Example (3): A spillway has been designed for a head of 2.80 m with a length 200 m. The discharge coefficient is C = 0.49 Calculate the discharge for this head. What will the discharge be for heads of 0.20 m and 1.50 m? What is the maximum discharge that can be passed over this spillway without cavitation?

Sol:

At the design head,

$$Q = C\sqrt{2g}LH^{3/2}$$

$$Q = 0.49 * \sqrt{19.62} * 200 * 2.8^{3/2}$$

$$Q = 2034 m^3/s$$

For $H = 0.2m,,,, \frac{H}{H_D} = \frac{0.2}{2.8} = 0.071$

From design chart 1 $\frac{C}{C_D} = 0.82$

$$C = 0.49 * 0.82 = 0.4$$

$$Q = 0.4\sqrt{19.62} * 200 * 0.2^{3/2}$$

$$Q = 32 m^3/s$$

$$H = 1.5m,,,, \frac{H}{H_D} = \frac{1.5}{2.8} = 0.54 \rightarrow \frac{C}{C_D} = 0.92$$

$$C = 0.92 * 0.49 = 0.45$$

$$Q = 0.45 * \sqrt{19.62} * 200 * 1.5^{3/2}$$

$$Q = 732 m^3/s$$

The maximum head that can be without cavitation on the crest,

$$H_{max} = 1.65 H_D$$

$$H_{max} = 1.65 * 2.8 = 4.62 m$$

For $\frac{H_{max}}{H_D} = \frac{4.62}{2.8} = 1.65 \rightarrow \frac{C}{C_D} = 1.08$

$$C = 1.08 * 0.49 = 0.53$$

$$Q_{max} = 0.53 * \sqrt{19.62} * 200 * 4.62^{3/2}$$

$$Q_{max} = 4663 m^3/s$$

Problems

Q1: Assume the dam in example (1), is 8m high. Re-compute the static head on the weir by accounting for approach velocity. What percent error was introduced in the static head by ignoring the approach velocity?

Q2: A spillway needs to be designed to carry a peak flow of $50m^3/s$ with the reservoir elevation 1m above the crest of the spillway. The elevation difference between the reservoir and the tail water is 15m. If an overflow spillway is used, with a discharge coefficient of 2.0, determine the length of the spillway crest required to handle the discharge. Also determine the discharge coefficient in British unit.

Q3: An overflow spillway 70m long was designed in 1960 for a flood capacity 900 m^3/s. It has now been established that according to new more stringent guidelines the design flood capacity of the spillway should be increased to 1200 m^3/s. As a consequence three possibilities are being considered:

1. Increasing the length of the spillway (without any change in the maximum flood reservoir level or in the freeboard); cost 300000$/m length,
2. Increasing the height of the dam (by providing an additional impermeable wave wall along the whole dam crest) to accommodate the increased flood level without a reduction of freeboard; cost 800000$/100mm increase in flood storage level,
3. Adding 2 siphons to the existing spillway; cost 6 million $.

a) On purely economic considerations which of the above three alternatives would you choose?
b) What factors other than spillway construction costs might influence your decision?
c) What are the advantage and disadvantages of siphon spillways?

Sol:

a) 1- $Q = C\sqrt{2g}LH^{3/2} \Rightarrow C_D = 0.75 \Rightarrow Q_{\max} = \frac{2}{3}Q$

$$Q = \frac{2}{3} * 0.75 * \sqrt{19.62} * 70 * H^{3/2} \Rightarrow H = 3.23m$$

$$1200 = \frac{2}{3} * 0.75 * \sqrt{19.62} * L * 3.23^{3/2} \Rightarrow L = 93.33m$$

