37

Hydraulic Structures

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37.1 Introduction

This chapter covers the principles of the hydraulic design of the more usual hydraulic structures found in Civil Engineering practice. These include reservoirs, dams, spillways and outlet works, energy dissipation structures, open channel transitions, culverts, bridge constrictions, pipe systems, and pumps.

37.2 Reservoirs

Classification According To Use

Reservoirs are used for the storage of water, for the purpose of conservation for later use, for mitigation of flood damages, for balancing a time varying supply and a time varying demand (for water supply or generation of hydropower), for maintenance of downstream flow (for water quality preservation, navigation, fisheries or wildlife habitat) and for storing excess runoff in urbanized areas, and so on. If a reservoir is built for one purpose only, it is said to be a *single objective* reservoir; and if it is built to satisfy multiple purposes it is called a *multiobjective* reservoir.

For example a single purpose reservoir could be built for water supply and a multipurpose reservoir could be built for flood control and hydropower. In the latter example the two objectives yield contradictory requirements. For flood control it is desired to have the reservoir as empty as possible in order to have as much free capacity as possible for a forthcoming flood, whereas the hydropower generation requires a reservoir nearly full in order to have the maximum head over the turbines and thus generate the maximum amount of power possible. These conflicting objectives are reconciled by assigning certain portions of the reservoir to the respective objectives. These proportions can vary throughout the year. For example, there are rainy seasons when the danger of flood is high and the reservoir portion assigned to flood control should be larger than during drought seasons. These time-varying storage assignments and the constraints on the amounts of water that can be released downstream at a certain time form the operating rules of the reservoir.

Reservoir Characteristics

The capacity-elevation and the area-elevation relationships are fundamental. Figure 37.1 shows the area and capacity curves for Lake Mead obtained from two surveys in 1935 and in 1963–1964. The decrease in capacity between the two surveys is due to the accumulation of sediment. The minimum pool level corresponds to the invert of the lowest outlet or sluiceway (Fig.37.2). The volume below this level is called **dead storage** and can be used for accumulation of sediments. The level corresponding to the crest of the spillway is the normal pool level and the volume difference between the normal pool level and the minimum pool level is the useful storage. When there is overflow over the spillway the additional storage above the normal pool is the **surcharge storage**. The discharge that can be guaranteed during the most critical dry period of record is the safe yield. Any additional release is the secondary yield.

For flood routing in reservoirs (see Chapter 31, "Surface Water Hydrology" for details) the water surface in reservoirs is generally assumed to be horizontal. However, this is not the case when the reservoir is narrow and long like a wide river. In such cases the water surface has a gradient that increases with

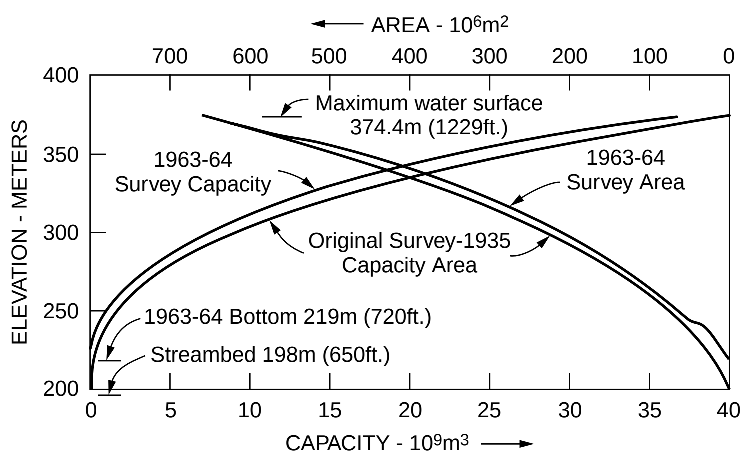

FIGURE 37.1 Area and capacity curves for Lake Mead. (*Source:* U.S Department of the Interior, Bureau of Reclamation. (*Design of Small Dams*. 1987. p. 531.U.S. Government Printing Office, Denver, CO.)

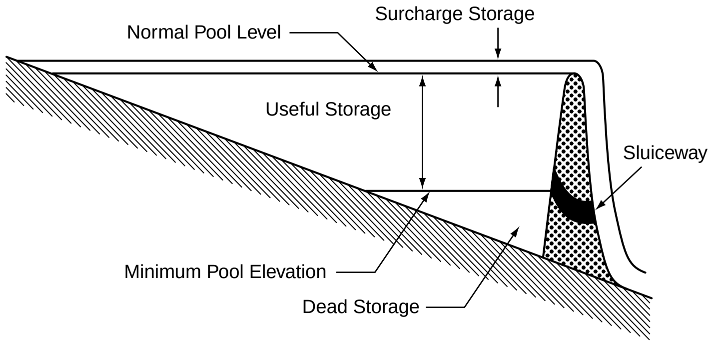

FIGURE 37.2 Storage pools of a typical reservoir. (*Source:* Martin, J.L. and Mc Cutcheon, S.C. *Hydrodynamics and Transport for Water Quality,* Lewis 1999, Fig. 2 p. 340.)

increasing discharge. Techniques for the calculation of water surface profiles under steady and time varying flows are discussed in Chapter 30, "Open Channel Hydraulics".

Capacity of a Reservoir

The simplest method of estimating the capacity of a reservoir to meet a specified demand uses the Rippl diagram or mass curve. The procedure involves the determination of the largest cumulative difference between a sequence of specified demands and a sequence of reservoir inflows. Series of historic or simulated inflows, Q_t, at a selected time interval are used. Monthly intervals are common for large reservoirs. The cumulative inflow in the reservoir is plotted as a function of time. This curve exhibits segments with steep upward slopes during periods of large inflow in the reservoir and segments with relatively flat slopes during periods of low inflows or droughts. This inflow is generally adjusted to account for the evaporation and seepage losses and required releases downstream. When the losses plus the downstream requirements exceed the inflow, the curve shows periods of decreasing cumulative adjusted inflow. After a sharp rise in the curve it may exhibit a sharp decrease in slope. When such changes occur, the reservoir is usually full. At these several bends in the adjusted cumulative inflow, lines are traced with a common slope equal to the demand, D_t, in volume per unit of time. These accumulated demand lines will be above the accumulated adjusted inflow curve for some periods of time (starting at time i and ending at time j). For such periods, the largest positive departure between the accumulated demand line and the cumulative adjusted inflow curve over a time horizon T represents the required storage of the reservoir, S. The procedure is generally performed graphically as shown in Fig. 37.3. Mathematically, the required storage is given by

$$S = \max_{T} \left[\sum_{t=i}^{j} \left(D_t - Q_t \right) \right] \tag{37.1}$$

When long data series are analyzed, or, if the demand varies, the computationally efficient sequent peak algorithm of Thomas and Fiering (1963) is preferred. The accumulated differences between inflows and demand are plotted as a function of time. The curve exhibits a sequence of peaks and troughs. The first peak and the next higher peak, called the sequent peak, are located. The difference between the first peak and the lowest subsequent valley represents the storage for the period. The next sequent peak is identified and the required storage after the first sequent peak is found. The process is repeated for the whole study period and the maximum storage is identified. Mathematically, the required storage capacity, S_t, at the beginning of period t is calculated from

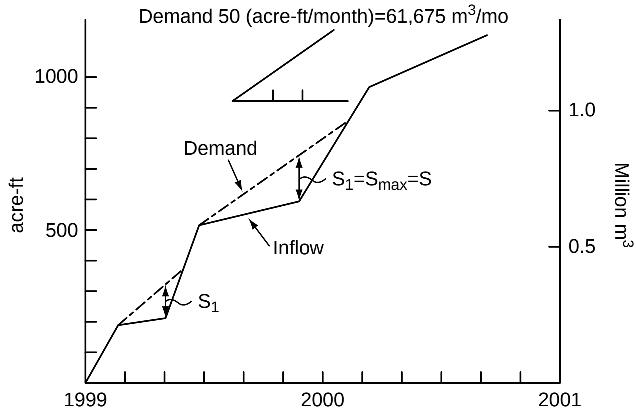

FIGURE 37.3 Estimation of reservoir capacity by mass curve method. 1 acre-ft = 1,233.5 m.[3]

$$S_t = \begin{cases} D_t - Q_t + S_{t-1} & \cdots if \quad D_t - Q_t + S_{t-1} > 0 \\ 0 & \cdots if \quad D_t - Q_t + S_{t-1} \le 0 \end{cases} \tag{37.2}$$

where D_t and Q_t are the demand and the inflow during the interval t.

Starting with $S_0 = 0$, consecutive values of S_t are calculated. The maximum value of S_t is the required storage capacity for the specified demand D_t.

Reservoir Sedimentation

The accumulation of sediments limits the useful life of a reservoir. The larger particles of sediments carried by the streams leading to reservoirs are deposited at the head of the reservoirs forming deltas. The smaller particles are carried in density currents and eventually are deposited in the lower parts of the reservoir near the dam. Methods of determining the suspended sediment carried by streams are discussed in Chapter 35 "Sediment Transport in Open Channels." Using these methods, theoretical predictions of the development of deltas over time can be made. One such approach can be found in Garcia (1999, p. 6.89–6.97). Instead, an empirical approach developed from observations of existing reservoirs is presented here. It can be used in a preliminary analysis.

The results of sediment samplings in streams can often be summarized by rating curves of the type

$$Q_s = aQ_w^b \tag{37.3}$$

where Q_s = the suspended transport (tons per day)
Q_w = the flow (cfs or cms)
a and b = coefficients

The trap efficiency of reservoirs is a function of the ratio of reservoir capacity to annual inflow volume. Envelopes and medium trap efficiency curves have been given by Brune (1953). The medium curve is shown in Fig. 37.4. Also shown is a relationship obtained by Churchill (1949) for Tennessee Valley Authority Reservoirs. The number of years to fill a reservoir can be estimated from the sediment inflow and the trap efficiency. For this purpose successive decreasing capacities of the reservoir are considered and the respective capacity-inflow ratios are calculated along with the trap efficiencies. By dividing the increments in reservoir capacity by the volume of sediment trapped, the life of each increment is calculated. These are then summed to obtain the expected life of the reservoir. Further discussion on Reservoir Sedimentation can be found in Appendix A of *Design of Small Dams*, U.S. Department of the Interior, Bureau of Reclamation (1987) and in Morris and Fan (1988).

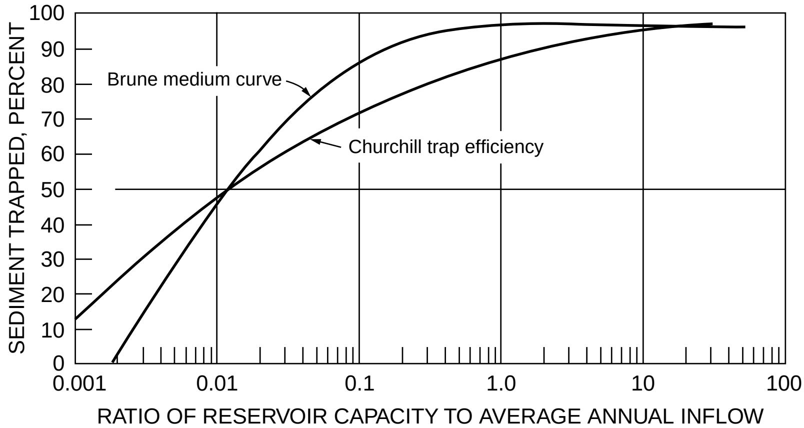

FIGURE 37.4 Trap efficiency curves. (Adapted from U.S Department of the Interior, Bureau of Reclamation, (1987) Design of Small Dams p. 542, Fig A.9.)

Many dams that have outlived their usefulness are decommissioned. Decommissioning is the complete or partial removal of a dam or substantial change in its operation. A primary issue to be considered in the removal of a dam is the disposal of the sediments that have accumulated in the reservoir. ASCE (1997) provides guidelines for retirement of dams and hydroelectric facilities.

Impacts of Dams and Reservoirs

Impacts due to the existence of a dam, and reservoir include: changes in upstream and downstream river bed morphology due to changed sediment load, changes in downstream water quality due to change in released water temperature and in constituent concentrations in the impounded water and a reduction in biodiversity due to decrease of freedom of movement of fish and organisms.

A reservoir induces a reduction in the upstream sediment transport capacity due to the decreased water velocity. Downstream of the reservoir, the trap efficiency of the pool reduces the sediment load below the transport capacity of the stream. As a result aggradation tends to occur upstream of the reservoir and degradation tends to occurs downstream. These effects are discussed in detail, for example, in Simons and Sentürk (1992) and in Sentürk (1994).

Reservoir water quality is affected by surface heating and by the absence of complete vertical mixing, at least during part of the year. The net thermal flux at the reservoir surface, H_n, is the algebraic sum of the absorbed short wave solar radiation, H_{sw}, the long wave atmospheric radiation, H_H, the back radiation from the lake surface, H_B, the heat loss due to evaporation, H_L, and the net flux due to conduction or sensible heat transfer:

$$H_n = H_{sw} + H_H - H_B - H_L - H_S \tag{37.4}$$

Formulas for the calculation of each of these heat transfer components can be found, for example, in Martin and McCutcheon (1999).

In the summer time, large reservoirs (with a depth of at least 10 m and a mean residence time of more than 20 days) tend to stratify in three main layers as shown in Fig. 37.5. The epilimnion or upper layer of relatively warm water is well mixed as a result of wind and convection currents. Below it is the thermocline or metalimnion, the layer in which the temperature decrease with depth is greater than in the overlying and underlying layers. At the bottom is the hypolimnion, characterized by a temperature that is generally cooler than the other strata of the reservoir. A criterion to determine the stratification potential is the densimetric Froude number, F_d. It is the ratio of the inertia forces of the horizontal flow to the gravity forces within the stratified impoundment and is a measure of potential of the horizontal flow to alter the thermal density structure of the reservoir from its gravitational equilibrium. It is given by

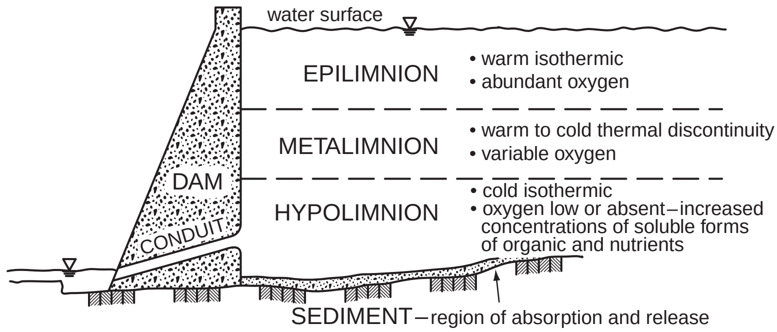

FIGURE 37.5 Typical summer stratification. (*Source:* Martin, J.L. and Mc Cutcheon, S.C., *Hydrodynamics and Transport for Water Quality,* Lewis 1999, Fig. 5, p. 343, taken from U.S. Army Engineers Waterways Experiment Station, Vicksburg, MS.)

$$F_d = \frac{LQ}{hV}\sqrt{\frac{\rho_o}{g\left(\Delta\rho/h\right)}} = \frac{U}{\sqrt{gh\left(\Delta\rho/\rho_o\right)}} \qquad (37.5)$$

where L = the reservoir length (unit L)
Q = the average flow through the reservoir ($L^3\ T^{-1}$)
h = the mean depth (L)
V = the reservoir volume (L^3)
g = the acceleration of gravity (LT^{-2})
ρ_o = the reference density (ML^{-3})
$\Delta\rho$ = the density difference over depth h (ML^{-3})
$\Delta\rho/h$ = the average density gradient (ML^{-4})
U = the average flow velocity (LT^{-1})

The average density gradient typically is of the order of 10^{-3} kg m^{-4} and the water density is 1000 kg m^{-3}. The reservoir is well mixed if $F_d \gg 1/\pi$, it is strongly stratified if $F_d \ll 1/\pi$, and it is weakly stratified or stratified off and on if $F_d \cong 1/\pi$. (Orlob, 1983). The stratification varies seasonally as shown in Fig. 37.6.

