LECTURE NOTES - IV

« WATER RESOURCES »

Prof. Dr. Atıl BULU

Istanbul Technical University
College of Civil Engineering
Civil Engineering Department
Hydraulics Division

CHAPTER 4

ARCH AND BUTRESS DAMS

4.1. ARCH DAMS

The ideal site for an arch dam is a narrow canyon where the abutments are of massive sound rock so that the horizontal load acting on the dam (water pressure force) may be safely transferred to the canyon walls by arch action. A rule of thumb is that if the ratio of the width at the top of the dam to the height of the dam is less than 5, then an arch dam should be considered for the site.

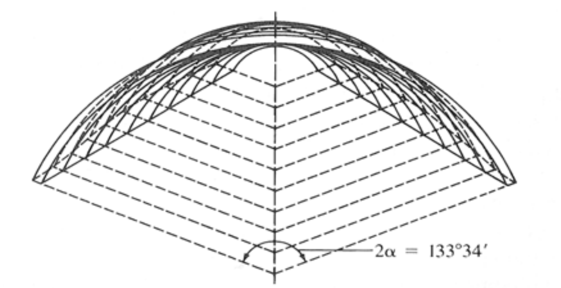

Figure 4.1 Arch dam geometry – plan view

Fig. (4.1) is a plan view showing the basic geometry of an arch dam. A dam like this is called a constant angle arch dam because the angle 2α is constant (approximately 133.6^0). It can be shown that for $2\alpha = 133.6^0$, the volume off concrete needed for the arch is minimum. This assumes all the force is transferred to the canyon walls. Because the width of the canyon increases from the bottom of the dam to the top, the arch radius will also have to increase with height. A simple approach to designing an arch dam is to divide it into several horizontal slices (separate arches) and to analyze each of these from the standpoint of hoop stress produced by water pressure.

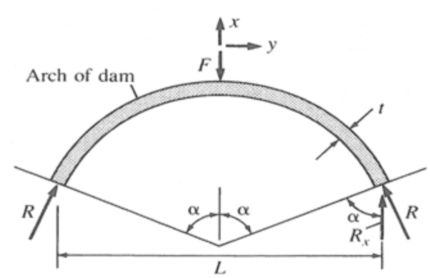

Figure 4.2. Forces on a single arch

Consider the arch shown in the Fig. (4.2), where the width between canyon walls is L. The horizontal component of water force acting downstream on one half of the arch will be resisted by one of the reaction components R_x, or

$$-\gamma_w h \Delta h \frac{1}{2} L + R_x = 0 \tag{4.1}$$

where h = the depth of water above the arch, and Δh = the thickness of the horizontal slice of arch being analyzed.

But,

$$R_x = R\cos(90^0 - \alpha) \tag{4.2}$$

therefore,

$$R\cos\left(90^0 - \alpha\right) = \gamma_w h \Delta h \frac{L}{2}$$

$$R = \frac{\gamma_w h \Delta h \frac{L}{2}}{\cos\left(90^0 - \alpha\right)}$$

Also,

$$R = \sigma \times t \times \Delta h \tag{4.3}$$

where σ = the normal stress in the concrete, and t = the thickness of the arch dam. Then, eliminating r from the above equations,

$$t = \frac{\gamma_w h \Delta h \frac{L}{2}}{\cos\left(90^0 - \alpha\right)\sigma} \tag{4.4}$$

By applying this equation to horizontal slice from the crest of the dam to the bottom, we can determine how the thickness of the dam should vary from top to bottom. Of course, an extremely thin dam at the top is undesirable; therefore, the minimum top thickness is usually taken as about L/60, where L is the length of arc of the crest of the dam.

EXAMPLE 4.1: Using simple arch analysis, design an arch for an arch dam. This particular arch is to be at elevation 300 m, and the width of the canyon wall at the elevation is 600 m. The maximum pool level for the reservoir r is to be 400. Assume the allowable compressive stress in the concrete is 1.4 kN/cm^2.

Solution: For minimum concrete volume 2α = 133.60^0,

$L = 600m$, h = 100 m , γ_w = 10 kN/m^3 , σ_c = 1.4 kN/cm^2

$\sigma_c = 1.4 \times 10^4 \; kN/m^2$

$$t = \frac{10 \times 100 \times \dfrac{600}{2}}{\cos\left(90^0 - \dfrac{133.60^0}{2}\right) \times 1.4 \times 10^4} = 29.59m$$

EXAMPLE 4.2: Prove that 2α = 133.57^0 is the arch angle that minimizes the volume of concrete required in a rib of an arch dam.