Increasing of length=93.33-70=23.33m
Cost=23.33*300000=6.99≈7$ million

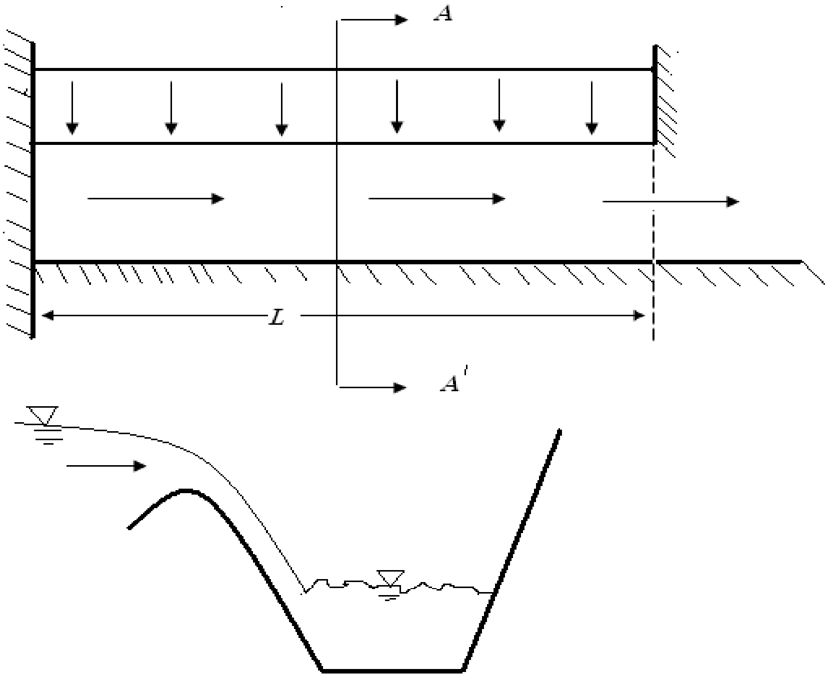
$$2\text{-}1200 = \frac{2}{3} * C_D * \sqrt{19.62} * 70 * H^{3/2} \Rightarrow C_D \rangle 0.75$$

Assume $C_D = 0.775 \Rightarrow H_2^{3/2} = 7.48m \Rightarrow H_2 = 3.82m$
Height increasing H2-H1=3.82-3.23=0.6m (600mm)
Cost=6*800000=4.8$ million
3- cost =6$ million
The second (number 2) is the best

Side-Channel Spillway:

A side-channel spillway carries water away from an overflow spillway in a channel parallel to the spillway crest.

- The side channel overflow was successively used at the Hoover dam (USA) in the late 1930's.
- The arrangement is advantageous at locations where a frontal overflow is not feasible, such as earth dams, or when a different location at the dam site yields a better and simpler connection to the stilling basin.
- Side channels consist of a frontal type of overflow structure and a spillway with axis parallel to the overflow crest.
- The specific discharge of overflow structure is normally limited to 10 m3/s/m, but for lengths of over 100 m.



Section A-A'

Side-channel spillway

The crest is usually a concrete gravity section, but it may consist of pavement laid on an earth embankment or the natural ground surface. **<u>This type of spillway is used in narrow canyons where sufficient crest length is not available for other types of spillway.</u>**

<u>Design Criteria:</u>

- The side-channel spillway must provide a slope steep enough to carry away the accumulating flow in the channel.
- Minimum slope and depth at each point along the channel is desired in order to minimize construction costs.
- For the above reasons; an accurate water surface profile for the maximum design discharge is important in this type of spillway design.

<u>Flow Profile Analysis for Side-Channel Spillway:</u>

1. The discharge through any section of the side-channel spillway at a distance x from the upstream end of the channel is

$$Q_x = xCH_a^{3/2}$$

2. The flow profile is analyzed by momentum principle, cannot be analyzed by the energy principle due to highly turbulent flow conditions that cause excess energy loss in the channel.

Consider the forces and change of momentum between two adjacent sections, Δx distance apart, in the side channel

$$\sum F = \rho(Q+\Delta Q)(V+\Delta V) - \rho QV$$

ρ is the density of water, V=average velocity Q=discharge at upstream section, Δ=the incremental change at the adjacent downstream section.

the forces in left-hand side include, weight component of water body between the tow sections in the direction of the flow, ($\rho gA\Delta x \sin\theta$); the unbalance hydrostatic forces;

$$\rho gA\bar{d}\cos\theta - \rho g(A+\Delta A)(\bar{d}+\Delta\bar{d})\cos\theta$$

and a friction force; F_f ; on the channel bottom. A=water cross sectional area, $\bar{d}$ =distance between centroid of the area and the water surface and θ=angle of the channel slope, momentum equation can be written as

$$\rho gA\bar{d}\Delta x\sin\theta + [\rho gA\bar{d} - \rho g(A+\Delta A)(\bar{d}+\Delta\bar{d})]\cos\theta - F_f$$
$$= \rho(Q+\Delta Q)(V+\Delta V) - \rho QV$$

S_0 =sinθ (for small angle) and $Q = Q_1, V+\Delta V = V_2, A = (Q_1+Q_2)/(V_1+V_2), and,,, F_f = \gamma AS_f\Delta x$; the above equation may be simplified to

$$\Delta d = -\frac{Q_1(V_1+V_2)}{g(Q_1+Q_2)}(\Delta V + V_2\frac{\Delta Q}{Q_1}) + S_0\Delta x - S_f\Delta x$$

where Δd is the change in water surface elevation between the two sections. This equation is used to compute the water surface profile in the side-channel spillway.

