Study of the Design of New Okhla Barrage, Kalindi Kunj, New Delhi

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Abstract: ***South Delhi is mainly divided in three phases. Okhla in the past has also lend its name to the New. The Okhla barrage, which was developed by Nitin Saxena, is also the starting point of the Agra Canal built in 1874; today it is also the location of the Okhla Sanctuary. The Agra Canal is an important Indian irrigation work which starts from Okhla in Delhi. The Agra canal originates from Okhla barrage, downstream of Nizamuddin Bridge, it opened in 1874. In the beginning, it was available for navigation, in Delhi, erstwhile Gurgaon, Mathura and Agra Districts, and Bharatpur State. Later, navigation was stopped in 1904 and the canal has since then, been exclusively used for irrigation purposes only. At present the canal does not flow in district Gurgaon, but only in Faridabad, which was earlier a part of Gurgaon. The Canal receives its water from the Yamuna River at Okhla, about 10 KM to the south of New Delhi. The weir across the Yamuna was the first attempted in Upper India upon a foundation of fine sand; it is about 800-yard long, and rises seven-feet above the summer level of the river. From Okhla the canal follows the high land between the Khari-Nadi and the Yamuna and finally joins the Bangangariver about 20 miles below Agra. Navigable branches connect the canal with Mathura and Agra. The canal irrigates about 1.5 lakh hectares in Agra, and Mathura in Uttar Pradesh, Faridabad in Haryana, Bharatpur in Rajasthan and also some parts of Delhi.***

Keywords: Okhla Barrage, Silt Excluder, Waterway, Discharge, Friction Loss and Crest Level

1. Introduction

Okhla Barrage

Okhla Barrage which is located in Delhi on river Yamuna diverts a large amount of water from the river into the Agra Canal from where the water is used

- NTPC for power generation
- Irrigation purposes.

Okhla, is a neighborhood around the old village in South Delhi district, though it is most known as **Okhla Industrial Area** (OIA).

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Original Design data of Okhla Barrage

Barrage

On river Yamuna 2.56km d/s of existing Okhla Weir in Delhi.

River Yamuna

Catchment Area 17950sq.km / 6930 sq. miles
Design Flood 9911.4 cumecs / 3.0 lac cusecs
Design H.F.L 202.17 m
Lacey's Waterway 444.60 m
Pond Level 201.35 m

The Barrage

Spillway Bays 22
Under sluice Bays 5
Length of each bay 18.30 m
Spillway crest 196.75 m
Under sluice Bays crest 195.85 m

Waterway

Total 552.09 m
Clear 494.10 m
U/S bed level 195.85 m
D/S bed level 191.45 m
No. and size of gates
22no.s-3 no's two tier gates
(18.3*1.5m, 18.3*3.6m)
19 no's (18.3*5.1m)
5 no.s-1 no. two tier gate
(18.3*1.5m;18.3*4.5m)

4 no's gates (18.3*6.0m)

Silt Excluder

No. of tunnels 14
Size of tunnels 2.3m*2.2m

Head Regulator

No. of Bays 8
Length of each bay 7.65m
U/S Floor level 198.35m
D/S floor level 197.90m
Free board above F.S.L 0.50m

Link Channels

Silt Ejector 1260 cusecs
Agra Canal 4000 cusecs
Gurgaon Canal 2240 cusecs

2. Design

Design of Barrage Components

1. Data used for the design

For the purpose of design of the barrage, following data has been used:

- Design flood discharge = 9911.4 cumecs
- Bed level of river = 195.85 m
- High Flood level = 202.17 m
- Pond level = 201.35 m

Assumed Data
Safe Exit Gradient = 1/5

- Retrogression = 0.5 m
- Discharge concentration = 20%
- Permissible Afflux = 1.0 m
- Lacey's silt factor, f = 1.0

3. Fixation of Crest Level and Waterway

Crest level: Average bed level of river = 195.85 which may be taken as crest level of under sluices, u/s floor of under sluices maybe kept same therefore no raised crest in under sluices but crest level of other barrage bays maybe 1-1.5 m higher than crest level of under sluices. We keep them 1.35 m higher-

i.e. 195.85 + 1.35 = 197.2 m

Waterway: The clear waterway as per the Lacey's wetted perimeter equation is given by,

$P = 4.75\sqrt{Q}$, Where Discharge (9911.4 cumecs)**,** P = 473 m

Provide a waterway = 1.2P such that 15-20% of Q is passed through under sluices provided waterway should be able to pass the entire Q.

