sewerage systems. With deep shafts, the annulus of flow may reach terminal conditions where the gravitational component is equalled by the friction at the shaft walls, and so there is a limit to the amount of energy to be dissipated at the base of the shaft.

22.2.7 Air problems in conduits

Air entrained at high velocity releases through gates and valves into conduits, e.g., at outlets from reservoirs, air entering from drop shafts or junctions and air entrained at hydraulic jumps, can lead to dangerous air and cavitation problems unless the conduits are adequately vented. Air can also collect and restrict the flow of water. Air release valves, often combined with vacuum relief to admit air if pressure falls below atmospheric, are therefore provided at high points. Vents are often provided in horizontal tunnels downstream of junctions where entrained air may enter. Air which has collected beneath the soffit tends to be carried forward by the flow, even against a small gradient, but with a variable flow may move upstream and downstream at different times. At vertical shafts in pressure conduits and at deeply submerged exits the intermittent escape of air produces shock waves due to slap on the soffit as water replaces the air. This effect can be minimized by vents for controlled air release.

Hydraulic jumps entrain air and when a jump in a conduit is in contact with the soffit much of the air is released downstream. Following model tests in a conduit with various slopes by Kalinske and Robertson and others, and several observations at full scale, the US Army Corps of Engineers use the formula:

\[ \beta = 0.03(F_i - 1)^{1.6} \]  

(22.12)

which gives higher values than found in the model tests to allow for scale effect. Here \( \beta \) is the air: water ratio \( Q_a/Q_w \), \( F_i = V_i/\sqrt{(gd)} \), \( V_i \) is the upstream velocity and \( d_i \) the effective upstream depth (= water area: surface width). A particular application of these formulae is the estimation of air demand downstream of gates or valves located in closed conduits, where high-velocity flow at part openings is transformed to full-conduit flow through a jump (see Figure 22.23). Full-scale observations in three different cases showed that with rectangular gate openings peak demand occurred at 60 to 85% opening, with a secondary peak at about 5%. Further analysis has been provided by Sharma.

\[ V/Q \]

\[ \% \]

\[ 2\% \]

\[ 10\% \]

\[ V_i \]

\[ d_i \]

\[ V_i = V/\sqrt{gd} \]

\[ \text{Water area} \]

\[ \text{Surface width} \]

\[ a \]

\[ 30\% \]

\[ 1\% \]


The air pumped by the jump may be carried downstream by the full-bore flow but, if the velocity is insufficient for this, air will collect immediately downstream of the jump and when a quantity of air has accumulated it will 'blow back' through the jump. Figure 22.23 shows the limiting conditions for the air just carried by the flow, as found by Kalinske and Robertson.

Sailer compared these curves with conditions in a number of full-scale inverted siphons and found verification in that five cases where blowback had occurred were represented by higher values of \( (F_i - 1) \) than shown by the curves, while others giving no trouble were on or below the curves. With large flows, blowback through the jump is, like 'blowout' at the exit, explosive and potentially dangerous.

22.3 Spillways

22.3.1 Purpose and types

A spillway is provided to remove surplus water from a reservoir and thus protect the dam and flanking embankments against damage by overtopping.

The best type and location of a spillway depends very much on the topography and geology of the dam site and adjoining area, and on the type of dam. Where the dam is of concrete or masonry founded on hard rock, the spillway may be within the dam, consisting either of a high-level overflow or of submerged orifices, discharging into the river bed beneath. In the case of an earth or rockfill dam, it is usual to site the spillway away from the deepest part of the dam; high-flanking ground or a saddle away from the dam site can be suitable locations where a spillway channel may be excavated and control structure provided (see, for example, Figure 22.24). Where the dam is built in a narrow gorge and there is no suitable separate site for the spillway, a side-channel spillway is often adopted (Figure 22.25).

If control is by a fixed ungated weir, the maximum retention level of the reservoir is the weir crest level; at times of spill the reservoir level rises and sufficient freeboard has to be allowed above maximum water level, which is the level at which the design maximum flood discharge is released. In the case of gate-regulated spillways, on the other hand, flood flows can be discharged with reservoir at retention level, which need never be exceeded. For a given dam height, retention storage can thus be greater but, because there is less flood storage, the spillway capacity also may have to be greater. The gates, however, enable the reservoir to be drawn down in advance of a flood peak, given adequate forecasting. Low-level orifices, having greater capacity than required for purposes of normal supply, have greater capability than has a gated crest overflow in drawing down a reservoir in the event of damage to the dam, an important aspect in areas where earthquake risk is present. But crest overflow weirs have a greater rate of increase of capacity as a reservoir level rises above normal, thus providing additional safety margin.

Cost is a major consideration in the choice between a regulated and an unregulated spillway, but spillways without gates have advantages in respect of reliability, absence of mechanical maintenance problems and no power requirements. They are therefore often adopted at remote sites and for small dams where the cost of gates would not be justified.

Siphon spillways carry some of the advantages of both gated and ungated spillways. They can be designed to prime and operate to maximum discharge within a small range of reservoir level and they are automatic, with no moving parts.

Another type of spillway, particularly suited for use with earth or rockfill dams is the bellmouth or 'morning glory' spillway, which can be built quite independently from the dam, and which is described further in section 22.3.5. If the reservoir is for water supply, the bellmouth and shaft are often combined in the same structure as a draw-off tower and the low-level tunnel can be used for river diversion during construction, as discussed and illustrated in section 22.4.1.

In many cases it is advantageous to provide more than one spillway. Instead of relying on a single spillway to control all
floods up to catastrophic, it may be safer and more economical to provide a main or service spillway, fully regulated and capable of controlling all floods up to perhaps a 20- or 50-yr return period, and a secondary or emergency spillway which will bring the combined capacity to the catastrophic level. The capacity of the emergency spillway should be adequate to control normal inflows alone for a reasonable period should the main spillway be damaged.

