1. INTRODUCTION

1.1 GENERAL

Ethiopia, the mother of blue Nile, Awash, Baro etc, all in all the tower of Water resource in east Africa. But millions of people are suffering for famine frequently. No one Dare to say this is due to lack of a drop of water to crop production except lock of efficient and organized agricultural technology. Moreover, nowadays the population of Ethiopia is growing at alarming rate and hence the demand of crop production is high.

Irrigation is essential to overcome deficiencies in rainfall and stabilize agricultural production so that it and only it is under the bottom of heaven to the country.

Furfuro small scale Irrigation project is one of such a major development projects to enhance a sustainable agricultural production and hence guarantee food security. It has a total area of to be irrigated 150 ha.

1.2 PROJECT AREA

1.2.1 LOCATION:-

Furfuro small scale irrigation project is found in SNNPR silite Zone at Dalocha woreda, Dalocha is located at a distance of 48 km from Butajira and 208 km from the regional town Awassa.

Furfuro diversion project is located on the left foot of the Gurage mountain chain, which is also the part of the central rift system of Ethiopia. It is proposed on the river furfuro.

The Name of the scheme is given from the river name furfuro. It is proposed to irrigate command areas located both sides of the river, namely Bilwanija and Temeda, right and left side of the river respectively.

1.2.2 ACCESS TO PROJECT SITE

The scheme is accessible on two ways. The first way is from the Dalocha woreda traveling 15 km up to agam and from agam 5 km up to the Temeda (Left side). The problem o the way of the left side is that there is seasonal river Dijo after traveling 2 km from Agama. It has no flows for three months in a year. Even if it has no flow for these months, the river bed is highly deposited by sand for most of its length and flow of water in the river is sudden. This route cans serve as an alternative.

The second way (i.e Bilwanja side) is more preferable than the first one even if it is a dry weather road . It is from the whereby traveling 15 km up to Wulbareg and from wulbareq 15 km up to the Bilwanja (Right side).

1.3 TOPOGRAPHY.

The Furfuro River drains most of its catchments area which list around the Gurage mountain chain. Furfuro River flows southeast direction. The land elevation of furfuro catch ment around 1890m asL

1.3 CLIMATE

Ethiopia lies with in the tropics; temperatures are modified considerably by altitude giving rise to 5 distinct Climatic zones that are traditionally recognized through out the country.

The characters tics of these climatic zones are shown in the following table.

Zone	Altitude (m) asl	Mean Annual Temperature,^{o}c	Classification
Beraha	Below 500	>27.5	Hot Arid
Kolla	500 – 1500/1800	20 – 27.5	Warm semi –arid
Woina Dega	1500/1800 – 2300/2400	16/17 – 20.0	Cool – semi – humid
Dega	2300/2400 – 3200	11.5 – 16/17	Cool and humid
Wurch	> 3200	< 11.5	Cold and moist

Based on the climatic zones classification the furfuro irrigation project area lies with in the woina dega zone and is characterized by distinct dry and wet seasons.

The Objectives of this project are:

- To conserve the wet season river flows for irrigation use during the dry season
- To rise the living standard of the farmer with increased food production and assured security.
- To improve the efficiency of water for crops production by utilizing the available source of water

2 GEOLOGY

2.1 GENERAL

Geology is concerned with the history and make up of the earth and materials like soil and rock. It is important to under stand the nature of the underlying rocks strata and the near surface materials. Soils are derived from geological materials, which may be consolidated or unconsolidated by the action of climate and vegetation.

Geology gives clear information about the fed materials of the foundation i.e. the riverbed materials of the site for the proposed project

2.2 REGIONAL GEOLOGY

Quaternary deposit is the regional unit of the area. The quaternary deposit in the area is named as Dino formation. Dino formation comprises of ignibrite tuff coarse pumic with role interaction of lacustrine sediments.

2.3 LOCAL GEOLOGY

Pumic and alluvial deposits are the major units which characterize the area. The pumic has yellow color. It is highly weathered and fragmented in the upper part. Its weathering mantel is estimated in the range of 1 to 1.5 m. It is seen exposed on river cut exposures and mountain and hill side exposures. It is covered by shallow to medium thickness soil despite. The soil deposit in the area can be broadly classified according to its color. The red colored clay soil and dark colored Clay soil. The soil deposit has heavy texture.

2.4 GEOLOGY OF THE HEAD WORK AREA

The diversion axis is located across the river on hard and tuff pumic rock. The pumic rock on borders is more fragmented and tribal than the exposure on the river bed.

The weathered pumic rock has yellowish gray color. The fresher of the units on the head work area has its own engineering significant. Fault and fissures with significant opening if encountered across the proposed weir axis may has significant transmitted which may affect the weir’s function. Seepage reduces the acquired head. Seepage through highly transmitted fissures may also induces up lift pressure on the weir During construction it ahs to be examined and the filling pervious materials has to be mine out and granted with proper materials . And cutoff trench has to be constructed across the fissure in order to avoid the problem mentioned above.

2.5 GEOLOGY OF THE MAIN CANAL

The main canal course is aligned on highly weathered pumic rock. It is weathered and fixable on the upper 1 m depth. It has light yellow color of tresh color and dark yellow weathered color. Water tight and stability is the character sties qualities of the units. It is water tight intact rock but fissure with significant permeability appear on its course.

On the part of the main canal is traversing on heavy clay soil with though and consolidated heavy clay soil with dark gray color. It is water tight and stable

2.6 Soils of site

During the feasibility study a method of visual observation to various fields to identify the type and the nature of the surface and sub-surface soils of the command area have been used. This is done using test pits in up ,mid and low land of the command.

The proposed command areas have a soil type ranging from moderate to heavy textured black soil , which are grouped under vertisol from these type of soils Clay Loam and silt clay loams are the dominant type of soils widely observed with in the command area. These soils are fertile soils having good water holding capacity and hold most of the plant nutrients.

3 HYDROLOGICAL ANALYSIS

3.1. General

Whenever a hydraulic structure is to be constructed on a river, it must be property planned and design keeping in view the damage to which it is susuptable and the catastrophic, which it is going to create in the event of its failure.

Therefore, while designing a hydraulic or Irrigation structures we have to determine a flood value against which those structures can be designed to be safe. Hydrological data is need for determine the minimum, mean and maximum discharge by the river at a proposed location of the structure, If the river is ungauged the peak discharge can be determined by calculating the design storm from available rainfall data and then synthesizing the flood hydrograph. This is the case that we have and the above parameter can be determined from the available 26 years of maximum. 24 hours/daily rainfall, at wulbareg meteorological station. This is in the catchments.

3.2. Determination of Return Period.

The return period T indicated the average interval between the Occurrence of flood equal to or greater than a given magnitude. The selection of the design return period depends on economic balance between the cost of periodic repair or replacement of the facility, and the cost of providing additional capacity to reduce the frequency of repair or replacement.

For small-Scale Irrigation project, it would be recommended that the project design flood once in 100 years be used for design of storage dams, the flood once in 50 years for design of diversion weirs, and the flood once in 10-20 years for design of drainage structures.