The density of tributary inflow usually differs from that of the surface water of the reservoir. Therefore, currents of water with slightly different densities, called density currents, are created. Depending upon the sign and magnitude of this density difference, the density currents can enter the epilimnion, the metalimnion or the hypolimnion, as shown in Fig. 37.7. When the inflow density is less than the surface water density, the inflow will float over the surface water. This condition often occurs in the spring when the inflows are warmer than the water in the reservoirs. If the inflow is cooler or carries larger concentrations of dissolved or particulate materials, then its density will be larger that that of the reservoir surface water. The inflow will then push the reservoir water ahead until the (negative) buoyancy force dominates and the inflow plunges beneath the surface as shown in Fig. 37.8. The plunge point can often be observed because of the accumulation of floating debris. After the flow plunges it follows the river channel as an underflow. This condition typically occurs in the fall when riverine waters are cooler than reservoir waters. If the underflow is not as dense as the bottommost layer in the reservoir, the underflow will separate from the bottom to form an interflow or intrusion as shown in Fig. 37.7. When cool water plunges as underflow or interflow it entrains some of the reservoir water creating a circulation pattern in the overlying surface waters. These reverse flows transport floating debris towards the plunge point where they remain. For further discussion on density currents in reservoirs see Alavian at al. (1992) and Martin and McCutcheon (1999).

Selective withdrawal release structures with a series of outlets at different elevations, as shown schematically in Fig. 37.9, can be used to select the layer(s) from which the water is released. This makes it possible to improve water quality within a reservoir and to reduce adverse impacts that temperature and

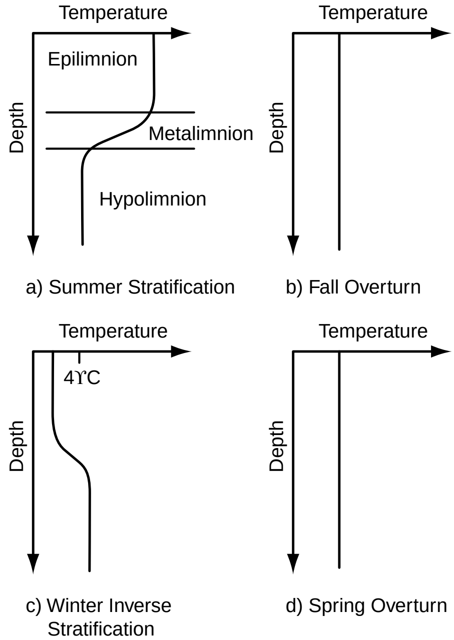

FIGURE 37.6 Idealized stratification cycle. (*Source*: Martin, J.L. and Mc Cutcheon, S.C., *Hydrodynamics and Transport for Water Quality,* Lewis 1999, Fig.7, p. 345.)

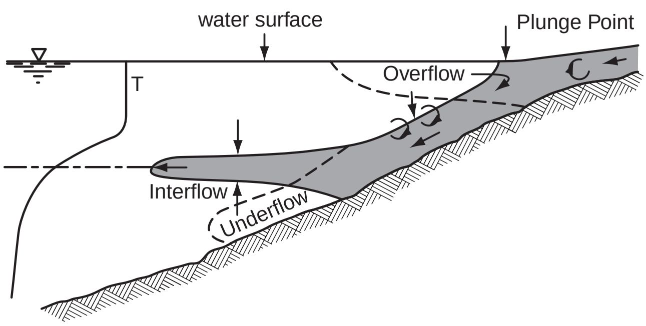

FIGURE 37.7 Density inflows to reservoirs. (*Source*: Ford, D.E. and Johnson, M.C. 1986, *An assessment of reservoir density currents and inflow processes,* U.S. Army Engineers Waterways Experiment Station, tech. Rept. 86–7, Vicksburg, MS.)

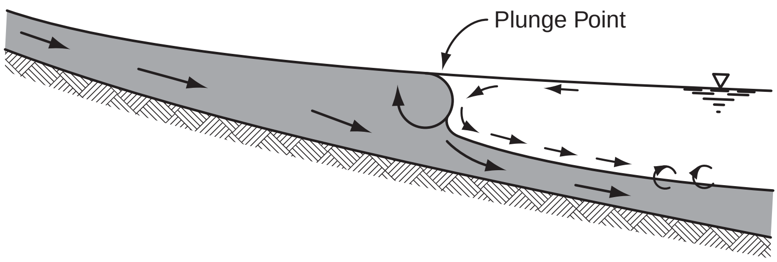

FIGURE 37.8 Plunge point, (*Source:* Ford, D.E. and Johnson, M.C. 1986, *An assessment of reservoir density currents and inflow processes,* U.S. Army Engineers Waterways Experiment Station, Tech. Rept. E-86–7, Vicksburg, MS.)

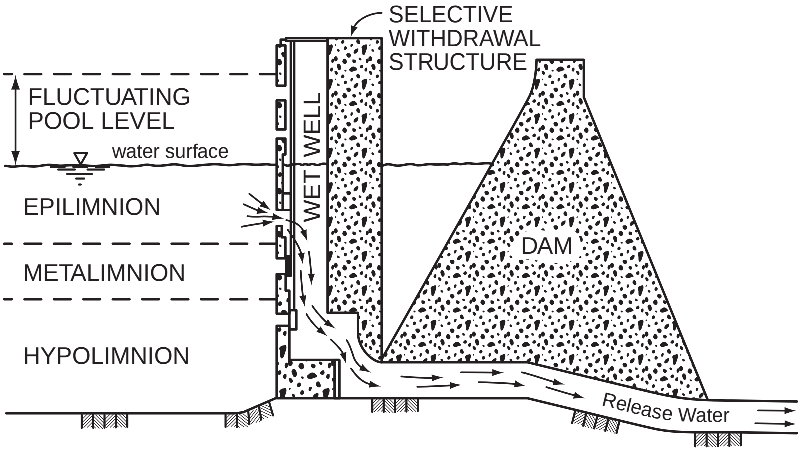

FIGURE 37.9 Selective withdrawal release structure. (*Source:* Martin, J.L. and Mc Cutcheon, S.C., *Hydrodynamics and Transport for Water Quality,* Lewis 1999, Fig. 3, p.341.)

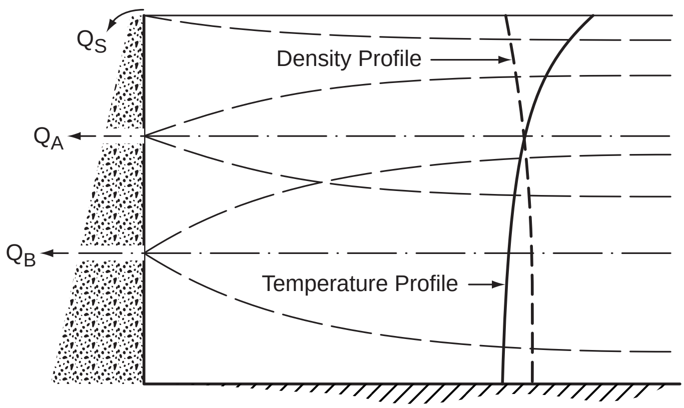

FIGURE 37.10 Schematic pattern of selective withdrawal to a stratified impoundment. (Source: Norton, W.B., Roesner, L.A. and Orlob, G.T. 1968, *Mathematical models for predicting thermal changes in impoundments,* EPA, Water Pollution Research Series, U.S.E.P.A., Washington, D.C.)

other water quality constituents may have on downstream water quality. Figure 37.10 shows a schematic withdrawal pattern. The U.S. Army Corps of Engineers has developed a computer program called SELECT to predict the vertical extent and distribution of withdrawal from a reservoir of known density and quality distribution for a given discharge from a specified location. Using this prediction for the withdrawal zone, SELECT computes quality parameters of the release such as temperature, dissolved oxygen, turbidity and iron (U.S. Army Engineer Waterways Experiment Station, 1992).

In winter ice begins to form when water is cooled to 0°C. When the air temperature T_a (°C) is lower than the melting point of ice, T_m (0°C), there is a thickening of the ice cover. The ice thickness h_i (m) after a time increment t (s) of freezing can be roughly estimated from

$$h_i \approx \alpha\left[\left(T_m - T_a\right)t\right]^{0.5} \tag{37.6}$$

where $\alpha = (2k_i/\rho_i\ \lambda_i)^{0.5}$ where k_i is the thermal conductivity of ice (2.24 W m^{-1} °C^{-1}), ρ_i is the ice density (916 kg m^{-3}), and λ_i is the latent heat of fusion of the ice (3.34×10^5 J kg^{-1}), (Ashton, 1982).

37.3 Dams

Classification and Physical Factors Governing Selection

Dams are generally classified according to the material used in the structure, and the basic type of design. Concrete gravity dams depend on their weight for their stability, concrete arch dams transfer the hydrostatic forces to their abutments by arch action, and in concrete buttress dams the hydrostatic force is resisted by a slab that transmits the load to buttresses perpendicular to the dam axis. Embankment dams can be subdivided into earth-fill dams and rock-fill dams.

The principal physical factors governing the choice of dam type are: the topography, the geology, the availability of materials and the hydrology. The topography usually governs the basic choice of dam. Low rolling plains would suggest an earth-fill dam, whereas a narrow valley with high rock walls would suggest a concrete structure or a rock-fill dam. Saddles in the periphery of the reservoir may provide ideal locations for spillways, especially emergency spillways. The foundation geology is also of major significance in the selection of the dam type. Good rock foundations are excellent for all types of dams, while gravel foundations are suitable for earth-fill and rock-fill dams. Silt or fine sand foundations are not suitable for rock-fill dams but can be used for earth-fill dams with flat slopes. The availability of materials (soils and rock for the embankments, riprap and concrete aggregate near the dam site) weigh heavily in the economic considerations. The hydrologic condition of stream flow characteristics and rainfall will influence the method of diversion of the flows during construction and the construction time. Finally, if the dam is located in a seismic-prone area the horizontal and the vertical components of the earthquake acceleration on the dam structure and on the impounded water must be considered in the analysis of the dam stability.

Stability of Gravity Dams

The principal forces to be considered are the weight of the dam, the hydrostatic force, the uplift force, the earthquake forces, the ice force and the silt force. The gravity force is equal to the weight of concrete, V_C plus the weight of such appurtenances as gates and bridges. This force passes through the center of gravity of the dam. The horizontal component of the hydrostatic force per unit width of the dam is (see Chapter 29, Fundamentals of Hydraulics, Application 29.1) is

$$H_w = \gamma h^2/2 \tag{37.7}$$

where γ = the specific weight of the water
h = the depth of water at the vertical projection of the upstream face of the dam

This force acts at a distance $h/3$ above the base of the dam (Fig. 37.11). The vertical component of the hydrostatic force, V_w, is equal to the weight of water vertically above the upstream face of the dam. This force passes through the center of gravity of this wedge of water. There also can be horizontal and vertical hydrostatic forces, H'_w and V'_w, respectively, on the downstream face of the dam. Water may eventually seep between the dam masonry and its foundation creating an **uplift pressure**. The most conservative design assumes that the uplift pressure varies linearly from a full hydrostatic pressure at the upstream face to the full tailwater pressure at the downstream face. The uplift force is then

$$U = \gamma\left[\left(h + h'\right)/2\right]T \tag{37.8}$$

where h and h' = the water depths at the upstream and downstream faces, respectively
T = the thickness or width of the dam base

The uplift force acts vertically upward through the center of gravity of the trapezoidal uplift pressure diagram. If there is a drain located at about 25% of the base width from the heel of the dam, the uplift

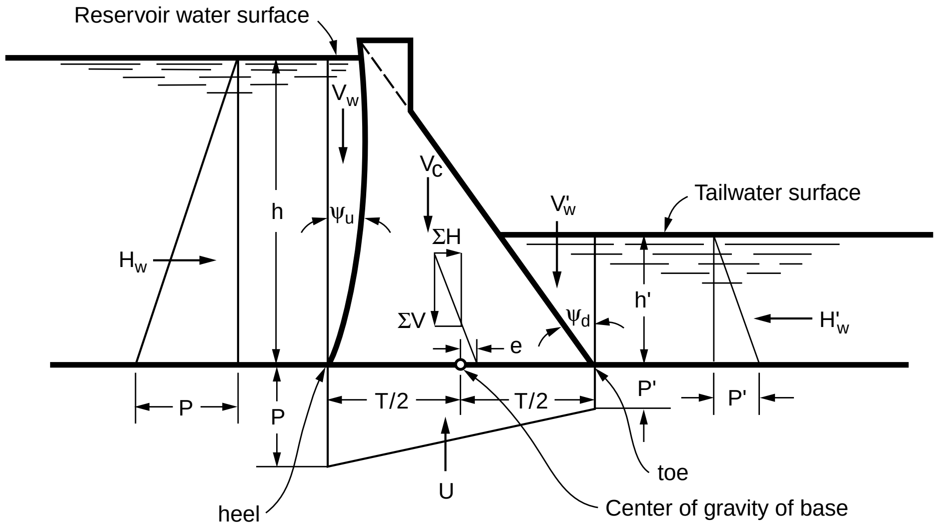

FIGURE 37.11 Forces acting on a concrete gravity dam. (Adapted from U.S. Department of the Interior, Bureau of Reclamation, *Design of Small Dams*, p.317, U.S. Government printing Office, Denver, CO.)

pressure can be assumed to decrease to 2/3 of the full upstream hydrostatic pressure at the drain and then to zero at the toe, assuming no downstream hydrostatic pressure. Field measurements of uplift pressures are quoted in Yeh and Abdel-Malek (1993). Suggested values of the earthquake accelerations in terms of the distance from the source of energy release and of the Richter scale magnitude as well as information on the increase in water pressure can be found in US Department of the Interior, Bureau of Reclamation, (1987). According to the same source an acceptable criterion for ice force is 10,000 lb/ft of contact between the dam and the ice for an assumed ice depth (see Eq. [37.6]) of 2 ft or more. An acceptable criterion for saturated silt pressure is to use an equivalent fluid with a specific weight of 85 lb/ft^3 for the horizontal component and 120 lb/ft^3 for the vertical component.

The stability analysis includes the following steps:

- Calculate the resultant vertical force above base of section, ΣV,
- Calculate the resultant horizontal or sliding force above base of section, ΣH,
- Calculate the overturning moment (clockwise in Fig. 37.11) about the toe of the section, ΣM_o,
- Calculate the stabilizing moment (counterclockwise in Fig. 37.11) about the toe, ΣM_s,
- Calculate the safety factor against overturning, $FS_o = \Sigma M_s/\Sigma M_o$,
- Calculate the available friction force $F_f = \mu \Sigma V$, where μ is the friction coefficient,
- Calculate the safety factor against sliding $FS_s = F_f/\Sigma H$,
- Calculate the distance from the toe to the point where the resultant of the vertical forces cuts the base $x = [M_s - \Sigma M_o]/\Sigma V$,
- Calculate the eccentricity of the load $e = T/2 - x$,
- Calculate the normal stress $\sigma = \Sigma V/A \pm Mc/I$ where c is the distance from the center of the section to its edge, I is the moment of inertia of the section about its centroidal axis and $M = e\Sigma V$ is the moment due to the eccentricity of the load,
- Calculate the stress parallel to the face of the dam $\sigma' = \sigma/cos^2\psi_u$,
- Verify that $FS_o > 2$,
- Verify that $FS_s > 1.0$,
- Verify that the stresses are within specifications.

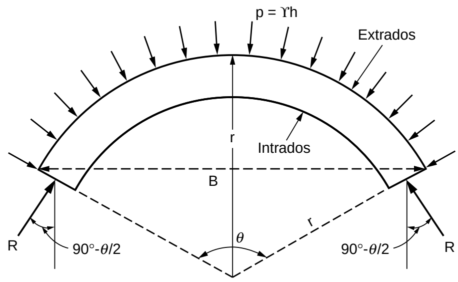

FIGURE 37.12 Hydrostatic pressure and resulting thrust on an arch rib.

Arch Dams

Arch dams are curved in plan so that they transmit part of the water pressure to the canyon walls of the valleys in which they are built. The arch action requires a unified monolithic concrete structure. Arch dams are classified as thin if the ratio of their base thickness to their structural height is less than 0.2 and thick if that ratio is larger than 0.3. The upstream side of an arch dam is called the **extrados** and the downstream side is the **intrados** (Fig. 37.12).

The structural analysis of arch dams assumes that it can be considered as a series of horizontal arch ribs and a series of vertical cantilevers. The load is distributed among these two actions in such a way that the arch and cantilever deflections are equal. This analysis is a specialized subject of structural engineering. The US Department of the Interior, Bureau of Reclamation (1977) has published an extensive book on the subject. Only the simplified cylinder theory is summarized below.