22.3.2 Channel spillways
The simplest form of spillway, consisting of a channel excavated in rock, is often used for small reservoirs and for emergency spillways. The control is usually provided by a hard sill or weir at the entrance. The channel downstream should be given sufficient slope to ensure that the weir will not be drowned by backwater effect. If the rock is erodible, a curtain wall with bed protection or stilling basin is needed to avoid erosion undermining the downstream end of the weir.

In the case of emergency spillways, a 'fuse plug' is often provided, consisting of an erodible bank across the channel. Its crest is below the dam crest level but above normal operating level. When overtopped it quickly erodes down to the level of the hard silt, bringing the full discharge capacity of the channel into operation. To avoid excessive draw-down of the reservoir, emergency spillways of this type are preferably wide and shallow. Bed and bank protection are usually minimal but it is essential that there is no risk of erosion progressing upstream and breaching the reservoir rim beneath the level of the hard silt.

22.3.3 Weirs
These may be any of the types described earlier on page 22/5 but are generally of the free-nappe profile type. It is usual for gates to close on to the weir face slightly downstream of the highest point of the crest, so that the jet at small openings will be projected downwards. Even so, sub-atmospheric pressures can develop, and for this and other reasons it is often desirable to avoid prolonged releases under high heads with small gate openings. Provision of separate sluices or valves for small releases is preferable.

Piers affect the rating curve, as described earlier (page 22/6). A typical rating curve is shown in Figure 22.26 (AB).

A side-channel spillway consists of a weir discharging into a parallel channel, as in Figure 22.25 where the weir is aligned on a ground contour normal to the axis of the dam. The channel must be of sufficient width and depth to allow for energy dissipation and the generation of exit flow without drowning the weir. Calculation is based on the momentum principle. Side-channel weirs have been investigated by El-Khashab and Smith.51

22.3.4 Low-level outlets
These may discharge through a dam or original ground into conduits, chutes or stilling basins. They are generally of rectangular section and regulated by radial gates (see, for example, Figure 22.27). When the reservoir is drawn-down so that the orifices are not fully submerged, flow is of the free-surface weir.
type. The rating curve of an orifice spillway with gates fully open therefore consists of two main parts, as may be seen from the example in Figure 22.26. The lower part CD relates to weir flow and the upper part EF to submerged orifice flow. Between the two is a transition DE. The discharge for weir flow is 
\[ Q = c_1(2g)\frac{BH_1^5}{5} \]
where \( Q \) is the discharge, \( c_1 \) is the weir coefficient, \( B \) is the width and \( H_1 \) is the upstream head above the sill. \( c_1 \) is generally a variable.

In the range of orifice flow, 
\[ Q = c_2\frac{B}{(2gH_2)} \]
where \( c_2 \) is the coefficient of discharge and \( H_2 \) is the effective head below reservoir level. If the jet springs clear, so that atmospheric pressure obtains around the whole periphery, \( H_2 \) is best measured from the centre of the jet at its exit from the orifice. If the jet emerges on a horizontal floor confined within vertical side walls, \( H_2 \) is more correctly measured from the soffit level as representing the effective elevation plus pressure head over the jet at the point of separation from the soffit. If there is back pressure \( H_2 \) is the differential head.

The requirements in design are those of high-pressure outlets, described on page 22/27. If these are met the coefficient can approach unity. In the case of Mangla spillway\(^4\) (Figure 22.27) it was approximately 0.95. Model tests are used to indicate pressures on the boundary surfaces and provide rating curves for full and partial gate openings.

### 22.3.5 Bellmouth, shaft and closed-conduit spillways

A bellmouth or ‘morning glory’ spillway normally consists of an overflow weir, circular in plan, but in some cases multisided, and a vertical shaft discharging into a tunnel or culvert carried through high ground with outfall into the downstream river. The weir can be provided with gates or siphons – Figure 22.28 shows an example of the latter.

The hydraulics of bellmouth and closed-conduit spillways are complicated by the number of potential controls and the entrainment and release of air. At low flows the bellmouth crest provides weir control; at a higher stage the throat at the foot of the bellmouth can exert orifice control; the bend at the foot of the shaft leading into the tunnel can also exert orifice control and if the tunnel flows full this may well control the discharge. To avoid instability due to controls operating intermittently, the range of each control should be clearly defined with stable transitions from one to another. It is best to reduce the number of potential controls to one or at most two. The weir (or siphons) provide the primary control and it is normal practice for the design maximum discharge to be reached in this range, with an adequate margin, before the weir is drowned by ‘gorging’ in the shaft and bellmouth due to controls in the system downstream. Similar considerations apply to spillways which are not of the bellmouth type but which have weir, gate or siphon as a primary control, discharging into shaft and tunnel. If, however, use is to be made of flood storage in the reservoir at higher levels, a bellmouth spillway may be allowed to become completely submerged.

As in the case of straight weirs, sharper curvature raises the coefficient of discharge. Profiles based on the shape of under-nappe of a jet have been designed for weirs circular in plan.\(^{12,25}\) In several cases of bellmouth spillways measures were necessary to reduce swirl and prevent vortex formation particularly at the highest flows, when it could greatly reduce discharge capacity. Vortex flow is induced by asymmetrical approach in the reser-
A typical rating curve is shown in Figure 22.26. Weir control is represented by the curve AG. At low flows there is a free surface flow in the bend and tunnel but with rising discharge and downstream conduit not flowing full the bend begins to act as an orifice with water level rapidly rising in the shaft. When it reaches crest level it begins to drown the weir flow. The bellmouth is then said to be 'gorged'. This is represented by the intersection of the two curves at G, above which the bend assumes control of the rate of flow. The rating curve of the spillway is therefore AGH with a short transition at G representing drowned weir control and bend orifice control at H.

If the bend is too sharp, flow from the bend to the conduit is very disturbed; if it is too easy the downstream culvert may flow full; a bend radius of 1.5 to 2 times the diameter is generally satisfactory.