However, there is no guarantee that such a flood would not occur during the useful life of the project as there is always some risk. To reduce the risk, the design return period of very important structures such as, spillways for very high dams is taken very long. The following table gives the recommended design return period and factor of safety.

Table:- 3.1 Safety Factor for Various Return Period .u

Type of structure	Project life (years)	Return period (years)	Safety factor (Percent)
Storage dams	30	200 100 50	86 74 54
Diversion weir and drainage structures	15	50 20	74 54

3.3. Maximum design flood estimation (life period of the structure).

The maximum design flood is the peak flood that corresponds to a certain return period. While designing hydraulic or Irrigation structures we, can neither choose a very high value nor can we choose a very law value. Because a very high value chosen for the design, will make the design very costly causing unnecessary expenditure, since such a high value flood may never occur during the lifetime of the structure.

In the same way, a very low value chosen for the design, if exceeded during the lifetime of the structure, will result in the failure of the structure causing much more damage than what it would have caused in the absence of the structure.

Therefore, it is necessary to determine the optimum maximum design that makes the structure safe. The following flood methods are commonly use for the estimation of the design rainfall.

1. Rational Method
2. Empirical Method
3. Unit hydrograph method
4. Flood frequency analysis
    - Gumbel's method
    - Log pear son's type III distribution method
    - Log Normal distribution Method
    - .Ven te chow method

3.3.1. Rational Method

According to the rational formula, the max flood is given by:

$$Rp = \frac{C*I*A}{3.6}$$

Where: - C= Coefficient of runoff = (runoff/rainfall)

A= Drainage area in Km2

I = the mean intensity in (mm/h) for a duration equal to Tc (time of Concentration) and an exceednce probability P.

Rp = Peak flood (mm)

This method is only applicable to catchments area of up to 50 km^2. But we have catchments area of 208km^2 There fore, this method is not used for this particular project.

3.3.2. <u>Empirical formulae</u>

The empirical formulae used for the estimating of the flood peak are essentially regional formulae based on the statistical correlation of the observed peak and important catchments property, but some of them have found a general existence. No particular formula will give precise results for the entire place.

Regression analysis study consisting of 42 catchments in Ethiopia ranging in size 200-9980 Km2 as developed by Dr. Admasu Gebeyehu (1989). The formula can be safely used for most river basin in Ethiopia .

$RT = \check{R}(1+K_T*C_r)$ (General formula)

Where:

$R= 0.87(A)^{0.70}$ (Dr. Admasu. Gebeyhu's)

A= Catchments Area Km^2

K_T = Frequency factor which is given by

$$\frac{-\sqrt{6}}{\pi}\left[0.57721+\ln\left(\ln\left(T/_{T-1}\right)\right)\right]$$

For T = 50 year return period

$$K_T = -\sqrt{6}/[\Pi]\left[0.57721+\ln\left(\ln\frac{50}{50-1}\right)\right]$$

=2.592

C_V = Average coefficient of variation

= 0.38 for most cases.

R = 0.87 (A) 0.7

R = 0.87(208) 0.7 = 36.49mm

RT = R $(1+K_T*C_r)$

= 36.49 (1+2.592*0.38mm)

RT = 72. 72.43mm

3.3.3. Unit hydrograph technique

A unit hydrograph is defined as the hydrograph of direct run-off resulting from one-unit depth (1cm) or rainfall excess occurring uniformly over the basin and at a uniform rate for a specified duration (D hrs) . (Subramanian, 2000)

This method is convenient for watershed areas ranging 25mm^2 to 5000km2. The proposed Project area (208Km2) is in this range, however, this method requires a large Number of observed date, for which more Number of gauging stations required to install in the water shed and hence in our case such conditions are not statistical for the method to be adopted.

3.3.4. Synder's Method

Synder's Method:- synder (1938) developed a set of empirical relation on the basis of analysis of large Number of hydrographs resulting from several Water shed ranging from 25km2 to 2500km2 in the site of united states (R.suresh, 1997) and with some modification in many other countries.

1) **Lag time**:-

$T_p = c_t\,(L..L_{ca})^{0.3}$

Standard duration of net rain fall, $t_r = \dfrac{tp}{5.5.}$

2) Peak- flow rate for standard duration tr

$$Qp = \frac{2.78 * Cp * A}{tp}$$

Where,

Ct, Cp = empirical constants, depend upon the watershed characteristics in which Ct approximately ranging from 0.3 to 0.65 and Cp from 0.56 to 0.69

A= water shed area (Km2)

L = Longer Length of Water shed, Km

Lca = Length along the main stream from gauging station to a point opposite the water shed centroid in km.

The synthetic unit hydrograph are developed on the basis of known physical characters tics of a gauged water shed, which is identical, both hydrologically and metrologically, ungauged water shed however, in our case there are not such metrological and hydrological similarity gauged water shed.

3.3.5. Flood Frequency Analysis

Hydrological Processes such as floods are exceedingly complex natural events. This makes the estimation of the flood peak a very complex problem leading to many different approaches. This method of frequency analysis is an approach to the prediction of flood flows and also applicable to other hydrological processes such as rain fall etc.

Some of the common used frequency distribution functions are-

- Gumbels' extreme value distribution
- Log Pearson type III distribution

3.3.5.1. Gumbel's Method

Gumbel distribution is one of the most widely used probability distribution function for extreme values in hydrologic and meteorological studies for prediction if flood peak, maximum rainfall

Gumbal defined a rainfall as the largest of the 365 daily falls and the annual series of rainfalls constitute a series of largest values of falls.

Gambell uses the general equation of hydrologic frequency analysis.

$X_{T} = x + K\sigma$

Where x_T = any variable such as Rain fall

X = mean of variate

The equation is applied under the following procedure

1. Arrange the maximum of the annual rainfall in descending order and determine the mean x and the standard deviation of the rain fall
2. Using relevant tables determine y_n and S_n appropnte for a given N.

   S_n = reduced standard deviation

   Y_n = Reduced mean
3. Find

   $Y_{(T)} = -\left(l_n . l_n \frac{T}{T-1} \right)$

   Where $Y_{(T)}$ = reduced variate
4. find K by equation

   $$K = \frac{Y(T) - Yn)}{Sn}$$

   Where:

   K =frequency factor.
5. Finally determine R_p by using equation

   $RT = R + K\sigma_{n-1}$

   RT = peak rain fall

   R = mean rainfall

Year	Peak rainfall Arranged in m^3/s descending order(Pp) mm	Ranking of rain fall	Frequency or return period(T)=N+1/m	Probability of excedenceP=1/T*100	R_P^2
1987	67.7	1	1.00	100.00	4583.3
1989	59.4	2	0.50	200.00	3528.4
1988	58.6	3	0.33	300.00	3434.0
1997	53.3	4	0.25	400.00	2840.9
1984	52.7	5	0.20	500.00	2777.3
1975	48.7	6	0.17	600.00	2371.7
1986	47.6	7	0.14	700.00	2265.8
1985	43.4	8	0.13	800.00	1883.6
1991	42.3	9	0.11	900.00	1789.3
1980	41.8	10	0.10	1000.00	1747.2
1972	41.3	11	0.09	1100.00	1705.7
1978	38.6	12	0.08	1200.00	1490.0
1996	37.2	13	0.08	1300.00	1383.8
1981	36.5	14	0.07	1400.00	1332.3
1995	35.3	15	0.07	1500.00	1246.1
1979	35.1	16	0.06	1600.00	1232.0
1990	34.8	17	0.06	1700.00	1211.0
1976	33.6	18	0.06	1800.00	1129.0
1977	33.3	19	0.05	1900.00	1108.9
1983	32.5	20	0.05	2000.00	1056.3
1973	30	21	0.05	2100.00	900.0
1992	29.7	22	0.05	2200.00	882.1
1982	29.1	23	0.04	2300.00	846.8
1974	25.4	24	0.04	2400.00	645.2
1994	24.3	25	0.04	2500.00	590.5
1993	23.5	26	0.04	2600.00	552.3

$\sum R_P = 1035.7$ $\qquad$ $\sum R_P^{\ 2} = 44533.1$

Using Gumbel’s Analytical Method

Mean peak dally rain fall.