The forces acting on an arch dam are the same as on a gravity dam, but their relative importance is not the same. The uplift is less important because of the comparatively narrow base and the ice pressure is more important because of the large cantilever action. If the arch has a radius r and a central angle θ, the horizontal hydrostatic force due to a head h is $H_h = \gamma\, h\, 2r \sin(\theta/2)$. (See Chapter 29 on Fundamentals of Hydraulics, Application 29.2). This force is balanced by the abutment reaction in the upstream direction $R_y = 2R\ sin\ (\theta/2)$. By equating the two forces the abutment reaction (Fig. 37.12) becomes $R = \gamma\, hr$. If the working stress of the concrete is σ_w, the thickness of the arch rib is $t = \gamma\, hr/\sigma_w$. It is seen that with this simplified theory the thickness increases linearly with the depth. The volume of a single arch rib with a cross section A is $V = rA\theta$, where θ is in radians. Because of the relationship between the thickness t and the radius, the angle that minimizes this volume of concrete can be shown to be $\theta = 133^o34'$. For this angle the radius r of the arch in a valley of width B is $r = (B/2)\sin(66^o47') = 0.544\, B$. One could select an arch of constant radius with an average angle around 133°34′. The angle would be larger at the top and smaller at the base. Alternatively one could select a fixed angle and determine the valley width B at various depths and calculate the radius r required and then the necessary thickness t. A compromise between these two cases consists in keeping the radius fixed for a few sections and varied in others.

Earth Dams

Rolled-fill earth dams are constructed in lifts of earth having the proper moisture content. Each lift is thoroughly compacted and bonded to the preceding layer by power rollers of proper design and weight. Rolled-fill dams are of three types: homogeneous, zoned and diaphragm. Early earth dams were homogeneous simple embankments as are many levees today. Most earth dams are zoned embankments. They can have an impermeable core made out of clay or a combination of clay, sand and fine gravel. This core can be flanked on the upstream and downstream sides by more pervious zones or shells. These zones support and protect the impervious core. The upstream zone provides stability against rapid drawdown while the downstream zone controls the seepage and the position of the lower phreatic surface. In addition there can be filters between the impervious zone and the downstream shell and a drainage layer below

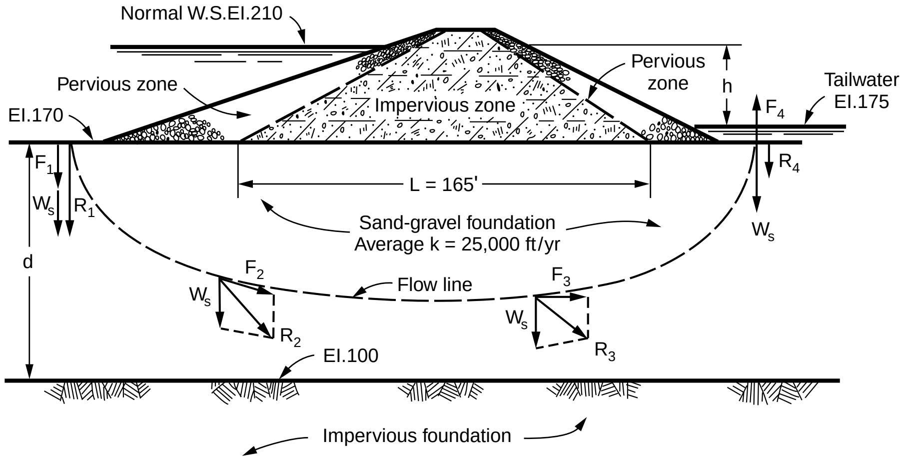

FIGURE 37.13 Seepage under an earth dam. (Adapted from U.S. Department of the Interior, Bureau of Reclamation, 1987, *Design of Small Dams*, p. 204, 205, U.S. Government Printing Office, Denver, CO.)

the downstream shell. Diaphragm-type dams are generally built on pervious foundations. Their impervious core is extended downward by a cutoff wall generally made of concrete. This wall is often accompanied by a horizontal impervious clay blanket under the base of the upstream face and extending further in the upstream direction. This lengthens the seepage path and reduces its quantity. The Bureau of Reclamation does not recommend this type of design for small dams because of potential cracking of the concrete wall due to differential movement induced by consolidation of the embankment.

The amount of seepage under a dam can be approximated roughly using Darcy's formula (see Chapter 34, Groundwater Engineering and Chapter 18 Groundwater and Seepage), $Q = K\,Ah/L$, where K is the hydraulic conductivity of the foundation material, A is the gross cross sectional area of the foundation through which the flow takes place and h/L is the hydraulic gradient, the difference of head divided by the length of the flow path. For the conditions shown in Fig. 37.13 with a hydraulic conductivity K of 25,000 ft/yr = 0.00079 ft/s, a head differential $h = 210 - 175 = 35$ ft over a distance $L = 165$ ft resulting in a hydraulic gradient $h/L = 35/165 = 0.212$ ft/ft, and a flow cross section area of $(170 - 100) \times 1 = 70$ sq ft per ft of width, the discharge per unit width is $Q = (0.00079)\ (0.212)\ (70) = 0.012$ cfs.

The flow through the foundation produces seepage forces due to the friction between the water and the pores of the foundation material. These forces on soil segments are labeled F_1, F_2 in Fig. 37.13 and W_s is the submerged weight of the soil segment. As the flow section is restricted, the flow velocity and the friction forces increase so that F_2 and F_3 are larger than F_1 and F_4. As the water percolates upward at the toe of the dam the seepage force tends to lift the soil. If F_4 is larger than W_s the soil could be "piped out." This is referred to as a piping failure.

The amount of seepage through an earth dam and its foundation, if the latter is pervious, can be estimated from a flow net. This is a network of streamlines (see Chapter 29, Fundamentals of Hydraulics, subsection on Description Fluid Flow) that are everywhere tangent to the flow velocity vector and the equipotential lines that are normal to the streamlines and are lines of equal pressure head. The network of lines form figures that tend to be squares when the number of lines becomes large. The streamlines are drawn in such a way that the amount of flow between consecutive streamlines is the same. The energy or head drop between consecutive equipotential lines is also the same. The exterior streamline along which the pressure is atmospheric is the phreatic line. It can be approximated by a parabola (Morris and Wiggert, 1972, p. 243). The amount of flow through a unit width of dam is $q = N'Kh/N$ where N is the number of equipotential drops, N' is the number of flow paths, i.e., the number of spaces between streamlines, and K is the hydraulic conductivity of the material. (For further details on flow nets, see Chapter 18, Groundwater and Seepage).

A simple method of stability analysis for small earth dams assumes that the surface of failure is cylindrical. The slide mass above an arbitrary slip-circle is divided into a number of vertical slices. The safety factor is equal to the ratio of the sum of the stabilizing moments to the sum of the destabilizing moments of the several slices about the center of the slip circle (for details see Chapter 21, Stability of Slopes).

The upstream face of earth dams must be protected against erosion and wave action. This can be achieved by covering the upstream face with riprap or a concrete slab. Both should be placed over a filter of graded material and the slab should have drainage weep holes. Likewise the downstream face should also be protected against erosion using grass or soil cement. Geotextiles are porous synthetic fabrics that do not degrade. They can be used for separation of materials in a zoned embankment, to prevent the migration of fines and to relieve pore pressure. Geomembranes are impervious and are used to reduce seepage (see Chapter 24, Geosynthetics). Additional information on earthfill and rockfill dams can be found, for example, in U.S. Department of the Interior, Bureau of Reclamation, (1987) and ASCE (1999).

Although flow overtopping an embankment is considered unacceptable, embankment protection consisting of specially designed concrete blocks has been tested for the Bureau of Reclamation (Frizell et al. 1994).

37.4 Spillways

Spillway Design Flood

Spillways are structures that release the excess flood water that cannot be contained in the allotted storage. In contrast *outlet works* regulate the release of water impounded by dams. As earth-fill and rock-fill dams are likely to be destroyed if overtopped, it is imperative that the spillways designed for these dams have adequate capacity to prevent overtopping of the embankment. For dams in the high hazard category, (i.e., those higher than 40 ft., with an impoundment of more than 10,000 acre-feet and whose failure would involve the loss of life or damages of disastrous proportions) the design flow is based on the probable maximum precipitation (PMP). Based on the PMP, the flood hydrograph is estimated and routed through the reservoir assumed to be full to obtain the spillway design flood. (See Chapter 31, Surface Water Hydrology for details on hydrograph estimation and flood routing).

The U.S. National Weather Service has developed generalized PMP charts for the region east of the 105^{th} meridian (Schreiner and Reidel, 1978) and the National Academy of Sciences (1983) has published a map that indicates the appropriate NWS hydrometeorological reports (HMR) for the region west of the 105^{th} meridian. (HMR 38 for California, HMR 43 for Northwest States, HMR 49 for Colorado River and Great Basin Drainage). The U.S. Army Corps of Engineers (1982) has issued hydrologic evaluation guidelines with recommended spillway design floods for different sizes of dams and hazard categories. For minor structures, inflow design floods with return periods of 50 to 200 years may be used if permitted by the responsible control agency (Hawk, 1992). Economic risk analysis is another approach in which the safety level and the design flood magnitude are determined simultaneously (Afshar and Mariño, 1990). Recent studies have indicated a tendency towards a modification of the policy requiring dams to accommodate the full probable maximum flood (Graham, 2000). For the UK, additions to the 1975 Flood Studies were reported by Reed and Field (1992). They also summarize procedures in nine other countries.

It is often economical to have two spillways: a *service spillway* designed for frequently occurring outflows and an *emergency spillway* for extreme event floods. Modifications of dams to accommodate major floods have been reviewed by USCOLD (1992).

Overflow Spillways

The overflow spillway has an ogee-shaped profile that closely conforms to the lower **nappe** or sheet of water falling from a ventilated sharp crested weir (See Chapter 29, Fundamentals of Hydraulics). Thus,

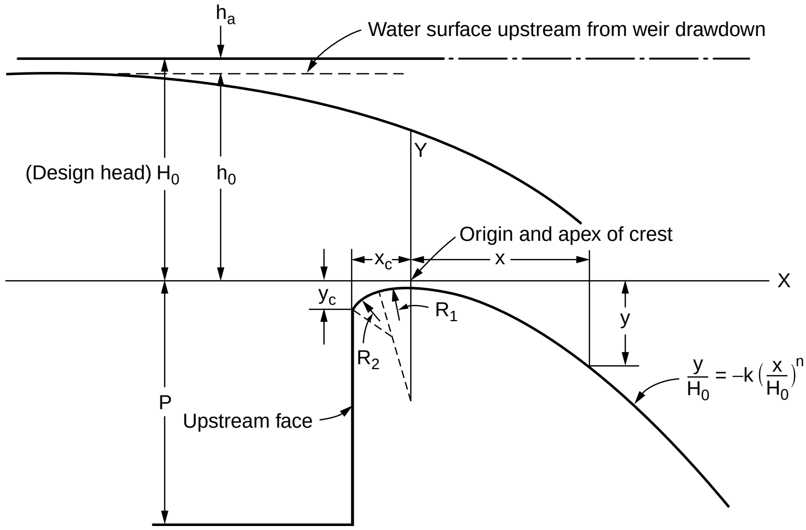

FIGURE 37.14 Ogee shaped overflow spillway profile. (Adapted from U.S. Department of the Interior, Bureau of Reclamation, *Design of Small Dams*, p. 366, U.S. Government Printing Office, Denver, CO.)

for flow at the design discharge, the water glides over the spillway crest with almost no interference from the boundary. Below the ogee curve the profile follows a tangent with the slope required for structural stability (see "Stability of Gravity Dams" earlier in this chapter). At the bottom of this tangent there is a reverse curve that turns the flow onto the apron of a stilling basin or into a discharge channel. The shape of the ogee is shown in Fig. 37.14 and can be expressed as

$$y/H_o = -K\left(x/H_o\right)^n \tag{37.9}$$

where x and y = the horizontal and vertical distances from the crest
H_o = the design head including the velocity head of approach h_a
K and n = coefficients that depend upon the slope of the upstream face of the spillway and h_a

For a vertical face and a negligible velocity of approach $K = 0.5$ and $n = 1.85$. Other values can be found in U.S. Department of the Interior, Bureau of Reclamation (1987). The discharge over an overflow spillway is given by (see Chapter 29, Fundamentals of Hydraulics, Application 29.11)

$$Q = C\,L\,H_e^{3/2} \tag{37.10}$$

where Q = the discharge
C = the discharge coefficient (units $L^{1/2}\ T^{-1}$)
H_e = the actual head over the weir including the velocity head of approach h_a
L = the effective length of the crest

The basic discharge coefficient for a vertical faced ogee crest is designated by C_o and is shown in Fig. 37.15 as a function of the design head $H_{o.}$ For a head H_e other than the design head, H_0, for an ogee with sloping upstream face and for the effect of the downstream flow conditions the U.S. Department of the Interior, Bureau of Reclamation (1987) gives correction factors to be applied to C_o to obtain the discharge coefficient C. When the flow over the crest is contracted by abutments or piers the effective length of the crest is

$$L = L' - 2\left(N\,K_p + K_a\right)H_e \tag{37.11}$$

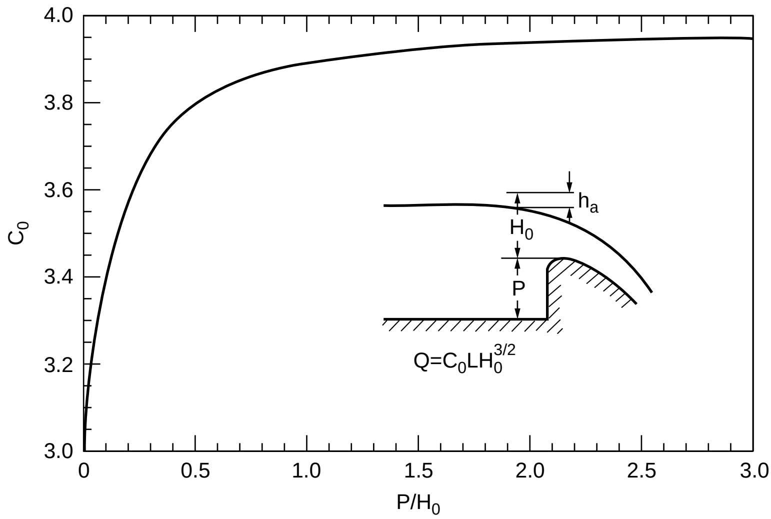

FIGURE 37.15 Discharge coefficient for Q in ft^3/s, L and H_o in feet for vertical faced ogee crest. (Adapted from U.S. Department of the Interior, Bureau of Reclamation, 1987, *Design of Small Dams*, p. 370, U.S. Government Printing Office, Denver, CO.)

where L' = the net crest length
N = the number of piers
K_p = the pier contraction coefficient with a value of 0.02 for square nosed piers with rounded corners (radius = 0.1 pier thickness), of 0.01 for rounded nose piers and of 0.0 for pointed nose piers
K_a = abutment contraction coefficient

K_a has a value of 0.20 for square abutments with headwall perpendicular to the flow and 0.10 for rounded abutment ($0.5\ H_o \leq r \leq 0.15\ H_o$) with headwalls perpendicular to the flow. For additional details concerning the hydraulics of spillways, see, for example, ASCE (1995) or Sentürk (1994) or Coleman et al. (1999).

Other Types of Spillways

In straight drop or free overfall spillway the flow drops freely from the crest, which has a nearly vertical face. The underside of the nappe must be ventilated. Scour will occur at the base of the overfall if no protection is provided. A hydraulic jump will form if the overfall jet impinges upon a flat apron with sufficient tailwater depth.

Chute spillways convey the discharge through an open channel placed along dam abutments or through saddles in the reservoir peripheries. They are often used in conjunction with earth dams. Generally, upstream of the crest the flows are subcritical, passing through critical at the control section and accelerating at supercritical velocity until the terminal structure is reached. Because of the high flow velocity all vertical curves and changes in alignment must be very gradual. Concrete floor slabs are provided with expansion joints that must be kept watertight and drains are provided at intervals under the slab to prevent piping.

