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Hydrological Analysis and Peak Discharge Determination

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Abstract: *Peak discharge Determination is one of the most important studies for Irrigation projects. The proposed of hydrologic design is to estimate maximum, average or minimum flood which the structure is expected to handle .This estimate has to be made quite accurately in order that the project can function properly. To estimate the magnitude of a flood peak the following alternative methods are available academically. In this study Hydrological analysis has been conducted based on 10years maximum daily rainfall data .The frequency analysis has been carried out by different methods and Log Pearson type III method is adopted .The peak discharge computed by United States Soil Conservation Service (USSCS) method is 1278m^3/sec .Since this method over estimates the design flood, we adopt the peak discharge calculated by slope -area method.*

Key Words: *Hydrological Analysis, peak discharge Determination, Gusha Temela Irrigation project, frequency Analysis, United States of soil conservation service method.*

I. Introduction

1.1 General

Ethiopia is situated in the horn of Africa, and is bordered by Sudan, Kenya, Somalia, Djibouti and Eritrea. The surface area is more than one million square kilometers and the country stretches from latitude 3° North to latitude 15° North of the equator and from 33° East to 48° East longitudes (MoWR, 2004). It has a large population of approximately 77.1 million people with an annual growth rate of 2.4% (FAO, 2008). The country has nine regional governments, Tigray, Afar, Amhara, Oromia, Somalia, Benshangul-Gumuz, Southern Nations Nationalities and Peoples, Gambella, Harari and two city states Addis Ababa and Dire Dawa. Ethiopia belongs to one of the poorest African countries, with 52% of the population living below the national poverty line (MoWR, 2004) and 31.3% of the population living below US$1 a day (World Bank in Teshome, 2003 p.24). Irrigation development plays an essential role in stabilizing crop production by either supplementing or replacing the need for natural precipitation. Irrigation makes agriculture more confidential. It stabilizes crop production by protecting against drought and by increasing crop yields, increases their income and crops that improve their diet Ebissa G. K. (2017). Despite the huge potential of the area, existing traditional farming practice is not in harmony with the needs and requirements of developing a productive and sustainable agriculture in Ethiopia. The food security situation has continued to deteriorate because of various factors including shortage of rain fall, high population growth, deforestation, soil degradation, pest out break and other related factors are threatening food security situation of the area. Although the initiation of farmer's traditional surface irrigation practice is appreciated, it is not in a position to provide sustainable supply source and effective utilization of water. Therefore, the development of Nanno Small Scale Irrigation Project diversion irrigation is expected to contribute towards alleviating these problems thereby increasing food supply and income source to the community and also at local and regional levels (Ebissa G. K. et al 2017). Fortunately, Ethiopia is lucky in that it has got ample source of surface and subsurface water for which it is known as "The Water Tower of East Africa." Moreover; the irrigation potential is estimated to be about 4.25 million hectare of which only 5.8% is irrigated.(source: Study carried out by International Water Management Institute-IWMI). Nowadays, implementation of small and medium scale irrigation schemes is being given priority in the water sector development strategy of Ethiopia.

1.2 Back Ground

Gusha Temela small-scale irrigation project is located in Arsi zone, in Digalo- Tijo district, which comprises 23 peasant associations (PA's) and three towns. The project area is especially located in the three adjacent PA's. Namely Gusha Temela, Chefe Gogessa and Ashebeka Walkitiel. The project area is accessible at 7km from Sagure town towards South direction on the main gravel road of Assela-Bale Robe towns. Based on the information from the report by OIDA total area of the district is about 1349.26km^2 of which 31.3% is arable land. The soil type in the district: clay soil 35%, loam soil 21% and red soil 44%.The midland and highland (about 78%) are the two major climate zones. So the project area is climatically categorized under highland. The altitude ranges 2000-3600m. a.m.s.l and temperature varies with in 15 ^{o}c to 20 ^{o}c while annual rainfall ranges from 900-1400mm.

As indicated on the report the area for PA's were about 5567ha. And it could be understood that out of this 82.77% was cultivated under different annual crops,1.82% was used as grazing area and the rest is utilized for different purposes The farmers practiced mixed type of farming. They are cultivating wheat, barely, bean, pea, and lin-seed (Telba). The beneficiary households are about 350 with total population of 2087 with average family size of six people per household.

The predominant crops grown in the project area; wheat, barely and lin-seed under rain-fed crop production. By traditional irrigation system 82 house holds were cultivating about 33.15 ha of land by diverting Temala River which is the potential river used for irrigation and the major crops grown by traditional Irrigation system are: potato, onion, cabbage and pepper. The following major constraints and problems were identified with regard to crop production under irrigated agriculture.

- Low input use due to its higher price or weak economic condition of farmers to Purchase input at prevailing market price.
- The use of traditional way of farming tools style under existing farming Practices.
- No modern diversion work.
- Poor supply canals with high seepage loss.
- In appropriate water application to the crops

1.3 Objectives

The main objective of this study is to upgrade the existing traditional scheme in the area so that food shortage problem will be alleviated. The project will lead farmers to increase the income of the people. It helps the community to utilize the available resources more wisely and to avoid conflict among users. Therefore, the following points are the aim of the study:

- To Analyze Hydrological data,
- To determine peak discharge,
- To calculate the peak flood,
- To Calculate lean Flow …etc.

1.4 Methodologies

In this study Hydrological analysis has been conducted based on 10years maximum daily rainfall data .The frequency analysis has been carried out by different methods. The peak discharge is computed by United States Soil Conservation Service (USSCS) method. Since this method over we estimate the design flood, the peak discharge by slope -area method. The methods followed in carrying out the above objectives are:

- The frequency analysis has been carried out by different methods.
- The peak discharge is computed by United States Soil Conservation Service (USSCS) method
- Analysis of secondary data such as rain fall, hydrological,… etc.
- Use of meteorological data to calculate the peak flood
- Referring of different literature reviews and guidelines related to the study.
- Use of contour map of the command area to read the weir length.
- Use of topography map (1:50,000 scale) to determine the catchment characteristics

II. Results and Discussions

2.1 General

Hydrologic design is important for safety, economy and proper functioning of hydraulic structures. The proposed of hydrologic design is to estimate maximum, average or minimum flood which the structure is expected to handle .This estimate has to be made quite accurately in order that the project can function properly.