4. Under Sluice Portion

Assume the waterway as below

(a) Under sluice portion:

6 bays of 15 m each = 90 m. 5 piers of 3 m each = 15 m. Overall waterway = 105 m

(b) Other barrage bays portion:

25 bays of 15 m each = 375 m.24 piers of 3 m each = 72 m
Overall waterway = 447 m
Assume a divide wall of 4.0 m thickness
Hence, total waterway provided between abutments = 105 + 447 + 4 = 556 m

To check whether maximum flood can pass through assumed waterway

Design H.F.L = 202.17 m
Permissible afflux = 1.0 m
Average discharge intensity, q = 9911.4/556 = 17.83 m3/s
u/s H.F.L = d/s H.F.L + Afflux = 202.17 + 1 = 203.17 m
Scour depth, $R = 1.35(q^2/f)^{1/3} = 1.35(17.83^2/1)^{1/3} = 9.21$ m
Velocity of approach, V = q/R= 17.83/9.21 = 1.93 m/s
Velocity head = $V^2/2g = (1.93)^2/(2 * 9.8) = 0.19$ m
u/s T.E.L = u/s H.F.L + velocity head= 203.17 + 0.19 = 203.36 m

Discharge formula for broad crested weir is given by,

$Q = 1.705 (L - K * n *H) * H^{3/2}$
Where, L = 90 m, n = no. of end contractions = 12, H = 6.51 m, K = 0.1
$Q = 1.705 (90 – 0.1 * 12 * 7.51) * (7.51)^{3/2} =$ 2833.541cumec

Design formula for sharp crested weir,

$Q = 1.84 (L - 0.1 * n * H) * H^{3/2}$
Where, L = 375 m, n = no. of end contractions = 50, H = 6.16 m
$Q = 1.84 (375 – 0.1 * 50 * 6.16) *(6.16)^{3/2} =$ 9682.77cumecs
Total discharge that can pass down the barrage = 2833.541 + 9682.77= 12516.31cumecs>> 9911.4 cumecs

Here it is found that the discharge passing down the barrage is very large as compared to the given discharge of 9911.4 cumecs, which is not suitable as per economic considerations.

Thus, the calculations have to be revised.

5. Design of under sluice portion

Discharge intensity and head loss under different flow conditions

i. For maximum flood

a) Without concentration and retrogression

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$q = CH^{3/2} = 1.70 * (7.52)^{3/2} = 35.06$cumec/m
d/s H.F.L = 202.17 m
u/s T.E.L = d/s H.F.L + afflux + velocity head= 202.17 + 1 + 0.210= 203.38 m
d/s T.E.L = d/s H.F.L + velocity head = 202.17 + 0.210= 202.38 m

Head Loss (HL) = u/s T.E.L – d/s T.E.L= 203.38 – 202.38 = 1.0 m

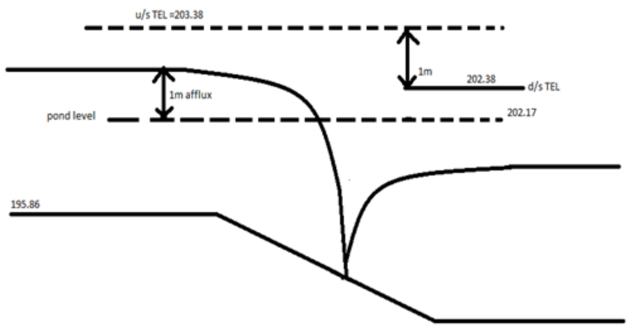

Figure 1: High Flood Condition with no retrogression

b) With 20% concentration and bed retrogression by 0.5 m

Discharge intensity is increased by 20%, therefore new discharge intensity is given as, q = 1.20 * 35.06 = 42.07 cumecs/m

New head required for this discharge intensity to pass, = $(42.07/1.7)^{2/3} = 8.5$ m
u/s T.E.L = 204.35 m
d/s H.F.L with 0.5 m retrogression=202.17-0.5 = 201.67 m
d/s T.E.L with 0.5 m retrogression = 201.67 + 0.210 = 201.88 m
Head Loss, HL = u/s T.E.L – d/s T.E.L= 204.35 – 201.88 = 2.47 m

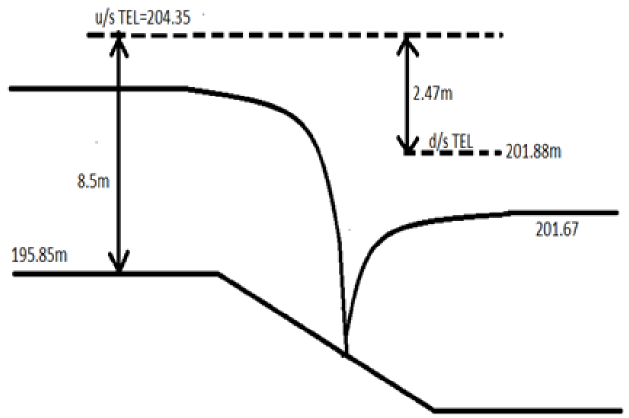

Figure 2: High Flood Flow with 20% concentration and 0.5m retrogression

ii. Flow at pond level (With all gates opened)

a) Without concentration and retrogression

Pond level (given) = 201.35 m
Head over crest of under sluices under this condition= 201.35 – 195.85 = 5.5 m

Head over the crest of other barrage bays= 201.35 – 197.20 = 4.15 m
Neglecting velocity of approach for this flow condition, the total discharge passing down the barrage is,

Q = Q1 + Q2
$Q = 1.705 (90 – 0.1 * 10* 5.5) * (5.5)^{3/2} + 1.84 (315 – 0.1 *42* 4.15) * (4.15)^{3/2}$
Q = 6481.8cumecs

