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Article

Addressing Water Resources Problem In Iraq through Improving Weir Design: The Case of Southern Baghdad

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Abstract: This study addresses Iraq's severe water crisis, focusing on the optimization of submersible weir designs, particularly in Southern Baghdad. The research identifies a critical gap in effective weir design for managing water resources amidst challenges like climate change and river interventions. Using the HEC-RAS model for hydraulic simulations, combined with remote sensing and geographic information systems, potential weir locations were assessed. The study aims to enhance flood control and reduce water wastage to the Arabian Gulf by improving weir design. The findings suggest that well-designed weirs can significantly enhance water security, protect agricultural lands, and support sustainable development in Iraq.

Keywords: Weir, Arabian Gulf, HEC-RAS, Remote Sensing.

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1. Introduction

Danger of water scarcity in the land of the two rivers [1]. Is that the most important issue facing the countries of the Tigris and Euphrates basins [2]. Is the mismatch between available resources and increasing demand [3]. As a result, Iraq's water imports have arisen [4]. Over time, this is expected to increase. Turkish projects have been completed [5]. In the Tigris and Euphrates basin, control and storage projects are very dangerous [6]. On Iraqi imports, the risks can be imagined. The annual imports of the Euphrates River decreased to (25.75) billion cubic meters per year in 1970. Compared, in the revenues of the 1990s. With its annual revenues of (8.97) billion cubic meters per year for the period from (1990-1997) [7].

It is worth noting that that year witnessed the filling of the Kiban in Turkey and the Tabqa in Syria [8]. Similarly, the Ataturk reservoir, which had a storage capacity of (48.7) billion cubic meters, was filled, while the Tigris decreased by (16.330) annual rate in 19970 compared to an annual revenue of 9 billion cubic meters for the period from 1992 to 1997. During the same period, the water wasted to the Arabian Gulf via the Shatt al-Arab was estimated at about (33.22) billion cubic meters annually (1993-1998) [9]. Accordingly, a strong policy must be developed for the Iraqi water resources file, especially of the south [10].

The proper management of this vital natural resource [11]. This study aims to establish submersible dams with a storage capacity of not less than (3 million cubic meters) as a project. The need to establish submersible dams within water resources projects is very important in order to control and enhance the management of these projects in Iraq. The importance of establishing submersible dams in all Iraqi governorates is to achieve water stability and regulate the flow of rivers, which contributes to protecting agricultural lands from floods and drought, enhancing food security, and providing the necessary water for industry and drinking. In addition, these dams contribute to generating electricity and providing new job opportunities, and an important tourist attraction, which enhances sustainable development in the country [12].

Surveying works by the competent directorates to confirm Weir proposal AL madain city shown in Fig (2&3) below covers an area of 210 km2 and found in south east Baghdad. Geographically located between 32° 29′ 00″ N latitude and 45° 50′ 00″ E longitude [13]. Which included locating the storage weirs and fixing the weirs height, width [14].

In addition to the lengths of the abutment retaining walls It is a preliminary site and not a final site that needs detailed studies [15]. But were need a plan to propose a weir, then study each basin to differentiate which proposal is more economically feasible [16]. More water storage in order to achieve spatial development for any region. At the same time preserve water from waste until reach a result that can propose a weir site [17].

Then the proposals are submitted on the basis of preference as a feasibility study and then submitted to the relevant departments where they conduct specialized hydrological studies to calculate the discharges and other matters of designing the proposed weir [18]. The figure (1), below represent the proposed site for the construction of this type of weir (Storage weir type).

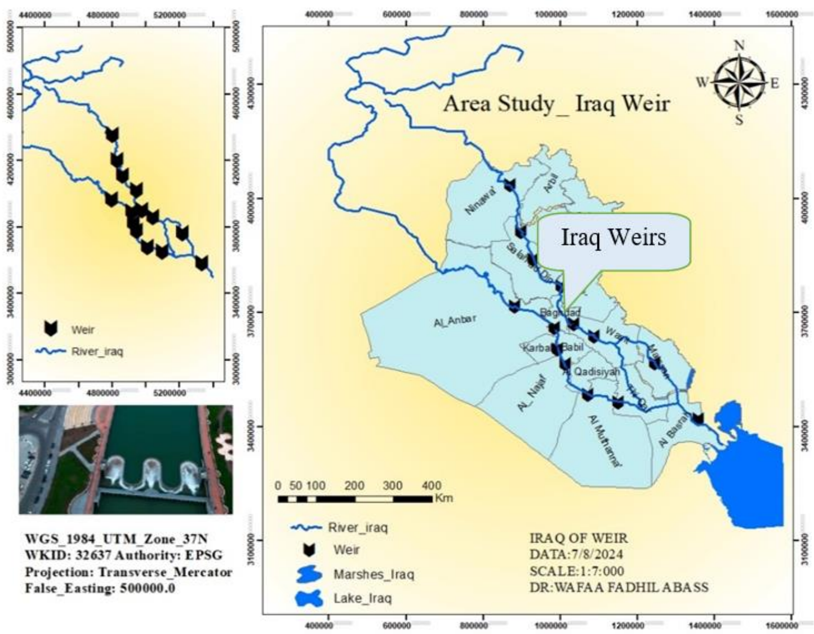

Figure 1. Site of weir -A types in Proposed Areas (GIS software) (the researcher,2024)

