CHAPTER 2

HYDRAULIC THEORY

Section I. Introduction

2-1. General. This section presents hydraulic design theory, available experimental data and coefficients, and discussions of certain special problems related to reservoir outlet works design. Generally, the presentations assume that the design engineer is fully acquainted with the hydraulic theories involved in uniform and gradually varied flow, steady and unsteady flows, energy and momentum principles, and other aspects such as energy losses, cavitation, etc., related to hydraulic design as normally covered in hydraulic handbooks and texts such as those by King and Brater (item 56) and Rouse (items 99 and 101). This manual is presented as guidance in the application of textbook material and as additional information not readily available in general reference material. The theory of flow in conduits from a reservoir is essentially the same for concrete and embankment dams. The application of the theory of flow through conduits is based largely upon empirical coefficients so that the designer must deal with maximum and minimum values as well as averages, depending upon the design objectives. To be conservative, the designer should use maximum loss factors in computing discharge capacity, and minimum loss factors in computing velocities for the design of energy dissipators. As more model and prototype data become available, the range between maximum and minimum coefficients used in design may be narrowed. An illustrative example, in which the hydraulic design procedures and guidance discussed in this manual are applied to the computation of a discharge rating for a typical reservoir outlet works, is shown in Appendix D.

2-2. Basic Considerations. The hydraulic analysis of the flow through a flood control conduit or sluice usually involves consideration of two conditions of flow. When the upper pool is at low stages, for example during diversion, open-channel flow may occur in the conduit. As the reservoir level is raised, the depth of flow in the conduit increases until the conduit flows full. In the design of outlet works, the number and size of the conduits and the elevations of their grade line are determined with consideration of overall costs. The conduits are usually designed to provide the required discharge capacity at a specified reservoir operating level, although adequate capacity during diversion may govern in some cases. Conduits should normally slope downstream to ensure drainage. The elevation of good foundation materials may govern the invert elevation of conduits for an embankment dam. If it is planned to use the conduits for diversion, a study of the discharge to be diverted at the time of closure of the river channel may limit the maximum elevation of the conduit. If the conduits are adjacent to the power penstocks, the level of which is governed by the turbine setting, it may be feasible and convenient to place all conduits on the same level. After limiting conditions are determined and preliminary dimensions and grades established by approximate computations, a more exact analysis may be made of the flow through the conduits. It is often more expedient to estimate the size, number, and elevation of the conduits and then check the estimated dimensions by an exact analysis rather than to compute the dimensions directly.

Section II. Conduits Flowing Partially Full

2-3. General. Analysis of partially full conduit flow is governed by the same principles that apply to flow in open channels. The longitudinal profile of the free-water surface is determined by discharge, geometry, boundary roughness, and slope of the channel. Reference is made to plate C-1 for illustration of the principal types of open-channel water-surface profiles. A study of the various profiles will indicate, for any particular conduit, where the discharge control is likely to be located and the type of water-surface profile that will be associated with the control.

2-4. Discharge Controls for Partially Full Flow.

a. Inlet Control. The control section is located near the conduit entrance and the discharge is dependent only on the inlet geometry and headwater depth. Inlet control will exist as long as water can flow through the conduit at a greater rate than water can enter the conduit. The conduit capacity is not affected by hydraulic parameters beyond the entrance, such as slope, length, or boundary roughness. Conduits operating under inlet control will always flow partially full for some distance downstream from the inlet.

b. Outlet Control. The control section is located at or near the conduit outlet; consequently, the discharge is dependent on all the hydraulic parameters upstream from the outlet, such as shape, size, slope, length, surface resistance, headwater depth, and inlet geometry. Tailwater elevation exceeding critical depth elevation at the outlet exit may influence the discharge. Conduits operating under outlet control can flow either full or partially full.

c. Critical Depth Control. Critical flow applies only to free surface flow and occurs when the total energy head (sum of velocity head and flow depth) for a given discharge is at a minimum. Conversely, the discharge through a conduit with a given total energy head will be maximum at critical flow. The depth of flow at this condition is defined as critical depth and the slope required to produce the flow is defined as critical slope. Capacity of a conduit with an unsubmerged outlet will be established at the point where critical flow occurs. A conduit operating with critical depth occurring near the entrance (inlet control) will have maximum possible free-surface discharge. The energy head at the inlet control section is approximately equal to the head at the inlet minus entrance losses. When critical flow occurs downstream from the conduit entrance, friction and other losses must be added to the critical energy head to establish the headwater-discharge relation. Critical depth for circular and rectangular cross sections can be computed with CORPS[o] H6141 or H6140 or from charts given in HDC 224-9[n] and 610-8,[n] respectively. Reference is made to TM 5-820-4[b] and to King's Handbook (item 56) for similar charts for other shapes.

d. Gate Control. It is generally necessary to compute surface profiles downstream from the gate for different combinations of gate openings and reservoir heads to determine the minimum gate openings at which the conduit tends to flow full. The transition from partly full to full flow in the conduit may create an instability that results in slug flow pulsations ("burping") at the outlet exit portal which can create damaging wave action in the downstream channel (item 2). Generally, this instability occurs near fully open gate openings and the outlet works are not operated in this discharge range for any extended period of time. However, it is particularly critical in projects that have a long length of conduit below the gate, and the conduit friction causes the instability to occur at smaller gate openings that are in the planned operating range of the outlet works. The conduit must be examined for slug flow where the ratio of downstream conduit length to conduit diameter or height exceeds 75 (i.e., $L/D \geq 75$). A larger conduit or steepened invert slope may be required to avoid this condition. Additional details and an example analysis are given in Appendix D.

