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FACTORS INFLUENCING FLOW IN LARGE CONDUITS

Report of the Task Force on Flow in Large Conduits
of the Committee on Hydraulic Structures

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INTRODUCTION

In 1958, a Task Force on Flow in Large Conduits of the Committee on Hydraulics Structures was authorized with the following objectives: To assemble and evaluate information on hydraulic characteristics, air entrainment, friction and other hydraulic losses in large pipes, tunnels, and conduits.

After a review of early suggestions, these objectives were developed as follows:

1. Friction losses in large lined, partly lined and unlined conduits flowing full;
2. other hydraulic losses associated with special elements in large conduits flowing full;
3. the effect of deposits and organic growths on head losses in large conduits flowing full; and
4. the effect of air entrainment on head losses in large conduits flowing full.

The work gave rise to observations on desirable research. This is dealt with in the section on "Areas Of Research."

Reflection on the development of theory and practice in establishing basic knowledge on the flow of fluids discloses the fundamentally different approaches of the scientist and the engineer. The scientist seeks to reveal knowledge, new knowledge, regardless of its end use. The engineer, conversely, is concerned with the application of that knowledge for the use of the

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human community, in the most practical manner yielding reliable results. The Task Force has approached its work from the viewpoint of the engineer.

FRICTION LOSSES

Background.—Attempts to establish the relationship between the friction losses and the physical characteristics of a conduit date back to 1775 when Chezy proposed a formula written as

$$h_f = \frac{1}{C^2}\frac{L}{R} V^2 \qquad (1)$$

in which h_f = the head loss caused by friction, R = the hydraulic radius, L = the length, V = the mean velocity and C = the dimensional roughness coefficient that Chezy, from the limited data then available, believed to be constant.

At approximately the middle of the 19th century, H. P. G. Darcy(1)[1] and J. Weisbach(2) suggested a similar equation based on the assumption that the average drag at the perimeter is proportional to the average energy of flow. This relationship can be written as

$$h_f = f \frac{L}{4R}\frac{V^2}{2g} \qquad (2)$$

in which f = the dimensionless Darcy-Weisbach friction factor.

Eqs. 1 and 2 were the forerunners of all subsequent endeavours to evaluate friction losses.

Exponential Formulas.—As experimental data were accumulated, it became apparent that both C and f varied not only with the boundary roughness but also with V and R. This led to the development of empirical expressions, such as Kutter's, which sought to relate C in terms of these factors. Manning's simplification of the complicated Kutter expression proved to be popular. He evaluated C as

$$C = \frac{1.486}{n} R^{1/6} \qquad (3)$$

using Kutter's n to represent roughness. Combining Eqs. 1 and 3 leads directly to the now familiar form of the Manning formula

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad (4)$$

in which S = the energy gradient or h_f/L. In metric units, the factor 1.486 drops out and Eq. 4 is sometimes written as the so-called Strickler formula

$$V = k_s R^{2/3} S^{1/2} \qquad (5)$$

in which k_s is simply the reciprocal of n.

The Manning formula was the model for additional equations of the exponential type

[1] Numerals in parentheses refer to papers listed in the Appendix—Bibliography.

$$V = \text{constant } R^x S^y \qquad (6)$$

Among these are the Scobey and the Hazen-Williams formulas. Such equations, though dimensionally inconsistent, are useful within the range of parameters for which they were established. The Manning formula, in particular, has been and still is widely used and there is a formidable body of experience and data underlying the selection of n for any particular surface.

Rational Approach.—Because the Darcy-Weisbach friction factor, by derivation, is dimensionless, experimenters who sought a physical understanding of fluid friction chose Eq. 2 as the framework of their research. L. Prandtl's theory of the boundary layer, proposed in 1904(3), was the basis on which the studies were pursued. Blasius first suggested that f, for smooth pipes, was a function of the Reynolds number, **R**, only. Hopf showed that, in the general

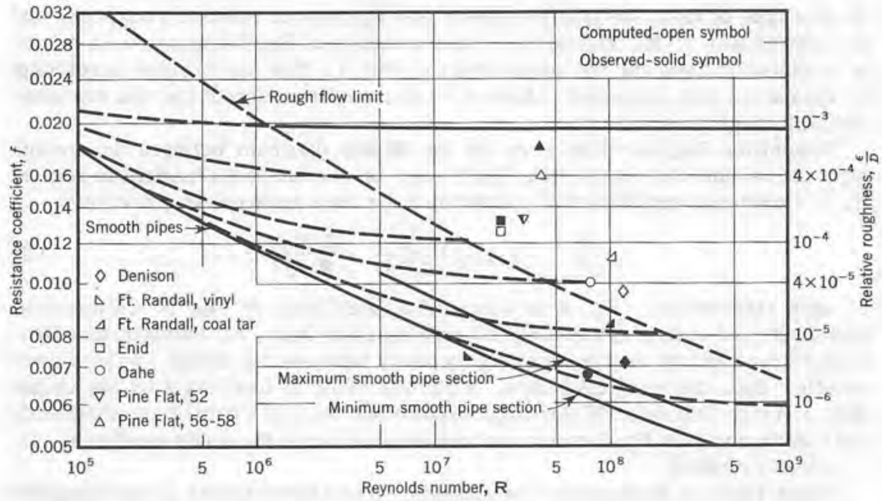


FIG. 1

case, f was related also to relative roughness. Davies and White, in 1929, provided for the general resistance diagram by distinguishing between hydraulically smooth surfaces (where f decreases as **R** increases) and rough surfaces (where f may or may not decrease as **R** increases, depending on the relative roughness).

Effect of Viscosity.—The classical smooth pipe tests of J. Nikuradse(4) demonstrated that the resistance coefficient f varies with Reynolds number, **R**. T. Von Karman(5) and Prandtl developed a smooth pipe equation based on the Nikuradse tests as

$$\frac{1}{\sqrt{f}} = 2 \log \mathbf{R} \sqrt{f} - 0.8 \qquad (7)$$

Eq. 7 is shown as the smooth pipe curve on a general resistance or Moody(6) diagram (see Fig. 1). Numerous field tests have shown that an hydraulically smooth condition can be achieved with both concrete and steel conduits over a wide range of Reynolds numbers. The smooth pipe curve also emphasizes that resistance coefficients determined for smooth pipes at small values of **R** yield too high a value of f for use in the design of conduits with large values of **R**.

Effect of Relative Roughness.—The rough pipe tests of Nikuradse are a valuable basis for determining the effect of relative roughness (k/D). The symbol k is the absolute roughness of the pipe wall and, in random roughness measurements, is taken as twice the root-mean-square of the height of the roughness elements. The von Karman-Prandtl(7) rough pipe equation, based on Nikuradse's tests, is

$$\frac{1}{\sqrt{f}} = 2 \operatorname{Log} \frac{D}{2\,k} + 1.74 \quad \ldots\ldots\ldots\ldots\ldots (8)$$

In this type of flow, the friction factor is a function of relative roughness and is independent of **R**. Therefore, representation of Eq. 8 appears as a series of horizontal lines on the upper right of Fig. 1. The fairly close agreement of measured and computed values of f offer confirmation of the von Karman-Prandtl rough pipe formula.

Transition Region.—The area on the Moody diagram between the smooth pipe curve and the rough flow limit may be considered a transition region. C. F. Colebrook and White(8), based on their own experiments, derived

$$\frac{1}{\sqrt{f}} = -2 \operatorname{Log} \left(\frac{k}{3.7\,D} + \frac{2.51}{\mathbf{R}\sqrt{f}}\right) \ldots\ldots\ldots\ldots\ldots (9)$$

to span this region. Eq. 9 is shown as dashed lines on Fig. 1. Comparison with observed values of f from prototype tests at Oahe, Ft. Randall, and Denison Dams indicate an apparent discrepancy because the actual f values were smaller than the computed ones. A careful study of field tests by the United States Army Engineer Waterways Experiment Station (WES) has resulted in the conclusion that Eq. 9 cannot be verified for large Reynolds numbers.

Lined Conduits.

Field Tests of Resistance Coefficients.—The United States Army Engineer hydraulic prototype test program monitored by the WES has arranged for piezometer installations in 28 large conduits. Pressure gradients are measured when test flows are available. Prior to testing, absolute roughness is measured by negative casts of elastic dental impression material from which positive plaster casts are made. The WES has standard field instructions for making such roughness measurements and cooperates with other organizations wishing to make similar observations. Table 1 reflects some of these observations.[2] The smooth pipe equation is considered applicable over the full range of Reynolds numbers. Apparently, concrete surfaces producing results that approach the smooth pipe curve can be achieved by careful construction with either steel or wood forms. In the case of Oahe, form tolerances were fully specified and form joint offsets were ground to specified limits. The Ft. Randall steel penstocks treated with vinyl appeared to be hydraulically smooth whereas the flood control conduits treated with coal tar produced essentially rough pipe flow. Wood forms constructed by ordinary methods

[2] "Hydraulic Design of Reservoir Outlet Structures," Engineering Manual 1110-2-1602, Govt. Printing Office, Washington, D. C., 1963.

yield an f value of approximately 0.013, in the rough flow region, and conduits roughened with use may show an f value of 0.018. More tests are needed to confirm the field data available. For large and lengthy conduits or tunnels, substantial savings in cost may be achieved if the conduits are designed to provide a smooth surface and consequently a reduced cross-sectional area. The cost of producing a smooth surface should be compared with the saving in cost resulting from reduced cross-sectional area for the same design discharge. When selecting a resistance coefficient for design consideration should be given to the manner in which construction of the conduit surfaces are to be specified. Test results cited above will serve as a guide to the results expected from certain methods of construction. In view of the scarcity of field data and the difficulty of achieving precision in field test methods, conservative methods should be used in design.