For proper control of flow at the bend and smoother flow in the conduit a deflector may be placed on the inside wall at the upstream of the bend. Where the conduit is used for river diversion during construction, a properly shaped bend can later be formed when the diversion intake is plugged. Unless the conduit is very short, with a free surface is desirable with sufficient air space above for entrained air to be released without trouble. Sufficient slope should be provided to ensure that the depth does not exceed the desired limit. As the result of a model study, Mussalli and Carstens recommend upper limits for the proportion of water flow in such conduits, ranging from 97% of the area, when the Froude number is 2, to 50% when it is 8.5. Where the velocity is high enough to entrain air it is desirable to provide an air vent at or near the bend. With high velocities it is also best to avoid bends and other conditions downstream which could cause a hydraulic jump to form in the conduit.

It is evident that the concrete surface at the base of the shafts of bellmouth spillways can be subjected to high impact loads by water, possibly with ice and logs, spilling from a great height. In some cases steel or cast-iron lining has been provided in this area, but from a survey of sixteen bellmouth spillways, of which eight with unlined concrete inverts had undergone a fair test, Bradley found no erosion of a serious nature. Dense concrete with smooth surface finish is called for here and in the conduit.

### 22.3.6 Siphon spillways

Compared with a free-surface weir, flow through a siphon reaches a high rate of discharge per unit width with only a small rise of reservoir level needed to prime the siphon. This permits a higher retention level for a given maximum water level, or alternatively a higher concentration of flow in a restricted width.

Reservoir retention level is equal to siphon crest level. As the reservoir level rises, the action of a siphon passes through the following successive phases: (1) weir flow, when water spills at low depth over the crest; (2) priming phase, when air is being extracted from the crown of the siphon; and (3) fully primed siphonic flow. When the reservoir falls, (3) gives way to (4), a depriming stage, when air is admitted in sufficient volume to break the siphonic action and the action vellerts to (1), weir flow. In recent years, many air-regulated (or partialized) siphons have been built. In these, phase (3) consists of two parts: in (a) when priming has occurred the entry of air continues so that the flow consists of an air–water mixture. The air intake is so designed that the volume of air admitted is insufficient to break the siphon (except at low flows) but is controlled by very small variations in reservoir level. As the reservoir rises further the volume of air is reduced until stage (b) is reached when the siphon flows 'blackwater', i.e. with no entrained air. This performance is illustrated in the typical stage: discharge function in Figure 22.29a. The advantage of air regulation is that, whereas without it the siphon on priming runs directly to a high blackwater discharge which if in excess of inflow will draw the reservoir down and lead to intermittent priming and depriming, an air-regulated siphon will remain in the fully primed phase over a wide range of flow, with continuous discharge matching the rate of inflow. Examples of air-regulated siphons are Eye Brook, Shek Pik (Figure 22.28) and Plover Cove (Figure 22.29b).

![Air-regulated siphon spillway](https://example.com/air-regulated-siphon-spillway.png)

**Figure 22.29** Air-regulated siphon spillway. (a) Typical stage: discharge curve; (b) Section through Plover Cove siphon spillway, Hong Kong (Consulting Engineers: Binnie and Partners)

Spillway siphons are generally designed to prime automatically when the reservoir has risen to a level such that: (1) an upstream air inlet is submerged; (2) the siphon outlet is sealed by a deflected jet or a downstream weir; and (3) the flow is sufficient to entrain and remove air from the crown of the siphon. Various priming devices have been used and the priming depth above crest level is in some cases as little as one-sixth of the throat diameter.

The blackwater discharge capacity of a siphon can be expressed as:

\[
Q = cA \sqrt{(2gH)}
\]  

(22.13)

where \(c\) is a coefficient allowing for head losses, \(A\) is the cross-sectional area of the flow at exit and \(H\) is the head from upstream reservoir to effective exit level — usually the downstream lip of the hood.

Ackers and Thomas reviewed the design and operation of siphon spillways. The value of \(c\) obtained in the Plover Cove study, Mussalli and Carstens obtained limits for the proportions of water flow in such conduits, ranging from 97% of the area, when the Froude number is 2, to 50% when it is 8.5. Where the velocity is high enough to entrain air it is desirable to provide an air vent at or near the bend.
siphon model (Figure 22.29b) was 0.68. That of the model of Shek Pik bellmouth siphon (Figure 22.28), where the shape was radial and no sealing weir was provided, was 0.66.

Surface waves in the reservoir are an important factor in siphon design. Model tests showed that despite provision of baffles, waves caused surging in the siphon but the air intakes could be designed to counterbalance the effects of surging and wave wash. The head in siphons is usually limited to about 7 m to avoid cavitation at the crest. Tests on the Plover Cove siphons showed that wave action resulted in transient pressures below average pressures, but a total head of 7.3 m was still feasible. Each case should, however, be examined in the light of the particular conditions obtaining.

22.3.7 Chutes

Chutes may be built into the downstream faces of concrete dams; longer chutes are often provided to convey the flow from side channel or other flanking spillways to the river bed downstream (see Figure 22.24). In general, high head spillways with chutes should not be used for routine releases of water for supply, because of the risk of cavitation damage with small gate openings, also because the chute may have to be taken out of service in the dry season for repairs, and because low flows can create problems of erosion downstream. The gradient is likely to be steep enough to generate high-velocity flow. As lateral changes of direction could result in overtopping of the side walls, any essential changes in the alignment should be made near the control structure where the velocity is relatively low and thereafter the chute should be straight. There should also preferably be no changes of alignment in the side walls where flow is supercritical because diagonal shock waves would be created which might cause overtopping downstream.

High-velocity flow can give rise to high pressure and the most careful precautions are necessary to avoid uplift pressures developing beneath the chute slabs. A high standard of surface finish is called for and the profile should contain only very gradual curvature. Joints between slabs should be keyed and bridged by flexible water stops sealed at intersections. Projections at the joints facing upstream should not be allowed but offsets facing downstream up to 12 mm are often accepted or even specified. Drains are generally provided beneath and parallel to all joints, so that in the event of leakage, uplift pressure cannot build up. For additional protection, chute slabs are often anchored to the foundation rock. Special care is needed where the chute rests on jointed or fissured rock because pressure can be transmitted from leakage at a higher level despite underslab drainage. Chutes at a steep slope are especially vulnerable and may call for deep anchors. Stability should be checked for all possible modes of failure.