Data availability

All water resources system must be planned for future event for which the exact time of occurrence can be forecasted. In order to forecast the hydrological events, data is necessary. The available data is daily maximum rain fall of 10 years and mean monthly flow of the river Gusha Temela.

2.2 Calculation of peak discharge

To estimate the magnitude of a flood peak the following alternative methods are available academically.

1 Rational method
2 Empirical method
3 Slope area method
4 Flood frequency analysis or USSCS (United State Soil Conservation Service) method.
5 Stream flow measurement method

The use of particular method depends upon:

- The desired objective
- The available data
- The importance of the project
- The size of the catchments

2.2.1 Rational Method

The rational formula is found to be suitable for peak flow prediction in small catchments up to $50Km^2$ in area. It is applicable in urban drainage design and in the design of small culverts and bridges. The equation of rational method is given by

$Q_P = \frac{1}{3.6}$ C. $I_{tc,\,p}$ .A (1)

Where Q_P = Peak discharge (m^3/sec)

C = Coefficient of run off

A = Drainage area in Km^2

$I_{tc,\,p}$ = the mean intensity of precipitation (mm/hr.) for duration equal and an exceedance probability p.

$I_{tc,\,p}$ =R50/Tc =315.5/2.114=149.2mm/hr.

$\Rightarrow$ Qp= $\frac{1}{3.6}$ *0.36*149.2*150=2238m^3/sec

- The use of this method to compute Q_P requires three parameters ; t_c, ($I_{tc,p}$) and C.

Rational method is not convenient for he determination of peak flood for river like Temela.

- Estimation of $I_{tc,\,p}$ requires some other regional constants based on catchment size difference.

2.2.2 Empirical Formula

The empirical formula used for the estimation of peak flood are essentially regional formula based on statistical correlation of the observed peak and important catchment properties. To simplify the form of the equation, only a few of the many parameters affecting the flood peak are used. For example almost all formulae use the catchment area as a parameter affecting the flood peak and most of them neglect the flood frequency as a parameter. In view of these the empirical formula are applicable only in the region from they were developed .

E.g. Dr Admassu's empirical formula is one of the applicable for some parts of Ethiopia

Qp =Q (1+Kz.Cv) (2) …….General formula

Q= $0.87A^{0.7}$

Where

A = catchments area (Km^2)

Kz = Frequency factor

= $\frac{-\sqrt{6}}{\pi}$ [0.57721+ln (ln ($\frac{T}{T-1}$))]

T = Return period

Cv = The average coefficient of variation.

= 0.38 for most catchment.

Qp=Q (1+Kz.Cv)

=57.63m^3/sec

The formula is safely adopted for Blue Nile basin under the given area range . However ,the water shed of Temela river is out of domain and hence we didn't adopt the result for this method.

2.2.3 Slope Area Method

According to the given parameters

Cross sectional area (A) =98m^2

Wetted perimeter (P) =36m

Hydraulic radius (R) =2.7

Water surface slope(S) =0.004

Manning roughness (n) = 0.04

From manning equation

$V= \frac{1}{n} * R^{2/3} * S^{1/2}$ (3)

= 3m/sec

Q =V*A

=290m^3/sec (Source OIDA report)

2.2.4 Frequency Analysis (United State Soil Conservation Service) Method

Testing For Out Lairs

Out lairs are data point that depart significantly from the trend of the remaining data . The depletion of high and low out lair can one sided 10% significantly affect the magnitude of statistical parameter computed data . Specially for small number data to find lower and higher out lairs. The following equation is used.

Lower out lairs ($XL_)$ = $\overline{X}$ -Kn * δ_X (3)

$\Rightarrow Q_L = 10^{XL}$

Higher out lairs (XL) = $\overline{X}$ + Kn * δ_X (4)

$\Rightarrow Q_H = 10^{XH}$

Where $\overline{X}$ = Mean value of logarithm transferred data.

Kn = factor taken from table which contains one side 10% significance

δ_X =Standard deviation of logarithmic transferred data.

Q_L =The lower discharge

Q_H =The higher discharge

Kn =2.036 (V.T.Chow, 1988)

$\overline{X}$ =2.30126, δx = 0.0604

$\Leftrightarrow X_L = \overline{X}$ - Kn *δ_X

= 2.30126 – 2.036*0.0604

= 2.1785

$\Rightarrow Q_L = 10^{XL}$

=150.83mm (Lower margin)

$\Leftrightarrow X_L = \overline{X}$ + Kn * δ_X

= 2.30126 + 2.036*0.0604

= 2.4244

$\Rightarrow Q_L = 10^{XL}$

=265.705 mm (upper margin)

Therefore , the data that is available from the OIDA report checked for further analysis between the lower and the higher value obtained from the calculation.

Rain fall frequency analysis

Some of the commonly used frequency distribution function for the prediction of extreme maximum rain fall values are:

1. Normal distribution method.
2. Gumbel distribution method.
3. Log Pearson type III distribution method.
4. Log normal distribution method.
5. Pearson type distribution method.

Taking return period of 50 years (Dr. Mr. Suberamanya,1989) the design rain fall is determined as follows.