$Rp = {}^{\sum Rp}/_{N}$ = $1035.7/_{26}$

= 39.84mm

∑(Rp-Rp) 2 = 3276.39mm

σ_{n-1} = $\sqrt{\sum\left({(x_i - x)^2}/_{N-1}\right)}$

where:

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

$= \sqrt{\sum {(Rp - Rp)^2}/_{N-1}}$

$= \sqrt{3276.39/_{25}}$

=11.45

Y_T = reduced variate a function of T and is given by

Yr = $-\left[l_n * l_n\left(T/_{T-1}\right)\right]$

= 3.902

Using tables determine Yn and Sn appropriate to given N

Yn = 0.5320

Sn = 1.0961

☛ K =(YT-Yn)/Sn

=(3.902-0.5320_/1.0961

K=3.0743

RT =R+K σ_{n-1}

= 39.84+3.0745*11.45

RT = <u>75.043mm</u>

3.3.5.2. <u>Log Pearson type III distribution method</u>

In this method the variate (i.e Rain fall in this case) is first trans formed into logarithmic form (base 10) and the transformed data is then analysed . If R is the variate of a random hydrologic series then the series of Z variate is thus obtained by

$Z = \log R_T$

The Value of the Variate Z_T for any recurrence interval T is given by

$Z_T = Z + K_Z\,\sigma_z$ ....(1)

Where:

K_Z = a frequency factor which is a function of recurrence interval T and the coefficient of skew C_S

σ_z = standard deviation of the Z Variety sample

and C_S = coefficient of skew of variate Z

$$= \quad N\sum(z-z)^3 \Big/ \left((N-1)(N-2)(\sigma_Z)^3\right)$$

Z = mean of the Z values

N = sample size = Number of years of recode

The variation of $K_Z = f(C_S, T)$ is given in table -------After finding Z_T by equation (1) , the corresponding value of R_T is obtained by eq

$R_T = \text{antilog}\,(Z_T)$

Using log Pearson type iii distribution method

$Z = \sum Z \big/ N$ =41.18/26=

=1.58

$\sum(Z-Z)^2$ =0.369115

$\sum(Z-Z)^3$ = 0.00758

$$\sigma_Z = \sqrt{\sum(Z-Z)^2 \big/ N-1} = \sqrt{0.369/25}$$

=0.1215

$$C_S = \frac{N\sum(Z-Z)^3}{(N-1)(N-2)(\sigma)\ 3} ==0.1$$

Year	Peak rainfall Arranged in m³/s descending order(Rp) m m	Rank	Z=logR$_{10}$	(Z-Z)	(Z-Z)^2	(Z-Z)^3
1987	67.7	1	1.83	1.83	3.351	6.134403
1989	59.4	2	1.77	1.77	3.146	5.580897
1988	58.6	3	1.77	1.77	3.125	5.525497
1997	53.3	4	1.73	1.73	2.982	5.148387
1984	52.7	5	1.72	1.72	2.965	5.104534
1975	48.7	6	1.69	1.69	2.848	4.805667
1986	47.6	7	1.68	1.68	2.814	4.721398
1985	43.4	8	1.64	1.64	2.681	4.39072
1991	42.3	9	1.63	1.63	2.645	4.301643
1980	41.8	10	1.62	1.62	2.628	4.260796
1972	41.3	11	1.62	1.62	2.611	4.219722
1978	38.6	12	1.59	1.59	2.517	3.993852
1996	37.2	13	1.57	1.57	2.467	3.873909
1981	36.5	14	1.56	1.56	2.441	3.81318
1995	35.3	15	1.55	1.55	2.396	3.707859
1979	35.1	16	1.55	1.55	2.388	3.690153
1990	34.8	17	1.54	1.54	2.376	3.663512
1976	33.6	18	1.53	1.53	2.330	3.55593
1977	33.3	19	1.52	1.52	2.318	3.528777
1983	32.5	20	1.51	1.51	2.286	3.45585
1973	30	21	1.48	1.48	2.182	3.222912
1992	29.7	22	1.47	1.47	2.169	3.194426
1982	29.1	23	1.46	1.46	2.143	3.137097
1974	25.4	24	1.40	1.40	1.974	2.77252
1994	24.3	25	1.39	1.39	1.920	2.660232
1993	23.5	26	1.37	1.37	1.880	2.577371

$\sum R$ = 1035.7 $\sum Z$ =41.18

$\sum(Z-Z)^2$ =0.36915 $\sum(Z-Z)^3$ =0.00758

From the above table for Tr = 50 and C_S = 0.185

We have K = 2.150

Now Z_T = z+K.σz

= 1.58+2.150*0.1215

Z_T = 1.841

Then RT = antilog (Z_T) = 10^{2T}

R_T = 69.379mm

3.3.5.3. **Log Normal distribution method**

Log normal distribution is a special case of log Pearson type III

Distribution with C_S = O. Thus, C_S isis taken as zero. The other statistics are as calculated above.

Or when the skew is zero i.e. C_S =0, the log – Pearson type III distribution reduces to log Normal distribution.

The value of K for a given return period T = 50 and C_S = 0 is read from table

K = 2.054.

The other statistics are Z = 1.58 and D_Z = 0.1215 as calculated about

Z_T = Z+ K σZ

= 1.58 + 2.054* 0.1215

Z_T = 1.830

R_T = antilog (Z_T) = $10^{ZT.}$

= antilog (1.830)

= 67.54m

3.3.5.4. **Ven Te chow method**

V.T chow made another modification of Gumbel's method by using the frequency factor.

The equation is given by;

$$R_T = a+b.X$$

Where:

$$X_{T\ =Log*Log}\left(\frac{T}{T-1}\right)$$

$$T=\frac{N}{m+1}$$

A, b: parameters estimated by the method of moments from the observed data.