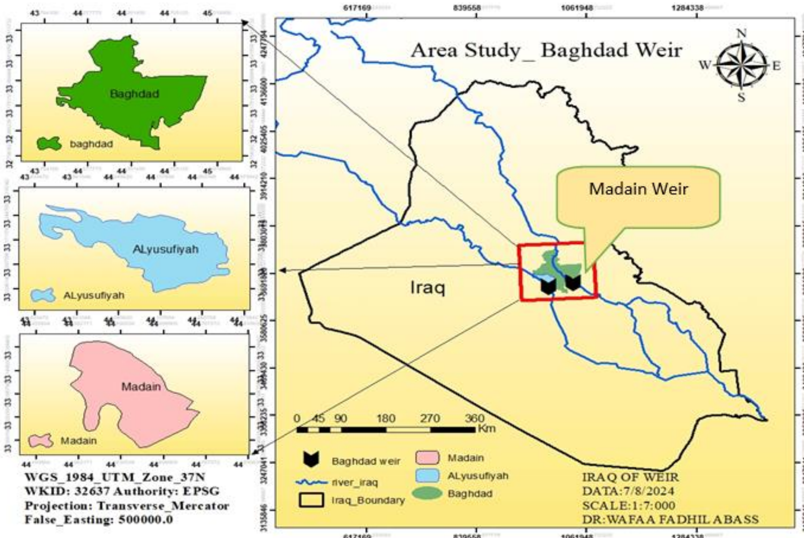

Figure 2. Site of weir types Figure on Euphrates &Tigris Rivers (GIS software) (the researcher,2024).

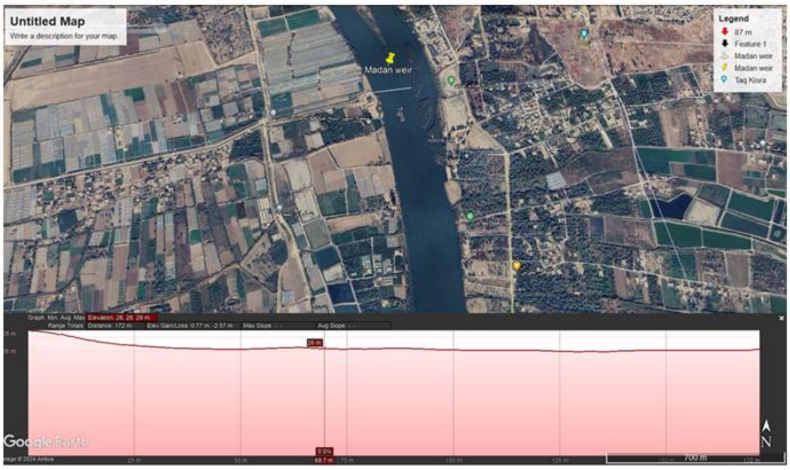

Figure 3. Google earth IMAGE for the study area type -A (global mapper) (the researcher,2024)

Hydrological Study.

The hydrological time series data used in this study are the maximum and minimum discharges of the Khaboor River, which are recorded in a station in Zakho city provided for the time period from (2004) to (2024) and as shown in table (1) below;

Table 1. Maximum and minimum discharge recorded of Khaboor river from period (2004 to 2024), at Cham Sirmo station

Dates	Max Discharge (m3/sec)	Minimum discharge (m3/sec)
08-03-04	391	5.13
26-04-05	386	5.37
04-02-06	674	7
06-05-07	429	7
01-04-08	109	3.6
20-12-09	309	3.5
17-03-10	321	3.5
23-04-11	996	4.3
13-04-12	435	14
28-01-13	656	33.12
31-01-13	810.8	20
28-01-14	612	10.9
11-04-15	413	13.5
12-04-16	407.6	16.1
15-04-17	737	14.8
28-01-18	612.8	10.9
17-03-19	737	33.1
18-03-20	1450	41.24
24-03-21	764	5.888
4-04-22	220	3.8
4-01-23	350	8.1
2-03-24	1200	44

From above data;

Max Discharge = 1450 m3/sec

Average discharge for total time series = 302.72 m3/sec

Average discharge from Max. discharges (column 2) = 592.12 m3/sec Average discharge from min. discharges (column 3) = 13.31 m3/sec Methodology [19].