Average discharge intensity, q = (6481.8/484) = 13.40cumecs/m

Normal scour depth, $R = 1.35 * (q^2/f)^{1/3} = 1.35 * (13.40^2/1)^{1/3} = 7.62$ m
Velocity of approach, V = q/R= (13.40/7.62) = 1.76 m/s
Velocity head = $V^2/2g = 1.76^2/(2 * 9.8) = 0.158$ m

u/s T.E.L = P.L + velocity head= 201.35 + 0.158 = 201.51 m

The downstream water level when a discharge of 6481.8 cumecs is passing can be found from stage discharge curve and is found to be 200.80 m.

d/s T.E.L = 200.80 + 0.158 = 200.96 m

Head Loss, HL = 201.51 – 200.96 = 0.55 m

Discharge intensity between piers = $1.70 * (5.5)^{3/2} = 21.93$cumecs/m

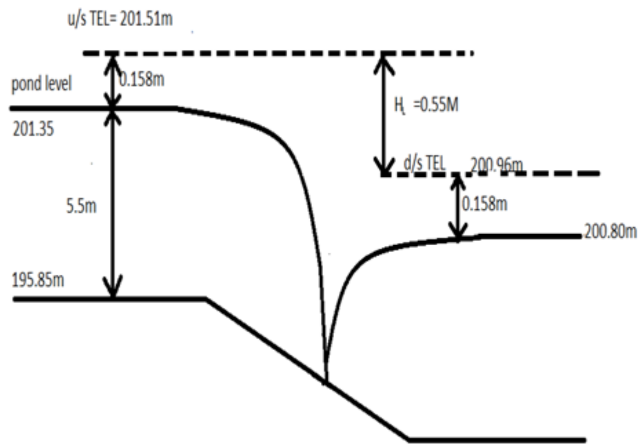

Figure 3: Pond Level Condition with no concentration and retrogression

b) With 20% concentration and 0.5 m retrogression

New discharge intensity = 1.2 * 21.93 = 26.316cumecs/m
New head required = $(26.316/1.70)^{2/3} = 6.21$ m
u/s T.E.L = 195.85 + 6.21 = 202.06 m
d/s H.F.L which was 200.80 m, is depressed by 0.5 m

new d/s H.F.L = 200.80 – 0.5 = 200.3 m
d/s T.E.L = 200.3 + 0.158 = 200.458 m
Head Loss, HL = 202.06 – 200.458 = 1.60 m

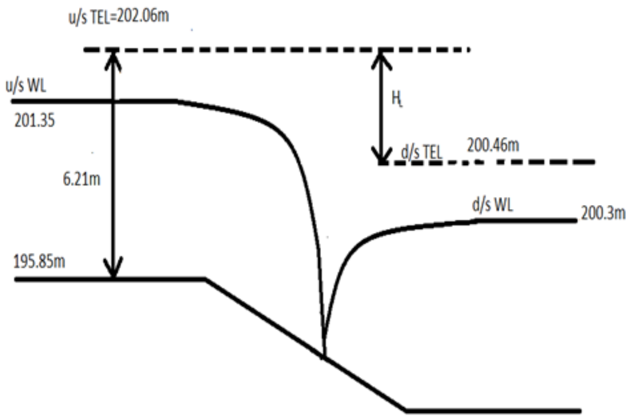

Figure 4: Pond Level Condition with 20% concentration and 0.5 m retrogression

The values of q, HL, the water levels and energy levels for all the four cases are tabulated in following table:

S.NO.	Item	High Flood Flow		Pond Level Flow	
		Without Conc. & Retrogression	With Conc. & Retrogression	Without Conc. & Retrogression	With Conc. & Retrogression
1.	**Discharge intensity**	35.06 cumec/m	42.04 cumec/m	21.93 cumec/m	26.32 cumec/m
2.	**u/s water level**	203.17 m	203.17 m	201.35 m	201.35 m
3.	**d/s water level**	202.17 m	201.67 m	200.80 m	200.30 m
4.	**u/s T.E.L**	203.38 m	204.35 m	201.51 m	202.06 m
5.	**d/s T.E.L**	202.38 m	201.88 m	200.96 m	200.46 m
6.	**Head loss**	1.0 m	2.47 m	0.55 m	1.6 m
7.	**Pre jump Depth (y_1) corresponding to Ef_1**	3.1 m	3.0 m	2.37 m	2.26 m
8.	**Post jump Depth (y_2) corresponding to Ef_2**	7.6 m	9.55 m	5.36 m	6.86 m
9.	**Length of concrete floor required = $5(y_2-y_1)$**	22.5 m	32.75 m	14.95 m	23.0 m
10.	**u/s Specific Energy ($Ef_1=Ef_2 + H_L$)**	9.62 m	13.02 m	6.73 m	9.18 m
11.	**d/s Specific Energy (Ef_2)**	8.68 m	10.54 m	6.22 m	7.61 m
12.	**Level at which jump will form(d/s T.E.L – Ef_2)**	193.7 m	191.34 m	194.74 m	192.85 m
13.	**Froude's No. ($F= q/\sqrt{gD_1^3}$)**	2.05 m	2.58 m	1.92 m	2.47 m

It can be seen from the table that the maximum value of 5(D2-D1) is 32.75 m for the worst case, i.e. high flood flow with concentration and retrogression. Hence, we provide a slightly conservative value of 34 m as the length of downstream floor.

