1. Design an ogee spillway with the following data.

i) Height of spillway crest above river bed =100m
ii) Design discharge = $12000m^3$
iii) Number of spans = 6
iv) Clear distance between piers =15m
v) Thickness of pier =3m
vi) Slope of downstream face of the overflow section = 0.8:1
vii) Assume any data if required.

Solution:

Clear water way (L') =clear d/ce b/n piers * number of span = 6x15=90m

Assuming $C_d = 2.30$, (value ranges b/n 2.1- 2.5)

From $Q= C_d L_e He^{3/2}$

$Q=2.30x90xHe^{3/2}$

$12000= 2.30x90x He^{3/2}$

He= 15.43m.

Check effect of height of spillway/approach velocity: Model tests have shown that the effect of approach velocity is negligible when the height of the spillway above the streambed is equal to or greater than 1.33 H_d ($P \geq 1.33 H_d$)

P/H_d= 100/15.43=6.48,

P/H_e = 6.48>1.33, effect on C_d is negligible.

Thus, the velocity of approach is small and $H_e \approx H_d$=15.43m

Effect of actual head: if the design head is equal to the actual head, (H_e/H_d) ratio is 1.0 and therefore, there is no effect on C_d.

Effect of upstream slope: Assumed to be vertical, hence no effect on C_d.

Effect of downstream apron: In this case, $d+h_d$=100+15.43=115.43m.

$\therefore (h_d + d)/H_e = 115.43/15.43 = 7.48 > 1.70$,hence no effect on C_d.

Effect on end construction: Let us assume K_p=0.02 and K_a=0.20.

Therefore, effective length

$$L_e = L' - 2(NK_p + K_a)(H_e)$$

$Le = 90.0 - 2(5x0.02 + 0.2)x15.43 = 80.74m$

$Q=C_dL_eH_e^{3/2}$

$12000 = 2.30x80.74 * H_e^{3/2}$

H_e= 16.59m

Substituting this value of H_e is equation of effective length,

$Le = 90 - 2(5x0.02 + 0.2)x16.59 = 80.05m$

Substitute on Discharge equation,

$12000=2.30x80.05xH_e^{3/2}$

$\Rightarrow H_e=16.7m$

Let's take the design head (H_e) of 16.70m.

Downstream profile: $X^n = KH_d^{n-1}y$

Velocity of approach $= \frac{Q}{A} = \frac{12000}{(90+5\times 3)\times(100+16.70)} = 0.98m/s.$

Head due to approach velocity is given by:

$$H_a = \frac{v^2}{2g}$$

$$H_a = \frac{(0.98)^2}{2x9.81} = \frac{(0.98)^2}{19.62} = 0.05m \approx \textit{small and negligible}$$
$$\therefore H_d \approx H_e$$

For a given upstream vertical condition i.e. vertical face, K=2.0, and n=1.850.

$$X^n = KH_d^{n-1}y$$

$x^{1.85}=2.00x\ (16.70)^{0.85}y=21.895y$

$Y= 0.0457(x)^{1.85}$..…………….d/s profile

Before determining the various co-ordinate of d/s we shall determine the tangent point.

The d/s slope is given 0.8H:1V

For d/s face of the overflow section, $\frac{dy}{dx} = \frac{1.0}{0.80} = 1.25$

From equation, Y= 0.0457(x) $^{1.85}$, differentiating WRT x

$$\frac{dy}{dx} = 1.85x0.0457(x)^{0.85} = 0.0845(x)^{0.85}$$

Combining the two equations,

$$0.0845x^{0.85} = 1.25$$
$$x = 23.80m.$$

The tangent point of the profile is at a distance of 23.8m from the origin.