Solution: Using Fig. (4.2),

Volume V of the concrete per unit height is,

$V = 2\alpha r A$

A = Cross-sectional area of arch.

A = kr , where k is a constant.

Since,

$$\sin\alpha = \frac{\dfrac{L}{2}}{r} \rightarrow r = \frac{L}{2\sin\alpha}$$

Substituting this to the volume equation,

$$V = 2k\alpha \times \left(\frac{L}{2\sin\alpha}\right)^2$$

$$V = \frac{kL^2}{2} \times \frac{\alpha}{\sin\alpha}$$

For minimum volume, $\dfrac{dV}{d\alpha} = 0$

$$\frac{dV}{d\alpha} = \frac{kL^2}{2} \times \frac{\left(\sin^2\alpha - 2\alpha\sin\alpha\cos\alpha\right)}{\sin^4\alpha} = 0$$

$$\sin^2\alpha = 2\alpha\sin\alpha\cos\alpha$$
$$\sin\alpha = 2\alpha\cos\alpha$$
$$2\alpha = \tan\alpha$$

Solving for α yields,

$$\alpha = 66.78^0 = 0.371\pi = 1.166 radian$$

This is the most economical angle. In actual practice, 2α varies between 100^0 to 140^0.

4.2. BUTTRESS DAMS

A buttress dam is essentially a hollow gravity dam. Buttresses of reinforced concrete rest on the rock foundation and support a watertight sloping face of the dam. Fig. (4.3) shows the general configuration of the nonoverflow section of buttress dam. The facing of this dam is a flat slab.

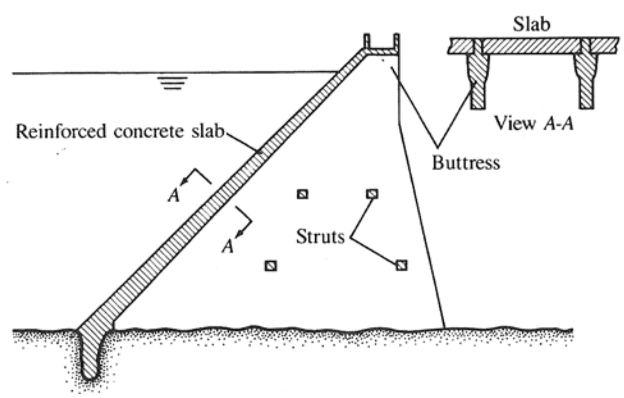

Figure 4.3. Buttress Dam

The gravity part of the stability of this dam comes from the dead load of the concrete in the buttresses and face and, more important, from the weight of the water above the sloping face.
The main advantage of the buttress dam is that it needs as little as 30 or 40 % of the concrete needed for a solid gravity dam. However, this advantage is usually offset by the added labor costs of building forms and placing reinforcing steel. Moreover, in areas subject to freezing temperatures, the face slabs of buttress dams have been known to deteriorate from this type of weathering. The flat-slab is well suited for low buttress dams.

4.3. RESERVOIRS

A reservoir is a manmade lake or structure used to store water. A dam reservoir has an uncontrolled inflow but a largely controlled outflow. The water available for storage is totally a function of the natural stream streamflow that empties into it.

4.3.1. Reservoir Capacity

Reservoir capacity is the volume of water that can be stored in the particular reservoir. It is the *normal maximum pool* level behind a dam. This can be calculated by using a topographic map of the region. First, the area inside different elevation contours is measured, and then a curve of area versus elevation can be constructed.

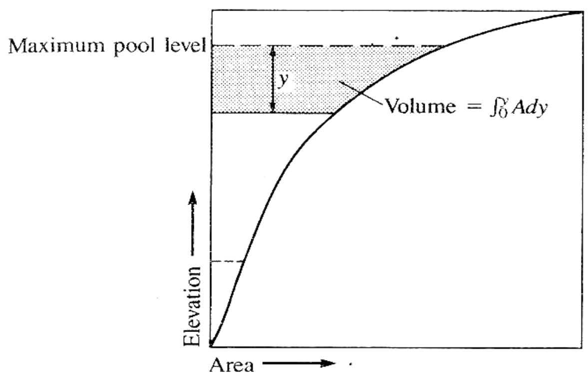

Figure 4.4. Area versus elevation for a reservoir

At any given elevation, the increment of storage in the reservoir at that elevation will be Ady, where dy is a differential depth. Then the total storage below the maximum level to any will be given by,