First term (right-hand side)=the change in water surface elevation between two section due to the impact loss caused by the water falling into the channel.

The last term=the change due to friction in the channel. Relating the water surface profile to a horizontal datum, we may write

$$\Delta z = \Delta d - S_0\Delta x = -\frac{Q_1(V_1+V_2)}{g(Q_1+Q_2)}(\Delta V + V_2\frac{\Delta Q}{Q_1}) - S_f\Delta x$$

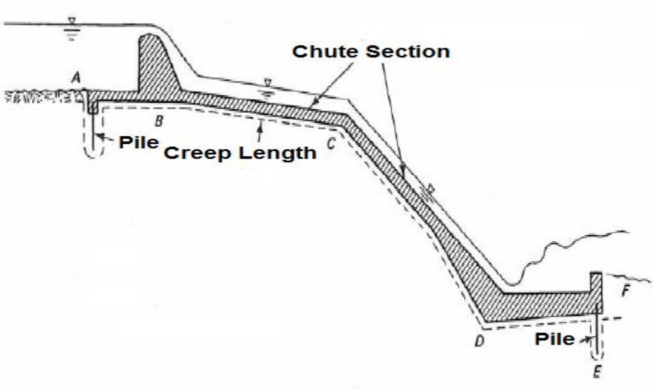
Note that when Q1=Q2 or when ΔQ=0, above equation becomes

$$\Delta z = \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) - S_f\Delta x$$

The last equation represents the energy equation for constant discharge in an open channel.

<u>Chute Spillway:</u>

In this type, water flows over the crest into a steep-sloped open channel that is called a ***chute or trough***, implies that the velocity of flow is greater than critical velocity.