The lowest level at which jump will form, is 191.34 m and hence, we provide the downstream floor at a level of say, 191.00 m.

Hence, the downstream floor is provided at R.L of 191.00 m and is equal to 34 m in length.

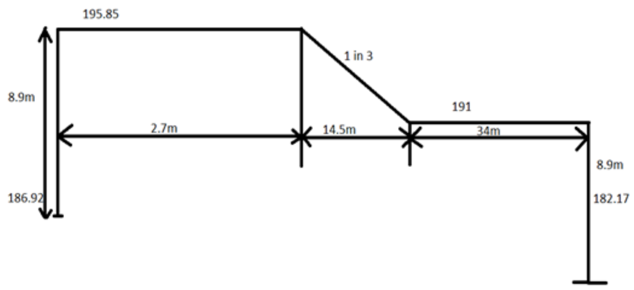

Figure 5: Reduced level of floor length, RL = 191m

6. Depth of sheet pile lines from scour considerations

i. Depth of scour

Total discharge passing through the under sluices = 2838.78cumecs
Overall waterway of under sluices = 105 m
Average discharge intensity = 2838.78/105 = 27.026cumecs/m

Depth of scour, R = $1.35(q^2/f)^{1/3} = 1.35(27.026/1)^{1/3} = 12.16$ m ≈ 13 m

ii. U/s sheet pile

On the u/s side, provide cut off at 1.25R below u/s water level= 1.25 *13 = 16.25 m
R.L of bottom of u/s cut off = 203.17 – 16.25 = 186.92 m
195.85 – 186.92 = 8.9 m
Provide sheet pile line at elevation of 188.7 m i.e. at a depth of 8.92 m.

iii. D/s sheet pile

On the d/s side, provide cut off at 1.5R below d/s water level= 1.5 *13 = 19.5 m
R.L level of bottom of d/s cut off = 201.67 – 19.5 = 182.17 m
Provide sheet pile line at an elevation of 182.17 m i.e. at a depth of 8.9 m

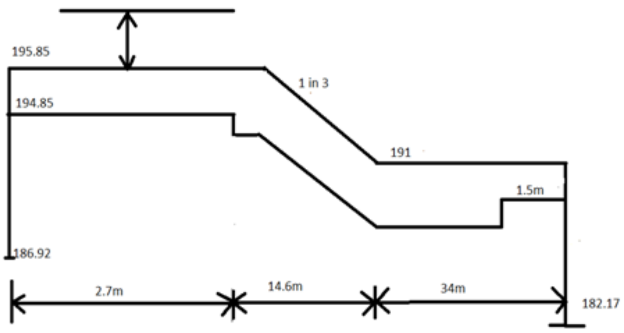

Figure 6: Total Floor Length

7. Pressure Calculations

For determining uplift pressures according to Khosla's theory, it is essential to assume the floor thickness at the upstream and downstream cut off.

Let us assume the floor thickness of 1.0 m at upstream end and 1.50 m at the downstream end, as in the figure.

i. Upstream pile line

$\phi_{E=}$ $_{(1/Л)}\cos^{-1}\left(\frac{(\lambda-2)}{\lambda}\right)$ = 36.34≈37%
Л = $(1+\sqrt{1+\alpha^2})/2$= 3.425
$\Phi_{D=}$ $_{(1/Л)}\cos^{-1}\left(\frac{(\lambda-2)}{\lambda}\right)$= $_{(1/Л)}\cos^{-1}\left(\frac{(3.425-2)}{3.425}\right)$ =25%
ϕ_{C1}= 100 – ϕ_E = 100 -37 =63%
$\varphi D1$ = 100 – ϕ_D= 100. 25 = 75%

D/S pile line:

d = 8.9 m
b = 51.3 m
α= b/d = 51.3/8.9 = 5.76

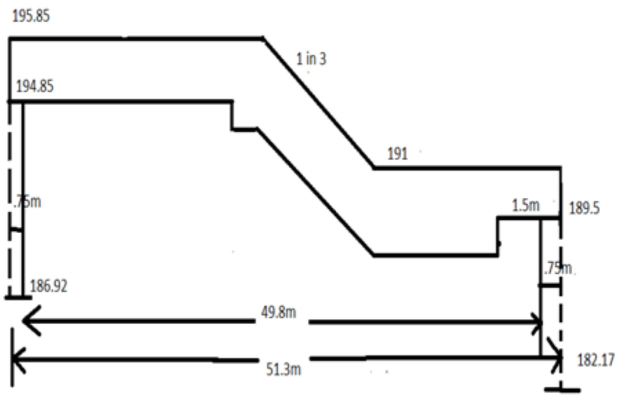

Figure 7: Correction of floor length

Table 2: Data of upstream pile line

Conditional flow	U/s water line (m)	D/s water line (m)	Head, H (m)	Ht./ elevation of sub soil HG line above datum					
				u/s pile line no.(1)			d/s line pile no. (2)		
				ϕ_{E1}= 100%	Φ_{D1}= 75%	Φ_{C1}= 68.19%	ϕ_{E2}= 34.11%	Φ_{D2}= 25%	Φ_{C2}= 0%
No flow, max static head	201.35	191 (no water d/s)	10.3	10.35 201.35	7.61 198.76	7.1 198.1	3.53 194.53	2.6 193.6	0 191
High flood conc. and retrogression	203.17	201.67	1.5	1.5 203.17	1.13 202.8	1.1 202.77	0.52 202.19	0.38 202.05	0 201.67
Flow at pond level (with conc. and retrogression)	201.35	200.30	1.05	1.05 201.35	0.79 201.09	0.72 201.02	0.36 200.92	0.26 200.56	0 200.30