2-5. Flow Profiles. EM 1110-2-1601[h] presents the theory involved in computing flow profiles for prismatic channels. Its application to the problem with a sample computation is given in Appendix D.

Section III. Conduits Flowing Full

2-6. General. The objective of the analysis of conduits flowing full is to establish the relation between discharge and total head and to determine pressures in critical locations. The solution is implicit and involves the simultaneous solution of the Darcy-Weisbach equation, the continuity equation, and the Moody diagram to determine the unknown quantities. A detailed explanation of the computational procedure is presented in Appendix D. The total head H , which is defined as the difference in elevation of the upstream pool and the elevation of the hydraulic (pressure) grade line at the exit portal, is consumed in overcoming frictional (h_f) and form (h_ℓ) losses and in producing the exit portal discharge velocity head (h_v). These component heads may be equated to the total head as follows:

$$H = h_f + h_\ell + h_v \tag{2-1}$$

Plate C-2 is a definition sketch showing the relation between these various components in an outlet works system.

2-7. Exit Portal Pressure Grade-Line Location. The elevation of the hydraulic (pressure) grade line at the exit portal for unsubmerged flow (into the atmosphere) is not as obvious as it may appear. Laboratory tests made at the State University of Iowa (item 103) have indicated that the elevation of the intersection of the pressure grade line with the plane of the exit portal is a function of the Froude number of the conduit flow. Plate C-3 shows the results of these and other tests for circular and other conduit shapes. The values of y_p/D are also dependent upon the condition of support of the issuing jet. The "Suggested Design Curve" on this plate is based upon analyses of model and prototype data. Plate C-3 indicates that a good approximation for the initial location is two-thirds the vertical dimension above the exit portal invert. Model and prototype tests have indicated the hydraulic (pressure) grade line at the exit portal can be depressed to near the conduit invert for certain geometrics and flow conditions (see Chapter 5, para 5-2d(2)). If the exit portal is deeply submerged, the hydraulic grade line at the outlet will be at the local tailwater elevation. However, at lower degrees of submergence the outflow will tend to depress the local water surface below the surrounding tailwater elevation. This depression and the accompanying hydraulic jump action for two-dimensional flow can be analyzed as described by Rouse or Chow (items 101 or 17, respectively). However, submerged conduit outflow into a wider channel is not subject to simple analysis. If submerged flow conditions are critical relative to conduit capacity, local pressures at the outlet, or stilling basin performance, a hydraulic model investigation will be needed.

Section IV. Gradients

2-8. General. The basic principle used to analyze steady incompressible flow in a conduit is the law of conservation of energy as expressed by the Bernoulli equation. Generalized so that it applies to the entire flow cross section, the expression for the energy at any point in the cross section in foot-pounds per pound of water is given by:

$$H = Z + \frac{p}{\gamma} + \alpha \frac{V^2}{2g} \tag{2-2}$$

where

H = total head in feet of water above the datum plane

Z = difference in elevation of the point and the elevation of a datum plane

p = pressure at the point, lb/ft^2

γ = specific weight of water, lb/ft^3

V = flow velocity, fps

g = acceleration due to gravity, ft/sec^2

α = dimensionless kinetic-energy correction factor

For many practical problems α may be taken as unity without series error.

2-9. Hydraulic Grade Line and Energy Grade Line. The hydraulic grade line, also referred to as the mean pressure gradient, is p/γ above the center line of the conduit, and if Z is the elevation of the center of the conduit, then $Z + p/\gamma$ is the elevation of a point on the hydraulic grade line. The locus of values of $Z + p/\gamma$ along the conduit defines the hydraulic grade line or mean pressure gradient. The location of the hydraulic grade line at any station along the conduit is lower than the energy grade line by the mean velocity head at that station as reflected by equation 2-2. See plate C-2 for a definition sketch of the energy grade line, hydraulic grade line, etc. The hydraulic grade line is useful in determining internal conduit pressures and in determining cavitation potentialities. Information on local pressure conditons at intakes, gate slots, and bends is given in the appropriate paragraphs of this manual. For purposes of structural design, pressure gradient determinations are usually required for several limiting conditions.

2-10. Mean Pressure Computation. The mean pressure at any station along a conduit is determined using the conservation of energy principle as expressed by the Bernoulli equation. The principle states that the energy at one station of the conduit (point 1) is equal to the energy at any downstream location (point A) plus any intervening losses. Expressed in equation form and in the units of equation 2-2,

$$Z_1 + \frac{p_1}{\gamma} + \alpha_1 \frac{V_1^2}{2g} = Z_A + \frac{p_A}{\gamma} + \alpha_A \frac{V_A^2}{2g} + H_{L_{1-A}} \qquad (2\text{-}3)$$

If the upstream station is taken in the reservoir near the conduit entrance where the velocity head is negligible, and $Z_1 + (p_1/\gamma)$ is taken as the pool elevation, equation 2-3 reduces to

$$\frac{p_A}{\gamma} = \text{pool elevation} - \frac{V_A^2}{2g} - H_{L_{1-A}} - Z_A \qquad (2\text{-}4)$$

Equation 2-4 is applicable to the general case of determining the mean pressure of any station along the conduit, with proper consideration being given to head losses due to friction and form changes between the entrance and station in question. For a uniform section, the pressure at any station (point A) upstream of the exit portal (point 2) can be determined by the following equation:

$$\frac{p_A}{\gamma} = Z_2 + y_p - Z_A + H_{L_{2-A}} \qquad (2\text{-}5)$$

where

p_A/γ = pressure head in feet of water at any station

$H_{L_{2-A}}$ = total hydraulic loss in feet between the exit portal and the station

$Z_2 + y_p - Z_A$ = difference in feet between the mean pressure gradeline elevation at the exit portal and the point elevation at the station in question.