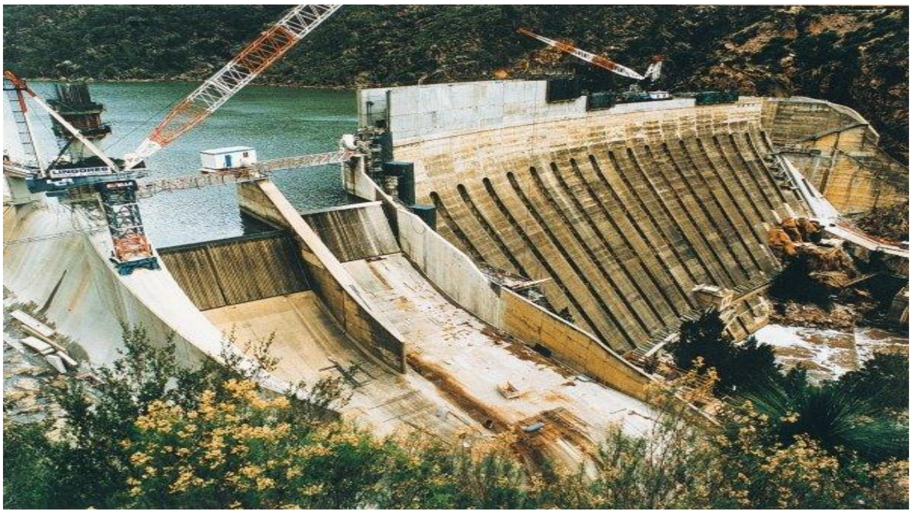



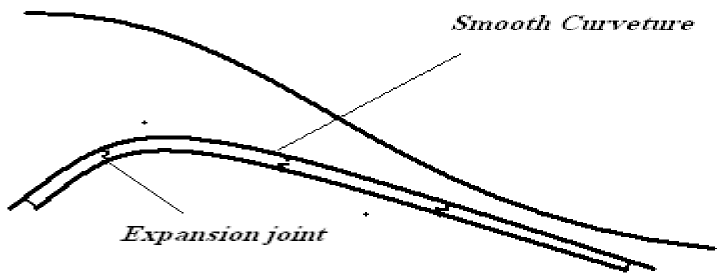
Chute spillway section

This type of structure consists of ***four parts as shown in Fig. above, an entrance channel, a control structure or crest, the sloping chute, and a terminal structure.*** The entrance channel at A is a relatively wide channel of subcritical flow. The control section at B is placed in line with or upstream from the centerline of the dam. The critical velocity occurs when the water passes over the control. Flows in the chute are ordinarily maintained at supercritical stage until the terminal structure DE is reached. Economy of excavation generally makes it desirable that from B to C, where a heavy cut is involved, the chute may be placed on a light slope. From C to D it follows the steep slope on the side of the river valley. An energy dissipating device is placed at the bottom valley D. The axis of the chute is kept straight as far as practicable. The velocity of flow increases rapidly in the chute with drop in elevation. ***It is preferable that the width of the control section, the chute, and the stilling basins are the same.*** Quite often, these widths are not the same, because of the design requirements of the spillway and stilling basin. Extreme care must be taken that the transitions take place gradually, or undesirable waves may develop. ***To prevent hydrostatic uplift under the chute, a cutoff wall (pile) is provided under the control structure and a drainage system of filters and pipes is provided.*** When the stilling basin is operating, there is a substantial uplift under the lower part of the chute and upstream part of the stilling basin floor. ***The floor must be made sufficiently heavy or be anchored to the foundation.***

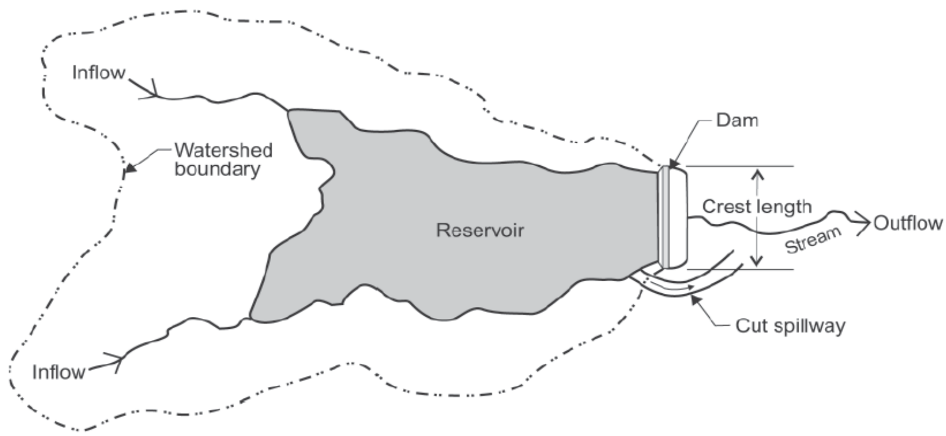
General Specification:

- The channel is usually constructed of reinforced-concrete slabs 0.25-0.5 m thick,
- Is relatively light and **is well adapted(ملائم) to earth or rock-fill dams.**
- Expansion joints are usually required in chute spillway at intervals of about 10m).
- Sometimes has a constant width but is usually narrowed for economy and then widened near the end to reduce discharge velocity.





A chute spillway may be constructed around the end of any type of dam when topographic condition permit and such location are preferred for **earth dams** to prevent possible damage to the embankment.



Slope of Chute Channel:

It is important that the slope of the chute in the upstream section BC should be sufficiently steep to maintain a supercritical flow to avoid formation of a hydraulic jump in the chute. Flow through a channel is given by Manning's formula,

$$Q = \frac{1}{n} AR^{2/3} S^{1/2}$$

Where,
Q = Discharge (m3/sec)
n = Roughness (Manning) coefficient
A = Cross-section area of the channel (m2)
R = A/P = Hydraulic radius (m)
S = Slope of bottom of channel.
For a rectangular channel of depth y and width b,
$A = By \rightarrow R \approx y$ (for a wide cross-section)
Under the condition of critical flow,

$$y_c = \sqrt[3]{\frac{q^2}{g}} \Rightarrow q = \frac{Q}{B}$$

$$Q = \frac{1}{n} Byy^{2/3} S_c^{\ 1/2}$$

$$\frac{Q}{B} = q_c = \frac{1}{n} y_c^{5/3} S_c^{1/2}$$

$$S_c = \frac{q_c^2 n^2}{y_c^{10/3}} = \frac{q_c^2 n^2}{\left(\frac{q_c^{2/3}}{g^{1/3}} \right)}$$

$$S_c = \frac{g^{10/9} n^2}{q_c^{2/9}} = \frac{12.64 n^2}{q_c^{0.222}}$$

The subscript c refers to the critical stage. Since the reliable information on the value of n is difficult, a conservative approach is indicated in the selection of n. The slope of the chute should be more than S_c for a supercritical flow.

