, new	B	8.0	Huacal pipe line, Sonora, Mexico	2.534	7.26	28.46	--------	105.4	117.6	.0103	.023	.422	1.31
5		B	10.0	do	2.534	4.65	11.68	--------	94.2	104.7	.0117	.029	.516	1.60

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*—Continued

FULL-RIVETED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	*° C.*						
6	FCS, 1919, 24 years.	B	10.0	Whittier Water Co. near Whittier, Calif. Pipe straight; lap riveted, double-dipped in hot asphaltum; velocities by color; loss of head by water and mercury columns.	--------	1.78	1.42	20.0	103.6	125.3	0.0108	0.024	0.388	1.22
		A				2.10	2.14	19.0	99.3	118.1	.0112	.026	.428	1.34
		A				2.33	2.73	20.0	97.6	115.0	.0114	.027	.448	1.41
		A				2.51	2.92	19.0	101.8	119.6	.0110	.025	.416	1.30
		A				2.54	3.16	19.0	99.2	116.1	.0113	.026	.440	1.38
		A				2.58	3.34	19.0	98.0	114.5	.0114	.027	.452	1.41
		A				2.60	3.30	19.0	99.3	116.1	.0112	.026	.440	1.38
7	FCS, 1919, 8 years.	B	10.63	Shobert ditch siphon, near Placerville, Calif.; lap seams, double riveted, of 14-gauge metal.	--------	1.74	1.52	--------	95.2	113.8	.0117	.028	.465	1.47
		B				1.86	1.76	--------	94.2	112.1	.0118	.028	.474	1.50
		C				2.05	2.51	--------	86.8	101.8	.0126	.033	.562	1.78
		B				2.07	2.21	--------	93.8	110.5	.0119	.029	.487	1.54
		B				2.20	2.65	--------	90.9	105.7	.0122	.031	.521	1.65
8	F C S, 1919, mixed.	A	10.87	City Garden siphon near Placerville, Calif. Pipe recently rebuilt.	--------	2.02	2.56	24.0	84.2	98.4	.0130	.036	.604	1.91
		A				2.28	3.20	24.0	84.8	98.2	.0129	.036	.599	1.90
		A				2.43	3.59	24.0	85.3	98.3	.0129	.036	.596	1.89
9	HS, 1876, 5 years.	A	10.93	North Bloomfield pipe, California, 5 years old; shop riveted in 20-foot lengths; slip joints in field.	3.07	4.71	8.50	--------	107.1	119.1	.0108	.022	.404	1.25
		A			3.97	6.09	13.34	--------	110.6	120.7	.0106	.021	.388	1.20
		A			4.52	6.93	16.95	--------	111.4	120.6	.0105	.021	.386	1.20
		A			5.64	8.66	25.59	--------	113.4	120.7	.0104	.020	.382	1.18
		A			6.53	10.02	33.09	--------	115.5	121.6	.0102	.019	.374	1.16
10	HD, 1851, new	B	11.22	Conduit No. 10. New sheet iron, dipped in bitumen. (See pipe No. 2 above for description.)	.892	1.296	.70	21.0	101.3	124.1	.0102	.025	.395	1.24
		A			1.967	2.782	2.54	21.0	114.0	132.9	.0103	.019	.338	1.06
		A			2.654	3.868	4.33	21.0	121.6	138.4	.0098	.017	.307	.97
		A			3.360	4.902	6.83	21.0	122.5	137.5	.0098	.017	.310	.97
		A				6.673	11.9	21.0	126.5	138.3	.0096	.016	.296	.93
		A				8.852	20.44	21.0	128.1	137.0	.0095	.016	.301	.95
		A				10.522	28.07	21.0	129.9	137.2	.0094	.015	.298	.94

11	FCS, 1917, 3 years.	A	12.0	U. C. 9 Lat. Okanogan project Washington. U. S. Bureau of Reclamation.	--------	1.128	.75	19.0	82.5	99.9	.0133	.038	.595	1.86
		A				1.294	.94	19.0	84.5	101.4	.0131	.036	.580	1.82
		A				1.333	1.00	19.0	84.4	101.1	.0131	.036	.581	1.82
12	HS, 1876, 5 years.	A	12.67	North Bloomfield pipe. 1 of 3 pipes laid side by side. (See No. 9 above and No. 15 below.)	4.02	4.60	6.68	--------	109.4	120.6	.0110	.022	.391	1.21
		A			6.10	6.96	14.28	--------	113.4	121.2	.0106	.020	.381	1.18
		A			7.57	8.65	22.19	--------	113.0	118.7	.0107	.020	.392	1.21
		A			9.38	10.71	33.18	--------	114.4	118.2	.0106	.020	.390	1.21
13	AMcLH, 1896	B	14.0	New Westminster, British Columbia. Pipe 3 years old.	.96	.90	.455	--------	78.1	94.6	.0141	.042	.661	2.05
		B			.99	.93	.455	--------	80.7	97.9	.0138	.039	.622	1.93
13a	1899	B	14.0	Same pipe as above. 3 years later	1.21	1.13	.584	--------	86.6	103.8	.0131	.034	.551	1.71
14	FCS, 1919, mixed	A	14.55	Carlisle siphon, near Placerville, Calif. Lap riveted; part new and part salvaged pipe.	--------	1.68	.878	24.0	102.7	120.7	.0117	.024	.405	1.28
		A				2.11	1.422	24.0	101.4	116.8	.0119	.025	.425	1.35
		A				2.36	1.70	24.0	103.9	118.9	.0116	.024	.411	1.30
		A				2.57	2.05	24.0	103.1	117.0	.0117	.024	.422	1.34
15	HS, 1876, 5 years.	B	14.76	North Bloomfield pipe, California. 1 of 3 pipes in same trench. (See Nos. 9 and 12 above.) Interior quite smooth and free from rust.	5.20	4.38	5.02	--------	111.6	121.9	.0111	.021	.380	1.18
		A			8.13	6.84	10.97	--------	117.8	124.8	.0106	.019	.357	1.11
		A			8.69	7.31	12.27	--------	119.1	125.6	.0105	.018	.352	1.09
		A			10.05	8.46	16.46	--------	119.0	124.0	.0105	.018	.358	1.11
		A			12.59	10.59	24.70	--------	121.6	124.6	.0104	.017	.350	1.08
		A			14.37	12.09	32.31	--------	121.3	123.0	.0104	.018	.356	1.10
16	ALA, 1896	A	16.0	Astoria, Wash. New	6.37	4.58	4.97	--------	112.3	121.5	.0112	.020	.378	1.18
17	FCS, 1919, 3 years.	A	16.0	Lateral No. 20 from Chatsworth high line, Los Angeles, Calif. Cylinder jointed riveted pipe; heavy coat asphaltum buries flat heads of rivets.	--------	2.55	1.433	26.0	116.5	132.4	.0108	.019	.332	1.06
		A				2.99	1.998	26.0	115.9	130.0	.0109	.019	.342	1.09
		A				3.31	2.497	26.0	114.8	127.7	.0109	.020	.352	1.12
		A				3.36	2.490	26.0	116.6	129.6	.0108	.019	.341	1.08
		A				3.49	2.658	26.0	117.2	130.0	.0108	.019	.339	1.08
		A				3.55	2.728	26.0	117.6	130.3	.0108	.019	.337	1.07
18	HS, 1878-9	A	16.99	Texas Creek pipe. 1 year old	31.72	20.14	66.72	16.0	131.1	126.8	.0100	.015	.325	1.01
19	FCS, 1919, 4 years.	B	17.85	Orange Blossom siphon, Oakdale, Calif. Practically straight in plan but on a long sag in profile; heavy asphaltum coat hides rivets.	--------	.688	.104	18.0	111.1	138.1	.0105	.021	.326	1.02
		B				1.143	.245	25.0	119.4	143.7	.0104	.018	.296	.94
		B				1.370	.346	22.0	120.8	143.3	.0104	.018	.294	.93
		B				1.509	.441	19.0	117.9	138.6	.0107	.019	.313	.98
		B				1.597	.439	20.0	125.2	147.2	.0102	.016	.279	.87
20	FCS, 1919, 4 years.	B	17.85	Birnbaum pipe line, Oakdale irrigation district, California. Straight, level line; rivets, flat-headed; heavy asphalt coat; velocity by color; mercury column at upper end, water column at lower end. This part of line probably free from silt, unlike No. 22.	--------	1.99	.852	29.0	111.6	127.7	.0113	.021	.356	1.14
		B				2.09	.961	24.0	110.4	125.7	.0114	.021	.367	1.16
		B				2.45	1.354	26.0	109.2	122.6	.0116	.022	.382	1.21
		B				3.01	1.98	26.0	110.8	122.6	.0115	.021	.378	1.20
		B				3.64	2.44	22.0	120.7	132.5	.0107	.018	.326	1.03
		B				4.31	3.53	24.0	119.0	128.6	.0109	.018	.339	1.07
		B				4.83	4.57	23.0	117.1	125.3	.0110	.019	.354	1.12
		B				5.28	5.415	24.0	117.6	125.0	.0110	.019	.355	1.12
		B				5.52	5.78	24.0	119.0	126.2	.0109	.018	.348	1.10
21	JBL, 1909, 8 years.	B	18.0	Force main, Buena Vista pump station, Los Angeles. Riveted steel, taper joints, 10-gauge metal, 5 bends of 90°, 52°, 43°, 47°, and 10°, respectively.	--------	2.43	1.588	--------	99.6	111.0	.0125	.026	.459	1.42
		B				3.05	2.381	--------	102.0	112.0	.0123	.025	.446	1.38
		B				3.63	3.336	--------	102.8	111.1	.0123	.024	.449	1.39
		B				4.29	4.605	--------	103.3	110.3	.0122	.024	.452	1.40
		B				4.88	6.275	--------	100.6	106.2	.0125	.025	.482	1.49
		B				5.79	8.975	--------	99.8	103.8	.0126	.026	.498	1.54

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*—Continued

FULL-RIVETED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	° *C.*						
22	FCS, 1919, 4 years.	B	20.0	Birnbaum pipe line, Oakdale irrigation district, California. (See No. 20 above and No. 31 below.) Same methods used. This part of line subject to silt accumulation more than other parts tested. Pipe, straight, double-dipped in asphaltum.	--------	1.585	0.631	24.0	97.7	111.5	0.0128	0.027	0.460	1.46
		B				1.62	.674	23.0	96.7	110.1	.0130	.028	.472	1.49
		B				1.67	.696	23.0	98.2	111.7	.0128	.027	.460	1.45
		B				2.425	1.483	26.0	97.5	107.5	.0130	.027	.482	1.53
		B				2.67	1.760	26.0	98.4	107.8	.0129	.027	.477	1.52
		B				3.70	3.28	22.0	100.1	107.6	.0128	.026	.478	1.51
		B				4.11	4.06	24.0	100.0	105.9	.0128	.026	.483	1.53
		B				4.20	4.22	24.0	100.1	105.9	.0128	.026	.483	1.53
		B				4.84	5.61	24.0	100.0	104.6	.0128	.026	.491	1.55
		B				4.84	5.52	23.0	100.9	105.5	.0127	.025	.483	1.53
		B				4.99	5.945	24.0	100.2	104.5	.0128	.026	.490	1.55
23	GWR	B	24.0	Rochester, N. Y., conduit No. 1	10.43	3.32	3.58	--------	78.5	81.6	.0162	.042	.784	2.43
23a	1890	C	24.0	Shorter reach of pipe 23	10.43	3.32	3.83	--------	75.8	78.7	.0166	.045	.839	2.60
24	EK, 1891	C	24.0	About one-third of pipe 23; 15 years old	10.52	3.35	3.46	--------	80.5	83.9	.0158	.040	.744	2.30
25	HS, 1873	A	25.8	Humbug pipe, California; 5 years old	45.92	12.60	16.41	--------	134.1	129.9	.0106	.014	.309	.96
26	HS, 1876	A	29.17	Cherokee siphon, California; 5 years old	50.00	10.78	11.72	--------	127.8	123.5	.0111	.016	.340	1.05
27	LJ, LeC	B	30.0	San Francisco, Calif	--------	--------	--------	--------	107.0	114.0	--------	--------	±.42	--------
28	1911	B	30.0	Spring Valley Water Co.; 42 years old	--------	--------	--------	--------	103.6	110.0	--------	--------	±.44	--------
29	LJ, LeC, 1911	B	30.0	Same system as above; 47 years old	--------	--------	--------	--------	84.0	91.5	--------	--------	--------	--------
30	IWS, 1894	B	33.0	Portland, Oreg.; new	36.37	6.13	3.57	--------	123.7	123.4	.0117	.017	.347	1.08
31	FCS, 1919	B	34.0	Birnbaum pipe line, Oakdale, Calif.; 4 years old.	--------	3.10	.923	26.0	121.3	127.2	.0118	.018	.339	1.08
32	IWS, 1894	B	35.0	Portland, Oreg.; new	36.37	5.45	2.526	--------	126.8	127.4	.0115	.016	.327	1.01
33	FSC, 1919. 2 years.	D	36.0	Pump lift, Lindsay-Strathmore irrigation district. California. Line straight in plan; on continuous upgrade in profile.	--------	1.57	.198	22.0	128.4	142.3	.0111	.016	.281	.89
		B				3.41	1.045	19.0	121.7	126.1	.0120	.017	.341	1.07
		B				4.95	2.389	23.0	116.9	117.2	.0124	.019	.383	1.21
		B				5.96	3.560	18.0	115.3	113.7	.0126	.019	.402	1.25
		B				6.16	3.550	22.0	119.4	117.9	.0122	.018	.376	1.18

34	FCS, 1917	A	36.0	Victoria Aqueduct, British Columbia; 2 years old.	----	1.81	.377	15.0	107.6	116.2	.0131	.022	.409	1.22
35	CH, 1892, new	C	36.0	East Jersey Water Co., cylinder joints, conduit from Belleville to South Orange Avenue line a long inverted siphon; contains 6 curves of radius less than 100 feet; of beveled plates.	4.0	.56	.04	----	102.2	120.7	.0113	.025	.404	1.25
		C			7.9	1.12	.20	----	91.5	101.2	.0148	.031	.540	1.67
		C			13.2	1.87	.55	----	92.0	97.9	.0151	.030	.560	1.73
		C			19.7	2.79	.97	----	103.4	107.5	.0137	.024	.462	1.43
		C			26.6	3.76	1.59	----	108.8	111.0	.0132	.022	.429	1.33
		C			33.6	4.75	2.35	----	113.1	113.5	.0128	.020	.408	1.26
		C			40.3	5.70	2.86	----	123.1	122.5	.0119	.017	.351	1.09
35a	1896	A	36.0	Same reach as above 4 years later	34.8	4.93	2.87	----	106.3	105.8	.0135	.023	.464	1.44
37	JN, LeC, 1911, 24 years.	A	36.0	Alameda pipe, Spring Valley Water Co., San Francisco.	24.03	3.44	1.458	----	104.3	106.4	.0136	.024	.467	1.45
38	JN, LeC, 1911	B	36.0	Another reach of above pipe	24.71	3.465	----	----	122.3	123.0	----	----	----	----
39	GWR	C	36.0	Rochester conduit, No. 1, 1890	10.43	1.47	.45	----	80.2	85.8	.0169	.040	.725	2.24
39a	EK	B	----	Same reach as above, year later, 1891	10.52	1.49	.43	----	83.0	89.1	.0161	.037	.676	2.09
39b	1890-91	7	36.0	Rochester, N. Y., conduit No. 1; constructed 1873; all tabular figures are averages of numbers of observations in column 3, covering years in column 2.	----	1.480	.440	----	81.6	87.5	.0166	.039	.700	2.17
	1892-1900	9				1.452	.438	----	80.1	86.0	.0169	.040	.723	2.24
	1901-1905	5				1.380	.431	----	77.9	83.7	.0173	.042	.760	2.35
	1906-1910	5				1.337	.401	----	77.2	83.1	.0174	.043	.774	2.40
	1911-1915	5				1.341	.409	----	76.6	82.4	.0176	.044	.787	2.44
	1916-1920	5				1.336	.410	----	76.0	81.8	.0177	.015	.796	2.47
40	EK, 1896	C	38.0	Rochester, N. Y., conduit No. 2, from overflow No. 1 to Mount Hope Reservoir on southern division, 26 miles long; constructed 1893-94, of steel plates ¼, 5/16, and ⅜ inch thick; quantity by rise in reservoir, loss of head by mercury gauges; cylinder joints.	----	.612	.051	----	96.5	112.0	.0131	.028	.461	1.43
		C				.758	.061	----	109.5	125.9	.0120	.022	.366	1.13
		C				.888	.081	----	110.9	126.4	.0121	.021	.360	1.12
		C				.965	.125	----	97.0	108.6	.0139	.027	.475	1.47
		C				1.06	.126	----	106.1	118.8	.0120	.023	.399	1.24
		C				1.13	.131	----	110.9	124.0	.0124	.021	.369	1.14
		C				1.19	.142	----	112.2	125.1	.0124	.020	.363	1.12
		C				1.22	.181	----	101.9	112.5	.0136	.025	.440	1.36
		C				1.26	.166	----	100.9	121.7	.0127	.021	.380	1.18
41	EK, 1897, 3 years.	B	38.0	Same reach of pipe as above, 1 year later. Kuichling says these tests, made "with utmost care." They are probably entitled to more weight than are the 1896 tests above; results are more uniform, but indicate a smoother pipe.	----	.637	.040	----	113.6	133.2	.0112	.020	.335	1.03
		B				.814	.062	----	116.6	134.4	.0114	.019	.326	1.01
		B				.898	.082	----	111.5	127.0	.0121	.021	.356	1.10
		B				1.02	.105	----	111.9	126.2	.0122	.021	.358	1.11
		B				1.11	.117	----	115.3	120.5	.0120	.019	.340	1.05
		B				1.15	.123	----	116.5	130.6	.0110	.019	.336	1.04
		B				1.22	.142	----	115.0	128.2	.0121	.019	.345	1.07
		B				1.24	.163	----	109.1	121.0	.0127	.022	.386	1.20
		B				1.25	.154	----	113.2	125.7	.0123	.020	.357	1.10
42	1895-1900	10	38.0	Rochester, N. Y. Conduit No. 2, reach about twice the length of Nos. 40 and 41 above; average values on same basis as No. 39b.	----	3.19	0.993	----	113.9	117.6	.0128	.020	.387	1.20
	1901-1905	5				3.19	1.048	----	111.2	114.4	.0130	.021	.408	1.26
	1906-1910	5				3.35	1.328	----	103.6	105.5	.0139	.024	.473	1.47
	1911-1915	5				3.24	1.331	----	99.7	101.6	.0143	.026	.508	1.58
	1916-1920	5				3.14	1.328	----	96.9	98.7	.0147	.028	.536	1.66
43	1895-1900	12	38.0	Rochester conduit No. 2, but on northern division while Nos. 40, 41, and 42 are on southern division; average values on same basis as No. 39b.	----	3.78	1.567	----	107.2	108.5	.0135	.022	.447	1.39
	1901-1905	5				3.63	1.519	----	104.7	106.2	.0138	.024	.465	1.44
	1906-1910	4				3.82	1.844	----	100.0	100.6	.0143	.026	.513	1.59
	1911-1915	5				3.60	1.647	----	99.6	100.7	.0144	.026	.514	1.59
	1916-1920	5				3.34	1.639	----	92.9	93.9	.0153	.030	.585	1.82
44	1912-1915	4	38.0	Rochester conduit No. 2. Rush Reservoir to Cobbs Hill Reservoir.	----	4.06	2.104	----	99.7	99.6	.0144	.026	.522	1.62
	1916-1920	4				3.80	2.065	----	93.9	94.0	.0152	.029	.580	1.80
45	IWS, 1896	B	42.0	Portland, Oreg. Main	36.37	3.78	1.215	----	115.9	117.1	.0128	.019	.386	1.20
45a	FMR	A	42.0	Same pipe as No. 45, 20 years later	34.36	3.57	1.16	----	112.1	113.2	.0132	.021	.411	1.27

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*—Continued

FULL-RIVETED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q= Quantity per second	V= Mean velocity per second	H= Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C= Chezy	C_w= Williams-Hazen	n= Kutter	f= Weisbach	Scobey: K_s= Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m= Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	*° C.*						
46	CH, 1896, new	C	42.0	East Jersey Water Co., New Jersey. Kearney extension; a few curves, taper joints, interior coating "unusually smooth"; Q by Venturi meter, loss of head by Bourdon-type gauges.	17.3	1.80	0.40	--------	96.2	101.5	0.0149	0.028	0.519	1.61
		C			17.4	1.81	.33	--------	106.5	113.3	.0136	.023	.424	1.32
		C			17.5	1.82	.53	--------	84.5	88.2	.0167	.036	.674	2.00
		C			19.55	2.04	.32	--------	121.9	130.0	.0120	.017	.328	1.02
		C			19.8	2.06	.60	--------	89.9	93.4	.0159	.032	.605	1.88
		C			20.7	3.09	.80	--------	116.8	119.0	.0126	.019	.372	1.15
		C			31.1	3.23	1.05	--------	106.6	108.2	.0137	.023	.449	1.39
		C			34.7	3.61	1.15	--------	113.8	115.1	.0130	.020	.397	1.23
		C			35.2	3.66	1.12	--------	116.9	118.4	.0127	.019	.377	1.18
		B			41.0	4.26	1.80	--------	107.3	106.7	.0137	.022	.455	1.41
47	CH, 1896, new	B	42.0	Same system as No. 46, conduit No. 2. Below Pompton Notch.	13.45	1.36	.19	--------	105.6	114.7	.0134	.023	.410	1.30
		B			16.8	1.75	.32	--------	104.6	111.4	.0138	.024	.438	1.36
48	CH, 1896, new	A	42.0	Another reach of same conduit as above; taper joints; Q by Venturi meter; loss of head by Bourdon-type gauges.	20.25	2.10	.47	--------	103.6	108.6	.0140	.024	.456	1.41
		B			21.3	2.21	.52	--------	103.6	108.2	.0140	.024	.457	1.42
		A			28.0	2.91	.85	--------	106.7	109.3	.0137	.023	.443	1.37
		B			29.0	3.02	.93	--------	105.8	107.8	.0138	.023	.451	1.40
		A			34.95	3.63	1.31	--------	107.2	107.9	.0137	.022	.448	1.30
		B			37.5	3.90	1.45	--------	109.5	109.8	.0135	.021	.433	1.34
		A			41.3	4.29	1.80	--------	108.1	107.4	.0136	.022	.449	1.39
		B			44.15	4.59	2.04	--------	108.6	107.0	.0136	.022	.447	1.39
		C			45.1	4.69	2.14	--------	108.4	107.0	.0136	.022	.452	1.40
		A			45.25	4.70	2.18	--------	107.6	106.1	.0137	.022	.458	1.42
		A			48.0	4.99	2.40	--------	108.9	107.0	.0136	.022	.449	1.39
49	EAB, 1918, 19 years.	B	44.6	Crystal Springs conduit, Los Angeles water supply. No. 49 covers whole reach included in parts 49a, 49b, 40c, and 49d; values erratic.	--------	2.545	.645	--------	104.7	107.7	.0140	.024	.456	1.415
49a		C				2.545	.468	--------	123.1	128.1	.0121	.017	.331	1.026
49b		C				2.545	.598	--------	108.5	112.2	.0135	.022	.423	1.312
49c		C				2.545	.713	--------	99.6	102.1	.0147	.026	.504	1.562
49d		C				2.545	.691	--------	101.1	103.8	.0144	.025	.488	1.514

50	JN, LeC, 1911	A	44.0	Spring Valley Water Co., San Andreas line. 14 years.	25.98	2.45	.547	------	109.4	113.5	.0135	.022	.415	1.28
51	JN, LeC, 1911	B	44.0	Spring Valley Water Co., Crystal Spring line. 27 years.	36.0	3.40	------	------	96.8	97.9	------	------	±.52	------
52	JN, LeC, 1911	A	44.0	do	37.0	3.50	1.681	------	89.1	88.5	.0162	.032	.648	2.01
53	CH, 1892, new	B	47.28	East Jersey Water Co., conduit No. 1. From station 309 to 575+87.	32.0	2.62	.65	------	103.5	105.6	.0143	.024	.471	1.46
53a	1893, 1 year	B	47.28	Same pipe as No. 53 but 1 year later. This reach below Pompton Notch.	31.4	2.57	.68	------	99.3	101.0	.0149	.026	.511	1.59
		B			31.7	2.59	.69	------	99.4	101.1	.0149	.026	.511	1.59
		B			34.3	2.81	.82	------	98.9	99.6	.0150	.026	.521	1.62
53b	1894, 2 years	B	47.28	Same reach as No. 53, 1 year later	25.05	2.05	.40	------	103.3	107.3	.0142	.024	.462	1.43
		B			37.15	3.05	.94	------	100.2	100.7	.0148	.026	.511	1.59
		B			43.6	3.57	1.16	------	105.6	105.2	.0142	.023	.468	1.45
53c	1896	A	47.28	Same reach as No. 53, 4 years old	36.4	2.98	.86	------	102.4	103.2	.0145	.025	.488	1.51
53d	1896, 4 years	A	------	Practically same reach as No. 53, from station 309 to 575+10.	53.7	4.40	2.03	------	98.4	95.8	.0151	.027	.549	1.70
		A			54.7	4.48	1.98	------	101.4	98.9	.0147	.025	.517	1.60
54	CH	A	47.34	From stations 309 to 923+55	66.6	5.45	2.47	------	110.4	106.7	.0137	.021	.444	1.38
54a	1896, 4 years	A		Same reach as No. 54, 4 years later	36.4	2.98	.80	------	106.1	107.3	.0141	.023	.454	1.41
		A			53.7	4.39	1.81	------	103.9	101.7	.0144	.024	.493	1.53
		A			54.7	4.47	1.91	------	103.0	100.6	.0145	.024	.502	1.56
54b	1896	A	------	Same line as No. 53 but stations 283 to 923+55	70.6	5.77	3.12	------	103.7	99.5	.0144	.024	.507	1.57
54c	1896, 4 years	A	47.4	Same line as No. 53 but stations from 283 to 1110.	64.6	4.46	1.90	------	102.9	100.5	.0145	.024	.503	1.56
[1] 55	CH, 1892, new	B	47.4	Same line as No. 53 but from stations 710+40 to 1052+96.	32.0	2.61	.51	------	116.3	119.6	.0120	.019	.373	1.16
		B			41.3	3.36	.94	------	110.3	110.7	.0136	.021	.426	1.32
		B			44.5	3.62	1.00	------	115.2	115.4	.0131	.019	.394	1.22
[1] 55a	1892, new	B	47.4	Includes No. 55 above	44.5	3.64	1.07	------	112.0	111.8	.0134	.021	.418	1.30
[1] 56	1894	A	47.4	Practically same as No. 54c	43.6	3.56	1.12	------	107.0	109.5	.0140	.022	.453	1.40
[1] 56a	1896, 4 years	A	47.4	Same conduit as No. 53 from stations 575+10 to 923+55; included in No. 56 above.	53.7	4.38	1.71	------	106.6	104.4	.0141	.023	.467	1.45
		A			54.7	4.46	1.83	------	104.9	102.5	.0143	.023	.485	1.60
		B			73.5	6.01	3.16	------	107.6	102.9	.0140	.022	.474	1.47
		A			74.0	6.04	3.15	------	108.3	103.6	.0139	.022	.468	1.45
		A			74.3	6.06	3.18	------	108.1	103.4	.0140	.022	.460	1.46
[1] 56b	CH, 1893, 1 year.	C	47.4	Same conduit as No. 53 from stations 710+40 to 923+55.	31.7	2.58	.45	------	122.4	120.5	.0123	.017	.337	1.04
		B			31.7	2.58	.56	------	109.7	112.4	.0136	.021	.419	1.30
		C			40.2	3.27	.91	------	109.1	109.6	.0137	.022	.434	1.34
[1] 56c	1894, 2 years	B	47.4	Same as No. 53 one-half year later	25.0	2.04	.33	------	113.0	118.3	.0131	.020	.386	1.20
[1] 57	1894, 2 years	A	47.4	Same line as No. 53 extended to 1117+00	43.6	3.55	1.08	------	108.7	108.5	.0138	.022	.440	1.36
[1] 57a	TM, 1907, 15.4 years.	D	47.56	Same system as No. 53 conduit No. 1, below Pompton Notch, stations 207+86 to 781+10, covers parts of Nos. 53 to 57 inclusive. (Note stations.)	------	1.177	.295	------	68.9	72.4	.0203	.054	.980	3.04
		B				1.356	.280	------	82.0	86.4	.0174	.038	.712	2.22
		B				2.306	.695	------	87.9	89.3	.0166	.033	.647	2.00
		A				2.477	.762	------	90.2	91.3	.0162	.032	.619	1.91
		A				3.917	1.020	------	89.8	87.6	.0164	.032	.652	2.02
		A				4.013	1.008	------	92.3	90.1	.0160	.030	.620	1.92
[1] 58	CH, 1892, new	B	47.52	Same conduit as No. 53, stations 11+50 to 257+80, above Pompton Notch.	63.2	5.13	2.07	------	113.3	110.2	.0116	.020	.421	1.31
		B			65.1	5.20	2.13	------	115.2	111.9	.0132	.019	.408	1.26
		B			66.1	5.37	2.25	------	113.8	110.3	.0133	.020	.419	1.30
		B			66.2	5.38	2.24	------	114.2	110.8	.0133	.020	.415	1.29
		B			66.6	5.41	2.28	------	113.9	110.3	.0133	.020	.419	1.30
[1] 58a	1896, 4 years	B	------	Same reach as No. 58 4 years later; unusual depreciation.	54.3	4.41	2.23	------	93.9	91.1	.0158	.029	.602	1.86
		A			54.9	4.46	2.24	------	94.7	91.8	.0158	.029	.593	1.84

[1] Not shown on Figure 3 because of congestion.

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*—Continued

FULL-RIVETED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation					
									C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	*° C.*						
[1] 58b	CH, 1896, 4 years.	A	-----	Same reach as No. 58 one-half year later	36.05	2.93	1.02	-----	92.2	92.2	0.0160	0.030	0.599	1.86
		A			36.65	2.98	1.03	-----	93.3	93.3	.0158	.030	.586	1.82
		A			57.0	4.63	2.05	-----	102.7	97.7	.0146	.024	.507	1.57
		A			57.0	4.63	2.34	-----	96.2	93.1	.0157	.028	.576	1.79
[1] 58c	TM, 1907	A	47.56	Same as No. 58, 11 years still later	-----	3.623	1.862	-----	84.4	82.4	.0173	.036	.737	2.28
[1] 59	CH, 1896, new	A	47.52	Same system as No. 53, conduit No. 2, parallel to conduit No. 1 above. Plates taper jointed, edges beveled; rivet beads well formed; pipe new; compare with No. 59a below, when 11 years old.	36.8	2.99	.89	-----	100.7	101.3	.0148	.025	.510	1.58
		A			37.1	3.01	.90	-----	100.8	101.4	.0147	.025	.509	1.58
		A			37.25	3.03	.89	-----	102.1	102.7	.0147	.025	.497	1.54
		A			48.0	3.90	1.40	-----	104.8	103.5	.0143	.023	.484	1.50
		A			56.7	4.61	1.99	-----	103.9	101.2	.0144	.024	.502	1.56
		B			57.7	4.69	1.96	-----	106.5	103.8	.0146	.023	.479	1.48
		B			57.95	4.71	2.04	-----	104.8	102.0	.0143	.023	.493	1.53
[1] 59a	TM, 1907, 11 years.	B	47.58	Same as No. 59 above, 11 years later; now blistered, tuberculated and pitted.	-----	3.382	1.377	-----	91.6	90.6	.0161	.031	.618	1.91
		B				3.558	1.739	-----	85.8	84.0	.0171	.035	.705	2.18
		B				3.826	1.798	-----	90.7	88.7	.0163	.031	.639	1.98
		B				4.324	2.550	-----	86.1	83.0	.0170	.035	.717	2.22
60	JBL, 1909	B	52.0	Siphon, Los Angeles, Calif., 5 years old	41.66	2.824	.502	-----	121.2	123.2	.0126	.018	.350	1.08
61	LJ, LeC, 1911	B	54.0	Spring Valley Water Co., San Francisco, Calif.	-----	-----	-----	-----	106.5	109.0	-----	-----	-----	-----
62	HFM, 1888, 3.4 years.	C	59.1	Monroe penstock No. 1, Essex Co., Lawrence, Mass. Tar-coated plate-iron pipe, covered with tubercles, filled in with mud and slime; *Q* by weir, *H* by piezometer tubes to gauge glass.	-----	1.125	.098	-----	102.4	109.4	.0144	.025	.453	1.41
		C				1.398	.160	-----	99.6	104.3	.0150	.026	.489	1.51
		C				1.712	.242	-----	99.2	102.1	.0152	.026	.503	1.56
		B				1.956	.327	-----	97.5	99.2	.0156	.027	.528	1.63
		B				2.197	.442	-----	94.2	94.7	.0162	.029	.571	1.76
		B				2.231	.441	-----	95.8	96.2	.0159	.028	.555	1.72
		B				2.305	.460	-----	96.9	97.3	.0158	.027	.543	1.68
		B				2.384	.511	-----	95.1	95.3	.0160	.029	.567	1.76
		B				2.490	.552	-----	95.5	95.2	.0160	.028	.563	1.74
		B				2.578	.602	-----	94.7	94.1	.0162	.029	.576	1.78

63	FCS, 1924	B	62.0	Siphon, Gage Canal, Calif. New[1]		1.93	.212		116.6	120.0	.0133	.019	.370	1.15
64	JWS, 1903, 0.8 year.	B	72.0	Jersey City Water Co., New Jersey. Plate-steel pipe, taper joints; plates from 5/16 to 11/16 inch thick; pipe dipped horizontally in gilsonite; line includes 10 gates each 48 inches in diameter with reducing and expanding sections.		1.14	.061	18.4	119.2	126.4	.0128	.018	.342	1.07
		B				1.37	.098		113.0	117.6	.0137	.020	.387	1.21
		B				1.52	.131		108.5	111.6	.0144	.022	.426	1.33
		B				1.83	.186		109.7	111.3	.0144	.021	.424	1.32
		B				2.09	.252		107.5	107.8	.0147	.022	.447	1.40
		B				2.60	.382		108.6	107.1	.0147	.022	.447	1.40
		B				2.69	.396		110.4	108.7	.0145	.021	.434	1.36
		B				2.69	.417		107.6	105.7	.0148	.022	.457	1.43
		B				3.29	.610		108.8	105.2	.0147	.022	.456	1.42
		B				3.30	.615		108.5	105.1	.0148	.022	.457	1.43
64a	JWS, 1904, 2 years.	B	72.0	Same pipe as No. 64, 14 months later. Mills estimates the loss of head when pipe was new to be about normal but that loss in this 14 months was 9 per cent above normal. (See 64b; compare with 69a.)		1.37	.127	13.0	99.3	102.3	.0156	.026	.501	1.54
		A				2.12	.303		99.4	99.0	.0159	.026	.523	1.61
		A				2.15	.307		100.2	99.7	.0158	.026	.515	1.58
		A				2.40	.390		99.2	97.8	.0160	.026	.531	1.64
		A				2.42	.394		99.5	98.0	.0159	.026	.528	1.63
		A				2.74	.497		100.4	97.9	.0158	.026	.525	1.62
		A				2.91	.568		99.7	96.8	.0160	.026	.536	1.65
		A				2.94	.575		100.1	97.1	.0159	.026	.532	1.64
64b	JWS, 1905, 2.95 years.	A	72.0	Same pipe as No. 64, 1 year later. Mills estimates loss since above tests to be 12 per cent above normal.		1.52	.178	17.0	93.0	94.6	.0167	.030	.578	1.80
		A				2.02	.297		95.7	95.3	.0164	.028	.561	1.75
		A				2.37	.413		95.2	93.6	.0166	.028	.576	1.80
		A				2.69	.528		95.6	93.0	.0166	.028	.579	1.80
		A				3.10	.682		96.9	93.4	.0164	.027	.571	1.78
64c	JWS, 1905, 5.3 years.	B	72.0	Same pipe as No. 64		1.4	.20		80.8	81.8	.0190	.039	.757	2.34
		B				2.15	.42		85.7	84.1	.0184	.035	.705	2.18
		B				2.44	.52		87.4	85.1	.0180	.034	.686	2.12
		B				2.75	.69		85.5	82.3	.0184	.035	.724	2.24
64d	JWS, 1908, 7.2	B	72.0	do		1.85	.27		91.9	91.9	.0170	.030	.602	1.86
		C				2.045	.39		84.5	83.3	.0185	.036	.720	2.23
		B				2.5	.50		91.3	89.0	.0173	.031	.630	1.95
		B				2.9	.655		92.5	89.3	.0171	.030	.622	1.93
64e	JWS, 1909, 9 years.	B	72.0	do		1.55	.21		87.3	88.2	.0178	.034	.656	2.03
		B				2.05	.36		88.2	87.2	.0178	.033	.661	2.05
		B				2.45	.51		88.6	86.2	.0178	.033	.667	2.07
		B				2.80	.67		88.3	85.1	.0178	.033	.680	2.11
64f	JWS, 1911, 13 years.	B	72.0	do		2.1	.386		87.3	86.0	.0179	.034	.678	2.10
		B				2.3	.455		88.0	86.2	.0178	.033	.671	2.08
		B				2.4	.523		85.7	83.4	.0183	.035	.713	2.21
		B				2.6	.585		87.8	85.1	.0179	.034	.684	2.12
		B				2.75	.650		88.1	85.0	.0179	.033	.682	2.11
		B				2.8	.700		86.4	83.1	.0182	.034	.711	2.20
		B				3.0	.780		87.7	84.0	.0180	.033	.695	2.15

[1] Not shown on Figure 3 because of congestion.

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*—Continued

FULL-RIVETED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=mv^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	*° C.*						
65	RAM, 1916, new.	D	72.0	Penstock, Halsey power house, Pacific Gas & Electric Co., California. Of riveted steel plates, ⅜ inch thick, butt jointed, triple riveted, double coated with graphite paint, brushed on; this series taken when pipe was new; Q by Venturi, H by differential gauge.	65.0	2.30	.186	--------	137.7	139.7	0.0117	0.014	0.274	0.85
		D			70.0	2.48	.288	--------	119.3	119.0	.0134	.018	.368	1.14
		D			86.0	3.04	.411	--------	122.4	120.3	.0132	.017	.357	1.11
		D			104.0	3.68	.598	--------	122.9	119.0	.0132	.017	.361	1.12
		D			130.0	4.88	1.109	--------	119.6	113.0	.0135	.018	.392	1.22
		B			176.5	6.24	1.928	--------	116.0	107.2	.0139	.019	.427	1.32
		B			214.0	7.57	2.789	--------	117.0	106.6	.0138	.019	.428	1.33
		B			257.5	9.11	4.016	--------	117.4	105.3	.0138	.019	.434	1.34
		B			290.0	10.26	5.163	--------	116.6	103.6	.0139	.019	.444	1.38
		B			313.0	11.07	5.941	--------	117.3	103.6	.0138	.019	.443	1.38
		B			332.0	11.74	6.534	--------	118.6	104.4	.0137	.018	.435	1.35
66	RAM, 1916, new.	D	72.0	Penstock, Halsey power house, another portion of same pipe as No. 65 above. Of plates ¼ to ⅜ inch thick, lap riveted with double row of rivets; tested same time as No. 65; same type apparatus.	65.0	2.30	.159	--------	148.9	152.1	.0108	.012	.234	.72
		D			70.0	2.48	.227	--------	134.4	135.3	.0120	.014	.290	.90
		D			86.0	3.04	.339	--------	134.8	133.5	.0120	.014	.294	.91
		D			104.0	3.68	.524	--------	131.3	127.8	.0124	.015	.316	.98
		D			138.0	4.88	.972	--------	127.8	121.4	.0127	.016	.344	1.07
		A			176.5	6.24	1.668	--------	124.8	115.9	.0130	.017	.370	1.15
		A			214.0	7.57	2.417	--------	125.7	115.1	.0130	.016	.371	1.15
		A			257.5	9.11	3.461	--------	126.4	114.1	.0129	.016	.374	1.16
		A			290.0	10.26	4.427	--------	125.9	112.5	.0128	.016	.380	1.18
		A			313.0	11.07	5.067	--------	127.0	112.9	.0129	.016	.376	1.16
		A			332.0	11.74	5.561	--------	128.6	113.9	.0127	.016	.373	1.16
67	RAM, 1913, new.	C	72.0	Penstock, Drum power house, Pacific Gas & Electric Co., California. Butt joints.	59.9	2.12	.270	--------	105.3	105.3	.0150	.023	.466	1.44
		B			70.8	2.50	.322	--------	113.8	112.9	.0140	.020	.406	1.26
		B			86.0	3.04	.463	--------	115.4	112.9	.0139	.019	.402	1.24

68	RAM, 1913, new.	B	72.0	Same penstock as No. 67, lap joints, double riveted.	70.8	2.50	.254	--------	128.1	128.4	.0125	.016	.320	.99
		B			86.0	3.04	.342	--------	134.2	132.9	.0121	.014	.297	.92
69	M-W-H, 1897, new.	D	72.22	Pioneer power plant, Odgen, Utah. Riveted steel penstock, butt joints with inside straps longitudinally only; value of some runs discounted because of leaking mercury manometers; pipe straight in plan; 12 vertical bends on 30-foot radius; plates from ⅜ to ¹¹⁄₁₆ inch in thickness by ¹⁄₁₆ inch increments; dipped in asphalt after shop riveting; loss of head by mercury manometers; *Q* by Venturi meter. Compare with 64.	16.0	.562	.0156	--------	116.0	129.0	.0122	.019	.345	1.07
		D			30.3	1.065	.0768	--------	99.1	104.1	.0154	.026	.401	1.52
		D			30.9	1.086	.0676	--------	107.7	113.7	.0142	.022	.417	1.29
		D			31.7	1.114	.056	--------	121.3	129.1	.0126	.018	.328	1.03
		D			44.8	1.575	.126	--------	114.4	117.8	.0137	.020	.382	1.18
		D			45.5	1.599	.109	--------	124.8	129.3	.0126	.016	.322	1.00
		D			45.7	1.606	.134	--------	113.1	116.2	.0139	.020	.391	1.21
		D			47.5	1.670	.169	--------	104.7	106.6	.0150	.024	.459	1.42
		D			47.5	1.670	.160	--------	107.6	109.8	.0146	.022	.436	1.35
		D			52.2	1.835	.187	--------	109.4	110.9	.0144	.022	.426	1.32
		C			60.9	2.141	.240	--------	112.7	113.1	.0141	.020	.408	1.26
		C			65.6	2.306	.353	--------	100.0	98.9	.0158	.026	.520	1.61
		C			65.8	2.313	.363	--------	99.0	97.7	.0160	.026	.532	1.65
		B			69.6	2.447	.340	--------	108.2	107.1	.0147	.022	.446	1.38
		B			69.9	2.457	.352	--------	106.8	105.5	.0149	.023	.458	1.42
		B			71.2	2.503	.359	--------	107.7	106.4	.0148	.022	.454	1.41
		B			73.8	2.594	.372	--------	109.6	108.2	.1046	.021	.438	1.36
		C			77.8	2.735	.477	--------	102.1	99.7	.0156	.025	.506	1.57
		B			87.1	3.062	.495	--------	112.2	109.4	.0143	.020	.425	1.32
		B			90.3	3.174	.582	--------	107.2	103.9	.0151	.022	.468	1.45
		B			103.9	3.652	.732	--------	110.0	105.6	.0146	.021	.451	1.40
		B			104.9	3.687	.732	--------	111.1	106.7	.0145	.021	.439	1.36
		B			105.3	3.701	.753	--------	109.9	105.4	.0147	.021	.452	1.40
		B			109.2	3.839	.769	--------	112.8	108.1	.0143	.020	.429	1.33
		B			109.4	3.846	.776	--------	112.5	107.8	.0143	.020	.433	1.34
69a	M-W-H, 1899, 2 years.	D	72.24	Same pipe as No. 69, 2 years later, tests presumably of more weight than for No. 69 above. Compare with 64a.	--------	.689	.059	--------	73.1	77.6	.0198	.048	.862	2.67
		D				1.120	.121	--------	83.1	85.6	.0182	.037	.702	2.18
		B				2.043	.286	--------	95.5	98.2	.0169	.027	.531	1.65
		B				2.577	.423	--------	102.1	100.3	.0156	.025	.504	1.56
		B				3.157	.645	--------	101.3	97.8	.0157	.025	.521	1.62
		B				4.339	1.117	--------	105.8	99.9	.0153	.023	.523	1.63
		B				4.375	1.258	--------	100.8	94.5	.0158	.025	.546	1.69
		B				4.713	1.343	--------	104.8	98.2	.0153	.023	.509	1.58
		B				4.779	1.370	--------	104.9	98.2	.0153	.023	.509	1.58
		B				5.320	1.696	--------	105.3	97.8	.0153	.023	.509	1.58
70	HFM, 1903, 22 years.	B	77.5	Munroe penstock No. 2, Essex Co., Lawrence, Mass. Of steel plates ⅜ inch thick, lap riveted in cylinder joints, asphalt coated; *Q* by weir, *H* by flush piezometer tubes attached to glass gauge tubes.	--------	1.215	.097	--------	97.0	100.0	.0160	.027	.522	1.62
		B				1.554	.163	--------	95.8	96.8	.0165	.028	.548	1.70
		B				1.720	.197	--------	98.5	96.7	.0164	.028	.548	1.70
		B				2.225	.347	--------	94.0	92.2	.0170	.029	.590	1.83
		B				2.500	.442	--------	93.6	90.9	.0172	.029	.603	1.87
		B				2.596	.461	--------	95.2	92.2	.0168	.028	.589	1.82
		B				2.771	.532	--------	94.6	91.1	.0170	.029	.598	1.85
		B				2.964	.617	--------	93.9	90.0	.0171	.029	.610	1.89
		B				3.300	.727	--------	96.3	91.7	.0167	.028	.585	1.81
71	HLD, 1923, 10 years.	B	84.0	Flow line, Big Creek No. 1, Cascada, Calif.; Southern California Edison Co.	129.5	3.37	0.738	--------	93.8	88.2	.0174	.029	.525	1.63
		B			242.0	6.29	2.070	--------	104.5	94.4	.0158	.024	.535	1.66
		B			355.5	9.24	4.176	--------	108.1	94.9	.0153	.022	.520	1.61
72	RAM, 1917, new.	C	84.0	Penstock, Wise power house, Pacific Gas & Electric Co. Lap joint, double rivets. (See 73, 74, 75 below.)	130.0	3.38	.491	--------	115.3	110.3	.0143	.019	.413	1.28
		B			175.0	4.55	.914	--------	113.8	106.2	.0145	.020	.437	1.35
		A			245.0	6.37	1.758	--------	114.8	104.4	.0144	.020	.444	1.38
		A			320.0	8.32	2.028	--------	116.2	103.5	.0142	.019	.445	1.38

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TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*—Continued

FULL-RIVETED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	° *C.*						
73	RAM, 1917, new.	C	84.0	Same penstock, as No. 72; lap joints; single rivets. (See 72, 74, 75.)	130.0	3.38	0.442	-------	121.5	116.8	0.0130	0.017	0.372	1.15
		B			175.0	4.55	.801	-------	121.5	114.0	.0137	.017	.383	1.19
		A			245.0	6.37	1.577	-------	121.3	110.7	.0137	.018	.398	1.23
		A			320.0	8.32	2.609	-------	123.1	110.2	.0135	.017	.396	1.23
74	RAM, 1917, new.	C	84.0	Same penstock as No. 72, butt strap riveted. (See 72, 73, 75.)	130.0	3.38	.448	-------	120.7	115.9	.0137	.018	.377	1.17
		B			175.0	4.55	.870	-------	116.2	108.6	.0142	.019	.419	1.30
		A			245.0	6.37	1.721	-------	116.1	105.6	.0143	.019	.435	1.35
		A			320.0	8.32	2.872	-------	117.4	104.6	.0141	.019	.436	1.35
75	RAM, 1917, new.	B	84.0	Combined reaches 72, 73, 74. (See 76 below for later tests on this penstock.)	130.0	3.38	.459	-------	119.3	114.4	.0139	.018	.386	1.20
		A			175.0	4.55	.865	-------	116.9	109.4	.0141	.019	.414	1.28
		A			245.0	6.37	1.687	-------	117.2	106.8	.0142	.019	.426	1.32
		A			320.0	8.32	2.810	-------	118.6	105.9	.0140	.018	.427	1.32
76	FCS, 1919, 2 years.	B	84.0	Wise penstock. Last end of reach includes No. 72 and upper end of No. 73 above, all lap riveted except 786 feet of butt strap pipe; capacity depreciating.	69.0	1.79	.182	23.0	100.3	99.8	.0161	.026	.512	1.59
		B			101.5	2.64	.395	25.0	100.4	96.9	.0163	.026	.531	1.65
		A			159.0	4.13	.955	20.8	101.0	94.1	.0162	.025	.548	1.70
		A			203.0	5.28	1.614	21.4	99.4	90.6	.0165	.026	.582	1.80
		A			203.0	5.28	1.580	21.3	100.4	91.7	.0163	.026	.569	1.76
		A			311.0	8.08	3.657	22.0	102.3	90.4	.0161	.025	.587	1.82
77	CH, 1887, 5 years.	C	103.4	Holyoke, Mass., trunk line. Wrought-iron plates, cylinder jointed; 5 years old; rusty inside, no tubercles, reach very short for such a large pipe; original coat all or nearly all gone.	29.08	0.5	.008	-------	121.8	134.1	.0126	.017	.319	.99
		C			58.16	1.0	.032	-------	120.4	125.1	.0135	.018	.342	1.06
		C			87.22	1.5	.082	-------	112.7	112.7	.0147	.020	.406	1.26
		C			116.25	2.0	.153	-------	110.1	107.4	.0152	.021	.439	1.36
		C			145.4	2.5	.242	-------	109.7	105.1	.0154	.021	.454	1.41
		C			174.4	3.0	.352	-------	109.0	102.8	.0155	.022	.467	1.45
		C			203.5	3.5	.490	-------	107.7	100.3	.0157	.022	.485	1.50
		C			232.6	4.0	.652	-------	106.7	98.3	.0159	.023	.500	1.55
		C			261.7	4.5	.835	-------	106.1	96.7	.0160	.023	.514	1.59
78	DWP, 1925, 0.8 years.	B	108.0	Oak Grove No. 3 penstock, Portland Electric Power Co., Portland, Oreg. Cylinder-jointed riveted-steel pipe, shop painted with	126.0	1.99	.154	-------	106.9	103.7	.0158	.022	.467	1.45
		B			167.0	2.62	.227	-------	115.9	110.7	.0147	.019	.408	1.26
		A			210.0	3.30	.326	-------	121.8	114.7	.0140	.017	.378	1.17

		A		red lead, field coated with graphite; quantity by multiple Pitot tube (Proebstel) method.	242.0	3.80	.413	....	124.7	116.2	.0138	.017	.366	1.13
		A			275.0	4.33	.543	....	123.9	114.2	.0139	.017	.376	1.16
		B			305.0	4.79	.599	....	130.5	119.9	.0132	.015	.343	1.06
		A			330.0	5.19	.759	....	125.6	114.3	.0137	.016	.372	1.15
		A			354.0	5.55	.866	....	125.7	113.8	.0137	.016	.375	1.16
		A			395.0	6.22	1.046	....	128.2	115.2	.0134	.016	.364	1.13
		A			425.0	6.67	1.190	....	128.9	115.2	.0134	.016	.363	1.12
		A			443.0	6.97	1.307	....	128.5	114.4	.0134	.016	.366	1.13
79	RW, 1922, new.	B	129.0	Pit No. 1, penstock, California. Triple-riveted butt joint.	714.0	7.86	1.212	....	137.7	120.2	.0128	.014	.328	1.02
		A			982.0	10.81	2.208	....	140.3	119.3	.0126	.013	.326	1.01
80	FCS, 1919, 7 years.	A	132.0	Deadman siphon, Los Angeles Aqueduct. Cylinder joints; plates ¼ to ½ inch thick, coated with asphalt paint, brushed on.	....	3.22	.310	22.0	110.3	101.3	.0160	.021	.470	1.48
		A			....	3.245	.313	24.0	110.6	101.6	.0159	.021	.468	1.47
		A			....	3.26	.306	25.0	112.4	103.1	.0157	.020	.454	1.43
		A			....	3.41	.333	25.0	112.7	103.2	.0157	.020	.455	1.44
		A			....	3.46	.343	24.0	112.7	103.1	.0156	.020	.454	1.43
81	FCS, 1917, 3 years.	B	168.0	Flow line, Ohio State Power Co., Fremont, Ohio.	....	3.665	.369	16.0	101.9	90.1	.0179	.025	.569	1.77
				GIRTH-RIVETED PIPE OR SCREW-JOINT WELDED PIPE										
202	JRF, 1892, old.	B	4.04	Old wrought-iron pipe, Nashua, N. H. Reach 51.5 feet between piezometers; interior described as "hardly square inch without bunches 1/16 inch high, of rough pebbly nodules of rust; numerous bunches 3/16 inch with about ⅛ inch between summit and valley."	....	0.954	2.45	20.0	66.6	88.6	0.0125	0.058	0.810	2.54
		B				1.878	9.38	20.0	66.8	84.3	.0125	.058	.853	2.68
		B				2.871	21.67	20.0	67.2	82.0	.0125	.057	.883	2.77
		B				4.628	56.15	20.0	67.3	79.1	.0124	.057	.922	2.89
		B				6.932	126.01	20.0	67.3	76.5	.0124	.057	.960	3.02
		B				9.619	241.15	20.0	67.5	74.8	.0124	.057	.988	3.10
		B				13.47	472.48	20.0	67.5	72.8	.0124	.056	1.019	3.20
		B				16.84	740.5	20.0	67.4	71.4	.0124	.057	1.045	3.28
		B				19.37	974.9	20.0	67.6	70.8	.0124	.056	1.055	3.31
		B				20.69	1126.0	20.0	67.2	70.0	.0124	.057	1.075	3.37
		B				22.92	1378.0	20.0	67.3	69.5	.0124	.057	1.084	3.40
204	JRF, 1892, new.	B	4.12	Experimental pipe, Nashua, N. H. New, straight, wrought-iron pipe, lap welded with screw joints; considered representative of ordinary new lap-welded 1892 pipes; tests at velocities below 1 foot per second excluded. Mr. Freeman describes inside as examined by a mirror on a long rod: "Almost every foot of interior contains ridges and bunches 1/100 to 2/100 inch high with rounded gently sloping edges and all of surface is a little scabby."	....	1.302	1.84	21.0	103.5	138.9	.0091	.024	.344	1.08
		B				1.653	2.74	21.0	107.8	142.5	.0089	.022	.325	1.02
		C				1.847	3.84	21.0	101.7	131.4	.0093	.025	.368	1.16
		C				1.854	3.86	21.0	101.9	132.8	.0093	.025	.368	1.16
		B				1.985	3.94	21.0	107.9	140.5	.0089	.022	.330	1.04
		B				2.644	6.59	21.0	111.2	141.9	.0087	.021	.321	1.01
		B				3.123	9.41	21.0	109.9	138.2	.0088	.021	.333	1.05
		B				3.607	11.80	21.0	113.4	141.4	.0086	.020	.316	1.00
		B				4.330	16.87	21.0	113.0	138.9	.0086	.020	.321	1.01
		B				4.750	19.84	21.0	115.1	140.5	.0085	.019	.317	1.00
		C				5.431	23.78	21.0	120.2	145.7	.0082	.018	.295	.93
		B				5.462	25.97	21.0	115.7	139.7	.0085	.019	.318	1.00
		B				7.796	51.48	21.0	117.3	137.8	.0084	.019	.321	1.01
		B				9.415	74.43	21.0	117.8	136.4	.0084	.019	.324	1.02
		B				11.69	111.47	21.0	119.5	136.2	.0083	.018	.321	1.01
		B				13.93	154.34	21.0	121.0	136.2	.0082	.018	.320	1.00
		B				17.12	229.37	21.0	122.0	135.1	.0082	.017	.321	1.01
		B				19.15	285.68	21.0	122.2	134.1	.0082	.017	.323	1.02
		B				19.43	298.69	21.0	121.4	133.0	.0082	.018	.328	1.03
		B				22.57	390.49	21.0	123.3	133.6	.0081	.017	.323	1.02
		B				23.41	417.73	21.0	123.6	133.6	.0081	.017	.322	1.01