Table 2. Discharges quantities used in hydraulic model of the weir

PF	Discharges in m3/sec	PF	Discharges in m3/sec
PF-1	1450	PF-22	33.1
PF-2	996	PF-23	20
PF-3	810.8	PF-24	16.1
PF-4	764	PF-25	14.8
PF-5	737	PF-26	14
PF-6	737	PF-27	13.5
PF-7	674	PF-28	10.9
PF-8	656	PF-29	10.9
PF-9	612.8	PF-30	7
PF-10	612	PF-31	7
PF-11	435	PF-32	5.888
PF-12	429	PF-33	5.37
PF-13	413	PF-34	5.13
PF-14	407.6	PF-35	4.3
PF-15	391	PF-36	3.6
PF-16	386	PF-37	3.5
PF-17	321	PF-38	3.5
PF-18	309	PF-39	302.714
PF-19	109	PF-40	220
PF-20	41.24	PF-41	350
PF-21	33.12	PF042	1200

2. **Materials and Methods**

2.1 Hydraulic Design for Weir Type A

The hydraulic study relied on the discharges mentioned in the hydrological study on the basis that the maximum discharge recorded in (2-3-2024) which is equal to (1450 m3/sec) is a very high discharge that has not occurred for many years, according to the engineers and hydrologic specialists in the field of hydrology Therefore, were relied on this discharge in the hydraulic study , since the body of the structure is solid concrete, so there is no risk for it if the discharge exceed this value, because in terms of stability and creep [20].

it depends mainly on the height of the weir .The simulation model for storage weirs was built using the HEC-RAS (Giustolisi et al.,2023).which is a computer program that models the hydraulics of water flow through natural rivers and other channels .In hydraulic simulation model the discharges in the hydrological study arranged in descending order and named with (PF) starting from the highest discharge (PF-1) to the lowest discharge (PF-38) as in table (2) , and these discharges used during operation process of the model also average total discharge was named (PF-39) and used in the operation of the hydraulic model.

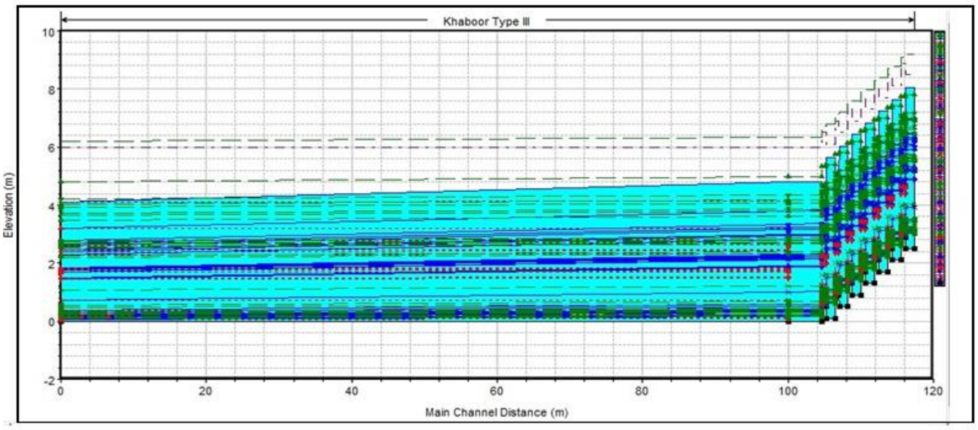

Figure 4. Flow profile of all discharges (PF-1 to PF -42) (the researcher,2024)

In addition to the above discharge quantities, the weir dimensions (length, width, and height), upstream slope and downstream slope that were calculated in the structural study report were used in the building the hydraulic model of the weirs, and the following results were obtained;

1. All discharges were flexibly passed over the weir without any hydraulic jump, as shown in profiles in the figure (4).
2. Depths of water at over the weir crest and downs steam (d/s) location when discharge was (PF-1) = 1450 m3/sec) = 4.76 m, and 4.82 m respectively as shown in figures (3 and 4).
3. The Froude Number in all sections is very few, and this indicates that there is no hydraulic jump. The discharges can be passed flexibly, the figure (5); represents the Froude number in all stations when passing discharge was maximum flow (PF-1 = 1450 m3/sec).

Cross Section Output

File Type Options Help

River: Khaboor Profile: PF 1

Reach Type III RS: 116.74 Plan: Plan 09

Plan: Plan 09 Khaboor Type III RS: 116.74 Profile: PF 1

E.G. Elev (m)	9.19	Element	Left OB	Channel	Right OB
Vel Head (m)	1.16	Wt. n-Val.		0.013	
W.S. Elev (m)	8.04	Reach Len. (m)	0.01	0.01	0.01
Crit W.S. (m)		Flow Area (m2)		304.59	
E.G. Slope (m/m)	0.000499	Area (m2)		304.59	
Q Total (m3/s)	1450.00	Flow (m3/s)		1450.00	
Top Width (m)	55.00	Top Width (m)		55.00	
Vel Total (m/s)	4.76	Avg. Vel. (m/s)		4.76	
Max Chl Dpth (m)	5.54	Hydr. Depth (m)		5.54	
Conv. Total (m3/s)	64894.9	Conv. (m3/s)		64894.9	
Length Wtd. (m)	0.01	Wetted Per. (m)		66.08	
Min Ch El (m)	2.50	Shear (N/m2)		22.57	
Alpha	1.00	Stream Power (N/m s)		107.44	
Frctn Loss (m)	0.00	Cum Volume (1000 m3)		28.99	
C & E Loss (m)	0.09	Cum SA (1000 m2)		6.39	