Side channel spillways are placed parallel to and along the upper reaches of the discharge channels. Thus flows pass over the crest and then turn approximately 90°to run into the parallel discharge channel. This layout is advantageous for narrow canyons where there is not sufficient space to accommodate an overflow or a chute spillway. Flows from the side channel can be directed to an open channel as shown in Fig. 37.16 or to a closed conduit or to a tunnel leading to the terminal structure. Discharge in the side channel increases in the downstream direction. Details of the analysis of such spatially varying flows are treated, for example, in Chow (1959), French (1985) and in Sentürk (1994).

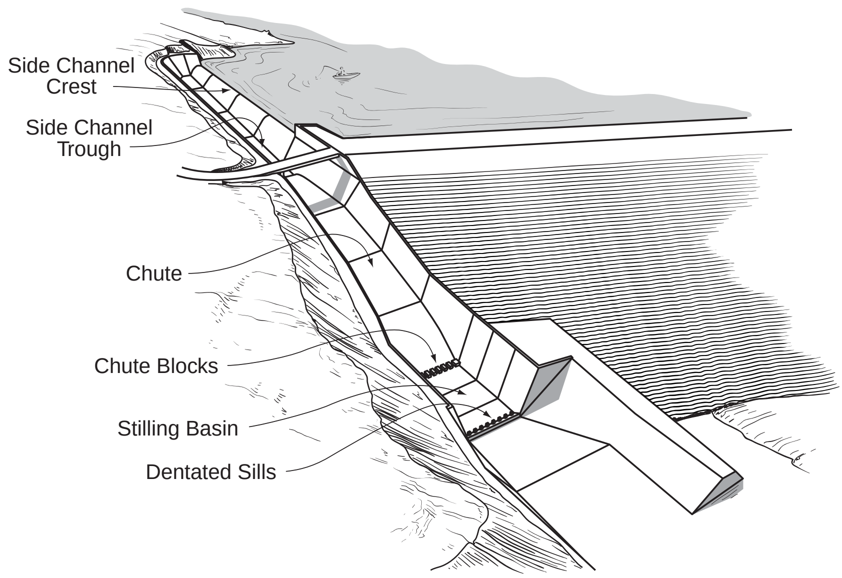

FIGURE 37.16 Typical side channel and chute spillway arrangement. (Adapted from Department of the Interior, Bureau of Reclamation, *Design of Small Dams*, p. 355, from Department of the Interior, Bureau of Reclamation.)

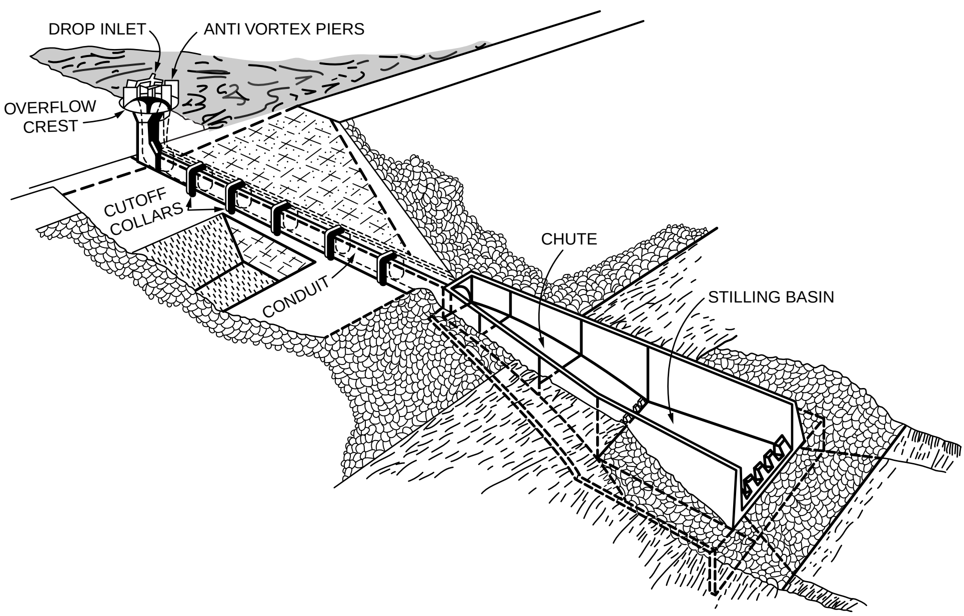

FIGURE 37.17 Shaft spillway. (*Source*: from U.S. Department of the Interior, Bureau of Reclamation, 1987, *Design of Small Dams*, p.358.)

In shaft or **morning glory spillways** the water first passes over a circular weir discharging into a vertical or sloping shaft followed by a horizontal or nearly horizontal tunnel leading to the terminal structure. The overflow crest is often provided with anti-vortex piers as shown in Fig. 37.17. At low flows the discharge over the spillway is $Q = C\ L\ h^{3/2}$ in which h is the head over the crest and L is the crest length. When the shaft is full and the inlet is submerged the spillway functions as a pipe flowing full. The

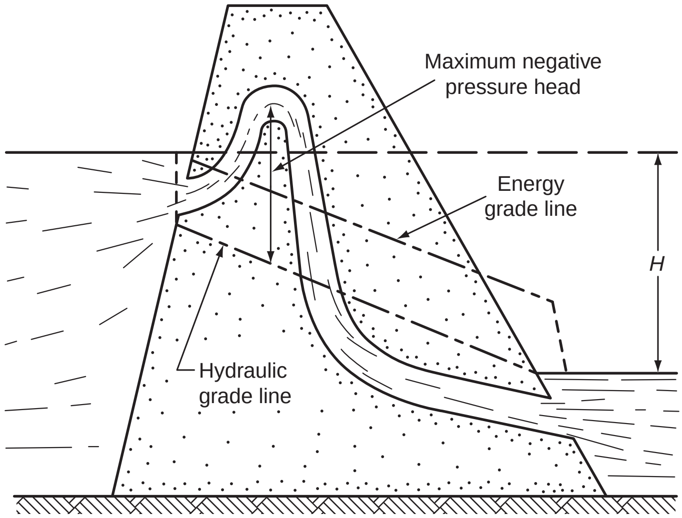

FIGURE 37.18 Siphon spillway. (*Source*: Morris, H.M. and Wiggert, J.M. *Applied Hydraulics in Engineering,* 1972, Wiley, Fig. 6–20, p. 265.)

discharge is then proportional to $H^{1/2}$, where H is the difference between the reservoir elevation and the elevation of the pipe outlet. Trash racks may be desirable to avoid clogging or damage by debris. In high shaft spillways there is the possibility of **cavitation** in the bend between the vertical shaft and the conduit. This situation must be avoided.

Siphon spillways can be used when large capacities are not required, space is limited and fluctuations of reservoir level must be maintained within close limits. (Fig. 37.18). When the siphon flows full the discharge is given by an orifice equation (see Chapter 29, Application 29.5).

$$Q = C_d A \left[2gH\right]^{1/2} \tag{37.12}$$

where the coefficient of discharge C_d is approximately 0.9, H is the difference in elevation between the reservoir and the tailwater surface if the siphon outlet is submerged, or if it is free flowing H is the difference in elevation between the reservoir and the siphon outlet.

In order to avoid cavitation the maximum velocity in the siphon must be limited to

$$V = \left[90.5\, r_i \, \log\left(r_o/r_i\right)\right] \Big/ \left(r_o - r_i\right) \tag{37.13}$$

in which V is the maximum velocity (in feet per sec), r_o and r_i are the outside and inside radii at the crown of the siphon, assuming a free vortex at the crown (Morris and Wiggert, 1972).

Without contraction at the outlet, the maximum permissible head, H, associated with this velocity is

$$H = \left(1 + k_e + k_f + k_b\right) V^2 \Big/ \left(2g\right) \tag{37.14}$$

in which k_e, k_f, k_b are the coefficients for entrance, friction and bend losses, respectively.

Labyrinth spillways are particularly advantageous when the available width is limited and large discharges must be passed. They concentrate the discharge into a narrow chute. They are particularly well suited for rehabilitation of spillways when the capacity has to be increased. They are more economical than gated structures. The total length of the crest, L_T, is

$$L_T = N\left(2\, L1 + A + D\right) \tag{37.15}$$

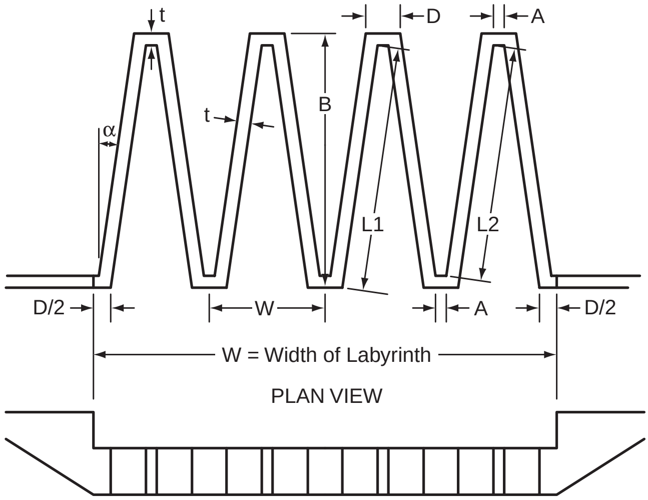

FIGURE 37.19 Labyrinth weir. (*Source*: Tullis, J.P., Amanian, N. and Waldron, D. 1995. ASCE, *Jour. of Hydraulic Engineering,* 121, 3, 247–255.)

where $L1$ is the actual length of the side leg and A and D are shown in Fig. 37.19 and N is the number of cycles (4 shown).

The effective length L of the crest in Eq. (37.10) is

$$L = 2\,N\,(A + L2) \tag{37.16}$$

where $L2$ is the effective length of the side.

Values of the discharge coefficient and details about the design of Labyrinth spillways can be found in Tullis et al.(1995) and in Zerrouk and Marche (1995).

Stepped spillways and channels have been used since antiquity. The steps significantly increase the energy dissipation over the spillway face thus reducing the size of the downstream stilling basin. Their design has been reviewed by Chanson (1993).

Cavitation

High spillways can experience flow velocities of 10 to 15 m/s or more. When the high velocity flow encounters a rapid convergence of the streamlines (perhaps caused by a surface irregularity or a bend), then cavitation is likely to occur. Cavitation is the formation and subsequent collapse of vapor cavities that occur when the fluid pressure gets below the vapor pressure. (Values of the vapor pressure head as a function of temperature are given in Chapter 29, Fundamentals of Hydraulics, Tables 29.1 and 29.2). On a spillway surface, when the streamlines of high velocity flow curve significantly at a surface irregularity, then the pressure drops along the converging streamlines. Abrupt convergence of the streamlines can produce pressure drops to the vapor pressure and vapor cavities begin to form. As the bubbles are swept downstream in a region of higher pressure, they collapse near the concrete boundary. The implosion of the bubbles creates a myriad of small high velocity jets that destroy the concrete surface. Measures to control caviatation include close construction tolerance and aeration of the flow by means of aeration devices. For further discussion of spillways considering cavitation and aeration see, for example, Wei (1993). Cavitation in pipes is discussed in a subsequent section.

Spillway Crest Gates

Several types of gates can be installed on spillway crests in order to obtain additional storage. By opening the gates, partial or full spillway discharge capacity can be obtained. The radial or **Tainter gate** has an

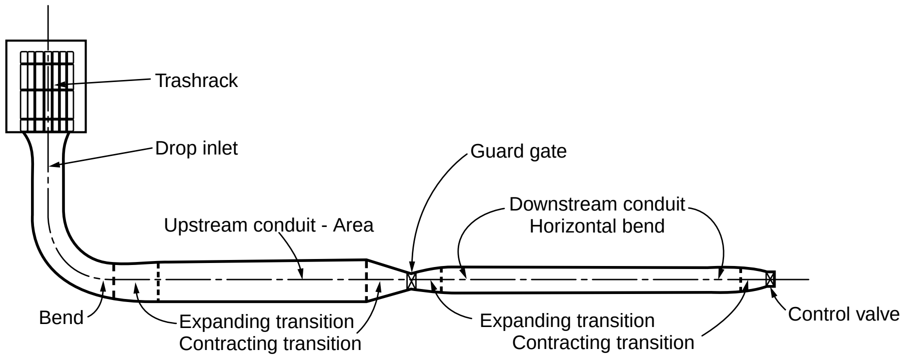

FIGURE 37.20 Outlet works pressure conduit. (Adapted from Department of the Interior, Bureau of Reclamation, 1987, *Design of Small Dams* Fig. 10.11, page 457, U.S. Government Printing Office, Denver, CO.)

upstream surface which is a sector of a cylinder. Thus the hydrostatic force goes through the pivot or trunnion located at the center of the circular arc. (see Chapter 29 on Fundamentals of Hydraulics, Application 29.3). Other types of crest gates include vertical lift gates, flap gates, drum gates, roller gates and inflatable gates. Design guidelines for spillway gates can be found in Sehgal (1996, 1993) and Sagar (1995). Gates that overturn when the reservoir level exceeds a specified elevation are called fuse gates (Falvey and Treille, 1995). Several gates are used and each gate is set to overturn with increasing reservoir levels so that only the number of gates needed to pass the flow are overturned. The fuse plug performs a similar function. It is an embankment that is designed to wash out in a predictable and controlled manner when capacity is needed in excess of that of service spillway and outlet works (Pugh and Gray, 1984). In this case the entire fuse plug fails. Horizontal boards or stop logs laid between grooved piers can be used in small installations. Flashboards or wooden panels held by vertical pins anchored on the crest of the spillway are sometimes used in small installations to temporarily raise the water surface.

37.5 Outlet Works

Components and Layout

The purpose of the outlet works is to regulate the operational outflows from the reservoir. The intake structure forms the entrance to the outlet works. It may also include trash racks, fish screens, and gates. The conduit entrance may be vertical, inclined or horizontal. The conduit may be free flowing or under pressure. For low dams the outlet may be a gated open channel. For higher earth dams the outlet may be a cut-and-cover conduit or a tunnel through an abutment. For concrete dams the outlet is generally a pipe embedded in the masonry or the outlet is formed through the spillway using a common stilling basin to dissipate the excess energy of both the spillway and outlet works outflows. Diversion tunnels used for the construction can in some cases be converted to outlet works. Examples of layout of outlet works may be found in U.S. Department of the Interior, Bureau of Reclamation (1987).

Hydraulics of Outlet Works

When the outlet is under pressure it performs as a system of pipes and fittings in series as shown in Fig. 37.20. It typically includes trash racks, an inlet, conduits, expansions, contractions, bends, guard gates that are usually fully open or fully closed for the purpose of isolating a segment of the system and control valves for the regulation of the flow. The total head H_T, the difference in elevation between the reservoir and the centerline of the outlet, is used in overcoming the losses and producing the velocity head at the exit

$$H_T = \sum h_i + h_v \tag{37.17}$$

where $\sum h_i$ = the sum of the applicable losses due to trash racks, entrance, bends, friction, expansion, contraction, gate valves

h_v = the exit velocity head

These losses are expressed as $h_l = KV^2/(2g)$, except for the contraction and expansion losses which are expressed as $h_l = K(V_1^2 - V_2^2)/(2g)$. Appropriate values of the coefficient K may be found in Table 29.9, Chapter 29 on Fundamentals of Hydraulics as well as in U.S. Department of the Interior, Bureau of Reclamation (1987) or in handbooks (Brater et al., 1996). Additional information on design of trashracks may be found in ASCE (1993).

When the outlet functions as an open channel, the flows are usually controlled by head gates. As the channel can be nonprismatic, the flow profile is calculated by the procedure described in the section on standard step method for gradually varied flow in nonprismatic channels (Chapter 30 on Open Channel Hydraulics). An example of design can be found in U.S. Department of the Interior, Bureau of Reclamation 1987. Guidelines for the design of high head gates that may be used in outlet works have been reviewed by Sagar (1995). The experiences of outlet works in the UK have been reviewed by Scott (2000).

37.6 Energy Dissipation Structures

When spillways or outlet works flows reach the downstream river a large portion of the static head has been converted into kinetic energy. Energy dissipation structures are therefore needed to prevent scour at the toe of the dam or erosion in the receiving stream or damage to the adjacent structures. As the flow from the spillway or the outlet works is usually supercritical, the hydraulic jump provides an efficient way of dissipating energy as the flow goes from supercritical to subcritical. The ratio of the depth d_1 before the jump to the conjugate depth d_2 after the jump is

$$d_2/d_1 = 1/2\left[\left(1+8\,F^2\right)^{1/2} - 1\right] \tag{37.18}$$

where $F = V_1/(gd_1)^{1/2}$ is the Froude number of the incoming flow and the head dissipated in the jump, h_j, is

$$h_j = \left(d_1 + V_1^2/2g\right) - \left(d_2 + V_2^2/2g\right) \tag{37.19}$$

where V_1 and V_2 are the velocities before and after the jump.