$$V = \int_0^y Ady \tag{4.5}$$

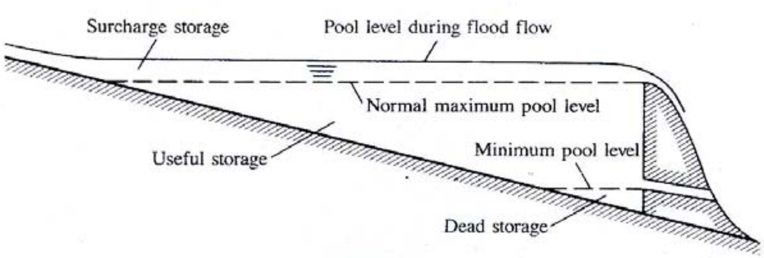

Figure 4.5. Storage relations for a reservoir with an uncontrolled spillway

4.3.2. Sedimentation in Reservoirs

All streams carry sediments that originate from erosion processes in the basins that feed the streams. After a dam is constructed across the steam and a reservoir is produced, the velocity in the reservoir will be negligible so that virtually all the sediment coming into the reservoir will settle down and be trapped. Therefore, the reservoir should be designed with enough volume to hold the sediment and still operate as a water storage reservoir over the project's design life. For large projects, the design life is often considered 100 years.

Sediment carried in a stream is classified as either *bed load* or *suspended load*. The bed load consists of the coarsest fractions of the sediment (sands and gravels), and it rolls, slides, and bounces along the bottom of the stream. The finer sediments are suspended by the turbulence of the stream. When the sediment enters the lower velocity zone of the reservoir, the coarser sediments will be deposited first, and it is in this region that a delta will be formed. The finer sediments will be deposited beyond the delta at the bottom of the reservoir.

The total sediment outflow from a watershed or drainage basin measured in a specified period is the *sediment yield*. The yield is expressed in terms of tons per square kilometer per year. The engineer designing a reservoir must estimate the average sediment yield for the basin supplying the reservoir to determine at what rate the reservoir will fill with sediment.

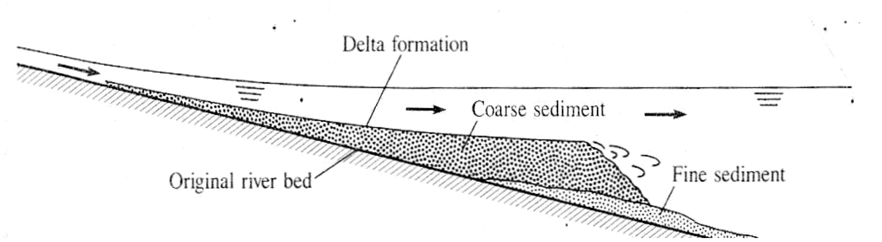

Figure 4.6. Deposition of sediment in a reservoir

For a given reservoir volume, V, the ratio of the reservoir volume decrease due to the deposited sediment can be estimated by this empirical equation,

$$R = 23\times10^{-6}G^{0.95}\left(\frac{A}{V}\right)^{0.8} \tag{4.6}$$

Where,

G = Sediment yield of the basin (kN/km^2/year)
A = Drainage basin area (m^2)
V = Reservoir volume (m^3)

Multiplying R ratio with the design life of the reservoir, T, will yield the percentage of the dead volume in the reservoir. The dead volume can be estimated over the period of design life by,

$$V_{dead} = R \times T \times V_{reservoir} \tag{4.7}$$

EXAMPLE 4.3: The total volume of a reservoir is V = 230×10^6 m^3 with a drainage basin of A = 1200 km^2. The design life of the project is T = 100 year and the density (specific mass) of the deposit is ρ = 2.65 ton/m^3. Calculate the dead volume of the reservoir.

Solution:

The sediment of the river for a river,

$$G = 1421A^{-0.229}$$
$$G = 1421 \times 1200^{-0.229}$$
$$G = 280\left(m^3 / km^2 / year\right)\left(volume\right)$$

The ratio of the reservoir volume decrease every year,

$$R = 0.000023 \times G^{0.95}\left(\frac{A}{V}\right)^{0.8}$$
$$R = 0.000023 \times 7279^{0.95} \times \left(\frac{1200 \times 10^6}{230 \times 10^6}\right)^{0.8}$$
$$R = 0.4(\%)$$