A review of existing spillways indicates that the actual slopes of the upstream section of the chute are 1 to 2 % or more. Unstable rapid flows occur when the Froude number exceeds 1.56 to 1.64. It is therefore likely that chutes designed with a conservative slope have a Froude number well over this limit and unstable flow will occur with bumpy surfaces.

Example: Determine the minimum slope in the upper reach of a chute section of 30 m width. The range of discharge is 150 to 2000 m^3/sec. n = 0.015.

Sol.

- Under the minimum flow conditions,

$$q = \frac{150}{30} = 5 m^3/s/m$$

$$S_c = \frac{12.64 n^2}{q_c^{0.222}} = \frac{12.64 * 0.015^2}{5^{0.222}} = 0.002$$

- Under maximum flow conditions, flow depth is by using Equ.

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \Rightarrow 2000 = \frac{1}{0.015} * 30 * y^{5/3} * 0.002^{1/2}$$

$$y^{5/3} = \frac{2000 * 0.015}{30 * 0.002^{1/2}} = 22.36 \Rightarrow y = 6.45 m$$

- Velocity of flow for maximum discharge,

$$V = \frac{Q}{A} = \frac{2000}{30 * 6.45} = 10.34 m/s$$

- Froude number,

$$F_r = \frac{V}{\sqrt{gy}} = \frac{10.34}{\sqrt{9.81 * 6.45}} = 1.3$$

Since 1.30 < 1.56, we have stable rapid flow.

Chute Sidewalls

Except for converging and diverging sections, chute channels are designed with vertical sidewalls, commonly of reinforced concrete 30 to 45 cm thick. The height of the walls is designed to contain the depth of flow for the spillway design flood. The water surface profile from the control section downward is determined for this purpose. An allowance is made for pier and waves roll waves, and air entrainment.

The water surface profile is computed by the methods of gradually varied flow. The initial values of discharge, velocity, and depth at the entering section are known. Since the flow in the chute is supercritical, computation proceeds in a downstream direction. It may be noted that in the steep channel, either S_2 curve (when $y_c > y > y_n$) or the S_3 curve (when $y_c > y_n > y$) is involved. In view of uncertainties involved in the evaluation of surface roughness, pier end waves, roll waves, and air entrainment buckling, a freeboard given by the empirical equation is added to the computed depth of the water surface profile.

$$Freeboard(m) = 0.6 + 0.0004 V' d^{1/3}$$

Where,

V' = Mean velocity in the chute section under consideration (m/sec)

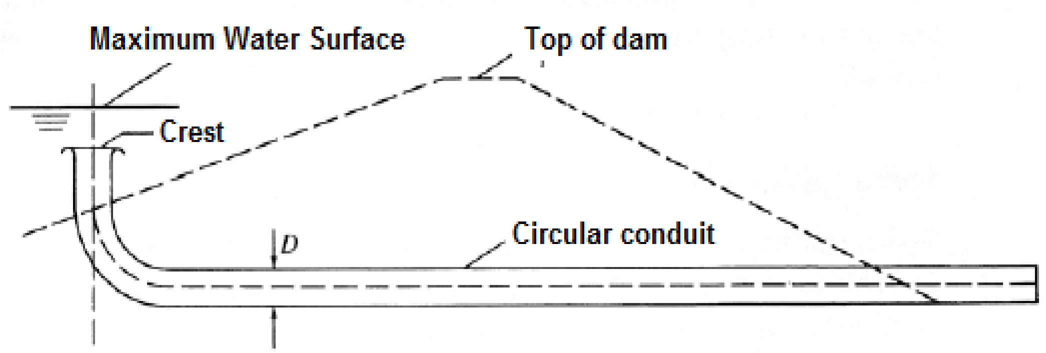
d = Mean water depth (m)

Shaft Spillway

In this type the water drops through a vertical shaft to a horizontal conduit that conveys the water past the dam.