The problems of cavitation caused by high-velocity flow are reviewed on page 22/17. Severe damage has occurred on some chute spillways which have been operated at high velocity and hence a careful assessment of the cavitation risk is needed for any chute spillway, especially if the total fall exceeds 50 m. Cavitation damage can be avoided if natural aeration of the flow from its free surface is high enough to provide several percent of volumetric air concentration at the bed of the chute in the region of high velocity.

The development of flow down the spillway is illustrated in Figure 22.30. A layer of slower-moving fluid influenced by friction at the solid boundary grows in thickness beyond the crest until it occupies the full depth of flow. This defines the 'point of inception' of air entrainment by turbulence at the surface. The entrained air diffuses down within the partially aerated region of flow to occupy the full depth of flow, usually some considerable distance down the spillway. It is only in this fully aerated region that the natural aeration at the solid boundary can be sufficient to prevent cavitation damage if the cavitation index (see page 22/32) drops below the safe limit. The likelihood of cavitation damage therefore depends on whether velocities rise too high as the flow accelerates down the chute before there is sufficient aeration at the bed. This is most likely at high discharge intensities. A method of calculation of the

![Figure 22.30 Development of flow down a typical spillway](image-url)
growth of boundary layer is given by Wood, Ackers and Loveless.\textsuperscript{59} If natural aeration is not sufficient, purpose-built air-entraining slots should be provided, fed from ducts at either side.\textsuperscript{64,65}

Calculation of depth of flow in chutes may be done by a step method beginning at the top, using curves showing energy against depth for the required discharge per unit width, similar to that of Figure 22.1. The calculation should be performed with two roughness coefficients, one representing a maximum, for use in determining side-wall height, and the other representing a minimum, for use in the design of energy-dissipating works. The height of side walls should include an allowance for bulking due to air entrainment. The US Corps of Engineers\textsuperscript{66} provide a design curve based on observed data, with the equation:

\[ c = 0.436 \log_{10}(S/q) + 0.971 \]  

(22.14)

where \( c \) is the ratio of air volume to air-plus-water volume, \( q \) the discharge per foot width in square metres per second, and \( S \) the sine of the angle of chute inclination.

### 22.3.8 Energy dissipation

The energy to be dissipated at the outfall from a spillway is very considerable. The means of protection of the dam and other structures from its erosive action depend largely on the rate of discharge and its head, the erodibility of the materials of the river bed and surrounding ground and on the proximity of the dam.

In the case of rivers in alluvium or other easily erodible ground, a stilling basin designed to contain a hydraulic jump is often provided. This may be of the rectangular type or a submerged roller bucket (see sections 22.1.6.1 and 22.1.6.2). The former is generally more efficient but more costly, especially in the case of large structures where deep retaining walls would be required. Where the river bed material is rock a ‘ski jump’, trajectory or ‘flip’ bucket is generally provided. This is elevated above maximum tailwater level, so that the jet trajectory carries the water into a plunge pool some distance from the bucket. Dissipators of this type are generally less costly and are suitable where the bedrock is resistant enough so that erosion does not progress back and endanger the foundations of the dam or other structures. Ski jumps have also been used where the river bed is of alluvium, and the structures are suitably protected against erosion.

The radius of flip buckets is not critical provided it is large relative to the depth of flow. The exit angle is important as it determines the throw distance; the angle is generally between 20 and 40° and the theoretical throw distance is given by \( x \) in the formula:

\[ \frac{x}{h} = \sin 2\theta + 2 \cos \theta \left( \sin^2 \theta + \frac{V}{h} \right)^{1/2} \]  

(22.15)

where \( h \) is the velocity head at the bucket lip, \( \theta \) the bucket exit angle, measured from the horizontal, \( y \) the vertical height of bucket lip above tailwater.\textsuperscript{11}

However, because of air resistance and internal shear the jet tends to break up and diffuse so that the actual trajectory distance may be 10 to 30% less than indicated by the formula. Elevators\textsuperscript{67} quotes examples of models and full-scale spillways.

Flip buckets are not necessarily of circular profile, nor axisymmetrical. There are several instances of buckets composed of flat deflecting surfaces, some designed to deflect to one side to suit downstream requirements.\textsuperscript{64}

In the design of the side walls, allowance must be made for the additional lateral pressure due to centrifugal force. This may be calculated by methods of Gumensky,\textsuperscript{68} Ballofet\textsuperscript{69} or the US Army Corps of Engineers.\textsuperscript{70} To avoid cavitation damage it is usual to avoid the use of teeth, and in some cases the lip edge is protected by stainless steel.

The size and depth of the plunge pool depend primarily on the discharge concentration and the characteristics of the materials which are eroded. In the formation of a deep pool the eroded material is lifted out by the flow. Incoherent alluvial materials are readily removed and form flat side slopes. Rock disintegrates into fragments by transient pressures in the joints and the fragments are reduced by abrasion until small enough to be removed.\textsuperscript{73,74} A plunge pool in rock has relatively steep side slopes and is less extensive in plan than one in alluvium. The erosion is not always confined to the plunge pool because the action of the jet creates large horizontal eddies which can extend back to the chute. Small flows and flows at low heads have shorter trajectories or may not be sufficient to sweep out of the bucket but spill over the lip, causing erosion beneath. In such cases special protection is needed (see, for example, Figure 22.23). Mason\textsuperscript{75} has reviewed experience of energy dissipation works at dam outlets.

### 22.4 Reservoir outlet works

#### 22.4.1 Intakes

The type of intake for drawing water from a reservoir depends on the type of dam and on the purpose of the supply. The velocity may be low and against a back pressure as, for example, in intakes for domestic water supply, and into penstocks for power generation, or it may be high, e.g. in spillways and into diversion tunnels during construction. With high velocity, special problems concerned with head loss and cavitation arise. If the dam is of concrete, the intakes may be located in the dam. If the dam is of earth or rockfill, a separate intake structure may be built (see Figure 22.31), leading into a tunnel, or a free-standing drawoff tower may be provided, sometimes combined with a shaft spillway as in Figure 22.32. A free-standing tower is particularly suitable where drawoff is required at several levels, as where the water is for domestic supply. In such cases a bottom drawoff or ‘scour’ sluice is generally provided; this is opened at intervals to prevent sediment from building up a deposit in the immediate vicinity of the lowest drawoff to supply.