X	0	1	2	3	4	5	6	7	8	9	10	11	12
Y= 0.0457(x) $^{1.85}$	0.000	0.046	0.165	0.349	0.594	0.897	1.257	1.672	2.141	2.662	3.235	3.859	4.533

13	14	15	16	17	18	19	20	21	22	23	23.8
5.257	6.029	6.850	7.719	8.635	9.598	10.607	11.663	12.765	13.912	15.105	16.091

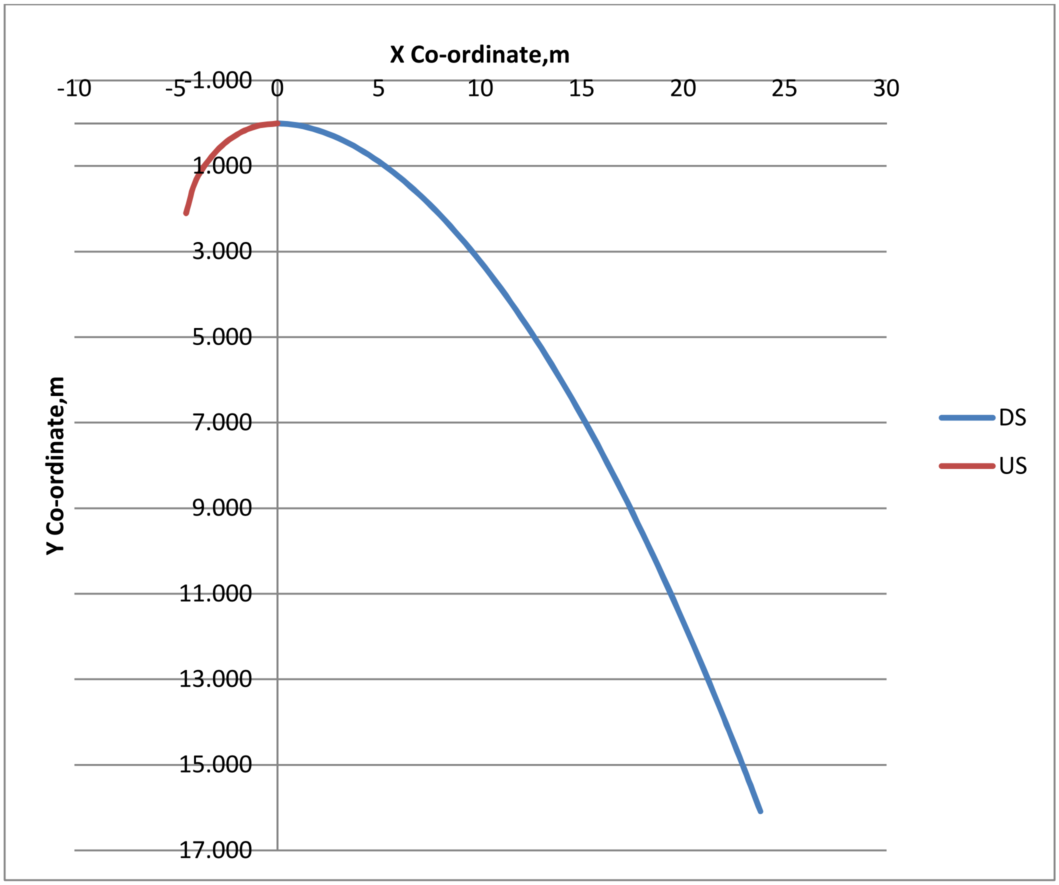
For upstream profile:

- the curve should go up to the point given by: 0.27H_d= 0.27*16.7 = 4.509 , substitute the value of H_d resulting the equation below,

$$Y = \frac{0.724(x + 0.270H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375}(x + 0.27H_d)^{0.625}$$

y= 0.06614 (x+4.509)$^{1.85}$+2.1042-1.2402(x+4.509)$^{0.62}$ upstream profile

x(m)	0	-1	-2	-3	-4	-4.509
y(m)	0	0.060924	0.263056	0.64191	1.309976	2.1042



2. The crest level of dam spillway has been kept at 723.70m while the maximum level in the reservoir is to be 734.50m calculate the maximum discharge through the overflow spillway, when the flow takes place through 5 units of 12.2m width each at the crest of the spillway.