The US Bureau of Reclamation has developed several types of **stilling basins** to stabilize the position of the jump and improve the energy dissipation. Figure 37.21 shows a Type III stilling basin for $F > 4.5$ and $V_1 < 60$ ft/s. The tailwater depth in the downstream channel is taken equal to the conjugate depth d_2. Similar information for other ranges of Froude numbers can be found in U.S. Department of the Interior, Bureau of Reclamation (1987).

Usually the conjugate depth d_2 and the tailwater depth TW in the discharging stream cannot be matched for all discharges. The elevation of the floor of the stilling pool can be set such that d_2 and TW match at the maximum discharge. If $TW > d_2$ at lower discharges the conjugate depth d_2 can be raised by widening the stilling basin and a closer fit between the two rating curves can be obtained. For some rating curves the tailwater and the conjugate depth rating curves can be matched for an intermediate discharge (see U.S. Department of the Interior, Bureau of Reclamation, 1987, p. 397). Whether the basin is widened depends on hydraulic and economic considerations.

The submerged bucket dissipator can be used when the tailwater is too deep for the formation of a hydraulic jump. There are two types: the solid bucket and the slotted bucket. Figure 37.22 shows the geometry of these dissipators. The dissipation is due to the formation of two rollers rotating in opposite

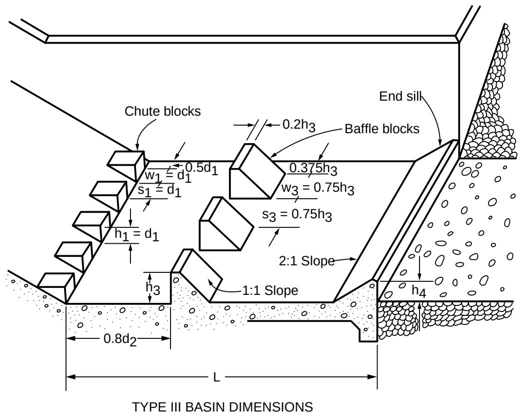

FIGURE 37.21 USBR stilling basin type III for Froude numbers larger than 4.5 and incoming flow velocity less than 60 ft/s. (*Source*: U.S. Department of the Interior, Bureau of Reclamation, 1987 *Design of Small Dams* Fig 9.41, page 391.)

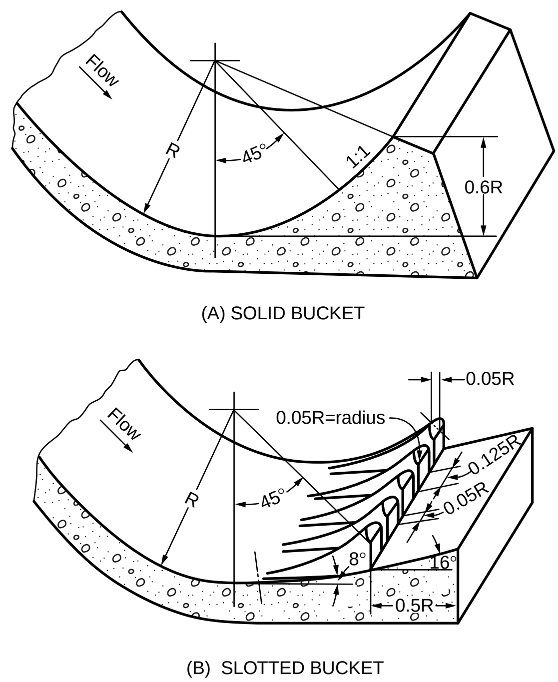

FIGURE 37.22 Submerged bucket dissipators.(*Source* U.S. Department of the Interior, Bureau of Reclamation, 1987, *Design of Small Dams*. Fig 9.45, page 398 U.S. Government Printing Office, Denver, CO.)

directions. Curves for their design can be found in U.S. Department of the Interior, Bureau of Reclamation (1987). The slotted bucket is the preferred design although the range of acceptable discharges is more limited. Other types of smaller dissipation structures are the straight drop spillway, the slotted grating dissipator and the impact type stilling basin. The latter type can be used with an open chute or a closed conduit and its performance does not depend on the tailwater.

The high flow velocity and air mixing below spillways and plunge pools can result in a supersaturation of nitrogen and oxygen in the water. This in turn can be a threat to anadromous fish. Geldert et al. (1998) have developed relationships to predict the dissolved gasses below spillways and stilling basins. This information can assist in the design and operation of such structures in order to mitigate high dissolved gas concentrations below them.

37.7 Diversion Structures

Construction of a dam across a perennial river generally requires the diversion of the flow so that the site can be dewatered and the foundation excavation can proceed in the dry. The diversion works typically include an upstream **cofferdam**, a downstream cofferdam and a conveyance structure. The upstream cofferdam directs the flow towards the conveyance structure and the downstream cofferdam protects the construction site from below. Cofferdams are temporary dams. Typically, they consist of circular cells connected to one another. The periphery of the cells is made of flat-web steel sheetpilings that reach the rock and provide effective water cutoff. The cells are filled with sand, gravel or a mix. The pressure of the fill material produces a hoop tension in the interlocks that provides watertightness and structural integrity. Free-standing cofferdams, without stabilizing inside berms, have been built to a height of 35 m (115 ft) (Fetzer and Swatek, 1988). The conveyance structure can be a channel, or a single tunnel or multiple tunnels. These conveyances can take the flow around the dam abutments or through the dam itself. In some cases part of the conveyance structure can be integrated in the outlet works. The discharge used for the diversion structures typically ranges from the 5-, 10-, 25- or the 50-year frequency flood depending on the risk of flooding that can be tolerated. The higher frequencies are used if the site is upstream of an urban area and if the cost of possible damage to completed work is important. Examples of diversion structures can be found in U.S. Department of the Interior, Bureau of Reclamation (1987), Swatek (1993) and Sentürk (1994).

37.8 Open Channel Transitions

Subcritical Transitions

Transitions are needed to connect channels of different cross sections, for example, to connect a trapezoidal channel to a rectangular flume or to a circular conduit to cross over a valley on an aqueduct or under a valley with an inverted siphon, respectively. These typically are contracting transitions. Likewise the transition from a rectangular flume or a circular conduit to a trapezoidal channel usually is through an expanding transition. Chow (1959) recommends an optimum maximum angle between the channel axis and a line connecting the channel sides of 12.5°. The drop of water surface, $\Delta y'$, for an inlet structure is given by

$$\Delta y' = (1 + c_i)\Delta h_v \tag{37.20}$$

and the rise in water surface, $\Delta y'$, in an outlet transition is given by

$$\Delta y' = (1 - c_o)\Delta h_v \tag{37.21}$$

where Δh_v is the change in velocity head and the coefficients c_i, c_o have the following values (Chow, 1959):

Transition Type	c_i	c_o
Warped	0.10	0.20
Cylinder quadrant	0.15	0.25
Simplified straight line	0.20	0.30
Straight line	0.30	0.50
Square ended	0.30+	0.75

The following Bureau of Reclamation formula can be used for the preliminary estimates of freeboard in channels less than 12 ft deep: $F = [C\,y]^{1/2}$ in which F is the freeboard, y is the depth, both in feet, and C is a coefficient varying from 1.5 to 2.5 for channels with discharges varying from 20 to 3000 ft^3/s, respectively. There are two approaches to the design of transitions: (1) a free water surface is assumed (for example two reversed parabolas) and the depth is calculated for assumed width and side slope (Chow, 1959, p. 310–317 and French, 1985); or (2) the boundaries are set first and the surface is calculated (Vittal and Chiranjeevi, 1983; French, 1985). Swamee and Basak (1991, 1992) have developed designs of rectangular and trapezoidal expansion transitions that minimize the head losses.

Supercritical Contractions

Contractions designed for subcritical flows will not function properly for supercritical flows. Generally, with supercritical flow, wave patterns are formed in the contraction and propagate in the downstream channel. Supercritical flow contractions are best designed for rectangular channels. The converging angles on each side produce two oblique hydraulic jumps that makes an angle with the original flow direction. A second pair of oblique jumps is created by the diverging angle at the downstream end of the contraction. Ippen and Dawson (1951) devised a design such that the disturbances caused by the converging angles are canceled by the disturbances caused by the diverging angles so that there is no wave pattern in the channel downstream of the contraction. This design will function properly only for the specified Froude number in the upstream channel. Additional details on the design of supercritical transitions can be found in Ippen (1950), Chow (1959), Henderson (1966), French (1985) and Sturm (1985). Numerical simulation of supercritical flow transitions has been discussed by Rahman and Chaudhry (1997).

37.9 Culverts

Flow Types

Culverts are short conduits that convey flows under a roadway or other embankment. They are generally constructed of concrete or corrugated metal. Common shapes include circular, rectangular, elliptical, and arch. Culverts can flow full or partly full. When the culvert flows full it functions as a pipe under pressure. When it flows partly full it functions as an open channel and the flow can be subcritical, critical or supercritical. (See Chapter 30, "Open Channel Hydraulics".) A culvert operates either under inlet or outlet control. If the culvert barrel has greater capacity than the inlet, then the culvert functions under inlet control. Conversely, if the barrel has less capacity than the inlet, the culvert operates under outlet control. Figures 37.23 and 37.24 illustrate inlet and outlet control flows, respectively. Partly full flow can occur with inlet control or with outlet control.

When operating under inlet control the flow becomes critical just inside the entrance and the flow is supercritical through the length of the culvert if the outlet is unsubmerged; if the outlet is submerged a hydraulic jump forms in the barrel. For low unsubmerged headwater the entrance of the culvert operates as a weir (Eq. [37.10]). When the headwaters submerge the entrance it performs as an orifice (Eq. [37.12]). From tests by the National Bureau of Standards, performed for the Bureau of Public Roads (now Federal Highway Administration), equations have been obtained to calculate the headwater above the inlet invert, for unsubmerged and submerged inlet control (Normann et al., 1985). These equations can be presented in a regression form that gives a direct solution for the inlet head given the discharge, the span and the rise of the culvert for the several culvert types. The equation and a table of regression coefficients can be found in U.S Federal Highway Administration (1999).

When operating under outlet control, for a full flow condition the total loss, H_L, through the conduit is

$$H_L = \left[1 + k_e + 2g\, n^2 L \big/ \left(K_M^2\, R^{1.33}\right)\right] V^2 \big/ 2g \qquad (37.22)$$

where k_e = an entrance loss coefficient
L = the length of the culvert
R = the hydraulic radius
n = Manning's roughness coefficient
V = the flow velocity

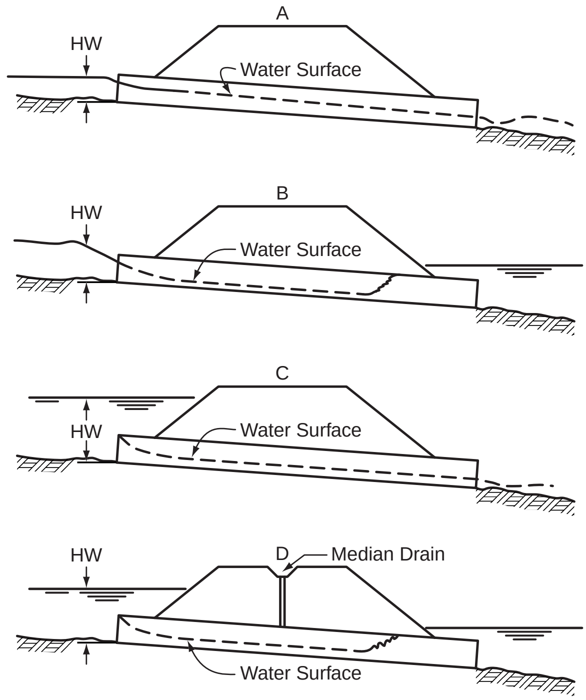

FIGURE 37.23 Types of inlet control: (A) inlet and outlet outlet unsubmerged, critical flow at entrance, supercritical flow along barrel, (B) inlet unsubmerged and outlet submerged, critical flow at entrance and supercritical flow upstream of hydraulic jump in culvert, (c) inlet submerged, outlet unsubmerged, critical depth at entrance and supercritical flow along barrel, (D) inlet and outlet submerged, similar to (B) with hydraulic jump in culvert. (*Source*: Normann et al. 1985.)

K_M has a value of 1 for metric units and 1.486 for customary English units (See Chapter 30, "Open Channel Hydraulics," Section 30.3).

For a full flow condition the energy and hydraulic grade line are shown in Fig. 37.25 and the relation between points at the free surface upstream and downstream of the culvert is

$$HW_o + V_u^2/(2g) = TW + V_d^2/(2g) + H_L \tag{37.23}$$

where HW_o = the headwater depth about the outlet invert
V_u = the upstream velocity of approach
TW = the tailwater depth above the outlet invert
V_d = the downstream velocity
H_L = the total loss through the conduit given in the preceding equation

For further details on the hydraulics and design of culverts see, for example, Tuncock and Mays (2001).

Inlets

For culverts operating under inlet control, the performance can be improved by reducing the contraction at the inlet and by increasing the effective head. These objectives can be achieved with tapered inlets. The simplest design is the side tapered inlet in which the side walls are flared between the throat and the face resulting in an enlarged face section. In addition, the effective head can be increased by depressing the inlet. This can be achieved by an upstream depression between the wingwalls or a sump upstream of the face section. A more elaborate design is the slope tapered inlet that has flared sidewalls. A fall is incorporated between the throat and the face sections, (Normann et al, 1985).

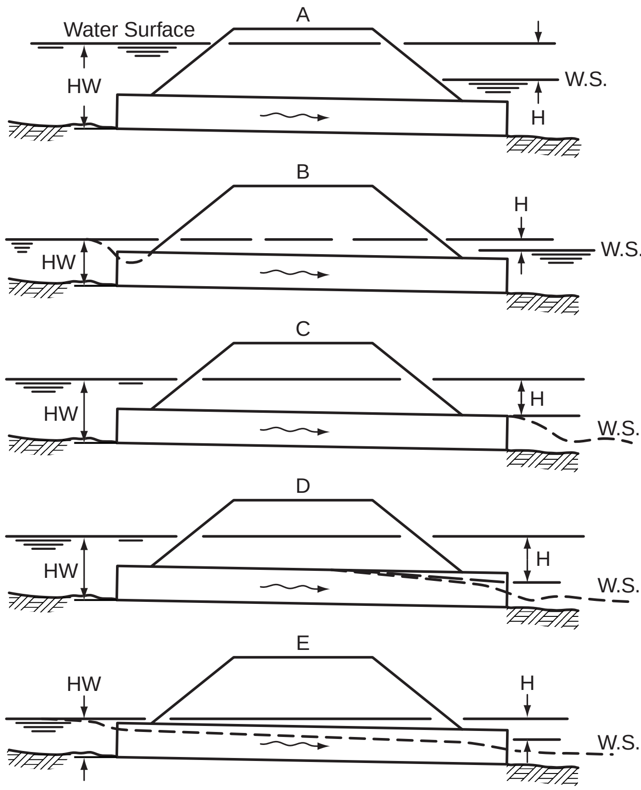

FIGURE 37.24 Types of outlet control: (A) inlet and outlet submerged, pipe flow, (B) inlet unsubmerged and outlet submerged, with surface drop at the entrance and contraction, (c) inlet submerged, pipe flow along barrel, no tailwater, (D) inlet submerged and outlet unsubmerged, outlet depth grater than critical, (E) inlet and outlet unsubmerged, subcritical flow over entire length, critical condition at outlet. (*Source*: Normann et al. 1985.)

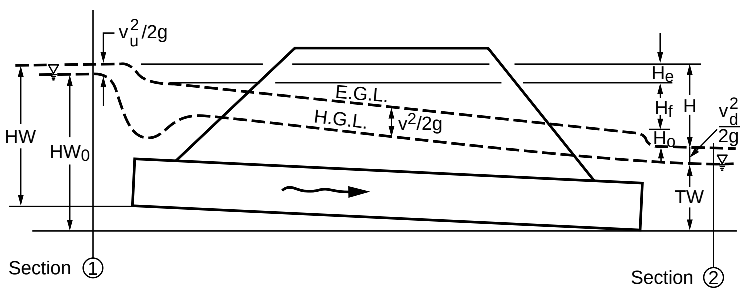

FIGURE 37.25 Energy and hydraulic grade line for full flow condition (*Source*: Normann et al. *Hydraulic Design of Highway Culverts*. 1985. Fig. III-8, p.36 Federal Highway Administration Report No. FHWA-IP-85–15.)