Deep intakes have advantages in that they will remain submerged at low reservoir levels, are less affected by vortices and are less susceptible to obstruction by ice and floating trash. Against these, the gate structure is more costly and access to the screens for cleaning more difficult.

A square edge or small radius edge to an orifice would result in flow separation and a \textit{vena contracta}, so orifices and sluice entrances, whether circular or rectangular in section, are usually shaped to a bellmouth. The head loss associated with the formation of a \textit{vena contracta} at a circular orifice can be greatly reduced by providing a simple bellmouth, as shown in Figure 22.33a, but for high velocities the curvature should be less to avoid low pressures which might result in cavitation damage. Compound curves of two or more radii and elliptical curves are often suitable profiles. A typical example of an elliptical profile is shown in Figure 22.33b. With this profile the minimum pressure at the boundary is approximately \( 0.1V^2/2g \) below the corresponding pressure in parallel flow in the orifice downstream where \( V \) is mean velocity. As this is a mean pressure, lower pressures may occur owing to fluctuations. For very high velocities this may not be acceptable and the profile may be...
based on the profile of a jet springing from a sharp-edged orifice or may be compound elliptical. In the case of important works, especially with high velocities, intake entry curves are usually tested in hydraulic models. Control may be by gate or valve, located at the intake or in a pressure conduit. Radial gates in rectangular orifices, as seen in Figure 22.27, are particularly suited for flood releases. If the gates are to be used for regulation at part openings under heads exceeding 10 m, the inverts immediately downstream are generally lined with steel as protection against cavitation. A second gate is often provided upstream of each service gate for emergency closure and to allow maintenance work on the service gate to proceed when the reservoir is at a higher level. This is generally a vertical-lift gate closing on to a steel sill but requiring side slots. The latter contribute to head loss and, where cavitation is a danger, require special design as, for example, is illustrated in Figure 22.34.

Where outlets fill a vital role in the safety of works, the possibility of failure or malfunction of a gate or valve should be considered and alternative measures provided for an emergency.

**22.4.2 Vortices**

Though a slight surface swirl may be of no consequence, a vortex with an air core extending to an intake can be harmful in reducing the discharge capacity of the intake, causing gate vibration or resulting in admission of air to pumps or turbines. Any tendency for a vortex to form in a model test should be carefully investigated because vortices form more readily and develop further at full scale.

A free vortex tends to form in accelerating flow towards a region of low pressure, as at a submerged intake. It is facilitated by boundary geometry consistent with vortex shape, and by an initial circulation in the reservoir, and is more marked the greater the pressure drop to the outlet relative to the depth below surface. Vortex action is reduced by deeper submergence of the intake, by reduced velocity at the intake and by obstructions to the rotation, such as horizontal grids and projecting walls.

Vortices are also a problem in pump sumps where even a slight swirl may affect pump efficiency and more refined measures are needed. Velocity of approach is generally limited to 1 m/s, but eddies can still form at points of separation. Sharp wall angles and regions of dead water should be avoided and expansions should be gradual. General guidelines are available but model tests are often needed to determine optimum pump sump geometry.

There are situations where vortices may be used to advantage, as in the vortex drop structures described earlier.

**22.4.3 Screens**

Screens or trash racks are provided at intakes to hydro-electric plants, pumps and water-treatment works. Log booms are often placed upstream, but it is generally required to intercept small debris and possibly fish. The spacing of the bars may be 2 to 20 cm depending on the duty. The main requirements in design are that the bars are stiff enough not to vibrate and are arranged for easy cleaning. As a general guide for screen area the mean velocity is usually limited to 0.6 m/s or less. Vibration is avoided if the dimensions of the bars are such that the natural frequency of the bars is higher than the forcing frequency. Screens may be fixed and cleaned by raking or lifted above water for cleaning. For ease of cleaning from above, the vertical bars generally project upstream of the lateral bars.
Figure 22.32  Combined drawoff tower and spillway, Seletar Reservoir, Singapore. Air-controlled syphons are used instead of valves for drawoff purposes. (Consulting Engineers: Binnie and Partners, Malaysia)

Figure 22.33 (a) Simple bellmouth; (b) elliptical roof profile for conduit intake with parallel sides and horizontal floor. (After US Army Corps of Engineers (1952–70) *Hydraulic design criteria*. US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi)

Figure 22.34  Typical gate slot with downstream offset to minimize cavitation. (After US Army Corps of Engineers (1952–70) *Hydraulic design criteria*. US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi)
22.5 Gates and valves

22.5.1 Gates

22.5.1.1 Uses and types

Gates are used to control flow in open channels or closed conduits by restricting or closing the waterway. They may be required:

1. For regulating the flow, when they must be capable of operating at any required degree of opening.
2. For emergency or guard purposes, when they must be capable of closing under any condition of runaway flow which could occur.
3. As bulkhead gates for closing a conduit for inspection, maintenance or construction works. When they are permanent installations, such gates are generally designed to open and close only under balanced pressures, but when used to close diversion tunnels during construction works, closure may be against a considerable flow. Stop logs are similar in function to bulkhead gates, but are in smaller units handled individually and placed above one another.

The types generally used in outlets from reservoirs and in spillways, barrages and canals are as follows.

Vertical lift gates. Vertical lift gates are supported by guides in slots at the side walls of the conduit. They may open by raising or by lowering; in some cases they are in two or even three parts, each operating independently. They may have sliding contact with the guides or may have wheels (fixed-wheel gates) or a moving train of rollers (Stoney gates). They have seals at the sides and (in orifices or closed conduits) also at the top and generally close on to a steel sill. Where it is required to allow passage of floating debris, or sensitive control of reservoir level, the top of the gate may consist of a hinged flap opening downwards. They are widely used for both weir and orifice control in spillways. They are also sometimes used in pressure conduits but need more space than vertical lift gates and problems of access and removal for maintenance have to be considered.

