Journal of Engineering Sciences
Vol 11, Issue 1, JAN/2020
ISSN NO:0377-9254

Attenuating the downstream flooding of Tagwai dam from the main channel using lateral weir

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Abstract: The aim of this research is to design a lateral weir that will attenuate the peak discharge that often results to downstream flooding from the main channel. The designed lateral weir is the most appropriate flood prevention outflow structure that should be installed along the Tagwai dam river channel. The lateral weir can also serve the purpose of irrigation through the sitting of a detention basin on the river bank. In this research, the peak discharge from the main channel for a chosen return period is estimated and a suitable lateral weir is designed that will attenuate the upstream peak flow thereby preventing flood damages downstream. River Flood occurs when high flow that goes downstream exceeds the retention capacity of the channel thereby leading to the spillage of such flow over the banks of the channel. Flooding is known to be hazardous when it happens. It can result in the destruction of farm lands, houses, livestock and even loss of human lives. Therefore, the need for the design of a lateral weir to divert flow laterally in order to prevent downstream flood and minimize erosion cannot be over emphasized.

Keywords: spillage, weir, retention, attenuate

1. Introduction

A Lateral weir (also known as a Side weir) is an overflow diversion structure set into the side of a channel with the purpose of allowing excess flood flows to spill over the weir crest when the flow in the main channel rise above the weir crest. Lateral weirs are typically used in irrigation, land, drainage, urban sewage systems and sanitary engineering. They are also widely used for storm relief, as well as head regulators of distributaries. The model parameters established can then be used to predict the extreme events of large recurrence interval (Pegram and Parak, 2004) Reliable flood frequency estimates are vital for floodplain management; to protect the public, minimize flood related costs to government and private enterprises, for designing and locating hydraulic structures and assessing hazards related to the development of flood plains (Tumbare, 2000). Nevertheless, to determine flood flows at different recurrence intervals for a site or group of sites is a common challenge in hydrology. Although studies have employed several statistical distributions to quantify the likelihood and intensity of floods, none had gained worldwide acceptance and is specific to any country (Law and Tasker, 2003). One of the most widely used methods employed in flood frequency analysis is the Gumbel's method.

1.1 Objectives

i. To estimate the peak discharge from the channel
ii. To reduce flow downstream of the lateral weir
iii. To design a suitable lateral weir that will attenuate the upstream peak discharge and prevent downstream flooding.

2. Study Area

The Tagwai dam situated at longitude 6˚ 39[1]-6˚ 44[1E] and latitude 9˚ 34-9˚ 37˚ N at the confluence of river Tagwai and Jidno south west of Minna. The dam has a vast catchment area, covering about 120km^2. The lake itself has a surface of 440ha and a tremendous capacity of 28.3 x 10^6 mm^2. There is secondary vegetation in the area which consists mainly of stress and open grass land. The climate is married by high temperatures and humidity. It had a maximum temperature of above 37.50^0 C record during the month of March and April and there is a fall in temperature to the minimum of about 20^0 C during the months of December to January.

3. Methodology

3.1 Field Study and Data Collection

The field (Tagwai dam) was visited a couple of times in order to carry out the field work successfully and to establish the need for the design and installation of a lateral outflow structure.Measurements of some flow parameters on the river were taken and

recorded. Parameters such as the channel flow depth and the surface width of flow of the channel. The surface width was measured linearly by two people with one person standing at the edge of opposite river bank holding one end of a measuring tape across a small bridge on the river channel in order to measure its width.The Niger State Water Board (Hydrology Section) was contacted for the collection of the annual maximum flood discharge data for the site under consideration (Tagwai Dam). Annual discharge record of 20 years was collected and generated.

3.2 Estimation of Peak Flow: Flood frequency analysis involves the fitting of a probability model to the sample of annual flood peaks recorded over a period of observation, for a catchment of a given region.

3.3 Gumbel's Method

The extreme value distribution introduced by Gumbel was adopted for estimating peak flow and is commonly known as Gumbel's distribution. Gumbel's equation is given by:

$$x_T = \bar{x} + K\sigma_{n-1} \quad (1)$$

Where

x_T = Value of the variate X with a return period T

$\bar{x}$ = Mean

σ_{n-1} = Standard deviation of the sample of size N

K = frequency factor expressed as

$$K = \frac{yT - yn}{Sn} \quad (2)$$

In which

y_T = reduced variate, a function of T (recurrence interval) and is given by

$$y_T = -[\ln.\ln\frac{T}{T-1}] \quad (3)$$

y_n reduced mean, a function of sample size N given in the Gumbel's extreme value distribution table

S_n = reduced standard deviation, a function of sample size N given in the Gumbel's extreme value distribution table

The discharge data were rearranged in descending order and the plotting position recurrence interval T_p for each discharge is obtained from the formula below

$$T_p = N + 1/m = 28/m \quad (4)$$

Where m = order number

N= Size number

Tp = Plotting position recurrence interval

3.4 Hydraulic Design Procedure

The side weir is then designed to remove the desired amount of flow from the channel when it is flowing at the design flow.