A comprehensive and detailed review of prototype observations can be found in the recently revised(15) U.S. Bureau of Reclamation (USBR) Engineering Monograph. Friction factor data for a wide range of sizes and types of conduit are listed.

TABLE 1.—OBSERVED RESISTANCE COEFFICIENTS

Surface Character	Coefficient f	Coefficient n
(a) $\mathbf{R} = 10^8$ (Approximate)		
Concrete, Wood Forms, Joints Ground (Oahe)	0.0068	0.0098
Concrete, Steel Forms (Denison)	0.0072	0.0103
Steel, Coal Tar (Ft. Randall)	0.0085	0.0114
(b) $\mathbf{R} = 10^7$ (Approximate)		
Steel, Vinyl (Ft. Randall)	0.0075	0.0107
Concrete, Wood Forms (Enid)	0.0130	0.0125
Concrete, Wood Forms (Pine Flat)	0.0132	0.0115
Concrete, Wood Forms, Roughened with Use (Pine Flat)	0.0181	0.0135

Erosion and Deterioration.—Suspended material, particularly abrasive sand, can cause erosion of concrete linings which becomes more pronounced as the velocity increases. Concrete inverts frequently become roughened in the course of time as a result of the removal of the matrix from the concrete by the abrasive action of the material rolling along the invert and the "plucking" action of the water on weakened particles of the concrete. Where larger pieces of rock are moved along a tunnel invert and lodge in a suitable depression, potholes gradually form. All of these factors increase the rugosity of the invert. Aggressive waters, particularly from swampy and upland areas, may also result in considerable roughening of the surface of a concrete lining.

Erosion can result from high velocities occasioning cavitation where the conduit or tunnel surfaces are improperly shaped for streamline flow. This is particularly instrumental in increasing roughness where the lining materials are of low resistance.

Rupture of Concrete Lining.—Failures of concrete linings are not uncommon. They are frequently caused by poor construction, especially the placing of the lining over wooden lagging. In time the wood swells, rupturing the lining, or rots, leaving a substantial void. Where linings are ruptured, for any reason, pieces may be torn by the action of the flow.

Partly-lined Conduits.—Colebrook(12) treats tunnels partly-lined longitudinally on the basis that the effective friction factor, f, is a direct weighted average of the component factors, depending simply on the relative proportion of each type of roughness. When a conduit consists of two or more differently-lined bores, each part can be handled separately and no problem occurs.

In the case, however, of partial lining circumferentially, it has been suggested that the linear assumption may not be valid because of the inter-play between the two flow regimes. A relationship that has been used to weight the composite friction factor slightly in the direction of greater roughness is

$$n_{actual} = n_r \left[\frac{p_r + p_s \left(\frac{n_s}{n_r} \right)^{3/2}}{p_r + p_s} \right]^{2/3} \qquad \text{.......... (10)}$$

in which n = the Manning coefficient, p = the perimeter, and the subscripts r and s = the rough and smooth sections, respectively.

Tests conducted at the Imperial College(13) in Great Britain tended to show that, at low Reynolds numbers and low absolute roughness, the composite friction factor was nearer the smooth friction factor than the arithmetic proportions would indicate. At high values of **R** and k, the converse appeared to be true.

Generally, the subject is still open and more data are required to resolve the matter. In view of the common practice of paving the invert of unlined rock tunnels, the question is not academic and seems to have been given less attention than is needed.

Unlined Conduits.—Recent investigations by L. Rahm(9)(10) on prototype unlined tunnels have shown that a relationship exists between the variation in the cross-sectional area and the Darcy-Weisbach coefficient, f. Colebrook(12) has also analyzed Rahm's data and presented it in summary form.

Rahm obtains cross-sectional areas taken approximately every 50 ft along the tunnel and, after excluding the upper and lower 1% for practical reasons, plots these data in an ordinary normal-distribution logarithmic diagram. A straight line is drawn approximating the curve obtained, passing through the point that corresponds to the mean cross-sectional area of the tunnel and at the frequency of 50%. The slope of the line represents the variation in cross-sectional area of the tunnel, and can be expressed in terms of percentage inclination, δ, by

$$\delta = \frac{A_{99} - A_1}{A_1} \; 100 \qquad \text{............. (11)}$$

in which A_{99} = the cross-sectional area corresponding to a frequency of 99% and A_1 = the area corresponding to a frequency of 1%.

TABLE 2.—UNLINED TUNNEL FRICTION COEFFICIENTS

No.	Tunnel Name[a]	Location	Type of Rock	Design Area, in square feet	Average Driven Area, in square feet	Percentage Overbreak	n	f	Variation in Cross-Sectional Area, δ
1	Cresta	California	Granite	578	656	13.5	0.035	0.075	27
2	West Point	California	Granite	180	222	23.5	0.033	0.080	NA
3	Bear River	California	Granite	82	93	14.0	0.028	0.066	NA
4	Balch	California	Granite	144	169	17.5	0.032	0.079	NA
5	Haas	California	Granite	151	184	21.9	0.030	0.068	26
6	Cherry	California	Granite	133	150	12.5	0.034	0.090	NA
7	Jaybird	California	Granite	177	195	10	0.032	0.077	NA
8	Apalachia	Tennessee	Quartzite & slate	380	431	13.5	0.038	0.095	NA
9	Alfta	Sweden	Granite-gneiss	323	364	12.7	0.036	0.086	29
10	Harspranget	Sweden	Granite	2045	2195	7.4	0.032	0.052	24
11	Jarpstrommen	Sweden	Slate	1130	1230	9.8	0.029	0.048	20
12	Krokstrommen	Sweden	Granite	970	1090	12.9	0.029	0.048	17
13	Porjus I	Sweden	Granite-gneiss	538	618	14.8	0.034	0.073	27
14	Porjus II	Sweden	Granite-gneiss	538	662	23.0	0.030	0.055	19
15	Selsforo	Sweden	Slate w/granite	753	865	15.0	0.044	0.114	39
16	Sillre	Sweden	Gneiss	54	71	32.0	0.034	0.102	37
17	Sunnerstaholm	Sweden	Granite-gneiss	323	386	19.7	0.039	0.104	37
18	Tasan	Sweden	Gneiss	183	185	1.2	0.033	0.081	33
19	Eucumbene-Tumut	Australia	36% Granite 64% Metam. Sedim.	400	445	11.2	0.029	0.054	NA
20	Tooma-Tumut	Australia	Granite	125	153	22.4	0.031	0.074	NA
21	Murrumbidgee-Eucumbene	Australia	10% Granite 90% Metam. Sedim.	100	127	27.0	0.036	0.104	NA
22	Big Creek 2	California	Granite	113	130	15.0	0.037[b]	NA	NA
23	Big Creek 3	California	Granite	434	515	18.7	0.035[c]	NA	NA
24	Big Creek 8	California	Granite	357	400	12.0	0.038[b]	NA	NA
25	Ward	California	Granite	211	274	29.9	0.036[b]	NA	NA
26	Big Creek 4	California	Granite	409	462	13.0	0.030[c]	NA	NA
27	Mammoth Pool	California	Granite	336	367	9.2	0.029[c]	NA	NA

[a] Tunnels 1 to 5 are owned by Pacific Gas and Electric Co., 6 by the City of San Francisco, 7 by Sacramento Municipal Utility District, 8 by the TVA, 9 to 18 by Swedish State Power Board, 19 to 21 by the Snowy Mountains Hydro-Electric Authority, and 22 to 27 by the Southern California Edison Co.

[b] Based on gross head loss observations including special losses at adits, bends, lined sections, etc.

[c] Based on net head loss observations on unlined sections only, or else on gross head loss measurements corrected by deducting special losses.

For the tunnels investigated, Rahm found that the frictional coefficient f (based on actual mean area) could be expressed in terms of the percentage inclination as

$$f = 0.00275\,\delta \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

Furthermore, δ has been found to be approximately equal to 200 t_m/R in which t_m = the average excess overbreak, in inches. This indicates that f is proportional to t_m and emphasizes the desirability of accurate drilling. Smoothness of the rock surface can be controlled to a great extent by close placing of contour drill holes, exact parallel drilling, and careful blasting. Heggstad states that in Norway the use of "smooth blasting" has resulted in the reduction of tunnel size of 20% to 30% over rough blasted tunnels. Model tests in Tasmania(16) indicate that, because of the "saw-tooth" effect, an upstream direction of driving yields a substantially lower friction factor.

A confirmation of the Rahm method was obtained recently at the Pirttikoski power plant in Finland(11). The tailrace tunnel is 2,000 m long, the mean area approximately 350 sq m and the rated flow approximately 500 cu m per sec. After blasting was completed, measurement of the cross-sections led to a predicted head loss of 0.72 m at rated flow. The actual head loss was subsequently found to be 0.75 m. Similar confirmation was obtained at the Mammoth Pool power tunnel in California.