Hinged leaf, bascule or flap gates. These are sometimes used for crest control where water depth is not great. They are hinged at the bottom and may be used for regulation with water spilling over them; they need venting. They have the advantage of allowing floating debris to pass at small openings but, although they can be of curved profile, the weir crest has to be rather wide to provide the recess. On many European rivers, bascule gates are used for regulating upstream water level, operated by hydraulic actuators located below the weir crest. Hinged gates can be made to open automatically by a simple mechanical device when the upstream water level rises to a given height. Gates hinged at the top are also used where the whole assembly is retractable to allow the passage of ships.

Drum and sector gates. Examples of these can be seen in Figure 22.35. These are crest gates which open downwards, retracting into a recess in the crest. They may be hinged on the upstream (drum) or downstream (sector). The upper surface can be shaped to suit the weir profile when fully open. In the examples shown, the gate consists of a watertight vessel con-
controlled by application of headwater pressure beneath; it is sealed at the hinge and gate seat. These gates can be arranged to operate automatically by the upstream water level. (The sector gates of the Thames Barrier are illustrated in Figure 22.11, page 22/11.)

**Bear-trap gates.** When raised, bear-trap gates are in the form of a flat ‘A’, with upstream and downstream leaves forming the two legs, hinged at the bottom, with seals at the hinges and the apex. The gate is raised by admitting water under pressure from the headwater. When lowered the upstream leaf overlaps the downstream leaf and both fold flat. Bear-trap gates have for long been used in Europe and the US for river regulation.

**Rolling gates.** Rolling gates consist of a roller with toothed-wheel meshing with an inclined toothed rack at each end. The gate is rotated by a chain and accordingly moves up and down the racks. Roller gates have been used for river regulation.

**Cylinder or ring gates.** Cylinder or ring gates moving on vertical axes have been used as crest gates on bellmouth spillways and for bottom outlets. Some of the former open by being lowered vertically into a recess in the weir crest, controlled by water pressure, others and the bottom outlet gates by being lifted from above. Lateral control of the gate motion is provided by guides.

### 22.5.1.2 Partial gate openings

These are orifices of which three sides are the fixed boundaries and the fourth is the gate lip. When unsubmerged, the jet springs clear from the gate lip forming a *vena contracta*, the dimensions of which depend on the shape of the gate skin and lip and on the upstream profile of the sill. It is therefore usual to calibrate gates by model tests. Details of calibration of various gates are available. Where gates are partially submerged downstream, calibration is complicated by the additional variable, and is also less reliable because of possible variation of flow pattern. Submergence may also lead to vibration problems.

### 22.5.1.3 Vibration

In general, vibration is a result of resonance where the frequency of a pulsating force is equal or nearly equal to the natural frequency of a flexible part of the structure. Gates are liable to vibrate when: (1) overtopped and not adequately vented; (2) when significant flow occurs both over and under a gate; (3) when the gate is partially or fully submerged; (4) when the location of flow separation is unstable; or (5) there is flow re-attachment. Vibration by the latter causes may occur at the bottom edge of a lifting or radial gate which should therefore be designed so that the flow separates at a sharp edge and cannot become re-attached by contact elsewhere. This is not always possible at small openings, so that vibration may occur in a limited range of opening. Flexible seals are a potential cause of instability and are often omitted from the bottom edges of gates for this reason. Many cases of gate vibration and remedial methods have been described.

### 22.5.1.4 Downpull and upthrust forces

These forces can act on the upper and lower edges of a gate as well as on the face. They affect the operating forces required and gates are often designed so that the resultant force is of assistance. In particular, if the top edge of a lifting gate is subjected to static pressure whereas the bottom edge is at atmospheric pressure because the lip is on the upstream edge, the resultant downpull assists in gate closure, a safety measure in case of power failure. Pressures measured on bottom edges of various shapes are available.

#### 22.5.1.5 Gate seals

Where a small amount of leakage can be tolerated, as in most works in the open, the seal at the bottom of a lifting or radial gate is usually metal to metal, between the gate edge and a steel sill set flush in the floor. A bottom slot would fill with debris and a projecting rubber seal may vibrate, but if leakage is to be minimal a rubber seal may be inset flush in the floor. Side and top seals can be metal to metal but with close tolerances these may be costly. Flexible rubber seals are therefore frequently used, in the form of strip tightly clamped with small projection, or moulded into bulbous shapes (e.g. music-note type) and arranged to be held in contact by the water pressure. As frictional resistance between metal and rubber seal increases with pressure, brass cladding is often used for sliding seals where the head exceeds 60 m. For very high heads, metal-to-metal contact may be required.

### 22.5.2 Valves

#### 22.5.2.1 Uses and types

Valves are used to regulate flow or pressure in pipes and conduits or to close them against flow, often at high pressure. A service valve, whether for regulation or closure, is generally protected by an upstream gate or guard valve which can be closed against flow to isolate the regulating valve for maintenance or repair, and to prevent leakage if the regulator is not adequately sealed. Valves may be ‘in-line’, i.e. with pipeline upstream and downstream, or ‘terminal’ at the discharge end of a pipeline. Variations in valve design are numerous; only a few types which are normally used in reservoir outlet and hydroelectric power systems are described below. Of these, gate, spherical, butterfly, needle and tube valves are generally used in-line, while needle, tube, hollow jet and sleeve valves are used as terminal regulators.