Figure 5. Properties of flow at cross section over crest of weir when. Discharge = 1450 m3/sec (the researcher,2024)

Cross Section Output

File Type Options Help

River: Khaboor Profile: PF 1

Reach Type III RS: 104.6 Plan: Plan 09

Plan: Plan 09 Khaboor Type III RS: 104.6 Profile: PF 1

E.G. Elev (m)	6.35	Element	Left OB	Channel	Right OB
Vel Head (m)	1.53	Wt. n-Val.		0.013	
W.S. Elev (m)	4.82	Reach Len. (m)	4.60	4.60	4.60
Crit W.S. (m)		Flow Area (m2)		265.08	
E.G. Slope (m/m)	0.000770	Area (m2)		265.08	
Q Total (m3/s)	1450.00	Flow (m3/s)		1450.00	
Top Width (m)	55.00	Top Width (m)		55.00	
Vel Total (m/s)	5.47	Avg. Vel. (m/s)		5.47	
Max Chl Dpth (m)	4.82	Hydr. Depth (m)		4.82	
Conv. Total (m3/s)	52242.6	Conv. (m3/s)		52242.6	
Length Wtd. (m)	4.60	Wetted Per. (m)		64.64	
Min Ch El (m)	0.00	Shear (N/m2)		30.98	
Alpha	1.00	Stream Power (N/m s)		169.46	
Frctn Loss (m)	0.00	Cum Volume (1000 m3)		25.80	
C & E Loss (m)	0.00	Cum SA (1000 m2)		5.75	

Figure 6. Properties of flow at d/s cross section of weir when. Discharge = 1450 m3/sec (the researcher,2024)

HEC-RAS Plan: Plan 09 River: Khaboor Reach: Type II Profile: PF 1

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Type II	117.94	PF 1	1450.00	2.50	8.04		9.19	0.000499	4.76	304.64	55.00	0.65
Type II	116.74	PF 1	1450.00	2.50	8.04		9.19	0.000499	4.76	304.59	55.00	0.65
Type II	116.73	PF 1	1450.00	2.90	7.07	7.03	9.11	0.001212	6.32	229.50	55.00	0.99
Type II	116.13	PF 1	1450.00	2.90	7.03	7.03	9.11	0.001256	6.39	226.95	55.00	1.00
Type II	116.12	PF 1	1450.00	2.10	7.64		8.79	0.000499	4.76	304.64	55.00	0.65
Type II	114.92	PF 1	1450.00	2.10	7.64		8.79	0.000499	4.76	304.59	55.00	0.65
Type II	114.91	PF 1	1450.00	2.50	6.67	6.63	8.71	0.001212	6.32	229.50	55.00	0.99
Type II	114.31	PF 1	1450.00	2.50	6.63	6.63	8.71	0.001256	6.39	226.95	55.00	1.00
Type II	114.3	PF 1	1450.00	1.70	7.24		8.39	0.000499	4.76	304.67	55.00	0.65
Type II	113.10	PF 1	1450.00	1.70	7.24		8.39	0.000499	4.76	304.62	55.00	0.65
Type II	113.09	PF 1	1450.00	2.10	6.27	6.23	8.31	0.001212	6.32	229.50	55.00	0.99
Type II	112.49	PF 1	1450.00	2.10	6.23	6.23	8.31	0.001256	6.39	226.95	55.00	1.00
Type II	112.48	PF 1	1450.00	1.30	6.84		7.99	0.000499	4.76	304.64	55.00	0.65
Type II	111.28	PF 1	1450.00	1.30	6.84		7.99	0.000499	4.76	304.59	55.00	0.65
Type II	111.27	PF 1	1450.00	1.70	5.87	5.83	7.91	0.001212	6.32	229.50	55.00	0.99
Type II	110.67	PF 1	1450.00	1.70	5.83	5.83	7.91	0.001256	6.39	226.95	55.00	1.00
Type II	110.66	PF 1	1450.00	0.90	6.44		7.59	0.000499	4.76	304.64	55.00	0.65
Type II	109.46	PF 1	1450.00	0.90	6.44		7.59	0.000499	4.76	304.59	55.00	0.65
Type II	109.45	PF 1	1450.00	1.30	5.47	5.43	7.51	0.001212	6.32	229.50	55.00	0.99
Type II	108.25	PF 1	1450.00	1.30	5.43	5.43	7.51	0.001256	6.39	226.95	55.00	1.00
Type II	108.24	PF 1	1450.00	0.50	6.04		7.19	0.000499	4.76	304.67	55.00	0.65
Type II	107.04	PF 1	1450.00	0.50	6.04		7.19	0.000499	4.76	304.62	55.00	0.65
Type II	107.03	PF 1	1450.00	0.90	5.07	5.03	7.11	0.001212	6.32	229.50	55.00	0.99
Type II	106.43	PF 1	1450.00	0.90	5.03	5.03	7.11	0.001256	6.39	226.95	55.00	1.00
Type II	106.42	PF 1	1450.00	0.10	5.64		6.79	0.000499	4.76	304.64	55.00	0.65
Type II	105.22	PF 1	1450.00	0.10	5.64		6.79	0.000499	4.76	304.58	55.00	0.65
Type II	105.21	PF 1	1450.00	0.50	4.67	4.63	6.71	0.001212	6.32	229.52	55.00	0.99
Type II	104.61	PF 1	1450.00	0.50	4.63	4.63	6.71	0.001256	6.39	226.95	55.00	1.00
Type II	104.6	PF 1	1450.00	0.00	4.82		6.35	0.000770	5.47	265.08	55.00	0.80
Type II	100	PF 1	1450.00	0.00	4.81	4.13	6.34	0.000774	5.48	264.71	55.00	0.80
Type II	0	PF 1	1450.00	-0.02	4.11	4.11	6.19	0.001256	6.39	226.92	55.00	1.00