Section V. Energy Losses

2-11. General. Energy losses within conduits fall into two general classifications: (a) surface resistance (friction) caused by shear between the confining boundaries and the fluid and (b) form resistance resulting from boundary alignment changes. Computational procedures for both types are given in the following paragraphs.

2-12. Surface Resistance (Friction).

a. General. Three basic equations have generally been used in the United States for computing energy losses in pressurized systems. The Manning equation has been used extensively for both free surface and pressure flow. The Hazen-Williams formula has been used for flow of water at constant temperature in cast iron pipes. The Darcy-Weisbach formula is adopted in this manual and is preferred because through use of the Moody diagram (plate C-4), the Reynolds number and the effective roughness properly account for the differing friction losses in both the transitional and fully turbulent flow zones.

b. Darcy-Weisbach Formula. The Darcy-Weisbach formula is expressed as

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \qquad (2\text{-}6)$$

where h_f is the head loss, or drop in hydraulic grade line, in the conduit length L, having an inside diameter D, and an average flow velocity V. The head loss (h_f) has the dimension length and is expressed in terms of foot-pounds per pound of water, or feet of water. The resistance coefficient f is a dimensionless parameter. Moody (item 73) has constructed one of the most convenient charts for determining resistance coefficients in commercial pipes and it is the basis for pipe-flow computations in this manual.

c. Effects of Viscosity. Nikuradse (item 82) demonstrated by experiments that the resistance coefficient f varies with Reynolds number $\mathbb{R}$. (Reynolds number is defined in plate C-4.) Von Karman and Prandtl (items 142 and 94, respectively) developed a smooth pipe equation based on the Nikuradse tests as follows:

$$\frac{1}{\sqrt{f}} = 2 \log_{10} \mathbb{R}\sqrt{f} - 0.8 \qquad (2\text{-}7)$$

This equation is shown as the "smooth pipe" on curve in plate C-4. Prototype tests have shown that a hydraulically smooth condition can exist in both concrete and steel conduits over a wide range of Reynolds numbers. Reference is made to plate C-4 for data from tests of concrete conduits and to HDC 224-1/1[n] for steel conduits.

d. Effect of Relative Roughness. The rough pipe tests of Nikuradse have served as a valuable basis for determining the effect of relative roughness (D/k). The symbol k represents the absolute roughness of the pipe wall, which for random roughness is taken as 2σ where σ is considered to be the root-mean-square of the height of the roughness elements. D represents the pipe diameter. The Von Karman-Prandtl (item 142) equation for a rough pipe and fully established turbulent flow is:

$$\frac{1}{\sqrt{f}} = 2 \log_{10} \frac{D}{2k} + 1.74 \qquad (2\text{-}8)$$

Thus, for this type of flow, the resistance coefficient is a function only of relative roughness and is independent of Reynolds number. Therefore, representation of the equation appears as a series of horizontal lines on the upper right-hand portion of plate C-4. Values of f based on prototype concrete conduit measurements are plotted in this plate. These values of k were obtained mathematically from hydraulic measurements and are essentially effective roughness values rather than physical values. Very few published roughness coefficients (items 16 and 30) are physical values and all should be considered as effective or hydraulic rather than absolute roughness values. Rouse (item 101) has proposed an equation that defines the lower limit of the rough flow zone as follows:

$$\frac{1}{\sqrt{f}} = \frac{\mathrm{IR}}{200} \frac{k}{D} \qquad (2\text{-}9)$$

The equation is shown as a dotted line in plate C-4.

e. Transition Region. The area on the Moody diagram between the smooth pipe curve and the rough flow limit may be considered as a transition region. Colebrook and White (item 18) published an equation based on their experiments to span the transition region. The equation is:

$$\frac{1}{\sqrt{f}} = -2 \log_{10} \left(\frac{k}{3.7D} + \frac{2.51}{\mathrm{IR} \sqrt{f}} \right) \qquad (2\text{-}10)$$

The relation is shown as dashed lines in plate C-4.

f. Noncircular Cross Sections. The Darcy f is expressed in terms of the conduit diameter and therefore is theoretically only applicable to conduits having circular cross sections. The concept of equivalent or hydraulic diameter has been devised to make it applicable to noncircular sections. This concept assumes that the resistance losses in a noncircular conduit are the same as those in a circular conduit having an equivalent hydraulic radius and boundary roughness.

$$D = 4R = \frac{4A}{P} \qquad (2\text{-}11)$$

where

R = hydraulic radius of the noncircular conduit

D = diameter of a circular conduit having the same hydraulic radius

A = conduit area

P = wetted perimeter

A WES study (item 19) has shown that the equivalent diameter concept is applicable to all conduit shapes normally used in the Corps' outlet works structures. Plate C-5 gives the relation between A, P, and R for various common conduit shapes. Geometric elements of rectangular, circular, oblong, and vertical-side horseshoe-shaped conduits showing full or partly full can be computed with CORPS° H2041, H6002, H2042, and H2040, respectively. See paragraph 4-2c for a discussion of when conduit shapes other than a circular section should be considered. Flow characteristic curves computed by the USBR (item 50) for their standard, curved-side, horseshoe-shaped conduit are presented in plate C-6. This shape is the same as that presented at the bottom of plate C-5.

g. Design Guidance for Roughness. The Colebrook-White equation (eq 2-10) is recommended for computing the resistance coefficient f since it is applicable to either smooth, transition, or rough flow conditions. Computations of discharge and head loss at given total heads for rectangular, circular, or oblong, and vertical-side horseshoe-shaped conduits flowing full can be computed with CORPS[o] H2044, H2045, and H2043, respectively. The solution is implicit; and without the aid of a computer, it is more convenient to graphically obtain values of f from a Moody-type diagram as illustrated in plate C-4. However, to use the Moody diagram requires knowledge of the effective roughness parameter. Recommended k-values for various conduit materials are shown below:

(1) Concrete. The following values of k are recommended for use in the design of concrete sluices, tunnels, and conduits.