Table 3: Data of upstream pile line

Distance from d/s end of crest i.e. start of glacis	glacis level (in m)	HFL q=42.07 cumec/m		Pond level flow q=26.32cumec/m	
		E_{f1} u/s TEL- glacis level (204.35-col 2)	y_1	E_{f1} u/s TEL- glacis level (202.06-col 2)	y_1
0	195.85	8.5	-	6.21	-
3	194.85	9.5	4.3	7.21	2.9
6	193.85	10.5	4	8.21	2.5
9	192.85	11.5	3.8	9.21	2.3
12	191.85	12.5	3.1	10.21	2.1
13.53	191.34	13.01	3	10.72	2.0

Pre Jump Profile calculation

Post jump profile

From table,
Froude No. for high flood condition, F = 2.58
$F^2 = (2.58)^2 = 6.7$
Depth D1 for high flood condition = 3 m
Froude No. for pond level condition, F = 2.47
$F^2 = (2.47)^2 = 6.1$
Depth D1 for pond level condition = 2.26 m

Now the following table is completed

Table 4: Post jump data

$\frac{x}{y_1}$ on plate	High Flood Flow $F^2 = 6.7$, y_1 =3m			Pond Level Flow F^2=6.1, y_1=2.26		
	$\frac{y}{y_1}$ from plate	Y	x =(col.1) *3	$\frac{y}{y_1}$ from plate	y	x = (col. 1) * 2.26
1	1.3	3.6	3	1.3	2.712	2.26
2.5	1.9	5.7	7.5	1.7	3.842	5.65
5	2.5	7.5	15	2.1	4.75	11.3
7.5	2.8	8.4	22.5	2.4	5.43	16.95
10	3	9	30	2.8	6.33	22.6
12.5	3.3	9.9	37.5	3	6.78	28.25

Hydraulic jump profile for two flow conditions, their H.G Lines and the uplift pressure diagrams are now plotted. The H.G Line and uplift pressure diagram for static head is also plotted.