Sedimentation and Scour

Special consideration must be given to the transport of sediments. Low flow velocities within the culvert may result in deposition of sediments, whereas high velocities may produce erosion at the outlet of the culvert. For further details on sediment transport see Chapter 35, "Sediment Transport in Open Channels" and Yang (1996). Protection against scour include cutoff walls, riprap armoring and energy dissipators. These protection devices are used if the outlet velocities are larger than 1.3 times the natural stream velocity, between 2.3 and 2.5 or greater than 2.5 times the natural stream velocity, respectively (Tuncock and Mays, 2001).

Software

The principal public domain computer program for the hydraulic design and analysis of culverts is: the interactive program HY8 that is part of the HYDRAIN (U.S. Federal Highway Administration, 1999), an integrated drainage design computer system. HY8 consists of four modules: (1) culvert analysis and design, (2) hydrograph generation,(3) routing, and (4) energy dissipation. HYDRAIN (version 6.1) can be downloaded from http://www.fha.dot.gov/bridge/hydrain.htm. The software package HEC-RAS, River Analysis System, developed by the US. Army Corps of Engineers, Hydrologic Engineering Center (2001) is a comprehensive suite of computer programs for water surface and river hydraulics calculations and includes hydraulics of bridges and culvert openings. HEC-RAS can be downloaded from http://www.wrc-hec.usace.army.mil/. These programs are discussed in more detail in Chapter 38, "Simulation in Hydraulics and Hydrology".

37.10 Bridge Constrictions

Backwater and Discharge Approaches

The hydraulics of bridges can be approached from the point of view of the highway engineer or from that of the hydrologist. The highway engineer is concerned with the amount of backwater created by a bridge constriction. Approach embankments are often extended in the flood plain to reduce the span of the bridge proper and thus decrease the cost of bridge crossings. The flow is thus forced to pass through a constriction that may produce a backwater. In contrast the hydrologist is concerned with the indirect determination of the discharge of a large flood from high water mark observations and from the geometry of the constriction. Indirect methods of discharge determination are needed when the flow rate is beyond the range of the rating curves at gaging stations.

Flow Types

Three different types of flow can occur at a bridge constriction. The first occurs when the flow is subcritical both in the stream and through the constriction. This is the flow type most generally encountered, illustrated in Fig. 37.26 and is discussed below. In the second type the flow is subcritical upstream of the bridge but goes through critical in the constriction. The flow can then immediately return to subcritical as it exits the constriction or the water surface can dip below the critical depth downstream of the constriction and then return to its normal depth through a hydraulic jump. Finally, the third type occurs when the flow is supercritical through both the stream and the constriction.

Backwater Computation

The backwater superelevation, h_1^*, upstream of a constriction with subcritical flow is given by Bradley (1978)

$$h_1^* = K^* \alpha_2 V_{n2}^2/(2g) + \alpha_1\left[(A_{n2}/A_4)^2 - (A_{n2}/A_1)^2\right] V_{n2}^2/(2g) \qquad (37.24)$$

where K^* = the backwater coefficient
A_{n2} = the gross water area in constriction measured below normal stage
$V_{n2} = Q/A_{n2}$ is the average velocity in the constriction
A_4 = the area at section 4 where the normal depth is reestablished
A_1 = the water area at section 1 including that produced by the backwater
α_1 and α_2 = the kinetic energy corrections factors at sections 1 and 2, respectively (See Eq. [29.11] in Chapter 29, "Fundamentals of Hydraulics")

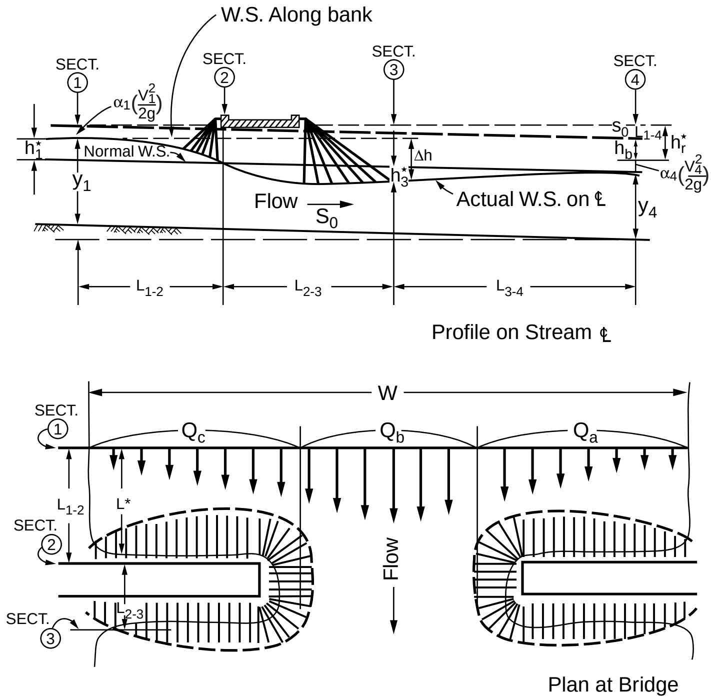

FIGURE 37.26 Normal bridge crossing with spillthrough abutments. (*Source*: Bradley, 1978, *Hydraulics of Bridges*. FIGURE 3 p. 7, Hydraulic Series No.1, Federal Highway Administration.)

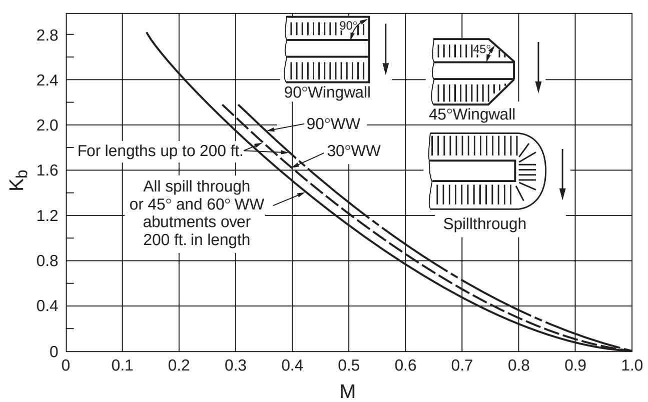

FIGURE 37.27 Backwater coefficient for wingwall (WW) and spillthrough abutments. (*Source*: Bradley, 1978, *Hydraulics of Bridges*, Fig 6, p. 14, Hydraulic Series No.1, Federal Highway Administration.)

α_1 can be approximated as $\alpha_1 = \Sigma q v^2/(QV_1^2)$ where q and v are the discharge and velocity in subsections of section1, respectively. Similarly α_2 can be approximated for section 2. In Fig. 37.27, the basic backwater coefficient K_b is given as a function of the bridge opening ratio, $M = Q_b/Q$, where Q_b is the flow in the portion of channel within the projected length of the bridge (Fig.37.26) and Q is the total discharge. Incremental coefficients to account for the effects of the piers, opening eccentricity, skewed crossing, dual bridges, bridge girder submergence, and backwater in the stream can be found in Bradley (1978). The sum of K_b and the incremental coefficients yields K^*.

Discharge Estimation

The discharge, Q, through a bridge constriction, under subcritical flow condition, can be calculated from Chow (1959)

$$Q = C\,A_3\left[2g\left(\Delta h - h_f + \alpha_1 V_1^2/(2g)\right)\right]^{1/2} \tag{37.25}$$

where C = a discharge coefficient
A_3 = the flow area at section 3
Δh = the drop in water surface between section 1 and section 3
V_1 = the mean velocity at section 1
h_f = the head loss between sections 1 and 3

This loss is calculated from

$$h_f = L_a\left[Q/(K_1 K_3)^{1/2}\right]^2 + L\left[Q/K_3\right]^2 \tag{37.26}$$

where L_a = the approach length from section 1 to the upstream face of the abutment
L = the length of the contraction
$K_1 = (K_M/n)\,A_1\,R_1^{2/3}$ is the conveyance at section 1 in which $K_M = 1$ for metric units and K_M = 1.486 for customary English Units

Similarly, K_3 is the conveyance at section 3. The discharge coefficient, C, depends on the shape of the abutment, the ratio of the contraction length to the contraction width, L/b, and the contraction ratio $m = 1 - K_b/K_1$, where b is the constriction width, K_b is the conveyance of the contracted section and K_1 is the conveyance of the approach section 1. Figure 37.28 gives the base discharge coefficient for the bridge opening with spillthrough abutments. Chow (1959) and Kindsvater et al. (1953) give curves for determination of the discharge coefficient for different abutment types and multiplicative correction factors to account for the effects of the Froude number, the rounding and chamfering of the abutment, the skewness and eccentricity of the bridge, the possible submergence of the bridge girders, the piers and piles.

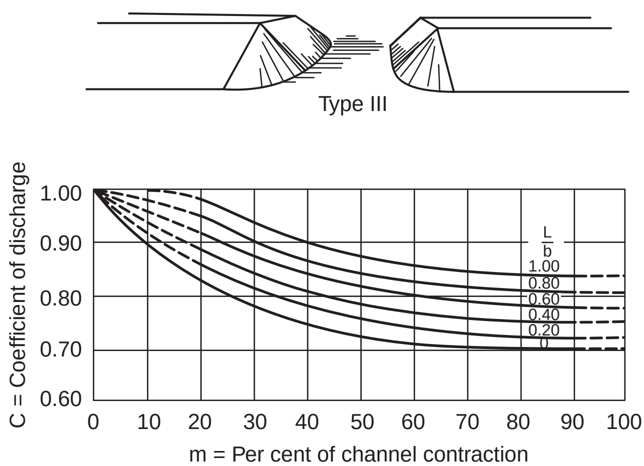

FIGURE 37.28 Discharge coefficient for spillthrough abutment bridge opening. (*Source:* Kindsvater, C.E., Carter, R.W. and Tracy, H.J., *Computation of Peak Discharge at Contractions,* U.S. Geological Survey, Circular No. 284.)

Scour

The previous discussion assumes that the channel is rigid, that is it does not aggrade nor degrade. However, long-term stream bed degradation and local scour may take place. Aggradation occurs when the sediment load supplied to a river reach exceeds its transport capacity. Aggradation can cause a reduction of bridge waterway openings. This in turn can result in increased upstream flooding and increased scour at the contraction (Johnson et al. 2001). Scour can occur during rapid flow events, when sediments are transported by the currents eventually undermining bridge pier foundations. Erosion and deposition can occur during the same flood event. (See Chapter 35, "Sediment Transport in Open Channels").

The Federal Highway Administration current practice in the determination of scour at bridges can be found in Richardson and Davis, (1995) and has been summarized by Tuncock and Mays (2001). This is a deterministic approach. Instead, Johnson and Dock (1998) propose a probabilistic approach for determining the likelihood of various scour depths for storm events of specified return periods. Monitoring scour is difficult; however, frequency modulated–continuous wave (FM-CW) reflectometry has potential for continuous monitoring of sediment depths (Yankielun and Zabilansky, 2000).

Software

The principal public domain computer programs for hydraulics of bridges are: HEC-RAS and WSPRO. The software package HEC-RAS, River Analysis System, developed by the US. Army Corps of Engineers, Hydrologic Engineering Center (2001), is a comprehensive suite of computer programs for water surface and river hydraulics calculations and includes hydraulics of bridges and culvert openings. HEC-RAS can be downloaded from http://www.wrc-hec.usace.army.mil. WSPRO is part of the HYDRAIN, an integrated drainage design computer system (U.S. Federal Highway Administration, 1999). It performs backwater calculations by the standard step method (see Chapter 30, "Open Channel Hydraulics"). HYDRAIN can be downloaded from http://www.fhwa.dot.gov/bridge/hydrain.htm. These programs are discussed in more detail in Chapter 38, "Simulation in Hydraulics and Hydrology."

37.11 Pipes

The hydraulics of flow in pipes is discussed in Chapter 29, "Fundamentals of Hydraulics."

Networks

For pipe network calculations it is convenient to express the friction loss, h_L, in a pipe by an equation of the form

$$h_L = KQ^n \tag{37.27}$$

where K includes the effects of the pipe diameter, length and roughness as well as the fluid viscosity.

For the Darcy-Weisbach formula (Eq. [29.21] in Chapter 29, "Fundamentals of Hydraulics")

$$K = 8fL\big/\left(\pi^2 gD^5\right) \quad \text{and} \quad n = 2 \tag{37.28}$$

where L = the pipe length
D = the diameter
Q = the discharge

The friction factor f is obtained from the Moody diagram (see Chapter 29). The formula is valid for consistent metric and English units. For the Hazen-Williams formula

TABLE 37.1 Williams-Hazen Coefficients

Pipe Material	Condition	Size	C
Cast Iron	New	all	130
	5 years old	≥ 12 in.	120
		8 in.	119
		4 in.	118
	10 years old	≥ 24 in.	113
		12 in.	111
		4 in.	107
	20 years old	≥24 in.	100
		12 in.	96
		4 in.	89
	30 years old	≥30 in.	90
		16 in.	87
		4 in.	75
	40 years old	≥ 30 in.	83
		16 in.	80
		4 in.	64
	50 years old	≥ 40 in.	77
		24 in.	74
		4 in.	55
Welded Steel	Same as Cast iron 5 years older		
Riveted Steel	Same as cast Iron 10 years older		
Wood Stave	Average value regardless of age		120
Concrete or concrete lined	Large sizes, good workmanship, steel forms		140
	Large sizes, good workmanship, wooden forms		120
	Centrifugally spun		135
Vitrified	In good condition		110
Plastic or drawn tubing			150

Source: Wood, D.J., 1980. *Computer Analysis of Flow in Pipe Networks Including Extended Period of Simulation, User's Manual*, Office of Continuing Education and Extension of the College of Engineering, University of Kentucky, Lexington, KY. With permission.

$$K = C_u C^{-1.852} L D^{-4.87} \quad \text{and} \quad n = 1.852 \tag{37.29}$$

where C_u = 10.654 for metric units and C_u = 4.727 for English units
C = the Hazen-Williams coefficient, typical values of which are given in Table 37.1

Two basic relationships must be satisfied in a network: the continuity or conservation of mass at each junction and the energy relationship around any closed loop. The continuity relationship requires that the sum of the flows entering a node be equal to the sum of the flows leaving it. The energy relationship requires that the algebraic sum of the head losses in any loop be zero using an appropriate sign convention, for example flows are positive in the counterclockwise direction. The same sign convention applies to all loops of the network. Minor losses due to fittings and valves, etc. and energy gains due to pumps are included in the appropriate segments.