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*—Continued

GIRTH-RIVETED PIPE OR SCREW-JOINT WELDED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation					
									C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\mu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	° *C.*						
206	JRF, 1892, new	B	5.12	Experimental pipe, Nashua, N. H. New, straight, wrought-iron pipe, lap welded with screw joints. (See No. 204 above for description.)	----	1.925	2.77	21.0	112.0	143.8	0.0090	0.021	0.312	0.98
		B				2.066	3.26	21.0	110.8	141.3	.0091	.021	.322	1.01
		B				3.929	10.86	21.0	115.4	140.5	.0089	.019	.315	.99
		B				7.714	39.51	21.0	118.8	137.2	.0087	.018	.318	1.00
		B				9.996	64.83	21.0	120.2	136.1	.0086	.018	.319	1.00
		C				10.63	86.65	21.0	110.6	123.7	.0092	.021	.381	1.20
		B				13.36	114.25	21.0	121.1	134.0	.0086	.018	.324	1.02
		B				14.53	133.67	21.0	121.7	133.8	.0085	.017	.324	1.02
		B				15.99	162.08	21.0	121.6	132.7	.0085	.017	.327	1.03
		B				17.86	199.50	21.0	122.4	132.5	.0085	.017	.326	1.03
		B				20.47	261.66	21.0	122.5	131.1	.0085	.017	.330	1.03
		B				20.59	261.89	21.0	123.1	131.8	.0084	.017	.327	1.03
208	JRF, 1892, new	B	6.14	do	----	.999	.73	23.0	103.4	136.9	.0097	.024	.349	1.10
		B				1.954	2.47	23.0	110.0	138.5	.0094	.021	.331	1.05
		B				2.446	3.64	23.0	113.4	140.7	.0092	.020	.318	1.01
		B				2.888	4.95	23.0	114.8	140.7	.0092	.020	.315	1.00
		B				4.805	12.85	23.0	118.5	139.8	.0089	.018	.311	.98
		B				5.696	16.62	23.0	123.5	144.2	.0087	.017	.291	.92
		B				7.391	29.00	23.0	121.3	140.2	.0088	.018	.310	.98
		B				9.529	48.13	23.0	121.4	135.9	.0088	.017	.318	1.00
		B				11.25	64.57	23.0	123.8	136.9	.0087	.017	.311	.98
		B				11.93	71.27	23.0	125.0	137.3	.0086	.016	.307	.97
		B				15.61	123.37	23.0	124.8	134.5	.0087	.017	.317	1.00
		B				16.90	143.34	----	124.8	133.7	.0087	.017	.319	1.01

210	JRF, 1892, new.	B	8.05	do.		1.479	1.047	21.0	111.6	140.5	.0098	.021	.324	1.02
		B				1.086	1.819	21.0	113.7	140.0	.0097	.020	.317	1.00
		B				3.087	4.17	21.0	116.7	139.1	.0095	.019	.317	1.00
		B				4.002	6.83	21.0	118.2	138.1	.0095	.018	.316	.99
		B				5.436	11.81	21.0	122.1	139.6	.0092	.017	.305	.96
		B				5.812	14.23	21.0	119.0	134.9	.0094	.018	.324	1.02
		B				5.872	14.60	21.0	118.7	134.5	.0094	.018	.326	1.02
		B				6.208	15.85	21.0	120.4	136.0	.0094	.018	.318	1.00
		B				6.768	18.86	21.0	120.3	135.0	.0093	.018	.321	1.01
		B				7.160	21.00	21.0	120.6	134.7	.0094	.018	.322	1.01
		B				7.474	22.75	21.0	121.0	134.7	.0093	.018	.321	1.01
		B				7.687	24.51	21.0	119.9	133.2	.0094	.018	.328	1.03
		B				8.158	25.43	21.0	124.9	138.4	.0091	.017	.304	.95
		B				8.254	27.15	21.0	122.4	135.3	.0092	.017	.317	1.00
		B				9.103	32.45	21.0	123.4	135.4	.0092	.017	.315	.99
		B				9.281	34.99	21.0	121.2	132.5	.0093	.018	.327	1.03
		B				10.37	42.46	21.0	122.9	133.4	.0092	.017	.322	1.01
		B				11.71	53.03	21.0	124.2	133.6	.0091	.017	.319	1.00
		B				12.31	57.71	21.0	125.1	134.2	.0091	.016	.316	.99
		B				12.95	60.33	21.0	122.8	131.0	.0092	.017	.330	1.04
		B				13.02	66.80	21.0	123.0	131.2	.0092	.017	.328	1.03
211	GHB, 1919, new.	C	8.94	Toconce line, Chile, South America. Welded rolled steel "line pipe," screw couplings; quantity measured by Venturi and weir; loss of head by pressure gauges.	----	2.03	2.32	----	97.6	117.4	.0111	.027	.438	1.36
		C				2.13	2.10	----	107.7	130.1	.0103	.022	.362	1.12
		C				2.67	3.34	----	107.0	126.9	.0104	.022	.374	1.16
		B				3.45	4.36	----	121.1	142.0	.0095	.018	.300	.93
		B				3.64	4.74	----	122.5	143.2	.0094	.017	.296	.92
		B				4.36	6.77	----	122.8	141.5	.0094	.017	.299	.93
212	WLDuM, 1915	C	10.0	Morenci, Ariz., 6 years old	----	4.08	6.82	----	108.2	122.9	.0106	.022	.387	1.20
214	WJR, 1910	B	15.5	Glenwood Springs, Colo., 22 years old	----	11.38	19.68	----	142.7	146.8	.0092	.013	.256	.79
216	FCS, 1917, 4 years.	B	36.0	Rattlesnake siphon, Spring Brook Water Supply Co., Wilkes-Barre, Pa. An inverted siphon pipe across a deep ravine, between sections of open channel; excessive loss of head caused by slimy algæ growth from ⅛ to ¼ inch thick.	----	1.221	.224	----	94.4	103.8	.0144	.029	.514	1.59
		B				1.885	.535	----	94.2	100.2	.0148	.029	.535	1.65
		B				2.206	.724	----	94.8	99.5	.0148	.029	.539	1.67
		B				2.694	1.301	----	86.2	88.6	.0161	.035	.603	2.06
		A				2.943	1.356	----	92.3	94.6	.0152	.030	.584	1.81
		A				3.706	2.184	----	93.7	94.4	.0150	.029	.579	1.80
		A				3.978	2.585	----	90.5	90.2	.0155	.032	.627	1.94
		A				4.570	3.078	----	95.1	94.4	.0148	.028	.573	1.78
216a	JHL, 1917, 4 years.	B	36.0	Same pipe as No. 216; tests conducted with company apparatus, results agree within reason.	----	1,221	.228	----	93.5	102.8	.0140	.030	.523	1.62
		B				1.885	.521	----	95.5	101.6	.0146	.028	.521	1.62
		B				2.206	.770	----	91.8	96.3	.0152	.031	.573	1.78
		B				2.694	1.286	----	86.7	89.1	.0160	.034	.656	2.03
		A				2.943	1.374	----	91.7	94.0	.0153	.031	.591	1.83
		A				3.978	2.609	----	89.9	89.8	.0156	.032	.633	1.96
		A				4.570	3.051	----	95.5	94.8	.0148	.028	.568	1.76
218	1912, 4 years	C	36.0	Montreal Water & Power Co. (See description this pipe on p. 114.)	21.21	2.99	.680	----	133.0	139.3	.0110	.015	.284	.88
		C			26.31	3.72	1.02	----	133.0	139.3	.0110	.015	.284	.88
220	1918	B	37.0	Conduit No. 3, Rochester, N. Y. New pipe	28.64	3.84	1.448	----	114.8	117.1	.0126	.020	.387	1.20
220a	1919, 1 year	B	37.0	Same pipe as No. 220, year later	28.06	3.76	1.452	----	112.4	114.5	.0129	.020	.404	1.25
220b	1920, 2 years	B	37.0	Same pipe as No. 220, 1 year still later	28.60	3.83	1.432	----	115.3	117.6	.0126	.019	.386	1.20
220c	1926, 8 years	B	37.0	Same pipe as No. 220, 6 years still later	26.34	3.53	1.428	14.4	106.3	108.5	.0135	.023	.449	1.39
222	1920	B	37.0	Rochester No. 3. From Rush Reservoir to Mount Hope Reservoir.	30.33	4.06	1.049	4.0	104.8	105.5	.0137	.023	.470	1.45
222a	1926	B	37.0		24.22	3.24	1.409	4.0	98.4	100.4	.0144	.027	.520	1.56

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*—Continued

GIRTH-RIVETED PIPE OR SCREW-JOINT WELDED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	° *C.*						
224	JBP, 1926, 17 years.	B	42.0	Springfield, Mass	39.65	4.12	1.447	1.0	115.8	116.1	0.0128	0.019	0.391	1.16
		B		Same line, 11 days later	39.87	4.14	1.452	1.0	116.2	116.4	.0128	.019	.387	1.15
225	FMR, 1922, 11 years.	A	44.0	Bull Run conduit No. 2, Portland, Oreg	78.51	7.43	3.090		139.7	135.0	.0110	.013	.295	.88
225a		A	52.0		78.51	5.32	1.472		133.3	130.1	.0117	.014	.309	.96
226	FMR	B	58.0	Bull Run conduit No. 3	114.5	6.24	1.502		146.6	140.6	.0100	.012	.262	.81
226a	1925	B	58.0	Another reach of same pipe as No. 226	114.5	6.24	1.676		138.5	132.5	.0115	.013	.293	.91
226b	1925	B	58.0	2.1 per cent butt-strap pipe	114.5	6.24	1.593		142.4	136.2	.0112	.013	.278	.86
226c	1925	A	58.0	Above 3 reaches combined	114.5	6.24	1.618		141.1	135.1	.0113	.013	.283	.88
[1] 227a	FCS, 1929, new.	A	65.0	Mokelumne Aqueduct, East Bay municipal utility district, California.	88.97	3.86	.636	24.	131.8	128.8	.0122	.015	.314	.99
[1] 227b		A	65.0		88.97	3.86	.567	24.	139.3	137.0	.0115	.013	.279	.88
[1] 227c		A	65.0		88.97	3.86	.580	24.	137.7	135.3	.0117	.014	.286	.91
228	RW, 1922, new	B	96.0	Penstock, Pit No. 1 California. Lap-welded, bump-joint pipe.	714.0	14.20	5.67		133.3	113.7	.0128	.014	.361	1.12
		B			982.0	19.54	10.18		136.9	114.0	.0124	.014	.354	1.10
230	RW, 1922	B	108.0	Same penstock as No. 228, lap-welded, bump-joint pipe; new.	714.0	11.22	3.264		130.9	112.4	.0131	.015	.370	1.15
		B			982.0	15.45	5.761		135.7	113.9	.0127	.014	.356	1.10

CONTINUOUS-INTERIOR PIPE [2]

302	FWF, JJW, 1917, 1 year.	D	3.628	Experimental pipe, Versailles, Pa. Straight lap-welded wrought steel pipe, fitted with Matheson joints between sections averaging 17 feet long; coated inside with coal tar; inside diameter determined by weighing water content, low velocities determined by volumetric tests, higher velocities by checked Venturi meter; loss of head by mercury	0.182	2.55	4.90	31.1	132.5	173.9	0.0075	0.015	0.222	0.714
		B			.202	2.83	8.37	32.8	112.5	144.7	.0084	.021	.311	1.004
		B			.223	3.11	9.97	25.6	113.3	144.5	.0084	.020	.310	.985
		B			.269	3.75	13.88	33.3	115.8	145.8	.0083	.019	.303	.978
		B			.293	4.08	14.90	28.3	121.6	152.6	.0080	.017	.279	.891
		B			.361	5.03	24.00	33.3	118.1	145.5	.0081	.018	.299	.965
		A			.370	5.15	24.82	25.0	118.9	145.3	.0081	.018	.296	.939
		A			.375	5.23	24.18	30.6	122.3	150.7	.0079	.017	.280	.900

		A		U tube manometers except for velocities above 8.63 for which a spring gauge was used.	.449	6.26	33.65	25.0	124.1	150.8	.0079	.017	.280	.890
		A			.459	6.39	34.71	31.7	124.8	151.4	.0078	.016	.277	.892
		A			.464	6.46	36.09	24.4	123.7	149.9	.0079	.017	.280	.887
		A			.535	7.45	47.67	31.7	124.1	148.7	.0079	.017	.282	.908
		A			.543	7.56	49.09	27.2	123.1	147.2	.0079	.017	.287	.915
		A			.611	8.52	61.23	31.1	125.2	148.6	.0078	.016	.280	.900
		A			.619	8.63	63.68	27.2	124.4	147.4	.0079	.017	.285	.909
		A			.628	8.75	66.25	24.4	123.6	146.3	.0079	.017	.288	.913
		A			.687	9.58	81.00	33.3	122.4	143.8	.0079	.017	.297	.959
		A			.697	9.71	81.42	30.6	123.8	145.2	.0079	.017	.291	.934
		A			.744	10.35	92.87	32.2	123.5	144.2	.0079	.017	.294	.947
		A			.765	10.65	95.75	28.9	125.2	145.9	.0078	.016	.287	.919
		A			.772	10.76	99.75	31.7	121.1	144.2	.0079	.017	.293	.943
		A			.840	11.71	116.50	30.6	124.8	144.3	.0078	.016	.291	.936
304	FWF, JJW, 1917, 1 year.	A	5.72	Experimental pipe, Versailles, Pa. (See No. 302 for general notes.)	.196	1.10	.927	26.1	104.6	138.4	.0096	.023	.342	1.088
		A			.355	1.99	2.631	33.3	112.4	142.5	.0092	.020	.315	1.017
		A			.535	3.00	5.436	32.8	117.9	145.2	.0089	.018	.299	.965
		A			.713	4.00	9.200	32.2	120.8	145.7	.0087	.018	.292	.940
		A			.892	5.00	14.18	32.2	121.6	144.2	.0087	.017	.295	.950
		A			1.070	6.00	20.34	32.2	121.8	142.4	.0087	.017	.299	.963
		A			1.248	7.00	27.24	31.7	122.8	141.9	.0086	.017	.299	.963
		A			1.426	8.00	35.46	32.2	123.0	140.8	.0086	.017	.302	.973
		A			1.605	9.00	44.38	31.1	123.7	140.2	.0086	.017	.302	.971
		A			1.783	10.00	53.81	30.6	124.9	139.7	.0085	.016	.300	.964
306	EWS, 1907	B	6.00	Cornell University, Ithaca, N. Y., test on 6 lengths of rusty wrought-iron pipe used for some years in a steam-heating plant; flange connected; loss of head by differential water guage; quantity by volumetric measurement; 2 opposite piezometer holes at each end of reach.	------	1.61	1.61	20.5	112.7	144.6	.0092	.020	.309	.971
		B				2.27	3.09	--------	114.7	143.1	.0092	.020	.308	.969
		B				2.87	4.762	--------	116.8	143.5	.0091	.019	.305	.957
		B				3.55	7.087	--------	118.4	143.2	.0090	.018	.303	.951
		B				[illegible]	8.366	--------	119.1	143.1	.0089	.018	.303	.951
		B				[illegible]	10.067	--------	120.0	143.2	.0089	.018	.300	.942
		B				4.80	12.49	--------	120.6	142.6	.0088	.018	.301	.944
		B				5.22	14.59	--------	121.4	142.6	.0088	.018	.300	.941
		B				5.67	16.98	--------	122.2	142.7	.0088	.017	.298	.937
		B				6.03	19.14	--------	122.4	142.2	.0088	.017	.299	.938
308	FCS, 1919, 3 years.	A	7.69	Lateral No. 21 from Chatsworth high line, Los Angeles. Bell-and-spigot joint pipe, coated with coal-tar dip.	--------	4.50	10.99	--------	107.3	123.8	.0101	.022	.387	1.227
		A				5.52	16.05	--------	108.9	123.7	.0100	.022	.383	1.215
		A				5.63	16.21	--------	110.6	125.5	.0099	.021	.372	1.181
		A				6.55	21.29	--------	112.3	126.0	.0098	.020	.368	1.166
310	FWF, JJW, 1917, 1 year.	C	8.00	Experimental pipe, Versailles, Pa. Perfectly straight, wrought steel pipe, with patent joints about 20 feet apart; coated with coal tar; quantity by Venturi meter; loss of head by mercury U tubes, after tests pipe taken down and slight deposit mud found.	.349	1.00	.38	25.6	125.6	164.2	.0087	.016	.243	.774
		B			.698	2.00	1.59	23.9	122.8	152.2	.0091	.017	.273	.864
		C			1.047	3.00	3.15	26.7	130.9	159.6	.0087	.015	.250	.797
		A			1.396	4.00	5.81	26.7	128.5	151.2	.0089	.016	.267	.851
		A			1.746	5.00	8.66	27.2	131.6	152.4	.0087	.015	.261	.830
		A			2.095	6.00	12.18	27.8	133.2	152.1	.0086	.0145	.259	.828
		A			2.444	7.00	16.48	25.6	133.6	150.7	.0086	.014	.262	.831
		A			2.793	8.00	21.19	25.0	134.6	150.4	.0086	.014	.261	.828
		A			3.142	9.00	26.65	24.4	135.0	149.6	.0086	.014	.262	.831
		A			3.316	9.50	29.58	24.6	135.3	149.1	.0086	.014	.263	.834
311	JD, 1926	A	14.00	Main supply line, Bend, Oreg. New pipe	8.909	8.334	13.62	2.0	132.7	139.8	.0096	.015	.287	.889

[1] These tests were made after bulletin had been submitted for publication. Description will be found in Appendix 2, p. 126.

[2] For compound pipes of this class see Nos. 156-162.

TABLE 1.—*Elements of experiments for the determination of friction losses in sheet metal and plate-metal pipe, with retardation coefficients in various formulas*

CONTINUOUS-INTERIOR PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cubic feet*	*Feet*	*Feet*	*° C.*						
312	ELM, 1926		[3]19.2	Colorado Springs, Colo.; municipal plant penstock, coupling joint pipe, nominal 20-inch outside diameter of plate from 5/16 to 9/16 inch thick; weighted average diameter 19.2 inches; quantity by Venturi meter low flows given D rating as gauge No. 2 not differential but was under nearly 500 pounds head, close reading impossible.	6.0	2.98	.99	----	149.7	168.8	0.0090	0.0116	0.209	0.65
		D			7.0	3.47	1.43	----	145.0	161.5	.0093	.0122	.226	.70
		D			8.0	3.97	1.99	----	140.6	154.2	.0096	.0130	.243	.75
		D			9.0	4.47	2.59	----	138.8	150.3	.0097	.0134	.254	.79
		D			10.0	4.96	3.30	----	136.5	146.6	.0098	.0138	.264	.82
		C			11.0	5.46	4.10	----	134.8	143.5	.0100	.0142	.273	.84
		C			12.0	5.96	4.94	----	134.0	141.7	.0101	.0143	.279	.86
		C			13.0	6.45	5.90	----	132.7	139.3	.0101	.0146	.287	.89
		B			14.0	6.95	6.94	----	131.9	137.5	.0102	.0148	.293	.91
		B			15.0	7.44	8.13	----	130.4	135.2	.0103	.0151	.301	.93
		B			16.0	7.94	9.49	----	128.8	132.7	.0104	.0155	.311	.96
		B			16.1	7.99	9.65	----	128.5	132.3	.0104	.0156	.312	.97
		B												
313	JSP, FCS, 1927.	A	26.0	Marin municipal water district California. Full welded line, very crooked; 1 year old.	10.68	2.90	1.11	----	118.2	127.5	.0116	.018	.344	1.06
		A			20.17	5.47	3.67	14.0	122.6	125.3	.0113	.017	.341	1.05
		A			22.26	6.04	4.46	----	123.0	126.1	.0113	.017	.342	1.06
314	CSRP, 1904	B	30.0	Coolgardie pipe line, Australia. Lock-bar pipe, butt joints, leaded sleeve.	----	1.889	.426	----	115.8	127.3	.0119	.019	.348	1.079
		B				2.115	.530	----	116.2	126.7	.0119	.019	.350	1.085
316	HFM, 1888, 2.3 years.	B	84.0	Pacific Mills penstock No. 2, Lawrence, Mass. Butt jointed, all rivet heads countersunk, "making a smooth interior surface."	----	2.589	.266	----	120.0	117.6	.0137	.018	.371	1.15
		B				2.728	.281	----	123.0	120.3	.0134	.017	.354	1.10
		B				2.772	.297	----	121.6	118.7	.0135	.017	.365	1.13
		B				3.179	.397	----	120.6	116.4	.0137	.017	.375	1.16
		B				3.524	.469	----	123.0	117.9	.0134	.017	.366	1.13

316a	HFM, 1899, 13 years.	B	84.0	Same pipe as No. 316, 11 years later	--------	2.075	.240	--------	101.3	99.7	.0160	.025	.512	1.59
		B				2.579	.381	--------	99.9	96.5	.0163	.626	.536	1.66
		B				3.379	.630	--------	101.7	96.4	.0161	.025	.530	1.64
		B				3.588	.735	--------	100.0	94.2	.0164	.026	.549	1.70
318	HFM, 1888, 2.3 years.	B	84.0	Pacific Mills penstock No. 3 parallel to No. 316. Same condition.	--------	2.699	.307	--------	116.5	113.5	.0141	.019	.396	1.23
		B				3.489	.516	--------	116.1	110.9	.0142	.019	.409	1.27
		B				3.790	.603	--------	116.7	110.7	.0142	.019	.408	1.26
		B				3.803	.608	--------	116.6	110.6	.0142	.019	.409	1.27
318a	HFM, 1899, 13 years.	B	84.0	Same pipe as No. 318, 11 years later	--------	3.612	.690	--------	104.0	98.1	.0158	.024	.510	1.58
		B				3.660	.692	--------	105.2	99.2	.0156	.023	.499	1.55
		B				3.670	.705	--------	104.5	98.5	.0157	.024	.508	1.57
		B				3.705	.724	--------	104.1	98.0	.0157	.024	.511	1.58

[1] Average.

TABLE 2.—*Elements of experiments for the determination of local friction losses in compound pipes, with coefficients of retardation in various formulas that will satisfy measured total loss of head*

Pipe No.	Experimenter, year, and age of pipe	Observation rating	Experimenter's number	Name and description of pipe	d= Various inside diameters	Total lengths for corresponding diameters	Quantity, Q, in cubic feet per second and total loss of head; h_f, in feet, for reach tested	V=Velocity, in corresponding size pipe	Coefficients of retardation and local losses: Kutter's n (approximate)	Williams-Hazen: C_w= Williams-Hazen	Williams-Hazen: Local loss of head for C_w as column 11	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: Local loss of head for K_s as column 13	Scobey: m=Viscosity considered $K_s=mv^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
					Inches	*Feet*		*Feet per second*			*Feet*		*Feet*	
150	JNT, 1876, new.	D	......	Rochester, N. Y., conduit No. 1. The 24-inch is cast iron for which C_w was assumed 130. (See next set.)	24.0	36,031.0	Q=13.384 h_f=143.8	4.26	......	[1] 130.0	......	[1] 0.34	......	[1] 1.05
					24.0	15,419.0		4.26	......	126.0	......	.320	......	1.02
					36.0	50,819.0		1.894	......	126.0	......	.320	......	1.02
150	JNT, 1876, new.	D		Same, with C_w assumed at 135, values K_s taken to conform to assumed C_w.	24.0	36,031.0	Q=13.384 h_f=143.8	4.26	......	[1] 135.0	......	[1] .32	......	[1] .99
					24.0	15,419.0		4.26	......	119.6	......	.361	......	1.12
					36.0	50,819.0		1.894	......	119.6	......	.361	......	1.12
152	RW, 1921, new.	C	......	Penstock, Spring Gap power house, Pacific Gas & Electric Co., California. During plant-efficiency tests over all losses through new penstock taken; quantity measured over weir; loss of head by pressure gauges, read to nearest 5 pounds. For this reason tests at lower flows given but a C rating. Great length of line makes tests at higher flows of material value.	36.57	2,410.0	Q=18.6 h_f=11.5	2.55	0.014+	103.6	2.07	.494	2.050	1.53
					36.07	1,320.0		2.62	.014+	103.6	1.20	.494	1.210	1.53
					30.21	2,310.0		3.74	.014−	103.6	5.06	.494	5.089	1.53
					30.00	1,375.0		3.79	.014−	103.6	3.09	.494	3.135	1.53
											11.42		11.493	
152	......	C	......		36.57	2,410.0	Q=25.7 h_f=23.1	3.52	.014+	97.8	4.22	.547	4.214	1.69
152	......	B	......		36.07	1,320.0		3.62	.014+	97.8	2.44	.547	2.460	1.69
					30.21	2,310.0		5.16	.014	97.8	10.16	.547	10.345	1.69
					30.00	1,375.0		5.24	.014	97.8	6.31	.547	6.390	1.69
											23.13		23.409	
152	......	B	......		36.57	2,410.0	Q=32.9 h_f=57.7	4.51	.017+	76.5	10.60	.842	10.409	2.61
					36.07	1,320.0		4.64	.017+	76.5	6.11	.842	6.079	2.61
					30.21	2,310.0		6.61	.017−	76.5	25.64	.842	25.518	2.61
					30.00	1,375.0		6.70	.017−	76.5	15.68	.842	15.676	2.61
											58.03		57.652	

					36.57	2,410.0	$Q=39.4$ $h_f=80.7$	5.40	.017	76.5	14.5		.838	14.581	2.60
					36.07	1,320.0		5.55	.017	76.5	8.5		.838	8.528	2.60
					30.21	2,310.0		7.01	.017	76.5	35.8		.838	35.695	2.60
					30.00	1,375.0		8.03	.017	76.5	22.0		.838	22.031	2.60
											80.8			80.835	
154	FCS, 1923, 14 years.	A		River siphon, Bitter Root Valley irrigation district, Montana. Conditions right for very accurate test.	71.0	276.0	$Q=268.5$ $h_f=37.9$	9.73	.0135+	106.4	1.25		.423	1.244	1.31
					60.37	133.3		11.23	.0135+	106.4	.85		.423	.850	1.31
					61.8	1,879.9		12.90	.0135−	106.4	16.79		.423	16.882	1.31
					66.37	2,703.2		11.23	.0135−	106.4	17.28		.423	17.255	1.31
					71.0	370.0		9.73	.0135−	106.4	1.67		.423	1.658	1.31
											37.84			37.899	
156	HLD, 1921, new.	B	D-1	Penstock, Kern River No. 3 power plant, Southern California Edison Co., California; a compound pipe consisting of 3 lengths of riveted pipe made of plate from ⅜ to 7/16 inch thick and 2 lengths of welded pipe of plate from 7/16 to 11/16 inch thick. The riveted pipes, of 84, 78, and 72 inches in diameter have longitudinal seams double and triple riveted, lapping joints. The girth seams are butt jointed with outside butt straps. The welded pipe consists of 853 feet of 66-inch and 870 feet of 60-inch pipe, the latter including a Venturi meter near the lower end. The girth joints of the welded pipe are double-riveted bump joints.	84.0	316.0	$Q=100.0$ $h_f=1.69$	2.60	.015+	[1] 105.0	.104		1.440	.100	[1] 1.36
					78.0	237.0		3.01	.015−	[1] 105.0	.112		1.440	.108	[1] 1.36
					72.0	206.0		3.54	.015−	[1] 105.0	.143		1.440	.139	[1] 1.36
					66.0	853.0		4.21	.011−	144.0	.512		.255	.513	.79
					60.0	870.0		5.09	.011−	144.0	.818		.255	.830	.79
						2,482					1.689			1.690	
156	....	C	C-1		84.0	316.0	$Q=101.6$ $h_f=1.56$	2.64	.015+	[1] 105.0	.107		1.440	.103	[1] 1.36
					78.0	237.0		3.06	.015−	[1] 105.0	.116		1.440	.111	[1] 1.36
					72.0	206.0		3.59	.015−	[1] 105.0	.148		1.440	.144	[1] 1.36
					66.0	853.0		4.28	.011	155.6	.453		.221	.457	.68
					60.0	870.0		5.17	.011	155.6	.735		.221	.744	.68
											1.559			1.559	
156	....	B	C-2		84.0	316.0	$Q=126.3$ $h_f=2.79$	3.28	.015+	[1] 105.0	.161		1.440	.156	[1] 1.36
					78.0	237.0		3.80	.015−	[1] 105.0	.174		1.440	.168	[1] 1.36
					72.0	206.0		4.47	.015−	[1] 105.0	.222		1.440	.217	[1] 1.36
					66.0	853.0		5.32	.011	137.7	.862		.274	.857	.85
					60.0	870.0		6.43	.011	137.7	1.383		.274	1.393	.85
											2.802			2.701	
156	....	A	D-2		84.0	316.0	$Q=151.7$ $h_f=4.34$	3.94	.015+	[1] 105.0	.224		1.440	.221	[1] 1.36
					78.0	237.0		4.57	.015−	[1] 105.0	.242		1.440	.230	[1] 1.36
					72.0	206.0		5.37	.015−	[1] 105.0	.313		1.440	.308	[1] 1.36
					66.0	853.0		6.38	.011	128.4	1.348		.307	1.359	.95
					60.0	870.0		7.72	.011	128.4	2.201		.307	2.213	.95
											4.328			4.340	

[1] Coefficient assumed in order to compute coefficient of other type pipe in compound line.

TABLE 2.—*Elements of experiments for the determination of local friction losses in compound pipes, with coefficients of retardation in various formulas that will satisfy measured total loss of head*—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	Experimenter's number	Name and description of pipe	d=Various inside diameters	Total lengths for corresponding diameters	Quantity, Q, in cubic feet per second and total loss of head; h_f, in feet, for reach tested	V=Velocity, in corresponding size pipe	Kutter's n (approximate)	Coefficients of retardation and local losses: Williams-Hazen: C_w=Williams-Hazen	Williams-Hazen: Local loss of head for C_w as column 11	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: Local loss of head for K_s as column 13	Scobey: m=Viscosity considered $K_s=mv^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
					Inches	*Feet*		*Feet per second*			*Feet*		*Feet*	
156	HLD, 1921, new.	A	C-4	Test C was made with unit No. 2 running 600 revolutions per minute; test D with the same unit running 500 revolutions per minute. For runs C-3 and C-5 the total loss of head was not measured.[2]	84.0	316.0	Q=174.9 h_f=5.48	4.55	[1]0.015+	[1]105.0	0.295	[1]0.440	0.291	[1]1.63
					78.0	237.0		5.27	[1].015−	[1]105.0	.315	[1].440	.313	[1]1.36
					72.0	206.0		6.19	[1].015−	[1]105.0	.406	[1].440	.403	[1]1.36
					66.0	853.0		7.36	.012−	131.3	1.697	.203	1.702	.91
					60.0	870.0		8.91	.012−	131.3	2.775	.203	2.770	.91
											5.488		5.470	
156	----------	A	D-3	----------	84.0	316.0	Q=200.3 h_f=7.52	5.21	[1].015+	[1]105.0	.370	[1].440	.376	[1]1.36
					78.0	237.0		6.04	[1].015−	[1]105.0	.408	[1].440	.406	[1]1.36
					72.0	206.0		7.09	[1].015−	[1]105.0	.523	[1].440	.521	[1]1.36
					66.0	853.0		8.43	.012−	125.7	2.380	.315	2.365	.98
					60.0	870.0		10.20	.012−	125.7	3.845	.315	3.852	.98
											7.535		7.520	
156	----------	A	C-6	----------	84.0	316.0	Q=223.8 h_f=9.20	5.82	[1].015+	[1]105.0	.466	[1].440	.464	[1]1.36
					78.0	237.0		6.75	[1].015−	[1]105.0	.500	[1].440	.500	[1]1.36
					72.0	206.0		7.92	[1].015−	[1]105.0	.639	[1].440	.644	[1]1.36
					66.0	853.0		9.42	.012−	125.7	2.909	.3115	2.889	.97
					60.0	870.0		11.40	.012−	125.7	4.698	.3115	4.704	.97
											9.212		9.201	
156	----------	B	C-7	----------	84.0	316.0	Q=249.7 h_f=11.70	6.49	[1].015+	[1]105.0	.569	[1].440	.570	[1]1.36
					78.0	237.0		7.52	[1].015−	[1]105.0	.612	[1].440	.614	[1]1.36
					72.0	206.0		8.83	[1].015−	[1]105.0	.787	[1].440	.791	[1]1.36
					66.0	853.0		10.51	.012−	122.6	3.736	.324	3.703	1.00
					60.0	870.0		12.71	.012−	122.6	6.012	.324	6.022	1.00

									11.716		11.700	
156	A	D-4	84.0	316.0	Q=250.9	6.52	[1] .015+	[1] 105.0	.577	[1] .440	.575	[1] 1.36
			78.0	237.0	h_f=11.86	7.56	[1] .015−	[1] 105.0	.616	[1] .440	.621	[1] 1.36
			72.0	206.0		8.88	[1] .015−	[1] 105.0	.795	[1] .440	.800	[1] 1.36
			66.0	853.0		10.56	.012−	122.3	3.745	.326	3.753	1.01
			60.0	870 0		12.77	.012−	122.3	6.125	.326	6.111	1.01
									11.858		11.860	
156	A	C-8	84.0	316.0	Q=274.2	7.13	[1] .015−	[1] 105.0	.683	[1] .440	.683	[1] 1.36
			78.0	237.0	h_f=14.17	8.26	[1] .015−	[1] 105.0	.725	[1] .440	.734	[1] 1.36
			72.0	206.0		9.40	[1] .015−	[1] 105.0	.886	[1] .440	.891	[1] 1.36
			66.0	853.0		11.54	.012−	121.0	4.512	.331	4.516	1.03
			60.0	870.0		13.96	.012−	121.0	7.352	.331	7.344	1.03
									14.158		14.168	
156	A	C-9	84.0	316.0	Q=299.0	7.71	[1] .015−	[1] 105.0	.781	[1] .440	.792	[1] 1.36
			78.0	237.0	h_f=16.74	9.01	[1] .015−	[1] 105.0	.856	[1] .440	.867	[1] 1.36
			72.0	206.0		10.58	[1] .015−	[1] 105.0	1.092	[1] .440	1.116	[1] 1.36
			66.0	853.0		12.58	.012−	121.0	5.348	.331	5.317	1.03
			60.0	870.0		15.22	.012−	121.0	8.691	.331	8.653	1.03
									16.768		16.745	
156	A	D-5	84.0	316.0	Q=300.5	7.81	[1] .015−	[1] 105.0	.803	[1] .440	.812	[1] 1.36
			78.0	237.0	h_f=16.79	9.06	[1] .015−	[1] 105.0	.863	[1] .440	.876	[1] 1.36
			72.0	206.0		10.63	[1] .015−	[1] 105.0	1.110	[1] .440	1.126	[1] 1.36
			66.0	853.0		12.65	.012−	121.4	5.340	.328	5.323	1.02
			60.0	870.0		15.30	.012−	121.4	8.691	.328	8.654	1.02
									16.807		16.791	
156	A	C-10	84.0	316.0	Q=319.0	8.29	[1] .015−	[1] 105.0	.891	[1] .440	.909	[1] 1.36
			78.0	237.0	h_f=18.95	9.61	[1] .015−	[1] 105.0	.962	[1] .440	.979	[1] 1.36
			72.0	206.0		11.28	[1] .015−	[1] 105.0	1.238	[1] .440	1.261	[1] 1.36
			66.0	843.0		13.43	.012−	120.8	6.048	.3306	6.019	1.02
			60.0	870.0		16.24	.012−	120.8	9.831	.3306	9.779	1.02
									18.970		18.947	

[1] Coefficient assumed in order to compute coefficient of other type pipe in compound line.

[2] Note that there are 2 types of pipe. For this reason it is necessary to assume the coefficient of retardation in 1 type and compute for the other type. As the influence of the welded pipe far overshadows that of the riveted pipe, due to greater lengths and velocities, the retardation factors have been assumed for the riveted pipe and the factor for the welded pipe computed so that the computed total loss of head would approximate the observed loss. Since an extended series of tests were run, extending into high velocities, this series is considered by the writer as giving better results on welded pipe with bump joints than the tests on Pit No. 1 penstock, involving shorter reaches of larger pipe. (See pipe Nos. 228 and 230.) If the total friction loss, in feet of head is plotted on logarithmic paper against flow in second-feet then the resulting straight-line curve closely approximates the line as computed by the Kutter formula, using a value of n=0.016 for riveted pipe and 0.0112 for welded pipe. From the total loss of head for each run was deducted the velocity head in the 60-inch pipe, the entry loss and the loss through the Venturi meter.

TABLE 2.—*Elements of experiments for the determination of local friction losses in compound pipes, with coefficients of retardation in various formulas that will satisfy measured total loss of head*—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	Experimenter's number	Name and description of pipe	d=Various inside diameters	Total lengths for corresponding diameters	Quantity, Q, in cubic feet per second and total loss of head; h_f, in feet, for reach tested	V=Velocity, in corresponding size pipe	Coefficients of retardation and local losses: Kutter's n (approximate)	Williams-Hazen: C_w=Williams-Hazen	Williams-Hazen: Local loss of head for C_w as column 11	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: Local loss of head for K_s as column 13	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
					Inches	*Feet*		*Feet per second*			*Feet*		*Feet*	
156a	HLD, 1921, new.	A	B-1		84.0	316.0	Q=99.8 h_f=2.21	2.59	[1] 0.015	[1] 105.0	0.104	[1] 0.440	0.100	[1] 1.36
					78.0	237.0		3.01	[1] .015	[1] 105.0	.111	[1] .440	.108	[1] 1.36
					72.0	206.0		3.53	[1] .015	[1] 105.0	.143	[1] .440	.130	[1] 1.36
					66.0	853.0		4.20	.013	120.0	.708	.355	.710	1.10
					60.0	870.0		5.08	.013	120.0	1.140	.355	1.153	1.10
											2.206		2.210	
156a	......	C	B-2	Penstock, Kern River No. 3 power plant, Southern California Edison Co., California. A compound pipe paralleling No. 156 above. This series of runs made for unit No. 1 running at 600 revolutions per minute. All other notes on No. 156 above apply equally to this pipe. Note that the same assumed values of retardation units on the riveted pipe result in about the same computed retardation units for the welded pipe.	84.0	316.0	Q=144.5 h_f=3.35	3.76	[1] .015	[1] 105.0	.208	[1] .440	.203	[1] 1.36
					78.0	237.0		4.36	[1] .015	[1] 105.0	.223	[1] .440	.218	[1] 1.36
					72.0	206.0		5.11	[1] .015	[1] 105.0	.286	[1] .440	.280	[1] 1.36
					66.0	853.0		6.08	.011	144.0	1.006	.249	1.008	.77
					60.0	870.0		7.36	.011	144.0	1.631	.249	1.641	.77
											3.354		3.350	
156a	......	B	B-3		84.0	316.0	Q=196.6 h_f=7.19	5.11	[1] .015	[1] 105.0	.367	[1] .440	.363	[1] 1.36
					78.0	237.0		5.93	[1] .015	[1] 105.0	.393	[1] .440	.391	[1] 1.36
					72.0	206.0		6.95	[1] .015	[1] 105.0	.505	[1] .440	.502	[1] 1.36
					66.0	853.0		8.27	.012	126.0	2.278	.312	2.252	.97
					60.0	870.0		10.01	.012	126.0	3.680	.312	3.678	.97
											7.223		7.186	
156a	......	A	B-4		84.0	316.0	Q=249.5 h_f=11.84	6.48	[1] .015	[1] 105.0	.569	[1] .440	.569	[1] 1.36
					78.0	237.0		7.52	[1] .015	[1] 105.0	.611	[1] .440	.614	[1] 1.36
					72.0	206.0		8.83	[1] .015	[1] 105.0	.785	[1] .440	.791	[1] 1.36
					66.0	853.0		10.50	.012	121.6	3.796	.3296	3.755	1.02
					60.0	870.0		12.70	.012	121.6	6.160	.3296	6.111	1.02

											11.921		11.840	
156a		A	B-5		84.0	316.0	$Q=296.5$ $h_f=16.48$	7.71	[1].015	[1]105.0	.790	[1].440	.792	[1]1.36
					78.0	237.0		8.94	[1].015	[1]105.0	.844	[1].440	.854	[1]1.36
					72.0	206.0		10.49	[1].015	[1]105.0	1.084	[1].440	1.098	[1]1.36
					66.0	853.0		12.48	.012	121.0	5.272	.331	5.214	1.03
					60.0	870.0		15.10	.012	121.0	8.526	.331	8.520	1.03
											16.516		16.478	
158	FCS, 1917, 2 years.	A		St. Mary's crossing siphon, Montana. Riveted; composed 1,432 feet of 90-inch, then 837 feet of 84-inch, then 953 feet of 90-inch. In computations 90-inch sections combined.	84.0	837.0	$Q=192.3$ $h_f=1.961$	5.00	.013−	120.5	.716	.341	.713	1.06
					90.0	2,062.0		4.35	.013+	120.5	1.253	.341	1.248	1.06
											1.969		1.961	
					84.0	837.0	$Q=235.3$ $h_f=2.842$	6.12	.013−	121.5	1.029	.336	1.034	1.04
					90.0	2,062.0		5.33	.013−	121.5	1.818	.336	1.808	1.04
											2.847		2.842	
160	HLD, 1921, new.	B		Little Brush Creek siphon. Kern River No. 3 conduit, Southern California Edison Co. Composed of 7 reaches with 6 tapers between. Reaches of same size combined for computations. No correction for "special" loss at tapers. Sharp curve at bottom of sag in siphon pipe.	114.0	488.0	$Q=350.0$ $h_f=1.6$	4.94	.015+	95.5	.443	.517	.441	1.60
					108.0	84.0		5.50	.015+	95.5	.099	.517	.099	1.60
					102.0	324.0		6.17	.015+	95.5	.349	.517	.348	1.60
					96.0	340.0		6.96	.015+	95.5	.707	.517	.712	1.60
											1.598		1.600	
160		B			114.0	488.0	$Q=358.0$ $h_f=1.70$	5.05	.015+	94.9	.469	.526	.467	1.63
					108.0	84.0		5.63	.015+	94.9	.106	.526	.105	1.63
					102.0	324.0		6.31	.015+	94.9	.371	.526	.370	1.63
					96.0	340.0		7.12	.015+	94.9	.752	.526	.757	1.63
											1.698		1.699	
160		B			114.0	488.0	$Q=368.0$ $h_f=1.80$	5.19	.015+	94.2	.495	.528	.495	1.64
					108.0	84.0		5.78	.015+	94.2	.110	.528	.110	1.64
					102.0	324.0		6.49	.015+	94.2	.392	.528	.392	1.64
					96.0	340.0		7.32	.015+	94.2	.804	.528	.801	1.64
											1.801		1.798	
160		B			114.0	488.0	$Q=390.0$ $h_f=2.10$	5.63	.015+	94.2	.579	.528	.577	1.64
					108.0	84.0		6.27	.015+	94.2	.129	.528	.130	1.64
					102.0	324.0		7.03	.015+	94.2	.458	.528	.458	1.64
					96.0	340.0		7.94	.015+	94.2	.934	.528	.935	1.64
											2.100		2.100	

[1] Coefficient assumed in order to compute coefficient for other type pipe in compound line.

TABLE 2.—*Elements of experiments for the determination of local friction losses in compound pipes, with coefficients of retardation in various formulas that will satisfy measured total loss of head*—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	Experimenter's number	Name and description of pipe	d= Various inside diameters	Total lengths for corresponding diameters	Quantity, Q, in cubic feet per second and total loss of head; h_f, in feet, for reach tested	V=Velocity, in corresponding size pipe	Kutter's n (approximate)	Coefficients of retardation and local losses: Williams-Hazen: C_w= Williams-Hazen	Williams-Hazen: Local loss of head for C_w as column 11	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: Local loss of head for K_s as column 13	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
					Inches	*Feet*		*Feet per second*			*Feet*		*Feet*	
160	----------	B	----------		114.0	488.0	Q=436.0	6.15	0.016+	91.6	0.721	0.553	0.714	1.71
					108.0	84.0	h_f=2.60	6.85	.016+	91.6	.161	.553	.161	1.71
					102.0	324.0		7.68	.016+	91.6	.567	.553	.566	1.71
					96.0	340.0		8.68	.016+	91.6	1.159	.553	1.158	1.71
											2.608		2.599	
160	----------	B	----------	Little Brush Creek siphon. Kern River No. 3 conduit, Southern California Edison Co.—Continued.	114.0	488.0	Q=459.0	6.48	.016−	90.9	.800	.560	.798	1.73
					108.0	84.0	h_f=2.90	7.21	.016−	90.9	.179	.560	.178	1.73
					102.0	324.0		8.09	.016−	90.9	.632	.560	.632	1.73
					96.0	340.0		9.13	.016−	90.9	1.289	.560	1.291	1.73
											2.900		2.899	
160	----------	B	----------		114.0	488.0	Q=459.0	6.48	.016	90.1	.815	.569	.811	1.76
					108.0	84.0	h_f=2.95	7.21	.016	90.1	.181	.569	.182	1.76
					102.0	324.0		8.09	.016	90.1	.642	.569	.642	1.76
					96.0	340.0		9.13	.016	90.1	1.309	.569	1.311	1.76
											2.947		2.946	

160		B			114.0	488.0	Q=488.0 h_f=3.40	6.88	.016	88.9	.938	.584	.932	1.81
					108.0	84.0		7.67	.016	88.9	.209	.584	.210	1.81
					102.0	324.0		8.60	.016	88.9	.743	.584	.740	1.81
					96.0	340.0		9.71	.016	88.9	1.510	.584	1.514	1.81
											3.400		3.396	
160		B			114.0	488.0	Q=513.0 h_f=3.60	7.24	.016−	90.5	1.015	.562	.990	1.72
					108.0	84.0		8.06	.016−	90.5	.221	.562	.222	1.72
					102.0	324.0		9.04	.016−	90.5	.784	.562	.784	1.72
					96.0	340.0		10.21	.016−	90.5	1.584	.562	1.604	1.72
											3.604		3.600	
160		B			114.0	488.0	Q=520.0 h_f=3.80	7.34	.016−	88.9	1.056	.578	1.046	1.78
					108.0	84.0		8.17	.016−	83.9	.234	.578	.234	1.78
					102.0	324.0		9.16	.016−	88.9	.828	.578	.826	1.78
					96.0	340.0		10.35	.016−	88.9	1.686	.578	1.692	1.78
											3.804		3.798	
160		B		Little Brush Creek siphon. Kern River No. 3 conduit, Southern California Edison Co.—Continued.	114.0	488.0	Q=528.0 h_f=3.80	7.45	.016−	90.3	1.049	.562	1.045	1.72
					108.0	84.0		8.30	.016−	90.3	.235	.562	.234	1.72
					102.0	324.0		9.31	.016−	90.3	.829	.562	.828	1.72
					96.0	340.0		10.51	.016−	90.3	1.687	.562	1.693	1.72
											3.800		3.800	
160		B			114.0	488.0	Q=538.0 h_f=4.20	7.59	.016+	87.1	1.160	.600	1.156	1.86
					108.0	84.0		8.46	.016+	87.1	.259	.600	.259	1.86
					102.0	324.0		9.48	.016+	87.1	.916	.600	.917	1.86
					96.0	340.0		10.70	.016+	87.1	1.870	.600	1.871	1.86
											4.205		4.203	
160		B			114.0	488.0	Q=544.0 h_f=4.40	7.67	.016+	86.0	1.210	.615	1.209	1.90
					108.0	84.0		8.55	.016+	86.0	.273	.615	.272	1.90
					102.0	324.0		9.59	.016+	86.0	.961	.615	.959	1.90
					96.0	340.0		10.82	.016+	86.0	1.958	.615	1.957	1.90
											4.402		4.397	
160		B			114.0	488.0	Q=554.0 h_f=4.60	7.82	.016+	85.6	1.266	.621	1.267	1.92
					108.0	84.0		8.71	.016+	85.6	.286	.621	.284	1.92
					102.0	324.0		9.76	.016+	85.6	1.008	.621	1.001	1.92
					96.0	340.0		11.62	.016+	85.6	2.040	.621	2.047	1.92
											4.600		4.599	

TABLE 2.—*Elements of experiments for the determination of local friction losses in compound pipes, with coefficients of retardation in various formulas that will satisfy measured total loss of head*—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	Experimenter's number	Name and description of pipe	d= Various inside diameters	Total lengths for corresponding diameters	Quantity, Q, in cubic feet per second and total loss of head; h_t, in feet, for reach tested	V=Velocity, in corresponding size pipe	Coefficients of retardation and local losses: Kutter's n (approximate)	Williams-Hazen: C_w= Williams-Hazen	Williams-Hazen: Local loss of head for C_w as column 11	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: Local loss of head for K_s as column 13	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
					Inches	*Feet*		*Feet per second*			*Feet*		*Feet*	
160	----------	B	------	Little Brush Creek siphon, Kern River No. 3 conduit, Southern California Edison Co.—Continued.	114.0	488.0	Q=570.0	8.04	0.016+	85.9	1.327	0.613	1.319	1.90
					108.0	84.0	h_t=4.80	8.96	.016+	85.9	.296	.613	.296	1.90
					102.0	324.0		10.05	.016+	85.9	1.053	.613	1.046	1.90
					96.0	340.0		11.34	.016+	85.9	2.125	.613	2.134	1.90
											4.801		4.795	
160	----------	B	------		114.0	488.0	Q=582.0	8.21	.016	86.6	1.358	.602	1.348	1.86
					108.0	84.0	h_t=4.90	9.15	.016	86.6	.302	.602	.303	1.86
					102.0	324.0		10.26	.016	86.6	1.065	.602	1.068	1.86
					96.0	340.0		11.58	.016	86.6	2.176	.602	2.182	1.86
											4.901		4.901	
160	----------	B	------		114.0	488.0	Q=600.0	8.46	.016+	85.1	1.466	.623	1.477	1.93
					108.0	84.0	h_t=5.36	9.43	.016+	85.1	.330	.623	.332	1.93
					102.0	324.0		10.57	.016+	85.1	1.245	.623	1.168	1.93
					96.0	340.0		11.94	.016+	85.1	2.320	.623	2.391	1.93
											5.361		5.368	

162	AK, 1926, new.	C	--------	Continuous-interior pipe. Gordon Valley pipe, Vallejo, Calif. Electric-welded; longitudinal seams butt-welded; girth seams hand lapwelded; cylinder joints; line sinuous; single-angle joints not to exceed 10°; line of 79,755 feet 24-inch and 35,260 feet 22-inch pipe; quantity measured by Venturi meter; loss of head by Bourdon type pressure gauge, reading to nearest pound only.	24.0	79,755.0	Q=5.42	1.73	.011+	132.6	34.77	.329	34.69	1.02
					22.0	35,260.0	h_t=58.±	2.05	.011+	132.6	23.90	.329	23.27	1.02
											58.67		57.96	
		B			24.0	79,755.0	Q=6.19	1.97	.011−	139.7	40.44	.296	40.12	.92
					22.0	35 260.0	h_t=67.±	2.34	.011−	139.7	27.19	.296	26.94	.92
											67.63		67.06	
		B			24.0	79,755.0	Q=6.98	2.22	.011−	138.5	50.88	.299	50.88	.93
					22.0	35,260.0	h_t=85.±	2.64	.011−	138.5	34.38	.299	34.20	.93
											85.26		85.08	
		B			24.0	79,755.0	Q=7.74	2.46	.011−	137.9	62.29	.301	62.05	.93
					22.0	35,260.0	h_t=104.±	2.93	.011−	137.9	41.96	.301	41.96	.93
											104.25		104.01	
		B			24.0	79,755.0	Q=8.35	2.66	.011−	141.4	69.55	.288	68.67	.89
					22.0	35,260.0	h_t=116.±	3.16	.011−	141.4	46.19	.288	46.19	.89
											115.74		114.86	
[1]164	FCS, 1929, new.	A	--------	Girth riveted pipe. East Bay municipal utility district, California. Pipe practically straight, across level floor of San Joaquin Valley. Pipe welded on straight seams but riveted on field seams. Note that double pipes are used on 3 river crossings. Last reach of 65-inch pipe welded all seams. See No. 227.	63.0	23,664.0		4.109	.012+	129.4	17.40	.309	17.35	.98
					61.0	31,835.5		4.385	.012+	129.4	27.31	.309	27.40	.98
					2-54.0	998.7		2.798	.012+	129.4	.42	.309	.42	.98
					61.0	16,137.6		4.385	.012+	129.4	13.85	.309	13.84	.98
					63.0	27,950.3	Q=88.97	4.109	.012+	129.4	20.57	.309	20.49	.98
					65.0	6,663.5	h_t=101.66	3.862	.012+	129.4	4.21	.309	4.18	.98
					2-54.0	991.6		2.798	.012+	129.4	.43	.309	.42	.98
					65.0	7,420.7		3.862	.012+	129.4	4.69	.309	4.65	.98
					2-54.0	945.8		2.798	.012+	129.4	.41	.309	.39	.98
					63.0	16,427.3		4.109	.012+	129.4	12.07	.309	12.05	.98
					65.0	780.0		3.862	.012+	129.4	.50	.309	.49	.98
						133,815.0					101.86		101.68	

[1] These tests were made after bulletin had been submitted for publication. Description will be found in Appendix 2, p. 127.