2.2.4.1 Normal Distribution Method

$X_T = \overline{X} + K_T * \delta_{n-1}$ (5)

Where X_T = annual maximum rain fall of T year return period

$\overline{X}$ = mean of annual maximum daily rain fall

K_T = frequency factor expressed as

$$K_T = W - \left(\frac{2.51557 + 0.80285W + 0.01033W^2}{1 + 1.143279W + 0.1992W^2 + 0.00135W^3}\right)$$

$\Rightarrow$ $W = (\ln(1/\rho_r)^2)^{1/2}$, $\rho_r = \frac{1}{T}$

$\Rightarrow$ $\rho_r = \frac{1}{50} = 0.02$

$W = \ln(1/0.02)^2)^{1/2}$

=2.797

K_T =2.06276=2.063

$X_{50} = \overline{X} + 2.063 * \delta_{n-1}$

=202.16+2.063*29.0759 = 262.143 mm

2.2.4.2 Gumbel Distribution Method

It is one of the widely used probability distribution function for extreme values in hydrologic and metrological studies for prediction of flood peaks , maximum rain falls , wind speed etc. and expressed by equation .

$RT = \overline{R} + K\sigma R - 1$ (6)

Where RT = annual maximum rain fall of T years return period

$\overline{R}$ = mean of the annul maximum daily rain fall .

K = frequency factor and expressed as

$$K = \frac{Y_T - \overline{Yn}}{Sn} \quad (7)$$

$\sigma R - 1$ = standard deviation of the sample size

$$N = \sqrt{\frac{\sum\left(R - \overline{R}\right)}{N-1}} \quad (8)$$

Let Y_T be a reduced variety , a function of T and is given by

$$Y_T = -\ln\left(\ln\left(\frac{T}{T-1}\right)\right) \quad (9)$$

Where $\overline{Yn}$ =reduced mean , it is a function of sample size

Sn = reduced standard deviation which is also a function of the sample size Yn and Sn are obtained from table .

This equation are used under the following procedure to estimate the rain fall magnitude corresponding to a given return period based on the maximum daily rain fall series.

Assemble the maximum daily rain fall data and note the sample size N .

Here the rain fall data is the variete $\overline{R}$. Find $\overline{R}$ and $\sigma R - 1$ for the given data .

- Using table determine Yn and Sn appropriate to given N
- Find Y_T for a given T by equation (9)
- Find K by equation (7)
- Determine the required R_T by equation (6)

Table 1 Determination of design storm arranging the maximum values in descending order .

S.No	R (mm) in descending order	$\overline{R}$ (mm)	R - $\overline{R}$ (mm)	(R - $\overline{R}$)^2
1	249	202.16	46.86	2193.986
2	228.7		26.54	704.3716
3	225		22.84	521.6656
4	213.2		11.04	121.8816
5	206.7		4.54	20.6116
6	202		-0.16	0.0256
7	200		-2.16	4.6656
8	170.6		-31.56	996.0336
9	165.4		-36.76	1351.1976
10	161		-41.16	1694.14
$\sum$	2021.6	$\sum$		7608.579

$$\overline{R} = \frac{\sum_{i=1}^{n} R_i}{n} = \frac{2021.6}{10} = 202.16$$

$$\sigma_{R-1} = \sqrt{\frac{\sum_{i=1}^{n}\left(R_i - \overline{R}\right)^2}{N-1}} = \sqrt{\frac{7608.579}{9}} = 29.075$$

Now, $KT = \frac{Y_T - Yn}{Sn}$, where $YT = -\ln(\ln(\frac{T}{T-1}))$, For T =50

Y_T = -ln (ln (50/49))
Y_T =3.952
Yn =0.49 (Refer table 3.3 in the annex 3)
Sn =0.9496 (Refer table 3.2 in the annex 3)

$\Rightarrow$ KT =3.587

So $R_T = \overline{R} + K_T \sigma_{R-1}$
= 202.16 + 3.587*29.075
= 315.55 mm

2.2.4.3 Log Person Type III Distribution Method.

In this method the verity is first transformed into logarithmic form (base 10) and the transformed data is then analyzed . If R is the verity of random hydrologic series then the series of Z varieties where
Z = log R (10)
are obtained, for this Z series for any recurrence interval T.

$Z_T = \overline{Z} + K_Z \sigma_R$ (11)
Where KZ = a frequency factor which is a function of T and the coefficient of skew ness ,C_S .
σ_R = standard deviation of the Z verity sample

$$= \sqrt{\frac{\sum\left(Z - \overline{Z}\right)^2}{N-1}}$$

$$C_S = \frac{N\sum\left(Z - \overline{Z}\right)^3}{(N-1)(N-2)(\sigma_Z)^3}$$

Where
$\overline{Z}$ =mean of the value
N = sample size =number of years of record
The variation of K2 =f(C_S, T) is given in table 3.4 in the annex 3 .

First find ZT by equation (11)
Then find R_T by equation (10)

Table 2 Calculation of design storm by log Pearson typeIII.

S.No.	Max. daily rain (mm)	Z = logR	$\overline{Z}$	Z- $\overline{Z}$	(Z- $\overline{Z}$)^3
1	249	2.396	2.3015	0.095	$8.36*10^{-4}$
2	228.7	2.359		0.058	$1.90*10^{-4}$
3	225	2.352		0.050	$1.26*10^{-4}$
4	213.2	2.329		0.026	$1.92*10^{-5}$
5	206.7	2.315		0.013	$2.37*10^{-6}$
6	202	2.305		0.003	$3.76*10^{-8}$
7	200	2.301		-0.001	$-9.13*10^{-10}$
8	170.6	2.232		-0.070	$-3.40*10^{-4}$
9	165.4	2.219		-0.083	$-5.72*10^{-4}$
10	161	2.207		-0.0947	$-8.49*10^{-4}$
$\sum$	2021.6	23.02		-0.0045	-0.021

$$\overline{Z} = \sum_{i=1}^{n} Z = 2.3015 \quad \sigma_R = \frac{\sqrt{\sum\left(z-\overline{z}\right)^2}}{\sqrt{(N-1)}} = 0.0604$$

$$C_S = \frac{N\sum\left(Z-\overline{Z}\right)^3}{(N-1)(N-2)(\sigma_Z)^3} = \frac{10*(-0.021)^3}{9*8*(0.0604)^3} = -0.005837$$

For T_{50} and C_S = 0.005837 from table , K_Z =2.05084

$Z_{50} = \overline{Z} + K_Z * \sigma_R$

=2.3015+2.05084*0.0604 =2.4253

$\Rightarrow X_T = 10^{2.4253}$ = 266.19mm

2.2.4.4 Log Normal Distribution

Log normal distribution is a special type of Pearson type III distribution with C_S= 0 i.e. from table 3.4 in the annex 3.