The following equations are derived from the method of least square

$\sum R = aN + b\sum X_T$---------------- (1)

$\sum(RX_T) = a\ \sum X_T+b\sum(X_T)^2$

Year	Peak rainfall Arranged in m^3/s descending order(Pp) mm	Ranking of rain fall	Frequency or return period T=(N+1)/m	X_T=log.log (T/T-1)	P.X_T	X_T^2
1987	67.7	1	27.00	-1.785	-120.872	3.188
1989	59.4	2	13.50	-1.476	-87.671	2.178
1988	58.6	3	9.00	-1.291	-75.660	1.667
1997	53.3	4	6.75	-1.157	-61.677	1.339
1984	52.7	5	5.40	-1.051	-55.382	1.104
1975	48.7	6	4.50	-0.962	-46.849	0.925
1986	47.6	7	3.86	-0.885	-42.123	0.783
1985	43.4	8	3.38	-0.816	-35.432	0.667
1991	42.3	9	3.00	-0.754	-31.905	0.569
1980	41.8	10	2.70	-0.697	-29.134	0.486
1972	41.3	11	2.45	-0.644	-26.577	0.414
1978	38.6	12	2.25	-0.593	-22.890	0.352
1996	37.2	13	2.08	-0.545	-20.266	0.297
1981	36.5	14	1.93	-0.498	-18.190	0.248
1995	35.3	15	1.80	-0.453	-15.999	0.205
1979	35.1	16	1.69	-0.409	-14.355	0.167
1990	34.8	17	1.59	-0.365	-12.707	0.133
1976	33.6	18	1.50	-0.321	-10.798	0.103

1977	33.3	19	1.42	-0.277	-9.229	0.077
1983	32.5	20	1.35	-0.232	-7.537	0.054
1973	30	21	1.29	-0.185	-5.548	0.034
1992	29.7	22	1.23	-0.135	-4.017	0.018
1982	29.1	23	1.17	-0.081	-2.365	0.007
1974	25.4	24	1.13	-0.020	-0.517	0.000
1994	24.3	25	1.08	0.053	1.293	0.003
1993	23.5	26	1.04	0.156	3.660	0.024

$\sum R = 1035.7$ $\sum R.X_T = -752.75$

$\sum X_T = -15.4255l$ $\sum (X_T)^2 = 15.044$

- 1035.7 =26 a+ (-15.4255) b

-752.75 = (-15.4255) a + (15.044) b

= 1035.7 =26 a – 15.426 b

-752.75 = 15.426 a ++ 15.044 b

From the above two equations a & b can be solved

As: a = 25.897 &

B = -23.492

Then $\Rightarrow R_T = a + bX_T$

$X_{T\,=}$ log .log $\underline{(T/T\text{-}1)}$

X_{50} = log .log ($\underline{50/49}$)

= - 2.057

R_T = 25.897 –23.492* (-2..057)

R_T = 74.220m

TABLE 3.3

Summery of the results for 50 years return period with different approaches.

No	Method	Rainfall magnitude (mm)
1	Gumbel's	**75.043**
2	Log Pearson type III	69.379
3	Log Normal	67.54
4	Ven te chow	74.22
5	Dr.Admassu Gebeyehu's formula	72.43

TABLE 3.4 Computation of peak discharge by scs method

step	Designation /Formula	symbol	unit	Measured Or computed	Remark
1	Area of catchments	A	$(Km)^2$	208	Measured from Top map
2	Length of main water course from watershed divide to proposed diversion or storage site	L_1 ,L_{15} L_2 , L_{16} L_3 , L_{17} L_4 , L_{18} L_5, L_{19} L_6, L_{20} L_7 , L_{21} L_8, L_{22} L_9 , L_{23} L_{10} , L_{24} L_{11}, L_{25} L_{12} , L_{26} L_{13} , L_{27} L_{14} , L_{28} , L_{29}	m	650 , 800 450 ,350 850 ,500 750 ,1450 700 ,300 2000 ,2850 250 ,1400 100 ,2000 150 ,1100 100 ,5850 900 ,1450 850 ,1400 250 ,1550 600 ,1950 , 1950	Measured from top map

3	Elevation of watershed divide opposite to the Head of the main Water course	H_1 , H_{16}	m	2640 ,2180	measured from top map
		H_2 , H_{17}		2600 ,2160	
		H_3 , H_{18}		2580 ,2100	
		H_4 , H_{19}		2560 ,2080	
		H_5 , H_{20}		2540 ,2060	
		H_6 , H_{21}		2520 ,2040	
		H_7 , H_{22}		2460 ,2020	
		H_8 , H_{23}		2440 ,2000	
		H_9 , H_{24}		2420 ,1980	
		H_{10} , H_{25}		2400 ,1960	
		H_{11} , H_{26}		2380 ,1940	
		H_{12} , H_{27}		2360 ,1920	
		H_{13} , H_{28}		2240 ,1900	
		H_{14}, H_{29}		2220 ,1880	
		H_{15}, H_{30}		2200 ,1870	
4	Slope of the main course	S_1 ,S_{16}		6.15 ,5.71	
	$S_{i+i} = \frac{Hi - Hi + 1}{Li + 1}$	S_2 ,S_{17}		4.44 ,12.00	
		S_3 ,S_{18}		2.35 ,1.38	
	i = 1 to 29	S_4 ,S_{19}		2.67 ,6.67	
		S_5 ,S_{20}		2.86 ,0.70	
		S_6 ,S_{21}		3.00 ,1.43	
		S_7 ,S_{22}	% ,m/m	8.00 ,1.00	
		S_8 ,S_{23}		20.00 ,1.82	Computed value
		S_9 ,S_{24}		13.33 ,0.34	
		S_{10} ,S_{25}		20.00,1.38	
		$S_{1\,1}$,S_{26}		2.22 ,1.43	
		$S_{1\,2}$,S_{27}		14.12 ,1.29	
		S_{13} ,S_{28}		8.00, 1.03	
		$S_{1\,4}$,S_{29}		3.33 ,0.51	
		S_{15} ,		2.50	
5	Time of concentration	T_C	br	10.67	Computed value

6	Rainfall excess duration $D \approx \frac{Tc}{6}$, if Tc<3 hr $D \approx 1hr$,if Tc>3	D	hr	1	
7	Time to peak $Tp = 1/2\ D + 0.6\ T_C$	T_p	hr	6.902	Computed value
8	Time base of hydrograph $Tb = 2.67\ tp$	T_b	hr	18.4280	Computed value
9	Lag time $T_L = 0.6\ T_C$	T_L	hr	6.402	computed
10	Peak rate of discharge Created by 1mm runoff excess on whole of the catchments $Qp = (0.21A)/tp$	Qp	m^3/sec	6.329	computed

- Time of concentration (T_C) of drainage basin is the time taken for water to travel from the hydraulically most distant part of the watershed to the point of the watershed out let .
- Time to peak :- is the time from the starting point to the peak point of the hydrographs.
- Lag time: - is the time difference between the centroid of the rainfall excess and the surface runoff.

3.4 Determination of Curve Number.

The curve Number (CN) is an index, which is used to estimate the direct runoff (depth) or rain fall excess, storm wise (By Suresh, 1997) . It is deter mined by the antecedent moisture conditions and the physical characteristics of the watershed such as land use, cover, soil classification and hydrologic condition..