TABLE 3.—*Elements of experiments for the determination of friction losses in sheet-metal and plate-metal pipes, with retardation coefficients in various formulas*

DREDGE PIPES

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cu. feet*	*Feet*	*Feet*	°*C.*						
402	IEH, 1919, new	A	15.0	At Taylorsville, Ohio. Pipe composed of 74 lengths of riveted soft steel and 56 lengths of welded hard steel; tests conducted with clear water only; loss of head by calibrated pressure gauges; velocities by color-velocity and salt-velocity methods; potassium permanganate used for color; pipe units 16 feet long, stovepipe joints; no apparent reason for great difference in capacity when compared with No. 404 below.	--------	6.92	13.4	--------	107.0	112.1	0.0116	0.023	0.434	1.34
		A				7.01	13.4	--------	108.3	113.6	.0114	.022	.425	1.32
		A				8.04	17.2	--------	109.7	113.8	.0113	.021	.419	1.30
		A				8.10	17.4	--------	109.8	114.0	.0113	.021	.418	1.29
		A				8.80	20.6	--------	109.7	113.0	.0113	.021	.423	1.31
		A				8.88	20.4	--------	111.2	114.7	.0112	.021	.412	1.28
		A				9.33	23.3	--------	109.3	112.1	.0113	.022	.428	1.33
		B				9.68	28.9	--------	106.2	108.3	.0116	.023	.456	1.41
		A				9.88	26.5	--------	108.6	110.8	.0114	.022	.437	1.35
		B				9.93	23.3	--------	116.4	119.3	.0108	.019	.380	1.18
		A				10.9	28.9	--------	114.7	116.6	.0109	.020	.395	1.22
		A				10.9	29.2	--------	114.1	116.0	.0109	.020	.399	1.24
		B				11.3	33.6	--------	110.3	111.5	.0112	.021	.429	1.33
		A				11.8	32.9	--------	116.4	117.7	.0108	.019	.387	1.20
		A				12.7	37.8	--------	116.8	117.5	.0107	.019	.387	1.20
		B				12.7	36.7	--------	118.6	119.4	.0106	.018	.376	1.16
		B				13.4	45.0	--------	113.0	112.9	.0110	.020	.416	1.29
		A				13.7	45.0	--------	115.5	115.4	.0108	.019	.399	1.24

404	IEH, 1919, new	B	15.0	At Taylorsville dam, Ohio. Pipe composed of 8 joints of welded hard steel and 76 joints of riveted soft steel; tests with clear water only; methods same as for No. 402 above; no apparent reason for great difference in capacity when compared with No. 402 above; predominance of riveted pipe on No. 404 should yield capacity inferior to that of pipe with more welded lengths like No. 402 above.	--------	7.70	11.6	--------	127.9	134.9	.0100	.016	.307	.95
		B				7.66	11.3	--------	128.9	136.1	.0099	.016	.301	.93
		C				8.74	16.5	--------	121.8	126.6	.0104	.017	.343	1.06
		B				8.94	15.7	--------	127.7	133.0	.0100	.016	.313	.97
		C				9.94	21.4	--------	121.6	125.1	.0104	.017	.349	1.08
		B				10.2	20.9	--------	126.2	130.0	.0101	.016	.324	1.00
		B				11.1	24.4	--------	127.1	130.1	.0100	.016	.322	1.00
		B				10.9	24.4	--------	124.8	127.8	.0102	.016	.334	1.04
		B				11.4	26.1	--------	126.2	128.9	.0101	.016	.328	1.02
		B				12.6	30.5	--------	129.1	131.0	.0099	.015	.317	.98
		C				13.6	36.0	--------	128.2	129.2	.0100	.016	.323	1.00
		B				13.4	36.7	--------	125.1	126.0	.0101	.016	.339	1.05
		B				14.8	41.0	--------	130.7	131.1	.0098	.015	.314	.97
		B				14.5	41.0	--------	128.1	128.4	.0100	.016	.326	1.01
		B				15.8	46.1	--------	131.6	131.4	.0098	.015	.312	.97
		B				15.4	44.9	--------	130.0	129.9	.0098	.015	.319	.99
406	IEH, 1919, new	D	15.0	At Englewood dam, Ohio. Reach *shorter than Nos. 402 and 404*; composed of 20 lengths riveted soft steel and 26 lengths welded hard steel and 60 feet of standard wrought-iron pipe; tests confirm No. 402 rather than No. 404.	--------	8.3	30.9	--------	84.5	85.7	.0139	.036	.709	2.20
		D				9.4	30.9	--------	95.7	97.0	.0126	.028	.560	1.74
		B				16.0	61.4	--------	115.5	113.9	.0108	.019	.405	1.26
		B				17.3	72.8	--------	114.7	112.4	.0109	.020	.414	1.28
		B				17.4	72.8	--------	115.4	113.0	.0108	.019	.410	1.27
		B				17.6	72.8	--------	116.7	114.3	.0108	.019	.401	1.24
		B				17.8	72.8	--------	118.0	115.7	.0106	.018	.392	1.21
		B				18.5	72.8	--------	122.6	120.2	.0103	.017	.364	1.13
		B				19.0	72.8	--------	126.0	123.4	.0101	.016	.347	1.08
408	FBM, 1902	B	32.25	Dredge Epsilon on Mississippi River. Pipe lengths joined by rubber hose; loss of head by mercury manometers; velocities by Pitot tube.	--------	20.1	38.6	--------	124.8	113.6	.0116	.017	.383	1.19
		B				20.9	38.6	--------	129.8	118.1	.0112	.015	.355	1.10
		B				21.1	38.6	--------	131.0	119.2	.0111	.015	.349	1.08
		B				21.4	39.8	--------	130.9	118.9	.0111	.015	.350	1.08
		B				22.4	43.5	--------	131.0	118.7	.0111	.015	.351	1.09
410	FBM, 1902	C	31.88	Dredge Kappa; ball-and-socket joints. (See No. 408.)	--------	21.77	45.5	--------	125.2	113.4	.0115	.016	.382	1.18
412	FBM, 1902	B	32.0	Dredge Flad; ball-and-socket joints. (See No. 408.)	--------	17.0	38.2	--------	106.5	97.1	.0132	.023	.516	1.60
414	FBM, 1902	A	32.0	Dredge Zeta; joints and apparatus like No. 408.	--------	14.2	26.5	--------	106.8	98.8	.0132	.023	.504	1.56
		A				15.0	30.6	--------	105.0	96.6	.0134	.023	.525	1.63
		A				16.2	34.1	--------	107.4	98.3	.0131	.022	.505	1.56
		A				16.3	36.6	--------	104.3	95.2	.0135	.024	.536	1.66
		A				16.7	37.0	--------	106.3	97.0	.0132	.023	.517	1.60
416	FBM, 1902	B	32.69	Dredge Iota; joints and apparatus like No. 408.	--------	17.1	29.6	--------	120.5	110.6	.0120	.018	.405	1.26
		B				18.3	34.3	--------	119.7	109.3	.0120	.018	.412	1.28
		B				18.6	35.9	--------	119.0	108.4	.0121	.018	.418	1.30
418	FBM, 1902	A	33.25	Dredge Beta; joints and apparatus like No. 408 above.	--------	17.9	35.8	--------	113.7	103.3	0.0126	.020	.457	1.42
		A			--------	19.3	40.0	--------	115.9	104.9	.0124	.019	.443	1.37
		A				20.8	45.2	--------	117.5	105.9	.0122	.019	.434	1.34
		A				21.3	47.9	--------	117.0	105.1	.0123	.019	.440	1.36
		A				21.6	51.4	--------	114.5	102.8	.0125	.020	.460	1.42
420	FBM, 1902	A	34.0	Dredge Gamma. (See No. 408.) Same connections; drop in pressure uniform throughout pipe.	--------	13.7	23.4	--------	106.4	98.1	.0134	.023	.509	1.58
		B				14.4	26.4	--------	105.3	96.6	.0135	.023	.523	1.62
		A				15.3	28.3	--------	108.1	98.9	.0132	.022	.499	1.55
		A				15.9	30.2	--------	108.7	99.2	.0131	.022	.495	1.53
422	FBM, 1902	B	34.25	Dredge Delta; joints and apparatus like No. 408.	--------	16.7	29.8	--------	114.1	104.0	.0126	.020	.452	1.40

TABLE 3.—*Elements of experiments for the determination of friction losses in sheet-metal and plate-metal pipes, with retardation coefficients in various formulas*—Continued

SPIRAL RIVETED PIPE

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	Coefficients of retardation: C_w=Williams-Hazen	Coefficients of retardation: n=Kutter	Coefficients of retardation: f=Weisbach	Coefficients of retardation: Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Coefficients of retardation: Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cu. feet*	*Feet*	*Feet*	*°C.*						
502	S&G, 1904 new	B	4.084	Experimental line at Cornell University, New York. 4 20-foot lengths of asphalt-coated pipe, flange-jointed; water running with the laps; loss of head by differential water columns; quantity measured in calibrated basin; series made in fall of 1904, although results are very consistent this series given but a B rating because of the short length of line.	--------	1.89	4.2	4	100.0	130.1	0.0094	0.026	0.383	1.15
		B				2.20	5.6	3	100.8	129.7	.0093	.025	.382	1.14
		B				2.73	8.2	4	103.4	131.0	.0092	.024	.372	1.12
		B				3.35	12.2	3	104.0	129.7	.0091	.024	.375	1.12
		B				4.04	16.9	4	106.6	131.2	.0090	.023	.364	1.09
		B				4.44	20.3	3	106.9	130.6	.0090	.023	.365	1.09
		B				4.56	21.3	3	107.2	130.6	.0089	.022	.364	1.09
		B				4.90	24.2	4	108.0	131.0	.0089	.022	.361	1.08
		B				5.48	29.8	3	108.9	131.0	.0088	.022	.360	1.08
		B				5.86	33.4	4	110.0	131.7	.0088	.021	.354	1.06
		B				6.08	35.7	3	110.0	131.4	.0088	.021	.355	1.06
		B				6.40	39.2	4	110.9	131.9	.0087	.021	.352	1.06
		B				6.77	44.0	3	110.7	131.1	.0087	.021	.355	1.06
		B				7.45	52.2	3	111.8	131.5	.0087	.021	.351	1.05
		B				7.83	56.9	4	112.6	132.0	.0086	.020	.348	1.05
504	S&G, 1905 1 year.	C	4.084	Same pipe, 1 year later; water running with the laps; same apparatus as listed above; results show a very slight depreciation in capacity; line	--------	1.530	2.86	8	98.1	139.6	.0094	.027	.390	1.18
		B				2.170	5.52	8	100.2	128.9	.0094	.026	.387	1.17
		B				2.408	6.74	8	100.6	128.4	.0093	.025	.388	1.18
		B				2.540	7.51	8	100.5	127.8	.0094	.025	.391	1.18

		B		had been drained in winter of 1904-05; the flat-head rivets and thin shell smoothed off by asphalt coat gives good surface; series rated as B grade because of short reach.	3.027	10.27	8	102.4	128.6	.0092	.025	.383	1.16
		B			3.045	10.27	8	103.1	129.4	.0092	.024	.378	1.14
		B			3.691	14.65	8	104.6	129.4	.0091	.024	.373	1.13
		B			4.362	20.06	8	105.6	128.9	.0090	.023	.374	1.13
		B			4.930	25.2	8	106.5	129.0	.0090	.023	.372	1.13
		B			5.210	27.8	8	107.2	129.3	.0089	.022	.369	1.12
		B			5.634	32.3	8	107.5	128.9	.0089	.022	.369	1.12
		B			6.117	37.8	8	107.9	128.6	.0089	.022	.369	1.12
		B			6.337	40.2	8	108.4	128.8	.0089	.022	.368	1.11
		B			6.606	43.7	8	108.4	128.4	.0088	.022	.369	1.12
		B			6.879	46.9	8	109.0	128.7	.0088	.022	.367	1.11
		B			7.036	49.2	8	108.8	128.3	.0088	.022	.369	1.12
506	S&G, 1905, 1 year.	B	4.084	Same pipe as No. 502, tested in fall of 1905; only difference from No. 504 above was that water flowed against the laps. The small reduction in capacity is worth considering when abrasive material in the water dictates the same reversal of lapping in taper joint plate pipe; given B rating because of short reach; temperature (8° C.) assumed same as above.	2.318	6.62	8	97.7	124.8	.0095	.027	.409	1.24
		B			2.757	9.57	--------	96.7	121.7	.0098	.028	.425	1.29
		B			3.370	13.57	--------	99.2	123.2	.0094	.026	.412	1.25
		B			3.509	13.83	--------	102.3	125.9	.0092	.025	.389	1.18
		B			4.116	18.71	--------	103.2	126.5	.0092	.024	.387	1.17
		B			4.138	18.86	--------	103.3	126.6	.0092	.024	.387	1.17
		B			4.176	20.44	--------	101.0	123.7	.0093	.026	.413	1.25
		B			5.027	27.8	--------	103.4	124.7	.0092	.024	.395	1.20
		B			5.834	35.9	--------	105.6	126.1	.0090	.022	.384	1.16
		B			5.977	38.1	--------	105.0	125.1	.0091	.023	.389	1.18
		B			6.813	48.8	--------	105.8	124.8	.0095	.023	.389	1.18
		B			7.097	52.4	--------	106.3	125.1	.0090	.023	.386	1.17
		B			7.360	55.5	--------	107.2	125.7	.0089	.022	.382	1.16
508	S&G, 1904	C	5.962	Experimental pipe; contains 2 old 20-foot joints; apparatus same as for No. 502 above; water flowing with lap.	1.60	1.97	1	102.4	130.7	.0099	.025	.374	1.11
		C			2.72	5.12	1	108.8	132.7	.0096	.022	.355	1.05
		C			3.03	6.2	4	109.3	133.3	.0095	.022	.349	1.05
		C			5.91	21.0	4	115.0	134.5	.0091	.019	.333	1.00
		C			6.64	25.9	4	117.2	134.9	.0090	.019	.329	.99
510	S&G, 1905	B	5.962	Same pipe in every way as No. 512 below. Series with water flowing with the laps.	2.60	4.5	8	110.1	136.0	.0094	.021	.340	1.03
		B			3.46	7.7	8	112.0	135.4	.0093	.021	.338	1.02
		B			3.61	8.4	8	111.9	134.8	.0093	.021	.338	1.02
		B			4.25	11.4	8	113.0	134.5	.0093	.020	.330	1.02
		B			4.96	15.0	8	115.0	135.4	.0091	.020	.332	1.01
		B			5.50	18.3	8	115.5	134.8	.0091	.019	.332	1.01
		B			5.99	21.1	8	117.1	136.0	.0090	.019	.326	.99
		B			6.54	24.9	8	117.7	135.8	.0090	.019	.326	.99
512	S&G, 1905, new.	B	5.943	Experimental pipe, flathead rivets, asphalt dipped, similar to Nos. 502, 504, and 506 above; series with water running with the laps. Note that there is no apparent difference in capacity when compared with next series (No. 514); this is important when conditions mentioned in 506 above holds.	1.977	2.93	6	103.7	130.3	.0098	.024	.370	1.10
		B			2.616	4.90	6	106.1	130.6	.0097	.023	.364	1.09
		B			2.921	5.67	6	110.2	134.8	.0094	.021	.341	1.02
		B			4.178	11.50	6	110.6	131.6	.0094	.021	.352	1.05
		B			4.615	13.46	6	113.0	133.5	.0093	.020	.340	1.02
		B			4.841	14.72	6	113.3	133.5	.0092	.020	.340	1.02
		B			5.605	20.06	6	112.4	130.8	.0093	.020	.350	1.05
		B			6.364	25.50	6	113.2	130.4	.0093	.020	.350	1.05
		B			6.628	27.01	6	114.5	131.7	.0092	.020	.342	1.02
		B			6.662	27.21	6	114.7	131.8	.0092	.020	.342	1.02

TABLE 3.—*Elements of experiments for the determination of friction losses in sheet-metal and plate-metal pipes, with retardation coefficients in various formulas*—Continued

SPIRAL RIVETED PIPE—Continued

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q=Quantity per second	V=Mean velocity per second	H=Loss of head per 1,000 feet	Temperature of water	Coefficients of retardation: C=Chezy	C_w=Williams-Hazen	n=Kutter	f=Weisbach	Scobey: K_s=Viscosity neglected $H=K_s\frac{V^{1.9}}{D^{1.1}}$	Scobey: m=Viscosity considered $K_s=m\nu^{0.1}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Inches		*Cu. feet*	*Feet*	*Feet*	*°C.*						
514	S&G, 1905, new.	B	5.943	Same pipe as No. 512, series with water running against the laps; experiments made in the fall of 1905. Note that the capacity, if anything, is greater than when water is running with the laps. Compare with No. 512 above. Temperature of water estimated same as in No. 504 above.	--------	1.512	1.71	8	103.8	133.3	0.0008	0.024	0.360	1.09
		B				2.584	4.52	----------	109.2	134.8	.0095	.022	.344	1.04
		B				2.706	4.97	----------	109.0	134.1	.0095	.022	.346	1.05
		B				3.236	6.91	----------	110.6	134.2	.0094	.021	.343	1.04
		B				3.297	7.08	----------	111.3	135.0	.0094	.021	.338	1.02
		B				3.890	9.66	----------	112.4	134.6	.0093	.020	.338	1.02
		B				4.357	12.46	----------	110.8	131.4	.0094	.021	.351	1.06
		B				5.041	15.97	----------	113.3	133.0	.0092	.020	.341	1.03
		B				6.078	22.35	----------	115.5	133.8	.0091	.019	.334	1.01
		B				6.577	25.68	----------	116.6	134.3	.0090	.019	.330	1.00
		B				6.602	26.24	----------	115.7	133.2	.0091	.019	.336	1.02
		B												

CORRUGATED PIPE

Pipe No.	Experimenter, year, and age of pipe	Observation rating	d=Inside diameter	Name and description of pipe	Q	V	H	Temperature	C	C_w	n	f	K_s	m
600	DLY, 1917, new.	A	8.88	Experimental line, Arlington, Va. Nominal size, 8 inches; line straight.	0.454	1.056	2.000	----------	54.9	66.4	0.0171	0.685	1.293	4.01
		A			.554	1.288	3.000	----------	54.7	65.2	.0171	.086	1.337	4.15
		A			.720	1.673	5.000	----------	55.0	64.2	.0171	.085	1.350	4.18
602	FHT, 1912, ½ year.	--------	24	Outfall sewer, El Paso, Tex. Straight reach, 1,037.8 feet.	4.96	1.58	1.322	----------	61.5	66.6	.0190	.068	1.184	3.67
					5.02	1.60	1.436	----------	50.7	64.4	.0201	.072	1.259	3.90
602a	----------------	--------	24	Same pipe as No. 502. Reach includes curve and above straight section.	4.865	1.55	1.547	----------	55.7	59.9	.0213	.083	1.430	4.46
					4.94	1.57	1.558	----------	56.4	60.6	.0211	.081	1.413	4.38

TABLE 4.—*Summary of series of experiments upon sheet and plate-metal pipe, including individual pipe equations, average coefficients, and percentage comparison of observed to calculated capacity*

[To be considered as supplementary to Tables 1, 2, and 3]

FULL RIVETED PIPE

Pipe No.	Experimenter	Year of tests	Age of pipe	Sheet or plate thickness (nominal)	Class	Approximate maximum pressure head	Inside diameter	Length of reach tested	Range of velocities	Individual pipe equations $H=MV^n$ and values of M' in $H=M'V^{1.9}$	Average values, excluding D ratings			
											Williams-Hazen C_w	Scobey		
												K_s	m	Variation of observed from computed capacity
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
		Year	*Years*	*Inches*		*Feet*	*Inches*	*Feet*	*Feet per second*					*Per cent*
1	Scobey	1917	2.0	0.0312	1a	3	3.88	404.5	0.6–1.3	$H=1.560\ V^{1.96}$	122.2	0.445	1.40	−7.2
2	Darcy	1850	New.	----------	1a	----------	7.71	365.3	.6–19.7	$H=.579\ V^{1.79}$	140.5	.304	.94	+11.4
3	Scobey	1917	New.	.050	1a	4	8.0	264.5	1.2–1.5	.564	132.1	.361	1.13	+2.3
4	Hawgood	1915	New.	.078	1a	179	8.0	4,354.0	7.2	.659	117.6	.422	1.31	−5.4
5	do	1915	New.	.078	1a	283	10.0	9,415.0	4.7	.630	104.7	.516	1.60	−14.0
6	Scobey	1919	24.0	.141	1a	3	10.0	1,769.6	1.8–2.6	.525	117.8	.430	1.35	+12.5
7	do	1919	8.0	.078	1a	25	10.03	221.0	1.7–2.2	.572	108.8	.502	1.59	−8.0
8	do	1919	3.5	.078	1a	50	10.87	845.3	2.0–2.4	.669	98.3	.600	1.90	−20.2
9	H. Smith	1876	5.0	.044–.091	1a	----------	10.92	684–731.0	4.7–10.0	$H=.501\ V^{1.83}$	120.5	.387	1.20	+3.1
10	Darcy	1850	New.	----------	1a	----------	11.22	365.5	1.3–10.5	$H=.413\ V^{1.78}$	135.1	.321	1.01	+8.3
11	Scobey	1917	3.0	----------	1a	4	12.0	1,144.0	1.1–1.3	.583	100.8	.583	1.83	−19.1
12	H. Smith	1876	5.0	.044–.091	1a	----------	12.07	685–718.0	4.6–10.7	$H=.349\ V^{1.92}$	119.7	.386	1.20	+3.2
13	Hawks	1896	3.0	----------	1a	----------	14.0	70,700.0	.9	.539	96.3	.642	1.99	−22.4
13a	do	1899	6.0	----------	1a	----------	14.0	70,700.0	1.1	.463	103.8	.551	1.71	−13.8
14	Scobey	1919	10.0	.062	1a	125	14.55	717.7	1.7–2.6	.337	118.4	.416	1.32	+2.0
15	H. Smith	1873	5.0	.044–.091	1a	----------	14.76	684–720.0	4.4–12.1	$H=.313\ V^{1.86}$	124.0	.359	1.11	+7.2
16	Adams	1896	New.	.109–.141	1a	----------	15.96	16,416.4	4.6	.277	121.5	.378	1.17	+.3
17	Scobey	1919	3.0	.141	1a	125	16.0	3,694.3	2.6–3.6	$H=.228\ V^{1.97}$	130.0	.340	1.08	+6.9
18	H. Smith	1878–9	1.0	.078–.156	1a	----------	16.99	4,438.7	20.1	.221	126.8	.325	1.01	+9.2
19	Scobey	1919	4.0	.078	1a	25	17.85	1,091.4	.7–1.6	.195	142.2	.302	.95	+15.5
20	do	1919	4.0	.078	1a	20	17.85	1,532.9	2.0–5.5	$H=.263\ V^{1.79}$	126.2	.356	1.13	+5.5
21	Lippincott	1909	8.0	.141	1a	300	18.0	3,150.0	2.4–5.8	$H=.257\ V^{2.01}$	109.1	.464	1.44	−4.2
22	Scobey	1919	4.0	.078	1a	30	20.0	2,012.2	1.6–5.0	$H=.262\ V^{1.94}$	107.5	.478	1.51	−9.5
23	Rafter	1890	14.6	.188–.250	1b	----------	24.0	10,541.0	3.3	.366	81.6	.784	2.43	−17.2

TABLE 4.—*Summary of series of experiments upon sheet and plate-metal pipe, including individual pipe equations, average coefficients, and percentage comparison of observed to calculated capacity*—Continued

FULL RIVETED PIPE—Continued

Pipe No.	Experimenter	Year of tests	Age of pipe	Sheet or plate thickness (nominal)	Class	Approximate maximum pressure head	Inside diameter	Length of reach tested	Range of velocities	Individual pipe equations $H=MV^x$ and values of M' in $H=M'V^{1.9}$	Average values, excluding D ratings: Williams-Hazen C_w	Scobey: K_s	Scobey: m	Scobey: Variation of observed from computed capacity
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
		Year	*Years*	*Inches*		*Feet*	*Inches*	*Feet*	*Feet per second*					*Per cent*
23a	Rafter	1890	14.6	0.188	1b		24.0	1,901.0	3.3	0.302	78.7	0.830	2.60	−20.1
24	Kuichling	1891	15.0	.188	1b		24.0	3,327.0	3.4	.348	83.9	.744	2.30	−14.6
25	H. Smith	1873	5.0	.062	1a		25.8	1,193.8	12.6	.133	129.9	.300	.96	+16.0
26	do	1876	5.0		1a	887	29.17	12,798.0	10.8	.128	123.3	.340	1.05	+10.3
27	LeConte	1911	5.0		1a		30.0	5 miles.		.153	114.0	±.42		−1.4
28	do	1911	42.0	.125–.156	1a		30.0	7½ miles.		.161	110.0	±.44		+20.0
29	do	1911	47.0		1a		30.0	1 mile.			91.5	±.60		
30	I. W. Smith	1896	1.0	.203	1a	250	33.0	34,176.0	6.1	.114	123.4	.347	1.08	+5.8
31	Scobey	1919	4.0	.109	1a	30	34.0	2,035.0	3.1	.108	127.2	.339	1.08	+8.0
32	I. W. Smith	1896	1.0	.203	1a	250	35.0	30,829.0	5.4	.101	127.4	.327	1.01	+9.1
33	Scobey	1919	2.0	.141	1a	162	36.0	896.1	1.6–6.1	$H=.076\ V^{1.14}$	118.7	.376	1.18	+0.5
34	do	1917	2.0	.312	1b		36.0	5,330.2	1.8	.122	116.2	.400	1.22	+5.6
35	Herschel	1892	New.	.250	1b		36.0	25,000.0	.6–5.7	$H=.150\ V^{1.76}$	110.0	.451	1.40	−1.3
35a	do	1896	4.0	.250	1b		36.0	24,720.0	4.9	.139	105.8	.464	1.44	+.3
37	LeConte	1911	24.0	.156–.188	1a		36.0	114,400.0	3.4	.130	106.4	.467	1.45	+8.5
38	do	1911	24.0	.156	1a		36.0	6 miles.	3.5	.138	123.0	±.46		+9.3
39	Rafter	1890	17.0	.188	1b		36.0	50,819.0	1.5	.216	85.8	.725	2.24	−12.0
39a	Kuichling	1891	18.0	.188	1b		36.0	50,820.0	1.5	.202	89.1	.676	2.09	−8.1
39b	——— Rochester, N. Y.	Seq.		.188	1b		36.0	50,799.0	Various.	See figure 7.				−4.0
40	do	1896	2.0	.250–.375	1b		38.0	46,339.0	.6–1.3	.142	119.4	.401	1.24	+6.7
41	do	1897	3.0	.250–.375	1b		38.0	46,339.0	.6–1.2	.098	128.4	.340	1.08	+15.6
42	——— Rochester, N. Y.	Various.	Various.	.250–.375	1b		38.0	91,640.8	3.1–3.4	See figure 7.				+9.3
43	do	Various.	Various.	.250–.375	1b		38.0	45,000.0	3.3–3.8	See figure 7.				+4.2
44	do	Various.	Various.	.250–.375	1b		38.0	40,890.6	3.8–4.0	See figure 7.				+5.8
45	I. W. Smith	1896	1.0	.188–.375	1b	370	42.0	50,965.0	3.8	.097	117.1	.386	1.20	+8.0
45a	Randlett	1915	21.0	.188–.375	1b	370	42.0	52,789.0	3.6	.103	113.0	.411	1.27	+22.4

46	Herschel	1896	New.	.312	1b		42.0	5,574.0	1.8-4.3	.113	109.5	.460	1.43	−2.3
47	do	1896	New.	.250-.312	1b		42	Various.	1.4-1.8	.108	113.1	.428	1.33	+1.5
48	do	1896	New.	.250-.312	1b		42.0	81,139.0	2.1-5.0	$H=.116V^{1.88}$	107.8	.449	1.39	−1.0
49	Bayley	1918	19.0		1a		44.0	Various.	2.5	.109	110.8	.456	1.42	+5.5
50	LeConte	1911	11.0	.250	1b		44.0	19,100.0	2.4	.099	113.5	.415	1.28	+15.2
51	do	1911	27.0	.188-.203	1a		44.0	27,000.0	3.4	.125	97.9	±.52		+4.8
52	do	1911	27.0	.188-.203	1a		44.0	4.3 miles.	3.5	.135	88.5	.648	2.01	−6.6
53	Herschel	1892	New.	.375	1b		47.28	26,687.0	2.6	.104	105.6	.471	1.46	−3.5
53a	do	1893	1.0	.375	1b		47.28	26,687.0	2.6-2.8	.114	100.7	.514	1.60	−7.1
53b	do	1894	2.0	.375	1b		47.28	26,687.0	2.0-3.6	.106	104.4	.480	1.49	−2.9
53c	do	1896	4.0	.375	1b		47.28	26,687.0	3.0	.108	103.2	.488	1.51	−2.3
53d	do	1896	4.0	.375	1b		47.27	26,610.0	4.4-4.5	.118	97.3	.533	1.65	−6.7
54	do	1892	New.	.250-.375	1b		47.34	61,445.0	5.4	.098	106.7	0.444	1.38	−.5
54a	do	1896	4.2	.250-.375	1b		47.34	61,445.0	3.0-4.5	.107	103.2	.483	1.50	−1.8
54b	do	1896	4.0	.250-.375	1b		47.40	61,075.0	5.8	.112		.507	1.57	−4.2
54c	do	1896	4.0	.250-.375	1b		47.40	83,000.0	4.5	.111		.503	1.56	−3.8
55	do	1892	New.	.250	1b		47.40	33,356.0	2.6-3.4	.088	115.2	.398	1.23	+5.4
55a	do	1892	New.	.250-.375	1b		47.40	74,396.0	3.6	.092	111.8	.418	1.30	+2.7
56	do	1894	2.0	.250-.375	1b		47.40	80,800.0	3.6	.100	106.5	.453	1.40	+.1
56a	do	1896	4.0	.250-.375	1b		47.40	34,845.0	3.0-6.1	$H=.111V^{1.86}$	103.4	.473	1.47	−.7
56b	do	1893	1.0	.250-.375	1b		47.40	20,415.0	2.0-3.3	.088	116.2	.397	1.23	+8.0
56c	do	1894	2.0	.250-.375	1b		47.40	20,415.0	2.0	.085	118.3	.386	1.20	+8.9
57	do	1894	2.0	.250-.375	1b		47.40	39,760.0	3.6	.097	108.5	.440	1.36	+1.7
57a	Merriman	1907	15.4	.250-.375	1b		47.56	48,324.0	1.2-4.0	$H=.148V^{1.86}$	86.2	.650	2.01	−8.1
58	Herschel	1892-1896	4.0	.250-.375	1b		47.52	24,630.0	2.9-5.4	.092				
58c	Merriman	1907	15.6	.250-.375	1b		47.56	23,871.0	3.6	.132	82.4	.737	2.28	−13.9
59	Herschel	1896	.3	.250	1b		47.52	24,648.0	3.0-4.7	.108	102.3	.496	1.54	−6.1
59a	Merriman	1907	11.0	.250	1b		47.56	5 miles.	3.4-4.3	.148	86.6	670	2.07	−12.6
60	Lippincott	1909	5.0	.188	1a	65	52.0	3,666.0	2.8	.070	123.2	.350	1.08	+8.7
61	LeConte	1911	0.0	.281	1b		54.0	3.2 miles.			100.0	±.40		+5.1
62	Mills	1888	3.4		1b		59.1	200.0	1.1-2.8	.076	98.8	.535	1.65	−7.3
63	Scobey	1924	New.	.312	1b		62.0	1,123.0	1.9	.061	120.0	.370	1.15	+9.5
64	J. W. Smith	1903	.8	.312-.688	1c		72.0	49,294.0	1.1-3.3	$H=.055V^{2.03}$	110.7	.428	1.34	+6.4
64a	do	1904	2.6	.312-.688	1c		72.0	49,294.0	1.4-2.9	$H=.068V^{1.97}$	98.6	.524	1.61	−2.7
64b	do	1905	3.0	.312-.688	1c		72.0	49,294.0	1.5-3.1	$H=.078V^{1.95}$	94.0	.573	1.70	−7.0
64c	do	1905	5.3	.312-.688	1c		72.0	49,294.0	1.4-2.8	$H=.110V^{1.77}$	83.3	.718	2.22	−15.6
64d	do	1908	7.2	.312-.688	1c		72.0	49,294.0	1.8-2.9	$H=.102V^{1.74}$	88.4	.643	1.00	−9.2
64e	do	1909	9.0	.312-.688	1c		72.0	49,294.0	1.6-2.8	$H=.089V^{1.90}$	86.7	.666	2.07	−9.6
64f	do	1911	13.0	.312-.688	1c		72.0	49,294.0	2.1-3.0	$H=.092V^{1.94}$	84.7	.691	2.14	−8.6
65	Monroe	1916	New.	.375	1c		72.0	310.9	2.3-11.7	$H=.052V^{1.97}$	105.1	.435	1.35	+5.3
66	do	1916	New.	.312-.375	1c		72.0	583.4	2.3-11.7	$H=.048V^{1.94}$	114.1	.374	1.16	+8.0
67	do	1913	New.	.375	1c		72.0	423.9	2.1-3.0	.059	111.5	.425	1.31	+6.6
68	do	1913	New.	.250-.312	1b		72.0	1,489.6	2.5-3.0	.043	130.6	.308	.95	+20.6
69	Marx, Wing, Hoskins	1897	.2	.375-.688	1c	484	72.22	0.8 mile.	.5-3.8	$H=.074V^{1.75}$	105.6	.457	1.36	+2.6
69a	do	1899	2.1	.375-.688	1c	484	72.24		2.0-5.3	$H=.073V^{1.88}$	98.1	.519	1.61	−2.5
70	Mills	1903	22.0	.375	1b		77.5	150.0	1.2-3.3	.076	93.5	.577	1.79	+3.1
71	Doolittle	1923	10.0		1b	178	84.0	6,410.0	3.4-9.2	$H=.028V^{1.73}$	92.5	.527	1.63	−1.6
72	Monroe	1917	New.	.438	1b	150	84.0	744.7	3.4-8.3	$H=.045V^{1.97}$	106.1	.435	1.35	+0.6
73	do	1917	New.	.375	1b		84.0	768.0	3.4-8.3	$H=.040V^{1.95}$	112.0	.387	1.20	+7.0
74	do	1917	New.	.375-.438	1c		84.0	1,070.6	3.4-8.3	$H=.038V^{2.05}$	108.7	.417	1.29	+7.7
75	do	1917	New.	.375-.438	1b		84.0	2,683.3	3.4-8.3	$H=.041V^{2.00}$	109.1	.413	1.28	+3.4
76	Scobey	1919	2.0	.312-.438	1b	150	84.0	3,696.2	1.8-8.1	$H=.057V^{1.99}$	93.0	.555	1.72	−10.8

TABLE 4.—*Summary of series of experiments upon sheet and plate-metal pipe, including individual pipe equations, average coefficients, and percentage comparison of observed to calculated capacity*—Continued

FULL RIVETED PIPE—Continued

Pipe No.	Experimenter	Year of tests	Age of pipe	Sheet or plate thickness (nominal)	Class	Approximate maximum pressure head	Inside diameter	Length of reach tested	Range of velocities	Individual pipe equations $H=MV^s$ and values of M' in $H=M'V^{1.9}$	Average values, excluding D ratings: Williams-Hazen C_w	Average values, excluding D ratings: Scobey K_s	Average values, excluding D ratings: Scobey m	Average values, excluding D ratings: Scobey Variation of observed from computed capacity
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
		Year	*Years*	*Inches*		*Feet*	*Inches*	*Feet*	*Feet per second*					*Per cent*
77	Herschel	1886	5.0	0.500	1c	-------	103.4	152.9	0.5– 5.4	-------	109.2	0.436	1.35	+9.4
78	Proebstel	1925	.8	.312–.688	1b	-------	108.0	33,920.0	2.0– 7.0	$H=.042V^{1.16}$	113.8	.380	1.17	+8.8
79	Wilkins	1922	New.	.438	1c	-------	129.0	231.0	7.9–10.8	.024	110.8	.327	1.02	+22.4
80	Scobey	1919	7.0	.250–.500	1b	245	132.0	3,324.5	3.2– 3.5	.033	102.5	.460	1.45	+3.3
81	do	1917	3.0	.312	1b	25	168.0	2,824.0	3.7	.031	101.9	.569	1.77	−10.6

GIRTH-RIVETED PIPE

Pipe No.	Experimenter	Year of tests	Age of pipe	Sheet or plate thickness (nominal)	Class	Approximate maximum pressure head	Inside diameter	Length of reach tested	Range of velocities	Individual pipe equations	Williams-Hazen C_w	K_s	m	Variation
202	Freeman	1892	"Old"	-------	2	-------	4.04	51.5	0.9–22.9	$H=2.66\ V^{1.99}$	76.3	0.072	3.05	-------
204	do	1892	New.	-------	2	-------	4.12	-------	1.3–23.4	$H=1.06\ V^{1.90}$	137.5	.327	1.03	+1.2
206	do	1892	New.	-------	2	-------	5.12	-------	1.9–20.6	$H=.777V^{1.93}$	134.9	.327	1.03	+1.2
208	do	1892	New.	-------	2	-------	6.14	-------	1.0–16.9	$H=.675V^{1.59}$	138.3	.316	1.00	+2.9
210	do	1892	New.	-------	2	-------	8.05	-------	1.5–13.0	$H=.485V^{1.91}$	135.6	.320	1.00	+2.3
211	Bayles	1919	New.	.342	2	-------	8.94	15,240.0	2.0– 4.4	-------	136.7	.326	1.01	+2.2
212	DuMoulin	1915	6.0	.307–.500	2	1,700	10.0	25,650.0	4.0	.473	122.9	.387	1.20	−2.1
214	Reynolds	1910	22.0	-------	2	450	15.5	6,229.0	11.4	.194	146.8	.256	.79	+38.1
216	Scobey } same pipe, same time	1917	4.0	-------	2	-------	36.0	1,273.1	1.2– 4.6	$H=.143V^{2.05}$	95.7	.577	1.70	−21.9
216a	Lance }							1,274.9	1.2– 4.6	$H=.140V^{2.05}$	95.5	.581	1.80	−22.2
218	—— Montreal	1912	4.0	-------	2	-------	36.0	36,000.0	3.0– 3.7	.085	139.3	.284	.88	+13.4
220	—— Rochester, N. Y.	1918	New.	.25	2	-------	37.0	50,697.9	3.84	.112	117.1	.387	1.20	−6.6
220a	do	1919	1.0	-------	2	-------	37.0	50,697.9	3.76	.117	114.5	.404	1.25	−8.0

220b	do	1920	2.0	----------	2	-------	37.0	50,697.9	3.83	.112	117.6	.386	1.20	−4.9
220c	Matthews, Rochester, N. Y.	1926	8.0	----------	2	-------	37.0	50,697.9	3.53	.130	108.5	.449	1.39	−7.8
222	do	1920	2.0	----------	2	-------	37.0	42,127.3	4.06	.136	105.5	.470	1.45	−12.9
222a	do	1926	8.0	----------	2	-------	37.0	42,127.3	3.24	.151	100.4	.520	1.56	−13.4
224	Porter	1926	17.0	----------	2	-------	42.0	39,053.0	4.1	.098	116.3	.389	1.16	+8.7
225	Randlett	1922	11.0	.25	2	270	44.0	77,829.3	7.4	.068	135.0	.285	.88	+19.7
225a	do	1922	11.0	.25 −.566	2	160	52.0	41,290.0	5.3	.062	130.0	.309	.96	+14.7
226	do	1925	New.	----------	2	-------	58.0	8,702.4	6.24	.046	140.6	.262	.81	+14.7
226a	do	1925	New.	----------	2	-------	58.0	7,918.6	6.24	.052	132.5	.293	.91	+8.1
226b	do	1925	New.	----------	2	-------	58.0	30,964.4	6.24	.049	136.2	.278	.86	+11.2
226c	do	1925	New.	.316−.566	2	-------	58.0	47,585.4	6.24	.050	135.1	.283	.88	+10.1
227a	Scobey	1929	New.	.500	2	-------	65.0	118,380.3	3.86	.049	128.8	.314	.99	+3.0
227b	do	1929	New.	.375−.500	2–3	-------	65.0	53,999.6	3.86	.044	137.0	.270	.88	+7.9
227c	do	1929	New.	.375−.438	2	-------	65.0	117,021.8	3.86	.045	135.3	.286	.91	+8.2
228	Wilkins	1922	New.	.388−.75	2	-------	96.0	240.0	14.2–19.5	.036	113.9	.357	1.11	−2.5
230	do	1922	New.	.50 −.626	2	-------	108.0	515.0	11.0–15.5	.032	113.1	.363	1.12	−3.4

CONTINUOUS INTERIOR PIPE

302	Speller	1917	1.0	----------	3	-------	3.628	1,000.0	2.6–11.7	$H=1.017\ V^{1.93}$	147.0	0.290	0.93	+4.3
304	do	1917	1.0	----------	3	-------	5.72	1,000.0	1.1–10.0	$H=.724\ V^{1.87}$	142.1	.304	.98	+1.6
306	Schoder	1907	"Old."	----------	3	-------	6.09	99.3	1.6– 6.0	$H=.664\ V^{1.87}$	143.1	.303	.95	+2.1
308	Scobey	1919	3.0	0.155	3	-------	7.69	585.8	4.5– 6.5	$H=.744\ V^{1.79}$	124.7	.378	1.20	−6.1
310	Speller	1917	1.0	----------	3	-------	8.0	1,000.0	.3– 3.3	$H=.400\ V^{1.91}$	153.2	.260	.83	+10.8
311	Dubuis	1926	New.	----------	3	-------	14.0	27,888.0	8.3	.241	139.8	.287	.89	+8.1
312	Mosley	1926	1.5	.312−.562	3	1,129	[1]19.2	12,528.3	3.0– 8.0	----------	138.6	.290	.90	+6.4
313	Peters and Scobey	1927	1.0	.25	3	-------	26.0	18,400.0	2.9– 6.0	$H=.150\ V^{1.89}$	126.3	.342	1.06	−2.7
314	Palmer	1904	New.	.25 −.316	3	320+	30.0	12–22 miles.	1.9– 2.1	.127	127.0	.340	1.08	−4.4
316	Mills	1888	2.3	----------	3	-------	84.0	100.0	2.6– 3.5	$H=.040\ V^{1.95}$	118.2	.366	1.13	−5.0
316a	do	1890	13.0	----------	3	-------	84.0	100.0	2.1– 3.0	$H=.057\ V^{1.95}$	96.7	.532	1.65	−15.2
318	do	1888	2.3	----------	3	-------	84.0	100.0	2.7– 3.8	$H=.042\ V^{2.00}$	111.4	.406	1.26	−10.0
318a	do	1890	13.0		3		84.0	100.0	3.6– 3.7	.060	98.4	.507	1.55	−13.0

[1] Average.

COMPOUND PIPES

150	Tubbs. Riveted and continuous interior.	1876	New.	0.188–0.25	-------	-------	24.0– 36.0	102,469.0	1.9– 4.3	----------	-------	-------	1.05	-------
152	Wilkins. Riveted steel	1921	New.	.25–1.062	[1]1d	-------	30.0– 36.6	7,415.0	2.6– 8.0	$h_f=0.002322\ Q^{1.87}$	-------	0.680	2.11	−13.2
154	Scobey. Riveted steel	1923	14.0	----------	1b	-------	61.8– 71.0	5,462.4	9.7–12.9	----------	106.4	.423	1.31	+14.0
156	Doolittle. Welded [1]	1921	New.	.375− .428	[1]2	-------	60.0– 84.0	2,482.0	2.6–16.2	$h_f=.000098\ Q^{2.13}$	-------	.304	1.14	+6.1
156a	do	1921	New.	.375− .438	[1]2	-------	60.0– 84.0	2,482.0	2.6–15.1	$h_f=.000194\ Q^{1.99}$	-------	.315	.98	+4.1
158	Scobey. Riveted steel	1917	2.0	----------	1b	-------	84.0– 90.0	2,899.0	4.4– 6.1		121.0	.338	1.05	+16.8
160	Doolittle. Riveted steel	1921	New.	.375− .75	[1]1d	330	96.0–114.0	1,136.0	4.9–11.9	$h_f=.000035\ Q^{2.23}$	89.8	.573	1.77	−5.0
162	Kempkey. Welded steel	1926	New.	.188− .25	3	-------	22.0– 24.0	115,015.0	1.7– 3.2	----------	138.0	.303	.92	+2.9
164	Scobey. Welded, except riveted girth seams.	1929	New.	.500	2	284	54.0–65.0	133,815.0	2.8– 4.1	----------	120.4	.309	.98	+3.9

TABLE 4.—*Summary of series of experiments upon sheet and plate-metal pipe, including individual pipe equations, average coefficients, and percentage comparison of observed to calculated capacity*—Continued

DREDGE PIPES

Pipe No.	Experimenter	Year of tests	Age of pipe	Sheet or plate thickness (nominal)	Class	Approximate maximum pressure head	Inside diameter	Length of reach tested	Range of velocities	Individual pipe equations $H=MV^n$ and values of M' in $H=M'V^{1.9}$	Average values, excluding D ratings: Williams-Hazen C_w	Scobey: K_s	Scobey: m	Scobey: Variation of observed from computed capacity
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
		Year	*Years*	*Inches*		*Feet*	*Inches*	*Feet*	*Feet per second*					*Per cent*
402	Houk	1919	New.		1a		15.0	2,075.0	6.9–13.7	$H=0.380\ V^{1.83}$	114.4	0.412	1.28	−4.2
404	do	1919	New.		1a		15.0	1,340.0	7.7–15.8	$H=\ .412\ V^{1.85}$	130.0	.323	1.00	+8.9
406	do	1919	New.		1a		15.0	780.0	8.3–19.0		110.6	.390	1.21	−1.4
408	Maltby	1902			1b		32.25	235.0	20.1–22.4		117.7	.358	1.11	+11.4
410	do	1902			1b		31.88	187.0	21.8		113.4	.382	1.18	+7.7
412	do	1902			1b		32.0	450.0	17.0		97.1	.516	1.60	−8.0
414	do	1902			1b		32.0	962.0	14.2–16.7		97.2	.517	1.60	−8.1
416	do	1902			1b		32.69	460.0	17.1–18.6		109.4	.412	1.28	+3.5
418	do	1902			1b		33.25	650.0	17.9–21.0		104.4	.447	1.38	−0.8
420	do	1902			1b		34.0	600.0	13.7–15.9		98.2	.505	1.57	−7.0
422	do	1902			1b		34.25	625.0	16.7		104.0	.452	1.40	−1.4

SPIRAL RIVETED PIPE

502	Schoder and Gehring	1904	New.	0.0375	1a	....	[2] 4.084	80.06	1.9- 7.8	$H=1.314\ V^{1.82}$	131.0	0.363	1.09	+5.1
504	do	1905	1.0	.0375	1a	....	[2] 4.084	60.01	1.5- 7.0	$H=1.314\ V^{1.84}$	129.4	.376	1.14	+2.8
506	do	1905	1.0	.0375	1b	....	[3] 4.084	80.1	2.3- 7.4	$H=1.430\ V^{1.84}$	125.0	.396	1.20	-2.6
508	do	1904	New.	.05	1a	....	[2] 5.962	60.01	1.6- 6.6	$H=.833\ V^{1.81}$	133.2	.348	1.04	+3.0
510	do	1905	1.0	.05	1a	....	[2] 5.962	60.01	2.6- 6.5	$H=.784\ V^{1.84}$	135.3	.334	1.01	+5.5
512	do	1905	New.	.05	1a	....	[2] 5.943	60.16	2.0- 6.7	$H=.813\ V^{1.83}$	131.9	.349	1.04	+2.8
514	do	1905	New.	.05	1b	....	[3] 5.943	60.16	1.5- 6.6	$H=.772\ V^{1.87}$	133.8	.342	1.03	+12.6
520	Greve and Martin	1921	New.	.0625	1a	....	[2] 4.13	40.0	1.6- 9.6	$H=1.154\ V^{1.86}$	....	.337	1.04	+6.5
521	do	1921	New.	.0625	1b	....	[3] 4.13	40.0	1.8- 9.1	$H=1.377\ V^{1.81}$	....	.435	1.35	+0.6
522	do	1921	New.	.0625	1a	....	[2] 6.01	40.0	2.3-19.7	$H=.843\ V^{1.91}$	....	.402	1.25	-2.9
523	do	1921	New.	.0625	1b	....	[3] 6.01	40.0	2.3-20.2	$H=.875\ V^{1.95}$	....	.439	1.36	0.0
524	do	1921	New.	.0625	1a	....	[2] 6.01	40.0	2.6-20.0	$H=.873\ V^{1.90}$	....	.412	1.28	-4.2
525	do	1921	New.	.0625	1b	....	[3] 6.01	40.0	2.3-16.6	$H=.979\ V^{1.87}$	....	.436	1.35	+0.5
526	do	1921	New.	.0625	1a	....	[2] 6.01	40.0	2.5-17.9	$H=.828\ V^{1.92}$	....	.409	1.27	-3.8
527	do	1921	New.	.0625	1b	....	[3] 6.01	40.0	2.4-17.7	$H=.902\ V^{1.94}$	....	.455	1.41	-1.7
528	do	1921	New.	.0625	1a	....	[2] 6.01	40.0	3.3-19.6	$H=.786\ V^{1.97}$	....	.436	1.35	-7.0
529	do	1921	New.	.0625	1b	....	[3] 6.01	40.0	2.6-17.0	$H=.875\ V^{1.94}$	....	.451	1.40	-0.1
530	do	1921	New.	.0625	1a	....	[2] 8.12	40.17	2.4-11.2	$H=.669\ V^{1.92}$	....	.461	1.43	-9.7
531	do	1921	New.	.0625	1b	....	[3] 8.12	40.17	1.8-10.9	$H=.739\ V^{1.96}$	....	.480	1.49	-4.5
532	do	1921	New.	.0625	1a	....	[2] 10.21	40.0	1.3- 5.8	$H=.554\ V^{1.83}$	....	.410	1.30	-5.0
533	do	1921	New.	.0625	1b	....	[3] 10.21	40.0	1.3- 6.0	$H=.597\ V^{1.90}$	....	.442	1.37	0.0

CORRUGATED PIPE

600	Yarnell	1917	New.	......		0.0	8.88	200.0	1.1-1.7	1.85	65.3	1.327	4.11	+3.6
602	Todd	1912	New.	0.0625			24.0	1,037.85	1.6	.571	65.5	1.221	3.78	+12.1
602a	do	1912	New.	.0625			24.0	2,807.75	1.6	.666	60.2	1.426	4.42	-1.0

[1] Classed according to type of predominating pipe.
[2] Flow with the laps.
[3] Flow against the laps.

DESCRIPTION OF PIPES

The descriptions in the following pages supplement the information given in Tables 1, 2, 3, and 4. Pipes upon which previous experimenters have made observations are described in Appendix 1.

Items such as "Capacity + 5.2 per cent" will be found after the descriptive headings. This means that the capacity of the pipe, based on the average observed coefficients for observations rated A, B, or C, is 5.2 per cent more than that required to satisfy the writer's formulas 9 to 11, inclusive (page 10), where the coefficient K_s is chosen for the type and age of pipe as given in the description. The temperature of the water was assumed at 15° C. when it was not given. If the temperature of water was given, then the capacity comparison is modified in accordance with column 8 of Table 6, which gives the percentage difference in velocity—hence in capacity—for temperatures differing from 15° C.

The pipes are placed in the following order:

FULL-RIVETED PIPES

No. 1.—Experiment S–111, 4-inch slip joint iron pipe—Lateral LJ 3–2, Okanogan project, Washington, United States Bureau of Reclamation—Capacity −7.2 per cent

Experiments were made on six pipes on this project, ranging from 4 to 12 inches in diameter. These pipes were constructed identically and will be described under the same number. They were of the stovepipe type, made of thin black iron sheets (22 to 18 gauge), shop-seamed in lengths of 8 feet and 9 feet 7 inches. Individual sheets were 2½ feet long, when rolled. Joints were formed by slipping the large end of each pipe unit, previously heated and dipped in asphaltum, over the smaller end of the laid section. The asphaltum and contraction by cooling combined, made tight joints under moderate pressures.

All the tests on this project show low carrying capacity for a pipe which when new and clean should have a reasonably high capacity. The low capacity was caused largely by sand and silt inside the pipe. The deposits came from contour-skirting main canals which fed the pipe laterals leading down the slopes and across the flats. All velocities were determined by direct observation with color. Any filling of the pipe section would not introduce an erroneous value of V as would be the case if Q were measured and V determined by the equation $V=\frac{Q}{A}$. The presence of sand results in values of K_s above the proper ones, as the observed loss of head results from a smaller conduit area than that of the original pipe.

The pipe, tested as series S 111, is a straight reach across a low sag in the topography. The maximum pressure head is less than 4 feet. Gauge No. 1 was a water column in a graduated glass attached to a piezometer tube thrust 9.1 feet down the pipe from the inlet end, while gauge No. 2 was a similar connection held 1.1 feet upstream from the outlet end. Between these piezometers was a reach of 494.5 feet. Velocities were determined by color shot into the inlet end and observed in the box at the outlet end of the pipe. The average values of $K_s = 0.445$ or $C_w = 122.2$ show the possibilities of this thin-sheet pipe as a hydraulic conveyor when free of silt or other obstruction.

No. 3.—Experiment S-110, 8-inch galvanized slip-joint pipe—Private lateral of Benjamin Dougal, near Omak, Wash.—Capacity +2.3 per cent

This pipe had been in service only two weeks. The horizontal alignment was straight but there was a sag in the profile, making an inverted siphon over a shallow depression. The joint units, riveted longitudinally with flathead rivets in 10-foot lengths, were slipped together in a hot asphalt bath. As the maximum pressure head was only about 4 feet this joint was water-tight, at least while new. The pipe was smooth on the interior except for the ends of 15 service hydrants located on its top at regular intervals.

Piezometer No. 1 was placed 3 feet from the intake end and No. 2, similar to No. 1, was held 0.6 foot upstream from the outlet. A graduated glass was used as gauge No. 1 and a hook gauge in a stilling box as gauge No. 2. All service hydrants were closed during the tests. Velocities were determined by timing a puff of permanganate solution, shot from a color gun into the pipe just below the inlet and observed at the outlet. The results of the four tests were quite consistent and indicate a value of K_s of 0.361 for new pipe of this type. The capacity of this pipe was 2.3 per cent more than is suggested for class 1a. The difference in capacity is too small to indicate a special rating for galvanizing over the usual asphaltum coat, at least for a new pipe.

No. 6. Experiment S-127, 10-inch lap-riveted pipe—California Domestic Water Co., operated by Whittier Water Co., Whittier Calif.—Capacity +12.5 per cent

About 1895 several miles of steel pipe were laid near Whittier. This pipe in 10-inch size, was made of sheets 30 by 32 inches and nine sixty-fourths inch thick. The sheets were lap-riveted with double rows of flat-buttoned rivets in the longitudinal seams and a single row in girth seams.

Before being assembled in 20-foot sections in the field they were double dipped in hot asphalt and after the 24 years of service attributed to them by the superintendent (1925), appeared to be in fair condition.

A straight reach of this pipe, 1,769 feet long, was selected for experiment. Piezometer tubes were thrust through $\frac{3}{16}$-inch holes at both ends of the reach. Pressure tubing from these piezometers led to graduated glass manometers—a mercury U-tube at gauge No. 1 and a water column at gauge No. 2. Velocities were determined by timing color solutions injected into the pipe through a $\frac{3}{16}$-inch hole a short distance below the holes for gauge No. 1. The color was observed in a standpipe near the upper end of a 6-inch lateral pipe into which the 10-inch pipe under test was discharging. Proper correction for difference in velocities in the pipes of different sizes was computed for the 15 feet traversed in the 6-inch line.

No. 7.—Experiment S–80, 10.63-inch riveted sheet-steel pipe—Shobert ditch siphon, Eldorado Ditch Co., Placerville, Calif.—Capacity –8 per cent

Water for irrigation was conveyed across a gulch in a siphon pipe installed about one year prior to the test, but the pipe had seen previous duty for seven years in mine service. It was of 14-gauge sheet metal, double lap-riveted in both longitudinal and girth seams. At the time of test all the inside coating had been worn off and the interior showed rust blisters. The line was buried about 2 feet with the low part of the siphon under a pressure head of about 25 feet. The inside diameters were measured at the inlet and outlet structures. Gauge No. 1 was a water column attached to a piezometer tube set in the pipe 15 feet from the inlet. Gauge No. 2 was a similar connection located 1 foot above the outlet. Velocities were determined by color, timed from inlet to outlet.

No. 8.—Experiment S–119, 10.87-inch lap-riveted steel pipe—City Garden siphon, Eldorado County Water Users Association, Calif.—Capacity –20.2 per cent

Water for irrigation purposes was conveyed between open ditches, across the sag of a saddle between two hills, in an inverted-siphon pipe 886.3 feet long, under a maximum pressure head of about 50 feet. This pipe had just been relaid at the time of test, about half its length being salvaged from pipe that had been in use some seven years in a mine, the other half being new pipe. It was all newly dipped in asphalt. The steel was of 14-gauge, cylinder-jointed with flat-head rivets—a single row at girth joints and a double row longitudinally. Loss of head was determined by piezometers and water columns in graduated gauge glasses. Gauge No. 1 was attached to a manifold into which two pressure tubes brought the average pressure from eight small holes in two piezometers, set neutral to the current through 3/16-inch holes bored through the pipe at positions on its circumference corresponding to 10 and 2 on a clock dial. Piezometer No. 2 was similar to No. 1 except that a single tube with four holes was used instead of two tubes. Gauge No. 1 was placed 39.4 feet from the intake end. Gauge No. 2 was 1.6 feet above the outlet and 845.3 feet from No. 1.