(a) Capacity. Conservatively higher values of roughness should be used in designing for conduit capacity. The k values listed below are based on the data presented in paragraph (c) below and are recommended for capacity design computations.

Type	Conduit Size ft	k ft
Asbestos cement pipe	Under 2.0	0.0003
Concrete pipe, precast	Under 5.0	0.0010
Concrete conduits (circular)		0.0020
Concrete conduits (rectangular)		0.0030

(b) Velocity. The smooth pipe curve in plate C-4 should be used for computing conduit flow velocity for the design of outlet works energy dissipators. It should also be used for all estimates for critically low pressures in transitions, bends, etc., as well as for the effects of boundary offsets projecting into or away from the flow.

(c) Miscellaneous. Available test data on concrete pipes and conduits have been analyzed to correlate the effective roughness k with construction practices in forming concrete conduits and in treatment of interior surfaces (HDC 224-1[n]). The following tabulation gives information pertinent to the data plotted in plate C-4. The type of construction and the resulting effective roughness can be used as guides in specific design problems. However, the k values listed are not necessarily applicable to other conduits of different sizes.

Plate C-4 Symbol	Project	Shape*	Size ft	k ft	Construction
			Precast Pipe		
●	Asbestos cement	C	1.2	0.00016	Steel mandrel
□	Asbestos cement	C	1.7	0.00008	Steel mandrel
▽	Neyrpic	C	2.82	0.00030	19.7-ft steel form
⊖	Denver #10	C	4.5	0.00018	12-ft steel form
⊐	Umatilla River	C	3.83	0.00031	8-ft steel form
⊤	Prosser	C	2.54	0.00152	Oiled steel form
⊏	Umatilla Dam	C	2.5	0.00024	4-ft sheet steel on on wood forms
⊥	Deer Flat	C	3.0	0.00043	6-ft steel form
×	Victoria	C	3.5	0.00056	4-ft oiled steel forms
▲	Denver #3	C	2.5	0.00011	12-ft steel form
◣	Denver #13	C	5.0	0.00016	12-ft steel form
▽	Spavinaw	C	5.0	0.00013	12-ft steel form
			Steel Form Conduits		
○	Denison	C	20	0.00012	
△	Ontario	O	18	0.00001	Hand-rubbed
▼	Chelan	C	14	0.00061	
■	Adam Beck	C	45	0.00018	Invert screeded and troweled
⊟	Fort Peck	C	24.7	0.00014	
◊	Melvern	H	11.5	0.00089	
♦	Beltzville	C	7	0.00009	
			Wood Form Conduits		
◒	Oahe	C	18.3	0.00004	Joints ground
+	Enid	C	11	0.00160	

(Continued)

* C = circular, O = oblong, R = rectangular, and H = horseshoe.

Plate C-4 Symbol	Project	Shape*	Size ft	k ft	Construction
			Wood Form Conduits (Continued)		
◑	Pine Flat 52	R	5 × 9	0.00103	
◐	Pine Flat 56	R	5 × 9	0 00397	Longitudinal planking
			Miscellaneous		
⊙	Quabbin	H	11 × 13	0.00015	Unknown

* C = circular, O = oblong, R = rectangular, and H = horseshoe.

(2) Steel.

(a) Capacity. The k values listed in the tabulation below are recommended for use in sizing cast iron and steel pipes and conduits to assure discharge capacity. The values for large steel conduits with treated interior surfaces should also be useful in the design of surge tanks under load acceptance. The recommended values result from analysis of 500 resistance computations based on the data presented in HDC 224-1/1[n] and in Table H of item 13. The data are limited to continuous interior iron and steel pipe. The recommended design values are approximately twice the average experimental values for the interior treatment indicated. The large increase in k values for large size tar- and asphalt-treated conduits results from heavy, brushed-on coatings.

Diameter ft	Treatment	k ft
Under 1.0	Tar-dipped	0.0001
1 to 5	Tar-coated	0.0003
Over 5	Tar-brushed	0.0020
Under 6	Asphalt	0.0010
Over 6	Asphalt-brushed	0.0100
All	Vinyl or enamel paint	0.0001
All	Galvanized, zinc-coated or uncoated	0.0006

(b) Velocity. The smooth pipe curve in plate C-4 is recommended for all design problems concerned with momentum and dynamic forces (stilling basins, trashracks, water hammer, surge tanks for load rejection, critical low pressures at bends, branches, offsets, etc.).

(c) Miscellaneous. The following tabulation summarizes the data plotted in HDC $224\text{-}1/1^n$ and can be used as a guide in selecting k values for specific design problems. However, the k values listed do not necessarily apply to conduits having different diameters.