The retention capacity (s) of the water shed can be predicted using curve Number, as defined by U.S soil conservation service (1996) given as

$$CN = \frac{254000}{254+S} \text{--------------- (a)}$$

The curve Number methods assumes the following stated equation

$$\frac{Q}{P-Ia} = \frac{P-Q-I_a}{S} \text{------------- (b)}$$

Where:

CN = curve number, percent of runoff

S = potential maximum retention a after runoff begins

Q = accumulated runoff in mm

P = accumulated rainfall in mm

I_a = Initial abstract ion

Through studies of several small water shed, I a was found to be approximated by the empirical equation as

Ia = 0.2.S

Substituting this approximate in eguation (b) , then

$$Q = \frac{(p-0.2S)^2}{P+0.8S} \text{-----------(C)}$$

12	13	14	15	16	17	18	19
Duration	Dairy point rainfall for return period of 50 year	Rainfall profile	Rainfall profile	Area of paint rainfall ration	Area rainfall	Incremental Rainfall	Descending order
Hr	mm	%	mm	%	mm	mm	number
0.0-1		45	33.769	57.68	19.478	19.478	(1) 19.478
1-2		58..75	44.088	64.48	28.428	8.950	(2) 8.950
2-3		66.25	49.716	72.68	36.134	7.706	(3) 7.706
3-4	75.043	72.5	54.406	75.68	41.174	5.04	(4) 5.04
4-5		76.25	57.22	77.68	44.448	3.274	(5) 3-274
5-6		78.75	59.096	77.44	45.764	1.316	(6) 1.316.

12	Fill in 0-Dhr,D-2Dhr ,---------------5D-6Dhr				
13	Determine the magnitude of the daily rainfall with the given recurrence				
14	Read from fig-----the rainfall profile (%) occurring in D_1,2D,3D ,4D,5D,6D hours and put in 14.				
15	Multiply 13 and 14 to find the rain fall profile (mm) and enter in 15				
16	Read from table ----------- areal to point rainfall ration for different duration in a particular catchments				
17	Multiply 15 and 16 enter in 17				
18	Calculate incremental rainfall by deducing the current aerial rainfall from the preceding areas rainfall as written in 18				
19	Assign order to the rainfall depths in descending order 1 to 6				
20	21	22	23	24	25
Rearranged order	Rearranged incremental rainfall	Commutative rainfall		Time of incremental hydrograph	
			Time of beginning	Time to peak	Time to end
Number	mm	mm	hr	hr	hr
6	1.316	1.316	0	6.902	18.428
4	5.04	6.356	1	7.902	19.428
3	7.706	14.062	2	8.902	20.428
1	19.478	33.540	3	9.902	21.428
2	8.950	42.490	4	10.902	22.428
5	3..706	46.196	5	11.902	23.428
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 Rain fall value to the rearranged order of 20 from 18				

<table>
<tr><td>22</td><td colspan="7">Fill in the corresponding incremental values of 21 by adding with the Rainfall values in the preceding duration.</td></tr>
<tr><td>23</td><td colspan="7">Fill in the time of beg up of the hydrograph as 0,D,2D,------5D hr</td></tr>
<tr><td>24</td><td colspan="7">Fill in the time of peak as tp, D+ tp, 2D+tp,---------5D+p or add tp in every value of 23 and mention in 24</td></tr>
<tr><td>25</td><td colspan="7">Add tp in every value of 23 and fill in 25</td></tr>
<tr><td>26</td><td>27</td><td>28</td><td>29</td><td>30</td><td>31</td><td colspan="2">32</td></tr>
<tr><td>Land use cover</td><td>Area ratio (%)</td><td>Hydrologic condition</td><td>Curve No ‘’C.N’’</td><td>Hydrologic sail group</td><td>Weighted ‘’CN’’</td><td colspan="2">‘’CN’’</td></tr>
<tr><td>Cultivated land</td><td>70</td><td>poor</td><td>88</td><td>C</td><td>61.6</td><td>AMC</td><td>CN</td></tr>
<tr><td>Forest land</td><td>6</td><td>fair</td><td>79</td><td>C</td><td>4.74</td><td>II</td><td>86.88</td></tr>
<tr><td>Grass land</td><td>15</td><td>poor</td><td>86</td><td>C</td><td>12.9</td><td></td><td></td></tr>
<tr><td>Bush l and</td><td>5</td><td>poor</td><td>86</td><td>C</td><td>4.3</td><td></td><td></td></tr>
<tr><td>Wood Land</td><td>2</td><td>poor</td><td>77</td><td>C</td><td>1.54</td><td></td><td></td></tr>
<tr><td>gullies</td><td>2</td><td>poor</td><td>90</td><td>C</td><td>1.8</td><td>III</td><td>94.44</td></tr>
<tr><td>26</td><td colspan="7">identify all types of land use cover such as cultivated land, forest l and, grass l and, Bush land , wood land and gullies from the catchments map or aerial photo.</td></tr>
<tr><td>27</td><td colspan="7">Find Ratios of each type land use cover to the catchments area & list in 27</td></tr>
<tr><td>28</td><td colspan="7">Accelerating treatment practice of each type of land use cove hydrologic condition corresponding to it from the catchments map</td></tr>
<tr><td>29</td><td colspan="7">Read from table --------of a curve numbers for hydrologic soil- cover complexes for Antecedent moisture condition class II.</td></tr>
</table>

30	Ascertain Hydrologic soils groups for each type of l and use corer as below Group A low runoff potential Group B moderate Run off potentials Group C soil having high runoff potential Group D soil having very high runoff potential
31	Multiply 27 and 29 and list in 31
32	Add 31. This curve number (CN) is corresponding to Antecedent moisture condition II (AMC-II). Find ''CN'' for AMC-III from table corresponding to AMC-II ''CN'' Va

Hydrologic Condition

Based up on l and coverage the hydrologic condition is said to be poor, fair or good as explained below.

Poor: - lass than 50% of the land is covered by canopy (area covered by plants). That is more run off

Fair: - 50 to 70% of the area is covered by canopy i.e. medium runoff.

Good: - >75% of the area is covered by canopy.

Antecedent Moisture Condition (AMC). :- There are three level of AMC in use

.AMC –I :- It implies lower runoff potential I.e. the water shed soil is dry enough for satisfactory cultivation to occur.