For C_S =0 and T_{50};K_Z =2.054

$\Rightarrow Z_{50} = \overline{Z} + K_Z * \sigma_R$ (12)

=2.3015 + 2.054 *0.0604 = 2.425

$\overline{X} = 10^{2.42556}$ =266.4mm

2.2.4.5 Pearson Type III Distribution Method

$X_T = \overline{X} + K_T * \sigma_{n-1}$ (13)

=202.16+2.05084*29.0759

=261.79 $\approx$ 262mm

Testing the goodness of fitness of probability

The D-index for the computation of first fit of various distribution is given by :

$$\text{D-index} = \frac{1}{\overline{X}} \sum_{i=1}^{5} \left| Xi - X_C \right| \quad (15)$$

Where Xi and Xc are the I[th] highest observed and computed values for the distribution. The distribution giving the least D-index is considered to be the best distribution

Table 3 Statistical Parameter

	Original series	Log transformed
Mean	202.16	2.31
Standard deviation	29.075	1.4635

Table 4 Normal Distribution of Rainfall Data

Rank	X_i	P=m/(n+1)	Kt	X_C	X_i-Xc
1	249	1/11	1.34	287.96	38.96
2	228.7	2/11	0.91	255.158	26.458
3	225	3/11	0.6	242.445	17.445
4	213.2	4/11	0.35	223.376	10.176
5	206.7	5/11	0.11	209.898	3.198
$\sum$					96.237

$X_C = X_i + K_T \sigma n - 1$

$$\text{D – index} = \frac{\sum_{i=1}^{5}\left|Xi - Xc\right|}{\overline{X}} = \frac{96.237}{202.16} = 0.476$$

Table 5 Log Normal Distribution of Rainfall Data

Rank	X_i	P=m/(n+1)	Kt	Xc	Xi-Xc
1	2.396	1/11	1.34	4.357	1.961
2	2.359	2/11	0.91	3.69	1.335
3	2.352	3/11	0.6	3.23	0.878
4	2.328	4/11	0.35	2.84	0.512
5	2.315	5/11	0.11	2.475	0.16
$\sum$					4.842

$X_C = X_i + K_T \sigma n - 1$

$$\text{D – index} = \frac{\sum_{i=1}^{5}\left|Xi - Xc\right|}{\overline{X}} = \frac{4.842}{2.31} = 2.096$$

Table 6 Pearson Type III of Rainfall Data

Rank	Xi	P=m/(n+1)	Kt	Xc	Xi-Xc
1	249	1/11	1.328	287.61	38.61
2	228.7	2/11	0.805	252.12	23.42
3	225	3/11	0.495	239.39	14.39
4	213.2	4/11	0.252	220.52	7.32
5	206.7	5/11	0.0891	209.29	2.59
$\sum$					86.32

$X_C = X_i + K_T \sigma n - 1$

$$\text{D – index} = \frac{\sum_{i=1}^{5}\left|Xi - Xc\right|}{\overline{X}} = \frac{86.32}{202.16} = 0.426$$

Table 7 Log person Type III of Rainfall Data

Rank	Xi	P=m/(n+1)	Kt	Xc	Xi-Xc
1	2.396	1/11	1.328	4.34	1.944
2	2.359	2/11	0.805	3.54	1.181
3	2.352	3/11	0.495	3.076	0.724
4	2.328	4/11	0.252	2.696	0.368
5	2.315	5/11	0.0891	2.445	0.13
$\sum$					4.347

$X_C = X_i + K_T \sigma n - 1$

D – index= $\dfrac{\sum_{i=1}^{5} |Xi - Xc|}{\overline{X}} = \dfrac{4.347}{2.31} = 1.8818$

Table 8 Gamble Distribution Method

Rank	Xi	P=m/(n+1)	Kt	Xc	IXi-Xc I
1	2.396	1/11	1.34	4.357	1.961
2	2.359	2/11	0.91	3.69	1.335
3	2.352	3/11	0.60	3.23	0.878
4	2.328	4/11	0.35	2.84	0.512
5	2.315	5/11	0.11	2.475	0.160
$\sum$					131.79

$X_C = X_i + K_T \sigma n - 1$

D – index= $\dfrac{\sum_{i=1}^{5} |Xi - Xc|}{\overline{X}} = \dfrac{131.79}{202.16} = 0.6519$

D-index is minimum incase of Pearson type III distribution and hence on the basis of D-index test it can be assumed that Pearson type III fits the data well.
Therefore, the design rainfall is 261.8mm.

Table 9 Peak Runoff calculations by United States Soil Conservation Service method

Step	Formula	Symbol	unit	Result
1	Area of catchment (this can be determined from 1:50,000 scale topographical maps or aerial photographs)	A	Km*2	150
2	Length of main water course from watershed divide to proposed diversion or storage site (Topographic maps)	L	m	
3	Elevation of water shed divide opposite to the head of the main water course(Topographic maps)	H1	M	
4	Elevation of stream bed at proposed or storage site (Topographic maps)	H2	m	
5	Slope of main water course S= $\dfrac{(H1-H2)}{L}$	S	m/m	
6	Time of concentration Tc= $\dfrac{1}{3000} * \left(\dfrac{L}{\sqrt{S}}\right)*0.77$	Tc	hr	2.114
7	Rain fall excess duration D $\approx \dfrac{Tc}{6}$, if Tc<3 hrs. D $\approx 1hr$, if Tc>3 hrs.	D	hr	0.3523
8	Time to peak t_p=0.5D+0.6Tc	tp	Hr	1.44
9	Time base of hydrograph Tb=2.67t_p	Tb	hr	3.85
10	Time lag Te=0.6tc	Te	hr	1.2684
11	Peak rate of discharge created by 1mm runoff excess on whole of the catchment Qp= $\dfrac{(0.21A)}{tp}$	Qp	M*3/S./m m	21.875