The discharge capacity of a valve may be expressed as:

\[
Q = CA_H/(2gH^2)
\]

where \(Q\) is the discharge, \(C\) the coefficient, \(A\) the cross-sectional area of the valve at entry and \(H\), the head loss across the valve

#### Gate or sluice valves

In their simplest form (Figure 22.36) these consist of a sliding leaf in a valve body with side slots, thus resembling a vertical lift gate with operating rod sealed to contain the pressure. The sealing contact is metal to metal and the leaf is usually wedge shaped to provide tight sealing when fully closed. Parallel guides prevent vibration at part openings, but gate valves are not suited for regulation except at low or moderate pressures. A bypass is usually provided to balance pressures but a valve for guard or emergency duty may have to close under unbalanced pressure. Advantages of gate valves include the simplicity of design and low head loss when fully open. Disadvantages are the considerable power required to operate under unbalanced heads, cavitation damage to the slots at high velocity and damage by abrasion of the sealing faces if sediment is carried in the flow. The main disadvantages of slots are overcome in the ‘ring-follower’ valve in which the gate when raised is followed by a cylindrical ring which effectively covers the slots. It requires a cavity for the ring beneath the conduit and is suitable only for guard purposes.

For regulation under high heads a special type of gate valve, termed the ‘jet-flow’ gate valve, has been devised by the USBR to operate free of cavitation. The flow is expanded, then sharply
contracted, at the boundaries upstream of the gate slot and springs clear into a vented surround downstream. Valves of this type have been successfully used in sizes up to 2.44 m diameter and under heads to 120 m.88

Spherical or rotary plug valves. These are used for guard and on-off duties. They generally consist of a short length of tube of the same diameter as the conduit and length about the same dimension, which is in line with the conduit in the open position and is rotated through 90° to effect a closure. This is enclosed in a body of roughly spherical shape. The great advantage of this valve is that it offers no obstruction to the flow when in the fully open position. Hydraulic characteristics are given by Guins.89

Butterfly valves. Butterfly valves (Figure 22.37a) are widely used as guard or isolating valves but under low- or medium-pressure differentials they can be used for regulation. The blade or disc is mounted on a shaft, or on two stub axles; rotation by hydraulic piston and crank is simple and direct. The obstruction caused by the blade in the fully open position inevitably results in head loss and downstream turbulence. The former is reduced by adopting a slim blade of greater diameter than that of the pipe. However, a slim blade may not have the strength to resist high-pressure loading, so to meet this need, blades in some valves consist of two thin parallel discs rigidly connected by structural members parallel to the flow; these have great strength while offering less resistance to the flow than corresponding solid blades. Where the valve is in a terminal position, or is guarding a terminal valve, the pressure of the surrounding fluid may be near atmospheric; this also calls for slim blades to avoid cavitation at high velocities. The resultant torque due to fluid pressure is always acting to close the valve, but for safety when closed it is best for the lower half of the disc (if the axis is horizontal) to close in the direction of flow. Guard valves are often designed to be opened under balanced pressure, for which a bypass valve is provided, but to close against full flow in case of emergency. Butterfly valves present special sealing problems because of the axles but this problem has been overcome and very low leakage rates have been achieved88 with rubber seals mounted on disc or body. In some valves the axles are offset to facilitate replacement of the sealing ring. For very high heads, rubber is not suitable and metal-to-metal seals are required.

Hydraulic and torque characteristics of valves with blades of various shapes are available.90-91

Needle valves. Needle valves have long been used for precise flow regulation in terminal locations. They consist of a needle or tapered plunger which moves axially within an orifice forming part of the valve body (see Figure 22.37b). The plunger is located by guides and operated by screw or hydraulic pressure. In the Larner–Johnson valve, actuation is by the pressure difference across the valve controlled by a pilot valve in the nose of the plunger. The plunger and body are precisely shaped so that the flow accelerates through the valve, to assist actuation, while avoiding cavitation under operating conditions. The discharge coefficient for a fully open valve is from 0.4 to 0.72 depending on the throat area ratio.

Tube valves. Tube valves which were used on some USBR reservoir outlets resemble needle valves but the part of the needle downstream of the sealing ring is omitted. This reduced cavitation problems but vibration was experienced with some valves. This valve has been satisfactorily operated fully submerged. The discharge coefficient, when fully open, based on valve outlet diameter is about 0.6. An in-line regulator of similar general form is shown in Figure 22.37c.90 This has tubular ports at its discharge end which direct jets towards the centre of the downstream pipe where excess head is dissipated, thus avoiding cavitation damage. The port openings are regulated by an axial movement of the plunger.

Hollow jet valve. This was developed by the USBR as successor to needle and tube valves and has been generally satisfactory as a high-pressure terminal regulator. It resembles a needle valve with the downstream half of the needle omitted, while the remaining half advances upstream to seal against a ring on the body (Figure 22.37d). The flow is deflected by the surrounding tubular body and is projected in the form of a jet with nearly parallel sides and hollow centre. Pressure is admitted to the plunger to assist operation, which is mechanical. When fully open the valve normally has a coefficient of discharge of 0.7 based on the outlet diameter.90-91 Hollow jet valves by the USBR are stated to operate satisfactorily when partially submerged up to centre level, but should not be operated fully submerged.90 The trajectory can be calculated approximately by the mechanics of a projectile. Though some aeration and dispersion of the jet occurs the jet fallout is concentrated in a relatively small area and in some cases erosion is a problem. A stilling basin has been developed for this valve.94

Fixed cone-sleeve valves. Also known as Howell–Bunger valves (Figure 22.38), these are widely used in free-discharge terminal applications, including pressure relief for turbines. They have a tubular body on the outside of which is a cylindrical sleeve. This is operated by screws or hydraulic servomotors and retracts to form an opening through which the discharge occurs, deflected outwards by the fixed cone. Discharge coefficient at normal maximum opening is approximately 0.85.94

Asymmetry of approach flow may lead to vibration; the valve should not be located too close to a bend in the conduit. A tapered contraction from conduit to valve assists to stabilize the flow. The sharply increasing diameter as the water leaves the valve forces the jet to break up, which is excellent for energy dissipation but sometimes creates problems due to fallout from drifting spray. The limits of the trajectory can be calculated by assuming projectiles at the upper and lower points and two sides of the jet, leaving the valve at velocity equal to \((2gH)^{1/2}\) where \(H\)
Example of butterfly valve. The valve diameter exceeds the conduit size to allow for the obstruction of waterway by the blade; Larner–Johnson needle valve, with internal pilot valve control; in-line regulating valve (Courtesy: Glenfield and Kennedy Ltd); hollow-jet valve (USBR design).