Velocities were determined by means of color shot into the pipe with the color gun through a 3/8-inch hole 5 feet nearer the inlet than tap No. 1. The color was observed in the outlet just beyond tap No. 2.

It was not feasible to vary the flow through a considerable range of velocities. All runs were taken with V between 2 and 3 feet per second. The capacity is much less than might be expected, probably because of a silted condition produced by the upper ditch.

No. 11.—Experiment S–108, 12-inch slip-joint iron pipe—Lateral U. C. 9, Okanogan project, Wash., United States Bureau of Reclamation—Capacity –19.1 per cent

For general description of pipes on this project see pipe No. 1, page 64. The U. C. 9 lateral extended through flat, cultivated lands, about 4 feet below the surface, and terminated in open boxes which gave access to it. The line consisted of three straight sections joined by two curved bends. In the intake box there was a Cipolletti weir, but measurements could not be made with it as it was completely submerged. These observations may be taken as indicating the maximum capacity of this line at the time of test. Velocities were determined by means of color injected under pressure at the intake end and observed at the outlet.

Loss of head was determined by use of piezometer tubes connected to graduated gauge glasses. Gauge No. 1 was dropped 6 feet down the intake end, and gauge No. 2 was thrust back into the pipe 1 foot above the outlet and 1,144 feet from gauge No. 1. Three observations were taken, at about the same velocity, the resulting value of K_s averaging 0.583. The ditch which brings water to this line could very well contribute silt to clog the pipe partially and cause the low capacity.

No. 14.—Experiment S-118, 14½-inch lap-riveted steel pipe—Carlisle siphon, Eldorado County Water Users Association, California.—Capacity +2 per cent

Water for irrigation purposes was conveyed across a gulch between open ditches in a steel siphon pipe 843 feet long and under a maximum pressure head of about 125 feet. This line was rebuilt in 1917–18 and tested in 1919. New pipe of 16-gauge metal, to the extent of 260 lineal feet was added to 583 feet picked over and salvaged from the 14-year-old line. The pipe was lap-riveted, with cylinder joints, of 16-gauge sheets. Both longitudinal and girth rivets were in double rows.

Taps for gauge No. 1 were placed 100 feet down from the inlet end. Two 5/16-inch holes were bored in the pipe symmetrically so that the two piezometer tubes pointed like the branches of the letter V. Each tube contained four pressure holes neutral to the current. A third hole, of the same size as the piezometer holes, was bored through the top of the pipe 8 feet nearer the inlet. Color shots were started through this hole, the nozzle of the color gun being thrust directly into the water prism. Piezometer No. 2 was thrust 1 foot upstream from a small leak hole in the top of the siphon 25.1 feet above the outlet. The gauges at both ends of the reach of 717.7 feet were water columns showing in graduated glass tubes. Velocities were determined by color timed from near gauge No. 1 to the outlet in an open ditch.

No. 17.—Experiment S-125, 16-inch riveted steel pipe. Lateral No. 20 from Chatsworth high line, Los Angeles water supply, California.—Capacity +6.9 per cent

The Chatsworth high-line canal skirts along a contour on the north side of San Fernando Valley, in southern California. From this canal steel pipe lines of various sizes convey irrigation water down the relatively steep slopes to the floor of the valley. Lateral No. 20 is typical of these pipes. It was rolled of 10-gauge steel, in shop-riveted units mostly 20 feet long. Both longitudinal and girth seams are in double rows of flathead rivets. The interior heads are hardly noticeable when buried in a coat of asphalt dip. This is classed as cylinder-joint pipe but for relatively thin sheets, any shoulder offset is nicely tapered by the flow of the asphalt. This is reflected in the high capacity. Experiments were made when the pipe was 3 years old.

This line gave excellent opportunity for reliable tests largely on account of the long reach—3,694.3 feet. A few gentle curves occurred in the plan view. In profile the line was on continuous down grade. Piezometers, neutral to the flow, were thrust through 5/16-inch holes bored in the pipe line. Pressure tubing connected the piezometers with mercury manometers of the U-tube form. Velocities were determined by timing color injected through a hole 1 foot from piezometer No. 1 and observed in a white-lined pan by withdrawing water at gauge No. 2. The whole line under test being on down grade, there was no obstruction caused by accumulated sand. The results of the series of tests show great consistency. The writer assigns great weight to this series in contemplation of all available data. The gentle curves and continuous down grade are elements that would tend to produce an excess of capacity.

No. 19.—Experiment S–120, 17.85-inch riveted-steel pipe—Orange Blossom siphon, Oakdale irrigation district, California.—Capacity +15.5 per cent

Water for irrigation is conveyed across the Stanislaus River, between open canals, in a siphon pipe of riveted steel. The total length of this pipe is 1,192.8 feet and the maximum pressure head is about 25 feet. The description of material, riveting, and dipping given for pipe No. 31 (page 69) applies to this line also. At the time of the test sufficient water was not available to run the line to capacity, and gauge No. 1 was placed 98 feet down a steep slope from the intake end, at a point about 6 feet lower in elevation than the water in the outlet ditch. This gave assurance that the siphon would be full and under pressure at piezometer No. 1 for all flows that might be available.

Piezometers for gauge No. 2 were thrust upstream into the pipe 3.2 feet from the outlet end. Both gauges were of the water-column type, in graduated glass tubes. Velocities were determined by injecting potassium permanganate color solution, into the line at gauge No. 1 and observing the color where the pipe discharged into the open canal. Five observations were made on this line, the maximum velocity possible being only 1.6 feet per second. For this velocity the total loss of head was but 0.479 foot for the test reach of 1,091.4 feet. As this and all similar losses for lesser velocities may well come within an appreciable percentage of experimental error, too much faith is not to be placed in the retardation coefficients. Although the values are discounted they can be considered as corroborating the values found for pipes Nos. 20 and 31, in this same system.

No. 20.—Experiment S–122, 17.85-inch riveted-steel pipe—Birnbaum pipe line Oakdale irrigation district, California.—Capacity +5.5 per cent

About 1 mile west of Oakdale, Calif., irrigation water is conveyed from one open canal to another at a materially lower level by means of a compound steel pipe. The first reach was 34 inches in diameter, 2,600 feet long. At the lower end a T connection led to a straight continuation of the line by a 20-inch pipe (No. 22) and a line at right angles, 18 inches in diameter (No. 20) on which this test was made. This line is perfectly straight in both plan and profile. The outlet is a foot or so higher than the intake end. A pressure head of about 20 feet is available at the intake. Inside diameters were determined from measurements of pipe sections remaining in the pipe yard. Color was timed from a point near the intake, where it was shot with a pressure gun, to the outlet of the pipe line in an open ditch. The reach was 1,532.9 feet long between piezometer holes. A mercury U-tube was used as gauge No. 1, while a water column was sufficient at gauge No. 2. Conditions for experimentation on this line were excellent and the results are particularly satisfactory for use in determining the coefficients for suggested formulas.

No. 22.—Experiment S–121, 20-inch riveted-steel pipe—Birnbaum pipe line, Oakdale irrigation district, California—Capacity —9.5 per cent

This reach was part of the line described as No. 20 and No. 31. It was perfectly straight and as previously stated was a continuation of the 34-inch pipe. There was practically no slope of the ground surface and the outlet of the pipe required a slight rise into a concrete pool of an open canal. These conditions would contribute to the accumulation of any sand that came out of the upper canal and probably account for the low capacity. It was not feasible to withdraw the water from the line and examine the inside. Eleven observations of velocities ranging from about 1.6 to 5 feet per second showed very consistent results when plotted on logarithmic paper. The computations were made using the observed velocity and loss of head and a pipe size which was assumed to be the same as the average of measured samples in the yard. If this line was obstructed with sand so as to make its effective size that of a 16-inch pipe, the values of the coefficients are consistent and near those found on the other two parts of the Birnbaum pipe line (Nos. 20 and 31).

No. 31.—Experiment S–123, 34-inch riveted-steel pipe—Birnbaum pipe line, Oakdale irrigation district, California—Capacity +8 per cent

This test of this pipe was made on a reach 2,035 feet long, in two straight sections joined by a curve through about 45°. Piezometers and gauges at both ends of the reach were identical. Holes 3/16-inch in diameter were drilled through the pipe at points corresponding closely to 10 and 2 on a clock dial. Piezometer tubes, each with four openings, neutral to the current, were thrust into the water prism. Pressure tubes led from these piezometers to mercury gauges of U-tube form. A line of levels and another of check levels determined the elevations of the gauge zeros in the graduated glasses. Velocities were determined by timing color, injected through one of the holes at gauge No. 1 and observed in a white-lined pan into which water was drawn at gauge No. 2. The beginning of this pipe extends down a moderate slope from the canal to the flat. After a few hundred feet the slope is quite gentle but enough to carry sand into the 20-inch pipe. Two inside diameters were measured in a sample pipe remaining from construction.

This pipe line was 4 years old at time of test. It was made of 12-gauge metal, joined with flathead rivets—a double staggered row longitudinally and a single row roundabout. When double dipped in a thick coat of asphalt, flat rivet heads are quite well masked. There is little obstruction at the lapping of adjoining plates, when of thin metal. The values of $K_s = 0.339$ and of $C_w = 127.2$ agree quite closely with the values found on the 18-inch pipe of similar construction leading off from the lower end of this pipe. (No. 20.)

The topography is such that all sand or silt flowing into the pipe from the canal above would accumulate in the flat-grade outlet pipe; that is, in the 20-inch pipe described as No. 22. The retardation factors found show this to have been the condition.

No. 33.—Experiment S-124, 36-inch riveted-steel slip-joint pipe—Lindsay-Strathmore irrigation district, California—Capacity +0.5 per cent

Water for irrigation in the Lindsay-Strathmore irrigation district, California is pumped up a steep slope from the low-line canal to a high-line flume through a 36-inch pipe of 10-gauge steel, seamed with taper joints and flat-head rivets. The outside is wrapped with asphalted paper. The length of this pipe is about 1,200 feet from pump to outlet. Tests were made for five runs of water at velocities ranging from 1.6 to 6.1 feet per second. Piezometer No. 1, was thrust into the pipe through a 3/16-inch hole located about 300 feet from the Venturi meter at the lower end of the line. At this point the pressure head was 162 feet, yet the small brass piezometer tube made a tight connection with the pipe shell, being held by taper and friction alone. The pressure head was observed in a mercury manometer of U-tube type. A second hole near by was used for the injection of color by inserting the nozzle of the color gun and pumping up the air reservoir until a pressure of 100 pounds was indicated by test with an automobile-tire gauge. This pressure in the gun was enough more than that in the pipe line to shoot the color into the flowing water prism without difficulty. Piezometer No. 2 was exactly like No. 1 and was inserted through the shell of the pipe 8 feet above the outlet in a concrete flume. The color was observed as it poured out from the pump line. The view in Plate 2, A shows the condition at this outlet eight years after the tests described. Gauge No. 2 was a water column in graduated glass. The velocity of the water as determined by shots of Congo red, was used in computing the results. Readings of the Venturi meter indicated slightly higher velocities than those indicated by the color. For most runs this difference was less than 2 per cent, but in one it was 4 per cent.

These tests were made in 1919 when the pipe was 1 year old. The average values of $K_s = 0.376$ and $C_w = 118.7$ for the four highest velocities probably reflect, to a reasonable degree, the true condition of this pipe. The long mercury column—more than 10 feet—used for gauge No. 1 required material correction for air temperatures where the latter varied from 31° to 35° C. (From 88° to 95° F.) For the lowest velocity in the pipe line the loss of head as determined from the top of the computed equivalent water column at gauge No. 1 to the top of the actual water column at gauge No. 2—less than 0.2 foot—might well come within the zone of influence usually attributable to experimental errors.

No. 34.—Experiment S-112, 36-inch lap-riveted plate steel pipe—Sooke Lake water supply, Victoria, British Columbia—Capacity +5.6 per cent

Water is conveyed to the city of Victoria from Humpback reservoir, by a steel pipe 56,667 feet long, of lap-riveted plates five-sixteenths-inch thick for 20,137 feet and three-eighths-inch plate for the balance. The plate was made by the open-hearth process. The rivets are of extra-soft steel. The pipe units are of 4-plate lengths, giving sections about 22 feet long. Each unit was dipped in hot asphaltic compound.

In 1917, a test was conducted on a reach of the three-eighths-inch plate pipe 5,330.2 feet long and near the upper end of the line. In profile this reach was quite sinuous, having seven sags with drain valves and seven summits with air valves. In plan view there were four curves aggregating 152°. Piezometer tubes for gauge No. 1 (four holes in each) were thrust into the pipe through three-sixteenths-inch holes 890 feet from the dam at positions corresponding to 10 and 2 on a clock dial. Piezometer No. 2, at the end of a long straight reach, had only four holes in a single tube. Both piezometers were attached to mercury manometers of the U-tube type. Quantity was determined by readings at 5-minute intervals on the Venturi meter at the pipe inlet from the reservoir.

No. 63.—Experiment S-134, 62-inch cylinder-jointed lap-riveted steel pipe—Gage Canal, Riverside, Calif—Capacity +9.5 per cent

In 1924, wooden box flume No. 9, conveying irrigation water was replaced with a 62-inch inverted-siphon pipe of five-sixteenths-inch steel plate wrapped with a protective coating and brush painted on the inside, after erection, with two coats of asphalt. The same year, a single test for loss of head was taken from inlet to outlet pools. The difference in elevation between the two pools was determined by a double line of levels and the velocity was determined by timing a shot of color (potassium permanganate) throughout the 1,123 feet of length. This test indicated the pipe to be in excellent condition.

No. 76.—Experiment S-129, 84-inch riveted-steel pipe—Penstock, Wise power plant, Pacific Gas & Electric Co., near Auburn, Calif.

Water for power and later for irrigation serves the Wise plant through a conduit of various types in the following sequence:

(a) 1,362.5 feet of 96-inch wood-stave pipe.
(b) 435.6 feet of tunnel.
(c) 925.3 feet of 84-inch lap riveted steel five-sixteenths-inch plate. (Includes a Venturi meter.)
(d) 180 feet of 84-inch lap-riveted steel three-eighths-inch plate.
(e) 786 feet of 84-inch butt-strap steel three-eighths inch plate.
(f) 878 feet of 84-inch lap-riveted steel three-eighths-inch plate.
(g) 788 feet of 84-inch lap-riveted steel seven-sixteenths-inch plate.
(h) 750 feet of 84-inch lap-riveted steel three-eighths-inch plate.
(i) 204 feet of 84-inch butt-strap steel three-eighths-inch plate.
(j) 822 feet of 84-inch butt-strap steel seven-sixteenths-inch plate.

The above sections were followed by tapering sections aggregating 1,361.5 feet. All the lap-riveted pipe was made with cylinder joints. Soon after completion of the conduit in 1917, tests were made by the company on reaches approximating sections *g* (No. 72, p. 112); *h*, (No. 73, p. 112); and *i* and *j* combined (No. 74, p. 112). The retardation factors for these tests may be taken as holding for a new pipe.

Two years later the writer conducted a series of tests on a reach of pipe 3,696.2 feet long, extending from the Venturi meter over sections *c*, *d*, *e*, *f*, *g*, and part of *h*. In plan view the reach is quite straight but it contains 10 major vertical angles, with 10 air valves and 1 blow-off. Difference in elevation between gauge points was determined by a checked line of levels. Mercury U-tube manometers were attached to piezometers thrust through the pipe into the water prism. Quantity was measured by the Venturi meter. From these data the loss in head was determined.

The original coating of this penstock was a graphite paint, applied with brush. Nodular tubercles quickly formed, each with a pit underneath. In comparing the two tests, values for the combined reaches *g*, *h*, *i*, and *j* listed as No. 75, are used. The 1919 tests indicate that the line had deteriorated about 14 per cent in two years. This decrease became so marked in 1923 that the company cleaned the interior of the penstock with a sand blast and painted it with two coats of red lead (*21*).

At the age of 2 years the capacity of this pipe had decreased from an excess capacity of 3.4 per cent, as noted for No. 75 above, to a deficiency.

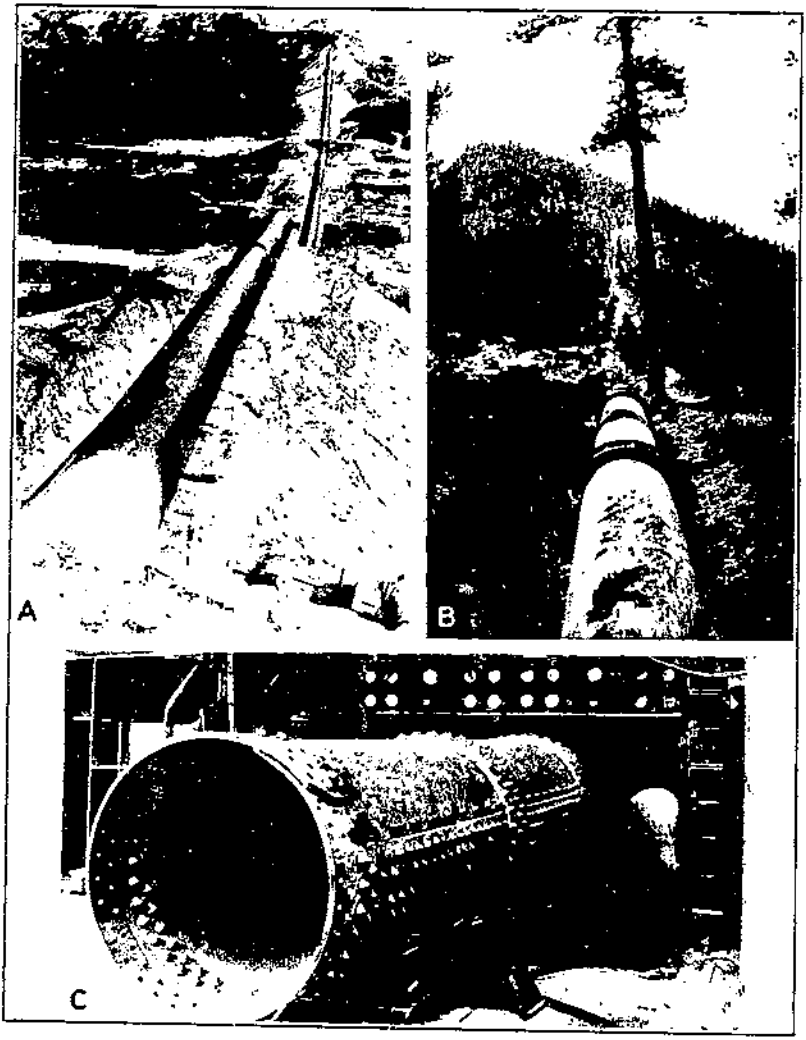
No. 80.—Experiment S-128, 132-inch riveted-steel pipe, cylinder joints—Deadman siphon, Los Angeles Aqueduct, California—Capacity +3.3 per cent

On the Los Angeles Aqueduct there are 13 inverted siphons of riveted steel and 8 of concrete. These siphons range in diameter from 7½, to 11 feet. Experiments for loss of head were made on Deadman siphon, near Saugus, in 1919. It was then about 7 years old. Water is conveyed for irrigation, power, and municipal use.

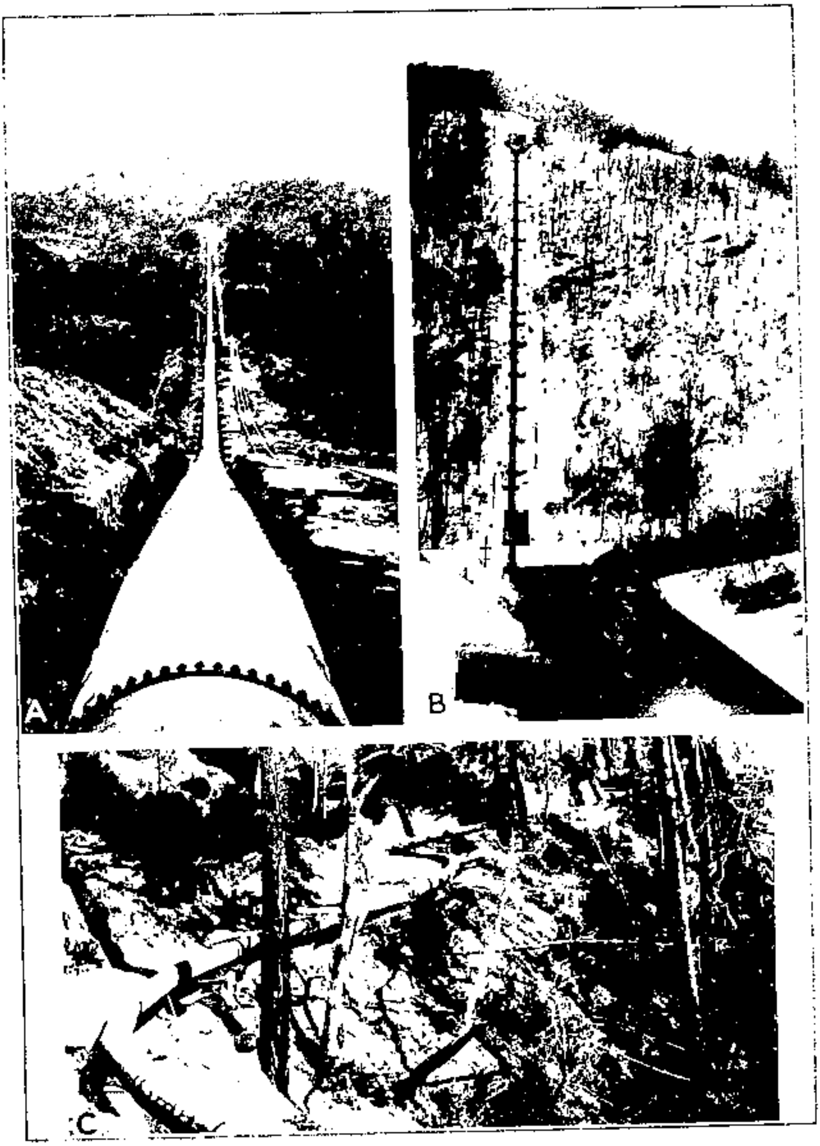
The total length of the siphon is 3,430 feet. The steel portion is 11 feet in diameter, of plates from one-fourth to one-half inch thick, with girth joints single riveted and longitudinal joints double riveted. The pipe was constructed with in-and-out or cylinder courses. The rivet heads are round and prominent. According to the final report on the aqueduct (*112, p. 220*), all the steel work was painted with hydrocarbon oil, residual from the manufacture of gas from California asphalt oil. This coating penetrates rust and rust scales on the metal. It was applied with brushes, without dilution in warm weather, but heated and diluted with distillate in cold weather or when put on cold plates.

The initial reach of this pipe is concrete. About 20 feet below the transition (in foreground, pl. 3, A) between the concrete and steel, holes three-sixteenths inch in diameter were drilled through the steel shell at positions corresponding to 10 and 4 on a clock dial. Through these holes piezometers were thrust and connected by a manifold to gauge No. 1, a mercury column of U-tube form. Near the outlet a similar layout of piezometers led to a water column. Practically all the steel portion of this siphon was included in the test reach, a distance of 3,324.5 feet. (Pl. 3, A.) The reach was straight in plan view but included seven vertical angles, each of which was less than 20°.

During the tests changes were made in the flow, for the purpose of producing several velocities, but the final range of velocities was not wide. Velocities were determined by timing color, shot from a pressure gun into the pipe line a few feet upstream from gauge No. 1, and observed through a manhole in the flow line just at the outlet end of the siphon. In order to detect the color a white dinner plate was suspended by four wires just below the surface of the water, and was lighted by a beam of sunlight reflected from a small mirror. A survey party furnished by the Los Angeles department of water supply ran two lines of levels across the valley between bench marks near the gauges while the observations were being taken. The tests indicated values of $K_s = 0.460$ or $C_w = 102.5$ for this line. It was designed with C in the Chezy formula equal to 90. The average value of C was 111.7 when the pipe was 7 years old.



A.—Deadman siphon, Los Angeles Aqueduct (Pipe No. 80). Riveted steel cylinder joints, 11 feet in diameter. Conveys water for irrigation and domestic use.
B.—A 20-inch O. D. pipe coupling joints continuous interior, class 3 penstock, Colorado Springs municipal power plant, Colorado. Pipe No. 312.
C.—Double butt strap pipe. A 48-inch pipe of heavy plate being cleaned by sand blast before coating. Note six rows of rivet heads on straight seams and four rows on girth seams.



A.—[illegible] Note [illegible] wrinkle [illegible]
B.—[illegible] No. [illegible]
C.—[illegible] A fail-welded pipe 26 inches in diameter. Pipe No. [illegible] curves.

No. 81.—Experiment S–115, 14-foot lap-riveted steel flow line, Ohio State power plant, Fremont, Ohio—Capacity −10.6 per cent

Water for power purposes is diverted from the Sandusky River and carried 2,861 feet through a 14-foot steel pipe to a large forebay adjoining a low-head power plant. The pipe is almost straight in both plan and profile. It is formed of plates five-sixteenths inch thick, cylinder-jointed in rings 6 feet 8½ inches long. At each girth joint is a stiffener ring of 4 by 6 inch L-section riveted to the pipe. As an additional precaution, heavy H-section steel members hold the pipe in circular form. A capacity test was made on a reach 2,824 feet long from a point 27 feet from the pipe intake to a point 20 feet above the forebay pool at the power house. Loss of head was found by determining the difference in elevation between a water column at the upper end and a mercury U-tube column at the lower end. Both gauges were of engine-divided manometer tubing. Velocity was determined by timing color from gauge No. 1 to the pool at the outlet.

GIRTH-RIVETED PIPES

[See pp. 113 and 126 in appendices]

No. 216.—Experiment S–116, 36-inch lockbar steel pipe—Rattlesnake siphon, *Spring Brook Water Supply Co.*, Wilkes-Barre, Pa.—Capacity −21.9 per cent

Water for domestic purposes is conveyed across Rattlesnake Hollow, a few miles from Wilkes-Barre, in a steel inverted siphon pipe connecting open canal sections on either side of the gulch. This pipe is about 1,321 feet long between inlet and outlet chambers. The two legs of the siphon are quite steep, with an intervening horizontal section. (Pl. 4, B.) The line is straight in plan view but includes nine vertical bends. Two series of experiments were made on this pipe at about the same time; one by a party under the direction of the writer, and the other by the company's engineers.

In the test made by the bureau the piezometers were all thrust into the water section through ³⁄₁₆-inch holes in the pipe. In those made by the company ½-inch corporation cocks were tapped into the shell of the pipe. The bureau's tap No. 1 was 45 feet from the intake end; the company's 0.3 foot nearer. Taps for No. 2 gauges were 3 feet above the outlet. The piezometers were spaced at approximately equal distances and all were connected to water-column gauges. Relative elevations across the gulch were determined with a Y-level and checked by piezometer readings with a velocity approximating static conditions; that is, a velocity of but 0.258 foot per second. This velocity caused a loss of head of 0.014 foot in the piezometers. Velocities determined by timing color injections were accepted as correct. A suppressed weir 4.99 feet in crest length was available, but the velocity of approach caused such a turbulent condition that the weir was taken as a rough check only. In all cases the color indicated more flow than that computed for the weir. The difference was accounted for by the uncomputed extra flow over the weir resulting from the indeterminate approach velocity. After the bureau's field party had tested such flows as could be made available, the gauges were turned over to the water company engineers who secured a few additional runs, some at high velocities. These observations with bureau equipment are included in the bureau series, while the company's piezometer readings are presented as a separate series, listed as No. 216a.

The retardation factors determined from this series should not be taken as indicating the capacity of this type of pipe, but as indicating the departure from normal capacity that is possible under certain conditions. An inverted siphon is of course subject to obstruction by heavy objects which may enter the open canal feeding the siphon, especially where the canal skirts a rather steep hillside, and this pipe was found to contain a coating from one-eighth to one-fourth inch thick of a slimy algæ growth.

No. 216a.—Same line as No. 216

At approximately the same time the tests described above were made the engineers of the Spring Brook Water Co. conducted an independent series on about the same reach of pipe. Their tests included runs at velocities from 0.26 foot per second up to 4.57 feet per second. The results at velocities below 1 foot per second were so erratic that they are not included in this bulletin, but the others, as shown in Tables 3 and 4 agree completely with those found by the bureau's field party.

CONTINUOUS-INTERIOR PIPE

[See p. 117 in Appendix 1]

No. 308.—Experiment S-126, 8-inch (outside diameter) bell-and-spigot joint steel pipe—Lateral 21 from Chatsworth high line, Los Angeles water supply, California—Capacity −6.1 per cent

This pipe is part of the network of steel lines that irrigate San Fernando Valley, within the city limits of Los Angeles. (See pipe No. 17 p. 67 for details of operation of these lines.) The nominal diameter given is the outside diameter, the inside being 7.689 inches. The interior coating is a coal-tar dip. Field joints are of the bell-and-spigot type, made tight with neat Portland cement calked into the joints.

The line is straight and on a continuous down grade, and regulated by a valve at the outlet end. A reach 585.8 feet between the intake and a branch line was selected for test. The upper gauge was a water column in graduated glass and the lower gauge a mercury column of the U-tube type. Both piezometers were thrust into the jet through small holes bored in the pipe shell.

Velocity was determined by timing color, shot into the pipe through the hole bored for piezometer No. 1 and observed by withdrawing water into a white-lined pan at gauge No. 2.

This pipe was 3 years old at time of test. A comparison with new experimental pipes of this type will be found under Nos. 302, 304 and 310, pages 117 and 118.

No. 313.—26-inch welded steel pipe. Municipal supply, Marin municipal water district, California (*166*)—Capacity −2.7 per cent

In 1926 a full-welded steel pipe was laid from Alpine Dam to serve several municipalities near the coast of Marin County, Calif. One year later flow tests were made on division No. 1, a very crooked section from the dam to Pine Mountain tunnel, a distance of 18,400 feet. The line was made in 14-foot single-plate sections, butt-welded on the straight seam. The girth joints were all butt-welded in the field with acetylene torch, the angle cut being made by means of an ingenious device perfected by the contractor. The sections were preheated and dipped in the same kind of coating as shown in Plate 1, C. Field joints were painted with the same coating. A portion of the alignment is shown in Plate 4, C. About 7,400 feet of the length was of short straight sections; the remaining 11,000 feet was in 133 short curves on 14-foot chords, the greatest angle at any one pipe joint being 16°. The aggregate angle of bends was 6,488° or 18 complete circles.

Two flow tests were made by J. S. Peters, chief engineer for the district. The third test was made some five months later by Mr. Peters and the writer. For the low flow, the two ends of the line were just submerged. Quantity was measured over a well-constructed rectangular steel weir, 8 feet long, with end and bottom contraction distances more than $3h$. The white-tiled pool above the weir was quiet and without appreciable velocity of approach. An acceptable float gauge showed the head on the weir crest. The outlet end of the pipe discharged freely into the tunnel section. The loss of head was computed from gauge readings and checked field levels.

The great length of line, compared with the diameter, makes these tests of unusual value. The second test was made by subjecting the inlet end of the pipe to the full head of water in the reservoir, some 77 feet, and by-passing the weir house through a Venturi meter, equipped with a recording device. As there was an unknown loss of head from the reservoir water through orifices, gates, valves, and Venturi meter the computations for this run were not accepted until after the third test, which was made like the second except that a mercury U-tube column was erected on the line 935 feet below the Venturi meter. For this run, it was found there was a loss of 23.4 velocity heads[5] (for the velocity of 5.47 feet per second existing in the 26-inch line), between the reservoir surface and the upper end of the 26-inch line at the weir house. The second test by Mr. Peters was then corrected for a loss of 23.4 velocity heads for the velocity of 6.04 feet per second in the pipe line. These two tests each covered a period of one hour. The data were then on a comparable basis for all three runs.

These tests may be compared directly with those made by Mr. Kempkey on the Gordon Valley line (No. 162, p. 123) as the pipes are similar except that the 26-inch line is slightly larger and very crooked. They were made by the same organization at about the same time.

CORRUGATED PIPE

No. 600.—8.88-inch experimental line, Arlington, Va. (*182, 183*) —Capacity + 3.6 per cent

In 1917 D. L. Yarnell, senior drainage engineer of the Division of Agricultural Engineering, conducted tests of flow through draintile and nominal 8-inch and 10-inch corrugated pipes. Detailed descrip-

[5] This term is used to express local losses through gratings, chambers, valves, Venturi meters, etc. generally found at the intake of municipal pipe lines. For example if the predominating velocity in the pipe connecting the "specials" is 8 feet per second, then the velocity head for 8 feet is $\frac{8^2}{2g}=1$ foot and if the aggregate loss is taken as 5 feet or determined to be 5 feet then this loss is expressed as five velocity heads and the losses for any other flow will approximate the same number of velocity heads for the velocity generated by that flow.

tion and photographs of test methods and equipment were reported (*182*). One hundred and forty-seven experiments were made on the two sizes of corrugated pipe, but only three runs, on the nominal 8-inch size, were made with the pipe running full. Elements of these tests are found in (*183*).

In essentials the layout was as follows: A straight reach of 200 feet was laid in earth placed in a wooden flume, so arranged by screw lifts that the slope could be changed. The screws allowed delicate adjustment so that the even values of H given in Table 3 were actually attained on the hydraulic gradient. The pipe units were held together by metal bands. Water, supplied by an 8-inch centrifugal pump, was measured over a 90° V-notch weir. Internal diameters were determined before units were laid.

For flows in both 8-inch and 10-inch sizes at various depths less than full, the values of Kutter's n obtained cover a wide range, but the predominating frequency of values from 0.019 to 0.020 indicates that a value of 0.020 is conservative for conditions approaching the ideal, as in these laboratory tests.

COMPOUND PIPES

[See pp. 121 and 126 in appendices]

No. 154.—Experiment S-133—64.2-inch (average) riveted steel pipe—River siphon, Bitter Root Valley irrigation district, Montana—Capacity + 14 per cent—Without allowance for time deterioration (because of polishing velocities) capacity + 2.1 per cent

Irrigation water is conveyed across the Bitter Root River, between open canal sections, in a steel siphon pipe of varying diameters as listed in Table 2.

The reach tested was 5,362 feet long, which did not include the first 287.5 feet of the 71-inch pipe at the intake end and 4.2 feet at the outlet end. The long reach, combined with a high velocity throughout, made the experiment of great value, results being but little influenced by unavoidable experimental errors. Both piezometers were inserted into the water prism through 3/16-inch holes bored in the side of the pipe. Even with this precaution, the water entered the siphon in such a turbulent condition that air was very evident at gauge No. 1 until removed from the pressure tubing leading up the hill from the piezometer tap to the graduated glass.

This was accomplished by an air trap placed just outside the steel pipe and made as follows: A glass Y-tube was inverted, with one short leg connected to the piezometer tube outside the siphon and the other short leg leading to the water-column gauge glass. The vertical leg of the Y was connected to the color gun, which was used as an air chamber. Bubbles of air finding their way into the piezometer tube, ascended the glass Y into the air chamber, leaving a column of solid water in the pressure tubing between the siphon and the gauge.

If air bubbles are allowed to remain in such a water column, the indicated elevation of the top of the column is erroneous. Where much air is entrained in a line being tested all the bubbles can not be expelled from the water column by blowing off the gauges. A continuous stream of bubbles comes along with the water, so that at any time the blow-off is closed the column between the gauge and the piezometer is still filled with a combination of air and water. However, with a trap such as that described, it is easy to secure a water column free of air.

The average velocity through the siphon from gauge No. 1 to the outlet was determined by the injection of fluorescein at tap No. 1 which was observed in the canal at the outlet of the pipe. The quantity, Q, was found by using the continuity equation, $Q = AV$, where A is the weighted average cross-sectional area of the pipe through which the color passes and V the average velocity. The total loss of head for the reach tested amounted to 37.9 feet. Just how much of this was lost in the different sized reaches was not determined.

No. 158.—Experiment S-103—84-inch and 90-inch riveted steel pipe, cylinder jointed, St. Mary River crossing siphon, Milk River project, United States Bureau of Reclamation, Montana—Capacity −16.8 per cent

Water for irrigation purposes is conveyed across the St. Mary River in an inverted siphon pipe of riveted steel. This siphon, between open canal sections, is composed of 1,432 feet of 90-inch pipe, then a 6-foot transition section to an 84-inch pipe 831 feet long, then an enlarging transition section back to a 90-inch pipe 953 feet long.

The pipe line is of the cylinder-joint type, with beveled edge plates having double rivet rows on the longitudinal seams and single rows on the girth seams. It was built under two contracts. Under the first it was assembled in field sections and given one coat of water-gas tar and two coats of coal-gas tar, 1 foot at each end being left without paint, so that final riveting would be on uncoated metal. All peeled spots were recoated. Approximately the entire last half of the pipe was constructed under the second contract. This portion of the line was given three coats of coal tar, cut with one-quarter its volume of water-gas tar and distillate, mixed in equal parts. This coat was applied at boiling temperature with a brush.

The siphon was finished in 1915. Two years later tests were made at two velocities. Gauge No. 1 was attached to piezometers at a point 315.8 feet from the intake end of the pipe, on the first 90-inch section. Gauge No. 2, attached to similar piezometers, was placed only 9 feet above the outlet end of the siphon, on the second 90-inch pipe. The loss of head was determined for a compound pipe—two 90-inch sections at the ends, joined by 6-foot taper sections to an 84-inch reach crossing the bridge above the river. The quantity of water was measured with a current meter in Spider Lake coulee flume, located just below the outlet of the siphon pipe.

The individual losses in the various sizes of pipe were not determined but the total loss can be allocated to each pipe size, by using the same values of K_s and C_s for each run of water.

ANALYSIS OF THE EXPERIMENTAL DATA

The extent of the data now available on measured losses of head for given velocities, in riveted-steel and analogous pipes, warrants the acceptance of some one of the existing formulas of flow or the revision of certain elements to accord with data accumulated from actual experience. If the necessary revision of elements is of moment a new formula must be derived.

To avoid going over old ground certain generally accepted premises will be stated here and the argument taken up from that point. Literature citations will indicate where more complete discussion of the premises can be found. The primary assumptions are:

1. *The exponential type of formula is preferable to any other form.*—It is fully understood that losses within most closed pipes, at velocities above Reynolds' critical velocity (*142*), i. e. at all "commercial velocities" require for the exponent of V a compromise value somewhat less than 2.00 (*140, 55, 147, 173, 151, 16, 180, 183*). A series of observations on a pipe, platted on logarithmic paper, with losses of head as ordinates and velocities as abscissas, gives a straight line whose slope indicates the exponent of V in formula 14, p. 79.

2. *A complete statement of the elements entering a formula for the loss of head must include the influence of temperature of the water in affecting viscosity (140, 18, 16, 89, 133).*—This influence is quite small and has generally been ignored heretofore, especially by American hydraulicians, as the variability of other factors in the various formulas did not warrant refinement in this one direction. In this bulletin water temperatures were considered, where available. Final recommendations are based on a temperature of 15° C. The last column of Table 6 shows the correction necessary if any other temperature is to be the criterion. It also shows that this correction is insignificant for all temperatures likely to be encountered.

3. *Consideration of viscosity most properly takes the form of acceptance of the law of dynamic similarity for fluid motion (142, 16, 133).*—This law[6] is to the effect that frictional losses in the flow of both liquids and gases are a function of $\frac{VD}{\nu}$.

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[6] There appear to be three general ways to apply the fact that resistance to flow in all fluids is a function of "Reynolds' number" $\frac{VD}{\nu}$. So far authorities are in agreement. With this as a base, Parry (*133*), for instance, develops constants for various kinds of pipes in terms of Chezy's C, but this method yields variable values of C for different velocities in the same conduit unless the loss of head varies as the square of the velocity. Other European writers develop a binomial type of formula using the above viscosity function in one of the terms. The latest of these formulas as given the writer in correspondence by Erik Lindquist (*199*), of Stockholm, follows:

$$\frac{2gDs}{V^2}=a\left(1+\frac{\frac{B}{3}}{\sqrt{\frac{VD}{\nu}}}\right)\left(1+\frac{K}{\sqrt{D}}\right)$$

where B diminishes and K increases with the roughness of the pipe.

The third way to utilize the law of similarity of fluid flow, and at the same time follow empirical results as indicated by fractional exponents, has been chosen by the writer, following the exhaustive studies of Biegeleisen (*16*).

If values of the exponents change, as is claimed by some authorities, there does not appear to be any change throughout the range of Reynold's number covered in the experimental data available, and this range is sufficient to encompass all present practice. The ease of handling the exponential formula, the definite tieing together of the exponents in algebraic form, and the relative simplicity of treatment by calculus, in economic studies, all encourage the use of the exponential formula rather than other types where "constants" are only approximately constant or so many in number that almost unlimited data would be required to set forth formulas definitely for various categories as is done in this publication.

The following argument, expanded and converted to the notation used in this bulletin was offered by Biegeleisen (*16, p. 44*) to utilize the law of fluid similarity in terms of exponential formulas already accepted.

In basic equation 6, page 9, $h_f=\frac{fLV^2}{2Dg}$ the general value of $f=a\left(\frac{\nu}{VD}\right)^n$ hence

$$\frac{h_f}{L}=\frac{a\left(\frac{\nu}{VD}\right)^n V^2}{2gD}=\left(\frac{a}{2g}\right)\left(\frac{\nu^n}{V^nD^n}\right)\left(\frac{V^2}{D}\right)=\left(\frac{a}{2g}\right)\nu^n\frac{V^{(2-n)}}{D^{(1+n)}}$$

Substituting x for $(2-n)$ then $1+n=2-x+1=3-x$ and $n=2-x$, and by writing $\frac{a}{2g}=\frac{M}{1{,}000}$ the following is obtained for the general formula for the flow of water in circular closed conduits.

$$\frac{1{,}000h_f}{L}=\frac{M\nu^{2-x}V^x}{D^{3-x}}\ \text{or}\ H=\frac{M\nu^{2-x}V^x}{D^{3-x}}.\quad \text{(See equations 19 and 20.)}$$

Acceptance of the above premises may be stated in proper algebraic form as follows: For a given surface the loss of head is influenced by three variables, which in order of magnitude are velocity, diameter, and temperature of the water. Within any given pipe, disregarding for a moment the viscosity,

$$H = MV^z \tag{14}$$

For a series of pipes of the same general characteristics but of varying diameters, the values of M follow the general equation

$$M = KD^{-x} \text{ where } x \text{ is negative.} \tag{15}$$

Substituting in formula 14

$$H = K\frac{V^z}{D^x} \text{ or } \log H = \log K + z \log V - x \log D. \tag{16}$$

This form has been offered many times, with various values of K, x, and z, depending on the experience data available at the date of study. Thus far viscosity has been disregarded but must be taken into account in the derivation of a precise formula. Considering the viscosity

$$K = m\nu^y \tag{17}$$

Substituting in formula 16

$$H = m\nu^y\frac{V^z}{D^x} \tag{18}$$

The solution for H now appears hopelessly involved, there being three knowns ν, D and V, and four unknowns, the exponents y, x, and z and the term m representing the internal characteristics of the pipe; but the law of similarity of fluid flow clarifies the situation. Converted to the writer's notation, Reynolds' classic work of 1883 (*142*), may be stated

$$H = \frac{m\nu^{2-z}\,V^z}{D^{3-z}} \tag{19}$$

Blasius' proof of this formula is given on page 78, showing a relationship such that, if enough observations on several pipes of the same characteristics are at hand, the exponent of V can be determined and the other two exponents disclosed. The mass of experience data listed in Tables 1, 2, 3, and 4 indicates the average exponent of V to be 1.9. Substitution in equation 19 above gives

$$H = \frac{m\nu^{0.1}\,V^{1.9}}{D^{1.1}} \tag{20}$$

For a working formula, considering viscosity of water at 15° C., substitution in equation16 gives

$$H = \frac{K_s V^{1.9}}{D^{1.1}} \tag{21}$$

This is the writer's basic formula No. 9 from which Nos. 10 and 11 are computed. Before the determination of K_s the factors of equation 20 should be considered as much depends upon it. It is not difficult to accept the relationship of the exponents of V and D as a consideration of some 25 or more exponential formulas offered during the past 75 years (listed in Table 5) indicates that the sum of these two exponents has nearly always been found approximately equal to 3. Reynolds' work was done with tubes of small caliber. The opportunity of checking his deductions came with the experiments of Saph and Schoder (*146, 147*) on small brass pipes. Their values of z and x were respectively 1.75 and 1.25 the sum of which again equals 3. Blasius (*18*), following the work of Reynolds and Saph and Schoder, proved the law of similarity but did not have sufficient data available from which to draw conclusions as to values of the coefficient, K, for various kinds of pipe. Biegeleisen and his coauthor (*16*) undertake a most extensive and careful analysis of the work of all previous authorities and show the mathematical correctness of equation 19. (See p. 79.) They then apply the experience of 1,761 gaugings on cast-iron pipe and find exactly the same exponents—1.9 for V and 1.1 for D—found by the writer for the pipes considered in this bulletin. However, they found 1.8 as the exponent for V and 1.2 for D on a study of steel pipe for which they had but little data, compared to that now available.

TABLE 5.—*Exponents in various formulas for metal pipes, based on the general formula* $H=K\frac{V^z}{D^x}$, *and showing approach of* $z+x$ *to a value 3.000 as first suggested by Reynolds*

No.	Name	Year	z	x	$z+x$	Kind of pipe
1	Chezy	1775	2.000	1.000	3.000	General.
2	Saint-Venant	1851	1.710	1.000	2.710	Cast iron.
3	Lampe	1873	1.802	1.250	3.052	Do.
4	Fanning	1878	2.000	1.000	3.000	Do.
5	Reynolds	1883	1.850	1.150	3.000	Old cast iron.
6	do	1883	2.000	1.000	3.000	New cast iron.
7	Unwin	1886	1.85	1.127	2.977	Asphalted cast iron.
8	Flamant	1892	1.750	1.250	3.000	All kinds.
9	Tutton	1890	1.818	1.127	2.945	New, asphalt coated.
10	do	1890	1.961	1.294	3.255	Old, asphalt coated.
11	do	1899	1.961	1.294	3.255	Lap riveted.
12	Fidler	1902	1.770	1.180	2.950	Bare riveted.
13	Saph-Schoder	1903	1.740	1.250	2.990	Various kinds.
14	Williams-Hazen	1903	1.852	1.167	3.019	All kinds.
15	Williams	1904	1.870	1.250	3.120	Do.
16	Unwin	1907	1.870	1.390	3.260	Riveted wrought iron.
17	Lea	1907	2.000	1.250	3.250	Clean riveted.
18	do	1907	1.880	1.250	3.130	Galvanized.
19	Moritz	1913	1.800	1.250	3.050	Riveted.
20	Biegeleisen	1914	1.800	1.200	3.000	Steel.
21	do [1]	1914	1.900	1.100	3.000	Cast iron.
22	Barnes	1916	1.795	1.092	2.887	Screw-joint, riveted.
23	do	1916	1.898	1.372	3.270	Single riveted.
24	do	1916	1.923	.846	2.769	Double riveted.
25	Giesecke	1917	1.770	1.274	3.044	Small black standard.
26	Corp and Ruble	1927	1.887	1.151	3.038	Wrought iron.
27	Wegmann-Aeryns	1925	1.856	1.342	3.198	Cast iron.
28	Flinn-Weston-Bogert	1927	1.950	1.250	3.200	Lap riveted.
29	Scobey [1]	1927	1.900	1.100	3.000	Riveted steel and analogous.

[1] Note that 1,761 observations gave Biegeleisen, in 1914, the same exponents for his formula for cast-iron pipe that the writer found for riveted-steel and analogous pipes, discussed in this bulletin.

In the writer's determination of 1.9 for z, the data were platted on logarithmic paper, H being used for ordinates and V for abscissas. (Fig. 3.) Observations manifestly inconsistent, when compared with others in the same series, were given a D rating (column 3, Tables 1, 2, and 3) and were not included in the analytical derivation of the equation of that series, nor were they considered in the comparison of observed to calculated capacity or in computing average values of C_w, K_s, or m. For each series covering any material range of velocities the individual pipe equation was derived by the centroid method (*152, p. 50*) and listed in column 11, Table 4, which shows the average value of z for various types of pipe that may be listed and weighted as follows:

	Average value of z
Full-riveted pipe, 33 series	1.902
Girth-riveted pipe, 6 series	1.950
Continuous-interior pipe, 9 series	1.912
Dredge pipe, 2 series	1.855
Spiral-riveted pipe, 21 series	1.881
Weighted average value for all pipes	1.900

Not only is the general average value of z equal to 1.900 but also the value of z is 1.902 for the dominating type.

TABLE 6.—*Values of $\nu^{0.1}$ (column 7) for use in formula 20, page 79, when the influence of temperature upon water viscosity is considered*

Temperatures		Viscosity μ C. G. S. units	Density ρ C. G. S. units	Kinematic viscosity ν: $\frac{\mu}{\rho}=\nu$ C. G. S. units	Kinematic viscosity ν: Column 5 ÷929.03 English units	$\nu^{0.1}$ English units	Variation in velocity, V at 15° C.=100 per cent
1	2	3	4	5	6	7	8
°C.	° F.	*Poises*	*Gr. per cu. cm.*	*Sq. cm. per sec.*	*Sq. ft. per sec.*		*Per cent*
0	32.0	0.017921	0.99987	0.01792	0.00001929	0.3377	−2.4
4	39.2	.015674	1.00000	.01567	.00001687	.3332	−1.5
10	50.0	.013077	.99973	.01308	.00001408	.3272	−.7
11	51.8	.012713	.99963	.01272	.00001369	.3263	−.6
12	53.6	.012363	.99952	.01237	.00001332	.3254	−.5
13	55.4	.012028	.99940	.01204	.00001296	.3245	−.3
14	57.2	.011709	.99927	.01172	.00001262	.3237	−.2
15	59.0	.011404	.99913	.01141	.00001228	.3228	.0
16	60.8	.011111	.99897	.01112	.00001197	.3220	+.2
17	62.6	.010828	.99880	.01084	.00001167	.3212	+.3
18	64.4	.010559	.99862	.01057	.00001139	.3203	+.4
19	66.2	.010299	.99843	.01032	.00001111	.3196	+.7
20	68.0	.010050	.99823	.01007	.00001084	.3188	+.8
21	69.8	.009810	.99802	.00983	.00001058	.3180	+.9
22	71.6	.009579	.99780	.00960	.00001033	.3173	+.9
23	73.4	.009358	.99757	.00938	.00001010	.3165	+1.0
24	75.2	.009142	.99733	.00917	.00000987	.3158	+1.2
25	77.0	.008937	.99708	.00896	.00000964	.3151	+1.3
26	78.8	.008737	.99682	.00876	.00000943	.3144	+1.5
27	80.6	.008545	.99655	.00857	.00000922	.3137	+1.6
28	82.4	.008360	.99627	.00839	.00000903	.3130	+1.7
29	84.2	.008180	.99598	.00821	.00000884	.3124	+1.8
30	86.0	.008007	.99568	.00804	.00000865	.3117	+1.9
35	95.0	.007225	.99406	.00727	.00000783	.3086	+2.4
40	104.0	.006560	.99225	.00661	.00000711	.3057	+2.9
45	113.0	.005988	.99025	.00605	.00000651	.3030	+3.4
50	122.0	.005494	.98807	.00556	.00000598	.3004	+3.9

While the list above fully warrants the adoption of a value of 1.9 for z, at least as against any other round-figure value, still it must not be overlooked that the individual values of z vary all the way from 1.73 to 2.14. The reason for low or high values of z has been discussed for 25 years. When Saph and Schoder (*146, 147*) found 1.75 as a consistent value of this exponent for smooth brass pipes of small diameter it appeared conclusive that this value was related to smoothness, and this was the concensus of opinion for many years. In looking over the available data on smooth steel, wood-stave, and concrete pipes (*152, 153*) the writer has found many pipes with frictional coefficients indicating very high capacities but with exponents of V in excess of 1.90. Conversely, D. L. Yarnell and his associates found an average value of z of only 1.986 for corrugated pipe culverts, the corrugations resulting in a very low capacity relative to that of other pipe materials (*183*). If z were wholly a function of smoothness then it should increase as the capacity in a given reach of pipe deteriorates with age. Likewise old pipes, shown in Figure 7 to have decreased in capacity with age, would show relatively high values of the exponent z. To demonstrate that there is nothing conclusive in this argument Figure 4 is offered, showing the values of z plotted against the age of pipe in years, with related gagings connected by light lines. The points disclose no definite trend.

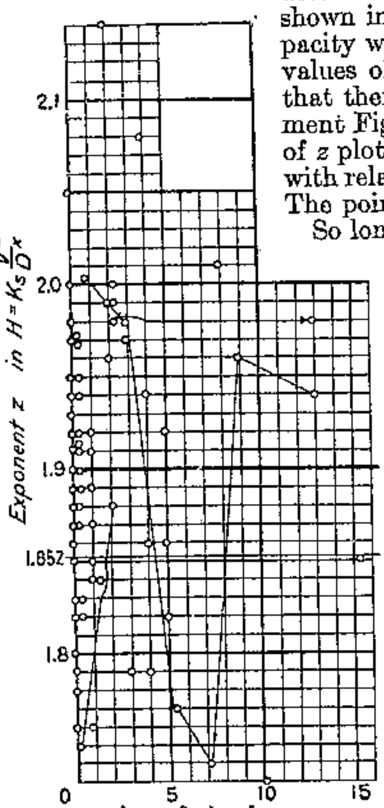

FIGURE 4.—If the exponent z is related to roughness of pipe, then it should be greater for older pipes. The diagram above does not show any definite tendency for an increase in z with age of pipe. Data for this diagram from Table 4, columns 4 and 11

So long as a wide variation appears in the exponent of V and this variation can not be rectified with present knowledge, a round figure for the exponent of V is all that is warranted. Hence, a value 1.9 for z is adopted as the average figure found above for several types of metal pipes. In formula 19, page 79, 1.9 is substituted for z, which gives tentative values for y and x, respectively, of 0.1 and 1.1. The value of $y=0.1$ could be much changed, or ignored altogether without materially affecting the formula, hence it will be accepted as part of the mathematical proof given on page 78. The value of x, on the other hand is of more moment. To test the tentative value of 1.1, Figure 5 was developed by plotting as ordinates values of M as taken from the individual formulas in Table 4, column 11, and values of M' for such observations as were too few to yield an individual formula (also from Table 4, column 11); and by plotting as abscissas the values of d. Lines at a slope of -1.1 through groups of points known to be good criteria indicate that this slope is as good as any other could be, hence the whole proof of Blasius and Biegeleisen is supported by empirical data. There remains only the selection of values of m in formula No. 20 for the various classes of metal pipes and from these values the selection of corresponding values of K_s in the working formula No. 21.