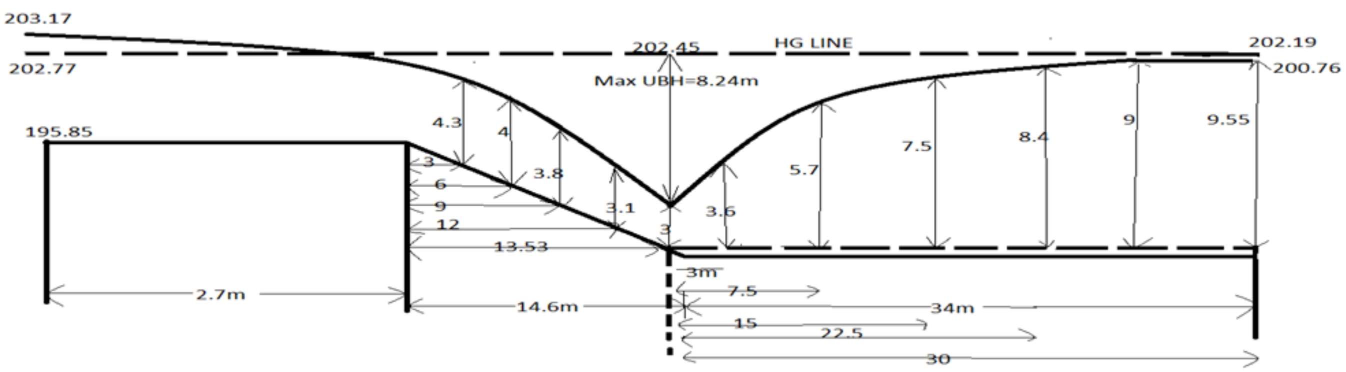

Figure 8: Unbalanced head in jump trough at HF flow

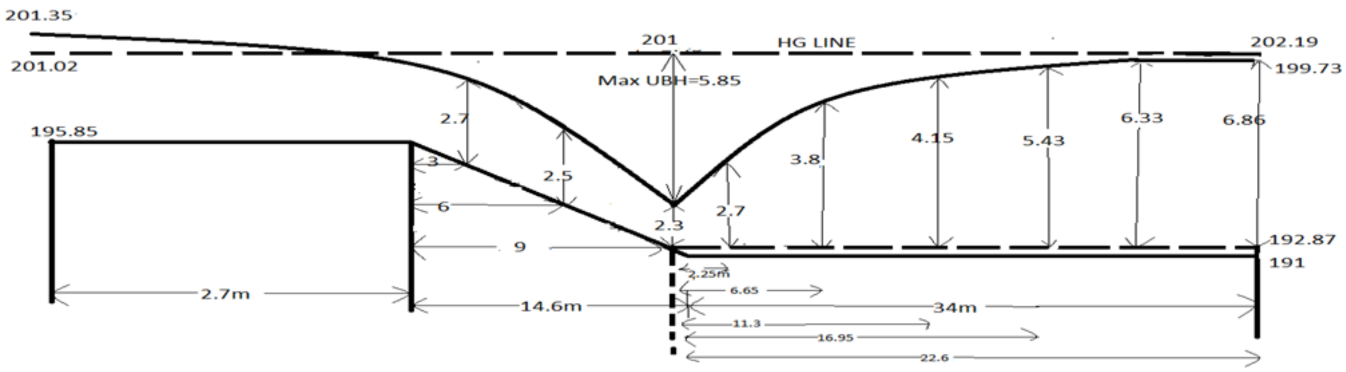

Figure 9: Unbalanced head in jump trough at pond level flow

H_j profile are plotted and from fig it is found that max UBH in the jump trough is 8.24 m.

Thickness of glacis shall therefore be designed for 2/3rd of head i.e.

= (2/3)*8.24=5.5m = or for static condition head, = Static head at pond level

8. Design of Other Barrage Bays portion

Discharge intensity and head loss under different flow conditions The values of q, HL, the water levels and energy levels for all the four cases are Tabulated in following table;

S. No	Item	High Flood Flow		Pond Level Flow	
		Condition 1(a) Without conc. & Retrogression	condition 1(b) 20% conc. 0.5 m retro.	Condition 2(a) Without conc. & retrogression	condition 2(b) 20% conc. 0.5 m retro.
1.	Discharge Intensity q(cumec/m)	28.26	34	16.46	19.7
2.	u/s water level	203.17 m	203.17 m	201.35 m	201.35 m
3.	d/s water level	202.17 m	202.17 m	200.80 m	200.80 m
4.	u/s T.E.L	203.38 m	204.20 m	201.51 m	202.05 m
5.	d/s T.E.L	202.38 m	201.88 m	200.96 m	200.46 m
6.	H_L	1 m	2.32 m	0.55 m	1.59 m
7.	E_{f1}	8.59 m	11.5 m	5.72 m	8.06 m
8.	Level at which jump will form (d/s TEL – E_{f2})	194.77 m	192.66 m	195.76 m	194.05 m
9.	E_{F2}	7.60 m	9.22 m	5.2 m	6.41 m
10.	Y_1	2.61 m	2.57 m	1.9 m	1.78 m
11.	Y_2	6.7 m	8.38 m	4.53 m	5.83 m
12.	Length of concrete floor required : i.e $5*(Y_2 – Y_1)$	20.45 m	29.05 m	13.15 m	20.25 m
13.	Froude's No. $F_1 = \frac{q}{\sqrt{gY_1^3}}$	2.14	2.63	2	2.65

The lowest level at which jump will form, is 192.5 m and hence, we provide the downstream floor at a level of say, 192.50 m.

Hence, the downstream floor is provided at R.L of 192.50 m and is equal to 31 m in length.

d/s glacis length with 3:1 slope = 3 (197.2 – 192.5) = 14.1m

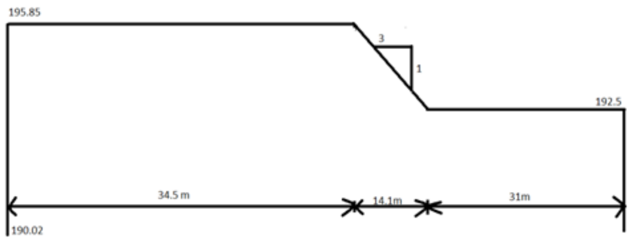

Figure 10: Total floor length

9. Design of Silt Excluder at Okhla Barrage

The silt excluder is to be designed for the Agra Canal head off taking from Yamuna River with dominant discharge of 9911 cumecs.

The other of canal and excluder are as below:

Canal discharge 250 cumecs
Width of under sluice span of the barrage where canal 15 meters
Head regulator is to be provided with an excluder.
River bed slope 1 in 5000
Average sediment diameter 0.32 mm
Head available for design 0.8 m
Manning's constant 0.016

Design:

A. *Escape Discharge*

Since the canal is of larger capacity, an escape discharge equal to 20% of canal discharge is chosen, i.e. 0.2 * 250 = 50 cumecs

Hence a discharge of 50 cumecs is selected.

B. *Width of Excluder*

Since the span of under sluice bay is 15 m, it is proposed to cover only one bay of the barrage.

C. *Design of Tunnels*

(a) Number of tunnels: Usually 4 to 6 tunnels are provided. In this case 6 tunnels are being provided.

(b) Since the width of under sluice bay is 15 m and thickness of divide wall is taken 0.6 m, the tunnel width at exit= (15 – 5*0.6)/6 = 2 m

Discharge through one tunnel = 50/6 = 8.33 cumecs

Let us adopt a discharge of 8.35 cumecs.

(c) The height of the tunnel is chosen such that the velocity through it is of the order of about 2 m/sec or more. At the exit the velocity may be taken higher up to 3 m/sec.