The tunnels listed in Table 2 represent as wide a range in size as would be encountered in practice. The rock is predominantly granite. The range in n is from 0.029 to 0.044 and f from 0.048 to 0.114—a substantial variation. It is evident, then, that the rock properties and the methods of excavation can result in large differences in head loss(17)(18).

LOSSES RESULTING FROM DEPOSITS AND ORGANIC GROWTHS

General Aspects.—Engineers concerned with the design and operation of water supply systems are generally aware of the occurrence and effect of organic growths and other deposits. Engineers dealing with the design and operation of tunnels and conduits for the conveyance of water for hydroelectric power, flood control, and irrigation developments are generally less informed on this subject and are frequently surprised by the additional head losses that develop.

Whereas, in certain cases, an actual decrease in net available flow area is involved, the more usual effect is an increase in rugosity along the surfaces under attack. The following classifications of deposits might be listed:

1. Tuberculation of unprotected steel linings;
2. calcareous deposits;
3. slime deposits;
4. organic growths; and
5. insect infestations.

Tuberculation.—The mechanism of tuberculation has been described by Campbell Brown(31) who emphasized that the resulting incrustations formed by the erosion of iron pipes, valves, and other iron work, where the exposed surfaces are not completely protected, can reach thicknesses of from 1 in. to

1-1/2 in. He has noted that the removal of the incrustations may greatly reduce the life of the pipe as a result of the more rapid corrosion of the remaining iron.

P. Parker(32) has found that the "limpet" formations making up the incrustations appear to originate in all waters, whether acid or neutral, where metal is exposed. He agrees with Brown that large mains are " . . . rarely choked by more than 1-1/2 inches of obstruction all round . . . ."

Calcareous Deposits.—Calcareous deposits are well-known to most hydraulic engineers. Brown(31) has described the deposit as "the translucent and semi-crystallized scale of calcium carbonate, formed by the escape of carbonic acid from water containing bicarbonate of lime."

J. F. Case(33) has observed that the 13-sq ft cross-sectional area of the old 16-mile Roman aqueduct at Athens was diminished by 20% because of calcareous incrustations.

There are one or two power plants in France in which tunnels have experienced material increases in head losses as a result of a coating of tufa that developed on the walls at low flow velocities. Such deposits have usually proven to be soft in nature and occur where water from calcareous regions is used.

Slime Deposits.—Brown has also drawn attention, in his comprehensive treatment, to slime deposits, stating that he had frequently observed that a black slimy lining formed wherever the flowing water was in contact with the sides of pipes, culverts or tunnels. The deposit occurred on protected as well as unprotected pipes, and on stonework, woodwork, brick, and rock surfaces, where the water had never been in contact with pipes nor with ironwork of any kind.

Discussion(37) on Brown's paper showed that, after 13 yr of service, the 42-in. diameter Vyrnwy main between Lake Vyrnwy and the downstream filter beds at Oswestry, near Liverpool, England, was only carrying approximately 75% of the flow of the analogous main downstream from the filters. This deterioration in carrying capacity was caused by the slime deposits.

Flynn, Weston, and Bogert(38) have found that the organisms that comprise organic and other growths in conduits tend to occur on the sides and top as compared with the invert, where the deposition of silt discourages the occurrence of organisms. P. Lamont(39) has examined many pipelines to ascertain the effect of age on their carrying capacity. Lunt(40) has recorded observations on the 20% loss in capacity of a conduit caused by black slime.

Colebrook(12) has tabulated pertinent facts with regard to slime formation in a number of tunnels. A. L. Pollard and H. E. House(41) cited experience with excessive head losses first observed in 1945 on a 4.75-mile-long, 11-ft-diameter pressure conduit. Slime to 5/8-in. thick had increased the losses more than 30% over design.

For the 8.3-mile-long Apalachia tunnel, R. A. Elder(42)(43) reported that all surfaces were found to be coated with a 5/16-in. layer of black mucilaginous material. It appeared that the deposit resulted in a decrease in friction coefficient, except for concrete linings in the downstream section in which it remained constant. Where the deposit had flaked on some of the steel pipes there was an increase in rugosity. Further observations, it is understood, have shown a slight increase in rugosity in the tunnel, because of the deposit.

A few French power plants, such as Monceau-la-Virolle, have experienced substantial increases in head loss because of deposits occasioned by microorganisms. These are removed by scraping from time to time, to maintain the head losses at an acceptable value. Values for the Nikuradse roughness value k at Monceau show this varies from 0.7 mm when the concrete-lined tunnel is clean to 1.7 mm when the algae are prolific.

Slime deposits are probably more frequently encountered in power developments supplied by lined and unlined tunnels than is currently (1965) realized. They can occur even in developments supplied by relatively cold water.

Organic Growths.—In some areas, organic growths of one type or another have occasioned considerable difficulty. These growths range from iron bacteria to fibrous, mossy-appearing accretions.

G. E. Arnold(34) has recorded the difficulty encountered with iron bacteria in the Coast Range tunnels of San Francisco's Hetch Hetchy water supply system. P. M. Lefever(35) has reported from experience at Conowingo, where material decreases in output were experienced as a result of extensive deposits on the trash racks, unit intakes, butterfly valve wickets, scroll cases, and wheels on most of the units. This growth varied in thickness from 1/8-in. to 1/2-in. and the surface was wavy. The material was gritty, slimy, and fibrous.

Removal of the deposit by cleaning, using high-pressure water and hand scraping, led to an increase in unit output of between 400 kw and 950 kw. Lefever indicated additional increases could have been secured if the trash racks were cleaned and the butterfly wickets and scroll cases scraped. Other deposits have been cited by R. L. Derby(36), notably a 5-ft-diameter conduit, 50 miles long, that suffered a decrease of 11% in its capacity because of growths of freshwater sponges.

Insect Infestations.—Derby(36) has cited experience with a 7,000-ft siphon, 4-ft in diameter. Shortly after the equipment was placed in service it was discovered that head losses were increasing. Examination revealed a growth, described as "like the hair of an Airedale's back," which was found to be the pupal cases of aquatic insects of the genus Limnophora. In the larval state, the insects build their pupal cases on the sides of the conduits by cementing silt and other material together. Maximum growth occurred at the entrance to the siphon.

At the Pit 5 hydroelectric plant, head losses in Tunnel No. 1 doubled within the first year of operation. W. Dreyer(44) reported: "Practically the whole area of the concrete lining in Tunnel No. 1 was found to be covered with the webs or nets which are spun and attached to the tunnel walls by the caddis-fly larvae. These deposits also extended into Tunnel No. 2 for a distance of about 1,000 ft to a point where the pressure (55 ft of head) was apparently too great for the larvae to live in."

G. R. Woodman(45) has reported on caddis-fly infestation at the Kern No. 3 hydroelectric plant. There was trouble in the late spring of each year with infestations of caddis-fly larvae forming in the upper 2,000 ft of the conduit. The formations were removed by hand scraping or streams of high pressure water.

Another interesting treatment of the influence of insect infestations appears in a 1926 report of the Hydraulic Power Committee of the Pacific Coast Electrical Association(46).

Remedial Treatment.—When difficulty is experienced with organic growths and other deposits, remedial treatment of one form or another can usually be applied.

For example, tuberculation of steel conduits can be eliminated by proper cleaning and the application of protective coatings, in the form of paint, coal-tar enamel, bitumastic preparations, or spun concrete. However, in the case of calcareous deposits, no effective and low-cost means of eliminating them from large conduits is known, other than scraping and use of high-pressure water jets.

Where slime deposits occur, the most effective means of treatment appears to be periodic drainage of the conduit and removal of the deposit by high-pressure water jets or scraping.

Where organic growths occasion losses in carrying capacity, these can normally be eliminated by the application of chlorine in either the free or combined form. Other chemicals may be useful in some cases. Cleaning may be necessary if the deposit has been allowed to build.

Insect infestations can only be dealt with effectively, it would appear, by either draining, scraping, and water-jetting the conduit or by applying protective coatings obnoxious to the insects. These methods have shown some success.

OTHER LOSSES

Introduction.—It is rarely possible to provide a straight, uniform circular conduit for conveying water from one point to another. Normally, between source and destination, the flow encounters a variety of deviations from such an ideal passage. These departures from uniformity, whether in the form of partial obstacles, changes in section, branches or bends, impose an additional loss on the flowing water that augments the loss resulting from frictional resistance. Because they generally represent only a small part of the total, such losses are commonly referred to as minor or secondary losses. Nevertheless, they can be an important consideration and their effect should always be investigated.

Generally, minor losses are the result of fully-developed turbulence and, thus, can be expressed in terms of the nominal velocity head of the flow, $V^2/2g$. But, because the velocity distribution of the liquid may be entirely upset as it traverses the loss-producing section, important changes in boundary conditions may also be expected. These will be reflected in a modification of wall resistance over that part of the conduit whose regime is disturbed—a distance of 20 diam or more.