This type is used when:

- There is inadequate space for other types of spillway,
- Undesirable to carry a spillway over or through an earth dam, but they are also often used with embankment dams because it is not safe for standard chutes to be constructed on the embankment.
- If topographic prevents the use of a chute or side-channel spillway around the end of the dam.

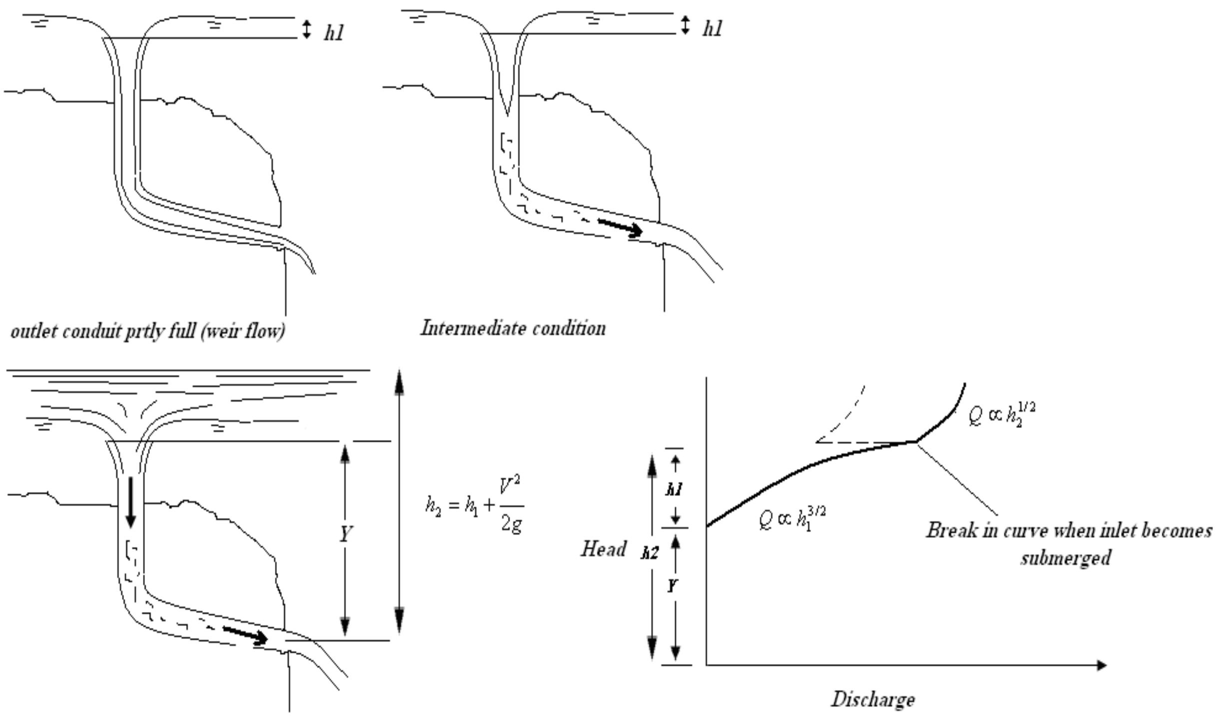


Typical shaft spillway through abutment dam

The major components of a shaft spillway are a ***circular crest section, a vertical shaft, an elbow in a vertical plane, a tunnel section, and a terminal structure.*** For large structures, the various components are usually constructed from concrete, with the tunnel and much of the vertical curve tunneled through rock.

There are three possible conditions of flow in a shaft spillway:

- At low heads the outlet conduit flows partly full, the perimeter of the inlet serves as a weir, and discharge of the spillway varies as ($h_1^{3/2}$).
- As the head is increased, water rises in the shaft, and the outlet may flow partly full (weir flow) or full (orifice flow).
- When the shaft is completely filled with water and the inlet is submerged, the discharge becomes approximately proportional to $h_2^{1/2}$ (pipe flow), where h_2 is the total head on the outlet.



-An abrupt transition between the shaft and outlet conduit may results in cavitations; hence a smooth transition is preferred in large dam. Hydraulic analysis of shaft spillways is difficult, and model tests are often employed.

-A desirable feature of shaft spillways is the hazarded of clogging with debris, trash racks or other types of protection are necessary to prevent debris from interring the inlet.