Reservoir volume decrease due to the sediment deposit every year,

$$V_{dead} = 0.004 \times 230 \times 10^6 = 0.92 \times 10^6 m^3$$

For the 100 year of design period,

$$V_{dead_{100}} = 100 \times 0.92 \times 10^6 = 92 \times 10^6 m^3$$

Useful storage is,

$$V_{useful} = \left(230 - 92\right) \times 10^6 = 138 \times 10^6 m^3$$

4.3.3. Wind-Generated Waves, Setup, and Freeboard

Whenever wind blow over an open stretch of water, waves develop, and the mean level of the water surface may change. The latter phenomenon, called *setup* or *wind tide*, is significant only in relatively shallow reservoirs. When a dam is designed, the crest of the dam must be made higher than the maximum pool level in the reservoir to prevent overtopping of the dam as the wind-generated waves strike the face of it. The additional height given to the crest of the dam to take care of wave action, setup, and possibly settlement of the dam (if it is earthfill) is called *freeboard*.

4.3.3.1. Setup

Consider the basin of water shown in the figure. The solid line depicting the water surface is the case when no wind is blowing; the water surface is horizontal. When the wind is blowing, a shear stress acts on the water surface, and because of this, the surface will tilt, as shown by the broken line in the Fig. (4.7).

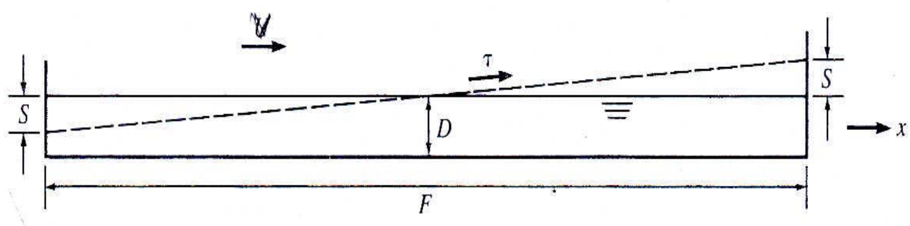

Figure 4.7. Definition sketch for setup

The amount of setup S is,

$$S = \frac{V^2 F}{KD} \tag{4.8}$$

Where,

D = Average reservoir depth (m),
V = the wind speed measured at a height of 10 m from the surface (km/h)
F = the wind fetch (km)
K = A constant ≈ 62000
S = Setup (m)

EXAMPLE 4.4: A reservoir is oval shaped with a length of 20 km and a width of 10 km. If the wind blows in a direction lengthwise to the reservoir with a velocity of 130 km/h, what will be the setup of the average water depth of the reservoir is 10 m?

Solution: The setup will be,

$$S = \frac{V^2 F}{KD}$$
$$= \frac{130^2 \times 20}{62000 \times 10} = 0.55m$$

4.3.3.2. Height of Wind Waves and the Run-Up

Allowances for wave height and the run-up of wind-generated waves are the most significant components of freeboard. The *run-up* of the waves on the upstream dam face, i.e. the maximum vertical height attained by a wave running up a dam face, is equal to H (wave height) for a typical vertical face in deep water, but can attain values over 2H for a smooth slope 1 in 2.

A *wave height*, H (m), (crest to trough) can be estimated by,

$$H = 0.34\sqrt{F} + 0.76 - 0.26\sqrt[4]{F} \qquad (4.9)$$

Where,

F = the fetch length (km),
H = Wave height (m)

For large values of fetch (F>20 km), the last two terms may be neglected. With the provision for the wind speed, the equation takes the form of,

$$H = 0.032\sqrt{VF} + 0.76 - 0.24\sqrt[4]{F} \qquad (4.9)$$

Where,

V = Wind velocity (km/hour)

The freeboard will be equal to set-up plus run up allowance for settlement of the embankment plus and amount of safety (usually 0.50m).

EXAMPLE 4.5: Calculate the wind set-up and wave height for a reservoir with 8 km fetch length. The average reservoir depth is 15 m. The wind velocity is V = 100 km/h. If the upstream of the dam is vertical, what will be the minimum freeboard to be given?

Solution:

The wind set-up is,

$$S = \frac{V^2 F}{KD}$$
$$= \frac{100^2 \times 8}{62000 \times 15} = 0.09m$$

The wave height,

$$H = 0.032\sqrt{VF} + 0.76 - 0.24\sqrt[4]{F}$$
$$H = 0.032\sqrt{100 \times 8} + 0.76 - 0.24\sqrt[4]{8}$$
$$H = 1.19m$$

Since the upstream side of the dam is vertical, the run-up height will be taken as the height of the wave. The freeboard,

$$H_{freeboard} = 0.09 + 1.19 + 1.19 + 0.5$$
$$H = 2.97m \cong 3.00m$$

0.5 m is the safety height.