Such considerations are basic to the research hydraulician and considerable work is now (1965) directed toward obtaining a better understanding of the nature of flow in the vicinity of disturbances to the flow regime. Conversely, the practicing engineer is more particularly interested in the net contribution of the minor loss toward the whole and is not especially concerned with the exact mechanism involved. Thus, the results of laboratory and prototype research have customarily been reported in the form of coefficients to be applied to the velocity head at the section in question, with the coefficient including (unless otherwise stated) the subsidiary effects noted above. This approach is more convenient than the alternative practice of converting minor losses to equivalent conduit length and treating them as additional frictional resistance.

In reviewing some of the more authoritative and up-to-date work in this field, no attempt will be made to provide a comprehensive list of all pertinent references. New material is continually being added to the literature, although perhaps not specifically in the conventional form, and many publications and papers dealing with related subjects contain data concerning minor losses. Occasionally, these data are consolidated and summarized in revised editions of standard hydraulics and hydroelectric handbooks such as are listed in the Appendix. Similar studies, as well as original data, are published by such governmental bodies as the United States Corps of Engineers Waterways Experiment Station (WES) and the Hydraulics Research Station in England, and by various university laboratories.

The following sections are intended to indicate what is available to the engineer seeking a rational basis for estimating the probable minor losses for a given conduit configuration. For this purpose, it is convenient to classify these losses into two groups: (1) At or near the intake; and (2) along the conduit proper.

Intake Losses.—Intake losses include entrance loss, trash rack loss, and head gate loss. It should be noted that accurate experimental determination of intake losses is dependent on the conduit being of sufficient length to permit a uniform friction gradient to be established, based on fully developed turbulence. A comprehensive treatment of the hydrodynamic principles involved and an analysis of the velocity and pressure distributions for various shapes of conduit inlets has been given by H. Rouse.(19)

TABLE 3.—REPRESENTATIVE K VALUES

Type of Entrance	Rouse (19)	King (20)	Addison (21)	Creager (22)
Inward projecting	1.0	0.78	0.70	0.56 - 0.93
Sharp-cornered	0.50	0.50	0.50	0.56
Slightly rounded	-	0.23	-	0.23
Bell-mouthed	0.01 - 0.05	0.04	0.04	0.06

Entrance Loss.—In addition to the pressure drop representing the velocity head, a loss is incurred at the entrance to a conduit analogous to that in a short tube. The magnitude of the coefficient, K, in the expression

$$h_L = K \frac{V^2}{2 g} \qquad (13)$$

depends largely on the geometry of the entrance. Representative published values are listed in Table 3.

Trash Rack Loss.—Creager gives the equation

$$K = 1.45 - 0.45 R - R^2 \qquad (14)$$

in which R = the ratio of net to gross area at the rack section. For a typical value of R = 0.65, the resulting K value is 0.74, to be applied to the velocity head for the net area. The Waterways Experiment Station (WES) has pub-

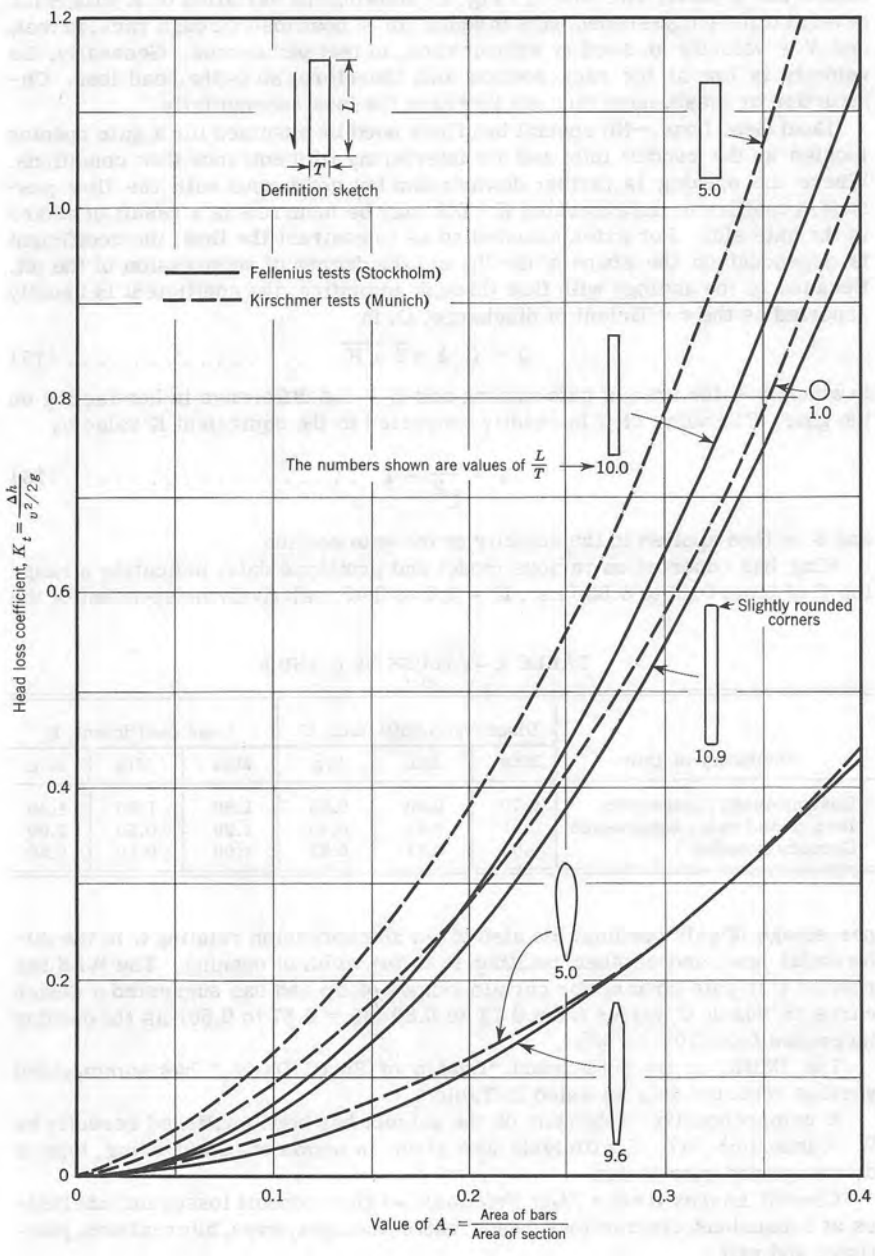

Definition sketch
L
T
5.0
Fellenius tests (Stockholm)
Kirschmer tests (Munich)
1.0
The numbers shown are values of $\frac{L}{T}$ → 10.0
Slightly rounded corners
10.9
5.0
9.6
Head loss coefficient, $K_t = \frac{\Delta h}{v^2/2g}$
Value of $A_r = \frac{\text{Area of bars}}{\text{Area of section}}$
0
0.1
0.2
0.3
0.4
0.2
0.4
0.6
0.8
1.0
1.2

FIG. 2

lished(23) a chart, HEC 010-7, (Fig. 2) showing the variation of K with R for several differently-shaped bars in which Δh = head loss through rack, in feet, and V = velocity at section without rack, in feet per second. Generally, the velocity is low at the rack section and, therefore, so is the head loss. Obstruction by trash, however, can increase the loss substantially.

Head Gate Loss.—No special head loss need be assumed for a gate opening located at the conduit inlet and not interfering with entrance flow conditions. Where the opening is farther downstream but continuous with the flow passage, a coefficient not exceeding K = 0.1 may be incurred as a result of eddies in the gate slot. For gates mounted so as to contract the flow, the coefficient is dependent on the shape of the lip and the degree of suppression of the jet. Because of the analogy with flow through an orifice, the coefficient is usually reported as the coefficient of discharge, C, in

$$Q = C\,A\,\sqrt{2\,g\,H} \qquad (15)$$

in which A = the area of gate opening and H = the difference in head acting on the gate. The value of C is readily converted to the equivalent K value by

$$K = \frac{1}{C^2} - 1 \qquad (16)$$

and K is then applied to the velocity at the gate section.

King has reported on various model and prototype data, indicating a range for C of from 0.62 to 0.83 (i.e., K = 1.6 to 0.45) relatively independent of the

TABLE 4.—VALUES OF C AND K

Geometry of gate	Discharge coefficient, C			Loss coefficient, K		
	Max	Min	Avg	Max	Min	Avg
Unsuppressed contraction	0.70	0.60	0.63	1.80	1.00	1.50
Bottom and sides suppressed	0.81	0.68	0.70	1.20	0.50	1.00
Corners rounded	0.95	0.71	0.82	1.00	0.10	0.50

percentage of gate opening. He also gives an expression relating C to the differential head and another relating it to the width of opening. The WES has related C to gate opening for certain shapes of lip and has suggested a design curve in which C varies from 0.73 to 0.80 (K = 0.87 to 0.56) as the opening increases from 10% to 80%.

The USBR, in its publication "Design of Small Dams," has summarized various reported data as listed in Table 4.

A comprehensive treatment of the subject has been published recently by W. Wunderlich(24). Coefficients are given in terms of gate opening, type of discharge and gate design.

Conduit Losses (Other Than Friction).—Other conduit losses include losses at expansions, contractions, bends, valve passages, wyes, bifurcations, junctions, and exit.