Figure 7. Froude number in all stations when passing discharge equal maximum flow (PF-1 = 1450 m3/sec) (the researcher,2024)

2.2 Hydraulic Design Sluice Gate

From hydraulic simulation mode

y1= 2.52 + 4.76 m = 7.28 m

Average discharge of time series data from hydrological study = 302.72 m3/sec

Usually, the sluice gates are designed to pass from 10% to 20 % percent of average of total discharge of the time series data.

After entering the value of y1 in the program and giving a different value of y2, we obtained that when y2 = 1.83 m will get the q = 12.42 m2 /sec (per 1m width) as showing in output result of the program.

If width of sluice open = 1.52 m

Discharge of one sluice gate Q = 12.42 * 1.52 = 18.88 m3/sec

Use two sluice gates …. Total discharge = 18.88 * 2 = 37.76 m3/sec

And 37.76/302.72 = 0.1247 =12.47% ok

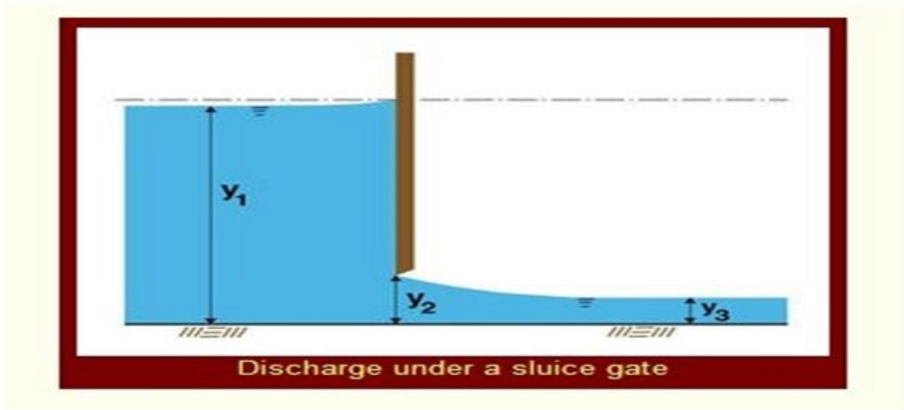

Figure 8. Discharge under a sluice gate (the researcher,2024)

2.3 Structural Analysis and Design of Weir Type _A

2.3.1 Elementary profile of storage weir

The elementary profile of a weir is a triangular section as showing in figure (9)

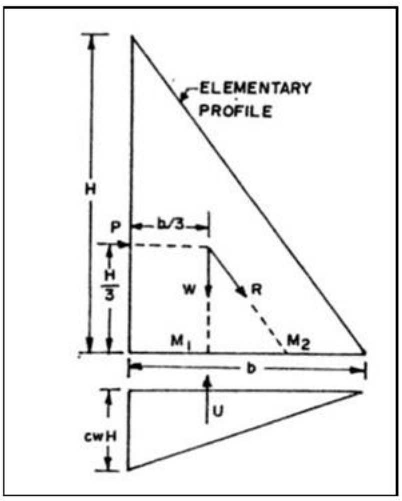

Figure 9. Elementary profile (the researcher,2024)

The shape like hydrostatic pressure diagrams the safe base with b is obtained by considering two criteria;

1. Stress criteria
2. Sliding criteria

Base width b on stress Criterion equation is

b=H/(S-C) ^0.5 Where b= width of weir

S= specific gravity of weir material and

C = uplift coefficient If uplift coefficient, C =0 b=H/S^0.5

So, in this weir b= 2.52 /2.2^0.5= 1.7 m

Base width b on Sliding Criterion

For no sliding, horizontal forces on the weir causing, sliding should be balanced by frictional resistive forces opposing, and in this case b= H/u(S-C) Where

u= coefficient lies from (0.65 to 0.75)

If C=0

Then b=H/u*S

For this weir b= 2.52 /0.65*2.2 = 1.77 m

So minimum width of elementary profile will be equal to 1.77m.

If the bottom width of weir = 1.77 m this mean it will be safe against sliding and material stress but as shown in the following calculations, the dimensions of the weir are much larger than dimensions that we obtained in the calculations of this item.

Since the weir will store the water and be used for tourism and entertainment purposes, the directorate Zakho Tourism suggested that they be sloped steps gradual to give more aesthetics.