12	13	14	15	16	17	18	19	
duration	**daily point rainfall of return period 50 year**	**rainfall profile**	**rainfall profile**	**areal to point rainfall**	**areal rainfall**	**Increme-ntal rainfall**	**descending order**	
hr	mm	%	mm	%	mm	mm	No.	
0-0.35	261.8	26	68.07	57	38.8	38.8	1	38.80
0.35-0.7		33	86.39	57.33	49.53	10.73	2	26.04
0.7-1.05		46	120.43	62.75	75.57	26.04	3	12.13
1.05-1.4		50	130.9	67	87.7	12.13	4	11.91
1.4-1.75		55	143.99	67	96.47	8.77	5	10.73
1.75-2.1		59	154.46	70.17	108.38	11.91	6	8.77

12	Fill in 0-D hr, D-2D hr,……,5D-6D.
13	Determine the magnitude of the daily rainfall with the given recurrent interval by applying statistical method
14	Read from Table 6 (IVA-1/B.1) the rainfall profile(%) occurring in D,2D,3D,….6Dhr.and enter in 14
15	Multiply 13and 14 to find the rainfall profile(mm) and enter in 15.
16	Read from Table (IVA-1/B.1) aerial to point rainfall ratio for different duration and particular catchment area. (method based on research conducted in India and influenced by return period, magnitude of storm shape and orientation of area etc.
17	Multiply 15 and 16 and fill in 17.
18	Calculate incremental rainfall by deducting the current areal rainfall from the preceding areal rainfall as listed in 18.
19	Assign order to the rainfall depths in descending order 1 and 6

20	21	22	23	24	25
Rearranged order	**Rearranged incremental rainfall**	**Cumulative rainfall**	**Time of incremental hydrograph**		
			Time of beginning	**Time to peak**	**Time to end**
NO	mm	mm	hr	hr	hr
6	8.77	8.77	0	1.44	3.85
4	11.91	20.68	.35	1.79	4.21
3	12.13	32.81	.7	2.14	4.56
1	38.8	71.61	1.05	2.49	4.91
2	26.04	97.65	1.4	2.84	5.26
5	10.73	108.38	1.75	3.19	5.6

20	From 19 mention the rearranged order as 6,4,3,1,2,5 (arbitrary but considering ascending and descending feature of the hydrograph ordinates, where peak value is around the middle of the hydrograph.
21	Fill in the corresponding incremental rainfall value to the rearranged order of 20 from 18.
22	Fill in the cumulative rainfall values of 21 by adding with the rainfall values in the preceding duration.
23	Fill in the time of beginning of hydrograph as 0,D,2D,….5Dhr.
24	Fill the time to peak as tp, D+tp, 2D+tp,….5D+tp or add tp in every value of 23 and mention in 24.
25	Add tp in every value of 23 and fill in 25

NO	**Formula**	**symbol**	**unit**	**result**
32	Find maximum potential difference between rainfall(p) and direct runoff(Q),which is given by the following formula: S= $\frac{25400}{CN}$ -254 CN=value corresponding to AMC III as found in 31	S	mm	27.28
33	Substitute the value of "S" in the following formula, giving the relation between direct runoff(Q) and rainfall(P) Q= $\frac{(P-0.2S)2}{(P+0.8S)}$	Q	mm	Q= $\frac{(P-5.56)}{(P+21.82)}$
34	Substitute the value of p1 as mentioned in 22,in the above formula and find the corresponding values of Q(34) Enter the value of Q in 36.	22		34
		mm		mm
		8.77		0.337
		20.68		5.379
		32.81		13.592
		71.61		46.694
		97.65		70.985
		108.38		81.197

35	36	37	38	39		
duration	**Cumulative runoff**	**Incremental Runoff)f**	**Peak runoff for increment**	**Time of begin-ing**	**Time to peak**	**Time to end**
hr	mm	mm	M^3/S	hr	hr	hr
0-0.35	.337	0.337	7.395	0	1.44	3.85
0.35-0.7	5.379	5.042	89	.35	1.79	4.21
0.7-1.05	13.592	8.213	121.27	.7	2.14	4.56
1.4-1.75	46.694	33.102	420.1	1.05	2.49	4.91
1.75-2.1	70.985	64.291	715.35	1.4	2.84	5.26
2.1-2.45	81.197	10.212	101.15	1.75	3.19	5.6
35	Enter the same time as in 12 i.e. 0-D, D-2D, 2D-3D, ……5D-6D					
36	There are the values of Q as found out in 34 corresponding to the value of P.					
37	Find incremental runoff by reducing the value of 36 by preceding values.					
38	Multiply 37 with peak rate of runoff corresponding to 1mm run off excess as found at 11.					
39	Plot triangular hydrograph with time of beginning, peak time and, time to end as mentioned in 23, 24, 25 and peak runoff as mentioned in 38.					
40	Plot a composite hydrograph (Table IVA-1/B.2and 3) by adding all the triangular hydrographs. The resultant hydrograph will be composite hydrograph of desired return period. The coordinate of the peak of tise hydrograph will give the peak runoff with desired return period.					