It was considered best to work toward round figures for values of K_s', rather than accept necessarily round figures for m and from these

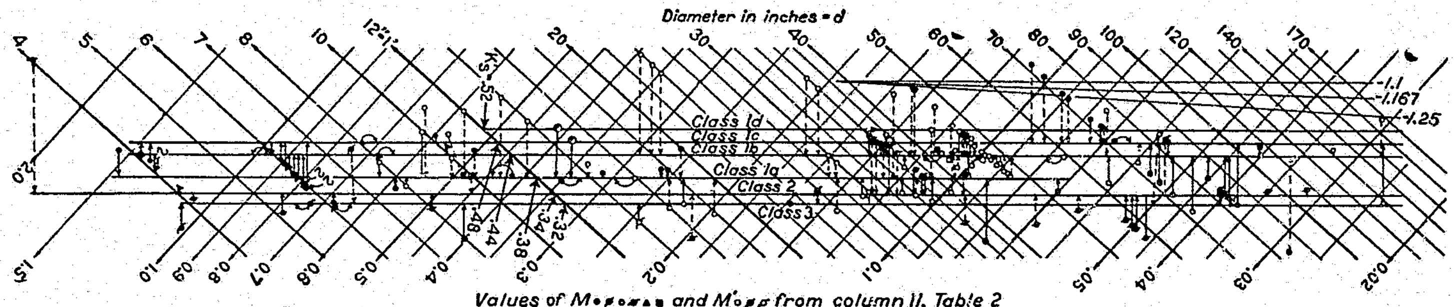

FIGURE 5.—Logarithmic diagram showing values of M and M' plotted against values of d for test of the exponent x in formula No. 15 after theoretical argument discloses this exponent should be −1.1. (See page 82.) Slope lines for −1.1 and −1.167 (Williams-Hazen) and for −1.25 (Saph-Schoder) are all given for comparison. Points are connected by arrows to proper class curve

get extended figures for K_s' in converting from one to the other. For the various subdivisions of class 1, page 12, the values of m from Table 4, column 14, were plotted as ordinates and shell thickness as abscissas. It was quite obvious that there was an increase in the values of m with the increase in shell or metal thickness. The influence of shell thickness was tabulated by LeConte (*116, p. 549*); (*132, p. 456*). It was evident also that pipes of thin-gauge metal, with well-buried flathead rivets, offered less resistance to flow than thick-plate pipes having appreciable offsets between individual pipe rings and single, double, or triple rows of bold rivet heads, themselves corresponding in size with plate thickness.

The selection of values of K_s', as given on page 12, was the result of careful weighing of the various points in Figure 6, finally supported by a comparison of observed to computed capacities for various basic

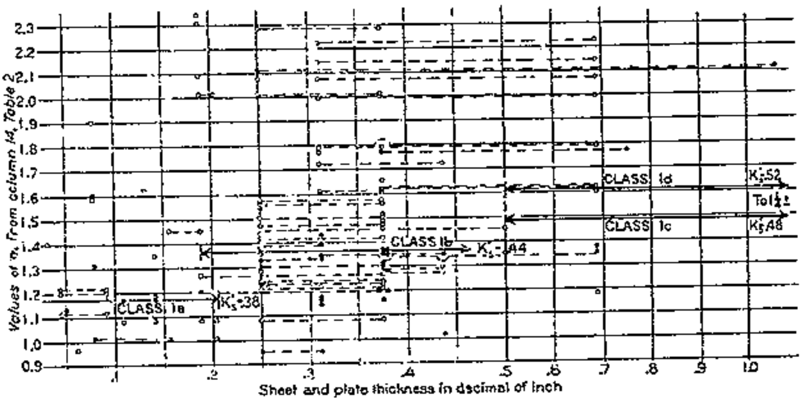

FIGURE 6.—For full-riveted pipes there is a tendency of retardation coefficients to increase with thickness of plate, largely because number and size of rivet heads also increase with plate thickness. Practically new pipes indicated by dots, older pipes by open circles. Points connected by dashed lines indicate range of plate thickness, within each tested, as listed in Table 4, column 5. Plotted in terms of m, which includes viscosity influence. Interpreted in terms of K_s' for use in working formula No. 9. Heavy horizontal lines, ending in arrow points, show range of plate thickness for each subdivision of class 1. The vertical position of these lines is determined by a round value of K_s' corresponding to the plotted value of m at 15° C.

assumptions of round-number values of K_s' shown by lines superimposed over the values of m. The values of K_s used in the comparison, of course, were modified to include the age-deterioration tendencies shown in Figure 7. It is to be regretted that the points make so scattered a pattern on the drawing; the explanation is that there are great intrinsic differences, due to the state of the interior coating, the extent of silting, the amount of curvature, number of valves, and other "specials," in pipes constructed under closely identical specifications. Had all observations been made on pipes of presumably equal interior surfaces, values could have been determined by the method of least squares. Such values would have been unassailable from a mathematical standpoint, but the writer is certain the values must be based on judgment formed from experience with pipe lines known to be typical, where conditions of experimentation were of the best. Individual values so chosen must then stand the test of comparison in adjoining classes of pipe. The argument for the various classes follows.

FULL-RIVETED PIPE

CLASS 1a. $K_s'=0.38$ FOR NEW PIPE

[See p. 12 for class description]

It was desirable to have values conservative; that is, below observed values where conditions were definitely known to be typical. For pipes of class 1a, of sheet metal seamed with flathead rivets, it was known that pipes 20, 31, and 33 fulfilled these conditions. It was also evident that the pipes tested by Hamilton Smith should be regarded in the same light, their capacities being slightly above those to be anticipated in design. This was true because the velocities he found were quite high and his descriptions indicate polished interiors. With these facts in mind a comparison of observed to computed capacities was made with three basic assumptions for values of K_s' for new pipes and a temperature of 15° C. The values tested were 0.36, 0.38, and 0.40, 0.38 being finally adopted as conservative. The last column of Table 4 shows the comparison between observed capacities and those computed with a value of 0.38 for a new pipe, and the value of K_s increased according to formula (22) for time deterioration. The irrigation pipes carrying water from open canals at low velocities nearly all show capacities below that called for by formula, because of bedding down of sand and silt deposit.

The writer's value of 0.38 for K_s' roughly corresponds with the Williams and Hazen $C_w=120$ for 30-inch pipe or 130 for 6-inch pipe. These values of C_w are from 10 to 20 points higher than they recommend for steel pipe. However, their discussion and data lead to the deduction that they had in mind plate thickness and rivets that would compare with class 1b, discussed below. In fact they speak of but one class of steel pipe. This type of pipe is made in the smaller sizes, for which Kutter's formula is particularly unsuitable. For a 30-inch pipe the value of n is about 0.012, while for a 6-inch pipe it is about 0.010.

CLASS 1b. $K_s'=0.44$ FOR NEW PIPE

[See page 12 for class description]

A study of Figure 6 shows that $K_s'=0.44$ is the round figure most acceptable for this group classification. In Table 4, column 15, it is shown that in most cases the observed capacity agrees very closely with the computed capacity. Where there is divergence it is generally very wide. In some cases the pipe descriptions disclose the reason. Generally speaking the agreement is closer than was the case for class 1a. This is reasonable because pipes of this class have been used for municipal supply to a greater extent than for irrigation use where there is greater opportunity for closure of the pipe by silting.

The value of $K_s'=0.44$ corresponds with reasonable closeness to the Williams-Hazen $C_w=110$ for 30-inch pipe which diminishes to 100 for 120-inch pipe. This group included most of the experimental results on which Williams and Hazen recommended their $C_w=110$ for new steel pipes. The corresponding values of Kutter's n for the 30-inch pipe is about 0.013, and for the 120-inch pipe about 0.015.

CLASS 1c. $K_s'=0.48$ FOR NEW PIPE

[See p. 12 for class description]

Figure 6 shows that the data for heavy plate pipe are quite meager. The few tests available indicate that a value of 0.48 is reasonably close. This is confirmed by extension of previous group coefficients.

The writer's value of $K_s'=0.48$ corresponds roughly with the Williams-Hazen $C_w=100$, a value that has been used quite extensively in the design of heavy penstocks. The corresponding values of Kutter's n are from 0.014 to 0.016. The data indicate that these are approximately correct for such penstocks when new but less favorable values should be used for the same lines when 10 to 20 or more years old.

CLASS 1d. $K_s'=0.52$ FOR NEW PIPE

[See p. 12 for class description]

While there are few tests available for lines of this class, this class nevertheless includes such pipes as the Jawbone siphon of Los Angeles Aqueduct and will also include some of the large pipe lines now planned, for which a projection of the available data appears warranted until such time as more actual capacity tests on extra large sizes have been made. It is suggested that a value of $K_s'=0.52$ be accepted tentatively. This value agrees reasonably well with the Williams-Hazen $C_w=95$ for a 10-foot pipe and $C_w=90$ for a pipe 24 feet in diameter. The equivalent value of Kutter's n is between 0.017 and 0.018.

SPIRAL-RIVETED PIPE, FOR FLOW WITH THE LAPS $K_s'=0.44$ FOR NEW PIPE AND FOR FLOW AGAINST THE LAPS $K_s'=0.48$

Very extensive series of tests on short reaches of small spiral-riveted pipe as set up for experimentation in laboratories are available, but no tests have been made on lines in commercial operation so far as is known. Appreciating the fact that the latter very seldom bear out results obtained in the laboratory, the tentative values of K_s suggested indicate slightly poorer capacities than those shown by the points in Figure 3. These tentative values are: For flow with the laps $K_s'=0.44$ and for flow against the laps $K_s'=0.48$. These values roughly correspond to the Williams-Hazen $C_w=110$ and $C_w=100$, respectively. The values of K_s resulting from tests on the smaller pipes indicate that tentative design values of $K_s'=0.38$ for flow in new pipe with the laps and $K_s'=0.44$ for flow against the laps are amply conservative; but it is to be noted in Table 4 that when the experiments of Greve and Martin are considered, the test values increase with the size of pipe, finally reaching 0.46 for flow with the laps in an 8.12-inch pipe and 0.48 in the same pipe with the flow against the laps.

It is suggested that the rifling effect produced by the spiraled laps and rivet heads may set up a condition of flow quite far removed from the regimen established in ordinary full-riveted pipe lines, and for this reason a high factor of safety, expressed in excess designed capacity, should be used in computing the flow in large spiraled lines in field service, although future tests on such lines may show that this warning is not justified.

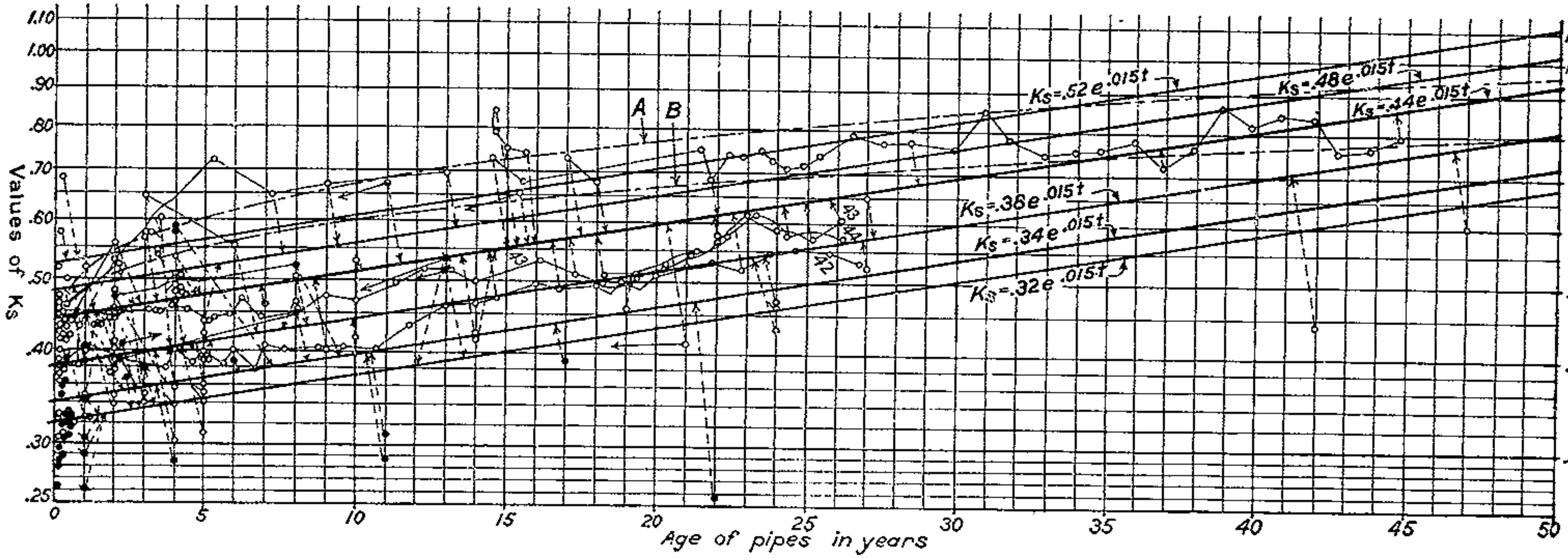

FIGURE 7.—Increase in retardation coefficient, indicating decrease in capacity, with age of pipes. Open circles show full-riveted pipes; dots, girth-riveted pipes; and barred dots, continuous-interior pipes. Points are connected by arrows to the equation curve for their particular class and type of pipe. All points connected by full lines indicate progressive values for the same reach of pipe. The A curve shows Mills's equation for riveted pipes within 1 mile of reservoir. The B curve for pipes beyond 1 mile from reservoir. Data for this graph from Table 4, columns 4 and 13

GIRTH-RIVETED PIPE

CLASS 2. $K_s'=0.34$ FOR NEW PIPE

[See p. 12 for class description]

A study of Table 1 shows that tests on the small pipes from 4 to 10 inches in diameter disclose a coefficient slightly more favorable than 0.34, while tests on the pipes from 36 inches to 58 inches show an unexplainable situation. Practically all of the pipes were of the lock-bar type. The writer will not comment on values of K_s applied to old pipe in accordance with the deterioration curves shown in Figure 7, but the results of tests on new pipe vary too much for pipes made by the same methods. The Rochester tests (Nos. 220 and 222) on two long reaches of pipe show a capacity but little better than that of an ordinary riveted pipe of class 1b and the Wilkes-Barre tests (No. 216) show still higher coefficients, the reason for which is understood, while the Portland tests (No. 226) on new pipe and the Montreal (No. 218), Springfield (No. 224), and Portland (Nos. 225 and 225a) tests on older pipe show capacities corresponding to a value of K_s from 0.26 to 0.30. Because of the wide divergence shown for pipes that should be alike to a reasonable degree the value of $K_s=0.34$ for class 2 should be taken as tentative, especially as regards lock-bar pipe, indicating a pipe with a capacity nearly 15 per cent above that of ordinary riveted-steel pipe of class 1b. This percentage agrees quite closely with recorded ideas of qualified engineers. The value of $K_s=0.34$ corresponds roughly with the Williams-Hazen $C_w=125$.

Only two short series of tests on uniform bump-joint welded pipe can be presented—Nos. 228 and 230. These show $K_s=0.36$ to agree quite closely, and future data may show that the deep recess at the bump will necessitate this value rather than the more favorable value, 0.34; still the general description of the welded pipe with bump-joints is such that it will be left in class 2 until proven incorrect by additional data. Tests on pipes 156 and 156a, both compound lines, confirm the choice of 0.34 so far as required assumptions warrant.

CONTINUOUS-INTERIOR PIPE

CLASS 3. $K_s'=0.32$ FOR NEW PIPE

[See p. 12 for class description]

The specifications for this class of pipe require a surface that approaches the best obtainable with coatings now in use. The tests on the straight experimental pipes at Versailles, Pa., Nos. 302, 304, and 310, show values of K_s, which are probably too favorable to be accepted for field installations with more or less curvature, although the values for Nos. 302 and 304 are supported by the Bend, Oreg., test (No. 311) on 5 miles of 14-inch full-welded pipe. The tests on the long Coolgardie line (No. 314) (which was one of the first lock-bar pipes, but is placed in class 3 rather than in class 2 because the field joints every 30 feet were butt joints, leaded under sleeves) and the test of Mills's No. 316 show surfaces only slightly inferior to that giving the value $K_s=0.34$. However, Mills's No. 318, paralleling and duplicating No. 316, shows a capacity but 6 per cent greater than that of an ordinary riveted pipe of class 1b; yet Mills's description of both of these pipes, with countersunk rivet heads, indicates that, when the lines were first put in service, he had high regard for the interior surface.

The recommended coefficient for this class of pipes, $K_s'=0.32$, is offered tentatively, there being a strong probability that much additional data may indicate a value $K_s=0.30$ to be warranted for a new pipe. The value $K_s'=0.32$ roughly corresponds to the Williams-Hazen $C_w=130$ for pipes above 30 inches in diameter, while for small pipes the corresponding value of C_w approaches 140.

DREDGE PIPES

$K_s'=0.38$ FOR NEW PIPE FOR CLEAR WATER AND $K_s'=0.44$ FOR NEW PIPE AND DREDGED MATERIALS

A study of the last column in Table 4 for dredge pipes (Nos. 402 to 422) shows that most of the observed capacities agree with the computed capacity within 10 per cent, on the assumptions that $K_s'=0.38$ for clear water and 0.44 for loaded water; however, the difference may be partially due to pipe characteristics and differences in joints rather than to class of materials carried.

CORRUGATED PIPES

$K'_s=1.40$ FOR NEW PIPE

For this type of pipe few data are available. Using the results of tests on Nos. 600 and 602 and comparing with the formula of Yarnell and associates *(183)* the writer suggests a tentative value of $K_s'=1.40$ or a value of Williams-Hazen $C_w=60$.

EFFECT OF AGE UPON CARRYING CAPACITY

Steel pipe lines are subject to several forms of chemical reaction tending to decrease capacity; wood-stave and concrete pipes are seldom seriously affected. Since steel was first used, constant efforts have been made to develop coatings to combat this tendency effectively. There are now available many dips and paints, usually with a base of asphalt, tar, or other hydrocarbons, which are so used that new pipes usually show a very smooth and glossy surface *(128)*[7]. For some of the newer coatings, made under various trade names, broad claims are made for long life and efficiency. Time alone will indicate just how effective these coatings are in the prevention of corrosion, tuberculation, and consequent loss of capacity. Until such proof is forthcoming it is more conservative to assume that all moderate-velocity steel lines will deteriorate in capacity.

Figure 7 shows the extent of this decrease on the basis of data now available. The diagram is on semilogarithmic paper, which results in straight lines for data that follow the laws of organic growth, compound interest, and the like. The ordinates show the increase in the retardation factor according to a logarithmic scale, while the abscissas show the age of the line in years according to an arithmetic scale.

For several lines, practically all of which are in the eastern part of the United States, there are available long records of periodic gagings. Some engineers believe that the more acid waters of the eastern States are conducive to decrease of capacity in greater degree than the generally harder western waters. If this be true, western pipes are not sufficiently represented in the diagram; hence, only one formula is derived, this being based on the tendency shown by the slant-

[7] This publication contains an extensive bibliography on corrosion.

ing lines drawn from values of the coefficient K_s' for new pipes of various classes.[8]

$$K_s = K_s' e^{0.015t} \text{ (See Table 7, Part A)} \qquad (22)$$

In Figure 10 the graphic presentation of formula 22 in the left diagram is supplemented by the right diagram, which may be used where the waters are known to be nonaggressive. Here the change in the value of K_s is taken at 1 per cent per year instead of 1½ per cent, as in the left local diagram. For such waters formula 22 has been modified to read

$$K_s = K_s' e^{0.01t} \text{ (See Table 7, Part B)} \qquad (22a)$$

TABLE 7.—*Values of coefficient K_s for any age pipe from 1 to 50 years*

Age of pipe t, in years	A Conservative values, based on experience with eastern waters. $K_s = K_s' e^{0.015t}$						B Suggested values for water known to be relatively inactive. $K_s = K_s' e^{0.01t}$					
	$K_s' =$ 0.32	$K_s' =$ 0.34	$K_s' =$ 0.38	$K_s' =$ 0.44	$K_s' =$ 0.48	$K_s' =$ 0.52	$K_s' =$ 0.32	$K_s' =$ 0.34	$K_s' =$ 0.38	$K_s' =$ 0.44	$K_s' =$ 0.48	$K_s' =$ 0.52
1	0.325	0.345	0.386	0.447	0.487	0.528	0.323	0.343	0.384	0.444	0.485	0.525
2	.330	.351	.392	.454	.495	.536	.326	.347	.388	.449	.490	.530
3	.335	.356	.397	.460	.502	.544	.330	.351	.392	.454	.495	.536
4	.340	.361	.404	.467	.510	.552	.333	.354	.396	.458	.500	.541
5	.345	.366	.410	.474	.517	.561	.336	.357	.400	.462	.505	.547
6	.350	.372	.416	.481	.525	.569	.340	.361	.404	.467	.510	.552
7	.356	.378	.422	.489	.533	.578	.343	.365	.408	.472	.515	.558
8	.361	.384	.428	.496	.541	.587	.347	.368	.412	.477	.520	.563
9	.366	.389	.435	.504	.550	.595	.350	.372	.416	.481	.525	.569
10	.372	.395	.442	.511	.558	.604	.354	.376	.420	.486	.530	.575
11	.377	.401	.448	.519	.566	.613	.357	.380	.424	.491	.536	.581
12	.383	.407	.455	.527	.575	.622	.361	.384	.429	.496	.541	.587
13	.389	.413	.462	.535	.583	.632	.364	.387	.433	.501	.547	.592
14	.395	.419	.469	.543	.592	.641	.368	.391	.437	.506	.552	.598
15	.401	.426	.476	.551	.601	.651	.372	.395	.442	.511	.558	.604
16	.407	.432	.483	.559	.610	.661	.375	.399	.446	.516	.563	.610
17	.413	.439	.490	.568	.619	.671	.379	.403	.450	.521	.569	.616
18	.419	.445	.498	.576	.629	.681	.383	.407	.455	.527	.575	.622
19	.426	.452	.505	.585	.639	.692	.387	.411	.459	.532	.580	.629
20	.432	.459	.513	.594	.648	.702	.391	.415	.464	.537	.586	.635
21	.438	.466	.521	.603	.658	.712	.395	.419	.469	.543	.592	.641
22	.445	.473	.528	.612	.668	.723	.399	.424	.473	.548	.598	.648
23	.452	.480	.537	.621	.678	.734	.403	.428	.478	.554	.604	.654
24	.459	.487	.545	.631	.688	.745	.407	.432	.483	.559	.610	.661
25	.466	.495	.553	.640	.698	.757	.411	.437	.488	.565	.616	.668
26	.473	.502	.561	.650	.709	.768	.415	.441	.493	.571	.623	.674
27	.480	.510	.569	.660	.720	.779	.419	.445	.498	.576	.629	.681
28	.487	.517	.578	.670	.731	.791	.423	.450	.503	.582	.635	.688
29	.494	.525	.587	.680	.742	.803	.428	.454	.508	.588	.641	.695
30	.502	.533	.596	.690	.753	.815	.432	.459	.513	.594	.648	.702
31	.509	.541	.605	.700	.764	.828	.436	.463	.518	.600	.654	.709
32	.517	.549	.614	.711	.776	.840	.441	.468	.523	.606	.661	.716
33	.525	.558	.624	.722	.787	.853	.445	.473	.529	.612	.668	.723
34	.533	.566	.633	.733	.799	.866	.450	.478	.534	.618	.674	.731
35	.541	.575	.642	.744	.811	.879	.454	.482	.539	.624	.681	.738
36	.549	.583	.652	.755	.824	.892	.459	.487	.545	.631	.688	.745
37	.557	.592	.662	.766	.836	.906	.463	.492	.550	.637	.695	.753
38	.566	.601	.671	.778	.849	.919	.468	.497	.556	.643	.702	.760
39	.574	.610	.682	.790	.862	.933	.473	.502	.561	.650	.709	.768
40	.583	.619	.692	.802	.875	.947	.477	.507	.567	.656	.716	.776
41	.592	.629	.703	.814	.888	.961	.482	.512	.573	.663	.723	.784
42	.601	.638	.714	.826	.901	.976	.487	.517	.578	.670	.731	.791
43	.610	.648	.724	.839	.915	.991	.492	.523	.584	.676	.738	.799
44	.619	.658	.735	.851	.929	1.006	.497	.528	.590	.683	.745	.807
45	.628	.668	.747	.864	.943	1.021	.502	.533	.596	.690	.753	.815
46	.638	.678	.757	.877	.958	1.037	.507	.539	.602	.697	.760	.824
47	.647	.688	.769	.891	.972	1.052	.512	.544	.608	.704	.768	.832
48	.657	.698	.780	.904	.986	1.068	.517	.549	.614	.711	.776	.840
49	.667	.709	.792	.917	1.001	1.084	.523	.555	.620	.718	.783	.849
50	.677	.720	.805	.931	1.016	1.101	.528	.561	.626	.726	.792	.857

[8] Allen Hazen, in a letter to the writer, says: "As a general proposition I think the notion that western waters are more quiet than eastern waters has no foundation in fact."

K_s is the increased value of K_s' for a pipe t years of age and e is the Naperian base, 2.7183. Expressed differently, formula 22 may be said to indicate that the increase in the retardation is at the rate of 1½ per cent a year, compounded.[9] The decrease in capacity is proportional to the change in $\frac{1}{K_s^{0.526}}$. (See formulas 10 and 11, p. 10.)

Table 7 gives values of K_s for ages of pipe up to 50 years. Figure 8 may also be used in estimating capacity at any age in terms of initial capacity.

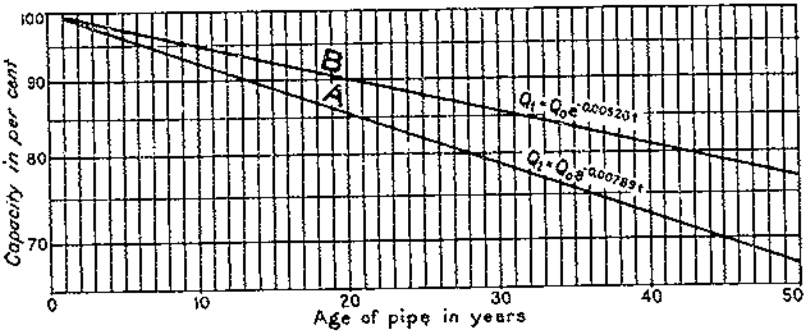

FIGURE 8.—Relative capacity for any age of pipe from 1 to 50 years, assuming capacity when new to be 100 per cent. The A curve is based on the deterioration tendency shown in Figure 7, and expressed in formulas 22 and 23. The B curve is based on formulas 22a and 23a. (See page 89 and 90.)

Where the rate of increase in K_s is at the rate of 1½ per cent a year, compounded (formula 22) then the capacity, Q_t at the end of t years may be expressed as

$$Q_t = Q_0 e^{-0.00789t} \tag{23}$$

and for the less active rate of increase in K_s (formula 22a)

$$Q_t = Q_0 e^{-0.00526t} \tag{23a}$$

For both these formulas Q_t is the capacity for any age, t years, in terms of the capacity Q_0 when the line was new.

CAPACITY OF STEEL PIPES

As with all other conduits, certain losses of head are involved in the conveyance of water in steel pipes. For long lines the losses due to retardation, usually termed friction losses, overshadow all the others. Attention is called to the discussion of entry head, velocity head, air in pipe, etc., in references (*152*, *153*). The discussion here is limited to the so-called friction losses.

The comparison in Table 8, based on the coefficients recommended for the various classes of riveted-steel and analogous pipe, is made for pipes assumed to be coated with the same material. The ordinary lap-riveted plate pipe of moderate thickness classed 1b is used as the basis of comparison, i. e., as 100 per cent, or rated at 100. The same difference is found at any assumed age. The capacity is proportional to $\frac{1}{K_s^{0.526}}$.

[9] From studies of the data on decrease in capacity then available to him, Mills (*122*, p. *222*) came to the tentative conclusion that the relative value of the coefficient of retardation should be increased $0.17\sqrt{t}$ for "tar coated riveted-steel conduits within 1 mile of feeding reservoir," and this increase should be reduced to $0.12\sqrt{t}$ "for portions further down stream." These two curves are shown in Figure 7 for one value of K', only, namely 0.44 for comparison with the writer's formula for time deterioration. It is to be noted that Mills' formulas provide for a rapid deterioration during the first years of operation, and this is borne out by some of the data, notably those for the east Jersey line. (No. 64.)

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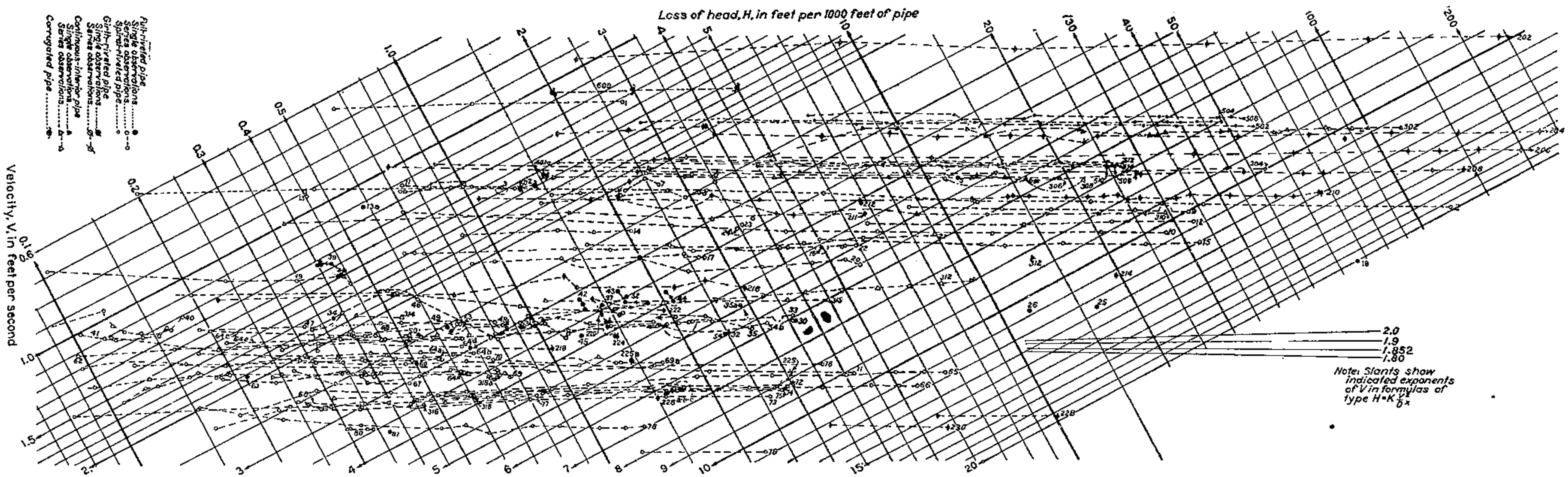


FIGURE 3.—Logarithmic diagram showing observations on riveted steel and analogous pipes

62210°—30. (Face p. 80.)

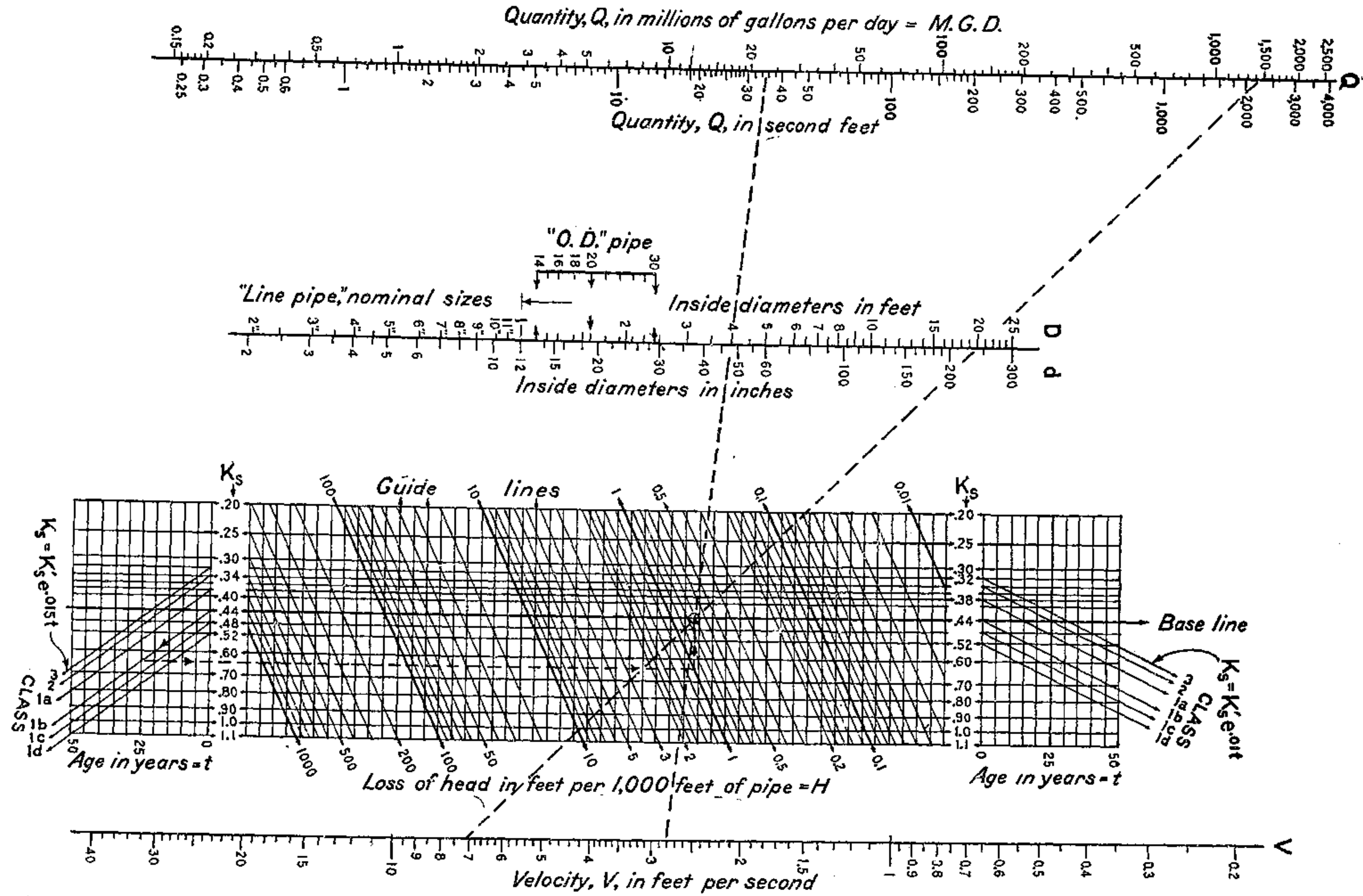
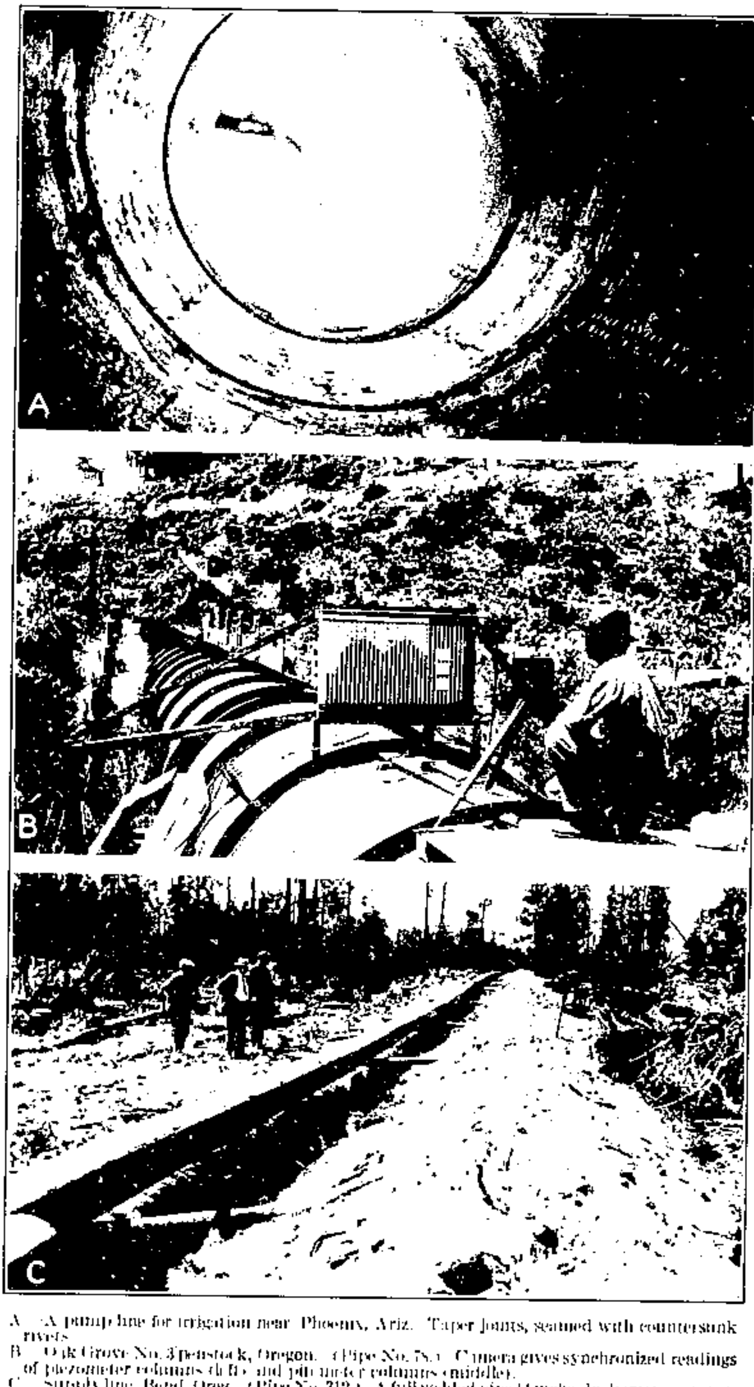
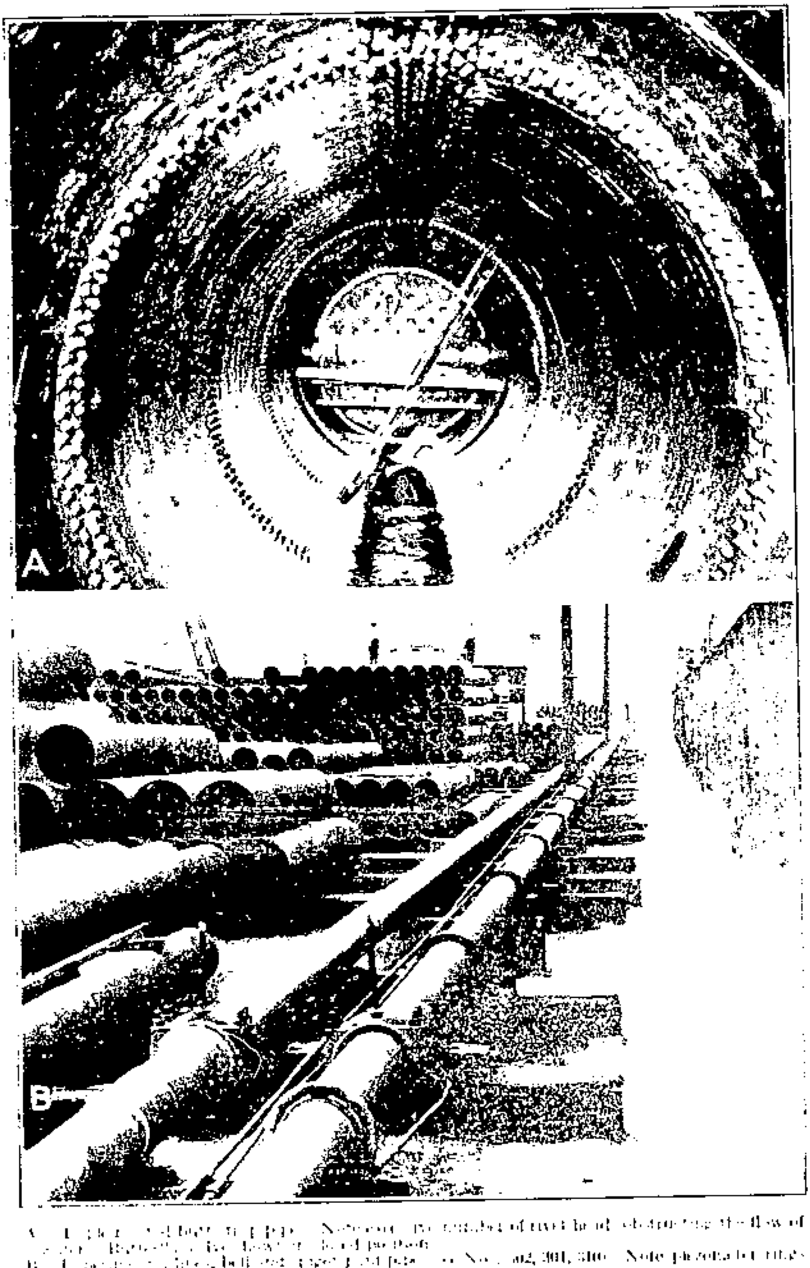


FIGURE 10.—Estimate diagram: hydraulic elements for all sizes, classes, and ages of steel pipe. For example: an ordinary full-riveted pipe of class 1b will be required for conveyance of aggressive water. When new the coefficient K_s' is 0.44, but for a pipe 25 years old coefficient K_s becomes 0.64 since $K_s = K_s' e^{0.015t}$. From the intersection of the 25-year line and the line $K_s = 0.64$ in the left local diagram follow the dashed line to slope line of 1 foot per 1,000 feet of pipe; thence by guide lines to base line. From this point as a pivot a straightedge gives simultaneous values of quantity, Q; diameter, D or d; and velocity, V. From the pivot point as determined above, two diagonals are shown. By following one it will be seen that a 48-inch pipe will carry 35.5 second-feet, or about 23,000,000 gallons per day, at a velocity of 2.82 feet per second. The other diagonal shows that a 20-foot pipe will carry 2,250 second-feet at a velocity of 7.17 feet per second. If nonaggressive waters are to be carried the right local diagram should be used. This is based on $K_s = K_s' e^{0.01t}$. If O. D. (outside diameter) pipe is considered, the proper local diagram will give needed values. It should be noted that for such a type the 30-inch pipe is only 29 inches inside diameter. In a similar way "line pipes" are given nominal sizes slightly different from the true inside diameters. Thus the diameter of a 3-inch line pipe is slightly larger than a true 3-inch diameter, while that of a 2½-inch line pipe is slightly smaller than a true 2½-inch diameter.

62210°—30. (Face p. 91.)



A.—A pump line for irrigation near Phoenix, Ariz. Taper joints, seamed with countersunk rivets.
B.—Oak Grove No. 3 penstock, Oregon. (Pipe No. 78.) Camera gives synchronized readings of piezometer columns (left) and pitometer columns (middle).
C.—Supply line, Bend, Oreg. (Pipe No. 312.) A full welded pipe 14 inches in diameter, reasonably straight in alignment.



A. [illegible] pipe. Note [illegible] of rivet heads obstructing the flow of [illegible] position.
B. [illegible] bell [illegible] pipe [illegible] Nos. [illegible] 302, 301, 300. Note [illegible] Hammer-weld pipe [illegible]

TABLE 8.—*Comparison of capacities of new riveted-steel and analogous pipes assuming the capacity of lap-riveted steel of moderate plate thickness (classed as 1b) as 100 per cent*

[The same percentage relations hold for any given age]

Class	K_s'	$\frac{1}{K_s'^{0.526}}$	Difference from base	Capacity difference	Remarks
				Per cent	
1a	0.38	1.663	+0.123	+7.99	Sheet metal, flat rivets.
1b	.44	1.540	0	0	Used as basis of comparison.
1c	.48	1.471	−.060	−4.48	Full riveted, heavy plates.
1d	.52	1.410	−.130	−8.44	Full riveted, extra heavy.
2	.34	1.764	+.224	+14.55	Girth-riveted pipe.
3	.32	1.821	+.281	+18.25	Continuous-interior pipe.

Figure 9 shows the relative capacities of pipes of identical types, classes, ages, and coatings, and differing only in size.

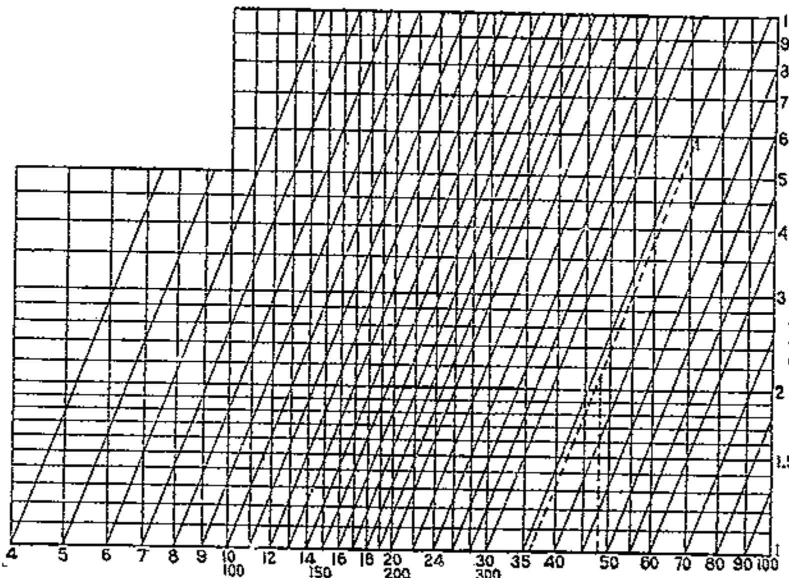


FIGURE 9.—Relative capacities of sheet and plate metal pipe, identical in type, and age, differing only in size. The dotted example shows that a 48-inch pipe will carry about 2.1 times as much as a 36-inch pipe and that a 72-inch pipe will carry almost 6 times as much as a 36-inch pipe

Since the friction loss is at present conceded to increase with passage of time, as a rule, it is not feasible to publish definite tables of capacity for each size of pipe as was done in the bulletins on wood-stave and concrete pipe (*152*, *153*). Instead, tables are given showing values of $V^{1.9}$, $D^{1.1}$, A, and K_s for any age of pipe, from which a rapid solution of formula 9 may be made with the initial values of K_s as modified by formula 12. The diagram on Figure 10 should be useful for preliminary study.[a] It is usually easier to work from the velocity in a particular size of pipe, converting the desired quantity of water into cubic feet per second, and solving the equation $V=\frac{Q}{A}$. For each size of pipe studied, solution of formula 9 will give

[a] Copies of this diagram suitable for mounting for desk use may be obtained by addressing the Bureau of Public Roads, P. O. Box 180, Berkeley, Calif.

the loss of head for each 1,000 feet of pipe. If an age of t years is assumed, values of K_s as given in Table 7 may be used.

FACTORS OF SAFETY

Broadly speaking, it is very difficult to forsee the probable condition of a steel pipe interior 10, 20, or 30 years in the future. The formulas developed appear to satisfy usual conditions quite closely, as can be seen from a casual review of column 15 in Table 4. However, occasional wide divergences are found between the observed and the computed capacity. The following factors of safety, expressed as percentage increases over the designed capacity, are recommended. For example, if 50 second-feet are required and a factor of safety of 10 per cent be used, all computations should be made on the basis of 55 second-feet. The factors are as follows:

Five per cent when only a rough approximation to the actual needs of the pipe is possible, when past experience with steel pipes and a similar water indicates that the water will not be unusually aggressive, when the water carries some abrasive and high velocities will probably insure a clean scoured condition at all times (note the pipes tested by Hamilton Smith), when the line is practically straight, when conditions of operation are such that no penalty attaches to a slight lack of capacity; 15 per cent for conditions intermediate between those above and those below; 25 per cent when the water is known to be very aggressive, when the coating is known to be inferior and the line is close to its reservoir, when the line is very crooked, when a colloidal silt is conveyed by the water and velocities below 3 feet per second are contemplated, when a heavy penalty attaches to appreciable lack of capacity.

Where long experience with a particular water shows positively that little decrease in capacity is to be expected, a rate of increase in the values of K_s may be taken as in part B of Table 7.