Expansion Loss.—Sudden expansions or enlargements incur a head loss theoretically expressed as

$$h_L = \frac{(V_1 - V_2)^2}{2\,g} \quad \ldots\ldots\ldots\ldots\ldots\ (17)$$

in which V_1 and V_2 = the velocities in the smaller and larger conduits, respectively. King states that the experimentally derived formula by Archer,

$$h_L = 0.01705\,(V_1 - V_2)^{1.919} \quad \ldots\ldots\ldots\ldots\ (18)$$

gives more satisfactory results. He provides a table, based on this relationship, which gives the coefficient K to be applied to the velocity head in the smaller pipe.

Gradual expansions or transitions entail a lesser head loss (because the eddies are suppressed) and are generally calculated as

$$h_L = K\,\frac{(V_1 - V_2)^2}{2\,g} \quad \ldots\ldots\ldots\ldots\ldots\ (19)$$

in which K depends on the angle of flare or divergence. J. G. Brown(25) gives values of K ranging from 0.20 to 1.07 for flare angles of 2° to 90°. King's corresponding values are considerably lower, ranging from 0.03 to 0.67. He also provides a table, based on these coefficients, that converts K to the more usual form (i.e., in terms of the larger velocity head). Creager also expresses K in the latter form, giving

$$K = \left(1 - \frac{a_1}{a_2}\right)^2 \sin\theta \quad \ldots\ldots\ldots\ldots\ldots\ (20)$$

in which a_1 and a_2 = the smaller and larger areas, and θ = one half of the flare angle.

It should be noted that all the above mentioned treatments exclude the friction loss within the enlargement section. When this is considered, Addison states that the optimum taper is 1 to 10, resulting in an approximate value of

$$h_L = 0.14\,\frac{(V_1 - V_2)^2}{2\,g} \quad \ldots\ldots\ldots\ldots\ldots\ (21)$$

Contraction Loss.—Sudden contractions incur a loss coefficient, K, of as much as 0.5 for pronounced differences in area. Rouse, King, and Creager provide data showing the variation of K with the ratio of diameters. Gradual contractions involve a negligible loss.

Bends.—The loss at a bend results from a distortion of the velocity distribution, thereby causing additional shear stresses with the fluid. The bend loss is calculated from

$$h_L = \frac{K_B\,V^2}{2\,g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (22)$$

in which h_L = head loss resulting from bend, K_B = bend loss coefficient, and V = velocity in pipe.

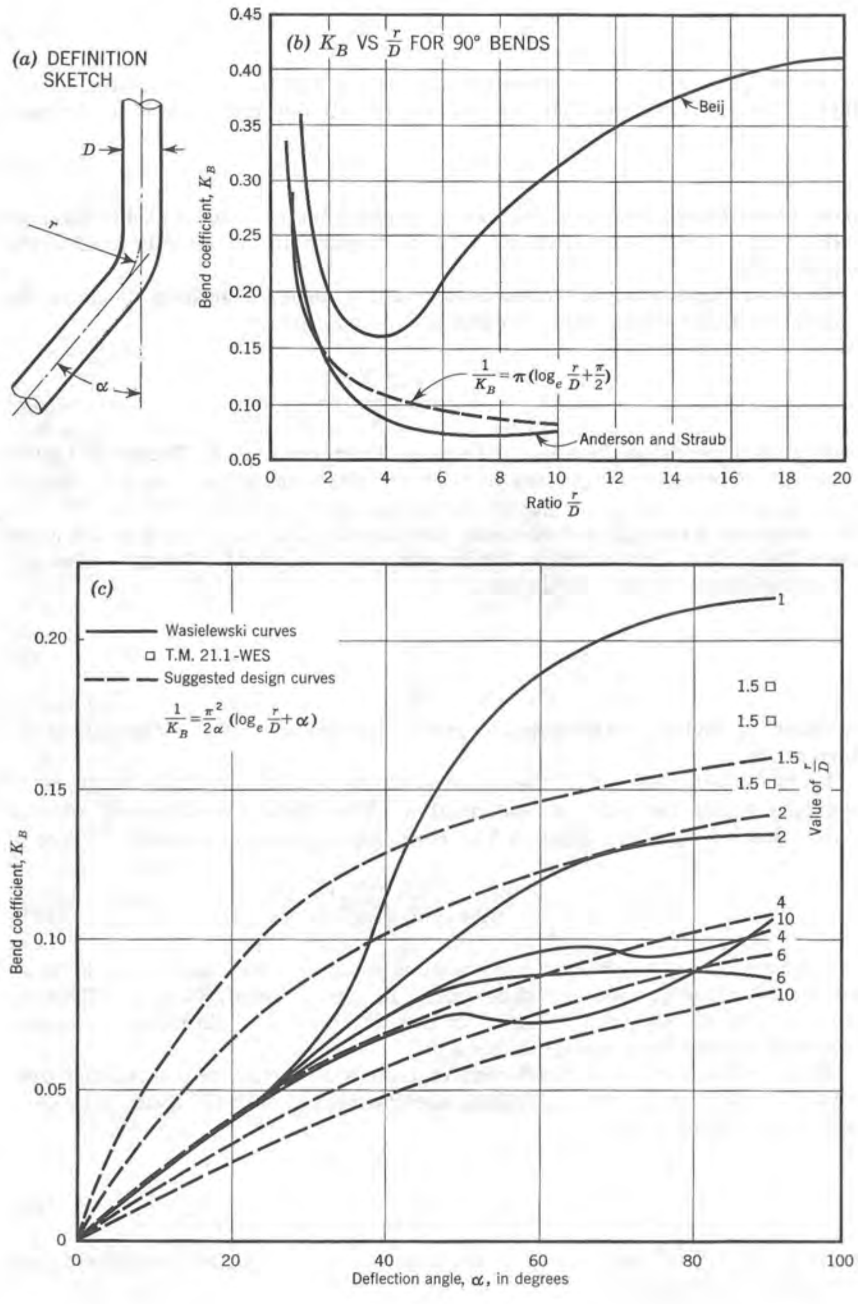

(a) DEFINITION SKETCH
D
r
α
(b) K_B VS $\frac{r}{D}$ FOR 90° BENDS
Beij
$\frac{1}{K_B} = \pi (\log_e \frac{r}{D} + \frac{\pi}{2})$
Anderson and Straub
Bend coefficient, K_B
Ratio $\frac{r}{D}$
(c)
Wasielewski curves
T.M. 21.1-WES
Suggested design curves
$\frac{1}{K_B} = \frac{\pi^2}{2\alpha} (\log_e \frac{r}{D} + \alpha)$
Value of $\frac{r}{D}$
Bend coefficient, K_B
Deflection angle, α, in degrees

FIG. 3

Customarily, bend loss coefficients do not include the effect of normal friction loss in the length of pipe involved. It is generally accepted that the principal parameters affecting the loss coefficient, K, are the deflection angle of the bend, the ratio of bend radius to conduit diameter, and the Reynolds number. The latter is usually either not considered or regarded as unimportant for the ranges reported.

Graphs and curves giving K as a function of these parameters are given by Rouse, King, Creager and the WES (HDC 228-1)(Fig. 3). There is reasonable agreement among these researchers, all indicating that a radius to diameter ratio of between 4 and 6 is the optimum, yielding a K of approximately 0.15 for a 90° bend. For smaller deflection angles, K is proportionally less.

Addison and Rouse indicate that the bend loss can be materially reduced by providing a grid of deflecting vanes extending across the passage. This is seldom done, however, because of the ordinarily minor magnitude of these losses.

Valve Passages.—For circular gate valves in the fully open position, the loss coefficients given by some references are listed in Table 5. For partial openings of this type of valve, the loss increases rapidly until, finally, free

TABLE 5.—CIRCULAR GATE VALVE LOSS COEFFICIENTS

	Diameter of Pipe, in inches	K
WES (Fig. 5)	to 18	0.18
Rouse	Not stated	0.19
King	12	0.07
Brown	24	0
Creager	Not stated	Less than 0.1

or "shooting" flow occurs. Coefficients for this condition are plotted on WES Charts HDC 330-1/1 and HDC 330-1, (Figs. 4 and 5). The basic equation used in Fig. 4 is

$$K_v = \frac{H_L}{V^2/2g} \qquad (23)$$

in which K_v = the valve loss coefficient, H_L = the head loss through valve, and V = average velocity in pipe.

In Fig. 5, the values are calculated from

$$Q = C\,A\,\sqrt{2g\,H_e} \qquad (24)$$

in which C = the valve discharge coefficient, A = the area based on the nominal valve diameter, and H_e = the energy head measured to the centerline of the conduit immediately upstream of the valve.

In Fig. 4, the data are for valves having the same diameter as the pipe, and for downstream pipe flowing full. In Fig. 5, the data are from USBR tests for free flow from 8-in. to 12-in. diam gate valves at the downstream and of conduit of the same nominal diameter as the valve.