Table 10 Composite Hydrograph

Time	**Q_1**	**Q_2**	**Q_3**	**Q_4**	**Q_5**	**Q_6**	**Q_{Total}**
0	**0**						**0**
0.35	**1.797**	**0**					**1.797**
0.7	**3.595**	**21.632**	**0**				**25.227**
1.05	**5.392333**	**43.264**	**29.475**	**0**			**78.13133**
1.4	**7.189833**	**64.896**	**58.95**	**102.475**	**0**		**233.5108**
1.75	**6.44**	**86.528**	**88.425**	**204.215**	**173.869**	**0**	**559.477**
2.1	**5.369**	**77.6**	**117.9**	**305.955**	**347.73**	**24.585**	**879.139**
2.45	**4.298**	**64.727**	**105.735**	**407.695**	**521.591**	**49.17**	**1153.216**
2.8	**3.227**	**51.854**	**88.186**	**366.285**	**695.452**	**73.755**	**1278.759**
3.15	**2.156**	**38.981**	**70.637**	**305.527**	**623.714**	**98.34**	**1139.355**
3.5	**1.085**	**26.108**	**53.088**	**244.769**	**520.254**	**88.193**	**933.497**
3.85	**0**	**13.235**	**35.539**	**184.011**	**416.794**	**73.5636**	**723.1426**
4.2		**0**	**17.99**	**123.253**	**313.334**	**58.9342**	**513.5112**
4.55			**0**	**62.495**	**209.874**	**44.3048**	**316.6738**
4.9				**0**	**106.414**	**29.6754**	**136.0894**
					0	**15.046**	**15.046**
						0	**0**

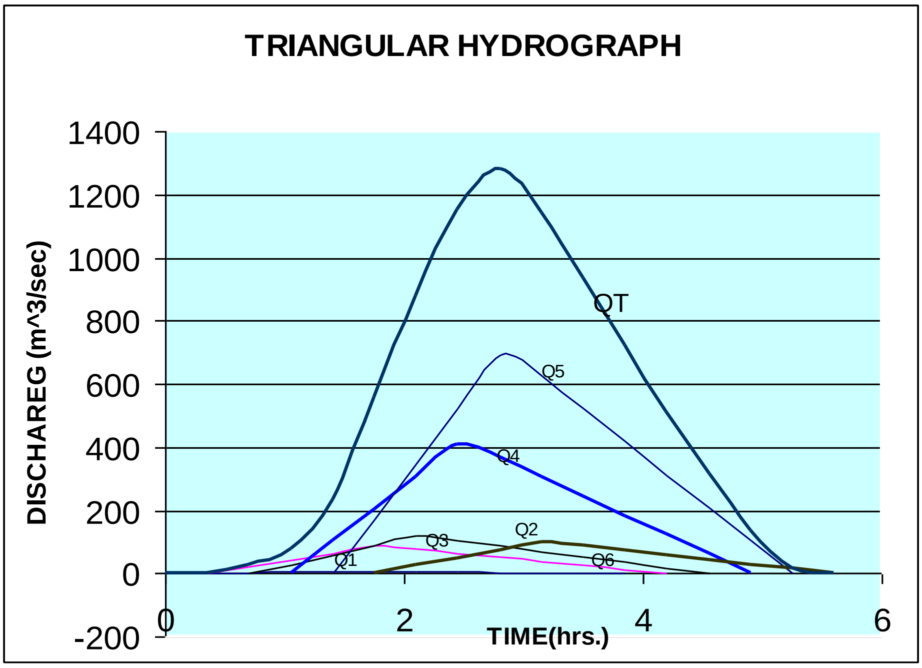

Figure 1 Triangular Hydrograph

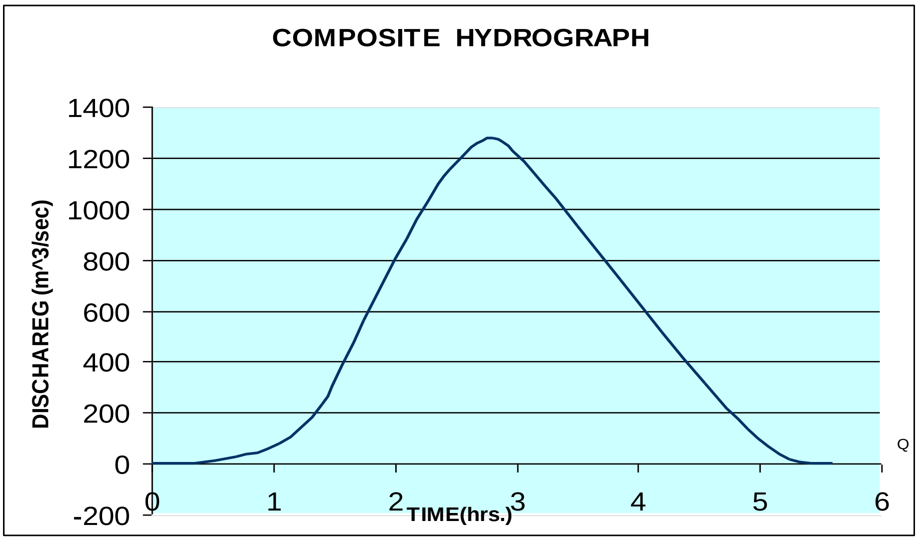

Figure 2 Composite Hydrograph

2.2.5 Stream Flow Measurement Method

The catchments of Sager and proposed weir site share common features such as; type of vegetation cover, soil type, agricultural practice and meteorological characteristics like the rainfall amount and distribution periods, and evaporation rates. However, the two catchment differ in some respects, particularly in general slope and catchment size. Therefore, the transposition of the flow recored was worked out based on drainage area ratio.

The drainage area of Sagure is 184.4 sq.m and that of the proposed weir site is 150 sq.m respectively, giving a drainage area ratio of 0.8.Therefore 0.8 plus 0.1 safety factor is used to transpose the Sagure flow to the proposed weir site.