And here again, the results of the hydraulic and hydrological analysis intervene to find the steps dimensions (Height and length of steps to achieve nape flow (Shalal). Depth of water over crest of weir(dc) = 1.73 m (from hydraulic model when average discharge = 302.72 m3/sec)

Let step height to be = 0.36 m

To find length of step from equation below

$$\frac{Ld}{h} = \left(\frac{dc}{h}\right)^{1.5} * \left(\sqrt[2]{h}/db\right) * \left(\sqrt{1 + 2 * h/db}\right)$$

$use\ db = dc\ (short\ distance)$

$$\text{So } \frac{Ld}{0.36} = \left(\frac{1.73}{0.36}\right)^{1.5} * \left(\sqrt[2]{0.36}/1.73\right) * \left(\sqrt{1 + 2 * \frac{0.36}{1.73}}\right)$$

$$\frac{Ld}{0.36} = 10.53 * 0.45 * 1.19$$

$$Ld = 5.63 * 0.36 = 2.0\ m$$

$$Cd = 0.615 \left(\frac{1}{1000\,h+1.6}\right)\{\,1 + o.5\left(\frac{h}{h+w}\right)\text{^}2\,\}$$

To find w from equation below

Use Cd = 0.94 (flow is near actual in hydraulic model)

By trial and error

W= 0.2 m

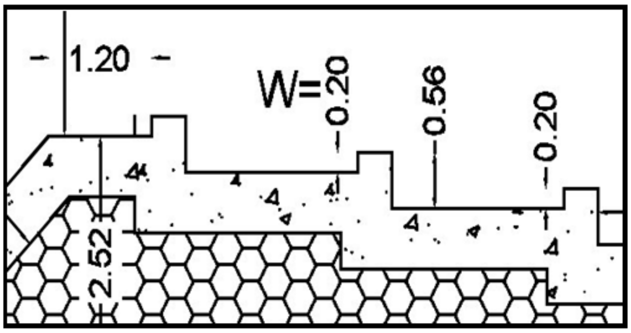

Figure 10. Width b on Sliding Criterion (the researcher,2024)

2.4 Scour depth for upstream cut off

To find scour depth of upstream cut off

$$R = 1.25 * 1.35\ (\sqrt{q/f})$$

q=discharge per meter length of weir = 1450/55 =26.36 where L = 55 m

From table (3) below (gravel bed soil) f = 3.89.

Table 3. Below (gravel bed soil) f = 3.89

S.No.	Type of bed soil	Size of particles mm	Lacey's silt factor f
1.	Coarse Silt	0.04	0.35
2.	Fine Sand	0.08 – 0.15	0.50 – 0.68
3.	Medium Sand	0.30 – 0.50	0.96 – 1.24
4.	Coarse Sand	0.7	1.47
		1.0	1.76
		2.0	2.49
5.	Gravel	5	3.89
		10	5.56
		20	7.88
6.	Boulders	50	12.30
		75	15.20
		90	24.30

$$R = 1.25*1.35\ (\sqrt[3]{26.36^2/3.89})$$

= 9.48 m

Depth of u/s cut off from HFL

Consider bed level of river =100

Max height of water in site of this weir (from hydraulic study) = 4.82 m

Level of bottom of u/s cut off wall (100+4.82)-9.48 = 95.34 m

Provide cutoff wall depth = 100-95.34 = 4.66 m

When there are two cutoff walls each will be with depth = 1.165 m

Use two cutoff walls each with depth = 1.165 m

Use two cutoff walls each with depth =1.65 m

Use also one cutoff wall with depth = 1.65 m for d/s

2.5 Length of d/s floor to resist the downstream from scouring

The length from to the end of d/s stone protection based on hydraulic simulation model to be = 6.5 (normal flow and the hydraulic jump will not happen)

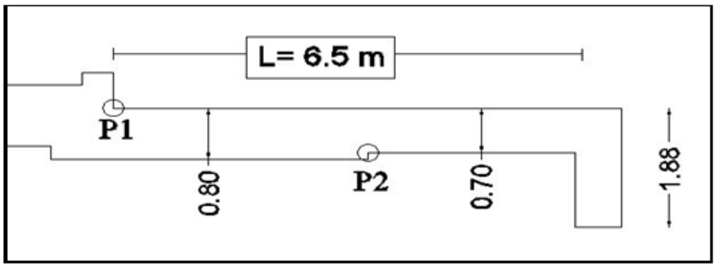

Figure 11. Length on end of weir (the researcher,2024)

2.6 Design of weir slab (floor thickness).

The Apron floor slab is subjected to pressures as a result of water leakage under the weir foundations. These pressures are resisted by the weight of the water column above it, as well as the weight of the concrete bed itself.