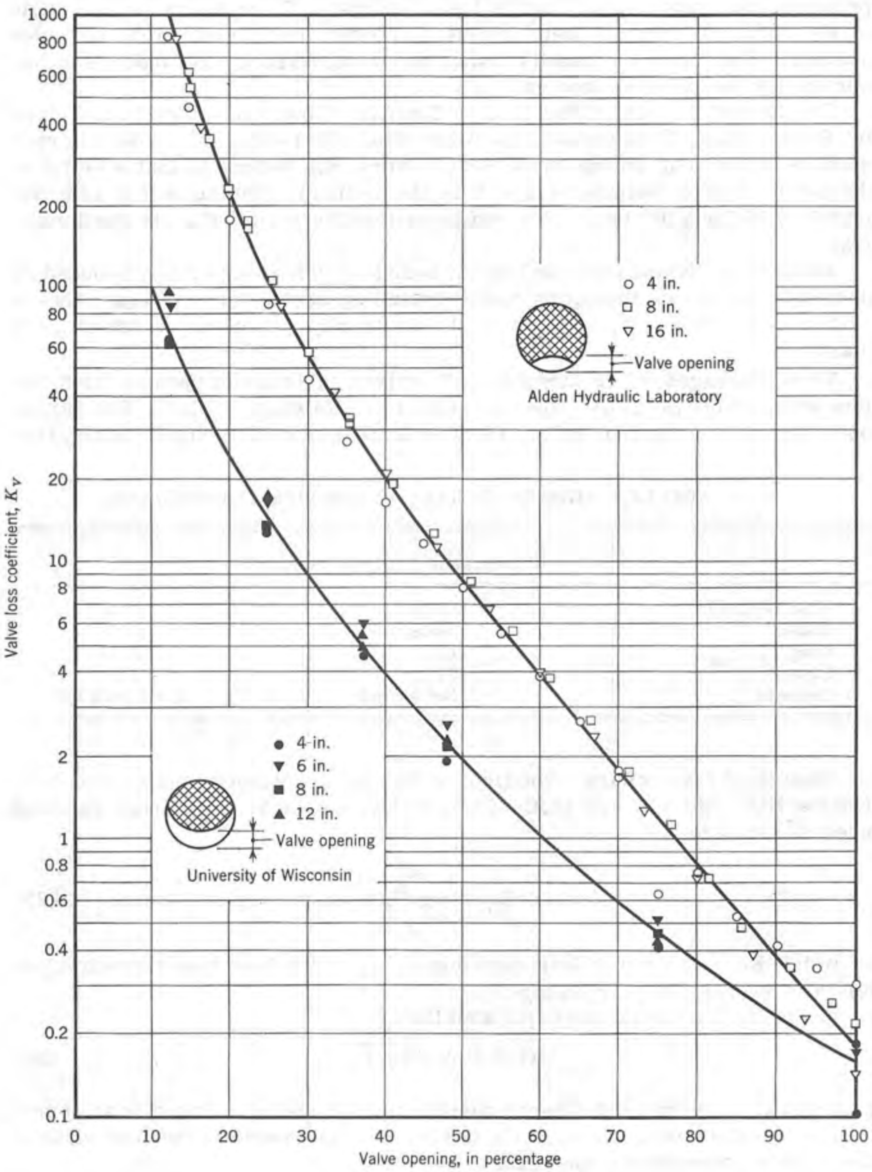

1 000
800
600
400
200
100
80
60
40
20
10
8
6
4
2
1
0.8
0.6
0.4
0.2
0.1
0
10
20
30
40
50
60
70
80
90
100
Valve loss coefficient, K_V
Valve opening, in percentage
4 in.
8 in.
16 in.
Valve opening
Alden Hydraulic Laboratory
4 in.
6 in.
8 in.
12 in.
Valve opening
University of Wisconsin

FIG. 4

Butterfly valves, even in the fully open position, present an obstacle to flow in the form of the disk thickness. Creager suggests K = t/d in which t = the disk thickness and d = the conduit diameter, with K to be applied to the velocity head based on the gross area. R. L. Mahon(26) observes that K = 0.25 is a commonly reported value. L. M. Boyd(27) notes that the loss can be reduced by the use of a converging section on the downstream side of

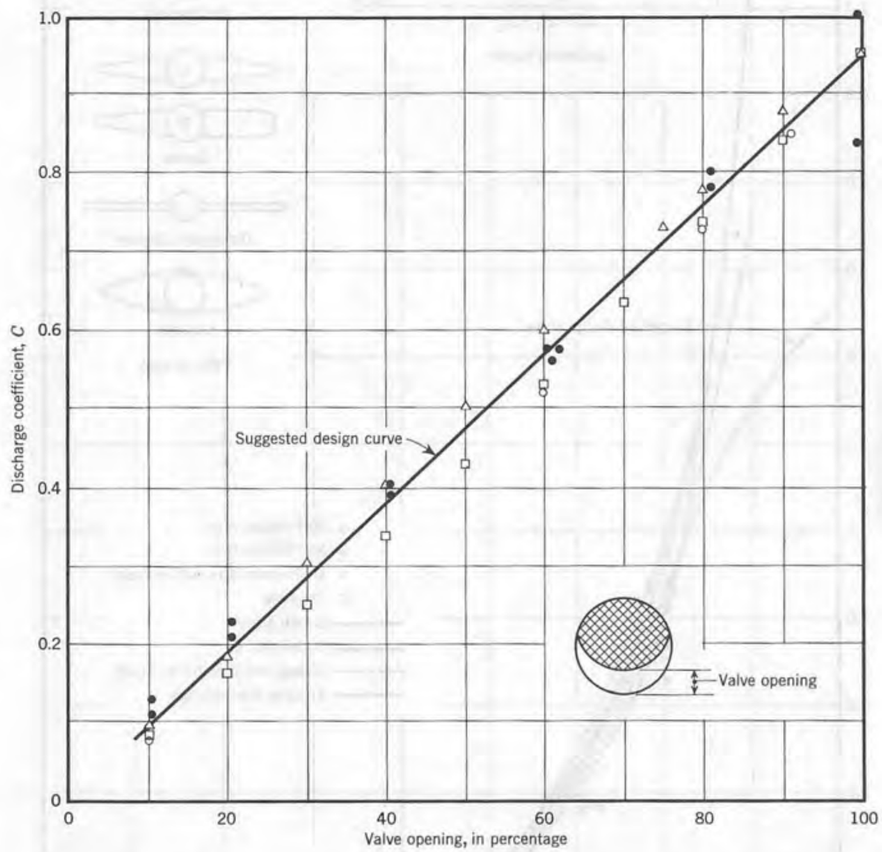
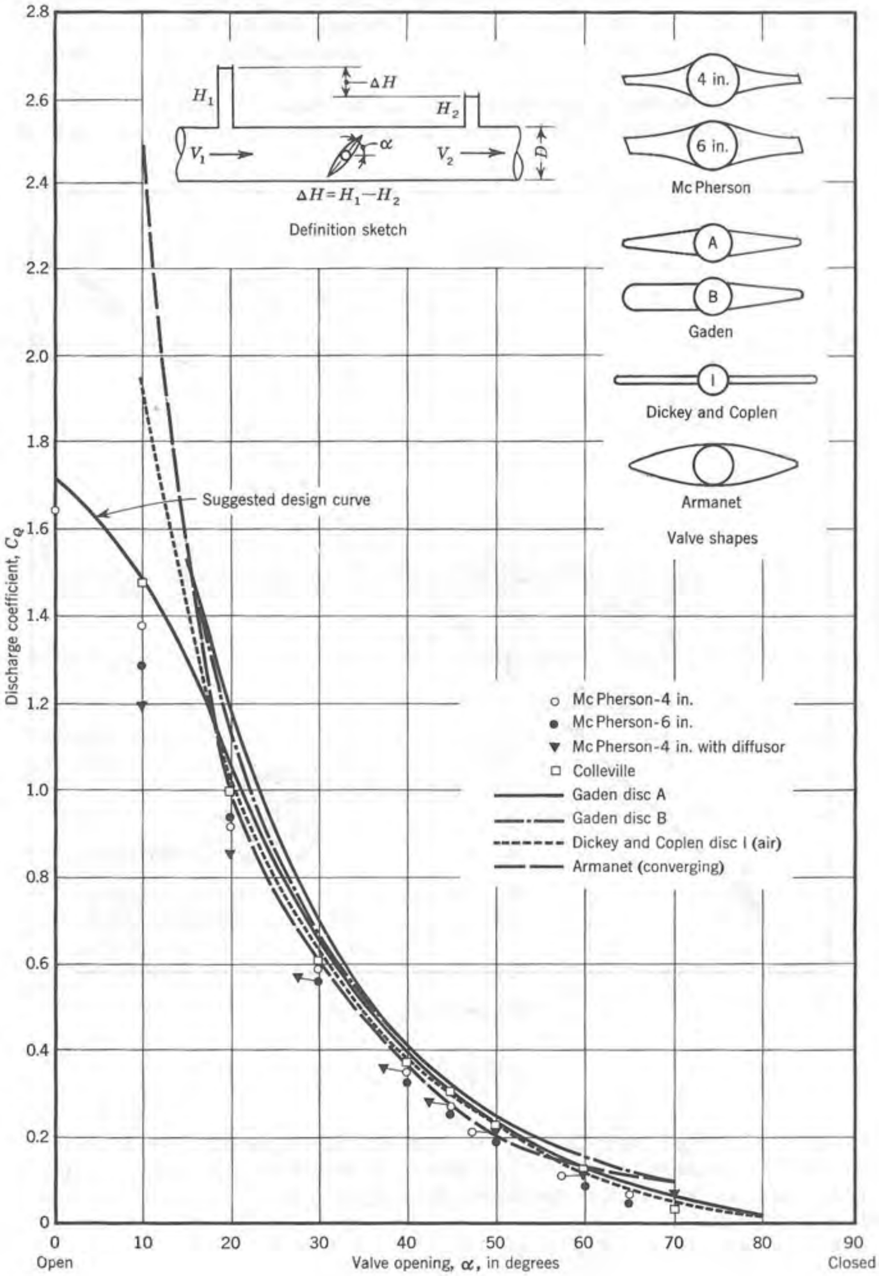


FIG. 5

the disk. The WES chart HDC 331-1 (Fig. 6) expresses the loss in terms of a modified discharge coefficient and shows, as would be expected, a rapid increase in loss for partial openings. Brown gives similar data for intermediate disk positions.