Table 11.4a Summary of maximum discharge

year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Des	Max. flow
1981	0.00	0.09	1.92	2.61	1.10	0.32	31.7	39.8	28.9	8.70	0.31	0.22	39.8
1982	0.22	0.22	0.52	2.10	1.70	0.67	1.76	6.23	2.00	1.68	0.50	0.43	6.20
1983	0.13	0.27	0.32	1.52	2.35	2.26	2.44	26.56	6.46	8.66	0.78	0.43	26.6
1984	0.29	0.23	0.41	0.31	0.51	0.41	1.32	1.10	1.57	0.44	0.23	0.80	1.60
1985	0.26	0.23	0.34	0.37	1.00	0.34	1.26	1.44	1.15	0.55	0.19	0.15	1.40
1986	0.11	0.15	0.31	3.10	4.10	5.10	5.44	8.11	1.84	0.94	0.28	0.17	8.10
1987	0.15	0.19	2.36	2.28	2.13	1.10	0.41	0.47	0.47	0.63	0.19	0.31	2.40
1988	0.21	2.00	0.37	0.63	0.41	0.63	22.5	39.8	3.31	1.84	0.51	0.23	39.8
1989	0.19	0.41	0.63	1.21	0.58	1.32	3.03	1.70	1.20	0.71	0.31	0.47	3.00
1990	0.31	2.28	2.05	3.12	0.61	0.55	1.98	3.12	2.60	1.21	0.34	0.29	3.10
1991	0.29	0.55	1.63	1.70	0.60	0.42	1.70	3.50	2.94	0.44	0.23	0.47	3.50
1992	0.41	0.34	0.21	0.51	0.71	0.80	1.27	30.1	7.70	1.20	0.37	0.34	30.10
1993	1.40	1.00	0.41	2.44	1.70	0.80	1.32	6.20	2.00	1.40	0.80	0.23	6.20
1994	1.40	0.17	0.41	0.29	0.23	0.55	17.58	14.10	0.68	0.88	0.47	0.78	17.60
1995	0.21	0.18	0.89	0.51	0.41	0.28	31.70	10.30	28.9	0.55	0.79	0.80	31.70
1996	1.40	2.30	2.40	0.41	0.59	1.80	5.70	30.10	2.30	8.70	0.90	0.70	30.10
1997	0.40	0.19	0.23	1.32	0.27	1.00	1.84	1.60	0.14	1.00	0.85	0.34	1.80
1998	0.47	0.80	1.57	0.62	4.10	1.21	0.20	10.90	14.4	8.41	0.80	0.11	14.40
1999	0.29	0.19	0.34	0.13	1.15	5.09	4.97	2.70	3.20	3.10	0.94	0.23	5.10
2000	0.19	0.11	0.11	0.90	0.41	5.10	31.7	39.80	28.90	8.70	0.90	0.80	39.80

Drainage catchment and rainfall ratio= 0.91

Table 12.4a Transposed maximum discharge

year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Des	Max. flow
1981	0.046	0.082	1.747	2.375	1.001	0.291	28.847	36.218	26.299	7.917	0.282	0.200	36.220
1982	0.200	0.200	0.473	1.911	1.547	0.609	1.601	5.669	1.820	1.528	0.455	0.391	5.642
1983	0.118	0.246	0.291	1.383	2.138	2.056	2.220	24.169	5.878	7.880	0.710	0.391	24.210
1984	0.264	0.209	0.373	0.282	0.464	0.373	1.201	1.001	1.428	0.400	0.209	0.728	1.456
1985	0.237	0.209	0.309	0.336	0.910	0.309	1.146	1.310	1.046	0.500	0.173	0.136	1.274
1986	0.100	0.137	0.282	2.821	3.731	4.641	4.950	7.380	1.674	0.855	0.255	0.154	7.371
1987	0.137	0.173	2.148	2.074	1.938	1.001	0.373	0.427	0.427	0.573	0.173	0.282	2.184
1988	0.191	1.820	0.337	0.573	0.373	0.573	20.475	36.218	3.012	1.674	0.464	0.209	36.220
1989	0.173	0.373	0.573	1.101	0.527	1.201	2.757	1.547	1.092	0.646	0.282	0.427	2.730
1990	0.282	2.075	1.865	2.839	0.555	0.500	1.801	2.839	2.366	1.101	0.309	0.263	2.821
1991	0.264	0.501	1.483	1.547	0.546	0.382	1.547	3.185	2.675	0.400	0.209	0.427	3.185
1992	0.373	0.309	0.191	0.464	0.646	0.728	1.155	27.391	7.007	1.092	0.337	0.309	27.390
1993	1.274	0.910	0.373	2.220	1.547	0.728	1.201	5.642	1.820	1.274	0.728	0.209	5.642
1994	1.274	0.155	0.373	0.263	0.209	0.500	15.998	12.831	0.618	0.800	0.427	0.709	16.020
1995	0.191	0.164	0.809	0.464	0.373	0.254	28.847	9.373	26.300	0.500	0.719	0.728	28.850
1996	1.274	2.093	2.184	0.373	0.536	1.638	5.187	27.391	2.093	7.917	0.819	0.637	27.390
1997	0.364	0.173	0.209	1.201	0.245	0.910	1.674	1.456	0.127	0.910	0.774	0.309	1.638
1998	0.428	0.728	1.428	0.564	3.731	1.101	0.182	9.919	13.100	7.653	0.728	0.100	13.100
1999	0.264	0.173	0.309	0.118	1.046	4.631	4.522	2.457	2.912	2.821	0.855	0.209	4.641
2000	0.173	0.100	0.100	0.819	0.373	4.641	28.847	36.218	26.300	7.917	0.819	0.728	36.220
mean=Qi													14.220
S.dv=$\sigma n-1$													13.420

Table 13.4a Summary of Peak Discharge

Method used	Result	Remarks
Rational	2238 m^3/s	It is used for catchments less than 50 sq.km
Empirical	57.67 m^3/s	Applicable only in the region from which they were developed.
United states soil conservation (USSCS)	1278.56 m^3/s	Dredging of lands to widen the weir length from (11m to 80m) which is technically difficult.
Slope area	290 m^3/s	It is between the two extremes of the largest and the lowest. It also seems an optimal design discharge to implement small scale projects.
Gauging	62.2m^3/s	It is so small that, designing with this discharge may be danger. Because the weir may washed away Before its design period.

2.3 Calculation of Lean Flow

Stream flow measurement station was established down stream of the proposed weir site. The total catchment area, which drains to this gauging station, is about 184.4 sq Km. Since the gauging is not at the weir site transposing of flow is important.