We use the following equation to find the thickness of the flooring at any point along the length floor slab after calculating the net uplift pressure.

$$t = 1.33 * \frac{h}{G-1}$$

Head loss up to any point along the floor foundation

$$= \frac{\text{Creep length to that point}}{\text{Total creep length}} * \text{total head}$$

Consider the creep to be equal to the leakage path under weir

= (26.6+ 1.65*6)

= 36.5 m

The critical thickness of floor slab will be at (point P1) end of weir) as shown in drawing no (ZW2_4) when there is now over flow

So, head loss pressures to that a point

$$= \frac{\text{Creep leng} \quad \text{to that point}}{\text{Total creep length}} * 2.52$$

= 1.84 m

And thickness of floor slab at this point   t at p1

= 1.33 *

= 0.69 m

Use t at (P1) = 0.8 m

To calculate the thickness of floor slab at (point P2)

Head loss to that point

$$= \frac{23.97+6.6}{3.7} * 2.52$$

= 2.1 m

And thickness t at P2

$$= 1.33 * \frac{2.52-2.1}{2.1-1}$$

= 0.43 m

Use thickness of slab floor = 0.6 m for all parts of slab floor (except for small part at end of weir = 0.7m), the final profile of the storage weir type _A will be as shown in drawing no (ZW2_4) after selecting upstream slope equal to 1H: 1V.

2.7 Structural Steel for Weir Type _A

Usually a floor slab of weir on rock foundation is reinforced in top face only, but slab on earth is reinforced in both top and bottom faces unless it is thin that reinforced is located in the center, the slabs on earth foundations are reinforced with minimum of 0.2 percent steel each way, and approximately 2/3 in top face and 1/3 in bottom with maximum reinforced of steel bar size 20 mm dia. on 25 centers and additional reinforcement should be provided at points subjected to high hydrostatic up lift pressures. Based on what was mentioned above, the reinforcement of steel bar was distributed as shown in the drawing no (ZW2_5). For example, in slab with a thickness of (60) cm, the reinforced steel was distributed as follows;

Minimum reinforcement

= 0.2/100 *60 *100 = 12 cm2 (per 1m = 100cm width)

For top face of slabs

2/3 * 12 = 8 cm 2   use steel bar dia. = 16 mm = 1.6 cm

Area of one bar size 1.6 cm = 2.0 cm2

Number of bars = 8 / 2 = 4 bar

Spacing = 100 / 4 = 25 cm

Use steel size 16 mm @ 20 cm center to centers

For bottom face of slabs

1/3 * 12 = 4 cm 2   use steel bar dia. = 16 mm = 1.6 cm

Area of one bar size 1.6 cm = 2.0 cm2

Number of bars = 4 / 2 = 2 bar

Spacing = 100 / 2 = 50 cm

Use steel size 16 mm @ 25 cm center to centers

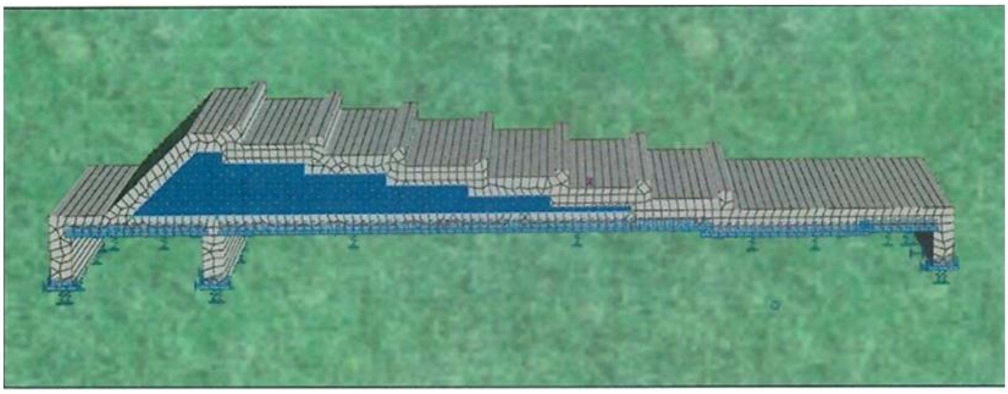

Figure 12. 3D model of the weir (the researcher,2024)

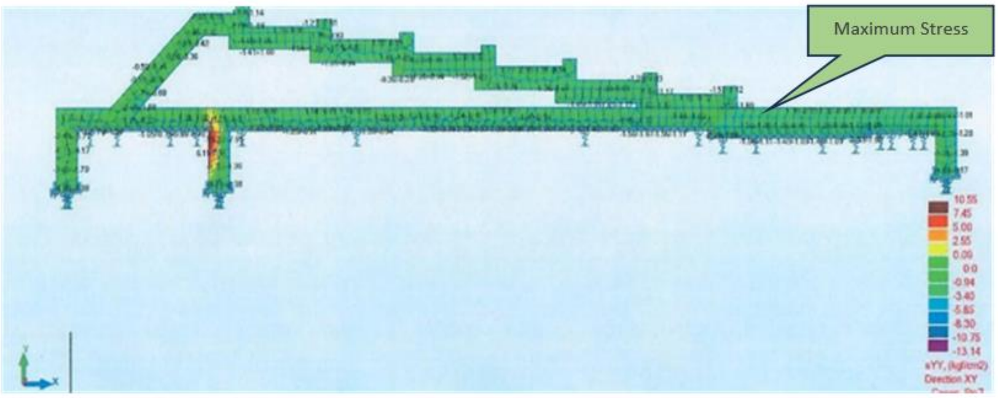

Figure 13. Maximum stress in weir structure due to combination load (the researcher,2024)