3.4.1 Determination of Flow parameters

Calculate the upstream Froude number

$$\mathbf{Fr} = \frac{v}{\sqrt{gy}} \quad (5)$$

Where

Fr = Froude number (unitless)

v = velocity of flow (m/s)

g = acceleration due to gravity (9.81 m/s)

y = flow depth (m)

3.5 Computation of critical depth in the channel

$$d_c = [Q^2/B^2g]^{1/3} \quad (6)$$

Where,

d_C = critical depth (m)

Q = discharge rate (m^3/s)

g = acceleration due to gravity (9.81 m/s)

B = width of main channel (m)

The weir height is assumed and set below the critical depth.

3.6 Computation of the specific energy

The specific energy relative to the weir crest elevation was computed as (α is taken as 1.0 upstream and 0.95 downstream):

$$E_w = \alpha (V^2/2g) + \alpha (d-c) \quad (7)$$

Determination of the lateral weir length

Computation of c/E_w ratio

The length of the weir is determined from this formula

$$L = 2.03 B (5.28 - 2.63 c/E_w) \quad (8)$$

3.4.5 Check to verify that flow target is satisfied

Solve for the downstream head on the weir and the flow goes through critical depth at upstream end

At critical flow

$$\alpha V_1^2/2g = \alpha h_1/2 \quad (9)$$

The the specific energy is

$$Ew = \alpha h_1/2 + \alpha h_1 \quad (10)$$

$$h_1 = Ew/(1.5 \alpha) \text{ and } h_2 = 0.1h_1 \quad (11)$$

Substitute h_2 into the specific energy equation and V_2 is obtained.

From the continuity equation Q = VA, Q2 is determined and it is compared to the target flow

4. Results and Discussion

4.1 Estimation of peak flow

Table 4.2 Annual maximum discharge records rearranged in descending order

Order number	Flood Discharge (m^3/s)	T_p (years)	$(x-\bar{x})$	$(x-\bar{x})^2$
1	8151	21	2135	4558225
2	7797	10.50	1781	3171961
3	7392	7.00	1376	1893376
4	7173	5.25	1157	1338649
5	7061	4.20	1045	1092025
6	6733	3.50	717	514089
7	6564	3.00	578	300304
8	6384	2.63	368	135424
9	6021	2.33	5	25
10	5919	2.10	-97	9409
11	5910	1.91	-106	11236
12	5679	1.75	-337	113569
13	5612	1.62	-404	163216
14	5602	1.50	-414	171396
15	5496	1.40	-520	270400
16	5145	1.31	-871	758641
17	5053	1.24	-963	927369
18	4280	1.67	-1736	3013696
19	4200	1.11	-1816	3297856
20	4151	1.05	-1865	3478225

Source: (NSWB, 2012)

$\sum(x-\bar{x})^2$= 25219091 σ_{n-1} = 1152 m^3/s $\bar{x}$ = 6016 m^3/s

From the Gumbel's extreme value distribution table (page 245 *Engineering Hydrology* tables 7.3 and 7.4)

For N= 20, y_n= 0.5236 and S_n = 1.0628

$y_T = -[\ln.\ln\frac{20}{19}] = 2.970195$

and K = Frequeny factor

$$K = \frac{2.970195-0.5236}{1.0628} = 2.30202$$

Choosing a return period of 20 years

$\bar{x}_{20}$= 6016 + (2.30202 × 1152) = 8666 m^3/s

4.2 Design of lateral weir

Estimation of peak discharge gave a value of 8666 m^3/s and the desired flow characteristic downstream is 3466 m^3/s (that is about 60% decrease)

4.2.1 Upstream Flow Condition

Flow depth, d = 1.7 m

Width across main channel, B = 82 m

Slope = 0.001

Manning roughness coefficient = 0.035 (for natural channels with vegetations along its banks)

Cross-sectional area of the channel upstream

A = 82 × 1.7 = 139 m^2

Wetted Perimeter (P)

P = 82 + 2(1.7) = 83.4 m

Hydraulic Radius (R)

$R = \frac{139}{83.4} = 1.67$ m

Flow Velocity, $V = \frac{1.49(1.67)\times(0.001)}{0.035}$ =1.9 m/s

Q = VA = 139 × 1.9 = 264 m^3/s

Froude number,

$Fr = \frac{v}{\sqrt{gy}} = \frac{1.9}{\sqrt{9.81\times1.7}} = 0.465 < 1$ (subcritical flow)