Preceding 5 day drain : < 12 mm (dormant season)

> 36 mm (growing season)

AMC – II ;- Average Condition

Proceeding 5 days drain: < 12-88 mm (dormant see son)

> 36 mm (growing season)

AMC – III ;- It implies highest run off potential

Preceding 5 days drain :- > 25 mm (dormant season)

< 23 mm (growing season)

(source: Hand book of hydrology MOA ,GDI, 1972)

No	Description Formula	symbol	unit	Example
33	Find maximum potential difference b/n rainfall (p) and direct runoff (Q) which is given in the following formula $S = \frac{25400}{CN} - 254$	S	mm	CN=94.44 S = 14.954
34	Substitute the value of ''S'' in the following formula given in the relation between direct runoff (Q) and rainfall (P) $Q = \frac{(p-0.25)^2}{P+0.85}$	Q	mm	$= \frac{(p-2.99)^2}{p+11.96}$
35	Substitute values of p, as mentioned in 22 in the above formula and find the corresponding			
		22		35

	values of Q (35) enter r the value of Q in 35.		P(mm) 1.316 6.356 14.062 33.540 42.490 46.196	Q(mm) 0.2111 0.619 4.711 20.512 28.655 32.099			
36	37	38	39	40			41
					24	25	
Duration	Cumulative runoff	Incremental runoff	Peak runoff for increment	Time of begin up	Time to peak	Time to end	Composite hydrograph
Hr	mm	mm	M^3/sec	Hr	Hr	Hr	T,Q
0-1	0.2111	0.2111	1.336	0	6.902	18.428	
1-2	0.619	0.408	2.582	1	7.902	19.428	
2-3	4.711	4.092	25.898	2	8.902	20.428	
3-4	20.512	15.801	100.055	3	9.902	21.428	
4-5	28.655	6.143	51.537	4	10.902	22.428	
5-6	32.099	3.444	21.797	5	11.902	23.428	
36	Enter the same time as in (12) i.e 0 -D, D -2D, ...,5D- 6D						
37	These are the values of Q as found out in 35 corresponded to the value of p						
38	Find incremental run off by reducing the values of 37 by preceding values.						
39	Multiply 38 with peak rate of runoff corresponding to 1mm runoff excess as found in 10						
40	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 39						

41	Plot a composite hydrograph by Adding all the triangular hydrograph. The resulted hydrograph will be composite hydrograph of desired return period. The coordinate of the peak of this hydro graph with give the peak runoff with desired return period

Tabel 3.5 Synthesis of complex Hydrograph.

Time	**Ordinates of unit hydrograph (m^3/sec)**							
	D	2D	3 D	4 D	5 D	6 D	Cumulative	Remark
0.	0	-	-	-	-	-	0	
1.	0.194	0	-	-	-	-	0.194	
2.	0.387	0.374	0	-	-	-	0.761	
3.	0.581	0.748	3.752	0	-	-	5.081	
4.	0.774	1.122	7.504	14.497	0	-	23.897	
5.	0.968	1.496	11.257	28.993	7.467	0	50.181	
6.	1.160	1.870	15.009	43.490	14.934	3.158	79.621	
7.	1.325	2.245	18.761	57.986	22.401	6.316	109.034	
8	1.209	2.560	22.513	72.483	29.868	9.474	138.107	
9	1.093	2.336	25.678	86.979	37.335	12.632	166.054	
10	0.977	2.112	23.431	99.204	44.802	15.790	**186.316**	**Q_{PEAK}**
11	0.861	1.888	21.184	90.523	51.099	18.948	184.503	
12	0.745	1.664	18.937	81.843	46.627	21.612	171.428	
13	0.629	1.439	16.690	73.162	42.156	19.720	153.796	
14	0.513	1.216	14.443	64.481	37.685	17.829	136.167	
15	0.397	0.992	12.196	55.800	33.213	15.938	118.536	
16	0.281	0.768	9.949	47.119	28.742	14.047	100.906	
17	0.166	0.544	7.702	38.439	24.271	12.156	83.27	
18	0.0496	0.319	5.455	29.758	19.799	10.265	65.646	
19		0.096	3.209	21.077	15.329	8.374	48.085	
20			0.962	12.396	10.856	6.483	30.697	
21				3.715	6.385	4.592	14.692	
22					1.914	2.701	4.615	
23						0.809	0.809	

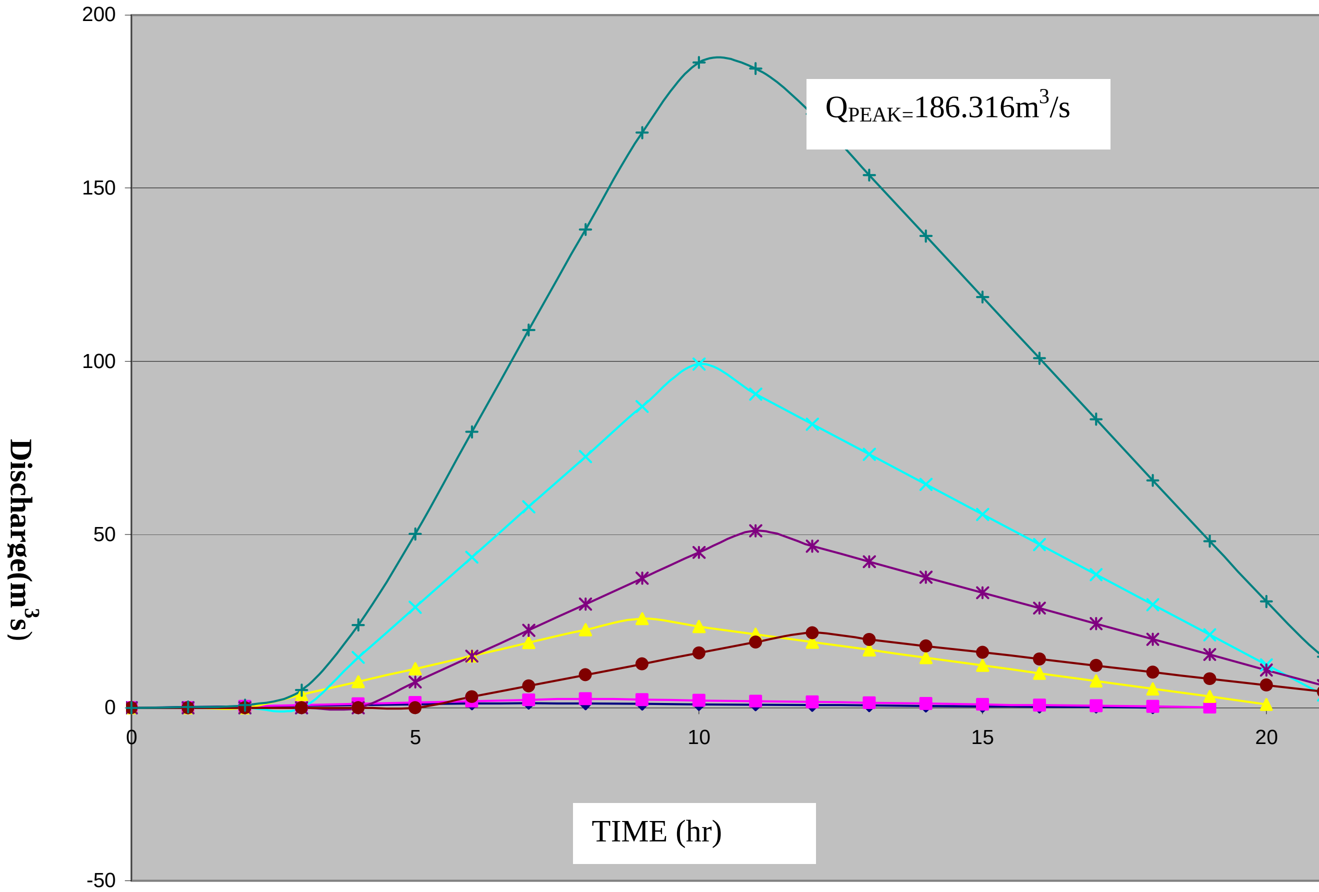


Fig . COMPLEX HYDROGRAPH

4 WATER DEMAND ASSESEMENT

4.1. General

Water requirement may be defined as the quantity of water regardless of its source, required by a crop or diversified patter of crops in a given period of time for its Normal growth under field condition at a place., It includes the losses due to evapo transpiration (Et) or consumptive use(cu) plus the losses during of irrigation water (un avoid able losses) and the quantity of water required for special operation such as land preparation, transplanting, leaching etc.