Project	Diameter ft	k ft	Remarks
Neyrpic	2.60	0.000010	Spun bitumastic coating
Neyrpic	2.61	0.000135	Uncoated
Milan	0.33	0.000039	Zinc-coated
Milan	0.49	0.000026	Zinc-coated
Milan	0.82	0.000071	Zinc-coated
San Gabriel	10.25	0.000004	Enameled
San Gabriel	4.25	0.000152	Enameled
Hoover	0.83	0.000133	Galvanized pipe
Fort Randall	22.00	0.000936	Tar-coated
Fort Randall	22.00	0.000382	Tar-coated
Fort Randall	22.00	0.000008	Vinyl-painted
Garrison	24.00	0.000005	Vinyl-painted

(d) Aging Effects. Interior treatment of pipes and conduits is of importance to their service life. Chemical, organic, and inorganic deposits in steel pipes and conduits can greatly affect resistance losses and conduit capacity over a period of time. Data by Moore (item 74) indicate that over a 30-yr period, incrustation of bacteria up to 1 in. thick formed in uncoated 8-in. water pipe. Similar conditions prevailed in 10-in. pipe where the bond between the pipe and the interior coal tar enamel was poor (item 38). Computed effective k values for these pipes were 0.03 and 0.02 ft, respectively. Data compiled by Franke (item 38) indicate that organic and inorganic incrustations and deposits in steel conduits up to 6 ft in diameter increased resistance losses by as much as 100 to 300 percent with effective k values increasing twenty to one-hundred fold. The data indicate that the interiors of some of the conduits were originally treated with a coat of bitumen. The changes occurred in periods of 5 to 17 yr.

(3) Corrugated Metal. The mechanics of flow in corrugated metal

and structural plate pipe are appreciably different from those occurring in steel and concrete pipe (items 44 and 117). Both the height of the corrugations (k) and their angle to the flow are important factors controlling the resistance coefficients (f) values. HDC 224-1/2 and 224-1/3[n] show the effects of pipe diameter, corrugation height and spacing, and flow Reynolds number for pipes with corrugations 90 deg to the flow. More recently Silberman and Dahlin (item 112) have analyzed available data in terms of pipe diameter, helix angle, and resistance coefficient and published a design chart based on these parameters. This chart is included as Plate C-7. The correlation shown indicates that pipe size and helix angle are of primary importance in resistance losses. The use of plate C-7 for the hydraulic design of corrugated pipe systems is recommended. Corrugated metal is not recommended for high pressure-high velocity systems (heads >30 ft, and velocities >10 fps). For this reason the published f values can be used for both capacity and dynamic design. Invert paving reduces resistance coefficients for corrugated metal pipe about 25 percent for 25 percent paved and about 45 percent for 50 percent paved.

(4) Unlined Rock Tunnels.

(a) General. Unlined rock tunnels have been used for flood flow diversion and hydropower tunnels where the rock is of sound quality. Generally, it is more economical to leave these tunnels unlined unless high-velocity flows are involved, considerable rock remedial treatment is required, or lining in fractured rock may be required. Existing resistance coefficient data have been studied by Huval (item 52) and summarized in HDC 224-1/5 and 224-1/6.[n] Field measurements of friction losses in the Corps' Snettisham diversion tunnel have been reported by WES (item 75). Accurate k values cannot be determined prior to initial tunnel blasting. Consequently, a range of probable k values based upon blasting technique and local rock characteristics must be investigated to determine tunnel size. Information of this type can sometimes be obtained by studying blasting techniques used and results obtained in the construction of tunnels in rock having similar characteristics. Adjustment to the tunnel size could be made after tunneling begins.

(b) Shape. Unlined rock tunnels are usually horseshoe-shaped. Structural stability normally requires a rounded roof. Economical blasting and rock removal operations usually require a flat or nearly flat invert.

(c) Limiting Velocities. Generally, velocities in unlined tunnels should not exceed 10 fps except during diversion flow when velocities up to about 15 fps may be acceptable. For a tunnel with downstream turbines, penstocks, or valves, it has been recommended that velocities be limited to 5 fps or less to prevent damage from migration of tunnel muck fines and rock falls.

(d) Rock Traps. Rock traps must be provided where damage to downstream turbines, stilling basins, etc., can result from rock fall material moving with the flow. Access to these traps is required for inspection and occasional cleaning out. The development of satisfactory rock trap design and size is presented in items 23 and 66. A rock trap designed to trap debris without interrupting the tunnel flow is described in item 47.

2-13. Form Resistance.

a. General. Energy losses caused by entrances, bends, gates, valves, piers, etc., are conventionally called "minor losses" although in many situations they are more important than the losses due to conduit friction discussed in the preceding section (item 118). A convenient way of expressing the minor losses in flow is

$$h_\ell = K \frac{V^2}{2g} \tag{2-12}$$

where

h_ℓ = head loss, ft

K = dimensionless coefficient usually determined experimentally

V = designated reference velocity, fps

g = acceleration due to gravity, ft/sec^2

The reference velocity in the following energy loss equations corresponds to a local reference section of the conduit at or near the point where the loss occurs. In a conduit with varying cross-sectional area (and inversely varying average velocity) along its length, the individual local loss coefficients (K's) can be adjusted to a single, general reference section for combining into a single total loss coefficient. To do this, each local coefficient (K) should be multiplied by a factor A_G^2/A_L^2, where A_G is the cross-sectional area at the general reference section and A_L is the area at the local reference section.

b. Sudden Expansion. In almost all cases the loss coefficient

K is determined by experiment. However, one exception is the head loss for a sudden expansion (items 101 and 118). Designating the smaller upstream section as section one and the larger downstream conduit as section two, equation 2-12 may be written as

$$h_\ell = \left(1 - \frac{A_1}{A_2}\right)^2 \frac{V_1^2}{2g} = K \frac{V_1^2}{2g} \tag{2-13}$$

in which

$$K = \left(1 - \frac{A_1}{A_2}\right)^2 \tag{2-14}$$

where A_1 and A_2 are the respective upstream and downstream conduit cross-sectional areas, and the reference velocity is the upstream velocity V_1. Note that the head loss varies as the square of the velocity. This is essentially true for all minor losses in turbulent flow. Furthermore, if the sudden expansion is from a submerged exit portal into a reservoir, $A_1/A_2 = 0$ and the loss coefficient K becomes unity and the head loss h_ℓ is equal to the velocity head. A plot showing K as a function of the area ratios is shown in plate C-8.