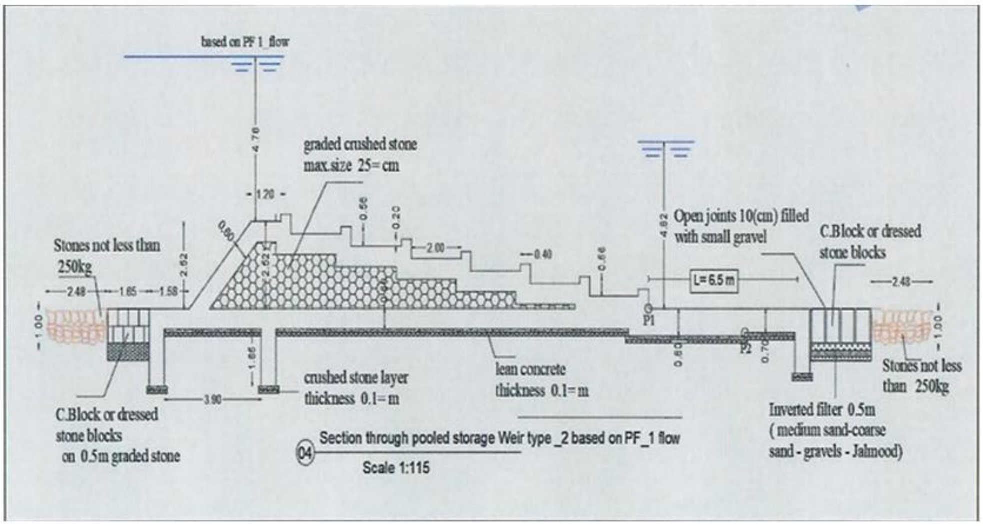

Figure14. Section though line center of the weir type-A (the researcher,2024)

3. Results and Discussion

A weir was designed in front of Southern Baghdad at station 33+44, as shown in Figure (11). The parameters were entered in the inline structure weir window, where Length of Sluice Gate is 7.28 m, width is = 1.77 m and the Froude coefficient is equal to 1. Several scenarios were conducted to achieve the 55 m. Optimum level that does not permit the passage of any flow exceeding m3/sec Use two sluice gate 37.76 m3/sec during the flow below 302.72m3/s during the season. After implementing the many scenarios, the optimum level for designing a weir as the following of:

1. In cause when the water level is at the edge of the weir (crest elevation of the weir), and after assigning the loads and combination to the structure and analyzing it according to the ACI 380 code, the following results obtained:
   a. Maximum Stress in the body of the weir = 6.37 kg/cm2
   b. Max. Stress in the upstream key wall = 5.37 kg/cm2
   c. Max. Stress in the middle key wall = 6.37 kg/cm2
   d. Max. Stress in the upstream Key Wall0.87 kg/cm2
2. In cause where there is a maximum flood and the highest water level reaches (5.54) m above the top of the weir, the following results obtained:
   a. Maximum Stress in the body of the weir =7.97 kg/cm2
   b. Max. Stress in the upstream key wall = 7.44 kg/cm2
   c. Max. Stress in the middle key wall = 7.97 kg/cm2
   d. Max. Stress in the downstream key wall = 1.88 kg/cm2

4. Conclusion

In this study, a one-dimensional HEC-RAS model was used to generate a hypothetical design model for submersible dams in the southeast of Tigris River in Baghdad Governorate. To further generate a map from the discharge data in Khabur River in the model, a geographic information system was used. In August 2024, a calibration and validation of the one-dimensional HEC-RAS model was carried out where Manning's roughness coefficient 'n' was the main calibration parameter. The flow data from Khabur River towards Tigris were used for calibration and validation. A strong correlation was observed between the observed and simulated flow data as a result. For Manning's Froude coefficient 'n' of 0.031 for Tigris River and 1 for Madan weir, the result showed a strong correlation between the observed and simulated flow results.

In this analysis, a one-dimensional HES-RAS model was used to generate a floodplain inundation model for the Tigris River. As it appears from the results above, the largest stress on concrete in both cases (1.88 kg/cm2), so the structure achieves safety under various type of loads and is not more than (7.97 kg/cm2) and it is very little in the downstream key wall combinations according to ACI 318 code. When strength of used concrete is (Class C 25 MPa). Based on the results, the following conclusions can be reached:

1. By combining remote sensing techniques and geographic information systems, it is possible to draw maps of the topography of water depressions to determine the locations of dams.
2. These Weir-type dams can be used in flat areas in all Iraqi governorates because they are less expensive, not exceeding one million dollars, the fastest time period for completion, ranging from 7 to 8 months. They are also used to raise the levels of the liquefaction stations, which leads to reducing water releases and is considered one of the ways to preserve water wealth and Tourist entertainment.
3. This integrated technology has a high and flexible ability to calculate the locations of dams in the main and tributary rivers in a short period of time and at a low cost

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