The equation used in Fig. 6 is

$$Q = C_Q D^2 \sqrt{g} \sqrt{\Delta H} \quad \ldots\ldots\ldots\ldots\ldots (25)$$



H_1
ΔH
H_2
V_1
α
V_2
D
$\Delta H = H_1 - H_2$
Definition sketch
4 in.
6 in.
Mc Pherson
A
B
Gaden
I
Dickey and Coplen
Armanet
Valve shapes
Suggested design curve
Mc Pherson-4 in.
Mc Pherson-6 in.
Mc Pherson-4 in. with diffusor
Colleville
Gaden disc A
Gaden disc B
Dickey and Coplen disc I (air)
Armanet (converging)
Discharge coefficient, C_Q
Valve opening, α, in degrees
0
Open
10
20
30
40
50
60
70
80
90
Closed
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0

FIG. 6

in which, Q = the discharge, in cubic feet per second, C_Q = the discharge coefficient, D = the valve diameter, in feet, g = the gravity constant, 32.2 ft per sec per sec, and ΔH = the pressure drop across the valve, in feet of water.

Needle or plunger valve losses are treated briefly in Creager. Coefficients for Howell-Bunger valves, flap gates, and reverse tainter valves are shown on corresponding WES charts.

Wyes and Bifurcations.—These configurations occur frequently in water supply and hydroelectric systems. Although there has been a considerable quantity of research, much of it is either theoretical or based on laboratory tests only. There is a paucity of systematic and codified data suitable for direct application to prototype installations. The problem is compounded because of the many combinations of size, angles, and fabricating methods involved.

E. Mosonyi(28) has noted that hydraulic losses at wye pieces are governed by the angle of bifurcation and by the ratio of cross-sectional areas. Diversion through a branch of part of the flow in a pipeline having a constant cross section will result in the velocities in the main pipe downstream from the point of bifurcation being suddenly reduced. The hydraulic phenomena thus corresponds to that in a sudden expansion. Sudden change in velocity and flow direction results in separation phenomena that can be reduced to minimal values by properly shaped corners and appropriate decreases in cross-sectional areas, to maintain constant flow velocities. He finds the loss coefficients for appropriately proportioned wyes to vary from 0.25 to 0.50, in contrast to those with improperly shaped wyes in which the coefficients may be appreciably higher. In fact, with 90° bifurcations the coefficient may attain the value of 1.0.

TABLE 6.—COEFFICIENTS OF HEAD LOSS AT BIFURCATIONS

Discharge Ratio for $Q = Q_m + Q_b$		Loss Coefficients for Bifurcations of			
		90°		Acute(30°) angles	
Q_b/Q	Q_m/Q_b	K_m	K_b	K_m	K_b
0.10	9	0.03	0.89	0	0.78
0.25	3	0	0.88	0	0.63
0.50	1	0	0.91	0	0.44
0.75	1/3	0.18	1.06	0.16	0.36
0.90	1/9	0.28	1.20	0.26	0.41

Based on the discharge delivered by the main conduit to the point of bifurcation as $Q = Q_m + Q_b$, Mosonyi has developed Table 6 giving values of the loss coefficients for various ratios of Q_b/Q and Q_m/Q_b, and for bifurcations at 90° and at 30°, in which the subscripts m and b denote main penstock and branch pipe, respectively.

He has observed that the local losses in the main conduit may assume negative values for small discharge ratios. This does not constitute a violation of the energy laws, because the increase originates from the decrease in velocity. Such negative loss coefficients are shown in the tabulation above by zero. Streeter(19) shows the necessity of including correction coefficients for the several velocity heads if the true energy balance is to be described.

A carefully conceived and executed test program(29) was conducted at the Lucendro power station in Switzerland to determine the head losses in a section of the 1.2 m diam welded steel penstock containing two 55° wye junctions. For $Q_m = Q_b$, the loss coefficient for each wye averaged nearly K = 1.2 (based on the velocity head in the main penstock). A minimum K of approximately 0.2 was achieved when approximately 40% of the main penstock flow was diverted through a branch. For the entire section, the total head loss was distinctly minimized when the flow was evenly divided between the two branches. The results confirmed earlier data obtained by the use of models.

Miscellaneous Junctions.—Loss coefficients for several tees and acute and obtuse-angled elbows are given by Creager and by Addison. More detailed data on various junctions are furnished by J. R. Freeman.(30)

Exit.—When a conduit empties into a lower reservoir, the condition is analogous to the sudden enlargement discussed previously. Unless a gradually tapered transition is provided, the entire velocity head is lost in the formation of eddies.

AIR BINDING

Introduction.—Although the phenomena of air entrainment and air binding have long been studied, they have not generally been considered a problem in flow through large conduits. This can be attributed to the fact that air effects are normally associated with conduits either flowing under partial vacuum (as found in some condenser cooling systems) or rising above the hydraulic gradient (as those in siphons). In these cases, the difficulties occasioned by air are handled through the provision of valves, pumps, and other mechanical devices.

There is reason to believe, however, that under certain circumstances other large conduits can be adversely affected by air phenomena. Moreover, the conditions may be such that the nature of the problem is not readily recognized; troubles caused by air are often unwittingly blamed on more common causes.

Dynamics of Air Bubbles.—An excellent treatment of the theory bearing on the formation and movement of air bubbles in conduits has been given by A. Veronese(55). A partial translation of his classic paper is deposited in the Engineering Societies Library as Appendix I. Of particular interest is the following:

"In conduits, except when they are not flowing full (in which case the action is the same as in any open channel), free air cannot exist in the water except in the form of bubbles of variable size which usually move, either in the direction of or against the current, directing themselves towards the upper portion of the conduit itself where they can be expelled by means of a suitable device. Between the two preceding conditions there are those cases in which the bubbles remain motionless or undergo local movement without either advancing or receding in the pipe.

"This occurs in three instances:

(1) When the pipe is almost horizontal and the water is either still moving with very low velocity.
(2) When there is discontinuity in the walls, or partial obstructions in the sections which cause eddying in which water and air bubbles remain in a local movement and therefore do not participate in the general motion along the pipe.
(3) In sloping conduits (i.e., where the water flows downward) under certain conditions in which the forces acting on the bubble are in equilibrium."

More recently, J. C. Kent(56) has shown that the velocity required to move an air bubble through a circular conduit is

$$V_{min} = C \sqrt{g\, D \sin \theta} \quad \ldots\ldots\ldots\ldots\ldots (26)$$

in which D = the diameter, θ = the angle of the conduit with the horizontal, and C is on the order of 1.4 (based on data from A. A. Kalinske and P. H. Bliss(52) and by WES).

Model studies on air-water phenomena can be misleading because of similitude difficulties. The following observation(57) by G. A. J. Young and L. Fellerman is pertinent:

"Since air-entraining vortices appear to be free surface phenomena it would be expected that a model with flow conditions and velocities corresponding to the same Froude number as the prototype would give the desired result. This does not seem to be the case, and in small models much higher velocities than those given by the equal Froude number relation have been found necessary to obtain similar air-entraining vortices, and there is evidence that, in a particular size range, air-entraining vortices only occur under similar submergence conditions when prototype velocities are used. The experimental evidence on which this equal velocity rule (i.e., model vel. = prototype vel.) is based covers pipes from 1" diameter to 30" diameter and a limited comparison of models and their prototype does not contradict this rule.

"Extrapolation of the equal velocity rule up to large installations would lead one to expect very large vortices, in some cases extending down to 100 ft. or more. Whether such large vortices exist in practice is not known, and the largest for which we have reliable evidence extended only 18 ft. deep. There is reason to believe that there is a change of law as the size of the installation is increased beyond the range of our experience.

"The equal velocity rule however errs on the safe side. If an air-entraining vortex occurs on the model we cannot be certain that it will occur on the prototype under similar submergence conditions but if the vortex is absent on the model at the equal velocity then it will be absent on the prototype. The basic causes of rotational flow become apparent on the model and remedies that are effective on the model will be equally effective on the prototype."

Effects of Air.—G. Maiguenet, in dealing with air entrainment by flow in secondary supply shafts, has observed (47):

"The presence of air in a pressure tunnel can be the source of grave inconveniences. Let us note principally:

1. The localization of an air pocket at the high point in a tunnel or at a change in slope, which occasions, by a marked loss of head, a diminution of discharge.
2. The 'slipping' of a pocket of air in a tunnel and the rapid elimination by an air vent can provoke a waterhammer by reason of the impact between two water columns, or the discontinuity linked to the loss of head for the air vent.
3. The supply of a turbine by emulsified water affects its operation by a drop in output and efficiency; the presence of air in a Pelton nozzle can be the cause of waterhammer shocks; the admission of air to a pump can occasion loss of priming."