c. Sudden Contraction. Plate C-8 also illustrates the loss coefficient K as a function of a ratio of the downstream to upstream cross-sectional areas. The head loss h_ℓ due to a sudden contraction is subject to the same analysis as the sudden expansion, provided the amount of contraction of the jet is known (items 101 and 118). Using the downstream conduit velocity V_2 as the reference velocity, equation 2-14 may be written as

$$h_\ell = \left(\frac{1}{C_c} - 1\right)^2 \frac{V_2^2}{2g} = K \frac{V_2^2}{2g} \tag{2-15}$$

in which

$$K = \left(\frac{1}{C_c} - 1\right)^2 \tag{2-16}$$

where C_c is the contraction coefficient (i.e., the area of the jet at the vena contracta section divided by the conduit area at the vena contracta). Thus, as illustrated by plate C-8, the head loss at the entrance to a conduit from a reservoir is usually taken as $0.5\ V^2/2g$, if the entrance is square-edged.

d. Transitions. Plate C-9 summarizes the available data for gradual expansions and gradual contractions in circular sections (conical transitions). Gradual expansions, which are referred to as conical diffusers (items 101 and 118) have been tested by Gibson (item 41), Huang (item 51), and Peters (item 92). These tests show the loss coefficient to be a function of the flare angle of the truncated cone. In the case of the gradual contraction, Schoder and Dawson (item 107) give the head loss in the upstream contracting section of a venturi meter as 0.03 to 0.06 $(V^2/2g)$, where V is the throat velocity. More recent data by Levin (item 59) gives loss coefficient values for flare angles up to 90 deg. Levin's data appear on the bottom of plate C-9. The loss coefficients shown in plate C-9 are applicable in equation 2-13 for both expansions and contractions where the reference velocity is in the smaller conduit. Approximate loss coefficients for rectangular-to-rectangular and rectangular-to-circular transitions have been published by Miller (item 72).

e. Bends.

(1) General. The mechanics of flow in bends is discussed by Yarnell (item 146), Hoffman (item 49), Anderson (item 4), and Zanker and Brock (item 147). Anderson includes detail summaries of the literature with many design graphs. More up-to-date but less detailed summaries are presented by Zanker and Brock.

(2) Losses. The bend loss, excluding friction loss, for a conduit is a function of the bend radius, conduit size and shape, and deflection angle of the bend. It has been found that the smoothness of the boundary surface affects the bend loss, but the usual surface of a flood control conduit permits it to be classed as smooth pipe for the determination of bend losses. Hoffman (item 49) and Wasielewski (item 144) have established that bend losses are independent of the Reynolds number for values in excess of 200,000. The Reynolds number need not be considered for computing bend losses for the design of flood control conduits, but it may be of importance in small-scale models of bends. Dimensionless loss coefficients based on equation 2-12 have been determined experimentally for bends in circular (items 49, 144, and 146) and rectangular (items 64 and 116) conduits.

(a) Circular Conduits. Loss coefficients for circular conduits having circular or single miter bends with deflection angles up to 90 deg are given in plate C-10. Bend loss coefficients for multiple miter bends in circular conduits with deflection angles from 5 to 90 deg are given in plate C-11 (item 4).

(b) Rectangular Conduits. Loss coefficients for rectangular conduits having circular and single miter bends have been published by Sprenger (item 116) and Madison and Parker (item 64). Plate C-12 shows the effects of Reynolds number and bend radii on rectangular conduits having 90 deg bends and height-width ratios of 0.5 and 2.0. Plate C-13 gives relative loss coefficients for rectangular conduits having circular bends varying from 10 to 180 deg (item 64). The bend loss coefficient from plate C-12 should be multiplied by the appropriate relative loss coefficient given in plate C-13. Plate C-14 shows the effects of Reynolds number ($\mathbb{R}$) on loss coefficients for various triple bend combinations with $\mathbb{R}$ in the vicinity of 10^5 (item 116).

f. Branches and Junctions.

(1) General. Branches (wyes, tees, etc.) are not normally found in outlet works but are encountered in the design of penstocks and water supply systems. Junctions (manholes) are frequently encountered in sewer (storm and domestic) design and junction boxes are occasionally used with gates as control structures for low-head outlet works. HDC $228\text{-}5^{n}$ presents design information on pressure change coefficients for junction boxes with in-line circular conduits and illustrates a procedure to compute the head loss for these structures.