Adopting exit velocity of 2.5 m/sec.

Now Area of the tunnel at exit

= 8.35/2.5 = 3.34 m2 and,

Height =3.34/2 = 1.67 m

Hence provide 1.67 m height of the tunnel at the exit. This height is provided throughout the tunnel length.

The tunnel widths at different sections are adjusted so as to give equal head loss in all the tunnels. This is done by trial and error method. The tunnel width in the straight portion works out to be 2.60 m.

Width of the tunnel at the entry can be approximately evaluated by the following criterion.

From above, the discharge intensity at entrance, for a tunnel height of 1.67 m, works out to be 3.876 cumecs i.e. at entrance a tunnel width of 8.35/3.876 = 2.15 m approximately.

For better smooth entry, the tunnel width at entry has been taken equal to twice at e*it i.e.

2*2 = 4 meters.

D. *Head loss in different tunnels:*

The head losses in different tunnels are calculated to ascertain if head losses in different tunnels are same. The calculation of head loss for the largest tunnel is shown below:

Head loss in Tunnel No. 1 (longest)

I. Friction loss in bell mouthing

Area = [(4 + 2.60)/ 2] * 1.67 = 5.51sq.m
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Wetted Perimeter = (4 + 2.60) + 2 * 1.67 = 9.94 m
R = A / P = 5.51 / 9.94 = 0.554 m
Average Velocity, Q / A = 8.35 / 5.51 = 1.52 m/sec
Friction loss by Manning's formula:
= (V2Ln2) / R4/3 = [(1.52)2*3.8*2.56*10-4] / 0.445
=0.00494 m
II. Friction loss in straight reach
Area = 2.6 * 1.67 = 4.342 sq.m
Wetted Perimeter = 2(2.6 + 1.67) = 8.54 m
R = A / P = 4.342 / 8.54 = 0.51 m
Velocity, V = 8.35 / 4.342 =1.923 m/sec.
Hf= [(1.923)2 * 80 * (0.016)2 / (0.51)4/3 = 0.186 m
III. Friction loss in bend
Average area = [(2..6 + 2) / 2]*1.67 = 3.84 sq.m
Wetted perimeter = 2.6 + 2 + 2 * 1.67 = 7.94 m
R = A / P = 3.84 / 7.94 = 0.48 m
Velocity, V = 8.35 / 3.84 =2.17 m/sec.
Hf= (V2Ln2) / R4/3 = [(2.17)2 * 14 * (0.016)2 / (0.48)4/3 = 0.045 m
IV. Friction loss in remaining length of tunnel
Area = 1.67 * 2 = 3.34 sq.m
Wetted perimeter = 2(1.67 + 2) = 7.34 m
R = A / P = 3.34 / 7.34 = 0.455 m
Velocity, V = 8.35 / 3.34 =2.5 m/sec.
Hf= (V2Ln2) / R4/3 = [(2.5)2 * 1 * (0.016)2 / (0.455)4/3 = 0.0046 m
Total friction loss = I + II + III + IV
=0.00494 + 0.186 + 0.045 + 0.0046 = 0.24054 M
V. Loss at entry
He = 0.2[(V12 – V22) / 2g]
Velocity at the entry = 8.35 / (4 * 1.67) = 1.25 m/sec
He = 0.2[(1.9232 – 1.252) / 2*9.8] = 0.0218 m
VI. Loss due to bend
Hb= F * (V2 / 2g) * (Ø/180)
Where, F = a coefficient which varies with radius and width of tunnel
F = 0.124 + 3.104 *(S /2R)1/2
Ø = angle of deviation = 180
R = radius = 45 m
S = width of tunnel = (2.6 + 2) / 2 = 2.3m
F = 0.124 + 3.104 *(2.3 /2* 45)1/2 = 0.62
Hence head loss at the bend = 0.774 * (2.17)2 /19.6 *18 / 180
=0.0186 m
VII. Head loss due to change in velocity
Hbe= 0.3 (V1
2 – V2
2) / 2g = 0.3 (2.52 – 1.9232) / 2 * 9.8
= 0.026 m
Hence total loss through tunnel 1
=0.24 + 0.0218 + 0.0182 + 0.026 = 0.306 m
The various head losses for all the tunnels are similarly calculated and are given in the
ne*t table.
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E. Tunnel Layout

To trap a major portion of coarse material, the tunnel bed level is kept such that the top slab is flush with the sill at the head regulator. If the thickness of top slab is 0.2 m then the bottom of the tunnel is 1.73 m from the regulator sill.

(ii) Approach

To increase the zone of suction at the upstream mouth, bell mouthing of the tunnels has been done according to

*2 / (0.75)2 + y2 / (0.25)2 =1

The radius of bell mouthing in plan varies from 2 to 8 times the tunnel width, the radii increasing for tunnels away from canal head regulator.

(iii) Exit

The tunnels have been throttled at the e*it to increase the velocity to prevent sediment deposits.

(iv) Bend Radius

It is kept 8 to 18 times the tunnel width. In this design it is kept varying from 10 to 17 times the tunnel width.

(v) The top slab has been protruded into the river by about 1.07 meter at the entry, to increase suction effect of the tunnels to draw in more sediment. The protrusion has been extended and elliptically shaped at the entry.