It may be defined as

CWR= ET or Cu + Application losses +special needs

Where: (Michael P537)

CWR= crop water requirement

Cu= Consumptive use

ET= evaporation spiriting

Application losses are the amount of water required to replenish the soil moisture deficit bock to field capacity (F.C) in the root- Zone of a crop cannot be applied exactly by means of irrigation. Some losses of water is, therefore, un avoidable under field conditions, these unavoidable losses are called application losses.

Special needs: Depending up on the field conditions and soil characteristics extra water is needed to meet purposes like Leaching of excess salts, pudding-preplanting irrigation etc.

4.2. **CROP WATER REQUIREMENT (CWR)**

The crop water requirement is the total quantity of water that a crop required right from the time of sowing to maturity continuously. The rate of use of water is not, however same for all crops. The rate of use varies with the kind of crop grown, time taken by the crop to mature, the temperature, and weather conditions.

The water requirement of crop is obtained by multiplying the potential evapotranspiration (ETO) with crop coefficient (kc) which will be discussed later. Potential evapotranspiration is the evapo transpiration from large vegetation of short height covering Land Surface with adequate moisture at all time and the crop characteristics on crop water requirement.

Evapotranspiration of the crop (Etc) is estimated using the equation

$$E_{tc} = Kc*ET_O$$

Where ETO= reference crop evapotranspiration

Kc= Crop Coefficient

The growth period of the crop is divided in to four stages

(a) Initial Stage: germination and early growth when the soil surface is not or is hardly covered by the crop (ground cover less than 10%)

(b) Crop development stage: From end of initial stage to attainment of effective full ground cover.(Ground covers approximately 70 to 80%).

(c) Mid-Season stage: from attainment of effective full ground cover to start of maturity.
(d) Late Season stage: from end of midseason state until full maturity of harvest. For more in formation see annex B, table –

4.3 CROP SELECTION AND CROPPING Pattern

4.3.1 <u>Crop Selection</u>:

To select the type of Crops grown in a certain area the following few points have to be considered.

- Adaptability to Climate
- Socio-economic aspect
- Adaptability to Irrigation
- Marketing value of crop (priority should be given to those with higher market value).
- Suitability for soil: The selected crop should give a maximum possible yield with little or no additional Inputs, which will increase the soil nutritive value and provide adequate drainage facility during high impoundment.

Based on the above criterion, maize, sorghum, Haricot Bean sweet potato, pepper, Tomato, Cabbage,& onion were selected the project area.

Cropping Pattern

Cropping pattern is the sequence of different crops growing in a regular order in any particular field. To determine the Irrigation requirement of the project area, an assessment should be made for different crops grown under irrigation, more over information about the crop characteristics such as length of growth cycle, rooting depth, crop factor etc should be collected.

In furfuro irrigation project there are commands of 150 ha. The proposed command areas have a soil type ranging from moderate to heavy textured black soils, which are grouped under vertisol.

The cropping pattern is calculated based on the type of soil (vertisol) by dividing the full year in to two seasons dry and wet seasons.

For calculating cropping pattern we need different data with respect to the given crops such as crop coefficient (Kc). Crop yield reductions factor (Ky) growing stage of crop and other data. Most of these data are taken from FAO books but there are same crops which are not stated in FAO books for instance, from the crop that are Selected for our commands Haricot Bean, Irish potato are not found in FAO However, what is done is that, we take the data of bean Dry for Haricot Been and potato for Irish potato. Because Haricot Bean and Bean dry are classified on the same group and the same reason for Iris potato.

- Table- 4.1 Proposed cropping pattern- in wet season

SI. No	crop	Area(%)	Planting date	Harvesting date	duration
1	maize	30	5/6	18/10	135
2	sorghum	20	15/4	18/8	125
3	Haricot Bean	20	5/7	23/10	110
4	S. Potato	20	5/6	13/10	130
5	pepper	10	5/8	13/12	130

- Table-4.2 proposed cropping pattern – in dry season

SI. No	crop	Area(%)	Planting date	Harvesting date	duration
1	maize	35	15/12	29/4	135
2	Tomato	10	20/2	14/5	145
3	Cabbage	10	15/1	25/4	100
4	Pepper	15	27/8	4/1	130
5	Haricot Bean	20	12/2	1/4	110
6	Onion	10	25/11	23/6	210

4.4. **Reference crop evapotranspiration (ETo**)

Reference crop evapotranspiration is defined as the rate at which water, if available, would be removed from the soil and plant surface of specific crop, arbitrarily called the reference crop. The reference crop adopted was actively growing grass clipped to a height of 8 to 15 cm and completely shading the soil.

Owing to the difficulty in obtaining accurate direct measurement of pan evaporation under field conditions, evapo transpiration is often predicted on the basis of climatology data. The approaches followed are to relate the magnitude and variation of evapotranspiration to one or more climatic factors (temperature, day length , humidity, sunshine, wind etc.)

However, the accuracy of such estimates depends primarily on the availability of the methods being used to represent the physical laws governing the process and the accuracy of the metrological and cropping data.

FAO 24 recommends using the combination equation methods for areas where measurement of temperature, humidity, wind and sunshine hour(duration) or radiation are available.

Estimation of ETO for the area of fururo was based on the penman-monteith combination equation with aerodynamic and surface resistance terms. It allows the calculation of evapotranspiration from metrological variables and resistance which are related to the stomata and aerodynamic characteristics of the crop .

To estimate the potential evapotranspiration for fufuro irrigation project the metrological and climatic data of Alaba Kolito. Which has the same latitude and longitude to that of Walberg metrological station found in furfuro command are used .

The widely used method to estimate ETO are as follows:

1. Blaney criddle method
2. Radiation method
3. pan evaporation method
4. Modified Penman method
5. pen man- monteith method

Blaney –Criddle Method

The Blaney–Criddle formula has generally given sufficient accurate estimates of seasonal consumptive use owing to the inclusion of locally developed crop coefficient factor (K) Basically, however, it assumes that the consumptive use of water is dependent only on temperature and day length which is not fully true. Crop water requirements have been found to vary widely between climates having similar air temperature but different humidity and wind conditions.

This equation is stated as:

E_{TO}= 0.46*P*C (t+17.4)

Where:

P= mean daily percentage of total annual day time hours obtained from Table

t= mean daily temperature in ^{0}C over the month considered.

c=Reference crop evapotranspiration in mm/day on minimum relative humidity sunshine hours and day time wind estimates .