(2) Experimental Data. Early interest in dividing and combining flow was generally limited to commercial pipe fittings (Vogel (item 141, 1928); Petermann (item 91, 1929)). In 1938 the USBR (item 135) published the results of experiments on junction losses. This was probably the first effort to minimize head losses and optimize pressure conditions in large diameter branching conduits through experimental design. The more recent works of Marchetti and Noseda (item 65), Syamala Rao (item 119), Ruus (item 105), and Williamson and Rhone (item 145) indicate the revival of interest in branches and junctions of large conduits. Miller (item 72) presents a summary of experimental data on dividing and combining flows in branches through 1970. Correlation of dimensionless loss coefficients from the literature is difficult because of the wide variations in geometry tested. Since structures of this type are not frequently used in reservoir outlet works, only the literature is cited to assist the designer.

g. Equivalent Length. Form losses may be expressed in terms of the equivalent length of pipe L_e that has the same head loss for the same discharge. Equating the head loss due to form losses and the Darcy-Weisbach equation,

$$f \frac{L_e}{D} \frac{V^2}{2g} = K \frac{V^2}{2g} \qquad (2\text{-}17)$$

in which K may refer to one form loss or the sum of several losses. Solving for L_e

$$L_e = \frac{KD}{f} \qquad (2\text{-}18)$$

For example, assume the total form loss coefficient in a 4-ft-diam conduit equals 20 (i.e., $K = 20$) and $f = 0.02$ for the main line; then to the actual length of conduit may be added $20 \times 4/0.02 = 4000$ ft , and this additional or equivalent length causes the same resistance as the form losses, within a moderate range of Reynolds numbers.

Section VI. Cavitation

2-14. General. (Items 8, 57, 97 and 127.) Cavitation is the successive formation and collapse of vapor pockets in low-pressure areas associated with high-velocity flow. Cavitation frequently causes severe damage to concrete or steel surfaces and it may occur at sluice entrances, downstream from gate slots, on edges of baffle blocks, at sharp bends in pipes, on tips of needle valves, etc. The roughening or formation of pockets in surfaces resulting from cavitation is commonly called "pitting." Surface erosion resulting from debris (rocks, gravel, etc.) is sometimes mistaken for cavitation, and cavitation damage may be difficult to determine from examination of the surface within the damaged area. Debris erosion may sometimes be identified by grooves in the direction of flow. While cavitation is normally associated with high-velocity systems, it can occur in low-velocity systems with certain local boundary geometry and flow conditions. The classical case is that of the venturi meter (item 99) in a low-head system (plate C-15). Cavitation is usually associated with closed systems such as in-line gates and valves, but it can occur locally in free-surface systems. Pressures in the cavitation range have been measured on a model of a navigation dam with a submergible tainter gate where the flow passages under the submerged gate had venturi-like characteristics. Similar flow conditions but with very high head losses can exist with lock culvert valves and with conduit gates operating under submerged conditions (plate C-15). In effect, cavitation can occur following any constriction when the back pressure in the system allows the jet flow piezometric head to approach the vapor pressure of water.

2-15. Theory.

a. General. Cavitation results from the sudden reduction of local pressure at any point to the vapor pressure of water. Such reductions in pressure are caused in water passages by abrupt changes in the boundary which causes a tendency of separation of the flow from the boundary, by constrictions which produce high velocities and low pressures, and by siphons in which pressures are reduced by reason of elevation. Vapor cavities form as bubbles in the low-pressure areas and collapse when a higher pressure area is reached a short distance downstream. The collapse ("implosion") is very rapid and sets up high-pressure shock waves or possibly small, high-velocity local "jets" in the water that cause damage to the nearby boundary. The basic equation associated with cavitation studies is

$$\sigma = \frac{\dfrac{(p_o - p_v)}{\gamma}}{\dfrac{V_o^2}{2g}} \tag{2-19}$$

where

σ = general dimensionless cavitation parameter

p_o = absolute pressure, lb/ft^2

V_o = average velocity of the flow

p_v = vapor pressure of the fluid at a particular temperature, lb/ft^2

γ = unit weight of the fluid

Abrupt boundary changes also cause large local fluctuations in pressures and velocities. Computation of these fluctuations is essentially impossible and cavitation potential can only be investigated under carefully controlled tests. In such tests a value of σ_i is determined for incipient cavitation by visual or specially instrumented observations. The value of σ_i applies only for the particular geometry tested. As long as σ's for other flow conditions exceed σ_i , cavitation is not expected to occur. The reader is referred to the book by Knapp, Daily and Hammitt (item 57) for additional discussion on the theory of cavitation.

b. Effects of Temperature. The vapor pressure of water (p_v or p_v/γ) varies somewhat with the temperature of the water. The vapor pressure of fresh water at 40°F is about 0.29 ft of water and at 70°F is 0.83 ft of water. The variation in vapor pressure is not large compared with the variation in atmospheric pressure due to elevation above sea level. For example, atmospheric pressure at sea level is 34 ft of water, whereas at Denver, Colorado (elevation 5332), atmospheric pressure is 28 ft of water (see HDC 000-2[n]). Thus, if the water temperature is 60°F, cavitation occurs at negative pressures of 33.4 and 27.4 ft of water at sea level and Denver, respectively.

2-16. Design Practice. Application of the theory of cavitation to practical design problems is difficult. Available design information on the magnitude of instantaneous pressure fluctuations is meager. In general, such fluctuations increase in magnitude with increasing total head. For this reason two minimum average pressure values are recommended for general design where the total head is less than 100 ft. These values are based on experience and should be conservative. Where boundary changes are gentle and streamlined, such as in entrances and transitions, minimum average local pressures as low as -20 ft of water can be expected to be cavitation-free. Where boundary changes are abrupt or the local flow is highly turbulent, such as at gate slots, offsets, and baffle piers of standard design, minimum average pressures should not be lower than -10 ft of water for safe design. In these highly turbulent cases, local instantaneous pressure fluctuations of ±10 ft of water or more can be expected. For higher heads, an average pressure exceeding 0 ft of water is often necessary as instantaneous pressure fluctuations can materially exceed atmospheric pressure.