TABLE 9.—*Values of $V^{1.9}$ for use in formula $H=K_s\frac{V^{1.9}}{D^{1.1}}$*

[For example, if V=2.84 feet per second then $V^{1.9}$=7.26]

V	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	V
Feet per second											*Feet per second*
0.6	0.379	0.391	0.403	0.416	0.428	0.441	0.454	0.467	0.480	0.494	0.6
.7	.508	.522	.536	.550	.564	.579	.594	.609	.624	.639	.7
.8	.654	.670	.686	.702	.718	.735	.751	.767	.784	.801	.8
.9	.819	.836	.853	.871	.889	.907	.925	.944	.962	.981	.9
1.0	1.000	1.019	1.04	1.06	1.08	1.10	1.12	1.14	1.16	1.18	1.0
1.1	1.198	1.22	1.24	1.26	1.29	1.30	1.32	1.35	1.37	1.39	1.1
1.2	1.414	1.44	1.46	1.48	1.50	1.53	1.55	1.57	1.59	1.62	1.2
1.3	1.645	1.67	1.69	1.72	1.75	1.77	1.80	1.82	1.84	1.87	1.3
1.4	1.895	1.92	1.95	1.97	2.00	2.03	2.05	2.08	2.11	2.13	1.4
1.5	2.161	2.18	2.21	2.24	2.27	2.30	2.33	2.36	2.39	2.41	1.5
1.6	2.44	2.47	2.50	2.53	2.56	2.59	2.62	2.65	2.68	2.71	1.6
1.7	2.74	2.77	2.80	2.83	2.86	2.90	2.93	2.96	2.99	3.02	1.7
1.8	3.06	3.09	3.12	3.15	3.18	3.22	3.25	3.29	3.32	3.35	1.8
1.9	3.39	3.42	3.45	3.49	3.52	3.56	3.59	3.63	3.66	3.70	1.9
2.0	3.73	3.76	3.80	3.83	3.87	3.91	3.94	3.97	4.01	4.05	2.0
2.1	4.09	4.13	4.16	4.20	4.24	4.28	4.32	4.36	4.40	4.44	2.1
2.2	4.48	4.52	4.56	4.59	4.63	4.67	4.71	4.75	4.79	4.83	2.2
2.3	4.87	4.91	4.95	4.99	5.03	5.07	5.11	5.15	5.19	5.23	2.3
2.4	5.27	5.31	5.36	5.40	5.44	5.49	5.53	5.57	5.62	5.66	2.4
2.5	5.70	5.74	5.79	5.83	5.87	5.92	5.96	6.01	6.05	6.10	2.5
2.6	6.14	6.19	6.24	6.28	6.32	6.37	6.41	6.46	6.50	6.55	2.6
2.7	6.60	6.65	6.69	6.74	6.79	6.84	6.89	6.93	6.98	7.03	2.7
2.8	7.07	7.12	7.17	7.22	7.26	7.31	7.36	7.41	7.46	7.51	2.8
2.9	7.56	7.61	7.66	7.71	7.76	7.81	7.86	7.91	7.96	8.01	2.9
3.0	8.06	8.11	8.17	8.22	8.27	8.32	8.38	8.43	8.48	8.53	3.0
3.1	8.58	8.63	8.69	8.74	8.79	8.85	8.90	8.95	9.01	9.06	3.1
3.2	9.12	9.17	9.23	9.28	9.33	9.39	9.44	9.49	9.55	9.61	3.2
3.3	9.66	9.72	9.77	9.83	9.89	9.95	10.00	10.05	10.11	10.17	3.3
3.4	10.23	10.28	10.34	10.40	10.46	10.52	10.57	10.63	10.69	10.75	3.4
3.5	10.81	10.86	10.91	10.97	11.0	11.1	11.2	11.2	11.3	11.4	3.5
3.6	11.40	11.5	11.5	11.6	11.6	11.7	11.8	11.8	11.9	12.0	3.6

TABLE 9.—*Values of* $V^{1.9}$ *for use in formula* $H=K\frac{V^{1.9}}{D^{1.1}}$—Continued

V	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	*V*
Feet per second											*Feet per second*
3.7	12.01	12.1	12.1	12.2	12.3	12.3	12.4	12.5	12.5	12.6	3.7
3.8	12.64	12.7	12.8	12.8	12.9	12.9	13.0	13.1	13.1	13.2	3.8
3.9	13.27	13.3	13.4	13.5	13.5	13.6	13.7	13.7	13.8	13.8	3.9
4.0	13.93	14.0	14.1	14.1	14.2	14.3	14.3	14.4	14.4	14.5	4.0
4.1	14.60	14.7	14.7	14.8	14.9	14.9	15.0	15.1	15.1	15.2	4.1
4.2	15.28	15.4	15.4	15.5	15.6	15.6	15.7	15.7	15.8	15.9	4.2
4.3	15.98	16.1	16.1	16.2	16.3	16.3	16.4	16.5	16.6	16.6	4.3
4.4	16.69	16.8	16.8	16.9	17.0	17.1	17.1	17.2	17.3	17.3	4.4
4.5	17.42	17.5	17.6	17.7	17.7	17.8	17.9	18.0	18.0	18.1	4.5
4.6	18.17	18.2	18.3	18.4	18.4	18.5	18.6	18.7	18.8	18.8	4.6
4.7	18.92	19.0	19.1	19.1	19.2	19.3	19.4	19.5	19.5	19.6	4.7
4.8	19.70	19.8	19.9	20.0	20.0	20.1	20.2	20.2	20.3	20.4	4.8
4.9	20.48	20.6	20.7	20.7	20.8	20.9	20.9	21.0	21.1	21.2	4.9
5.0	21.28	21.4	21.4	21.5	21.6	21.7	21.8	21.9	21.9	22.0	5.0
5.1	22.10	22.2	22.3	22.3	22.4	22.5	22.6	22.7	22.8	22.9	5.1
5.2	22.93	23.0	23.1	23.2	23.3	23.4	23.4	23.5	23.6	23.7	5.2
5.3	23.78	23.9	23.9	24.0	24.1	24.2	24.3	24.4	24.5	24.6	5.3
5.4	24.63	24.7	24.8	24.9	25.0	25.1	25.2	25.2	25.3	25.4	5.4
5.5	25.51	25.6	25.7	25.8	25.9	25.9	26.0	26.1	26.2	26.3	5.5
5.6	26.40	26.5	26.6	26.7	26.8	26.8	26.9	27.0	27.1	27.2	5.6
5.7	27.30	27.4	27.5	27.6	27.7	27.8	27.9	27.9	28.0	28.1	5.7
5.8	28.22	28.3	28.4	28.5	28.6	28.7	28.8	28.9	29.0	29.1	5.8
5.9	29.15	29.2	29.3	29.4	29.5	29.6	29.7	29.8	29.9	30.0	5.9
6.0	30.09	30.2	30.3	30.4	30.5	30.6	30.7	30.8	30.9	31.0	6.0
6.1	31.05	31.2	31.3	31.3	31.4	31.5	31.6	31.7	31.8	31.9	6.1
6.2	32.03	32.1	32.2	32.3	32.4	32.5	32.6	32.7	32.8	32.9	6.2
6.3	33.02	33.1	33.2	33.3	33.4	33.5	33.6	33.7	33.8	33.9	6.3
6.4	34.02	34.1	34.2	34.3	34.4	34.5	34.6	34.7	34.8	34.9	6.4
6.5	35.04	35.1	35.2	35.3	35.4	35.5	35.7	35.8	35.9	36.0	6.5
6.6	36.07	36.2	36.3	36.4	36.5	36.6	36.7	36.8	36.9	37.0	6.6
6.7	37.11	37.2	37.3	37.4	37.5	37.6	37.8	37.9	38.0	38.1	6.7
6.8	38.17	38.3	38.4	38.5	38.6	38.7	38.8	38.9	39.0	39.1	6.8
6.9	39.25	39.4	39.5	39.6	39.7	39.8	39.9	40.0	40.1	40.2	6.9
7.0	40.34	40.4	40.6	40.7	40.8	40.9	41.0	41.1	41.2	41.3	7.0
7.1	41.44	41.6	41.7	41.8	41.9	42.0	42.1	42.2	42.3	42.4	7.1
7.2	42.55	42.7	42.8	42.9	43.0	43.1	43.2	43.4	43.5	43.6	7.2
7.3	43.68	43.8	43.9	44.0	44.1	44.3	44.4	44.5	44.6	44.7	7.3
7.4	44.83	44.9	45.1	45.2	45.3	45.4	45.5	45.6	45.8	45.9	7.4
7.5	45.98	46.1	46.2	46.3	46.5	46.6	46.7	46.8	46.9	47.0	7.5
7.6	47.16	47.3	47.4	47.5	47.6	47.8	47.9	48.0	48.1	48.2	7.6
7.7	48.34	48.5	48.6	48.7	48.8	49.0	49.1	49.2	49.3	49.4	7.7
7.8	49.54	49.7	49.8	49.9	50.0	50.1	50.3	50.4	50.5	50.6	7.8
7.9	50.76	50.9	51.0	51.1	51.3	51.4	51.5	51.6	51.8	51.9	7.9
8.0	51.98	52.1	52.2	52.4	52.5	52.6	52.7	52.8	53.0	53.1	8.0
8.1	53.23	53.3	53.5	53.6	53.7	53.8	54.0	54.1	54.2	54.3	8.1
8.2	54.48	54.6	54.7	54.8	55.0	55.1	55.2	55.3	55.5	55.6	8.2
8.3	55.75	55.9	56.0	56.1	56.2	56.4	56.5	56.6	56.8	56.9	8.3
8.4	57.03	57.2	57.3	57.4	57.5	57.7	57.8	57.9	58.1	58.2	8.4
8.5	58.33	58.5	58.6	58.8	58.9	59.0	59.1	59.2	59.4	59.5	8.5
8.6	59.64	59.8	59.9	60.0	60.1	60.3	60.4	60.5	60.7	60.8	8.6
8.7	60.96	61.1	61.2	61.4	61.5	61.7	61.8	61.9	62.0	62.2	8.7
8.8	62.30	62.4	62.5	62.7	62.8	63.0	63.1	63.2	63.4	63.5	8.8
8.9	63.66	63.8	63.9	64.1	64.2	64.3	64.4	64.6	64.7	64.9	8.9
9.0	65.02	65.2	65.3	65.4	65.6	65.7	65.9	66.0	66.1	66.2	9.0
9.1	66.40	66.5	66.7	66.8	67.0	67.1	67.2	67.4	67.5	67.7	9.1
9.2	67.80	67.9	68.1	68.2	68.4	68.5	68.6	68.8	68.9	69.1	9.2
9.3	69.20	69.3	69.5	69.6	69.8	69.9	70.0	70.2	70.3	70.5	9.3
9.4	70.62	70.8	70.9	71.1	71.2	71.3	71.5	71.6	71.8	71.9	9.4
9.5	72.06	72.2	72.4	72.5	72.6	72.8	72.9	73.1	73.2	73.3	9.5
9.6	73.50	73.6	73.8	73.9	74.1	74.2	74.4	74.5	74.7	74.8	9.6
9.7	74.96	75.1	75.3	75.4	75.5	75.7	75.8	76.0	76.1	76.3	9.7
9.8	76.44	76.6	76.7	76.9	77.0	77.2	77.3	77.5	77.6	77.8	9.8
9.9	77.93	78.1	78.2	78.4	78.5	78.7	78.8	78.9	79.1	79.3	9.9

V	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	*V*
10.0	79.43	81.0	82.5	84.0	85.6	87.1	88.7	90.3	91.9	93.6	10.0
11.0	95.20	96.9	98.5	100.2	101.9	103.6	105.3	107.0	108.7	110.5	11.0
12.0	112.3	114.1	115.9	117.7	119.5	121.3	123.2	125.1	127.0	128.9	12.0
13.0	130.8	132.7	134.6	136.6	138.6	140.5	142.5	144.5	146.5	148.5	13.0
14.0	150.5	152.6	154.6	156.7	158.8	160.9	163.0	165.1	167.3	169.4	14.0
15.0	171.6	173.8	176.0	178.2	180.4	182.7	184.9	187.2	189.4	191.7	15.0
16.0	194.0	196.3	198.6	201.0	203.3	205.7	208.1	210.5	212.9	215.3	16.0
17.0	217.7	220.2	222.6	225.1	227.5	230.0	232.6	235.1	237.6	240.1	17.0
18.0	242.7	245.2	247.8	250.4	253.0	255.7	258.3	260.9	263.6	266.2	18.0
19.0	268.9	271.6	274.4	277.1	279.8	282.5	285.3	288.0	290.8	293.6	19.0

TABLE 10.—*Values of $D^{1.1}$ for use in the formula* $H=K_s\frac{V^{1.9}}{D^{1.1}}$

[Example, $d=38$ inches, is the equivalent of $D=3.167$ feet, for which $D^{1.1}=3.55$ feet]

[For use in the determination of $V=\frac{Q}{A}$ the values of area, A are given. For $d=38$ inches, $A=7.88$ square feet]

Diameter		$D^{1.1}$	Area A	Diameter		$D^{1.1}$	Area A	Diameter		$D^{1.1}$	Area A	Diameter		$D^{1.1}$	Area A
d	D			d	D			d	D			d	D		
Ins.	*Ft.*	*Ft.*	*Sq. ft.*	*Ins.*	*Ft.*	*Ft.*	*Sq. ft.*	*Ins.*	*Ft.*	*Ft.*	*Sq. ft.*	*Ins.*	*Ft.*	*Ft.*	*Sq. ft.*
4.0	0.333	0.298	0.087	26.0	2.167	2.34	3.69	56.0	4.667	5.44	17.10	108.0	9.0	11.2	63.62
4.5	.375	.340	.110	27.0	2.250	2.44	3.98	57.0	4.750	5.55	17.72	114.0	9.5	11.9	70.88
5.0	.417	.382	.137	28.0	2.333	2.54	4.28	58.0	4.833	5.66	18.35	120.0	10.0	12.6	78.54
5.5	.458	.424	.165	29.0	2.417	2.64	4.59	59.0	4.917	5.77	18.99	126.0	10.5	13.3	86.59
6.0	.500	.467	.196	30.0	2.500	2.74	4.91	60.0	5.000	5.87	19.63	132.0	11.0	14.0	95.03
6.5	.542	.510	.230	31.0	2.583	2.84	5.24	61.0	5.083	5.98	20.29	138.0	11.5	14.7	103.9
7.0	.583	.552	.267	32.0	2.667	2.94	5.58	62.0	5.167	6.09	20.97	144.0	12.0	15.4	113.1
7.5	.625	.596	.307	33.0	2.750	3.04	5.94	63.0	5.250	6.20	21.65	150.0	12.5	16.1	122.7
8.0	.667	.640	.349	34.0	2.833	3.15	6.30	64.0	5.333	6.31	22.34	156.0	13.0	16.8	132.7
8.5	.708	.684	.394	35.0	2.917	3.25	6.68	65.0	5.417	6.41	23.04	162.0	13.5	17.5	143.1
9.0	.750	.729	.442	36.0	3.000	3.35	7.07	66.0	5.500	6.52	23.76	168.0	14.0	18.2	153.9
9.5	.792	.774	.493	37.0	3.083	3.45	7.47	67.0	5.583	6.63	24.48	174.0	14.5	18.9	165.1
10.0	.833	.818	.545	38.0	3.167	3.55	7.88	68.0	5.667	6.74	25.22	180.0	15.0	19.7	176.7
10.5	.875	.863	.601	39.0	3.250	3.66	8.30	69.0	5.750	6.85	25.97	186.0	15.5	20.4	188.7
11.0	.917	.909	.660	40.0	3.333	3.76	8.73	70.0	5.833	6.96	26.74	192.0	16.0	21.1	201.1
11.5	.958	.954	.721	41.0	3.417	3.86	9.17	71.0	5.917	7.07	27.50	198.0	16.5	21.8	213.8
12.0	1.000	1.000	.785	42.0	3.500	3.97	9.62	72.0	6.000	7.18	28.27	204.0	17.0	22.6	227.0
13.0	1.083	1.09	.922	43.0	3.583	4.07	10.08	74.0	6.167	7.40	29.87	210.0	17.5	23.3	240.5
14.0	1.167	1.19	1.069	44.0	3.667	4.17	10.56	76.0	6.333	7.62	31.50	216.0	18.0	24.0	254.5
15.0	1.250	1.28	1.227	45.0	3.750	4.28	11.04	78.0	6.500	7.84	33.18	222.0	18.5	24.8	268.8
16.0	1.333	1.37	1.396	46.0	3.833	4.38	11.54	80.0	6.667	8.06	34.91	228.0	19.0	25.5	283.5
17.0	1.417	1.47	1.576	47.0	3.917	4.49	12.05	82.0	6.833	8.28	36.67	234.0	19.5	26.2	298.6
18.0	1.500	1.56	1.767	48.0	4.000	4.59	12.57	84.0	7.000	8.51	38.48	240.0	20.0	27.0	314.2
19.0	1.583	1.66	1.969	49.0	4.083	4.70	13.10	86.0	7.167	8.73	40.34	246.0	20.5	27.7	330.1
20.0	1.667	1.75	2.182	50.0	4.167	4.81	13.64	88.0	7.333	8.95	42.24	252.0	21.0	28.5	346.4
21.0	1.750	1.85	2.405	51.0	4.250	4.91	14.19	90.0	7.500	9.18	44.18	258.0	21.5	29.2	363.1
22.0	1.833	1.95	2.640	52.0	4.333	5.02	14.75	92.0	7.666	9.40	46.16	264.0	22.0	30.0	380.1
23.0	1.917	2.05	2.885	53.0	4.417	5.12	15.32	94.0	7.833	9.62	48.19	270.0	23.0	31.5	415.5
24.0	2.000	2.14	3.142	54.0	4.500	5.23	15.90	96.0	8.000	9.85	50.27	288.0	24.0	33.0	452.4
25.0	2.083	2.24	3.409	55.0	4.583	5.34	16.50	102.0	8.500	10.53	56.74	300.0	25.0	34.5	490.9

ESTIMATE TABLES AND DIAGRAM AND SOLUTION OF TYPICAL PIPE PROBLEMS

Problem 1.—An inverted-siphon pipe is required to carry 3.5 second-feet of water for irrigation across a depression 40 feet deep. The siphon will connect sections of open concrete-lined canal. The distance, as measured along the line to be followed by the pipe, is 273 feet. A total drop in water surface between water in the intake end and water at the outlet end of the siphon is 2.5 feet. Required: The size of thin-sheet flat-head riveted pipe needed to convey the desired quantity of water when the pipe is 20 years old on the basis of deterioration as in formula 22.

Design for an overload of 15 per cent as a factor of safety. Thus quantity designed for will be $1.15\times 3.5=4.03$ second-feet. Try a 12-inch pipe. From Table 10, the area, $A=0.785$ square feet. Hence, the velocity would be $\frac{4.03}{0.785}=5.14$ feet per second, for which the velocity head equals to $\frac{V^2}{2g}=0.411$. The velocity head and entry head necessary to generate the velocity and to get the water into the pipe at the inlet end will be about 1.5 times the velocity head, or $1.5\times 0.411=0.617$ feet. The difference in water surface available for he friction loss becomes $2.50-0.617=1.883$ feet. Since the pipe vill be 273 feet long the loss per 1,000 feet would be $\frac{1.883}{0.273}=6.90$ feet. The type of pipe required would come under class 1a, for which K_s when the pipe is new is 0.38 but when the pipe is 20 years old would be 0.513 (from Table 7A). Substituting in formula 9, page 10, the oss of head per 1,000 feet would be

$$H=K_s\frac{V^{1.9}}{D^{1.1}}=0.513\,\frac{5.14^{1.9}}{\left(\frac{12}{12}\right)^{1.1}}=0.513\,\frac{22.4}{1.000}\begin{pmatrix}\text{from Table 9}\\ \text{from Table 10}\end{pmatrix}=11.5 \text{ feet.}$$

which requires more head than is available, hence a 12-inch pipe is too small.

Try a 14-inch pipe in the same way. The area $A=1.069$ square feet. The velocity will be $\frac{4.03}{1.069}=3.77$ feet per second, for which the velocity head is $\frac{3.77^2}{2g}=0.221$ feet. The combined velocity and entry heads will be taken as $1.5\times 0.221=0.331$ feet. The fall available for friction loss becomes $2.50-0.331=2.169$ feet. The loss per 1,000 feet will be $\frac{2.169}{0.273}=7.94$ feet. Substitution in formula 9, as before gives

$$H=K_s\frac{V^{1.9}}{D^{1.1}}=0.513\,\frac{3.77^{1.9}}{\left(\frac{14}{12}\right)^{1.1}}=0.513\,\frac{12.5}{1.19}\begin{pmatrix}\text{from Table 9}\\ \text{from Table 10}\end{pmatrix}=5.39.$$

Since the pipe is only 273 feet long the actual friction loss would be 1.47 feet while the fall available for friction would be 2.169 feet. Thus there is ample margin of safety and a 14-inch pipe of this type will be slightly oversize. If available, it will be found that a 13-inch pipe would satisfy the requirements reasonably closely. After determining the fall per 1,000 feet of pipe available for friction losses, then any of the usual problems can be solved by means of the nomogram, Figure 10.

Problem 2.—A pipe line, to convey 50,000,000 gallons per day 20 years hence, for a distance of 10 miles through open country calling for little curvature, with a total available fall of 65 feet, is required to convey "active" water for municipal purposes. The pressure head is between 50 and 100 feet. What will be the required diameter of (1) riveted-steel pipe, (2) lock bar pipe with riveted girth seams?

For the conditions involved assume a factor of safety of 15 per cent. The required capacity will be $50+7.5=57.5$ million gallons per day, which is equal to approximately 89 second-feet. From page 12 it is found that a riveted pipe of class 1b, having a coefficient of retardation of $K_s=0.44$ when new, can be considered while the lock-bar pipe of class 2 will have a coefficient when new of $K_s=0.34$.

From Table 7A the value of $K_s=0.44$ when new becomes 0.594 for a pipe 20 years old and the value of K_s of 0.34 for the new lock-bar pipe becomes 0.459 in 20 years. Of the 65 feet total fall available assume 3 feet are to be lost at valves, screens, and meter.

This leaves 62 feet for friction losses. For each 1,000 feet of line this will be $\frac{62}{5.280\times 10}=1.174$ feet. Substituting in formula 9, page 10:

$$1.174=0.594\,\frac{V^{1.9}}{D^{1.1}},\ \text{or}\ 1.96\,D^{1.1}=V^{1.9}$$

From the continuity equation $Q=AV=89$ second-feet.

Table 11 shows the results of tentative trials of different sizes of riveted-steel pipe using data from Tables 9 and 10.

TABLE 11.—*Tentative trials of different sizes of riveted-steel pipe—problem 2*

d	D	$D^{1.1}$	$1.96\,D^{1.1}=V^{1.9}$	V	A	$Q=AV$	Remarks
Inches	*Feet*	*Feet*		*Feet per second*	*Square feet*	*Second-feet*	
54	4.50	5.23	10.25	3.40	15.90	54.1	Too small.
60	5.00	5.87	11.50	3.62	19.64	71.2	Do.
66	5.50	6.52	12.78	3.82	23.76	90.8	Too large.
64	5.33	6.31	12.36	3.75	22.6	84.8	Too small.
65	5.42	6.41	12.56	3.78	23.4	88.5	Just under 89. Better use 66-inch.

In a similar way, for the lock-bar pipe:

$$1.174=0.459\,\frac{V^{1.9}}{D^{1.1}}\ \text{or}\ 2.56\,D^{1.1}=V^{1.9}\ \text{and}\ Q=AV=89\ \text{as before.}$$

Table 12 shows the results of trials of different sizes of this type of pipe.

TABLE 12.—*Tentative trials of different sizes of lock-bar pipe—problem 2*

d	D	$D^{1.1}$	$2.56\,D^{1.1}=V^{1.9}$	V	A	$Q=AV$	Remarks
Inches	*Feet*	*Feet*		*Feet per second*	*Square feet*	*Second-feet*	
60	5.00	5.87	15.03	4.16	19.64	81.7	Too small.
64	5.33	6.31	16.15	4.32	22.6	97.7	Too large.
61	5.08	5.98	15.30	4.20	20.6	86.5	Too small.
62	5.17	6.09	15.60	4.24	21.0	89.0	Just right.

For the particular problem stated, the pipe of class 2 has the advantage to the extent of from 3 to 4 inches in diameter.

COMPARISON OF CAPACITIES, RIVETED STEEL AND ANALOGOUS PIPES WITH CAST-IRON, CONCRETE, AND WOOD-STAVE PIPES

Table 13 has been prepared for the purpose of comparing pipes of various types and materials in the important matter of carrying capacity. With the exception of cast-iron pipe all types and materials in general use have been investigated by the Division of Agricultural Engineering and the tables are based on the results of the investigations. From this table curves can be quickly developed on logarithmic paper for a comparison of any sizes of pipes for any coefficients between those listed. This table likewise permits a comparison between results as computed by formulas here recommended and results as computed by the Williams–Hazen or the Kutter formula.

For pipes of the types investigated in this study basic values are given in columns 3 to 11, inclusive. The proper value of K_s will be determined by the class and age of the pipe considered. (See Effect of Age on Carrying Capacity, page 88.) For cast-iron pipe, reference is made to the latest recommendations of Williams and Hazen (*180, p. 22*): "For new cast-iron pipe 130 (in the Williams–Hazen formula) remains the appropriate value; but 140 is sometimes reached, and there is a record of one pipe with a value of 147." Williams and Hazen point out that all iron and steel pipe deteriorates in capacity with age, and suggest that a fair value for computation of cast-iron lines is a coefficient of 100, and for steel pipe a coefficient of 95.

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TABLE 13.—*Velocities as computed by various formulas for given sizes of pipe with given friction heads*

Diameter (*d*)	Friction head (*H*)	Velocities in feet per second																							
		Scobey, K_s for metal pipe									William-Hazen, C_w						Kutter, *n*				Scobey				
																					C_s Concrete pipe				Wood stave pipe
		0.30	0.32	0.34	0.36	0.38	0.40	0.44	0.48	0.52	140	130	120	110	100	90	0.009	0.011	0.013	0.015	0.400	0.370	0.345	0.310	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26
Inches	*Feet*																								
6	5.0	2.94	2.85	2.75	2.68	2.60	2.53	2.41	2.30	2.20	2.86	2.65	2.45	2.24	2.04	1.83	2.93	2.23	1.78	1.46	2.74	2.54	2.36	2.12	2.52
6	10.0	4.24	4.10	3.97	3.86	3.74	3.64	3.47	3.31	3.18	4.15	3.85	3.55	3.26	2.96	2.67	4.15	3.17	2.52	2.07	3.88	3.59	3.34	3.00	3.70
6	20.0	6.10	5.91	5.72	5.55	5.39	5.25	4.99	4.77	4.57	6.02	5.59	5.16	4.73	4.30	3.87	5.88	4.49	3.57	2.93	5.48	5.07	4.73	4.25	5.45
12	1.0	1.88	1.82	1.76	1.71	1.66	1.62	1.54	1.47	1.41	1.85	1.72	1.59	1.45	1.32	1.19	2.16	1.67	1.35	1.12	1.89	1.75	1.63	1.46	1.62
12	5.0	4.40	4.24	4.11	3.99	3.88	3.77	3.59	3.43	3.29	4.40	4.09	3.78	3.46	3.15	2.83	4.89	3.79	3.06	2.54	4.23	3.91	3.63	3.28	3.96
12	10.0	6.33	6.12	5.93	5.75	5.59	5.44	5.17	4.94	4.74	6.40	5.95	5.49	5.04	4.58	3.14	6.93	5.37	4.33	3.60	5.98	5.53	5.15	4.63	5.87
30	.5	2.22	2.15	2.09	2.02	1.96	1.91	1.82	1.74	1.66	2.27	2.10	1.94	1.78	1.62	1.45	2.90	2.23	1.85	1.56	2.37	2.19	2.05	1.84	2.09
30	1.0	3.20	3.10	3.00	2.91	2.83	2.75	2.62	2.50	2.40	3.30	3.06	2.82	2.58	2.36	2.12	4.07	3.23	2.65	2.24	3.35	3.10	2.89	2.60	2.94
30	8.0	9.57	9.25	8.96	8.69	8.45	8.23	7.82	7.47	7.16	10.13	9.40	8.68	7.96	7.24	6.51	11.64	9.21	7.57	6.37	9.48	8.77	8.18	7.34	9.33
		0.32	0.36	0.40	0.44	0.48	0.52	0.56	0.60	0.64							0.011	0.013	0.015	0.017					
48	.2	1.74	1.64	1.55	1.47	1.41	1.35	1.30	1.25	1.21	1.86	1.73	1.59	1.46	1.33	1.19	1.93	1.60	1.36	1.18	2.01	1.86	1.73	1.56	1.63
48	1.0	4.06	3.82	3.61	3.43	3.28	3.15	3.03	2.92	2.82	4.43	4.12	3.80	3.48	3.17	2.85	4.44	3.68	3.13	2.72	4.50	4.16	3.88	3.48	3.99
48	3.0	7.24	6.81	6.44	6.12	5.85	5.61	5.39	5.20	5.03	8.02	7.45	6.87	6.30	5.72	5.15	7.73	6.42	5.46	4.74	7.79	7.20	6.72	6.03	7.34
72	.2	2.20	2.07	1.96	1.86	1.78	1.71	1.64	1.58	1.53	2.40	2.23	2.06	1.89	1.71	1.54	2.54	2.12	1.81	1.58	2.59	2.40	2.23	2.01	2.13
72	1.0	5.14	4.83	4.56	4.35	4.16	3.98	3.83	3.69	3.56	5.72	5.31	4.90	4.49	4.09	3.68	5.80	4.84	4.14	3.62	5.79	5.36	5.00	4.49	5.20
72	2.0	7.41	6.96	6.58	6.26	5.98	5.73	5.51	5.31	5.14	8.31	7.72	7.12	6.53	5.94	5.35	8.23	6.86	5.87	5.12	8.19	7.58	7.07	6.35	7.64
96	.2	2.60	2.45	2.31	2.20	2.10	2.02	1.94	1.87	1.81	2.88	2.67	2.47	2.26	2.06	1.85	3.07	2.58	2.22	1.94	3.10	2.87	2.67	2.40	2.57
96	1.0	6.07	5.71	5.40	5.14	4.90	4.70	4.52	4.36	4.21	6.86	6.37	5.88	5.39	4.90	4.41	6.95	5.84	5.02	4.40	6.93	6.41	5.98	5.38	6.27
96	1.5	7.52	7.06	6.68	6.36	6.07	5.82	5.60	5.40	5.22	8.54	7.93	7.32	6.71	6.10	5.49	8.52	7.16	6.16	5.39	8.49	7.85	7.32	6.58	7.84

		0.36	0.40	0.44	0.48	0.52	0.56	0.60	0.64	0.68							0.014	0.015	0.017	0.020					
120	.2	2.78	2.63	2.51	2.40	2.29	2.21	2.13	2.06	1.99	3.32	3.08	2.84	2.61	2.37	2.13	2.77	2.58	2.27	1.91	3.57	3.30	3.08	2.76	2.97
120	.6	4.97	4.70	4.47	4.27	4.09	3.93	3.80	3.66	3.55	5.99	5.56	5.13	4.71	4.28	3.85	4.83	4.49	3.95	3.34	6.18	5.71	5.33	4.79	5.45
120	1.0	6.50	6.15	5.84	5.58	5.35	5.15	4.97	4.80	4.65	7.90	7.33	6.76	6.20	5.64	5.07	6.24	5.81	5.11	4.32	7.97	7.37	6.88	6.18	7.23
144	.2	3.10	2.93	2.78	2.66	2.55	2.45	2.36	2.29	2.21	3.71	3.45	3.18	2.92	2.65	2.39	3.13	2.92	2.57	2.18	4.00	3.70	3.45	3.10	3.33
144	.6	5.52	5.22	4.97	4.74	4.55	4.37	4.21	4.07	3.95	6.71	6.24	5.76	5.28	4.80	4.32	5.43	5.06	4.46	3.78	6.92	6.40	5.97	5.36	6.14
144	1.0	7.22	6.83	6.50	6.21	5.95	5.72	5.52	5.34	5.16	8.85	8.22	7.58	6.95	6.32	5.69	7.00	6.53	5.76	4.89	8.94	8.27	7.71	6.93	8.15
180	.1	2.45	2.31	2.20	2.10	2.01	1.94	1.87	1.81	1.75	2.94	2.73	2.52	2.31	2.10	1.89	2.56	2.39	2.12	1.81	3.25	3.00	2.80	2.52	2.62
180	5	5.71	5.40	5.14	4.90	4.70	4.52	4.36	4.21	4.09	7.00	6.50	6.00	5.50	5.00	4.50	5.70	5.32	4.71	4.02	7.26	6.72	6.27	5.63	6.41
180	1.0	8.22	7.78	7.40	7.06	6.77	6.52	6.28	6.07	5.88	10.18	9.46	8.73	8.00	7.27	6.54	8.06	7.52	6.65	5.69	10.27	9.50	8.86	7.96	9.42
240	.1	2.89	2.73	2.60	2.48	2.38	2.29	2.21	2.14	2.07	3.52	3.27	3.02	2.77	2.51	2.26	3.10	2.90	2.58	2.21	3.89	3.60	3.35	3.01	3.16
240	.5	6.74	6.37	6.06	5.79	5.56	5.34	5.15	4.98	4.81	8.39	7.79	7.19	6.59	6.00	5.40	6.82	6.38	5.66	4.84	8.69	8.04	7.50	6.74	7.73
240	1.0	9.70	9.18	8.73	8.34	7.99	7.69	7.42	7.17	6.94	12.21	11.33	10.46	9.59	8.72	7.85	9.61	9.00	7.98	6.83	12.29	11.37	10.60	9.53	11.35
288	.1	3.20	3.03	2.88	2.75	2.64	2.54	2.45	2.37	2.29	3.95	3.67	3.38	3.10	2.82	2.54	3.49	3.27	2.91	2.51	4.36	4.03	3.76	3.38	-------
288	.4	6.65	6.29	5.98	5.71	5.48	5.27	5.07	4.91	4.76	8.35	7.75	7.15	6.56	5.96	5.36	6.82	6.39	5.68	4.88	8.72	8.06	7.52	6.75	-------
288	.8	9.57	9.05	8.61	8.23	7.89	7.59	7.32	7.07	6.85	12.14	11.27	10.40	9.53	8.67	7.80	9.62	9.01	8.00	6.87	12.33	11.40	10.63	9.55	-------

The use of the Kutter formula for closed conduits is discouraged by nearly all present-day authorities on the flow of water in pipes and channels; however, there are engineers who prefer to use it at least for comparison. The writer wishes to point out, however, that the value of Kutter's n corresponding to a 6-inch continuous-interior steel pipe is approximately 0.009, but when found for a 6-foot pipe $n=0.012\pm$. Likewise, while $n=0.015$ is about right for a new riveted pipe 8 feet in diameter, a value of 0.013 would satisfy the conditions with the lower velocities in the same riveted pipe if its diameter were only 30 inches.

In columns 22 to 25, inclusive of Table 13 are found data on velocities in various grades of concrete pipes. After the writer's investigation of the flow in concrete pipe, coefficient and data as shown in column 23 were recommended for the highest grade pipe and lined tunnels, but more recent experiments indicate that flow as shown in column 22 is attained. While no definite data have been offered as to deterioration in capacity of concrete pipe it is advisable to provide for such reduction by means of a factor of safety applied as a design overload unless previous experience with the particular water carried shows that no deterioration is to be expected. When the modern concrete pipe line has attained a reasonable age there will be more opportunity of settling the question as to the extent of any capacity reduction. The wood-stave pipe also does not appear to sustain any marked degree of capacity reduction from time deterioration and the writer believes the data in column 26 will be found to represent within 5 per cent the actual flow in most stave lines up to 15 years of age.

CONCLUSIONS

There is a material difference in the carrying capacities of steel pipes. Other things being equal, the difference is due to the type of unit assembly and the field joints.

Present indications, based on the performance of pipes of various ages, are that all iron and steel pipes lose capacity progressively when in use. Time alone will determine the extent of immunity afforded by some of the newer coatings. (This paragraph does not apply when it conflicts with the third paragraph below.)

The general term "riveted pipe" is insufficiently distinguishing for use in capacity specifications. Plate thickness, type and size of rivets, and method of making joints all have noticeable influence on capacity.

In irrigation use, where pipes are fed by canal water, extensive silt deposits may be expected unless scouring velocities are available. For velocities under about 5 feet per second many of the irrigation pipes of diameters under 14 inches, when tested showed marked lack of capacity compared with the same types of pipe under conditions which did not contribute silt deposits.

No deterioration in capacity need be anticipated throughout the life of a pipe carrying water containing abrasive detritus at velocities above 10 feet per second. It is quite probable that neither coating nor tuberculation could survive the erosion. However, the life of the pipe would be shortened by the scouring action.

The capacities of riveted steel and analogous pipes, when new, fit in with comparable categories of other materials to an extent that leaves little room for dispute. Future research and improvement should be directed toward the retention of the original capacity by preservation of the coating, which should protect the steel from chemical action regardless of the activity of the water or the structure of the steel. Loss of capacity caused by such natural barriers as débris or silt is, of course, common to all pipes, although plate offsets and rivet heads offer initial obstructions which tend to produce such accumulations in greater degree than in some other kinds of pipes, other factors remaining equal.

Assuming the capacity of full-riveted pipe with plate less than one-half inch thick as 100 per cent, then continuous-interior pipe (class 3) without rivet heads will carry about 18 per cent more, and girth-riveted pipe, but continuous on the straight seams (class 2), will carry nearly 15 per cent more. With the same base, thin-sheet (gauge-metal) pipes with flat-head rivets will carry 8 per cent more; heavy plate pipe of cylinder or taper joints will carry about 4 per cent less; and heavy plate, butt-strap pipe will carry about 8 per cent less.

ACKNOWLEDGMENTS

The writer desires to acknowledge indebtedness to the various engineers and managers of irrigation, municipal, and power systems who permitted and aided in tests upon the pipes in their charge. Acknowledgment is also made to the engineers of the United States Bureau of Reclamation for active cooperation. Where data have been secured from other sources the necessary references are given in the literature citations.

APPENDIX 1

The following pages are devoted to abstracts of descriptions of experiments made by agencies other than the Division of Agricultural Engineering, Bureau of Public Roads. The number before each description refers to the corresponding numbers in column 1, Tables 1, 2, 3, and 4.

FULL RIVETED PIPE

No. 2.—7.71-inch wrought-iron riveted-sheet pipe—Paris, France (*35; 159; 67; 122, p. 135*)—Capacity +11.4 per cent

This line of pipe, 365.3 feet long, was tested when new by H. Darcy, in 1851. The pipe units, about 9.5 feet long, had been bitumen coated before being joined with screw couplings. The material was smooth quality sheet iron, slightly tapering, with longitudinal seams held by rivets 0.2 inch in diameter. Quantity of water was determined by a calibrated tank and loss of head by mercury manometers. This line and line No. 10 are classed as full-riveted on the assumption that retardation effects of exposed screw threads at the section joints would approximate those of a band of flathead rivets. On this assumption the capacity was more than that called for by the writer's formula.

Nos. 4 and 5.—Riveted-sheet steel pipe—Huacal Dam, Sonora, Mexico (*78, p. 601*)—Capacity −14.9 per cent for 10-inch pipe, and —5.4 per cent for 8-inch pipe

A new pipe line between reservoir and mill tanks at Nacozari, comprising 1.13 miles of 8-inch and 1.78 miles of 10-inch riveted pipe of 14-gauge steel sheets, asphalt coated in 20-foot sections for the lower 3,600 feet of the 10-inch pipe and 10-foot sections for the rest. Buried lengths were connected by driving slip joints after the coating at the joints had been heated by burning kerosene. Exposed lengths were flange bolted, the flanges being riveted to pipe sections. Capacity tests were made by H. Hawgood when the line was new. Velocity was measured by color (potassium permanganate) through both 8-inch and 10-inch sizes and finally checked by volumetric measurement in mining tanks at the end of the line. The difference between highest and lowest determinations by these methods is 1.33 per cent. A study of the retardation elements for these two pipes shows a very marked friction loss as compared with other thin-sheet new pipes, this being especially true of the 10-inch line.

No. 9.—11-inch lap-riveted sheet-iron pipe with taper joints—North Bloomfield, Calif. (*158; 159, p. 241*)—Capacity +3.1 per cent

In 1876, Hamilton Smith made tests of the capacity of three pipes in the same trench. These lines, about 11, 13, and 15 inches in diameter, respectively, and about 700 feet long, were tested when 5 years old. All were of sheet iron from 0.044 to 0.078 inch in thickness, single riveted in the shop in lengths about 20 feet long, then immersed in a boiling bath of asphaltum and coal tar. Field joints were made by insertion of the smaller end into the larger end of 20-foot lengths, stovepipe fashion. When tested the rivet heads were worn smoother than when new. Interior surfaces were quite smooth and practically free from rust. The three pipes were laid side by side across a sharp ravine with the outlet tank about 25 feet below the box at the inlet.

The quantity, Q, was determined by a calibrated weir, some 600 feet by ditch, from the outlet. The maximum discharge was tested while the pipes were in ordinary use. Lesser velocities were obtained by continuing the outlet end of the pipes by similar pipe units, on the same general rising inclination as that leading to the outlet tank. This of course decreased the total loss of head and hence decreased the flow. Elevations were determined by two level lines carefully run from head water to tail water and loss of head determined after making allowance for velocity and entry heads. Pipe diameters were found by measuring some 14 circumferences at equidistant points with a steel tape and making allowance for shell thickness but not thickness of coat, given as possibly 0.001 foot. Water surface elevations in inlet and outlet pools were read to 0.01 foot partly and 0.005 foot partly.

No. 10.—11.22-inch wrought-iron riveted-sheet pipe—Conduit No. 10, Paris, France (*35; 159; 67; 122, p. 135*)—Capacity +8.3 per cent

This line appears to have been of identical construction with No. 2 above, except that the pipe units tapered to a diameter 0.04 foot less at one end than at the other. The tests were conducted by H. Darcy in a similar manner to that already described. (p. 101.)

No. 12.—13-inch lap-riveted sheet-iron pipe, taper joints—North Bloomfield, Calif. (*158; 159, p. 241; 67*)—Capacity +3.2 per cent

One of three pipes described under No. 9 above. The capacity is in accord with that of No. 9.

No. 13.—14-inch lap-riveted steel pipe, cylinder joints—New Westminster, British Columbia (*79, pp. 77, 90; 80; 81*)—Capacity −22.4 per cent for No. 13 and −13.8 per cent for No. 13a

Tests are reported by A. McL. Hawks, covering two observations on the water supply trunk line for New Westminster, British Columbia. This steel pipe was 70,700 feet long; cylinder joints; 14 inches inside diameter of smaller course; hot asphalt dipped; riveting double in straight seams, single in girth seams, 42 inches apart, forming complete ring around pipe one-fourth inch less than inside pipe shell; vertical and horizontal alignment practically straight. Tests were made in April, 1896, and February, 1899, when pipe was 3 and 6 years old, respectively. Quantity of water measured by weirs, checked by rise in reservoir.

Test No. 13 was made at velocities less than 1 foot per second. This fact probably caused difficulty and error in measurements as the resulting capacity was much less than it should be.

No. 15.—16-inch lap-riveted sheet-iron pipe, taper joints—North Bloomfield, Calif. (*158; 159, p. 241*)—Capacity +7.2 per cent

One of three pipes described under No. 9 above. The capacity is slightly more than that of the other pipes in the trench.

No. 16.—16-inch riveted-steel pipe, cylinder joints—Astoria, Oreg., city supply (*57, p. 1; 86, p. 29*)—Capacity +0.3 per cent

Tests, reported by A. L. Adams, were made shortly after the completion of the supply main for Astoria, Oreg., consisting of 19,131 feet of wood stave and 16,417 feet of steel pipe. Inside diameter of the smaller courses was full 16 inches. Sheets were 4 feet long of 10-gauge and 12-gauge metal. Hot asphalt bath was applied after all riveting. Field girth joints were leaded under sleeves. Straight seams were double riveted and shop girth seams single riveted. Capacity tests were made of both stave and steel lines. Slope was determined by elevations of water in open standpipes 35,547 feet apart, the loss simultaneously determined in the previous 4,188 feet of stave pipe being deducted. Quantity of water was determined by uniform rise in the reservoir over a period of 18 hours. The long reach and high velocity should have given a very good test.

No. 18.—17-inch lap-riveted wrought-iron pipe, Texas Creek, Calif. (*158, 159*)—Capacity +9.2 per cent

This pipe, experimented upon by Hamilton Smith, was constructed of 14-gauge to 9-gauge wrought-iron sheets, double riveted in the long seams into sections about 20 feet long. These sections were put together in stovepipe fashion for 1,350 feet; on the remaining 3,090 feet an inner sleeve and outer band, with lead between was used. The line was laid in 1878 and tested in 1878 and 1879, while carrying clear mountain water. It was unusually heavily coated with asphaltum and coal tar. The inner surface was described as appreciably rougher than those of Nos. 9, 12, and 15. The length was measured twice. The interior diameter was accurately determined from many measurements as 1.416 feet. Loss of head was taken as the difference in levels between two water tanks, 303.62 feet, corrected for entry and velocity losses. The great loss of head caused a velocity of over 20 feet per second. For the 1879 test, described as the more acceptable, quantity was determined by flow over a 5.5-foot weir at the intake end of the pipe. The high velocity would produce a smooth interior, conducive to a high capacity.

No. 21.—18-inch riveted-steel pipe, taper joints—Force main to Solano Reservoir, Los Angeles, Calif.—Capacity −4.2 per cent

In correspondence with the writer, J. B. Lippincott reports an excellent series of observations on the capacity of the line from Buena Vista pumping station to Solano Reservoir. The essential details follow. Pipe was laid in 1901 and tested in 1909. It was made of 10-gauge sheet steel, riveted in taper joints, and probably coated with asphalt dip. The line has one bend of 90° on a radius of 3 feet, 9 inches, and other bends of the common elbow type of 52°, 43°, 47°, and 10°, respectively. The quantity, Q, was determined by displacement of pump, 1 per cent being deducted for slippage, although this was known to be practically negligible. Loss of head was measured from a point 40 feet beyond the pump to the reservoir 3,150 feet distant. Various observations showed that the pump discharge held steadily throughout the measurements. Mr. Lippincott says: "The facilities for observation were exceptionally favorable as there are no outlets from the main and the level of the water in the reservoir, into which the pipe was discharging, did not fluctuate more than an inch during the whole time of the test."

No. 23-24.—24-inch lap-riveted wrought-iron pipe, cylinder joints—Hemlock Lake conduit, Rochester, N. Y. (*140, p. 20; 86, pp. 13, 30; 117; 144; 122, p. 203*)—Capacity −17 to −20 per cent in 1890 and −15 per cent in 1891

Fourteen years after the construction of the first large riveted-metal pipe in the eastern States, G. W. Rafter made capacity tests to ascertain the changes which had taken place since its construction in 1876. (See No. 150, p. 121.) The essentials of his test were as follows. Quantity was determined by the rise in the reservoir and loss of head by water piezometers on a reach 1,901 feet long and by Bourdon-type guages on a reach 10,541 feet long. Results were based on a discharge of 6,742,000 gallons per day. These tests were important as being the first showing either great decrease in capacity, or that the 1,876 tests were very inaccurate (*86, p. 13*). One year after the Rafter test a similar test on a 3,327-foot reach of the same pipe (No. 24) was reported by Kuichling (*117*). The retardation factors found showed a higher capacity than those of the year before.

No. 25.—25.8-inch lap-riveted wrought-iron pipe—Humbug siphon, North Bloomfield, Calif. (*158; 159, p. 241; 67, p. 140*)—Capacity +16 per cent

Contemporary with the riveted wrought-iron pipes laid in San Francisco (Nos. 28–29), were similar pipes which conveyed much of the water used in hydraulic mining. Hamilton Smith made tests on several of them between 1873 and 1877. The Humbug pipe line consists of a double-barreled inverted siphon across Humbug Canyon, leading as a main supply line to North Bloomfield mine, Calif. The pipes were each nominally 26 inches in diameter, of sheet iron only one-sixteenth inch thick, seamed with a single line of light rivets which formed little obstruction to the flow of water. There were no sharp bends. The pipe was laid in 1868 after being coated with hot asphaltum and coal tar. The tests were made in 1873. Quantity was determined by flow through apertures, with computed velocity checked by timed velocity of wood blocks and stones. The test was not given full weight by Smith, who questioned the quantity determination. The high velocity insured a polished interior, as indicated by the excess capacity.

No. 26.—30-inch lap-riveted wrought-iron pipe—Cherokee siphon, California. (*159, p. 237; 67, p. 156; 9, p. 56; 127, p. 191*)—Capacity +10.3 per cent

An example of the bold construction in the days of hydraulic mining in California is found in this long siphon pipe made of sheet iron but operating under a maximum pressure head of 887 feet. Hamilton Smith reports tests he made when the line was 5 years old. The elements of these tests follow. The line was 2½ miles long, conveying water for mining purposes across a canyon. Pipe was double-riveted 16-gauge sheet iron, coated when new with boiling asphaltum and coal tar. It was tested for capacity in 1876, the quantity of flow being determined by flow through "standard orifices." Length and loss of head were ascertained by engineers of the company owning the pipe. At the time of test the interior surface was very smooth except for rivet heads which formed "noteworthy obstructions" for over half the length.

Because of high velocities and clean scoured condition the excess capacity may be taken as verification of the formula for average conditions.

No. 27.—30-inch riveted laminated wrought-iron Baden-Merced line, Spring Valley Water Co., San Francisco, Calif. (*136*)—Capacity, −1.4 per cent

In 1911, J. N. LeConte of the University of California made gaugings on many of the trunk lines of the San Francisco supply. These measurements are of especial value as they covered very long reaches of old pipe. Some of these pipes are among the oldest sheet-metal lines of large size in this country. They were all dipped in refined natural asphalt. The detail measurements of some of the gaugings are not now available as the original notes can not be located. The others are quite complete and the presumption is that the same methods of testing were used throughout. As it is possible that complete figures will be available at some future time it is thought best to include meager data at this time and thus reserve space for additional elements. In these tests the quantity was determined by pitometer traverses across two diameters. The loss of head was found by tested gauges of the Bourdon type. This reach of the Baden-Merced line, was 5 miles long and approximately 5 years old at time of test.

No. 28.—30-inch San Andreas line, Baden to College Hill, Spring Valley Water Co., San Francisco, Calif.—Capacity +29 per cent

See No. 27 for general notes. This line, laid in 1870, of 9-gauge and 11-gauge sheets, shows values of $K_s = 0.44$ and C_w equal to 110 after 42 years of service. This is a remarkable showing for either the efficiency of the original coating on this metal or the nonaggressive qualities of the water carried.

No. 29.—30-inch riveted laminated wrought-iron pipe.—Pilarcitos line, Daly Hill to Lake Honda, Spring Valley Water Co., California (*136*)

See No. 27 for general notes of the LeConte tests. This line was 1 mile long. At 47 years of age the value of C_w was 91.5, which indicates much more deterioration with age than does No. 28 above at 42 years.

No. 30.—33-inch riveted-steel pipe, cylinder joints—Bull Run conduit No. 1, Portland, Oreg. (*160; 116, p. 503*)—Capacity +5.8 per cent

I. W. Smith reports tests on cast-iron and riveted-steel pipes from 24 to 42 inches in diameter. The riveted steel portion of the line extended from the source in Bull Run River to a reservoir near Willamette River. Tests were made in 1896 on three sizes of pipe as follows:

Size	Length	Bends	Total angle	Maximum radius	Minimum radius
Inches	*Feet*		°	*Feet*	*Feet*
33	34, 176	85	404	38	14
35	39, 829	141	782	38	38
42	50, 965	225	1, 033	38	14

Quantity of water was measured over a weir at the intake end of line, and also by the rise of the reservoir surface. The pipe was all made of 60-inch sheets with cylinder joints, double riveted on straight seams and single riveted on girth seams. Size was computed on the basis of interior diameter of smaller rings. Shop units were coated with hot asphaltum, which rounded off the exposed edges of the rings. Both the 33 and 35 inch lines were built of No. 6 steel (0.203 inch thick).

No. 32.—35-inch riveted-steel pipe, cylinder joints—Bull Run supply main, Portland, Oreg. (*160; 116, p. 503*)—Capacity +9.1 per cent

For details of tests by I. W. Smith see No. 30 above.

No. 35.—36-inch lap-riveted steel pipe, cylinder joint—Belleview to South Orange Avenue, Newark, N. J. (*86, p. 32*)—Capacity −1.3 per cent

General data under No. 58 apply also to this pipe. It is a 5-mile inverted siphon across a valley, mostly in city streets. All is of ¼-inch steel plate. There are six curves with radii of 83 feet or less; all well rounded and smoothly finished. No. 35a, a single run taken when the pipe was 4 years old, indicated the usual decrease in carrying capacity. The results agree very closely with the writer's formula when type, class, and age of pipe are all considered.

No. 37.—36-inch riveted laminated wrought-iron pipe—Alameda pipe line from Niles to Belmont—Spring Valley Water Co., San Francisco, Calif. (*136*)—Capacity +8.5 per cent

See No. 27 for general notes of the LeConte tests. This experiment was carried out in 1911 on a reach of 114,400 feet from the beginning of the line to Belmont, Calif. The pipe was made of 7 and 9 gauge wrought-iron plates laid in 1887–88 and was still in good condition in 1925, except 2 miles through swamp land. The test included 6,467.5 feet of San Francisco Bay-crossing pipes, two pipes each 16 inches in diameter, and two each 22 inches, of ⅜-inch steel tubing with ball-and-socket joints.

No. 38.—36-inch riveted-iron pipe—Alameda pipe line—Spring Valley Water Co., San Francisco, Calif. (*136*)—Capacity +9.3 per cent

See No. 27 for general notes of the LeConte tests. This pipe, a portion of the 20-mile Alameda line from Niles to Belmont, Calif., was laid in 1887–88. It was made of 9-gauge wrought-iron plates.

No. 39.—36-inch riveted wrought-iron pipe—Conduit No. 1, Rochester, N. Y. (*140, p. 14; 116; 144; 122, p. 203*)

Rochester, N. Y., was one of the first eastern cities to install a large riveted-steel pipe for municipal supply. Conduit No. 1, from Hemlock Lake to Rush Reservoir, was built in 1873–1875. Conduit No. 2, built in 1893–94, also conveys water between these reservoirs and extends around and beyond Rush Reservoir, a distance of nearly 9 miles, to Cobb's Hill Reservoir. Its intake end is in White Bridge overflow chamber, some 12,000 feet from Hemlock Lake via a brick conduit. Conduit No. 3, built in 1914, extends from the overflow mentioned to Cobb's Hill Reservoir, with a pipe connecting to Rush Reservoir as it passes. In discussion of these conduits "southern division" should be understood to cover the distance from Hemlock Lake to Rush Reservoir, and "northern division" the distance between Rush Reservoir and Cobb's Hill Reservoir or Mount Hope Reservoir.

The Rafter and Kuichling tests, listed as numbers 39 and 39a, were on part of the compound pipe consisting of 50,776 feet of 36-inch wrought-iron riveted pipe at the intake from Hemlock Lake to air valve No. 53; then 15,500 feet of 24-inch wrought-iron riveted pipe; then 35,772 feet of 24-inch cast-iron pipe to Rush Reservoir. This line, now called conduit No. 1, was first tested for capacity when new. Quantity was determined by the rise in Rush Reservoir, and levels by difference in elevation of water surfaces in the two reservoirs. (See No. 150 for further comment.) Deductions possible apply only to the compound line, as local losses were not determined. After 14.6 years tests were again made and a great reduction in capacity was noted. From 1897 to 1920, many tests were made at varying intervals, on the reach extending from Hemlock Lake to air valve 53, thus covering only the 36-inch pipe. This pipe was made of 3/16-inch plates and coated with tar. The reach had at least 60 abrupt angles or elbows ranging from 5° to 40° and 37 air valves. The indicated capacity for the reaches of conduit No. 1 was appreciably below that called for by the writer's formula.

Nos. 40 and 41.—38-inch lap-riveted steel pipe—Conduit No. 2, Rochester, N. Y. (*101, 116*)—Capacity +6.7 per cent for 1896 runs and +15.6 per cent for 1897 runs

For general notes on the Rochester mains see No. 39. In 1896 and 1897, E. Kuichling made capacity tests at various velocities for a reach in the "middle section" of the new conduit nearly 9 miles long. This pipe was constructed in 1893–94 of ¼ to ⅜ inch plates. In 1896, tests were conducted by timing the rise in the reservoir over periods of from 8 to 26 hours. Allowance was made for evaporation and leakage. The loss of head was measured by mercury columns corrected for temperature. The resulting values of Chezy's C are not consistent, and Kuichling calls these observations "only approximate." For this reason the observations have been given a C rating. However, similar tests run in 1897 are described as "made with the utmost care" and it is noted that the retardation factors are reasonably consistent.

No. 42.—38-inch lap-riveted steel pipe, cylinder joints, southern division, conduit No. 2, Rochester, N. Y. (*116; 144, pp. 62, 66; 122*)—Capacity +9.3 per cent

See pipe No. 39 for general description of layout of Rochester conduits. This portion of conduit No. 2, from White Bridge overflow to Rush Reservoir, 17.3 miles long, was made of plates one-fourth to three-eighths inch thick, laid in 1893–94. This reach was gauged by the water department of the city of Rochester once or oftener each year from 1897 to 1920. The quantity of water was determined by the rise in the reservoir over periods ranging from 3 to 12 hours. These tests, 28 in number, showed a gradual decrease in capacity. In order to save space and make this tendency to decrease more apparent, these tests are grouped (Table 1) into 5-year periods, values of all functions and retardation factors being averaged after the latter were computed for each individual test. However, in Figure 7, showing the deterioration in capacity of various pipes, the individual values are plotted.

No. 43.—38-inch lap-riveted steel pipe, cylinder joints, northern division, conduit No. 2, Rochester, N. Y. (*116; 144, pp. 62, 65; 122*)—Capacity +4.2 per cent

See pipe No. 39 for general layout of Rochester conduits. This portion of conduit No. 2, from Rush Reservoir to Mount Hope Reservoir, 8.6 miles long, of plates one-fourth to three-eighths inch thick, was laid in 1893 and 1894. The city of Rochester made periodic gaugings, 27 in number, and the resulting data are grouped as described under No. 42 above. The duration of the test periods ranged from 4 to 10.8 hours.

No. 44.—38-inch lap-riveted steel pipe, cylinder joints, northern division, conduit No. 2, Rochester, N. Y. (*116; 144, pp. 62, 69*)—Capacity +5.8 per cent

See pipe No. 39 for general layout of Rochester conduits. This portion of conduit No. 2, from Rush Reservoir to Cobb's Hill Reservoir, 7.7 miles long, was made of plate one-fourth to three-eighths inch thick. During the period from 1912 to 1920, the city of Rochester made eight gaugings each covering a period of four to seven hours. The resulting data are placed in two groups as described under No. 42 above.

No. 45.—42-inch riveted-steel pipe, cylinder joints, Bull Run conduit No. 1, Portland, Oreg. (*160; 116, p. 503*)—Capacity +8 per cent in 1896 and +22.4 per cent in 1915

For details of tests by I. W. Smith see No. 30 above. The 42-inch part of the line was constructed of metal 0.188, 0.234, and 0.375 inch thick. In August, 1915, another test of this portion of conduit No. 1 was made by F. M. Randlett. (No. 45*a*.) The line was then 21 years old, but the retardation coefficients showed little increase in the roughness of the pipe. The indications are that the water conveyed in the Portland mains is distinctly nonaggressive. The writer is indebted to B. S. Morrow, engineer for the bureau of water works of Portland, for data covering the test by Mr. Randlett.

No. 46.—42-inch lap-riveted steel pipe, taper joint—Kearney extension, Newark, N. J. (*86, p. 33*)—Capacity—2.3 per cent

General data under No. 58 apply to this pipe also. This line was 55 per cent of 5/16-inch plate with the balance about equally divided between ¼ and ⅜ inch plate. Herschel comments to the effect that the interior coating was "unusually smooth."

Nos. 47 and 48.—42-inch lap-riveted steel pipe, taper joint—conduit No. 2, below Pompton Notch, Newark, N. J. (*86*)—Capacity within 2 per cent of formula

Some five years after the construction of conduit No. 1 (No. 58) a second pipe was laid alongside it. Above Pompton Notch the new line was of the same diameter, 48 inches, but below the notch the size was decreased to 42 inches. The type of construction was changed from cylinder courses to taper courses. The 42-inch line is some 55 per cent of ¼-inch plate and the balance nearly all of 5/16-inch plate. Presumably the other data regarding all capacity tests and coating can be found under description of No. 58 on page 108.

No. 49.—44-inch taper-jointed riveted-steel pipe—Crystal Springs conduit, Los Angeles, Calif.—Capacity +5.5 per cent

This pipe, extending from Crystal Spring settling basin to Buena Vista tunnel, a total length of approximately 16,900 feet, was laid about 1889 to supply domestic water to the city of Los Angeles. In 1918, E. A. Bayley conducted tests for retardation losses over four sections of this line, as follows:

Pipe No.	Location	Feet	Remarks
49a	Settling basin to manhole No. 7	491.1	Perfectly straight.
49b	Manhole No. 7 to manhole No. 8	1,654.8	A few bends.
49c	Manhole No. 8 to manhole No. 9	1,739.6	
49d	Manhole No. 9 to manhole No. 10	1,013.3	
49	Total, settling basin to manhole No. 10	4,898.8	

Throughout the reach the line was under slight pressure, the water rising in each manhole to the hydraulic gradient. Lines of checked levels were run between ends of the reach and bench marks established directly over each point of observation. Every few minutes throughout the four hours of test, the elevation of the water surface in the manholes was determined from these datum points. The surface and hence the flow remained quite constant throughout the run, the greatest variation being only 0.04 foot, and this in but one reading. The quantity of water was determined by 24 sets of five observations each with a recently-rated cup-type current meter.

The retardation coefficients, as determined from these observations, are not consistent. The first reach, No. 49a, was too short for a good test. The total loss of head between water surfaces included entry and velocity heads, which involved more than 30 per cent of the total. However, these items as measured checked with the theoretical entry and velocity heads. The coefficients for this reach indicate the smoothest section of the pipe. Usually the initial reaches of pipe, just below a reservoir, show the greatest losses

More weight should be given the longer reaches of pipe, where the gross loss of head overshadows experimental errors, especially as measurements of loss were only taken to the nearest hundredth foot.

No. 50.—44-inch riveted-steel pipe—San Andreas pipe line—Spring Valley Water Co., San Francisco, Calif. (*136*)—Capacity +15.2 per cent

See No. 27 above for general notes on the LeConte tests. This line made of ¼-inch iron plates, extending from San Andreas to Baden, a distance of 29,000 feet, was laid in 1898. Test was made on a portion 19,100 feet long.

No. 51.—44-inch riveted laminated wrought-iron pipe—Crystal Springs pipe line, from Millbrae to Sierra Point—Spring Valley Water Co., San Francisco, Calif. (*136*)—Capacity +4.8 per cent

For general notes on the LeConte tests see No. 27 above. The quantity of flow was measured over a 13¼-inch weir at University Mound and loss of head determined with tested gauges of the Bourdon type. This line was 18 miles long, made of No. 6 and No. 7 wrought-iron plates, and was laid in 1885. It still gives good service (1925).

No. 52.—44-inch riveted laminated wrought-iron pipe—Crystal Springs line, from Sierra Point to University Mound—Spring Valley Water Co., San Francisco, Calif. (*136*)—Capacity −6.6 per cent

For general notes of the LeConte tests see No. 27 above. This is another part of the 18-mile line listed above (No. 51). Although the two parts of the line are of the same age, No. 52 shows the lesser capacity.