4.2.2 Computation of critical depth, d_c

At critical depth Fr = 1 and Q = AV

$= Q / (B\, Dc^{3/2}\, g^{1/2}) = (\sqrt[3]{1.02})^2$

Dc= 1.02 m

The height of weir crest (c) is assumed and set to be 0.5m (below the critical depth)

4.2.3 Computation of Specific Energy

$Ew = \alpha (V^2/2g) + \alpha (d-c)$

$Ew = 1.2 (1.9^2/2\times9.81) + 1.0 (1.7 - 0.5) = 0.22 + 1.2$

Ew = 1.42 m

4.2.4 Lateral Weir Length is calculated thus;

$L = 2.03B (5.28 - 2.63\, c/E_w)$

And c = 0.5 m

$c/E_w = \frac{0.5}{1.42} = 0.352$ (this indicates a falling water surface profile)

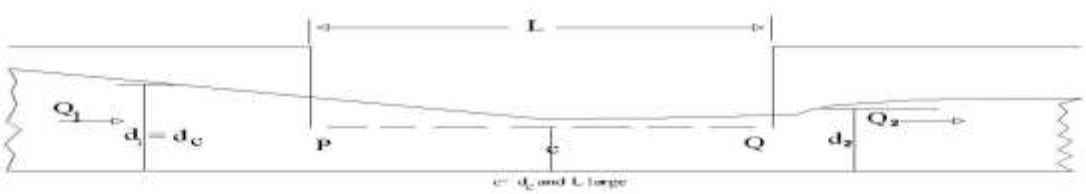

Figure 1: Falling Water Surface Profile

$L = 2.03 (82) \times [5.28 - 2.63 (0.352)]$

$= 166.5 \times [5.28-0.926]$

L = 722.8 m

4.2.5 Check length for flow target

Downstream head on the weir

$h_2 = 0.1\, h_1$

$h_1 = Ew/1.5\, \alpha$

$h_2 = 0.1\, Ew/1.5\, \alpha = \frac{[(0.1)(1.42]}{[1.5(1)]}$

h_2 = 0.095 m

The value of h_2 is substituted into the specific energy equation to derive V_2 (α as 1.4 and α' as 0.95)

$Ew = \alpha (V^2/2g) + \alpha (h_2)$

$1.4 \{V^2/ 2 \times 9.81\} + 0.95 (0.045) = 1.42$

$V_2 = 2.23$ m/s

$y_2 = h_2 + c = 0.095 + 0.5 = 0.595$

$Q_2 = AV = (82)(0.595) \times 2.23$

$Q_2 = 108.8$ m^3/s

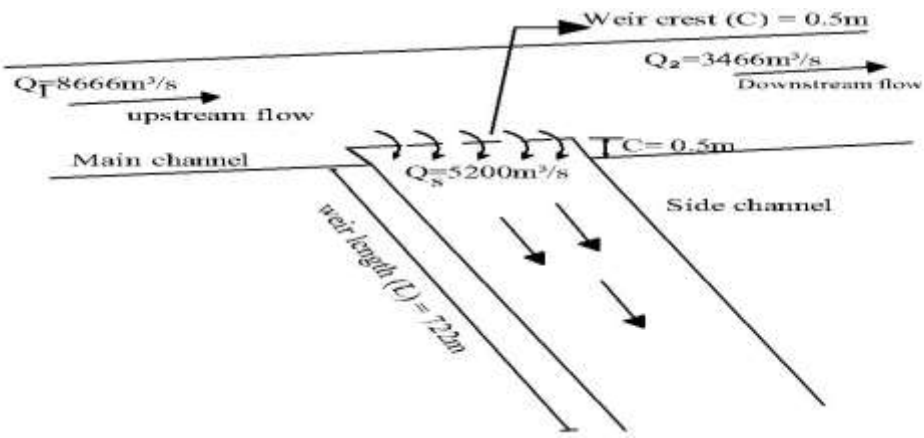

Figure 2: Top View of the Designed Lateral Weir

5. Discussion of Results

The peak discharge was estimated using Gumbel's method which proves to be a reliable flood frequency analysis tool. Peak discharge estimated gave 8666m^3/s. The weir crest was set at 0.5m below the critical depth and it resulted in a falling water profile. At peak flow, the computed lateral weir length is capable of discharging about 60% (5200 m^3/s) of the total estimated peak flow and the remaining 40% (3466 m^3/s) flows downstream which is safe.

6. Conclusion

It can conclusively be said that the designed lateral weir is the most appropriate flood prevention outflow structure that should be installed along the Tagwai dam river channel. The lateral weir can also serve the purpose of irrigation through the sitting of a detention basin on the river bank.

7. Recommendations

It is recommended that the appropriate authorities should finance the installation of a lateral weir in the side of the river as it will go a long way in preventing the occurrence of flooding in the nearest future.

It is recommended that further researches should be carried out on the other types of lateral weirs using this research work as a base line for further findings since it is not a common feature in this part of the divide.

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