Transposing of stream flow records to the weir site

1) Froster type 3 method

$$\sigma = \sqrt{\frac{\sum X^2}{n-1}} \quad , (15)$$

σ = Standard deviation

$$Cs = \frac{\sum X^3}{(n-1)\sigma^3} \quad , (16)$$

Cs =coefficient of skew ness

$$\overline{Cs} = Cs\,(1+\frac{8.5}{n}) \quad , (17)$$

$\overline{Cs}$ =Adjusted skew ness coefficient

Table 14 Calculation of K_{adj}

Average base flow by floating		Transferred value	
Volume (m^3/sec)	month	Volume (m^3/sec)	month
0.2464	April	.132	April
0.231	Feb/march	.066	Feb/march
0.3	Oct	.178	Oct
Average=.259		Average=.126	

$$K_{adj} = \frac{\text{mean by floting method}}{\text{mean transfered}} \quad , (18)$$

Kadj is adjustment factor

Therefore $K_{adj} = 0.259 / 0.126 = 2.06$

We have used the above parameter to calculate the following table.

Table 15 Lean Flow Analysis

Year	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec
1981	0.00	0.00	0.00	0.55	0.05	0.05	0.41	2.02	0.73	0.37	0.0	0.00
1982	0.09	0.09	0.09	0.26	0.34	0.22	0.32	2.20	0.67	0.37	0.20	0.13
1983	0.05	0.05	0.02	0.13	0.32	0.32	0.37	1.90	2.70	0.87	0.50	0.32
1984	0.23	0.17	0.17	0.19	0.17	0.04	0.31	0.51	0.41	0.19	0.20	0.98
1985	0.15	0.15	0.11	0.17	0.17	0.15	0.11	0.67	0.41	0.19	0.20	0.11
1986	0.08	0.09	0.08	0.24	0.23	0.24	0.51	1.55	0.71	0.19	0.10	0.13
1987	0.11	0.11	0.15	0.34	0.21	0.29	0.11	0.29	0.29	0.19	0.20	0.15
1988	0.15	0.13	0.15	0.15	0.21	0.23	0.41	1.63	1.15	0.55	0.20	0.19
1989	0.19	0.18	0.19	0.34	0.21	0.21	0.34	0.85	0.59	0.29	0.30	0.29
1990	0.29	0.29	0.44	0.51	0.34	0.29	0.51	1.32	1.00	0.34	0.20	0.23
1991	0.19	0.19	0.21	0.29	0.26	0.21	0.59	1.05	0.47	0.23	0.20	0.19
1992	0.19	0.19	0.17	0.23	0.29	0.34	0.47	0.67	0.67	0.37	0.30	0.26
1993	0.23	0.38	0.19	0.19	0.41	0.29	0.41	1.40	0.71	0.41	0.20	0.21
1994	0.51	0.38	0.47	0.50	0.55	0.80	1.40	1.55	0.90	0.28	0.70	0.55
1995	0.19	0.17	0.17	0.17	0.19	0.19	0.23	1.70	0.64	0.31	0.80	0.25
1996	0.17	0.17	0.16	0.19	0.19	0.23	0.31	1.84	0.55	0.37	0.30	0.25
1997	0.19	0.13	0.08	0.15	0.11	0.15	0.55	0.17	0.19	0.17	0.30	0.15
1998	0.13	0.15	0.21	0.15	0.17	0.19	0.34	2.08	0.63	0.82	0.30	0.29
1999	0.21	0.13	0.13	0.26	0.09	0.09	0.11	1.00	0.52	0.59	0.20	0.11
2000	0.11	0.08	0.04	0.26	0.05	0.24	0.41	2.02	0.53	0.37	0.30	0.25
mean	0.17	0.17	0.16	0.26	0.23	0.24	0.41	1.32	0.72	0.37	0.28	0.25
Std	0.10	0.13	0.12	0.13	0.12	0.16	0.27	0.63	0.52	0.20	0.19	0.20
Cs	1.60	2.05	1.47	1.25	0.86	2.48	2.60	0.00	3.17	1.43	1.76	2.65
CS	2.28	2.92	2.09	1.78	1.22	3.53	3.70	0.00	4.52	2.04	2.50	3.78
K80%	0.73	0.60	0.80	0.80	0.85	0.60	0.62	0.84	0.60	0.80	0.70	0.62
Q80% g=Qmean+K 80%* std	.097	0.09	0.07	0.16	0.12	0.14	0.24	0.79	0.40	0.22	0.10	0.13

III. Conclusions

Peak discharge Determination is one of the most important studies for Irrigation projects. The proposed of hydrologic design is to estimate maximum, average or minimum flood which the structure is expected to handle .This estimate has to be made quite accurately in order that the project can function properly. To estimate the magnitude of a flood peak the following alternative methods are available academically. This study was carried out to assess Hydrological Analysis and peak discharge determination. The proposed project of Gusha Temela diversion weir irrigation is predicted to bring both beneficial and adverse impacts on physical, biological and socio-cultural environment. Although the implementation of this diversion weir irrigation project has many benefits, obviously it will also bring a number of adverse impacts to the physical, biological and socio-cultural environment.

The following conclusions are drawn after the design of Gusha Temela SSIP.
In this study Hydrological analysis has been conducted based on 10years maximum daily rainfall data .The frequency analysis has been carried out by different methods and Log Pearson type III method is adopted .The peak discharge computed by United States Soil Conservation Service (USSCS) method is $1278m^3/sec$ .Since this method over estimates the design flood, we adopt the peak discharge calculated by slope -area method.
The peak discharge of Temela River is $290.00m^3/s$ and the weir is designed based on this value. Lean flow of the area is found to be $1.2m^3/s$, The peak duty is found to be 1.31 l/s/ha and the available lean flow is sufficient to fulfill the water demand of the crops and downstream requirement. Hence there is no need to construct any storage pond for the designed cropping pattern. Generally the use of irrigation in the study area is found to be essential to alleviate food shortage and to increase source of income of the community.

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