Solution:

Assume pointed nose piers and rounded abutments (Kp = Ka= 0)

Effective length of spillway crest

Le= L\`-2 (N*kp+Ka) He

L\`=5*12.2= 61m

Maximum flood rise over the crest, H = 734.5-723.70

= 10.8m

Neglecting velocity of approach and taking C=2.2

Le = 61-2(4*0+0)10.8=61m

$Q = CLe\ H^{3/2}$

$Q = 2.2*61*10.8^{3/2}=4763\ m^3/s$

If square nosed piers and square abutments are assumed: (Kp = 0.02, Ka= 0.2)

Le= 61-2(4*0.02+0.2)10.8=54.952m

$Q= 2.2*54.952*10.8^{3/2}= 4291 m^3/s$

3. Compute the discharge over an ogee shaped spillway whose coefficient of discharge is 2.2 and head of 6m on the crest. The flow takes place through 4 units of 12.5m width each at the crest of the spillway. The spillway crest is 10m above the bottom of the approach channel which has the same width to that of the spillway. Take K_a =0.10 and K_p=0.015 and pier thickness t=1.0m.

Solution: $He= H_d+Ha$

$He \approx H_d$, neglecting velocity head

First neglect velocity of approach Va= 0 which implies $H_e=H_d$

$$L_e = L - 2*\lfloor NK_p + K_a \rfloor H_e = 50 - 2*[3*0.015 + 0.10]*6$$

$$L_e = 48.26m$$

The discharge Q over the spillway is estimated to be

$$Q = CLH_e^{\frac{3}{2}} = 2.2*48.26*[6]^{\frac{3}{2}} = 1560\ m^3/\sec$$

Then the velocity of approach is computed as

$$V_a = \frac{Q}{A} = \frac{156}{(50+3)*16} = 1.84m/\sec$$

The total head over the weir is:

$$H_e = H_d + \frac{V^2}{2*g} = 6 + \frac{1.84^2}{2*9.81} = 6.173m, take\ 6.20m$$

Finally, the discharge over the weir is calculated to be:

$$Q = CLH_e^{\frac{3}{2}} = 2.2*48.26*[6.2]^{\frac{3}{2}} = 1639\ m^3/\sec$$

Note: This question could be also iterated or recalculated for more accurate results. How?

4. An overflow spillway is designed for a head of 2.8m having a waterway length of 200m and the discharge coefficient for this given head is assumed as 0.75. What will the discharge be for heads of 0.2m and 1.5m, and what is the maximum discharge that can be passed over this spillway (assuming the dam freeboard to be high enough and the spillway to be well constructed) without cavitation?

$$Q = \frac{2}{3}\sqrt{2}g^{1/2}bC_dH^{3/2}$$

- For H_d=H_{max} the pressure is atmospheric and C_d = 0.745.
- For H_d>H_{max} the pressure is greater than atmospheric, coefficient of discharge will be b/n 0.578 <C_d <0.745.
- For H_d <H_{max} negative pressures result, reaching cavitation level for H=2H_d with C_d=0.825.

For safety it is recommended not to exceed the value H_{max}=1.65H_d with C_d=0.81.

Solution

At the design head

$$Q = 2/3 \times 0.75\sqrt{2}g^{1/2} \times 200 \times 2.8^{3/2} = 590.66 \times 0.75 \times 2.8^{3/2} = 2075\,\text{m}^3\text{s}^{-1}.$$

For $H = 0.20\,\text{m}$, $H/H_d = 0.20/2.80 = 0.071$; thus $C_d \simeq 0.58$ and

$$Q = 590.66 \times 0.58 \times 0.2^{3/2} = 31\,\text{m}^3\text{s}^{-1}.$$

For $H = 1.50\,\text{m}$, $H/H_d = 1.50/2.80 = 0.536$; by interpolation between 0.578 ($H/H_d = 0.05$) and 0.75 ($H/H_d = 1$), $C_d = 0.666$ and

$$Q = 590.66 \times 0.666 \times 1.5^{3/2} = 723\,\text{m}^3\text{s}^{-1}.$$

The maximum head for no cavitation. $H_{max} = 1.65H_d = 1.65 \times 2.8 = 4.62\,\text{m}$. For this condition, $C_d = 0.81$, and therefore

$$Q_{max} = 590.66 \times 0.81 \times 4.62^{3/2} = 4757\,\text{m}^3\text{s}^{-1}.$$