Nos. 53 to 57.—47-inch riveted-steel pipe, East Jersey Water Co.—Various portions of conduit No. 1, below Pompton Notch, Newark Supply Main (*86, p. 27; 122, p. 211*)

Description is similar to that of No. 58 below, except that this reach of 16 miles is less broken and the pipe is one long inverted siphon, all below the gradient, but crossing two high hills. Reach No. 53 is mostly of ⅜-inch plate. About half of No. 54 is three-eighths inch thick and a third one-fourth inch thick. No. 55 is mostly ¼-inch plate. Half of No. 56 is ¼-inch plate and the balance of 5/16 and ⅜-inch plate. Two-thirds of No. 57 is ¼-inch plate, and the balance divided between 5/16 and ⅜ inch plate.

Eleven years after the tests just described were made Thaddeus Merriman experimented upon a reach of the same line about 9 miles long (57a). No comment was made regarding the condition of the pipe interior or the methods of test, but they are probably similar to those mentioned under the other Merriman tests of this water system. (See No. 58.)

A study of the last column in Table 2, for this series of numbers, shows that the capacity depreciated about in accordance with formula No. 12, page 10, and the agreement with the writer's capacity formula is quite close with a few exceptions.

No. 58.—48-inch cylinder-joint, lap-riveted steel pipe—Conduit No. 1, above Pompton Notch, East Jersey Water Co., Newark, N. J. (*85; 86, p. 27; 122, p. 208*)—Capacity at 15½ years, −13.9 per cent

Soon after the completion of the second large riveted trunk line in the eastern part of this country (No. 39 was said to be the first), tests were conducted by J. Waldo Smith, then assistant to Clemens Herschel, who reported these and other tests to the total number of 115 (*86*). The line was constructed of lap-riveted plates from one-fourth to three-eighths inch thick, rolled into cylinders, alternately larger and smaller, which were dipped in asphalt. Plate edges were beveled, and rivet heads, driven in both shop and field, were well formed. All bends, both vertical and horizontal, were made with one of four predetermined curves. The maximum radius was that of a 10° curve—573 feet. The reach above Pompton Notch, more than 5 miles long, extended from station 0 to station 309. The major portion of this reach was experimented upon in 1892 and 1896. Smith measured the flow with Venturi meters and the loss of head with tested gauges of the Bourdon type. These gauges are acceptable for very long reaches of pipe but are not graduated fine enough for short reaches.

Eleven years after tests reported by Herschel, an additional experiment was made by Thaddeus Merriman. Nearly the same reach was tested (No. 58c). Quantity was measured by the old Venturi meter, after the throat had been cleared of deposits of organic growth from one-eighth to three-fourths inch thick.

No. 59.—48-inch taper-joint riveted-steel pipe—Conduit No. 2 (above Pompton Notch), East Jersey Water Co., New Jersey (*86, p. 28; 122, p. 210*)—Capacity when new, −6.1 per cent; when 15½ years old, −12.6 per cent

For general description of tests reported by Herschel, see No. 58 above. Reach No. 59 was put in use in 1896, parallel to conduit No. 1 above. Note that a change was made from cylinder to taper joints. Since this test and No. 58 were both made on new pipes of about the same size, laid parallel, it would appear that they should give comparative data on the cylinder and taper joints. If the average values of retardation factors be taken at face value it appears that the pipe with taper joints carries only 92 per cent as much water as the pipe with cylinder joints. However, a comparison of the tests made in 1907 by Merriman indicates that the cylinder-joint pipe (No. 58) is slightly below the taper-joint pipe (No. 59) in relative capacity.

Eleven years after tests by Herschel, listed as No. 59 above, a series of four tests was made by Thaddeus Merriman (June and October of 1907, No. 59a). The June tests covered a reach of 28,535 feet and the October tests a reach of 24,641 feet. The pipe was very rough, the tubercles ranging from 25 to 40 per square foot. The deepest pitting was about one-sixteenth inch. Before the tests the meter was cleaned as described for No. 58c.

No. 60.—52-inch lap-riveted pipe, taper-joint siphon, main supply conduit, Los Angeles city water works, California—Capacity +8.7 per cent

In correspondence with the writer, J. B. Lippincott reports details of a test on a siphon in the main supply pipe that had been in continuous operation for 5 years (*112, p. 81*). It was built of three-sixteenths-inch plate with taper joints, each 4 feet long. Straight seams were double riveted and girth seams single riveted. The pitch of the flat-headed rivets was 1¼ inches. Diameter was one-eighth inch scant at lap or girth seams. The developed length of the siphon was 3,666 feet, with a maximum sag of 65 feet below the hydraulic gradient. Loss of head between water surface at intake and outlet, as determined by checked lines of levels, was 1.84 feet. No corrections were necessary for velocity changes or entry head.

No. 61.—54-inch riveted laminated wrought-iron pipe—Alameda pipe line, Spring Valley Water Co., San Francisco, Calif. (*136*)—Capacity +5.1 per cent

See No. 27 above for general notes of the LeConte tests. This line, of nine-thirty-seconds-inch plate, was laid in 1903.

No. 62.—59-inch riveted-steel pipe—Munroe penstock No. 1, Munroe Mill, Mass. (*122, p. 202*)—Capacity −7.3 per cent

These tests were made on a line described by Mills as the "roughest tar coated plate iron pipe which the writer has seen." The original diameter was 4.96 feet but this had been contracted by tubercles, after three years, to 4.925 feet. The penstock originally was coated "with some coal-tar preparation." Mills supplies no description of plate thickness, method of jointing, or rivets. Flush piezometers set 200 feet apart, brought water columns to gauge glasses beside a single scale, by means of small copper tubes. Quantity was measured over a weir below the outlet. The results of this series of tests were remarkably consistent for such a short reach on a large pipe.

No. 64.—72-inch taper-jointed riveted-steel pipe of Jersey City Water Co., New Jersey (*122, p. 217*)—Capacity when new, +6.4 per cent—at 2 years, −2.7 per cent; at 5 years, −15.6 per cent; and at 13 years, −8.6 per cent

This series of tests was made on a reach 9.3 miles long. The pipe was made of plates from five-sixteenths to eleven-sixteenths inch thick, with one longitudinal seam double riveted; girth joints, from 5 to 7½ feet apart, were single riveted. The pipe rings were dipped horizontally in Gilsonite. In the line were ten 48-inch gate valves, requiring reducing and enlarging tapers between sections of standard diameter, also 22 horizontal and 94 vertical changes in direction. At summits were 28 air valves and at sags 15 blow-offs. There were also 53 manholes. Tests were made by J. Waldo Smith when the pipe was new and at various times thereafter until the line was 13 years old. The quantity of flow was measured through a Venturi meter at the downstream end of the line. The method of determining loss of head was not described.

No. 65.—72-inch butt-jointed, triple-riveted steel pipe—Penstock, Halsey power house, Pacific Gas & Electric Co., Placer County, Calif. (*127*)—Capacity +5.3 per cent

In 1916, tests were made by R. A. Monroe on the Halsey penstock much as were those described for the Drum and Wise plants. For description of piezometer rings and gauges see No. 68 below. On this penstock, however, a Venturi meter was available for the measurement of quantity. After passing the meter and reach No. 66, water flows through a butt-strap pipe of which a reach 319.9

feet long was chosen for test. As the whole penstock was new and the same methods of experimentation were used on both No. 65 and No. 66 a direct comparison of butt-strap and lap-riveted pipe is possible. The reaches were short and the retardation factors erratic for the lower velocities. However, for those above 6 feet per second the average value of C_w was 114.1 for the cylinder-joint, lap-riveted reach, and 105.1 for the butt-jointed reach with an excessive number of rivet heads and the interior butt straps. The plate thickness was three-eighths inch, but this had no direct influence on the capacity, as in the lap-riveted pipe. (See No. 66 for other notes.)

No. 66.—72-inch lap-jointed, double-riveted steel pipe—Penstock, Halsey Power house, Pacific Gas & Electric Co., Placer County, Calif. (*127*)—Capacity +8.9 per cent

In 1916 tests by R. A. Monroe were made on Halsey penstock, similar to those on the Drum and Wise penstocks. For description of piezometer ring and gauges, see No. 68. On this penstock, however, a Venturi meter was available for the measurement of quantity. A series of 11 flows at velocities from 2.3 to 11.7 feet per second gave a most acceptable set of tests, especially as it is stated that proper correction was made for difference in temperature between the water in the penstock and that in the static line. This reach was 583.4 feet long and contained three bends—one horizontal and two vertical. Plate thickness varied from one-fourth to three-eighth inch. This line was new at time of test. The tested reach was on the final steep slope leading to the power house. Beyond the lower ring the water entered the butt-strap pipe of the same size (No. 65). Two coats of a special graphite were brushed on both inside and outside of this penstock. It was designed by use of Williams-Hazen formula, with $C_w=100$. The observed values are erratic for velocities below 6 feet per second. For the higher velocities the average value of C_w for this new pipe was about 114.

No. 67.—72-inch butt-jointed, triple-riveted steel pipe—Penstock, Drum power house, Pacific Gas & Electric Co., Placer County, Calif. (*127*)—Capacity +6.6 per cent

Simultaneous with the tests described as No. 68 below, identical tests were conducted by Mr. Monroe on the new butt-strap pipe. A reach 423.9 feet long was chosen along a relatively straight portion of the line. The same type of equipment was used and losses were determined for the same flows and one additional. These simultaneous tests indicated that the loss of head caused by the excessive number of rivet heads through the butt straps more than offset that at the enlargements and contractions of lap-riveted joints where fewer rivets were used. This reach is part of the same penstock and is painted as was No. 68. The indicated capacity agreed fairly well with that of No. 65, which is a comparable pipe. However, the range of velocities for No. 67 did not include any beyond about 3 feet per second, while the results of tests on No. 65 did not become stable until a velocity of 6 feet was reached.

No. 68.—72-inch lap-jointed, double-riveted steel pipe—Penstock, Drum power house, Pacific Gas & Electric Co., Placer County, Calif. (*127*)—Capacity +20.6 per cent

In 1916 a single penstock served the Drum power house. It consisted of a 72-inch lap-jointed pipe for 3,341.6 feet; then 795 feet of butt-strap pipe of the same nominal size. This penstock was given one shop coat of green graphite and a field coat of black graphite, both coats being brushed on. Immediately after construction, R. A. Monroe made tests on the first sag of the penstock, 1,489.6 feet in length of reach. Piezometers were connected to the pipe at four points on the circumference. These piezometers were attached to a 2-inch air pipe paralleling the penstock. At the lower ring a differential gauge, containing carbon tetrachloride, served to indicate the loss of head between rings. The quantity of flow was determined by measuring the fall of water surface in the forebay, with corrections for predetermined seepage loss and rainfall. This penstock was designed by the use of Williams-Hazen formula, using a value of $C_w=100$. If the two observations at velocities less than about 3 feet per second can be considered indicative the value of C_w when the pipe was new was about 130. The results of these tests should be used with caution.

No. 69 and 69a.—72¼-inch double butt-strap, triple-riveted pipe—Penstock, Pioneer Electric Power Co., Ogden, Utah (*70, 116, 117*)—Capacity when new +2.6 per cent; when 2 years old, −2.5 per cent

One of the first large riveted-plate pipes built in the West serves a power plant at the mouth of Ogden Canyon, Utah. The main conduit is 31,600 feet long of which 27,000 feet is wood-stave pipe, 72½ inches in diameter, between reservoir and surge chamber; 4,600 feet is a riveted steel penstock averaging 72¼ inches in diameter. Soon after its completion in 1897 Professors Marx, Wing, and Hoskins of Stanford University tested the retardation losses in both stave and steel portions of the line. Results for the stave portion were entirely unexpected and without precedent. Tests were repeated in 1899 but with approximately the same results (*152*). Practically the entire length of the portion of the steel line tested consisted of riveted pipe, straight in plan view but with profile broken by 13 vertical elbows of 30 feet radius and one of 40 feet. Joints abutted, with longitudinal straps both inside and out. Plates were from three-eighths to eleven-sixteenths inch thick, with one-sixteenth inch increments. Sections were 9 feet 2 inches long, shop riveted and then dipped in a mixture of grade C, California asphalt and natural liquid asphalt maltha. The process of dipping took nearly one hour per section, but gave good results. Slow withdrawal developed a smooth glossy coat with very few wrinkles. Obstructions to flow consisted of six straight and four girth rows of low cone rivet heads and a continuous band of interior strap 16 inches wide and three-eighths to one-half inch thick along the top of the pipe. Loss of head was determined by mercury gauges of pot-and-column type. Quantity of water was measured with a Venturi meter. Enough points were observed to justify the consideration of these experiments among the best for this type of pipe.

For velocities below 2 feet per second the results of the tests are very erratic, hence only the observations at velocities above 2 feet per second were considered in arriving at the equations for the pipe flow and in computing average values.

No. 70.—77-inch lap-jointed riveted-steel pipe—Munroe penstock No. 2, Essex Co., Lawrence, Mass. (*122, p. 198*)—Capacity +3.1 per cent

This line was constructed in 1881 of cylindrical plates three-eighths inch thick. The joints were lapped and riveted. The pipe was coated with asphalt which appeared to be effectual in retarding tuberculation but which acquired a coating of slime and a few patches of sponge. Tests by H. F. Mills from time to time from 1888 to 1895 indicate deterioration according to the formula

$$H=0.0412\ (1+0.17\sqrt{t})\ V^2, \text{ where } t \text{ is time, in years of age.}$$

Details of these tests were not given by Mr. Mills.

In 1896 the penstock was repainted with coal-tar pitch "that did not harden well but remained for a time sticky to the touch." This painting was done after the pipe had been brushed but not scraped. According to Mr. Mills' tests made after an interval of three days indicated.

$$H=0.0547\ V^2$$

and other tests made from time to time until 1903 showed the coefficient to increase up to 0.069 or a deterioration of nearly 13 per cent. Seven years after the pipe was repainted the series of tests listed as No. 70 were made.

For the capacity tests mentioned, the quantity of flow was measured over a weir below the pipe outlet. Flush piezometers were set on the pipe 150 feet apart and connected by copper pressure pipes to glass tubes placed near a single scale. The reach is very short, but Mills was in a position to make careful piezometer connections.

No. 71.—84-inch riveted-steel pipe—Flow line to Big Creek No. 1 power house, Southern California Edison Co., Cascada, Calif.—Capacity −1.6 per cent

Water from Huntington Lake flows through 0.73 mile of 12-foot tunnel and 9-foot conduit before reaching the 7-foot pipe tested by H. L. Doolittle. Gauges were installed over a reach 6,410 feet long, No. 1 being located on the lower end of the 9-foot pipe just above the taper section, and No. 2 on the 7-foot pipe a short distance above a valve into the steep penstock. The computations were corrected for difference in velocity heads in the two sizes of conduit. This line was 10 years old at the time of test. Longitudinal joints were double riveted and girth joints single riveted. It was factory-painted with two coats of hot mineral asphalt.

No. 72.—84-inch lap-jointed, double-riveted steel pipe—Penstock, Wise power house, Pacific Gas & Electric Co., Placer County, Calif. (*127, 21*)—Capacity +0.6 per cent

Within a year after completion of this line R. A. Monroe conducted a series of tests, comprising four flows at velocities from 3.4 to 8.3 feet per second. These tests were made on three sections of the 84-inch penstock, having different types of riveting. (See Nos. 73 and 74 below.) This reach and No. 73 included the last part of reach No. 76, tested by the writer two years later. Four piezometer rings were installed on the line. This reach is located between rings 1 and 2, ring 2 being the upper end of the next reach; and so on. The quantity of flow was measured with a Venturi meter located near the upper end of the penstock. Loss of head was determined by a differential gauge set up between the two rings of each reach. Carbon tetrachloride was used in a U-tube to indicate the difference in pressure or friction loss between the two rings. This reach was 744.7 feet long, made of plates seven-sixteenths inch thick, with cylinder joints, double riveted. The steel portion of the penstock was given two coats of graphite paint, inside and outside. The steel penstock was designed by the use of the Williams-Hazen formula, with $C_w=100$. The average value of C_w found when the pipe was new was 106, which did not leave much margin for deterioration. (See No. 76.)

No. 73.—84-inch lap-jointed, single-riveted steel pipe—Penstock Wise power house, Pacific Gas & Electric Co., near Auburn, Calif. (*127, 21*)—Capacity +7 per cent

This reach, between rings 2 and 3, was immediately below No. 72. It consisted of 768 feet of single-riveted pipe of plates three-eighths inch thick. (See No. 72 for general description of this penstock and methods of test.)

No. 74.—84-inch butt-jointed, triple-riveted steel pipe—Wise power house, Pacific Gas & Electric Co., Placer County, Calif. (*127, 21*)—Capacity +3.4 per cent for No. 75

This reach adjoined No. 73. It was 1,070.6 feet between piezometer rings 3 and 4. Plates were from three-eighths to seven-sixteenths inch thick. The pipe rings were all of the same size, ends and edges abutting. However, the excessive number of rivets through the butt straps caused greater retardation loss than would be found in a similar pipe with lap joints. (See No. 72 for general statements and methods of tests for this penstock.)

The series indicates a higher capacity for this butt-strap pipe than is usually found.

No. 75 covers tests on reaches 72, 73, and 74 when combined, for comparison with No. 76. When new this long reach, classed as 1b had an excess capacity. Note its deterioration in the following two years. (See No. 76.)

No. 77.—103-inch lap-riveted wrought-iron pipe, cylinder joints, Holyoke testing flume, Massachusetts. (*86, p. 29; 122, p. 200*)—Capacity +9.4 per cent

In connection with the turbine-testing laboratory at Holyoke, there is a riveted wrought-iron pipe 153 feet long. Herschel reports tests he made on this short pipe, in 1887. This line was constructed in 1881 with cylinder joints, three plates of ½-inch metal to the course. At the time of test, when the pipe was 5 years old, little if any of the original coat remained. Herschel says "it was rather rusty inside although not affected with the tubercular disease which is the bane of cast-iron pipe." This statement senses the misunderstanding that existed up to the time of the 1890 Rochester tests, that wrought-iron and steel pipes were not subject to tuberculation. Herschel gave his experiments upon this pipe a B rating. The writer believes that his results show remarkable consistency when the extremely short reach of pipe, compared to the large diameter, is considered.

No. 78.—108-inch cylinder-jointed riveted-steel pipe—Flow line, Oak Grove plant, Portland Electric Power Co., Oregon (*3, 138*)—Capacity +8.8 per cent

In connection with turbine efficiency tests of unit No. 1, D. W. Proebstel developed data enough to indicate satisfactorily the losses in the 9-foot pipe line between the reservoir intake and the surge tank above the steep power drop. Through the courtesy of Chief Engineer C. O. Dunn and Mr. Proebstel these data were reported to the writer for this bulletin. From the intake to the surge chamber the conduit consisted of a concrete duct 12 feet 4 inches diameter, 1,340 feet in length, then the steel pipe, 9 feet in diameter and 33,920 feet long, then a tunnel 14 feet in diameter extending 262 feet to the surge tank. The pipe line was built of steel plates from nine thirty-seconds to eleven-sixteenths inch in thickness. The shop units were 33 feet long; in five rings, cylinder jointed with single rows of rivets in both longitudinal and girth seams. The field joints were similar riveted girth seams. As shown in Plate 5, B stiffeners were used where the plate thickness was less than three-eighths inch.

Before leaving the shop the pipe was painted with red lead; in the field it was again painted with graphite.

Proebstel made 10 observations in May, 1925, after the plant had been in operation 10 months. His determinations of quantity of flow were made by a system of multiple Pitot and piezometer orifices (*138, p. 77.*) An X-tube structure was installed in the pipe. In each of the legs of the X were 10 Pitot orifices reflecting the pressure heads plus velocity heads for equal-area zones across the section of the pipe. Piezometer orifices at each side and at the top of the pipe gave the pressure head alone. (See three columns at left in pl. 5, B.) Each orifice was connected by pressure tubing to a gauge glass set over a graduated background. Simultaneous readings on all glasses were effected by use of a camera. The apparatus is shown on the pipe in plate 5, B.

The friction loss in the steel pipe was taken as the difference in water levels at intake and surge tank, less 50 per cent of the velocity head for the velocity in the steel pipe and also deducting the loss in the large concrete duct. For the maximum flow this total deduction comprised only 2.5 per cent of the total fall. Any error in the assumptions was negligible.

No. 79.—129-inch triple-riveted steel pipe, butt-joints—Penstock, Pit No. 1, power plant Pacific Gas & Electric Co., Shasta County, Calif. (*124*)—Capacity +22.4 per cent

In 1922 Roy Wilkins made a series of tests for the company on the various types of pipe constituting the penstocks of the Pit River plant. This particular reach consisted of a straight section of large pipe, made up of steel plates seven-sixteenth inch thick, butt-strap joined and triple riveted. While this reach was short the velocities were relatively high. See No. 228 for methods and equipment. The capacity, if truly indicated by two observations at high velocities, is far above that of any other butt-strap pipe listed.

GIRTH-RIVETED PIPE OR SCREW-JOINT PIPE

Nos. 202, 204, 206, 208, 210.—Screw-joint pipe—Experimental lines, New Hampshire (*122, p. 125*)—Capacity from +1.2 to +2.9 per cent

In 1892 John R. Freeman made a series of tests on short lengths of lap-welded screw-joint pipe. Complete descriptive notes are in Table 1. The relative capacity of the old pipe (No. 202) was not computed as the age was not given, but the interior must have been in a very bad condition. Straight experimental lines nearly always yield test results better than should be anticipated in practice.

No. 211.—9-inch welded rolled-steel "line pipe"—Screw-couplings, Toconce line, Chile (*11*)—Capacity +2.2 per cent

After construction of a 98-kilometer water-supply conduit across high desert country to serve a copper-mine camp with domestic water, capacity tests were made on the first 5 kilometers below the supply springs. These experiments and general data on the line were reported by G. H. Bayles. The conduit throughout the reach tested was constructed of standard "line pipe" of nominal 9-inch size. This has an actual inside diameter of 8.94 inches. The joints in this kind of pipe are formed by screw threads inside of sleeve couplings. The threads taper so that there is a length of six or eight threads exposed on the inside, even after the units are tightly joined. The protective coating for this line was not specified. Unit lengths were set at 20 feet as a standard for computation but random lengths were shipped.

The capacity tests were made just after the line was finished. Quantity was measured through a Venturi meter and also over a weir at the head of the line. Loss of head was measured by pressure gauges. As the total loss ranged from about 35 feet for the lowest velocity to about 103 feet for the highest velocity it is apparent that the tests are reasonably acceptable, even taking into account the fact that close reading is not possible on the ordinary pressure gauge. In the article from which these data are taken the tabular figures for loss of head are not given but the platted points were scaled from a diagram. Because of the lack of consistency in the lower velocities and the necessity for scaling basic figures, the data are given only a C rating for the three low points, but a B rating for the three very consistent high points.

No. 212.—10-inch wrought-steel screw pipes—Morenci, Ariz. supply line (*45*)—Capacity —2.1 per cent

In 1908 new triple pipe lines were built to supply water to the town of Morenci for domestic and mining purposes. The lines were notable because of being subject to a maximum pressure head of 1,700 feet in a single pumping lift. The three lines were laid side by side, from the pumping plant to storage tanks about 5 miles away and 1,525 feet higher. All pipe was of wrought steel. The sections were threaded for 2½ inches with standard threads and screwed together with recessed line couplings. This description would place this pipe in class 2. The items in Table 1 under No. 212 are based on the following statement by W. L. DuMoulin (*45*):

When pumping through a 10-inch line, at the rate that represents 1,000 gallons of actual water per minute, the total head amounts to 1,700 feet. This makes the friction head about 175 feet in 25,650 feet of smooth, screwed, wrought-steel pipe; or 6.82 feet of friction head per 1,000 feet of length.

In recent correspondence with the writer Mr. DuMoulin gives the following additional information:

Special care was taken to check up the accuracy of the data given. The valves in one pump were carefully overhauled and so were in good condition. The plungers had been in service but a short time. Water-pressure gauges had been tested. The amount to be allowed for slippage and leakage had been determined by a series of tests. The water leaving the storage tank at Morenci was metered by a Venturi meter. The particular pump was operated at a uniform rate for about eight hours so that it delivered water at a rate of 1,000 gallons of actual water per minute through one of the 10-inch pipe lines, into one of the storage tanks at Morenci. The necessary information was taken to determine the actual quantity of water delivered into the storage tank which amount was also checked by the Venturi. This special run was made to check up the data to be published so that it would be as reliable as it might be feasible to obtain.

No. 214.—16-inch (outside diameter) lap-welded steel pipe, Glenwood Springs, Colo. (*143*)—Capacity +38.1 per cent

The trunk line of Glenwood Light & Water Co., serving Glenwood Springs, was tested after 22 years use. Pipe was 16 inches in outside diameter, of lap-welded steel; inside diameter was 15½ inches. The line was 6,229 feet long, of which the first 640 feet was spiral-riveted steel pipe full 16 inches in diameter. The alignment was free from sharp curves and well below the hydraulic gradient at all places. The pipe was laid in 1888 and tested in 1910. The elements follow: Quantity was determined by progressive fall in three wood-stave tanks at the upper end of the line. Simultaneously, recently checked pressure gauges were read at the power house at lower end of the line. Gross loss of head between water surface in the tanks and the power house was corrected for entry and velocity heads, leaving a net average loss due to friction, of 128.4 feet for the total distance of 6,229 feet. The average velocity during the 25-minute run was computed as 11.38 feet per second—a velocity so high that experimental errors and any question as to reliability of gauge were reduced to a minimum. Interior examination "showed the pipe to be very smooth on the inside and little corroded on the outside." In computing the retardation factors for the welded pipe the loss through the spiral-riveted pipe was first deducted, an arbitrary coefficient of 110 in the Williams-Hazen formula being used. Any reasonable assumption would have resulted in about the same values for the welded pipe. The high velocity in this line evidently produced a polished condition as the capacity found was far better than should be expected of a pipe 22 years old. If the high velocity can be assumed to maintain an interior surface as good as that of a new pipe of this type, the capacity is still excessive by 16.1 per cent.

No. 218.—36-inch lock-bar steel pipe—Montreal Water & Power Co., Montreal, Canada—Capacity +13.4 per cent

In correspondence with the manufacturers of the pipe, the Montreal Water & Power Co. reported tests of capacity on the 36,000 foot lock-bar conduit constructed in 1907, 1908, and 1909. Water was pumped into this line and measured through a Venturi meter. During a period of steady flow for three hours the loss of head shown by recording gauges was 28.27 feet. The gauges were compared with a gauge tester both before and after the run. During this test 11,800 gallons per minute were carried through the first 12,000-foot line where 2,333 gallons per minute, as measured by a Venturi meter, were diverted. Thus for the remaining distance the flow was 9,467 gallons per minute. With this information a common value for the friction factors was computed which would satisfy the flow in both reaches of pipe and agree with the total loss of head for the two reaches. The results agreed fairly well with the runs on the Bull Run No. 2 and No. 3 pipes in Portland, Oreg. (Nos. 225 and 226), but showed much better capacity than the Rochester lines (Nos. 220 and 222). It is difficult to compare this line when 3 years old with the Springfield line (No. 224) when 17 years old.

No. 220.—37-inch lock-bar steel pipe—Conduit No. 3, Rochester, N. Y. (*144, pp. 62, 70*)—Capacity when new −6.6 per cent; when 8 years old, −7.8 per cent

See description under No. 39 for general layout of the Rochester main pipes. Conduit No. 3 extending from overflow No. 1 (2¼ miles north of Hemlock Lake) to Rush Reservoir and Pinnacle Avenue tee, is composed of 7.75 miles of 37.08-inch cast-iron pipe constructed in 1914 and 17.75 miles of 37-inch lock-bar steel pipe of plate one-fourth inch thick completed in 1918. It contains 4 valves and 10 vertical and 20 horizontal curves, totaling 3,947 feet in length. The 1920 report of the department of engineering of Rochester (*144*) gave results of observations over periods of from four to five hours taken in the fall of 1918, 1919, and 1920. In correspondence with the writer, I. E. Matthews, in charge of the system, furnished data on an observation in October, 1926. These data indicated a pipe when new as 21 per cent inferior in capacity to the average of the Montreal and Portland lines (Nos. 218 and 226), and the deterioration in capacity as shown by the progressive gaugings was some 8 per cent in 8 years. It should also be noted that the gauging when the line was 2 years old indicated a capacity in excess of that when the line was new. Another comparison is that the line when new showed only about the capacity of the Springfield line (No. 224) when 17 years old.

No. 222.—37-inch lock-bar steel pipe—Conduit No. 3, Rochester, N. Y. (*144, pp. 62, 70*)—Capacity when 2 years old −12.9 per cent; when 8 years old, −13.4 per cent

See description under No. 39 for general layout and No. 220 for additional details. Between Rush Reservoir and Cobb's Hill Reservoir there was a line composed of 8,336.5 feet of 36-inch cast-iron and 42,134.2 feet of lock-bar steel pipe. This was part of conduit No. 3, finished in 1918. The 1920 report mentioned above (*144*) contained record of a test for capacity in 1920 when the line was 2 years old, and I. E. Matthews, in correspondence furnishes a report of a test on December 1, 1926. Each of these observations extended over a period of three hours. There were 33 horizontal deflections averaging 2° 08′ each, and 108 vertical deflections average 1° 53′ each; three 36-inch valves, one tee and 784.6 feet of cast-iron pipe. This line showed an excessive loss of head for this type of pipe. The 1926 test indicates that this line has approached the Wilkes-Barre pipe (No. 216) in deterioration of capacity.

No. 224.—42-inch lock-bar pipe, West Parish filters to Provin Mountain reservoir, Springfield, Mass.—Capacity +8.7 per cent

In recent correspondence with the writer, E. E. Lochridge, chief engineer of the municipal water works of Springfield, submitted the results of tests on a reach of 42-inch pipe 39,053 feet long. The pipe was the lock-bar type in units 30 feet long, with taper joints, single riveted between units; each unit shop dipped vertically in pitch. The interior was apparently in excellent condition on the last partial inspection in 1926. The pipe was put in service about January 1, 1910. The experiments were conducted on March 31, and April 10, 1926, by J. B. Porter, and the computations worked up by Allen Hazen. The essential data follow:

Quantity of water was taken as the mean of readings of combined flow through six Venturi meters at the filter plant and the reading of the single large Venturi plus the reservoir rise at Provin Mountain. The two items thus averaged, varied less than 2 per cent from the mean. Moreover, the lesser flow was indicated at the reservoir, which agrees with the fact that there is more or less leakage on the 7 miles of line. From regulator house to reservoir the loss of elevation by checked levels was nearly 59 feet. From this gross loss was deducted 8 velocity heads for $V=4.13$ feet per second or $8\times0.266=2.13$ feet. Mr. Hazen assumed that the 8 velocity-heads lost were made up as follows: 2 at entrance, 1.64 at each of two 30-inch gates, and 2.73 at the 36-inch gate at Provin Mountain bottom chambers.

No. 225 and 225a.—44-inch and 52-inch lock-bar steel pipe—Bull Run conduit No. 2, Portland, Oreg.—For No. 225, capacity +19.7 per cent, and for 225a, capacity +14.7 per cent

In 1911 the city of Portland, laid conduit No. 2 paralleling No. 1 (see pipes 30, 32, and 45) from Bull Run River to the city. In 1922 F. M. Randlett conducted capacity tests on long reaches of the pipe. In correspondence with the writer

B. S. Morrow gave the data of Mr. Randlett's tests. The loss of head was determined by water columns attached to piezometers located at the ends of the reaches. The quantity of flow was measured through a Venturi meter and with weirs located near the outlet end of the line. From Bull Run River to Lusted's Hill the line was 52 inches in diameter, of plate steel from one-fourth to seven-sixteenths inch thick, with two longitudinal seams held in lock bars. The sections were 30 feet in length with field seams girth riveted. The line was provided with 4-inch air valves on the summits and 8-inch blow-offs at the depressions. The flow tests at the pipe's age of 11 years showed little or no deterioration in capacity. This statement appeared to apply to all the Portland lines taking water from Bull Run River.

From Lusted's Hill to the end of the line the pipe was 44 inches inside diameter, of ¼-inch plate, with similar lock bar straight seams and riveted girth seams.

Two reaches of the 52-inch line were tested. Between stations 1601+59.9 and 1486+70 the total loss of head was 29.65 feet, giving a loss per 1,000 feet of 2.580 feet with an indicated value of the Williams-Hazen C_w of 96. The writer has not tabulated this test because Mr. Randlett says air was trapped in the summits along this part of the line, as a result of faulty action of air valves. The second reach, between stations 1486+70 and 1073+80, a distance of 41,290 feet, gave a loss of head of 1.472 feet per 1,000 feet of pipe. The computed values of K_s=0.309 or C_w=130 are more indicative of this type of pipe and agree more nearly with the values found on the 44-inch line following the 52-inch pipe; that is, a value of K_s=0.285 or C_w=135.

No. 226.—58-inch lock-bar steel pipe, Bull Run conduit No. 3, Portland, Oreg.—Capacity from +8.1 to +14.7 per cent

In correspondence with the writer J. C. Stevens gave the elements of tests made under the direction of F. M. Randlett and B. S. Morrow. This series covered four reaches of the conduit:

Section	Length	Total angles	
	Feet	°	
A	2,850.0	283.6	Contains 1 Venturi meter, 58 inches to 24-inch throat.
B	8,702.4	415.2	
C	7,918.6	596.8	
D	30,961.4	1,448.4	Includes 1,468 feet of triple-riveted butt-strap pipe.

This series of tests was on a new pipe which diverted raw water from Bull Run River for municipal use in the city of Portland. The pipe was built in 30-foot sections joined by the usual riveted-taper method. The water flowed with the laps. Prior to the tests the flow was gradually built up for two weeks during which the air valves were watched. Loss of head was determined with water columns attached to piezometers at points where but little pressure existed. Quantity was taken as the average of flows indicated by two Venturi meters and a weir located near reservoir No. 1 at the outlet of the pipe. The total loss of head in section A was 7.22 feet, of which so much was ascribed to loss through the meter that results are not given in Table 1.

The results of these tests agreed quite closely with those on the Montreal line (No. 218) but disagreed with those on the Springfield (No. 224) and the Rochester (No. 220 and 222) lines.

No. 228.—96-inch bump-jointed lap-welded pipe—Pit No. 1, Penstock, Pacific Gas & Electric Co. Shasta County, Calif. (*124*)—Capacity −2.5 per cent

Shortly after construction of Pit No. 1 power plant on Pit River, Calif., tests for retardation losses were made at two flows of 712 and 982 second-feet. These tests, made by Roy Wilkins, comprised measurements on the tunnel from forebay to penstocks and on one of the twin steel penstocks. The other steel pipe was held idle and served as a static water column. Six piezometer connections were made at the inside ends of the horizontal diameters. Between the pipes these piezometers were connected to the two legs of a mercury U-tube manometer. Thus one leg was subject to static pressure alone and the other to the same static head less the velocity head, entry losses, and friction loss from the intake of the penstocks to the station of the connections. Water was run in both penstocks up to the time of tests so that the water was of the same temperature. The sequence of important test stations follows:

Gauge 1 was on the 129-inch pipe just below the standpipe and valve at the penstock entrance from the surge chamber at the tunnel outlet. Gauge 2 was located 231 feet down a straight pipe to which it was attached just above a 43-foot taper section from the 129-inch to the 108-inch pipe. On the 108-inch line, gauge 3 was located at the outlet of the taper just mentioned. This location was of questionable desirability for a single piezometer connection, but was probably the best available, as the same doubt would be attached to any single connection within at least 100 diameters below a straight taper section without curve transitions. Gauge 4, at the lower end of the reach of 108-inch pipe, was located 515 feet from gauge 3 and just above a 39-foot taper from the 108-inch to the 96-inch pipe. Gauge 5, on the 96-inch pipe just below the taper, was subject to the same doubt as gauge 3. Gauge 6 was 240 feet down the 96-inch pipe beyond an upward angular bend. Quantity of water was measured over a Francis suppressed weir 130 feet long. It was appreciated that Francis's 10-foot-maximum weir formula was being greatly extended, but this method is customary and checked the flow computed from the velocity-head differential at gauges 2 and 3, the taper section being used as a Venturi meter without the usual expanding portion.

This reach of pipe had the longitudinal seams lap-welded and the girth seams riveted in the "bump." (Fig. 1.) The pipe rings, of $^{11}/_{16}$-inch and $^{3}/_{4}$-inch steel, were all of the same diameter. The reach tested was 240 feet long between gauges 5 and 6, and contained one expansion joint and three vertical angles ranging from $15\frac{1}{2}°$ to 23°. In plan view it was straight. At the upper end it was joined by a 39-foot taper section to the 108-inch pipe (No. 230). The lower end was 220 feet above the turbines. Note that the retardation factors were in close accord with those on the adjoining 108-inch sections. (No. 230 below.)

Nos. 228 and 230 support the tentative formula and coefficients suggested in this bulletin for inclusion of bump-joint pipe in class 2, as they both are within 3.5 per cent of the formula if the two runs in each pipe are taken as indicative. Note that the values of C_w are consistent between 112 and 114. These values, for new pipes, are very far from the value of 140 in the Williams-Hazen formula suggested in some catalogues for this type of pipe.

No. 230.—108-inch bump-jointed lap-welded steel pipe—Pit No. 1 penstock, Pacific Gas & Electric Co., Shasta County, Calif. (124)—Capacity −3.4 per cent

These tests cover two flows at velocities of about 11 and 15 feet per second respectively. The reach, 515 feet long between gauges 3 and 4, is straight in plan view but contains two angles of about 20° each in profile. It is joined by taper sections to the 129-inch pipe above it (No. 79) and to the 96-inch pipe below. (See No. 228 for methods and equipment and comments on relative capacities.) It contains three expansion joints. In construction it is identical with the 96-inch pipe described as No. 228 except that the plates run from one-half to five-eighth inch in thickness. The retardation factors are very close together.

CONTINUOUS-INTERIOR PIPE

Nos. 302 and 304.—4-inch and 6-inch (outside diameter) lap-welded wrought-steel pipe, bell-and-spigot joints—Experimental pipe, Versailles, Pa.—Capacity +4.3 per cent for No. 302 and +1.6 per cent for No. 304.

In correspondence with the writer, F. N. Speller gives results of tests on three reaches of experimental pipe (Nos. 302, 304, and 310) set up perfectly straight in both plan and profile. (Pl. 6, B.) These tests were made by F. W. Frederick and J. J. Wilson with assistants, under the general direction of Mr. Speller. In all cases the lines were laid side by side, 1,000 feet long between piezometer connections. Quantity of water was determined by volumetric measurement for small flows and a Venturi meter for larger flows. The Venturi meter had been checked the previous year by pitometer readings. Pressure head was determined by mercury manometers of U-tube form, attached to manifold rings having access to the water prism through eight evenly spaced $\frac{1}{4}$-inch holes. Loss of head was, of course, the difference in elevation of the water columns equivalent to the mercury columns. Gauge No. 1 was placed 105 feet downstream from the elbow at the pump; likewise gauge No. 2 was 105 feet above a return bend. The pipes were 1 year old. Each had been dipped in coal-tar pitch. When torn down the lines were found to have deposits of mud up to one-sixteenth inch thick on the lower third of the perimeter. The 4-inch and 6-inch pipes (outside diameters) were respectively 3.628 and 5.72 inches inside diameter. The lengths averaged 17 feet 1 inch for the 4-inch size and 18 feet 11 inches for the 6-inch size. The inside diameter of the 4-inch size was determined by weighing the water contents of six average pieces. The inside of the 6-inch size was measured with micrometer calipers.

These tests appear to have been carefully conducted and are given an A rating for all observations consistent with each other. However, the writer has found experimental pipe in straight reaches to yield results that can not be duplicated with certainty in commercial operation. These pipes and No. 310 should be compared with the writer's series No. 308 on a field line of the same kind of pipe.

No. 309.—6-inch wrought-iron pipe, flange-connected—Cornell University, N. Y. (*122, p. 129*)—Capacity +2.1 per cent

H. F. Mills gives the elements of experiments by E. W. Schoder of Cornell University. This series was made on a straight reach of lengths of 6-inch pipe that "had been used in a steam-heating main for some years." The total length was 122 feet of which 99.33 feet was tested for the loss of head. Gauge No. 1 was placed 20.04 feet downstream from the intake end. Each piezometer consisted of two opposite holes in the shell, into which ⅛-inch cocks were screwed. Pressures from the two holes at each end of the reach were brought together and indicated in a water differential gauge. Quantity was measured volumetrically in a calibrated tank of 500 cubic feet capacity. Because of the short reach tested this series has been given a B rating. However, the results are quite consistent and do not show the relatively low capacity that might be expected from the description.

No. 310.—8-inch lap-welded steel "line pipe" fitted with bell-and-spigot joints—Experimental pipe, Versailles, Pa.—Capacity +10.8 per cent

This is one of three pipes described under Nos. 302 and 304. The only different conditions applying to the larger pipe follow: Unit lengths averaged about 20 feet. Inside measurements were taken with micrometer calipers on each of the 84 units. This method was checked by filling four units of pipe with water and weighing this water. The agreement in size was so close that the nominal diameter, 8 inches, was used in the calculations. (See comments under Nos. 302 and 304.)

No. 311.—14-inch full-welded pipe—Main supply line, Bend, Oreg.—Capacity +8.1 per cent

The city of Bend, Oreg., in 1926 completed a new water-supply main, consisting in sequence of 27,888 feet of 14-inch pipe; 20,691 feet of 12-inch pipe and 13,515 feet of 16-inch pipe. At the end of each reach the pressure was fully relieved. In correspondence with the writer, the late John Dubuis, then engineer in charge of the project, reported the following tests on the 14-inch pipe made soon after the line was put in service: Water at the diversion dam entered screen chambers and passed over two 6-foot rectangular weirs with suppressed contractions. Below the weirs a taper section 6 feet long led to the 14-inch pipe tested. This line was formed of 30-foot units, field welded together by the torch method. Each unit was formed of a single sheet, shop welded along the straight seam and then dipped in a hot compound.

After the field joints were made all uncoated spots were given two coats of asphalt applied with a brush. As shown in Plate 5, C, the seams as formed by the weld were evidenced by a small irregular bead, which was smoothed by the asphalt coat. At the time of test the 14-inch pipe had been running at full capacity—the water backed up into the weir chamber—for several weeks. The quantity of flow was the sum of the flow over one weir under a head of 0.36 foot and over the other under head of 0.37 foot. The total fall as determined by checked levels, was 381.09 feet. Of this, 1.24 feet was deducted for entry and velocity head for the velocity of 8.34 feet per second. This deduction was based on a coefficient of 0.5 for entry loss into the 24-inch opening to the taper section and a coefficient of 0.1 of the difference in velocity heads for the taper between the 24-inch opening and the 14-inch pipe. To these losses was added the velocity head for the velocity in the 14-inch line. The writer regards this as an excellent test; the great length of pipe, the high velocity, and the great loss of elevation due to retardation of flow were all such that any conceivable error would not materially affect the results. It will be extremely interesting to observe the progressive capacity of the line, which receives clear untreated water from a mountain stream in a lava terrain.

No. 312.—20-inch outside diameter pipe, coupling-jointed, lap-welded pipe—Lake Moraine-Ruxton Park penstock—Colorado Springs municipal power plant, Colorado—Capacity +6.4 per cent

This line, constructed in 1924, was dipped in an asphaltic compound, both inside and out, and laid on a continuous falling grade. Bolted couplings were used throughout with curves having a radius of from 100 feet to 717 feet; 36.5 per cent of the alignment is on curves and 63.5 per cent on tangent. (Pl. 3, B.)

At the upper end, some 200 feet below the outlet of Lake Moraine, a 16-inch relief valve was installed to relieve back pressure caused by operation of the automatic hydroelectric plant connected therewith. Within the power house at Ruxton Park a Venturi meter is located, just ahead of the nozzle. Capacity tests were made under the direction of E. L. Mosley, manager of the power department for the city of Colorado Springs. During the tests a new recording pressure gauge, graduated from 0 to 15 pounds per square inch, was placed on the relief valve, and in the power house at Ruxton there was placed a new recording gauge, graduated from 400 to 600 pounds per square inch; and all data taken in these tests were by means of these instruments, and simultaneous readings were taken at each end of the line. Readings of the upper gauge are acceptable for such a relatively long line, but small losses of head on the lower gauge, recording pressures in excess of 1,000 feet, are, of course, very questionable.

The actual length of pipe laid between the two pressure gauges is 12,528 feet, made up as shown in Table 14.

TABLE 14.—*Dimensions of Lake Moraine-Ruxton Park penstock*

Length (L)	Shell plate thickness	Inside diameter (d)	Inside area (a)	Weighting product, $L \times a$
Feet	*Inches*	*Inches*	*Sq. ins.*	
3,426.1	5/16	19.38	294.83	1,010,117
4,852.6	3/8	19.25	291.04	1,412,301
2,695.7	7/16	19.12	287.27	774,304
1,136.5	1/2	19.00	283.53	321,918
418.3	9/16	18.87	279.81	117,044
Total 12,528.2				3,635,804 = 290.2 square inches.

The weighted average inside area is 290.2 square inches, or 2.015 square feet, which is the area of a pipe 19.2 inches in diameter. All computations are based on this value of d.

The elements of the experimental data as given in Table 1 are themselves taken from a smoothed curve based on the original tests. A table of data from Mr. Mosley gives the measured loss of head for each unit quantity from 0 to 16 second-feet. Observations for flows below 6 second-feet are not included in the table, being considered by the writer to have little value. For $Q=6$ second-feet the observed total loss of head was but 12.4 feet, while the total pressure head recorded at gauge No. 2 was more than 1,100 feet. As the flows are increased the recorded loss of head becomes a larger part of the total head. In other words, these data are considered the more reliable. For a flow of 16 second-feet, with an average velocity throughout the reach of 7.94 feet per second, the total loss of head is 120.9 feet, with a pressure head at the lower gauge of 1,008 feet. Thus for the higher flows the values of the retardation coefficients are reasonably acceptable and agree quite closely with comparable values in other series.

In column 3 of Table 1 it will be found that unacceptable D ratings are assigned to flows below 10 second-feet and the ratings are increased as the friction heads become a greater percentage of the total heads as indicated on the lower pressure gauge. For the flows rated C and B the average capacity compared with the writer's formula is excessive by 6.4 per cent. However, it will be noticed that all the retardation factors have credible values for the higher flows but are without precedent for the lowest flow included in Table 1, and these values were completely incredible for the flows in the zone of greatest experimental errors not included in the table.

No. 314.—30-inch steel lock-bar pipe—Coolgardie supply main, Western Australia (*111, 131, 173, 130 177*)—Capacity −4.4 per cent

The first of the modern long conduits was finished in 1902. In order to serve the gold field of Coolgardie, it was necessary to pump water across 307 miles of desert country from the Helena River near Perth, in Western Australia. For an additional 44 miles the flow was by gravity. The static pumping lift was 1,290 feet and with the estimated friction loss the total pumping head was 2,655 feet, distributed among eight pumping plants. The type of pipe chosen was the forerunner of the present lock-bar pipe. It consisted of a steel line of ¼ and 5/16-inch plate; its inside diameter was 30 inches. The shop units were exactly 28 feet long with the single long seam formed by a straight "locking bar." These units were true cylinders without taper; thus the ends of adjoining sections abutted together with a flush interior. The girth joints were formed by calking lead in the annular space between the pipe ends and sleeve joint rings 8 inches in width. This pipe with "continuous interior" comes under class 3 as described on page 12, rather than under class 2, which applies to the standard riveted girth joint of the present lock-bar type of pipe.

The coating of this line received particular attention, as maximum temperatures of 170° F. were to be encountered. The greater number of the pipe units were dipped in coal tar mixed with equal parts of refined Trinidad bitumen. Soon after completion of theline tests were made and reported by C. S. R. Palmer in his comprehensive paper on this, the longest conduit in the world (131). The line was equipped with eight Venturi meters—one at each pumping plant. These furnished a method of determining quantity. The exact manner in which the loss of head was measured was not given by Mr. Palmer, but such great lengths of pipe were available that there should have been no extraordinary error in his measurements.

The results of these tests indicated a value $K_s=0.35$ or of $C_w=127$ for this pipe when new.

In his paper on this line Longley (*111*) gave a table showing the depreciation in capacity from year to year, in terms of coefficients in the Williams-Hazen formula. Longley mentions a coefficient when the pipe was new of $C_w=135$, and notes that there is inconsistency between this figure and another of $C_w=128$ resulting from tests, also made when the pipe was new. The latter value is probably related to the data given in Table 1. In the design of the pipe a coefficient equivalent to $C_w=96$ was used. The actual value for the new pipe was probably quite close to 130. At the age of 6 years this value, which is proportional to the capacity, had diminished to an average of 97.6, representing a variation from 111 at the inlet end to 95 at the outlet end. In 1920, after 18 years of use, the average value of C_w had become 77 without any noticeable difference at the two ends. The rapid loss of capacity is ascribed to tuberculation within the pipe, there being no possibility of sand or other débris deposits. As the tuberculation was the result of chemical reactions requiring a supply of dissolved oxygen a deaeration treatment was started in 1917. The results of this treatment are difficult to determine definitely, as the line was badly tuberculated when the treatment was started. However, the evidence indicates that depreciation in capacity had been stopped on 7 of the 12 sections of pipe.

In the discussion of Colonel Longley's paper the statement is made by F. N. Speller that mixtures of coal tar and asphalt should be avoided; that either one when properly applied by itself lasts longer than the combination.

Considering the conditions of installation, the capacity of this line when new supported the value of $K_s=0.32$ for new pipes of continuous-interior type.

Nos. 316 and 318.—84-inch butt-joint riveted pipe (rivet heads countersunk)—Penstocks Nos. 2 and 3, Pacific Mills—Essex Co., Lawrence, Mass. (*122, p. 194*)—Capacity at 2.3 years, No. 316 was −5 and No. 318 was −10 per cent, and at 13 years, No. 316 was −15.2 and No. 318 was −13 per cent

These short penstock pipes were tested under direction of H. F. Mills. Reaches 100 feet long were chosen from the total length of 190 feet. The quantity of flow was determined by weir or by opening of the gate wheel as calibrated against velocity measurements by long floats in the leading canal. Flush piezometer connections led to glass gauge tubes. Penstocks were made of steel plates, riveted with butt joints and rivet heads countersunk. Plates were shop coated with two coats of hot asphalt and linseed oil; a third coat was put on when the pipe was in place, "thus forming a smooth hard interior surface throughout."

First tests were made when the pipe was 2 years old. They were repeated from time to time for 11 years. Average deterioration in capacity of the two penstocks is expressed by $H=0.0320(1+0.17\sqrt{t})V^2$ where t is age in years. Mills gives details of the tests at 2 and at 13 years of age, with some derived figures for intervening tests, but the latter are not included in this bulletin. Mills describes the gradual growth of tubercles, sponge, slime, and mud, toward the end mentioning "many tubercles the size of a pea," "rough surface" due to mud and tubercles, and "tubercles, one per square inch."

While Mills's description of these lines clearly indicates that they should be classed as "continuous-interior" pipe of the highest grade, the retardation factors do not support this assumption.

COMPOUND PIPE

No. 150.—36-inch riveted wrought-iron and 24-inch cast-iron pipe—Conduit No. 1, Rochester, N. Y. *(170, 159, 140, 86, 122)*

Hemlock Lake conduit, now called conduit No. 1 of the Rochester supply mains, was laid in 1873–1875 between Hemlock Lake and Rush Reservoir. It was a compound pipe of the following general specifications: 9.62 miles of 36-inch riveted wrought-iron pipe three-sixteenths inch thick; then 2.92 miles of similar pipe 24 inches in diameter, of plate three-sixteenths and one-fourth inch thick; then 6.12 miles of 24-inch cast-iron pipe. In 1876 a test of capacity was made under the direction of J. N. Tubbs, then chief engineer. It comprised nothing more nor less than a determination of the total fall from intake to outlet and a quantity measurement by rise in water at Rush Reservoir. The maximum flow, as made public at that time *(170)* was 9,293,000 gallons per day. Assuming a value of 130 in the Williams-Hazen formula for the cast-iron pipe, the computed value for the 24-inch and 36-inch riveted steel pipe would be $C_w=153.5$, which would show a far higher capacity than present-day data indicate. If the assumed value of C_w for the cast-iron pipe be set at 140—the maximum value suggested by Williams and Hazen—the computed value for the riveted pipe becomes 134.6, still a very high value, in the light of present knowledge.

Table 2 gives figures based on the average of three flows taken several days apart, in 1876. The base data are taken from Herschel's notes *(86, p. 16)*, quoted from a report by Kuichling. The resulting retardation factors, computed after assumptions are made as to the value of C_w for the cast-iron reach, all indicate a very high capacity for this type of pipe, but are not entirely unsupported by other tests. The writer gives these tests a D rating, indicating that they should not be given weight; but he believes they should be included in a comprehensive paper such as this, for historical reasons. Later experiments on the 24-inch and 36-inch riveted lines are described as Nos. 23, 24, and 39. The results of the 1876 tests became the cause of bitter controversy during the "Nineties," as the information had been used in the design of a large pipe in New Jersey, under a guarantee of capacity that was not met by the completed pipe. Herschel describes this controversy quite fully *(86)* and comes to the conclusion that the Rochester conduit had never carried the quantity claimed in the published reports of the 1876 tests *(170)*. In the light of present extensive data, the writer agrees with Herschel, but the 1876 test was apparently given full weight by such authorities as Hamilton Smith *(159, p. 235)* and Mills *(122)*, who were both contemporaries of Tubbs. Both writers give Tubbs' data without adverse comment as to the excessive capacity indicated.

No. 152.—Riveted-steel compound pipe—Penstock, Spring Gap power house, Pacific Gas & Electric Co., California

This plant is served by a penstock 7,415 feet long made up as follows:

Length	Diameter	Pipe characteristics
Feet	*Inches*	
2,410	36.57	Lap riveted, made of $\frac{1}{4}$ to $\frac{9}{16}$ inch plates.
1,320	36.07	Triple-riveted double butt-strap pipe, $\frac{9}{16}$ to $\frac{5}{8}$ inch plates.
2,310	30.21	Triple-riveted butt-strap pipe, $\frac{3}{8}$ to $\frac{13}{16}$ inch plates.
1,375	30.0	Triple-riveted butt-strap pipe, $\frac{3}{8}$ to $\frac{15}{16}$ inch plates.

During tests for efficiency of the machines, made in 1921 when the penstock was new, the over-all loss of head throughout the compound pipe was determined by Roy Wilkins.

Quantity of water was measured over a weir. Loss of head was determined by pressure gauge reading only to the nearest 5 pounds. Flow ranged from 18.6 to 39.4 second-feet, developing a loss of head from 11.5 feet to 80.7 feet. The reach was so long and, for the higher velocities, the losses were so appreciable that the usual errors of such a pressure gauge were minimized; however, the lesser reading to nearest 5 pounds may be much in error.

This pipe had been given one brushed coat of red lead in linseed oil. It was designed after the Williams-Hazen formula, with $C_w=100$. These tests indicated that an average value of $C_w=76.5$ satisfies the loss for its particular velocity in each size of new pipe. However, this method does not assign a variable loss to the various kinds of pipe, the quoted value resulting from the two highest flows. Flows at the lesser velocities indicated values of C_w up to 104. These values are so far apart that no definite deductions are possible, except that flow in heavy butt-strap pipe is characterized by a very high retardation loss.

No. 156.—Riveted and welded compound pipe—Penstock, Kern River No. 3 power house, Southern California Edison Co., Kern County, Calif.—Capacity +6.1 per cent for No. 156 and +4.1 per cent for No. 156a.