The first systematic numerical procedure for the calculation of the flows of liquids in pipe networks was proposed by Hardy Cross (1936). It includes the following steps: (1) assume a discharge in each pipe, Q_g, so that the continuity requirement is satisfied at each node, (2) for each pipe loop calculate a first order correction to the discharge

$$\Delta Q = -\Sigma K Q_g^x \Big/ \left[\Sigma \left| x K Q_g^{x-1} \right|\right] \tag{37.30}$$

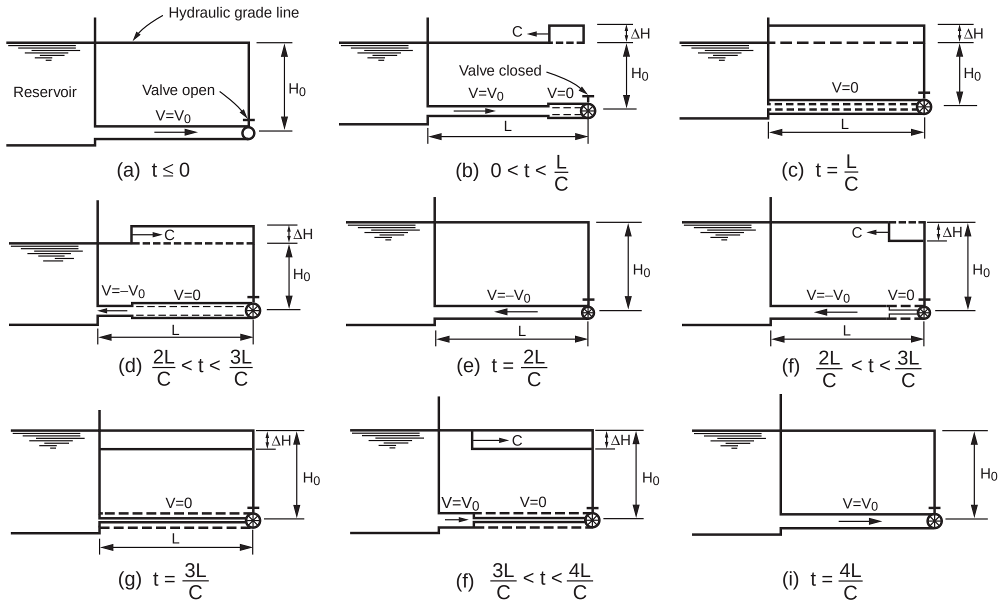

FIGURE 37.29 Water hammer cycle due to instantaneous valve closure.

taking into account the sign convention in the numerator of the right hand side,(3) in each pipe loop add the corrections algebraically to flow in each pipe (note that a pipe that is common to two loops has different signs depending upon which loop is considered and this pipe will receive two corrections), (4) with Q_g being the new flows, calculate a new correction ΔQ for each loop, (5) in each loop add the correction algebraically to each pipe, (6) repeat steps 4 and 5 until the correction becomes sufficiently small. An example involving 7 pipes, 2 reservoirs, and 1 pump can be found in Lansey and Mays (1999). The main advantages of the Hardy cross method are that it provides an understanding of the procedure and that the calculations can be done by hand for small networks. Its main limitation is that it solves the equations one at a time. More recent methods use more efficient numerical techniques such as the linear theory, the Newton-Raphson method or the gradient method to solve the large system of equations for the nodes and loops. For large networks computer programs are used. (see section on software later in this chapter). For further discussion on pipe networks see, for example, Lansey and Mays (1999).

Hydraulic Transients and Water Hammer

A hydraulic transient is a situation where conditions, such as flow velocity and pressure, are time varying. Some of the common conditions creating a transient are: a change in a valve opening; operation of check valves or pressure relief valves; starting or stopping of pumps; changes of power demand on hydraulic turbines; pipe break; trapped air in pipeline; filling or flushing of pipes, etc. Large and rapid changes in velocity can create high transient pressures. If the pipe is not designed to withstand the high transient pressures or if controlling devices are not included to limit the increase in pressure head, damage (including rupture) can occur to the pipe or to the connected equipment and machinery.

When the flow of a liquid in a pipe is stopped abruptly due to a rapid valve closure, for example, the kinetic energy is transformed into elastic energy and a train of positive and negative pressure waves travels up and down the pipe until the energy is dissipated by friction. (Fig. 37.29). When the liquid is water this is known as **water hammer** because the transient noise in small pipes sounds as if it is being hit by a hammer. The elasticity of the liquid and of the pipe material need to be taken into account. Consider the elastic properties of the water and the pipe: the bulk modulus of the liquid, E,(about 3×10^5 psi or

2 GN/m^2 for water) and the modulus of elasticity of the pipe material E_p (about 30×10^6 psi or 200 GN/m^2 for steel). The velocity or *celerity, c*, of the pressure wave is given by

$$c^2 = (E/\rho)\left[1 + E D/(E_p t_p)\right]^{-1} \tag{37.31}$$

in which ρ is the fluid density, t_p is the thickness of the pipe wall and D is the pipe diameter.

For an initial flow velocity V, the rise in pressure head due to the sudden valve closure is obtained from the momentum principle as

$$\Delta H = \Delta p/\gamma = V c/g \tag{37.32}$$

This is the pressure head obtained when the time of closure of the valve, t_c, is less than the time for the round trip travel of the pressure wave $2L/c$. For a longer closing time, t, the pressure head can be approximated as $(t/t_c)\ \Delta H$. However, more accurate results can be obtained by numerical integration of the transient flow equations. (Morris and Wiggert (1972), Wylie and Streeter (1993), and Borg (1993).

Figure 37.29 illustrates the pressure wave propagation without friction. Diagram (a) shows the initial steady state hydraulic grade line and velocity V_0 with the valve open. When the valve is suddenly closed the head rise Δh is calculated by Eq. (37.32) and the pressure wave travels upstream with the celerity c calculated from Eq. (37.31). Diagram (b) illustrates the condition for $0 < t < L/c$. Behind the wave the velocity is zero, the pressure is increased, and the pipes expands. The mass of water entering the pipe is equal to the increased volume of the pipe plus the added mass stored due to the increased water density. When $t = L/c$ the pressure wave arrives at the reservoir as shown in diagram (c). The pressure in the pipe is $H_0 + \Delta H$, the velocity is zero and the increased pressure exists all along the pipe. This is a non-equilibrium situation, the compressed fluid then flows from the pipe into the reservoir at a velocity $-V_o$ and the reflected pressure wave recedes. The cycle continues as shown in diagrams (e) to (i). For an in-depth treatment of water hammer see, for example, Martin (1999), Wylie and Streeter (1993), Borg (1993), Rich (1963), Parmakian (1963), Chaudhry (1987), and Jaeger (1977). There are computer programs for the analysis of water hammer (see section on software).

Surge Protection and Surge Tanks

There are two types of transient events that need to be controlled: the downsurge or low pressure event that occurs with pump power failure and the upsurge or high pressure event caused by the closure of a downstream valve. Surge control protection devices include several types of valves such as check valves and surge relief valves (Martin, 1999). Another device is the surge tank or standpipe. A surge tank is a vertical tank connected to the pipeline that typically extends above the maximum grade line. The surge tank diameter is substantially larger than that of the pipe to avoid spilling. The standpipe has a smaller diameter, possibly less than the pipe, and is used if spillage can be allowed. Normally the standpipe is designed high enough so as to avoid spillage during normal shutdown.

Surge tanks are standpipes that are installed in large piping systems to relieve the water hammer pressure when a valve is suddenly closed and to provide a reserve of liquid when a valve is suddenly opened. In hydropower installations they are located close to the turbine gates. In pumping installations they are located on the discharge side of the pumps to protect against low pressures during stoppage of the pumps. A simple surge tank is connected directly to the penstock (Fig. 37.30). An orifice surge tank has an orifice in the connection between the tank and the pipe, often with a larger coefficient of discharge for flow out of the tank. A differential surge tank consists of two concentric surge tanks, the inner one is usually a simple surge tank that provides a rapid response but has a small volume. The outer and larger tank is usually an orifice tank.

Consider a horizontal pipe of cross-sectional area A and length L between a reservoir and a surge tank of cross section S, in which the instantaneous water level is at an elevation y above that of the reservoir

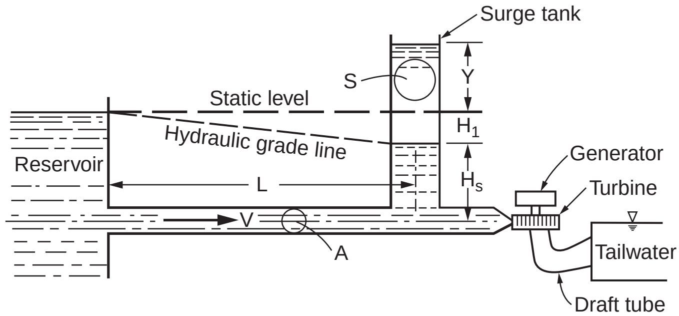

FIGURE 37.30 Simple surge tank.

(Fig. 37.30). V_o is the steady state flow velocity in the pipe. At the time of closure the surge water elevation obtained neglecting friction, is

$$y_{max} = V_o\left[(A/S)(L/g)\right]^{1/2} \tag{37.33}$$

If the pipe is fairly long the friction should be included. This results in a differential equation that requires numerical solution (Morris and Wiggert, 1972). The minimum cross-sectional area required for stability of the surge tank derived by Thoma and cited by Rich (1963) and by Coleman et al. (1999) is

$$S = (AL)/(2gkH_s) \tag{37.34}$$

where $k = H_l/V^2$ is the ratio of the head loss between the reservoir and the surge tank to the square of the flow velocity in the conduit

H_s = the steady state head in the surge tank

This area is multiplied by a minimum safety factor of 1.5 for simple surge tanks and 1.25 for orifice surge tanks, to obtain reasonably fast damping of the water oscillations according to Coleman et al. (1999), Borg (1993) and Rich (1963). For a more detailed treatment of surge tanks and other surge suppressing devices see Coleman et al. (1999), Wylie and Streeter (1993), Borg (1993), Rich (1963), Chaudhry (1987), and Jaeger (1977).

Valves

Valves are used to regulate the flow and pressure in pipes and to perform many other functions. These include prevention of reverse flow through pumps, protection of pipes and pumps from overpressurization, prevention of transients etc. Head loss coefficients for several types of valves are shown in Table 29.9 (Chapter 29, "Fundamental of Hydraulics"). Some valves are used to prevent flow in certain sections of pipe. They are normally fully open or fully closed. The gate valves are of this type. Other valves are used to control the flow. Examples of control valves are the butterfly valve, the cone, ball and plug valves, the globe valves. Howell-Bunger valves and hollow jet valves are free discharge valves used to release water from reservoirs, for aerating water, for flood control or irrigation. Check valves are used to prevent reversal of the flow. Figures 37.31 and 37.32 show simplified sketches of several types of control and check valves.

Cavitation

Cavitation is a process similar to boiling. It consists of rapid vaporization and condensation. For boiling, vapor cavities are formed due to temperature increase. The vapor cavities rise to the surface and explode releasing vapor to the atmosphere. For cavitation, the vapor cavities are formed when the fluid pressure drops below the vapor pressure. The cavity will collapse if there is a local pressure in the cavitation region

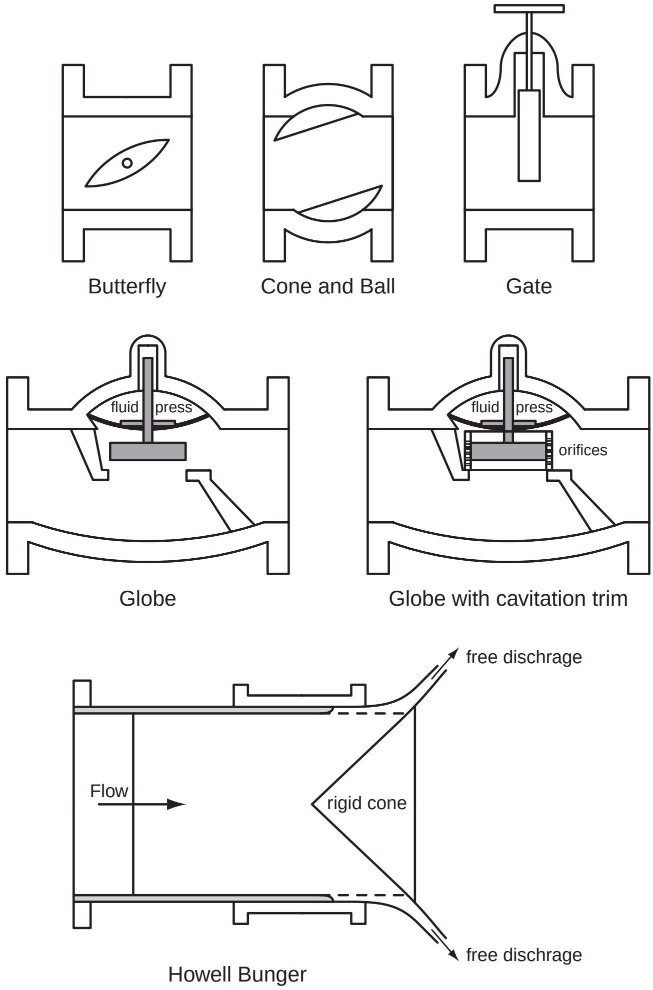

FIGURE 37.31 Schematic of typical control valves. (*Source:* Tullis, J.P., 1989, *Hydraulics of Pipelines, Pumps, Valves, Cavitation, Transients.* John Wiley & Sons, New York, NY. Fig 4.1 p.83.)

that is greater than the vapor pressure. In a venturi, for example, the pressure is minimum at the throat where the cavity may form and collapse occurs as the cavity moves from the throat into the diffuser where the pressure increases with distance. The shock waves produced by the collapsing cavities produce pressure fluctuations that can induce vibration of the system. Cavitation creates noise. The noise created by cavitating valves can be quite intense, varying from a hissing or crackling sound to loud roars with intermittent explosions. The collapse of cavities close to solid boundaries can causes considerable damage to almost any surface. It also causes corrosion of metal surfaces. Cavitation can also occur in hydraulic machinery reducing its efficiency and damaging the impellers. The onset of cavitation can be expressed by a cavitation number. For valves the cavitation number σ is generally defined as

$$\sigma = \left(p_d + p_b - p_{va}\right) / \Delta p \tag{37.35}$$

where p_d = the pressure downstream (10 diameters)
p_b = the barometric pressure
p_{va} = the absolute vapor pressure
Δp = the net pressure drop

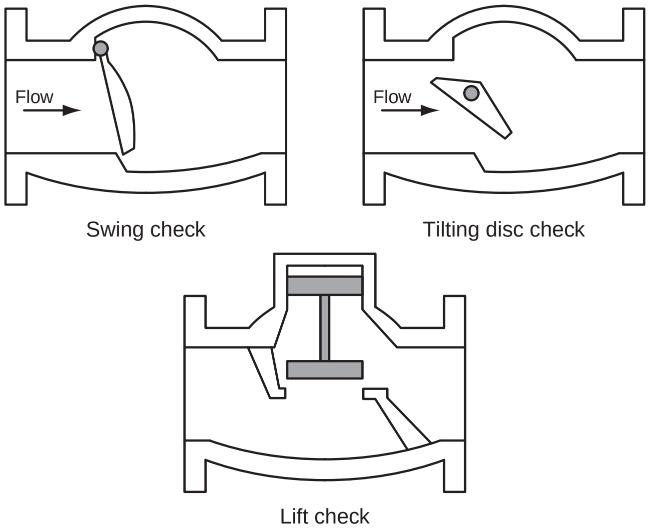

FIGURE 37.32 Schematic of typical check valves. (*Source:* Tullis J.P., 1989, *Hydraulics of Pipelines, Pumps, Valves, Cavitation, Transients.* John Wiley & Sons, Nrew York, NY. Fig 4.10, p.112.)

The cavitation parameter for pumps is discussed in the section on pumps. The vapor pressure head of water as a function of temperature is given in Chapter 29, "Fundamentals of Hydraulics," Tables 29.1 and 29.2. Tullis (1989) gives experimental values of the cavitation parameter for several types of valves and for several intensities of cavitation.

Forces on Pipes and Temperature Stresses

The fluid pressure in a pipe creates a circumferential tension stress called hoop stress given by

$$s = (pD)/(2t) \tag{37.36}$$

where p = the pressure (static plus water hammer)
D = the inside diameter
t = the thickness of the pipe wall

Unsupported pipe segments act as beams. The loads include the weight of the pipe, the weight of the fluid and any superimposed load. Forces on pipe bends and anchor blocks are obtained by the momentum equation as illustrated in Chapter 29 (Application 29.8).

Buried pipes must support the loads due to gravity earth forces and live loads. Load and supporting strength depend on installation conditions. Design details and specifications can be found in ACI, ASTM, AASHTO or FHWA specifications and industry manuals. ASCE (1992) Manual of Practice 77 (Chapter 14 on structural requirements) gives a good state-of-the-art review.

For concentrated and distributed loads superimposed on buried pipes the reader is referred to the AASHTO Code, the Portland Cement Association 1951) and the American Concrete Pipe Association (1988) for wheel loads, as well as to ASCE (1992) for a discussion of the Boussinesq theory for concentrated loads.

A temperature change ΔT on a pipe of modulus of elasticity E and coefficient of thermal expansion α will induce a longitudinal stress, assuming fixed ends, given by

$$\sigma = \alpha E \Delta T \tag{37.37}$$

For steel approximate values of the physical constants are $E = 30 \times 10^6$ psi and $\alpha = 6.5 \times 10^{-6}$ °F.$^{-1}$

Software

EPANET, developed by the U.S. Environmental Protection Agency (2000), is a public domain software. It performs extended period simulation of hydraulics and water quality behavior in a pressurized pipe network. In the hydraulic analysis it places no limit on the size of the network, computes friction headloss using the Hazen-Williams, Darcy-Weisbach or Chézy-Manning formulas, includes minor losses, constant or variable speed pumps, variable geometry surge tanks, etc. It uses a gradient algorithm for the solution of the hydraulic equations. In addition to the hydraulic modeling, EPANET models the movement of non-reactive tracer material through the network over time, models the movement and fate of a reactive material as it grows or decays (e.g., chlorine residual) with time, models the age of water throughout a network, tracks the percent of flow from a given node reaching all other nodes over time, etc. EPANET 2 can be downloaded from http://www.epa.gov/ORD/NRMRL/wswrd/epanet.html.