Figure 22.37

Blow-off valve

Pilot valve

Plunger nose

Regulating valve

Blow-off valve

Figure 22.38

Fixed-clone sleeve valve. (Courtesy: Glenfield and Kennedy Ltd)

is the pressure plus velocity head in the valve body, and initial trajectory according to the angle of the cone, normally 45° to the axis. Fallout distance is, however, appreciably reduced by air resistance and affected by wind. In some installations a cylindrical hood is provided to restrict the dispersion of the jet, which then becomes tubular in form. To avoid vibration and failure by fatigue this should be rigid and is often of steel with concrete surround. Because of the large air demand, free access of air is essential.

The advantages of sleeve valves are simplicity and relatively low cost, low actuating power and the energy-dissipating char-
acteristics of the jet. They have been made in sizes up to 3.850 m diameter and (smaller valves) for heads up to 280 m. Several cases of damage due to vibration have been recorded, particularly fatigue failure of the ribs attaching the fixed cone to the body, but this weakness has been overcome by increasing rigidity and avoiding causes of instability, in particular by fairing the leading edges of the ribs. Valves of this type have been operated fully submerged but trouble has occurred with partial submersion.

Submerged sleeve valves. Located in a stilling well, a valve of this type is an excellent terminal regulator. As developed by the National Engineering Laboratory, the valve has an internal sleeve sliding in a perforated cylinder. The perforations result in numerous small jets, which can be stilled in a small chamber, and the perforations can be graded to obtain any desired discharge/stroke characteristic. In some valves large ports are provided with the latter object but causing less obstruction and the full opening of the sleeve can be utilized, without perforations or ports. In this case the discharge coefficient is greater but a larger stilling well is needed. Dimensions of stilling wells providing adequate energy dissipation may be determined from Figure 22.15, but shallower, wider wells of about equal volume also have been found satisfactory.

Multijet sleeve valves have also been developed as energy dissipators in pipelines.\(^5\) Energy is dissipated by small jets and, over a wide range of pressure differential, the valve is generally free from vibration and cavitation problems.

### 22.5.3 Air demand

Air vents are provided downstream of gates and valves in conduits, to relieve low pressures which develop due to regulating and to avoid cavitation or column separation following valve closure. The rate of air demand of a hydraulic jump is discussed on page 22/20. During closure of the valve, downstream pressure falls and the water standing in the vent pipe is gradually drained. At the same time the water in the conduit, if flowing full, is decelerated, but if the conduit is long and the valve closes before the vent is admitting air, pressure might fall to cavitation level. In such cases it is necessary to limit the speed of valve closure, especially when approaching final closure. Pressure can be estimated by a step calculation.

### 22.6 Cavitation

In high-velocity flow and regions of low pressure, local pressure may approach the level of vapour pressure causing cavitation bubbles to form and become entrained in the flow. When these reach regions of higher pressure they collapse, producing extremely high local transient pressures which can damage solid boundaries. Examples of cavitation damage have occurred in high-velocity flow past rough surfaces and joints in concrete, in stilling basins below high spillways, downstream of submerged orifices and in valves and rotary pumps. Metal becomes damaged by pitting; concrete begins to disintegrate.

Cavitation in valves, pipe fittings and pumps is characterized by a high-pitched tapping sound and the discharge coefficient or pump efficiency may be affected. Cavitation damage usually occurs immediately downstream of the point of lowest pressure but if the cavities are formed away from solid boundaries as, for example, by fluid shear downstream of a submerged orifice, they can be carried some distance in the flow and in some cases collapse harmlessly in the fluid, but if the collapse occurs near to or on a solid boundary this can be damaged. The collapse of a diversion tunnel at Tarbela dam was attributed to this cause.\(^6\)

An index of cavitation potential is the cavitation number:

\[
\sigma = \frac{H_t - h}{H_t + H_i} \quad \text{or} \quad \frac{p_2 - p_1}{\rho V^2/2} \quad (22.17)
\]

where \(H_t\) is the pressure head and \(p_2\) the pressure at the point concerned, \(h\) the vapour pressure head, \(H_i\) the total (static + velocity) head, \(\rho\) the density of water, \(p_1\) the vapour pressure and \(V\) the mean or relevant velocity.

The heads and pressures appear as differences so should be related to the same datum. If the datum is atmospheric pressure, \(h\), will be negative, usually taken as \(-10\) m at sea-level. For the second expression the pressures are usually absolute. The cavitation number represents the ratio of pressure drop required to initiate cavitation to the velocity head available and therefore indicates the potential for cavitation. The number at which cavitation occurs depends on the boundary geometry and flow pattern. If the number for the onset of cavitation in a given situation is known through research or experience it can be used to test whether cavitation will occur over a range of velocities and pressures and (subject to small variations due to scale effect) in situations of geometrical similarity but different absolute dimensions, as in scale models.

The risk of cavitation damage on concrete surfaces in contact with high-velocity flow increases rapidly with increasing velocity, velocities in the range 20 to 40 m/s being of particular concern. It is necessary to provide a very smooth finish; specifications often call for offsets to be ground to a flat slope such as 1:30.\(^7\) In stilling basins below high spillways, concrete baffle blocks may be protected by steel cladding. Use of special additives in the concrete can increase its resistance to damage. Damage can also be alleviated or prevented by aeration as, for example, provided by suction of air at offsets through special ducts.\(^6,8\) In the case of vertical and radial gates and gate valves, mild cavitation may occur with \(\sigma = 2.0\) and more severe cavitation with \(\sigma = 1.0\). In valves of other types the critical value of \(\sigma\) differs in different designs and with the amount of opening. Butterfly valves were found to have incipient cavitation characteristics with \(\sigma = 1.5\) for 30° opening but 3.9 for 80° opening.\(^9\)

### References

11. US Army Corps of Engineers (1952–70) Hydraulic design criteria. US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
22/34 Hydraulic structures