This plant is served by two penstocks, each having the following sequence of sizes and types of pipe. Plate thickness varies from three-eighths to seven-sixteenths inch:

Length	Size of pipe	Plate thickness	Type of riveting
Lin. ft.			
316	84-inch	⅜-inch	Riveted pipe.
237	78-inch		Do.
208	72-inch	7/16-inch	Do.
853	66-inch		Lap-welded bump joint.
870	60-inch [1]		Do.

[1] Including Venturi meter.

Longitudinal seams in riveted pipe are double-riveted and triple-riveted lap joints while all girth seams in riveted pipe are butt joints with outside butt strap. In the welded pipe the girth joints are all bump joints with double lines of rivets. The inside of this penstock was brushed with one coat of graphite.

A series of tests by H. L. Doolittle was made on the over-all losses, from which were deducted the velocity heads for the velocities in the 60-inch pipe, plus the loss through the Venturi meter, plus the losses at entrance to the various tapers as the pipe became smaller. The quantity of flow was measured through Venturi meters and the loss of head was measured with mercury U tubes. The Edison Co., in sending the writer data for these tests, included a logarithmic diagram showing a straight line loss-of-head law on the assumption of a value of n in the Kutter formula of 0.0112 for the welded pipe and 0.016 for the riveted pipe. Corresponding round-number assumptions in the Williams-Hazen formula would have been $C_w=130$ for welded pipe and 100 for the riveted pipe. It should be noted that the Pacific Gas & Electric Co. found a value for similar bump-jointed welded pipes alone as about 114. (See Nos. 228 and 230.)

This penstock was designed with the use of the Kutter formula using $n=0.012$ for the welded pipe and 0.016 for the riveted pipe.

The computations in columns 11 and 13, Table 2, were based on assumed values of $C_w=105$ and $K_s=0.44$ for the riveted pipe and computing the resulting values of C_w and K_s for the welded pipe. These assumptions were made because only about 10 per cent of the friction loss is due to the flow in the riveted pipe; furthermore, enough data on riveted pipes are at hand to justify the values of C_w and K_s for such pipes, although all the data available are needed to support deductions made for welded and other pipes of types 2 and 3. It should be noted that Wilkins' tests on Nos. 228 and 230—welded pipes with bump-girth joints—showed capacities slightly deficient in capacity when judged by the writer's formula for pipes of class 2.

No. 160.—Compound riveted steel pipe—Little Brush Creek siphon, Kern River No. 3 conduit, Southern California Edison Co., California (*127, p. 5*)—Capacity −5 per cent

Immediately after the completion of the conduit and power house at No. 3 plant, Kern River, Calif., experiments were made by Doolittle (*127*). Correspondence with the company disclosed that the data were platted directly on the diagram given in the cited report. From the platted points the elements given in Table 2 have been computed.

This siphon was 1,170 feet in length and the bottom of the sag lies 327 feet below the hydraulic gradient. This gave two straight legs joined by an abrupt bend at the sag. From inlet to outlet the sequence of pipe sizes was as follows:

Length	Diameter	Plate thickness	Type of riveting
Feet	*Inches*	*Inches*	
245	114	⅜	Double-riveted lap, cylinder joints.
40	108	----------	Triple-riveted lap, cylinder joints.
104	102	----------	Double-riveted, butt joints.
340	96	23/32	Triple-riveted, butt joints.
120	102	----------	Double-riveted, butt joints.
44	108	----------	Triple-riveted lap, cylinder joints.
243	114	⅜	Double-riveted lap, cylinder joints.

It is to be noted that there were six changes in diameters, by 6-inch increments. These changes were effected by taper transition sections. The values C_w and K_s satisfy the observed loss of head for the proper velocity in each individual length of pipe for each given quantity of flow. The lengths of pipe of the same size are combined for computations. No correction was attempted for the local losses in the taper sections and in the bend at the sag. The pipe is considered as being all of type 1d as the losses in the butt-strap pipe predominate. Without the adjustments the average values of K_s and C_w are 0.573 and 89.7, respectively. However, it is to be noted that K_s changes from 0.517 for the lowest flow to 0.623 for the highest, the corresponding change in C_w being from 95.5 to 85.1. The losses through the taper sections and for the bend at the sag should not be ignored. When more data are available this series of tests may contribute additional evidence as to losses in such taper sections and bends.

It is to be noted that the greatest local loss occurs through the sections of heavy butt-strap pipe. This is taken as sufficient to place the whole line in class 1d.

No. 162.—24-inch and 22-inch welded-steel compound pipe—Municipal supply, Gordon Valley pipe line, Vallejo, Calif.—Capacity +2.9 per cent

In correspondence with the writer, Augustus Kempkey submits data on a preliminary test for capacity of the new trunk line supplying the city of Vallejo, Calif. This pipe is full welded, composed of 59,267 feet of 24-inch pipe of 3/16-inch plate, 20,488 feet of 24-inch pipe of ¼-inch plate, and 35,260 feet of 22-inch pipe of 3/16-inch plate, making a total length of 115,015 feet, or about 22 miles. The field sections were about 28 feet long, each made of two cylindrical courses, the ends of the smaller cylinders fitting into the larger ones successively. The longitudinal joints were butt welded with an electric arc; the field joints, at every alternate girth seam, were hand welded with an electric arc, the smaller rings being lapped into the larger. Sections were shop dipped in a hot asphaltum compound and then wrapped with a mica covering as a soil protection; field joints were painted with the same asphaltic compound and wrapped.

Shortly after completion of the line a set of preliminary runs was made to ascertain its approximate capacity. A new Venturi meter offered an excellent means for the measurement of the flow. The overall loss of head was determined by reading pressure gauges of the Bourdon type, these readings being to the nearest pound only. Thus the error of reading might be one-half pound either way, covering a range of 2.3 feet of head at each end of the line. However, the total loss of head was so great, especially for the greater flows, that the data are probably more valuable than more precise readings on a short reach of pipe, although Mr. Kempkey does not offer them as the results of finished tests. This line has no projecting rivet heads; on the other hand, the interior is not "continuous," as the term is used in this bulletin. However, a comparison will be made with class 3—continuous-interior pipe—on the ground that computations are made on the basis of the interior diameter of the smaller rings of pipe and the lower velocities, with the consequence that the lower friction losses in the larger rings compensate for the enlargements and contractions of the lapping cylindrical courses. The average value of K_s=0.302 (C_w=138), compares with 0.304 for the welded portion of No. 156, with 0.287 for the straighter full-welded line at Bend, Oreg. (No. 311), and with 0.342 for the very crooked Marin line (No. 313). These values all indicate that the writer's recommended K_s'=0.32 is conservative.

DREDGE PIPE

No. 402.—15-inch steel pipe, Taylorsville dam, Ohio (93)—Capacity −4.2 per cent

At the time of the construction of hydraulic-fill dams for the Miami conservancy district, Ohio, a series of tests on the loss of head in dredge pipes carrying clear water was carried out by I. E. Houk. Identical methods were employed for all three pipe lines tested. Loss of head was determined by a carefully calibrated pressure gauge. Velocities were ascertained by lowering packages containing 1 ounce of potassium permanganate and 1 pound of salt into the suction pipe, which were well distributed in the water by the pump runner and observed at the outlet ends of the line. The permanganate's purple color was visible when clear water was being tested. The salt solution was electrically observed, electrodes of copper and zinc strips being placed one-fourth inch apart in the outlet jet. The needle deflection on a voltmeter checked the optical indication of the color, indicating the feasibility of using the salt alone when mud and gravel were running in the pipe. The pipe listed as No. 402 was 2,075 feet in length, composed of 74 lengths of riveted soft steel and 56 lengths of welded hard steel. The pipe units were approximately 16 feet long and were fitted together by "stovepipe" joints. This reach contained one flap valve; one 30°, four 45°, and two long-radius 90° bends, and one Y unit. Obviously a pipe in this service would be highly scoured on the inside.

No. 404.—15-inch steel pipe—Taylorsville dam, Ohio (93)—Capacity +8.9 per cent

Exactly similar to the tests described under No. 402 above were those conducted by I. E. Houk upon another reach of dredge pipe composed of 8 joints of welded hard steel and 76 joints of riveted soft steel assembled by slip joints into a length of 1,340 feet. This reach included one flap valve, one 30° bend and two 45° bends, and two long-radius 90° elbows. This series also was run with clear water alone. The indicated capacity was superior to that of No. 402, although the relative amounts of welded pipe and other conditions of retardation would tend to produce the opposite effect.

No. 406.—15-inch steel pipe—Englewood dam, Ohio (93)—Capacity −1.4 per cent

Mr. Houk reports a series of observations made on a reach of dredge pipe at the Englewood hydraulic-fill dam in a manner identical with that described under No. 402. This reach was 780 feet long, consisting of 20 lengths of riveted soft steel and 26 lengths of welded hard steel dredge pipe and 60 feet of standard wrought-iron pipe bolted together with screw flange joints. In the reach were two 15° bends, one long-radius 90° elbow, and one flap valve. Clear water was used for this series of runs.

Nos. 408 to 422, inclusive.—Lap-riveted steel pipe, from 31.88 to 34 inches, inside diameter—Discharge pipes from Mississippi River dredges (*115, 15*)—Average capacity less than −1 per cent

F. B. Maltby gives some notes on the loss of head due to friction in pipes conveying dredged materials, the dredges being the large type used on the Mississippi River (*115, p. 458*). This loss was measured by piezometers inserted in the pipe lines at various points and by simultaneous readings of gauges attached to these piezometers. Velocities in the pipes were determined with Pitot tube. For all the lines except the *Beta* (No. 418) the loss included that at the joints as well as through the unit lengths of pipe. The *Beta* line was straight, of ¼-inch plate, 33 inches inside diameter, made up in rings about 5 feet in length, these rings lapping in the direction of flow. All rivet heads were countersunk, the pipe being perfectly smooth and bright inside as a result of scour of the dredged material. For the *Beta* line capacity was 0.8 per cent less than that called for by the writer's formula. The pipe connections on the *Kappa* and the *Flad* were ball-and-socket joints permitting deflections of about 20° either way. The indicated capacities were 7.7 per cent excessive and 8 per cent deficient, respectively. For the other six dredges the joints were short pieces of rubber hose which slipped over the ends of adjoining lengths. These lengths were generally 50 feet, but the *Beta* line had some 100-foot lengths. Mr. Maltby platted his observations for loss of head, but the writer's figures are taken from the table given by Mr. Berridge in (*15*).

SPIRAL-RIVETED PIPE

Nos. 502 to 514.—4-inch and 6-inch spiral-riveted steel pipe, asphalt coated—Experimental lengths, Cornell University, New York (*151*)

In 1904 and 1905 experiments were conducted on 60-foot lengths of nominal 4-inch and 6-inch spiral-riveted asphalt-coated pipe, the flow being tested both with and against the laps. The results of these tests were reported by E. W. Schoder and H. A. Gehring. The quantity of flow was measured volumetrically. The loss of head was measured by water-column differential gauges. All pipes were 120 feet long, of which the lower 40 feet comprised the test length of the nominal 4-inch pipe and the lower 60 feet that of the 6-inch size. All piezometer holes were 1 inch in diameter, carefully reamed out and all located in similar position with relation to the laps. The mean diameters of the experimental sections were 4.084 inches for the nominal 4-inch and 5.962 and 5.943 inches for the nominal 6-inch. The flat-head rivets and asphlatum coat contributed to a capacity in excess of that indicated for similar pipe when galvanized only, such as those listed under Nos. 520 to 533. If the tests on pipes 502 to 514 be taken as indicative, conservative values of K_s are 0.38 for new-pipe flow with the laps and 0.40 for flow against the laps. The value for flow with the laps would thus be identical with that offered for ordinary riveting in sheet-metal pipes in class 1a. (See discussion under Nos. 520 to 533, following.)

Nos. 520 to 533.—4-inch, 6-inch, 8-inch, and 10-inch spiral-riveted steel pipe, galvanized—Experimental lengths, Purdue University, Indiana (*73*)

Beginning in 1917, thesis work of senior students of Purdue University, supplemented by work and further computations of F. W. Greve and R. R. Martin, resulted in extending the work on spiral-riveted pipes to include those up to 10 inches in diameter. Experiments were conducted on reaches approximately 40 feet long, of galvanized pipe. (Note that Nos. 502–514 were on asphalt-coated pipes.) All pipes were of 16-gauge metal. Tests were made both with and against the laps. Loss of head was determined by differential gauges attached to piezometer rings with multiple-hole connections to the water prism. The gauges were water-and-air columns for low flows and mercury columns for higher velocities. The quantity of water was determined by weight.

The results of these tests showed capacities for the 4 and 6 inch sizes of galvanized pipe to be but slightly less than Gehring and Schoder found for similar pipe when asphalt-coated (Nos. 502–514). However, for the 8-inch and 10-inch sizes the relative capacities were materially less. The only difference apparent was that between the asphalt coats and the galvanizing coats, the asphalt tending to round over the flat rivet heads and leave them less in evidence than was the case with the galvanized coats.

The capacity computations, based on values of $K_s=0.38$ for flow with the laps and 0.44 for flow against the laps, showed a slight deficiency, while Schoder's tests on asphalt coated pipes showed a slight excess capacity. In the final recommendations, values of $K_s=0.44$ for flow with the laps and $K_s=0.48$ for flow against the laps, are offered as more conservative for field installations.

CORRUGATED PIPE

No. 602.—24-inch corrugated iron pipe—Outfall sewer, El Paso, Tex. (*168, 106*)—Capacity +12.1 per cent for No. 602 and −1 per cent for No. 602a

In August, 1913, the city of El Paso commenced discharging sewage through a temporary outfall of 24-inch corrugated pipe, 3,784 feet long and between a pumping plant and the outlet in the Rio Grande. The line was of standard galvanized iron, the sections lapping one corrugation. Soon after completion of the line F. H. Todd conducted tests for the determination of capacity. Gauges were set at three points on the line, giving the loss of head through a reach of straight pipe 1,038 feet long and through a reach containing a curve, 1,770 feet long. At the outlet end the water was checked by a partition board so that the pipe flowed full throughout its length. Quantity of water was determined by a well-made rectangular weir between the outlet and the river. The piezometer connections were similar to those used by the writer, consisting of small brass tubing sealed at the end but drilled on the side. These tubes were thrust into the water prism at the horizontal diameter of the pipe, through the small-diameter corrugations. This arrangement placed the actual piezometer orifice well out in the water prism, away from the heavy eddies caused by the corrugations. From the piezometers, rubber tubing led to gauge glasses mounted beside paper scales. These glasses were set by levels so that the loss of head was determined by direct readings of the three water columns. The elements of the observations were not given in *(168)* but were furnished in correspondence by Mr. Todd.

The results of these observations can probably be best discussed in terms of the Kutter formula, as all the meager data on this type of surface were developed in the same way. For the straight pipe the average value of n was 0.01985, while for the section including the curve this average n was 0.0212. It is interesting to compare these figures with those found by D. L. Yarnell and his associates in the laboratory at Iowa City, Iowa: "The coefficient of roughness, n, in the Kutter formula, for the corrugated metal pipe ranges from 0.019 for the 12-inch size to 0.023 for the 30-inch size" *(183, p. 10)*. An increase in the value of n with an increase in pipe size is characteristic of all pipes.

APPENDIX 2

No. 227.—Experiment S-135a-c-d—65-inch welded steel pipe with riveted field seams—Mokelumne Aqueduct, East Bay municipal utility district, California—See No. 164 (38)—Capacity, reach 227a, +3 per cent; reach 227b, +7.9 per cent; reach 227c, +8.2 per cent

The new aqueduct, leading from Pardee Reservoir, on the Mokelumne River, to the cities of Oakland and Berkeley, and other towns, comprising the East Bay municipal utility district, was placed in operation during the summer of 1929. A short time later a test for the determination of the initial coefficients of retardation was made in informal cooperation with the engineers of the district. This test was made on four separate reaches of steel pipe line aggregating some 80 miles in length.

The various units of this pipe were fabricated in the shop by bending two plates, each 30-feet long, to the curve of the pipe and electric-welding the two straight seams. (Plate 1, D.) Thus the only girth seams were those made in the field, for the most part by riveting through a bell-and-spigot joint, formerly called a "straight-bump" or through butt straps. That is, the various units are cylinders of equal size.

Mercury U-tube columns were set up at five points between the dam at the reservoir and the pumping plant at Walnut Creek. The pressure head in the pipe line operated on the mercury gauges through standard ¾-inch valves in the pipe connections of five air valves at summits. All of the connections were thus identical and the difference in elevations of the summits of the various water columns equivalent to the observed mercury columns of course gave the loss of elevation as the water flowed from gauge to gauge. The quantity of flow was measured at two Venturi meters, one near the dam and the other at the pumping plant mentioned above. These had just been checked by observations of the mercury test-columns. The metered flow was checked by two observations of the velocity in the last 10 miles of line, timing shots of potassium permanganate and of fluorescein. A correction of about one-half per cent was necessary. The test was made at but one flow, the maximum possible for the stage of water in the reservoir, at much less than full capacity. The booster pump at Walnut Creek was operated to maintain a uniform pressure of but 5 or 6 pounds at the intake pipe. Readings of the various gauges and of the Venturi meters extended over a period of 5 hours, at intervals of 15 minutes. Three of the mercury columns reflected pressure heads of less than 50 feet but two of them required the long leg of the mercury column to be from 19 to 22 feet—believed to be an unprecedented head for a portable mercury gauge. For each gauge a single sight with a Y level carried the elevations from bench marks set in the concrete tops of the valve boxes. These bench marks represented the final level computations for many parallel level lines.

The necessary temperature readings were taken in order to convert the particular mercury column for the particular temperature at that moment, to the equivalent water column at the average particular temperature in the whole aqueduct, 80 miles long, that had a bearing on the pressure balanced by the mercury column.

The writer regards these observations as deserving higher regard than any similar tests previously made.

The coefficients of retardation all indicate a capacity slightly above that when computed by the coefficients recommended in this bulletin. However, this excess is not greater than should be expected since the conditions for this aqueduct were more favorable than would usually be the case. The reaches are long and relatively straight in both plan and profile. The coating was considered among the best obtainable, regardless of cost. As shown in the photograph (Plate 1, D), there were no wrinkles or bumps, so often found after dipping.

Reach 227a, about 22 miles long, of bell-and-spigot pipe, in plate 7/16 and ½ inch thick, extended from the dam, out through the rolling foothills to the floor of the valley. It had more curvature and more undulations of profile than the other portions tested. The retardation coefficients indicated a capacity some 5 per cent less than that of the more distant portions of the line. This phenomenon was commented upon by Mills and a formula was developed to indicate the difference in lines close to reservoirs and those farther away. However, the line had been in operation such a short time that the writer does not believe this accounts for the difference in coefficients. The coefficients for this reach indicate capacity less than 1 per cent better than do the coefficients for the reach just beyond, No. 164, a compound pipe some 25 miles long. Reach 227b lies beyond the compound pipe, on the flat floor of the valley. The straight lengths so far exceed the curvature that it can be considered as a straight pipe. While about 10 miles in length, this reach contains nearly 5 miles of pipe with field girth joints abutting without rivet heads. Some of these joints were butt-welded with acetylene torch but for the most part they were electric welded under butt-strap sleeves. This reach classifies midway between classes 2 and 3. The coefficients for this reach are in close conformity with those found by Randlett on the Portland, Oreg., pipe (No. 226) and by Kempkey for the Vallejo, Calif., pipe (No. 162). Reach 227c some 22 miles long was all single-riveted under butt-straps, of plates either ⅜ or 7/16 inch thick. The thickness of plate does not affect the capacity in this case as the pipe sections were all cylinders of identical size with abutting ends. Here, too, the capacity is more than would be computed for this type of pipe according to the writer's formula, but it is believed that the conditions surrounding this pipe warrant such an excess over that computed for usual conditions. (See pipe No. 164, which is a reach of compound pipes placed between 227a and 227b.)

No. 164.—Experiment S–135b—Compound pipe, 54 to 65 inch, Mokelumne Aqueduct, East Bay municipal utility district, California—See No. 227 (38)—Capacity +3.9 per cent

Just following the reach described as No. 227a the aqueduct at present (1929) consists of some 26 miles of pipe of various sizes, all of ½-inch plate, welded or lock-bar seamed in the longitudinal joints and mostly double-riveted under butt straps at the field girth seams. The sequence and distribution of sizes is given in Table 2. The only part requiring explanation is at the triple river crossings where the flow in a single pipe is divided between two 54-inch pipes, again returning to a single pipe after each crossing. The assumption is made that the flow divides equally; hence that the velocity is but half that if the full flow went through one 54-inch pipe. However, the retardation loss is, of course, running simultaneously in the two pipes so it is computed for half flow in a single pipe. This reach is so long that local losses at the transitions between sizes can be ignored. The overall loss of head between mercury columns was nearly 102 feet. Aside from the dips at the river crossings this reach extends over perfectly flat country and has but little curvature. The coefficients of retardation are slightly better than for No. 227a, preceding. Likewise they are inferior to those for Nos. 227b and 227c following, showing a progressive improvement in capacity factors as distance from the reservoir is attained. (See No. 227 above for description of tests.)

LITERATURE CITED

(1) Anonymous.
1900. New hydraulic data. Engin. Rec. 41: 217.

(2) ——
1914. Condition of the interior of a 48-in. riveted-steel water main after 17 years' service. Engin. News 71: 1254–1256, illus.

(3) ——
1923. Construction of a 9-foot pipe line six miles long. Engin. News-Rec. 91: 1067, illus.

(4) ——
1925. Iron bacteria and water-pipe incrustation. Surveyor and Munic. and County Engin. 68: 241–242.

(5) Alexander, C. W. L.
1905. Resistance offered to the flow of water in pipes by bends and elbows. Inst. Civ. Engin. [England], Minutes Proc. 159: 341–364, illus.

(6) AMERICAN WATERWORKS ASSOCIATION.
1925. WATERWORKS PRACTICE; A MANUAL ISSUED BY THE AMERICAN WATERWORKS ASSOCIATION. 790 p., illus. Baltimore.

(7) ATHERTON, D. H.
1926. FLUID FLOW IN PIPES OF ANNULAR CROSS-SECTION. Mech. Engin. [New York] 48: 1112–1114, illus.

(8) BALCH, L. R.
1913. INVESTIGATION OF HYDRAULIC CURVE RESISTANCE. EXPERIMENTS WITH THREE-INCH PIPE. Wis. Univ. Bul. Engin. Ser. 7 (3), 52 p., illus.

(9) BARNES, A. A.
1916. HYDRAULIC FLOW REVIEWED. 158 p., illus. London and New York.

(10) BAXTER, H. D. G.
1923. VELOCITIES OF FLOW IN DREDGE PIPES. Mil. Engin. 15: 139–140, illus.

(11) BAYLES, G. H.
1919. CONSTRUCTING A 98-KILOMETER WATER CONDUIT IN CHILE. Engin. News-Rec. 83: 593–599, illus.

(12) BAYLIS, J. R.
1926. PREVENTION OF CORROSION AND "RED WATER." Jour. Amer. Waterworks Assoc. 15: 598–633, illus.

(13) BEHRENS, H.
1926. DIE BERECHNUNG DER ROHRLEITUNGEN FÜR KALTWASSERVERSORGUNGEN. Gas und Wasserfach 69: 313–318, illus.

(14) BENZENBERG, G. H.
1897. COATING OF CAST-IRON AND STEEL RIVETED PIPE. Munic. Engin. 13: 280–287, illus.

(15) BERRIDGE, H.
1912. FORMULA AND CONSTANTS FOR FRICTION LOSS IN DREDGE DISCHARGE PIPES. Engin. News 68: 117–118, illus.

(16) BIEGELEISEN, B., and BUKOWSKI, R.
1914. GRUNDLAGEN ZUR BERECHNUNG VON WASSERROHRLEITUNGEN. Gesundheits-Ingén. 37: 477–495, 554–560, 593–601, illus.

(17) BIER, P.
1927. STEEL PENSTOCK DESIGN BY A GRAPHICAL METHOD. Engin. News-Rec. 99: 629–634, illus.

(18) BLASIUS, H.
1913. DAS AEHNLICHKEITSGESETZ BEI REIBUNGSVORGÄNGEN IN FLÜSSIGKEITEN. Mitt. über Forschungarb. Geb. Ingenieurwesens, No. 131. Berlin.

(19) BOWIE, C. P.
1912. PROTECTING PIPE LINES AGAINST ALKALI. Canad. Engin. 22: 689–690.

(20) BRABBÉE, C. W.
1912. REIBUNGSWIDERSTANDE IN WARMWASSERHEIZUNGEN. Mitt. Prüfungsanst. für Heizungs und Luftungseinrichtungen. Pamphlet 5. Berlin.

(21) BRAGG, G. H.
1924. REMOVING AND PREVENTING RUST IN PENSTOCKS. Elect. World 83: 239–240.

(22) BRIGHTMORE, A. W.
1907. LOSS OF PRESSURE IN WATER FLOWING THROUGH STRAIGHT AND CURVED PIPES. Inst. Civ. Engin. [England], Minutes Proc. 169: 315–336, illus.

(23) BRINKHAUS, P.
1926. GLEICHUNGEN ZUR BESTIMMUNG DES DRUCKVERLUSTES IN WASSERREHRLEITUNGEN. Gas und Wasserfach 69: 165–168.

(24) BRUSH, C. B.
1888. SOME FACTS IN RELATION TO FRICTION, WASTE AND LOSS OF WATER IN MAINS. Amer. Soc. Civ. Engin. Trans. 19: [89]–105, illus. [Discussion by J. T. Fanning, p. 110–116, illus.]

(25) BRUSH, W. W.
1926. THE PREVENTION OF CORROSION OF PIPE. Jour. Amer. Waterworks Assoc. 16: 173–180, illus.

(26) CALAME, J., and GADEN, D.
1926. CALCUL DE LA PERTE DE CHARGE DANS LES GALERIES SOUS PRESSION. Bul. Tech. de la Suisse Romande 52: 74–80.

(27) Chezy, A.
1775. MÉMOIRE SUR LA VITESSE DE L'EAU CONDUITE DANS UNE RIGGOLE DONNÉE. Ref. No. 86. [Unpublished manuscript.]
(28) Christen, T.
1903. DAS GESETZ DER TRANSLATION DES WASSERS IN REGELMASSIGEN KANÄLEN, FLÜSSEN UND RÖHREN. 169, illus. Leipzig.
(29) Coffin, F. C.
1896. THE FRICTION IN SEVERAL PUMPING MAINS. Jour. New England Waterworks Assoc. 10: 234–243, illus.
(30) Coker, E. G., and Clement, S. B.
1903. AN EXPERIMENTAL DETERMINATION OF THE VARIATION WITH TEMPERATURE OF THE CRITICAL VELOCITY OF FLOW OF WATER IN PIPES. Roy. Soc. [London] Phil. Trans., Ser. A 201: 45–61, illus.
(31) Corp, C. I., and Ruble, R. O.
1922. EXPERIMENTS ON LOSS OF HEAD IN VALVES AND PIPES OF ONE-HALF TO TWELVE INCHES DIAMETER. Wis. Univ. Bul. Engin. Ser. 9 (1), 143 p., illus.
(32) Creager, W. P., and Justin, J. D.
1927. HYDRO-ELECTRIC HANDBOOK. 897 p., illus. New York and London.
(33) Crimp, W. S., and Bruges, C. E.
1895. A NEW FORMULA FOR THE FLOW OF WATER IN SEWERS AND WATER-MAINS. Inst. Civ. Engin. [England], Minutes Proc. 122: 198–201, illus.
(34) Daley, D. H.
1911. LOSS OF HEAD IN SCREW PIPE ELBOWS AND TEES. Cornell Civ. Engin. 20: [107]–113.
(35) Darcy, H.
1857. RECHERCHES EXPÉRIMENTALES RELATIVES AU MOUVEMENT DE L'EAU DANS LES TUYAUX. 2 v., illus. Paris.
(36) Darrach, C. G.
1878. THE FLOW OF WATER IN PIPES UNDER PRESSURE. Amer. Soc. Civ. Engin. Trans. 7: [114]–121.
(37) Davey, T. F.
1926. VELOCITY AT TANGENT OF CURVES OF 36-IN. PIPE. Engin. News-Rec. 97: 905, illus.
(38) Davis, A. P.
1926. FIELD WELDING EXPERIENCE ON 65-IN. MOKELUMNE PIPE. Engin. News-Rec. 97: 912–914, illus.
(39) Davis, G. J.
1911. INVESTIGATION OF HYDRAULIC CURVE RESISTANCE. Experiments with two-inch pipe. Wis. Univ. Bul., Engin. Ser. 6 (4), 60 p., illus.
(40) Defour, H.
1926. JAUGEAGES AU MOULINET DANS LES CONDUITES FORCÉES. Bul. Tech. de la Suisse Romande 52: 102–108, illus.
(41) Dibble, B.
1915. LEAD AND TAR PAINTS ON HYDRAULIC APPARATUS, MINIDOKA POWER PLANT. Reclamation Rec. 6: 425–426, illus.
(42) Dole, R. B.
1906. THE USE OF FLUORESCEIN IN THE STUDY OF UNDERGROUND WATERS. U. S. Geol. Survey Water Supply Paper 160: 73–85.
(43) Duane, J
1893. THE EFFECT OF TUBERCULATION ON THE DELIVERY OF A 48-IN. WATER MAIN. Amer. Soc. Civ. Engin. Trans. 28: [26]–31.
(44) Dubuat-Nancy, L. G.
1816. PRINCIPES D'HYDRAULIQUE ET D'HYDRODYNAMIQUE. 3 v., illus. Paris.
(45) Du Moulin, W. L.
1915. THE PUMPING PLANT OF THE MORENCI WATER COMPANY. Amer. Soc. Civ. Engin. Trans. 79: [1268]–1318, illus.
(46) Dupuit, J. E. J.
1863. TRAITÉ THÉORETIQUE ET PRATIQUE DE LA CONDUITE ET DE LA DISTRIBUTION DES EAUX. Ed. 2, 2 v., illus. Paris.
(47) Durand, W. F.
1921. HYDRAULICS OF PIPE LINES. 271 p., illus. New York.

(48) ECHOLS, W. H.
1910. ON THE FLOW OF WATER IN PIPES, CONDUITS, AND OPEN CHANNELS AND THE LOSSES OF ENERGY DUE TO ITS MOTION. Va. Univ. Phil. Soc. Sci. Ser. Bul. 1, 186 p.

(49) ——
1912. FLOW OF WATER IN ARTIFICIAL CHANNELS: CLEAN PIPES. Va. Univ. Phil. Soc. Sci. Ser. Bul. 1 (10), 243 p.

(50) ELLIS, D.
1925. IRON BACTERIA AND PIPE INCRUSTATION. Engineering [London] 120: 371–372.

(51) ESSEX, E. H.
1927. THE ESTIMATION OF VELOCITIES IN WATER CONDUITS. Engineering [London] 124: 105, illus.

(52) EUSTICE, J.
1910. FLOW OF WATER IN CURVED PIPES. Engineer [London] 110: 23–24, illus.

(53) FANNING, J. T.
1893. PRACTICAL TREATISE ON HYDRAULIC AND WATER-SUPPLY ENGINEERING. Ed. 11, 644 p., illus. New York.

(54) ——
1906. A PRACTICAL TREATISE ON HYDRAULIC AND WATER-SUPPLY ENGINEERING; RELATING TO THE HYDROLOGY, HYDRODYNAMICS, AND PRACTICAL CONSTRUCTION OF WATERWORKS, IN NORTH AMERICA. Ed. 16, rev., enl., 644 p., illus. New York.

(55) FIDLER, T. C.
1898–1902. CALCULATIONS IN HYDRAULIC ENGINEERING; A PRACTICAL TEXT-BOOK FOR THE USE OF STUDENTS, DRAUGHTSMEN, AND ENGINEERS . . . 2 v., illus. London, New York, [etc.].

(56) FINKLE, F. C.
1905. THE MOST ECONOMIC TYPE OF HYDRAULIC POWER CONDUITS. Engin. Rec. 52: 263–264.

(57) FITZGERALD, D.
1896. FLOW OF WATER IN 48-IN. PIPES. Amer. Soc. Civ. Engin. Trans. 35: [241]–275, illus.

(58) FLAMANT, A. A.
1892. ÉTUDE SUR LES FORMULES DE L'ÉCOULEMENT DE L'EAU DANS LES TUYAUX DE CONDUITE. Ann. Ponts et Chausées (7) 4: 301–345, illus.

(59) FLAMANT, A. A.
1900. HYDRAULIQUE. Ed. 2, considérablement augm., 689 p., illus. Paris.

(60) FLINN, A. D., WESTON, R. S., and BOGERT, C. L.
1927. WATERWORKS HANDBOOK. Ed. 3, 871 p., illus. New York.

(61) FLYNN, P. J.
1892. IRRIGATION CANALS AND OTHER IRRIGATION WORKS, INCLUDING THE FLOW OF WATER IN IRRIGATION CANALS AND OPEN AND CLOSED CHANNELS GENERALLY. 398 p., illus. San Francisco.

(62) FOLWELL, A. P.
1902. NOTES ON LOST HEAD IN WATER SUPPLY SYSTEMS. Engin. News 47: 303–304, illus.

(63) FRANCIS, J. B.
1874. MEMORANDUM AND TABLES EXHIBITING THE RESULTS OF SOME OF DARCY'S EXPERIMENTS ON FLOW OF WATER IN CAST-IRON PIPE. Amer. Soc. Civ. Engin. Trans. 2: 45–51.

(64) FRIEND, C. A.
1894. AN EXPERIMENT ON THE FRICTION IN A 21-IN. WATERMAIN. Engin. News 33: 223.

(65) FROMM, K.
1923. STRÖMUNGSWIDERSTAND IN RAUHEN ROHREN. Ztschr. Angew. Math. und Mech. 3: 339–358, illus.

(66) GALE, J. M.
1869. ON SOME RECENT ADDITIONS TO THE LOCH KATRINE WATERWORKS. Inst. Engin. & Shipbuilders of Scotland Trans. 12: 125–144.

(67) GANGUILLET, E., and KUTTER, W. R.
1907. A GENERAL FORMULA FOR THE UNIFORM FLOW OF WATER IN RIVERS AND OTHER CHANNELS. Translated from German . . . by R. Hering and J. C. Trautwine, jr., 240 p., illus. New York.

(68) GIESECKE, F. E.
1917. THE FRICTION OF WATER IN PIPE AND FITTINGS. Tex. Univ. Bul. 1759, 42 p., illus.
(69) ——
1926. FRICTION OF WATER IN ELBOWS. Jour. Amer. Soc. Heating and Ventilating Engin. 32: 461–472, illus.
(70) GOLDMARK, H.
1897. THE POWER PLANT, PIPE LINE, AND DAM OF THE PIONEER ELECTRIC POWER COMPANY AT OGDEN, UTAH. Amer. Soc. Civ. Engin. Trans. 38: [246]–305, illus.
(71) GOULD, E. S.
1896. THE MODIFIED DARCY FORMULA AND THE NEW WESTMINSTER STEEL MAIN GAGINGS. Engin. News 36: 90–91.
(72) GREVE, F. W., JR.
1918. FLOW OF WATER THROUGH ONE AND ONE-HALF INCH PIPE AND VALVES. Purdue Univ. Engin. Expt. Sta. Bul. 1, 21 p., illus.
(73) —— and MARTIN, R. R.
1921. FLOW OF WATER THROUGH SPIRAL RIVETED STEEL PIPE. Purdue Univ. Engin. Expt. Sta. Bul. 8, 32 p., illus.
(74) HAGEN, G.
1855. ÜBER DEN EINFLUSS DUR TEMPERATUR AUF DIE BEWEGUNG DES WASSERS IN ROHREN. K. Akad. Wiss. Berlin p. 17–98, illus.
(75) HALE, R. A.
1896. TUBERCLES IN IRON AND STEEL PENSTOCKS. Jour. New England Waterworks Assoc. 10: 244, illus.
(76) HASTINGS, L. M.
1894. SOME MEASUREMENTS OF THE FLOW OF WATER IN A 36-INCH AND 30-INCH COMPOUND PIPE. Jour. New England Waterworks Assoc. 8: 189–194, illus.
(77) ——
1899. THE USE OF STEEL FOR WATER MAINS. Jour. New England Waterworks Assoc. 13: 314–324.
(78) HAWGOOD, H.
1915. HUACAL DAM, SONORA, MEXICO. Amer. Soc. Civ. Engin. Trans. 78: [564]–601.
(79) HAWKS, A. McL.
1896. FLOWAGE TEST OF A 14-IN. RIVETED STEEL MAIN AT NEW WESTMINSTER, B. C. Engin. News 36: 76–77, 90.
(80) HAWKS, A. McL.
1899. SECOND TEST OF A 14-IN. RIVETED STEEL MAIN AT NEW WESTMINSTER, B. C. Engin. News 42: 4–5, illus. Discussion p. 26, 89, 107.
(81) ——
1899. FRICTION COEFFICIENT FOR RIVETED STEEL PIPE. Amer. Soc. Civ. Engin. Trans. 42: [155]–159.
(82) HAZEN, A.
1911. STEEL AND CAST-IRON PIPE. (Abstract) Engin. Rec. 63: 681–682.
(83) HENNY, D. C.
1926. FACTORS GOVERNING AIR INLET VALVES ON PIPE LINES. Engin. News-Rec. 97: 294–297, illus.
(84) HERING, R.
1892. [ON THE HYDRAULICS OF THE HEMLOCK LAKES CONDUIT OF THE ROCHESTER, N. Y., WATERWORKS.] Discussion. Amer. Soc. Civ. Engin. Trans. 26: 40–44.
(85) ——
1896. THE CAPACITY OF THE EAST JERSEY WATER COMPANY'S CONDUIT. Engin. News 35: 59–60.
(86) HERSCHEL, C.
1897. 115 EXPERIMENTS ON THE CARRYING CAPACITY OF LARGE, RIVETED, METAL CONDUITS . . . 130 p., illus. New York.
(87) ——
1897. ON THE ORIGIN OF THE CHEZY FORMULA. Jour. Assoc. Civ. Engin. Soc. Phila. 18: 263.
(88) ——
1913. THE TWO BOOKS ON THE WATER SUPPLY OF THE CITY OF ROME OF SEXTUS JULIUS FRONTINUS, WATER COMMISSIONER OF THE CITY OF ROME, A. D. 97, A PHOTOGRAPHIC REPRODUCTION OF THE SOLE ORIGINAL LATIN MANUSCRIPT . . . Ed. 2, 296 p., illus. Boston.

(89) Heywood, F.
1925. The flow of water in pipes and channels. Inst. Civ. Engin. [England], Minutes Proc. 219: 174–191, illus.

(90) ——
1925. The flow of water in pipes and channels. Engineering [London] 119: 41–42.

(91) Hill, N. S., jr.
1907. Tuberculation and the flow of water in pipes. Amer. Waterworks Assoc., 27th Ann. Convention Proc. 1907: [303]–374, illus.

(92) Hopf, L.
1923. Die Messung der hydraulischen Rauhigkeit. Ztschr. Angew. Math. und Mech. 3: 329–339, illus.

(93) Houk, I. E.
1921. Velocity tests in hydraulic dredge pipe . . . Engin. News-Rec. 86: 18–19, illus.

(94) Hubbell, C. H.
1901. A study of hydraulics. Jour Engin. Soc. 26: 155–192, illus.

(95) Hubie, M.
1927. Note sur diverses formules et l'écoulement de l'eau dans les conduits et les aqueducs de grandes dimensions. Ann. Ponts et Chaussées. (I–1927.)

(96) Hughes, H. J. and Safford, A. T.
1916. A treatise on hydraulics. 505 p., illus. New York.

(97) Jones, J. O.
1925. New tables for computing loss of head in pipes. Engin. News-Rec. 94: 240–242, illus.

(98) King, H. W.
1929. Handbook of hydraulics. Ed. 2, 523 p., illus. New York and London.

(99) Krey, H.
1927. Die Quer—Geschwindigkeitskurve bei turbulenter Strömung. Ztschr. Angew. Math. und Mech. 7: 107–113, illus.

(100) Kuichling, E.
1896. Comparison of original computations and actual gauging of the new steel conduit of Rochester, N. Y. waterworks. (Extracts from Ann. Rpt.) Engin. Rec. 33: 383–385.

(101) Kuichling, E.
1897. Abstract from report to examining board, Rochester, N. Y. Jour. New England Waterworks Assoc. 13: 138–146.

(102) Kunigk, W. A.
1926. Plotting a life line for Tacoma's water supply conduit. Engin. News-Rec. 96: 562–563, illus.

(103) Kutter, W.
1877. Die neuen Formeln über die Bewegung des Wassers. 95 p., illus. Vienna.

(104) Lang, H.
1923. Das Gesetz für den Durchflusswiderstand in Rohrleitungen kreisformigen Querschnittes. Bautechnik. (Apr. 27, 1923): 174–175.

(105) Ledoux, J. W.
1921. Some observations concerning water-supply mains. Jour. New England Waterworks Assoc. 35: 354–361, illus.

(106) ——
1925. Relative corrosibility of water supply. Engin. and Contract., Waterworks 63: 101–106, illus.

(107) ——
1926. Economic design of pipe lines. Engin. News-Rec. 96: 250.

(108) Leslie, J.
1854–55. Observations on the flow of water through pipes, conduits and orifices. Inst. Civ. Engin. [England], Minutes Proc. 14: 273–287, illus.

(109) Lindqcist, E.
1926. Formler för berakning an medalhastigheten i öppna ledningar. 513-580, illus. Stockholm.

(110) Lippincott, J. B.
1926. Life of steel pipe in certain soils dependent on protective coating. Western Construction News 1 (May 25): 40–43, illus.

(111) Longley, F. F.
1925. the coolgardie pipe line in australia, and measures taken to correct corrosion. Jour. New England Waterworks Assoc. 39: 421–444, illus.
(112) Los Angeles. Board of Public Service Commissioners.
1916. complete report on construction of the los angeles aqueduct. 319 p., illus. Los Angeles.
(113) McAuliffe, P. J.
1920. pipe-line velocities necessary to transport gravel. Engin. News-Rec. 85: 988–990, illus.
(114) Mallett, F. S.
1921. the flow of water in pipes and pressure tunnels. Inst. Civ. Engin. Proc. 208: 9–19, illus.
(115) Maltby, F. B.
1904. hydraulic dredging on the mississippi river. Amer. Soc. Civ. Engin. Trans. 54C: [391]–478, illus.
(116) Marx, C. D., Wing, C. B., and Hoskins, L. M.
1898. experiments on the flow of water in the six-foot steel and wood pipe line of the pioneer electric power company at ogden, utah. Amer. Soc. Civ. Engin. Trans. 40: 471–527, illus.
(117) ——
1900. experiments on the flow of water in the six-foot steel and wood pipe line of the pioneer electric power company at ogden, utah. Amer. Soc. Civ. Engin. Trans. 44: [34]–54, illus. [Discussion by E. Kuichling, p. 55–59; by D. Fitzgerald p. 86–88.]
(118) Marzolo, F.
1925. considerazioni sul calcolo pratico delle condotte d'acqua. Ingegneria 4: 44–49, illus.
(119) Maury, D. H.
1924. large supply mains. Jour. Amer. Waterworks Assoc. 12: 1–60, illus.
(120) Metcalf, L.
1914. the protection of riveted steel pipe. Engin. and Contract. 42: 598–599.
(121) Mills, H. F.
1879. experiments upon piezometers used in hydraulic investigations. Amer. Acad. Arts and Sci. Proc. 14 (W. S. 1879): 26–53.
(122) ——
[1923]. the flow of water in pipes. (with historical and personal note by j. r. freeman and introductory outline by k. r. kennison.) 236 p., illus. [Providence, R. I.]
(123) Moir, D.
1926. applying cement lining by centrifugal means to cast iron and steel pipes. Jour. Amer. Waterworks Assoc. 15: 685–689, illus.
(124) Monroe, R. A.
1923. test of friction losses made on large penstocks. Engin. News-Rec. 91: 598–599, illus.
(125) Moritz, E. A.
1913. flow of water in pipes. Engin. Rec. 68: 667–670, illus.
(126) Mouret, G.
1921. antoine chezy. histoire d'une formule d'hydralique. Ann. Ponts et Chaussées (11) 61: 165–268.
(127) National Electric Light Association.
1923. penstock design report of hydraulic power committee. Natl. Elect. Light Assoc. 80: 383–432, illus.
(128) National Tube Company.
1924. national pipe standards. 656 p., illus. Pittsburgh.
(129) Nicholas, A. J.
1927. Calculation of fluid friction in pipes. Power 66(2): 60, illus.
(130) O'Brien, P. V.
1916. engineer reports on corrosion of 350-mile australian steel pipe line 12 years old. (Coolgardie Line.) Engin. Rec. 74: 501-502.

(131) PALMER, C. S. R.
1905. COOLGARDIE WATER SUPPLY. Inst. Civ. Engin. [England], Minutes Proc. 162: 50–103, illus.

(132) PARKER, P. A. M.
1925. THE CONTROL OF WATER, AS APPLIED TO IRRIGATION, POWER AND TOWN WATER SUPPLY PURPOSES. Ed. 2, 1055 p., illus. London.

(133) PARRY, E.
1920. ON A THEORY OF FLUID FRICTION AND ITS APPLICATION TO HYDRAULICS. Eng. Elec. Jour. (Oct. 1920), p. 3–19, illus.

(134) PERRY, L.
1925. NEW TESTS OF LOSS OF HEAD IN 2-IN. BLACK WROUGHT-IRON PIPE. Engin. News-Rec. 94: 272–273, illus.

(135) POISEUILLE.
1842. RECHERCHES EXPÉRIMENTALES SUR LE MOUVEMENT DES LIQUIDES DANS LES TUBES DE TRÈS-PETITS DIAMETRES. Compt.-Rend. Acad. Sci. [Paris]15: 1167–1186.

(136) PRACY, G. W.
1926. EXPERIENCE WITH RIVETED PIPE IN SAN FRANCISCO WATERWORKS. Engin. News-Rec. 96: 844–846, illus.

(137) PRINCE, G. T.
1916. TABULATED DATA, WITH EXPLANATORY NOTES, RELATING TO FLOW OF WATER UNDER PRESSURE THROUGH CLEAN CLOSED PIPES. 149 p., illus. New York.

(138) PROEBSTEL, D. W.
1925. MEASURING WATER FLOW IN CONDUITS. MULTIPLE PITOT AND PIEZOMETER TUBES USED—APPLIES OLD PRINCIPLE COMBINING SIMPLICITY, SPEED, AND ACCURACY—PHOTOGRAPHIC READINGS. Elect. World 85: 711–712, illus.

(139) PROPER, J. W. F. O.
1926. NOG EENS: VERTOONEN ALLE HYDRAULICA FORMULES EENE TEMPERATUR FOUT. Waterstaats-Ingenieur 14: 102–112, illus.

(140) RAFTER, G. W.
1892. ON THE HYDRAULICS OF THE HEMLOCK LAKE CONDUIT OF THE ROCHESTER, N. Y., WATERWORKS. Amer. Soc. Civ. Engin. Trans. 26: [13]–22, illus.

(141) REEVE, S. A.
1911. THE FLOW OF WATER IN CYLINDRICAL CONDUITS. Amer. Soc. Civ. Engin. Trans. 74: [206]–215, illus.

(142) REYNOLDS, O.
1883. AN EXPERIMENTAL INVESTIGATION OF THE CIRCUMSTANCES WHICH DETERMINE WHETHER THE MOTION OF WATER SHALL BE DIRECT OR SINUOUS AND OF THE LAW OF RESISTANCE IN PARALLEL CHANNELS. Roy. Soc. [London], Phil. Trans. 174, Pt. 3: 935–982, illus.

(143) REYNOLDS, W. L.
1916. ROUGHNESS OF STEEL PIPE AFTER 22 YEARS' SERVICE. Engin. News 76: 896–897.

(144) ROCHESTER WATERWORKS BOARD.
1920. ROCHESTER, N. Y. Annual reports 1917 to 1920, inclusive. 337 p., illus.

(145) RUSSEL, S. B.
1894. THE FLOW OF WATER THROUGH NEW CAST-IRON PIPE. Engin. Rec. 29: 312.

(146) SAPH, A. V., and SCHODER, E. W.
1902. EXPERIMENTS AT DETROIT, MICH., ON THE EFFECT OF CURVATURE UPON THE FLOW OF WATER IN PIPES. (Discussion) Amer. Soc. Civ. Engin. Trans. 47: 295–323, illus.

(147) ———
1903. AN EXPERIMENTAL STUDY OF THE RESISTANCE TO THE FLOW OF WATER IN PIPES. Amer. Soc. Civ. Engin. Trans. 51: 253–312, illus. [Discussion (by letter) by G. H. Fenkell p. 323–325; by E. C. Thrupp, p. 315–316].

(148) SCHAPHORST, W. F.
1925. HOW TO CALCULATE PIPE CAPACITIES. Modern Min. 2: 28–29, illus.

(149) SCHODER, E. W.
1909. CURVE RESISTANCE IN WATER PIPES. Amer. Soc. Civ. Engin. Trans. 62: [67]–96, illus. [Discussion by C. J. Davis, p. 97–112, illus.]

(150) ———
1910. FRICTION HEAD HYDRAULICS AND PIPE FLOW DIAGRAMS. Cornell Civ. Engin. 18: [288]–297, illus.
(151) ——— and GEHRING, H. A.
1908. THE FLOW OF WATER IN SPIRAL RIVETED AND OTHER PIPES. Engin. Rec. 58: 240–241, illus.
(152) SCOBEY, F. C.
1916. THE FLOW OF WATER IN WOOD-STAVE PIPE. U. S. Dept. Agr. Bul. 376, 96 p., illus. (Revised 1926.)
(153) ———
1920. THE FLOW OF WATER IN CONCRETE PIPE. U. S. Dept. Agr. Bul. 852, 100 p., illus. Discussion by ALLEN, K., BENT, A. S., FINKLE, F. C., HAZEN, A., LIPPINCOTT, J. B., and NEWELL, H. D.
(154) SCOTT, W. A.
1926. STEEL-WELDED PIPE FOR WATER SUPPLY. Engin. World 29: 221–222, illus.
(155) SHARP, J.
1914. SOME CONSIDERATIONS REGARDING CAST IRON & STEEL PIPES. 142 p., illus. London, New York, [etc.]
(156) SHERMAN, C. W.
1900. EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH. (Discussion.) Amer. Soc. Civ. Engin. Trans. 44: 84–86.
(157) SIMPSON, J.
1854–55. TESTS ON 12-INCH AND 19-INCH CAST-IRON PIPES. Inst. Civ. Engin. [England], Minutes Proc. 14: 316–317.
(158) SMITH, H., JR.
1883. THE FLOW OF WATER THROUGH PIPES. Amer. Soc. Civ. Engin. Trans. 12: [119]–125, illus.
(159) ———
1886. HYDRAULICS, THE FLOW OF WATER THROUGH ORIFICES, OVER WEIRS, AND THROUGH OPEN CONDUITS AND PIPES. 362 p., illus. New York and London.
(160) SMITH, I. W.
1896. FLOW OF WATER IN WROUGHT AND CAST IRON PIPES FROM 28 TO 42 INS. IN DIAMETER. Amer. Soc. Civ. Engin. Trans. 36: [197]–211, illus.
(161) SPELLER, F. N.
1921. CONTROL OF CORROSION IN IRON AND STEEL PIPE. Mech. Engin. 43: 661–662, 697, illus.
(162) ———
1925. THE CORROSION PROBLEM IN STEEL AND IRON PIPE. Fire and Water Engin. 78: 349–379, illus.
(163) STRICKLER, A.
1923. BEITRÄGE ZUR FRAGE DER GESCHWINDIGKEITSFORMEL UND DER RAUHIGKEITSZAHLEN FÜR STRÖME, KANÄLE, UND GESCHLOSSENE LEITUNGEN. Mitteilungen des Amtes für Wasserwirtschaft No. 16.
(164) SULLIVAN, M. E.
1900. NEW HYDRAULICS. 301 p. Denver, Colo.
(165) TALBOT, B.
1926. LININGS AND COVERING FOR WATER PIPES. Jour. New England Waterworks Assoc. 40: 446–456.
(166) TAYLOR, H. E.
1926. MARIN COUNTY JOB IS "CROOKEDEST PIPE" LINE; HAS MANY UNUSUAL CONSTRUCTION FEATURES. Mod. Irrig. 2 (9): 30, 32–33, illus.
(167) THRUPP, E. C.
1908. FLOWING-WATER PROBLEMS. Inst. Civ. Engin. [England], Minutes Proc. 171: 346–359.
(168) TODD, F. H.
1913. AN OUTFALL SEWER OF CORRUGATED IRON PIPE AND MEASUREMENTS OF ITS INTERNAL FRICTION. Engin. News, 69: 774–775, illus.
(169) TRAUTWINE, J. C., JR.
1904. RESISTANCE DUE TO BENDS IN PIPES. APPLICATIONS OF EXISTING FORMULAS TO A GIVEN CASE. Amer. Waterworks Assoc. Pro[c.] 24: 328–334, illus.

(170) Tubbs, J. N.
1877. ANNUAL REPORT OF EXECUTIVE BOARD ROCHESTER, N. Y., WATER-WORKS. p. 17–69. Rochester, N. Y.

(171) Tutton, C. H.
1899. FLOW OF WATER IN PIPES. Jour. Engin. Soc. Assoc. 23: 151–165, illus.

(172) Unwin, W. C.
1880. ON THE FRICTION OF WATER AGAINST SOLID SURFACES OF DIFFERENT DEGREES OF ROUGHNESS. (Abstract) Roy. Soc. [London] Proc. 31: 54–58, illus.

(173) ——
1907. A TREATISE ON HYDRAULICS. 339 p. illus. London.

(174) Walker, W. H.
1912. THE RELATIVE CORROSION OF IRON AND STEEL PIPE AS FOUND IN SERVICE. (Abstract) Jour. New England Waterworks Assoc. 26: 7–39, illus.

(175) Walter, F. J.
1926. AN EASY METHOD TO DETERMINE FRICTION LOSSES IN WATER PIPE. Railway Engin. Maintenance 22: 181–182, illus.

(176) Wegmann, E., and Aeryns, A. N.
1925. NEW FORMULA FOR FLOW OF WATER IN CLEAN CAST-IRON PIPE. Engin. News-Rec. 95: 100–102, illus.

(177) Weller, W. K.
1927. PREVENTION OF CORROSION IN STEEL PIPE LINE. Water Works 66: 342–343, illus.

(178) Weston, E. B.
1890. THE RESULTS OF INVESTIGATIONS RELATIVE TO FORMULAS FOR THE FLOW OF WATER IN PIPES. Amer. Soc. Civ. Engin. Trans. 22: [1]–65, illus.

(179) Williams, G. S.
1899. THE FOUNDATION OF OUR KNOWLEDGE OF HYDRAULIC CURVE RESISTANCE. Mich. Tech. (n. s.) 12: 48–59.

(180) Williams, G. S. and Hazen, A.
1920. HYDRAULIC TABLES. Ed. 3. 115 p., illus. New York.

(181) —— Hubbell, C. W., and Fenkell, G. H.
1902. EXPERIMENTS AT DETROIT, MICH., ON THE EFFECT OF CURVATURE UPON THE FLOW OF WATER IN PIPES. Amer. Soc. Civ. Engin. Trans. 47: [1]–196, illus.

(182) Yarnell, D. L.
1922. THE COEFFICIENT OF ROUGHNESS IN CORRUGATED-IRON PIPE. Engin. News-Rec. 88: 352, illus.

(183) —— Nagler, F. A., and Woodward, S. M.
1926. FLOW OF WATER THROUGH CULVERTS. Univ. of Iowa studies. Studies in Engin. Bul. 1, 128 p., illus.

(184) —— and Woodward, S. M.
1920. THE FLOW OF WATER IN DRAIN TILE. U. S. Dept. Agr. Bul. 854, 50 p., illus.

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