KYPIPE, is a computer program for the solution of pipe networks developed at the University of Kentucky (Wood, 1980). It uses a Newton method for the solution of the hydraulic equations.

FORTRAN computer programs for water hammer analysis can be found in Wylie and Streeter (1993) and in Chaudhry (1987). A FORTRAN program for water level oscillations in a simple surge tank is given in Chaudhry (1987). Tabular presentations of the numerical integration of the unsteady flow equations that can be adapted to spread sheet software such as EXCEL, LOTUS, QUATTRO PRO, etc. can be found in Morris and Wiggert (1972, p. 335) for water hammer and in Rich (1963) for water hammer, surge tanks and stability analysis.

37.12 Pumps

Centrifugal Pumps

Centrifugal pumps are those most commonly used in civil engineering applications. The rotating part of the centrifugal pump is the impeller. It consists of blades or vanes attached to the hub. If the blades are enclosed by plates or shrouds on the top and bottom sides, the impeller is closed. Impellers without shrouds (i.e., open impellers) are less prone to become clogged when the liquids contain suspended matter. Closed impellers, however are more efficient. In radial flow impellers the fluid is forced outward in the radial direction, which is perpendicular to the axis (see Fig. 37.33), while in axial flow impellers the fluid exits along the axis. In the mixed flow pumps the impeller imparts velocities that have both radial and axial components. The flow exits from the impeller into a casing called the volute. Centrifugal pumps can be single stage or multistage. Deep well pumps often are multistage, which is equivalent to several stages or impellers in series so that the total head generated by the pump is the sum of the heads imparted to the fluid at each stage.

The impeller exerts a torque on the fluid. This torque can be calculated as the change in angular momentum. This is obtained by multiplying the terms of the momentum equation (Eq. [29.15])) by the lever arm r, to yield

$$T = \rho Q\left(V_{t2} r_2 - V_{t1} r_1\right) \tag{37.38}$$

where V_{t1} and V_{t2} = the tangential components of the flow velocities at the entrance and at the exit of the impeller, respectively

r_1 and r_2 = the radii at the entrance and exit of the vane

If e is the pump efficiency, then the power to be supplied to the pump shaft by the motor is (1HP = 550 ft.lb/s in English units and 1kW = 1000 N.m/s in SI units)

$$HP = (T\omega)/(550e) = \left(\gamma Q H_p\right)/(550e); \quad kW = T\omega/1000e = \gamma Q H_p/1000e \tag{37.39}$$

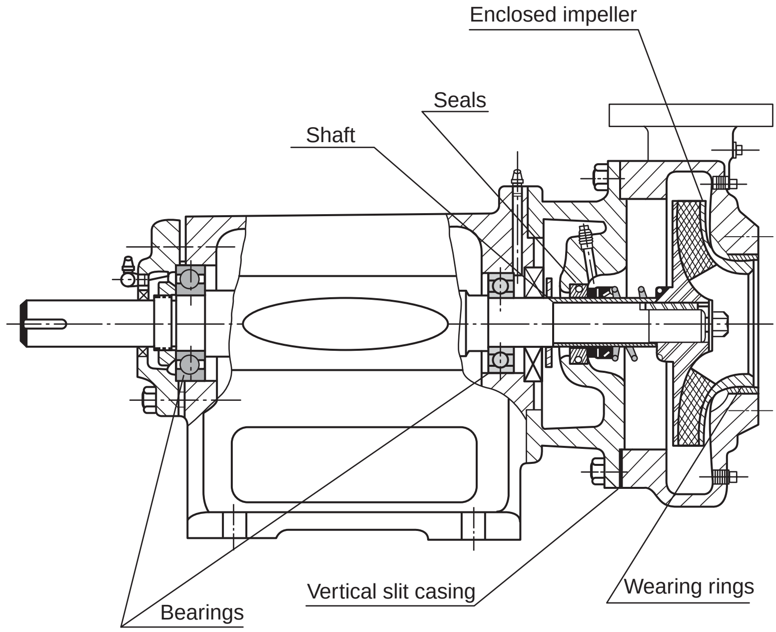

FIGURE 37.33 Radial pump cross section (Adapted from Peerless Pump Co.)

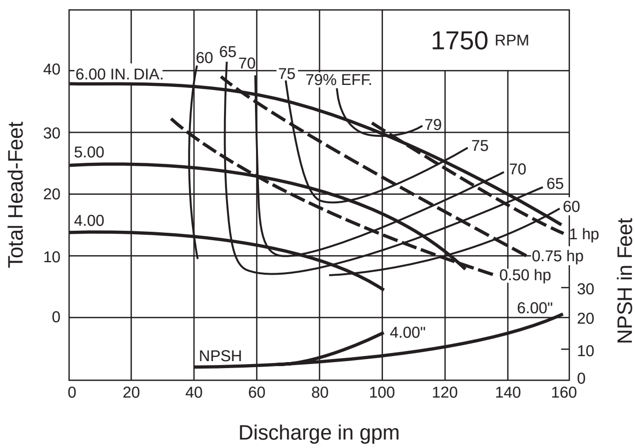

FIGURE 37.34 Characteristic Curves of a Centrifugal Pump. (Adapted from Peerless Pump Co.)

where H_p is the head imparted by the pump to the fluid. H_p can be obtained from the energy equation written between the suction side (subscript *s*) and the discharge side (subscript *d*) of the pump:

$$z_s + p_s/\gamma + V_s^2/2g + H_p = z_d + p_d/\gamma + V_d^2/2g \tag{37.40}$$

Pump Characteristics

Figure 37.34 is an example of pump characteristic curves. Pump manufacturers supply characteristic curves for their several designs and operating speeds. The head at zero discharge is called the shutoff head. The head and capacity corresponding to the maximum efficiency are the nominal head and discharge of the pump. Also shown is the Net Positive Suction Head (NPSH) curve. The NPSH is the difference between the total head on the suction side of the pump above the atmospheric head, p_a/γ, and the vapor pressure head, p_v/γ:

$$NPSH = p_a/\gamma + p_s/\gamma + V_s^2/(2g) - (p_v)/\gamma \tag{37.41}$$

If the liquid surface elevation in the supply reservoir is below the axis of the impeller by a distance z_s, there is a suction lift equal to $z_s + h_{fs}$ (where h_{fs} is the friction head loss in the suction pipe) and the pressure head on the suction side of the pump will be negative, i.e., below atmospheric:

$$p_s/\gamma = -z_s - h_{fs} - V_s^2/(2g) \tag{37.42}$$

To avoid cavitation, the suction pressure head, p_s/γ, must be such that the available NPSH calculated by the above equation be larger than the required NPSH determined from the manufacturer curves or specifications. Equivalently, the calculated value of the cavitation parameter defined as $\sigma = NPSH/H_p$ must be below the critical value supplied by the manufacturer. H_p is the head developed by the pump:

$$H_p = (z_d - z_s) + (p_d - p_s)/\gamma + \left(V_d^2 - V_s^2\right)/2g \tag{37.43}$$

where the subscripts d and s refer respectively to the discharge and suction sides of the pump.

The basic similarity parameter for pumps is the specific speed defined as

$$N_s = N Q^{1/2}/H_p^{3/4} \tag{37.44}$$

where N_s = the specific speed
N = the rotational speed in rpm
Q = the discharge in gp
H_p = the head in feet

The specific speed can be interpreted as the rpm of a homologous pump operating at a discharge of 1 gpm under a head of 1 ft. For multistage pumps, H_p is the head per stage. The specific speed can be derived from the similarity parameter $W_p/(\rho D^5 n^3)$ obtained in Chapter 29 by noting that the power W_p is proportional to $\gamma Q H_p$ and that the discharge Q is proportional to $D^2 H_p^{1/2}$ or equivalently that the diameter D is proportional to $Q^{1/2}/H_p^{1/4}$. Figure 37.35 shows the optimum efficiency of water pumps as a function of the specific speed.

Pump Systems

Pumps generally operate as a part of a system including one or more pipes and perhaps other pumps in series or in parallel. When pumps operate in series the discharge is the same through each pump but the head imparted to the fluid is the sum of the heads imparted by each pump. When the pumps operate in parallel the head is the same but the system discharge is the sum of the discharges of the individual pumps. The operating head and discharge of a pump-pipe system is at the intersection of the pump and pipe discharge-head curves as shown in Fig. 37.36, where Δz is the static lift and Σh_f is the sum of the head losses in the suction and discharge pipes. For further details on pumps and hydraulic machinery see, for example, Krivchenko (1994).

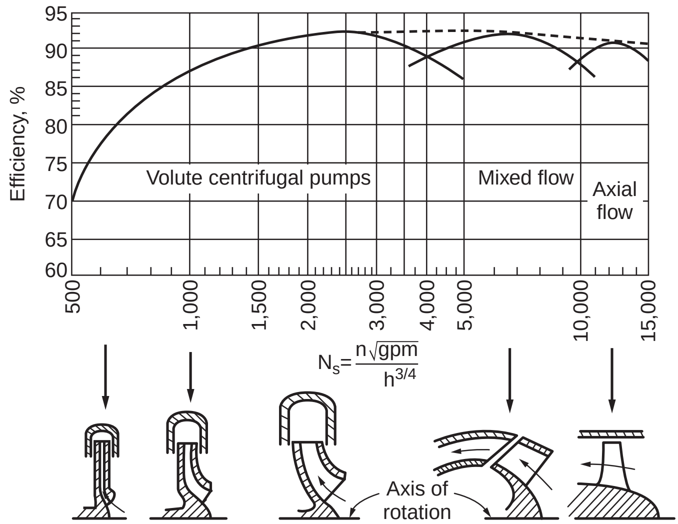

FIGURE 37.35 Optimum efficiency of water pumps as a function of specific speed. (Source: Daugherty, R.L., Franzini, J.B. and Finnemore, E.J. 1985. *Fluid Mechanics with Engineering Applications,* 8th ed. McGraw-Hill, New York.)

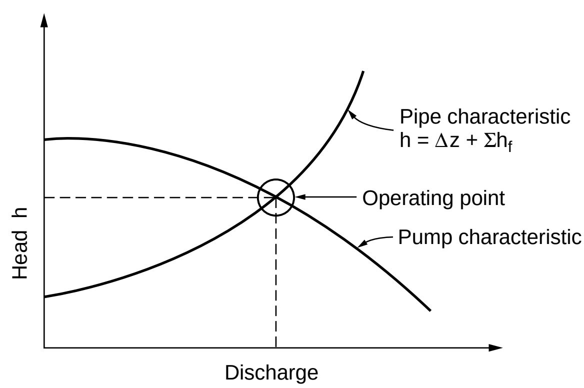

FIGURE 37.36 Pump and pipe system.

Defining Terms

Cavitation — The formation of water vapor cavities due to reduction of local pressure in the water. The cavitation bubbles collapse as they are swept into regions of higher pressure. Continuous implosion of cavitation bubbles severely damages concrete and metal surfaces.

Cofferdam — A temporary dam used to maintain dewatered a portion of a construction site.

Dead storage — That part of the reservoir storage below the elevation of the lowest outlet.

Diversion of flow — Rerouting of flow so that portion of a dam site can be dewatered during construction.

Extrados — Upstream side of an arch dam.

Freeboard — Vertical distance between the maximum probable flood water level or the maximum reservoir setup and wave action level and the crest of the dam.

Intrados — Downstream side of an arch dam.
Kaplan turbine — An axial flow turbine with adjustable runner blades.
Morning glory spillway — A spillway consisting of a circular overflow section leading into a vertical shaft connected to a horizontal tunnel. The vertical cross section resembles that of a morning glory flower.
Nappe — The upper and lower nappes are the upper and lower water surface profiles that water takes as it flows over an ventilated sharp crested weir.
Penstock — A large pipe to carry water to the turbines of a hydroelectric power plant.
Roller compacted concrete (RCC) — A mixture of portland cement, water, aggregate and pozolan with zero slump used in RCC dams.
Run-up — Height to which a wave will rise when it hits a sloping surface.
Setup — Rise in water surface on leeward side of reservoir due to tilting caused by wind.
Stilling basin — A structure, usually located at the foot of a spillway, to dissipate energy over a short distance and reduce flow velocity by means of a hydraulic jump.
Surcharge storage — That part of the reservoir storage above the maximum operating level, usually taken as the elevation of the spillway crest.
Surge tank — Tank designed to contain pressure upsurges due to rapid reduction of flow velocity in a pipeline due to, for example, a quick valve closure.
Tainter gate — A radial gate used to control flow over a spillway.
Uplift pressure — Upward vertical hydrostatic pressure at the interface between the bottom of a dam and foundation or at other interface.
Water hammer — A pressure transient in a pipe system due to a rapid reduction of flow velocity caused, for example, by an adjustment of the setting of a control valve or a change in the operation of a turbine or a pump.

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Further Information

Davis' Handbook of Applied Hydraulics (Zipparro and Hasen, 1993) provides a detailed treatment of reservoirs, natural channels, canals and conduits, dams, spillways, fish facilities, hydroelectric power, navigation locks, irrigation, drainage, water distribution and wastewater conveyance.

Mays' "*Hydraulic Design Handbook*" (1999) contributes design information on many topics including pipe and open channel flows, sedimentation and erosion hydraulics, spillways and energy dissipators, pump and water distribution systems, urban and highway drainage, risk/reliability of hydraulic structures as well as environmental hydraulics and treatment plant hydraulics.

Sentürk's text "*Hydraulics of Dams and Reservoirs*" (1994) presents an extensive treatise on the hydraulic design of reservoirs, dams, spillways and associated structures. Each chapter presents solved examples and proposed problems.

The textbook entitled "*Hydraulic Structures*" by Novak et al. (1996) gives an excellent coverage of dam engineering and other hydraulic structures such as diversion works, drainage and drop structures, hydroelectric power development and pumping stations as well as some discussion of coastal and offshore engineering. Several worked examples are included.

Sturm's textbook entitled "*Open Channel Hydraulics*"(2001) contains a chapter on Hydraulic structures dealing with spillways, culverts and bridges.

The U.S. Army Corps of Engineers "Hydraulics Design Criteria" (1955–70) includes practical formulas, nomographs, design standards on most hydraulic structures.

USCOLD (1991) lists key references for hydraulic design on the following subjects: cavitation in hydraulic structures, increasing discharge capacity of existing projects, spillway design floods, fish passage, gas transfer at hydraulic structures, trashrack vibrations, energy dissipation and terminal structures for spillways and outlet works and thermal stratification and instream thermal simulation.

ICOLD (1980, 1992) has published several bulletins (35, 37, 50, 65, 66 and 86) on the environmental effects of dams and reservoirs. Bulletin 35 is technical while the other reports include case histories in several countries.

The International Association of Hydraulic Research (IAHR, 1987–1995) has published a series entitled *Hydraulic Structures Design Manual* which includes 9 volumes on the following specialized topics:

1. J. Knauss (ed), 1987. Swirling flow problems at intakes.
2. H.N.C. Breuaers and A.J. Raudkivi, 1991. Scouring.
3. E. Naudascher,1991. Hydrodynamic forces.
4. I.R. Wood (ed), 1991. Air entrainment in free surface flows.
5. M. Hino (ed) Water quality and its control.
6. A.J. Raudkivi, 1993. Sedimentation: Exclusion and removal of sediment from diverted water.
7. E. Naudascher and D. Rockwell, 1993 Flow induced vibrations: An engineering guide.
8. D.S. Miller (ed) 1994. Discharge characteristics.
9. Visher & W.H. (eds) 1995. Energy dissipators.

Each important hydraulic structure built by the Bureau of Reclamation of the U.S. Department of the Interior or by the U.S. Army Corps of Engineers is the object of detailed reports. As an example see the "Technical Record of Design and Construction, Glen Canyon Dam and Power Plant" by the U.S. Department of the Interior, Bureau of Reclamation (1970).