R. T. Richards has additionally noted (48):

"Air binding in a pipeline is the trapping of air within the water passage in a manner which prevents the pipe from flowing full. In poorly laid out or poorly vented pipelines air binding may be the source of head losses which can seriously reduce the pipeline capacity."

D. F. Denny and Young, in an investigation concerned with the prevention of vortices and swirl at intakes(49) show that air carried into an intake by vortices may easily reach 5% of the flow of water (10% of the flow in extreme cases) and can occasion vibration or corrosion damage to pipes and tunnels, as well as serious decreases in the efficiency of waterwheels and pumps. For example, they note 1% of air as being capable of reducing the efficiency of a centrifugal pump by as much as 15%.

Conversely, F. T. Mavis and F. Bustamante conducted a basic model study simulating the effects of air entrapment in unlined water tunnels through rock(50). The investigation was designed to answer the question: To what degree is the hydraulic roughness factor for unlined rock tunnels affected by air entrapment in the overbreak of the tunnel vault? Their investigation was made on a 4-in.-diam, 12.5-ft-long, semicircular lucite model that simulated in pockets in the flat roof section an irregularity in excess of 10% of the height of conduit. Data was secured photographically to minimize observational errors of human origin. The tests showed consistently that there was no difference in effective roughness of a straight conduit whether the roof pockets were filled with water or air. However, they did observe: ". . . a slight increase in velocity for a given slope where air was injected to form a thin, irregular blanket moving along the roof of the conduit."

In practice, however, undesirable effects can and do occur. Generally, three conditions must be satisfied: (1) there must be a source of air in excess of that normally held in solution in the flow; (2) there must be a mechanism whereby air is released from the flow and is able to collect into large air pockets or bubbles; and (3) this collected air must be in a dynamic equilibrium which keeps it from moving along the conduit.

Sources of Air.—Air can enter and accumulate in a conduit by the following means:

1. During filling, air can be trapped along the crown at high points or at changes in cross-sectional size or shape;

2. air may be entrained at the intake, either by vortex action or by means of a hydraulic jump association with a partial gate openings; and

3. air dissolved in the inflowing water may come out of solution as a result of a decrease in pressure along the conduit.

In the first instance above, the entrapment process can be substantial if the filling is performed at such a rate that the release of air upstream does not occur. In any case, it appears reasonable to assume that natural pockets in the crown could, to a large extent, remain filled with air.

Vortices and swirl are caused by the presence of persistent rotational flow in the body of water approaching the intake. Where the core of the vortex is sufficiently deep and the geometry of the intake is vulnerable, considerable quantities of air can be forced into the flow.

The hydraulic jump as an air-entraining mechanism has been studied, among others, by Kalinske and J. M. Robertson(51), Kalinske and Bliss(52), K. Haindl and V. Sotornik(53), and Haindl(54). When the jump occurs downstream from a partly-open gate, a steady stream of air bubbles may be "pumped" downstream, being replenished through the air vent.

Release of dissolved air is a possibility that is often overlooked. However, because the solubility of air in water is proportional to the pressure, it is apparent that there is a tendency for free air to come out of solution in cases where the hydraulic gradient converges with the conduit. Localized pressure drops, such as occur at constrictions, may also act to release air. At standard conditions, water contains over 2% dissolved air by volume. When the intake is deep, the percentage rises substantially. Hence, a significant volume of air may become available for dissolution under decreasing pressure conditions.

Remedial Measures.—As in the case of many engineering problems, prevention would seem to be the best approach. Considering the foregoing, several design and construction principles appear to be obvious:

1. The combination of a deep intake and a decreasing hydraulic gradient should be avoided;

2. vortices that threaten to supply air to a conduit should be suppressed by floating baffles or similar devices;

3. partial gate openings that result in hydraulic jumps should be minimized; and

4. natural traps or pockets along the crown should be avoided.

When some air entrainment is inevitable, as in secondary feeder shafts supplying a main conduit, the provision of a deaeration chamber of enlarged area is usually required. In this connection, M. Galassini(58) has observed that ". . . reduction of velocity is the only effective method of separating the air bubbles from the fluid mass." Such a chamber, of course, must be provided with adequate pumping capacity to vent the accumulated air.

AREAS OF RESEARCH

For more than 200 years, engineers active in the field of water conveyance have been theorizing, experimenting and compiling data on the matter of friction losses. Out of this wealth of material have emerged numerous concepts as to how such losses are related to the physical characteristics of the conduit and the flow regime. Generally, these endeavours have followed one of two paths—the formulation of exponential relations of the Manning type or the establishment of better criteria for evaluating the actual roughness factor in the so-called rational relationship of the Darcy-Weisbach form. Whereas the significance and limitations of both approaches are now generally understood, the need for more information remains.

In the case of exponential-type equations, the question is raised of obtaining sufficient reliable data so that the range of applicability of each variety of formula may be more clearly defined and that coefficients such as Manning's n or Hazen-Williams C may be more accurately predicted by designers. This, in turn, implies that the dimensions, characteristics and performance of complete conduits be reported to the profession. Each project can thus provide a meaningful contribution toward a more complete understanding of an approach which undoubtedly will continue in popularity.

Rational methods, as characterized by the use of the general resistance or f versus **R** diagram, still require considerable research to clarify the role of spacing, shape, and size of roughness projections. Understanding of the turbulent boundary layer has been considerably enhanced during the past 30 years both by theoretical analyses and by newly developed shear measurement techniques. Attention is now being focused on translating a given configuration of roughness elements into a predictable effect on head loss. The course of hydraulic research in this field has logically been leading from the descriptive, empirical, and macroscopic toward the quantitative, experimental, and microscopic.

It would be desirable if some of the newer and more advanced techniques in this field were adopted or at least attempted by a larger number of organizations. Such concepts as using plaster casts to analyze roughness and using air to simulate water (to determine friction parameters of conduits while under construction), as done by Electricité de France, can not only lead to immediate economic benefit but can also help in the search for methods of replacing approximations by certainty in the field of fluid friction.

In this connection, there is also a need for more information on secondary or nonfriction losses in large conduits. Until 1965, prototype data in this regard has tended to be fragmentary or piecemeal. Designers continue to base their estimates largely on laboratory results—some of which date back many years—with mixed success as to accuracy. Considering the extent of field testing that is being done, it would appear that the opportunity is available to obtain a substantial quantity of specific measurements of these losses. This, in turn, could lead to a systematic compilation that would more closely reflect the actual contribution of such losses toward the whole.

As mentioned previously, the head losses caused by air phenomena may often be confused with the friction losses generated by wall roughness. Although the cause and the treatment are quite different, the symptoms are similar and, therefore, the diagnosis is difficult. In light of the considerable research effort that has always been directed toward the general problem of head losses in conduits, it would seem that the airbinding phenomenon warrants more attention.

It is believed that a distinct contribution to knowledge on friction and other losses in large conduits flowing full would result from the collection, analysis, and compilation of loss data for major hydraulic works in North America and elsewhere. A substantial beginning is Engineering Monograph No. 7 of the U.S. Bureau of Reclamation(15), which covers friction losses in lined and fabricated conduits. Expansion of this tabulation to include unlined tunnels and secondary losses would enhance its value. The results, perhaps in a form analogous to the Register of Large Dams, would benefit the profession by bringing together, for the first time, a continually up-to-date record of what has been achieved in this field.

ACKNOWLEDGMENTS

Initial membership of the Task force consisted of: F. B. Campbell, Waterways Experiment Station; T. J. Rhone, U. S. Bureau of Reclamation; J. B. Cooke, Pacific Gas and Electric Co.; H. H. Ambrose, University of Tennessee; and F. L. Lawton, Chairman; Aluminum Laboratories Limited, Montreal, Canada.

It is a matter of great regret that Mr. Ambrose died as a result of an accident; his contribution as a member was most helpful. J. B. Cooke, on leaving Pacific Gas and Electric Co. to take up consulting engineering, resigned. He was succeeded by J. E. Schumann Jr., of Pacific Gas and Electric Co. M. D. Lester of Aluminum Laboratories, Ltd., also participated actively in the preparation of this report.

The Task Force is grateful to the Waterways Experiment Station, Vicksburg, Miss. for the use of Figs. 2 to 6.

This report is respectfully submitted by the Task Force on Flow in Large Conduits, of the Committee on Hydraulic Structures.

Frank B. Campbell
Thomas J. Rhone
James E. Schumann, Jr.
Frederic L. Lawton, Chairman

APPENDIX.—BIBLIOGRAPHY

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KEY WORDS: air entrainment; conduits (water); friction; head losses; reports; water tunnels

ABSTRACT: Problems associated with flow in large conduits are examined, with emphasis on head losses. The several approaches to estimating friction losses in lined, unlined, and partly lined conduits are considered, together with recent prototype data. Similar treatment is given for secondary or minor losses. Sections on organic growths and deposits and on air binding phenomena are also included. An extensive bibliography is provided.

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