2-17. Preventive Measures. Once pitting has started in an outlet conduit, the effect of cavitation may be accelerated by the existence of a depression or hole in the surface which intensifies the local turbulence and the negative pressures in the area just downstream from the depression. Thus, early repair of pitted surfaces is important and should be done preferably with a more resistant material. Stainless steel welding has been used to repair cavitation damage to steel surfaces such as gate frames and turbine blades. Successful repairs have been made to concrete surfaces with epoxy concrete or mortar. The cause of cavitation should be determined and corrected or avoided if due to a particular operating condition. The preventive measures to be taken in the design of outlet works conduits depend on particular conditions as follows:

a. Improvement of the shape of water passages to minimize the possibility of cavitation. Examples are the streamlining of conduit entrances, increasing the amount of offset and decreasing the rate of taper downstream of gate slots, and using larger bend radii.

b. Increasing the pressure by raising the hydraulic grade line at disturbance areas, which may be accomplished by flattening any downward curve, restricting the exit end of the conduit, or increasing the cross-sectional area in such localities as gate passages to decrease the velocity and increase the pressure.

c. Introducing air at low-pressure areas to partly alleviate negative pressure conditions and to provide air bubbles in the flow that will reduce the formation of cavitation pockets and cushion the effects of their collapse. In the design of high-head outlet conduits, it is often desirable to combine any two or all three of the above preventive measures. It is especially desirable to maintain a substantial back pressure in the vicinity of entrances, roof openings, bulkhead slots, and gate slots whenever the velocity is sufficiently high to produce cavitation. For long conduits, the pressure gradient will ordinarily produce the required back pressure, but for short conduits, gate passages frequently must be enlarged or exit constrictions provided to produce the back pressure. When conduits are to be operated at part-gate opening, special care should be taken to provide streamlined shapes at the aforementioned locations and downstream therefrom because back pressure will not be provided when the conduits flow partly full. The floor and walls of a conduit just downstream from a high-head gate are particularly vulnerable when operated at small openings for an extended period of time (items 93 and 136). It is especially important that during construction, small protrusions resulting from incorrect monolith alignment, concrete spills, unground welded joints, etc., not be permitted.

2-18. Boundary Layer. (Items 101 and 106.) Conduit systems are generally designed on the assumption that the boundary layer generated in the flow by the shear between the fluid and the boundary is fully developed and exists the entire length of the uniform conduit section. Tests at WES (item 129) and other places show, in fact, that conduit lengths of about 40 diameters are required for the boundary layer to become fully developed. A recent study reported by Wang (item 143) showed that for rough pipes, the wall shear stress became fully developed in about 15 diameters and the velocity profile was almost fully developed in 50 diameters for a Reynolds number range of 1.2×10^6 to 3.7×10^6. In sluices and conduits of very small length-diameter ratios, the exit portal flow can contain a central core having a velocity head approximating the full reservoir head. Energy dissipators for very short conduits should be designed using the total reservoir head.

2-19. Air Demand. Under certain conditions of operation, the pressure in a conduit may fall considerably below atmospheric pressure. Subatmospheric pressures, approaching the vapor pressure of water, may be accompanied by large fluctuations that can cause dangerous vibration or destructive cavitation, particularly in the gate section, and are therefore undesirable from the operating standpoint as well as for structural reasons. Large reductions in these pressure fluctuations can be effected by providing air vents through which air will flow into the conduit where less than atmospheric pressure exists. The vents usually open through the conduit roof immediately downstream from the service gate. (See para 3-17 for details.) Air requirements are most critical in this area and reach a maximum value when the service gate is operated at about three-quarters open under the highest head. It is particularly important that the air vent opening extend across the full width of the conduit, that the high-velocity air actually spreads across the full width, and that the water flow does not impinge into the opening. An illustrative example showing the methods used for determining the size of air vent required and for computing the pressure drop in such an air vent is presented in HDC 050-2.[n] The air discharge which must be supplied by air vents is dependent upon the rate of air entrained by high-velocity flow and upon the rate of air discharged above the air-water mixture at the conduit exit. Both factors are variable and are influenced by the hydraulic and structural features of the conduit and the method of conduit operation. Plate C-16 indicates the types of flow that cause air demand and the relative amounts. When conduit discharge is not influenced by tailwater conditions and a hydraulic jump does not form in the conduit, the jet issuing from a small gate opening forms a fine spray or mist that fills the conduit and is dragged along the conduit by the underlying high-velocity flow, finally producing a blast of air and spray from the exit portal. At large gate openings, a partial hydraulic jump is formed in the conduit and the jet will entrain air as previously cited; but the air inflow from the vent at the top of the conduit will be entrained by the turbulence of the jump and drawn by the jump action into the conduit flow downstream. Both conditions of water flow in the conduit result in reduced pressures at the back of the service gate and at the vent exit, thus causing air inflow through the vent. Air demand, in most instances, is not subject to a rigid analysis. Quantitative estimates of air requirements for design purposes have been based principally on empirical application of appropriate experimental and prototype data. A paper by A. A. Kalinske and J. M. Robertson (item 55) correlated experimental data obtained on the rate of air entrainment by the hydraulic jump as a function of Froude number. Data on the prototype has also been obtained. A summary of existing data is presented in plate C-17. Data presented by Sharma (item 111) indicate that the air demand for free flow and spray conditions may be about 3 and 6 times, respectively, that for the hydraulic jump condition.

2-20. Air Flow. Air vent flow encountered in the hydraulic design of outlet works is generally treated as an incompressible fluid and consequently conveyance systems are designed using conventional hydraulic theory and procedures. In extremely high-velocity systems (>200 fps) the air should be treated as a compressible fluid and the system designed accordingly. Scott (item 109) has prepared many flow charts for designing air conveyance systems.