F. Escape Channel

NO special outfall channel is required as the sediments and escape discharge will pass down the barrage.

10. Design of Canal Head Regulator

Fixation of Crest level and water way

Full Supply discharge = 250 cumecs
Anticipated maximum full supply level of canal =201.10m
Bed level of canal = 196.0 m
Safe e*it gradient for canal bed material = 1/5
The crest level of canal head regulator is kept 1.2 - 1.5 m higher than crest level of under sluices.
The crest level of under sluices = 195.85 m
Pond level = 201.35 m
u/s H.F.L = 202.17 m
The crest level of regulator is kept 1.5m high than under sluices.
As silt excluder is used, raise crest level by 1 m and further by 1.05 m.
_Crest level of regulator = 195.85 + 1.5 + 1 + 1.05 = 199.4 m
Fig 5.1 (Fixation of Crest Level and Waterway)
Now fix the waterway for regulator, such that the full supply discharge of 250 cumecs can pass through it.
Discharge 'Q' through regulator is given as,
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Q = 2/3 C1 * L *√2g {(h+h1)3/2 – ha
3/2} + C2 * L * h1*√(2g * (h + ha))
Here, C1 = 0.577, C2 = 0.80
Neglecting head due to velocity of approach, 'ha'
Here, Q = 250, h = 0.25, h1 = 1.7
Now,
250 = 2/3 * 0.577 * L *√(2 * 9.8)2 * (0.25)3/2
+ 0.80 * L * 1.7 *√(2*9.8*0.25)
250 = 0.212 L + 0.3010 L
L = 77.59 ~ 77.6 m
Provide 10 bays of 7.8 m each, giving a clear water way of 78 m.
Provide 9 piers of 1.5 m each
Overall water way of regulator = 78 + (9 * 1.5) = 91.5 m

Hydraulic conditions for various flow conditions

(i) Full supply discharge passing down regulator during high flood

When u/s water level is 202.17 m, water shall pass over the regulator and the gated opening provided between the silt level and pond level shall have to be adjusted by partially opening this gate.

Let the gate opening be '*' meters. The discharge can then be calculated by submerged orifice formula i.e.,

Q = Cd * A *√ (2gh)
Here, Q = 250 cumecs
A = L ** = 78.* m2
Cd = 0.62
h = head causing flow = 202.17 – 201.10 = 1.07 m
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250 = 0.62 * 78.**√(2*9.8*1.07)
* = 1.13 m
Velocity of flow through opening, v = 250/(78 * 1.13) = 2.83 m/s
Loss of head at entry = 0.5 * v2/2g
= 0.5 * (2.832/(2*9.8))= 0.204m
T.E.L just u/s of gate = 202.17 + 0.199 = 202.37 m
T.E.L just d/s of gate = 202.37 – 0.204 = 202.16 m
d/s water level = 201.10 m
Head Loss, HL = 202.16 – 201.10 = 1.06 m
Discharge intensity, q = 250/78 = 3.20 cumecs/m
(ii) Full supply discharge passing down regulator at pond level
Head Loss, HL = 201.35 – 201.10 = 0.25 m
Discharge intensity, q = 3.20 cumecs/m
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Depth of sheet piles from scour considerations

Discharge intensity, q = 3.20 cumecs/m
Depth of scour, R = 1.35 * (q2/f)1/3
= 1.35 * (3.202/1)1/3 = 2.93 m
(i) d/s sheet pile
Provide d/s cutoff upto 1.5R below d/s water level = 1.5 * 2.93
= 4.39 m
R.L of bottom of d/s cutoff = 201.10 – 4.39 = 196.71 m

Table: Data of high flood flow and pond level flow

S.No.	Item	High flood flow condition	Pond level flow condition
1	Discharge Intensity (q)	3.20cu/m	3.20cu/m
2	Upstream Water level	202.17m	201.35m
3	Downstream Water level	201.10m	201.10m
4	U/S T.E.L	202.17m	201.35m
5	D/S T.E.L	201.10m	201.10m
6	Head Loss(HL)	1.06m	0.25m
7	D/S Specific Energy(Ef2)	2.05m	1.90m
8	U/S Specific Energy(Ef1= Ef2+HL)	3.11m	2.15m
9	Level at which jump will form(D/S T.E.LEf2)	199.05m	199.2m
10	Pre jump depth D1 corresponding to Ef1	0.5m	0.6m
11	Post jump depth D2 corresponding to Ef2	1.80m	1.70m
12	Length of concrete floor required beyond jump 5(D2- D1)	6.5m	5.5m
13	Froude's Number, F=q/√gD1 3	2.89	2.19

11. Discussions

1. The barrage has been designed independently by only considering the required data.
2. There have been some changes made in the original specifications of the barrage as per the requirements.
3. The dimensions and number of gates of the under sluice and other barrage bays have been changed corresponding to the economic conditions.
4. The crest level of head regulator has been increased more as a silt excluder has to be provided.
5. In the design of silt excluder, the tunnels have been provided in one bay of under sluice only, in contrast to the two bays in the original design. As, the silt content in the Yamuna river decreases to some e*tent upon reaching that portion of Delhi.

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