The Radiation Method:

Is recommended in the absence of detail data on humidity and wind speed and the formula is used particularly under humid conditions (FAO 24)

The equation can be stated as follows

E_{TO} = C (W*Rs)

Where:

E_{TO} = reference crop evapotranspiration in mm/day for the period considered.

R_s = solar radiation in equivalent evaporation in mm/day.

W = Weighing factor which depends on temperature and altitude.

C = adjustment factor which depends on mean humidity and day time wind Conditions.

The pan-evaporation Method:

Is not considered accurate due to the extra-energy that is received from the side of the pan, the higher vapor pressure deficit and air temperature moreover, the data are often affected by in adequate maintenance of the pan, algae growth etc.

The equation stated as

$$E_{TO} = K_{pan} * E_{pan}$$

Where:

E_{TO} =Reference crop evapotranspiration

The modified-penman:

Is the most widely used method as it considers too many parameters for the calculation of E_{TO}. But it does not take into consideration the crop resistance and also over estimate the potential E_{To} by more than 25% (principle of Hydrology, DELFT,1992).

The equation is stated as:

$$E_{To} = C\ [W.Rn + (1-W).f(u).(ea-ed)]$$

Where :

E_{TO}= reference crop evapotranspiration mm/day

W= temperature related weighting factor

Rn= net Radiation in equivalent evapotranspiration in mm/day

F(u) = Wind Related function

(ea-ed) = difference between the saturation vapor pressure at mean air temperature and the mean actual vapor pressure of the air, both in mbar.

C= adjustment factor to compensate for the effect of day and night Weather Condition.

The penman- monteith equation:

Becomes the new standard for estimating potential E_{TO} according to 1990 FAO s meeting. The method is suitable to directly estimate the potential ET if the crop resistance is known (Principle of Hydrology , DELFT ,1992). And this approach is proved to be superior in Lysimeter experiments (JONSENET,AL 1990). That gives close result with the actual value .

By considering all the above reasons we have adopted for penman- monteith method for calculating of ETo by crop wat 4 windows version 4.3 computer program for the Alaba kolito station. The result is shown on table below .

Table 4.3 ETo calculated by crop wat- 4 computer program.

Data Source: C:\ETHIOPIA\ALABA-KO.PEN

Country : Ethiopia Station : ALABA KOLITO

Altitude: 1850 meter(s) above M.S.L.

Latitude: 7.22 Deg. (North) Longitude: 38.06 Deg. (East)

Month	MaxTemp	MiniTemp	Humidity	Wind Spd.	SunShine	Solar Rad.	ETo
	(deg.C)	(deg.C)	(%)	(Km/d)	(Hours)	(MJ/m2/d)	(mm/d)
January	27.6	10.8	66.0	78.0	8.5	20.4	3.81
February	28.1	12.1	62.0	78.0	8.6	21.8	4.17
March	27.7	12.3	67.0	78.0	9.0	23.3	4.43
April	27.0	12.9	70.0	78.0	8.4	22.5	4.28
May	26.9	12.0	84.0	78.0	7.9	21.0	3.94
June	25.3	12.3	82.0	95.0	6.6	18.6	3.53
July	23.5	12.8	83.0	52.0	5.2	16.7	3.06
August	23.8	12.6	83.0	35.0	5.3	17.3	3.15
September	25.1	12.5	84.0	69.0	5.1	17.2	3.29
October	26.6	10.5	82.0	69.0	8.5	21.8	3.94
November	27.1	9.2	69.0	86.0	9.3	21.8	3.97
December	27.8	7.9	66.0	86.0	8.8	20.4	3.80

Average	26.4	11.5	74.8	73.5	7.6	20.2
3.78						

➔ Table 4.3 ETO calculated by cropwat-4 Computer program.

Manual Calculation of ETo by penman – monteith method.

- Month:- February
- Latitude :- 7:22N.L
- Elevation :- 1850 meter
- Mean monthly temp :- 20.1 ^oC
- Mean Relative humidity:- 62.0%
- Mean Observed sunshine hrs:- 8.6hrs
- Wind Velocity at 2m heights:- 18.0 $^{km}/d$
- Nature of the surface : close –ground green crop

PET or ETO can be calculated using penman's equation, incorporating some modifications suggested by other investigation is given by:-

$$PET = \frac{A.Hn + E_a r}{A + r}$$

Where:

PET: Potential evapotranspiration

A = Stop of the saturation vapor pressure Vs temprature curve at the mean air temperature .

Hn= Net incoming solar radiation of energy, expressed in mm of evaporable water per day .

Ea = A parameter including wind velocity and saturation deficit in $^{mm}/day$:

r = Psychosomatic constant
= 0.49 mm of Hg/^{O}c

Then H_n is given by

$$Hn = Hc\,(1-r)\left[a + b\frac{n}{N}\right] - \delta T_a^{\,4}(0.56 - 0.092\sqrt{ea}\,) * \left[0.10 + 0.9\frac{n}{N}\right]$$

Where:-

Hc:- mean incident solar radiation at he top of the atmosphere on the horizontal surface, expressed in mm of evaporable water per day.

- r = Reflection coefficient (Aledo) of the given area .
- a = a constant depending on Latitude = 0.29 cost
- b = a constant having an average value = 0.52
- n = actual duration of bright sunshine in hrs.
- N = Maximum possible hours of bright sunshine (mean value)

- F = Stefan – Beltzman constant
  $= 2.01*10^{-9}$ mm/day
- Ta= mean air temperature.
  $= 273+^{o}C$
- C_a = actual mean Vapor pressure in the air in mm of H_g The parameter Ea
  is defined as

The parameter Ea is defined as.

$$Ea = 0.35\left[1+\frac{V_2}{160}\right]*(es-ea)mm/day$$

Where:-

V_2 = mean wind velocity at 2m above the ground in km/day
es = saturation vapor pressure at mean air temperature in mm of Hg .
(from annex A table 1:6)
ea = actual mean vapor pressure of air in mm of Hg .

With the help of the above equation, and using the value of A, e ,r, hc and N from Annex A table 1:6, 1:6, 1:3 ,1:4 ,1:5 respectively, PET or ETo can be determined for the given area.

Solution.

Using penman's equation .

$$ETo\ or\ Cu = \frac{AH_n + E_a r}{A+r}$$

From Annex <u>A</u> table <u>1.6</u>
A = 1.06 mm/^{0}C (by interpolation from the table)
Es = 17.66 mm of Hg (by interpolation from the table)
From annex A table 1.4
Hc = 14.21mm of water /day
From Annex A table 1.5
N= 11.9 hrs

$\therefore \frac{n}{N} = \frac{8.6}{11.9} = 0.72$ From given data
ea = R.H*es = 0.62*17.66 = 10.95 mm of Hg.
a = 0.29cos 7.22 = 0.29
b = 0.52
δ $= 2.01*10^{-9}$ mm/day
Ta= 273+20.1 = 293.1K
$\delta\ T_a^4 = 2.01*10\text{-}9*(293.1)^4$
= <u>14.83</u>

$r = albedo\ for\ close\ ground\ crop\ as\ \ 0.5$

$H_n = 14.21(1 - 0.25)[0.29 + 0.52*0.72] - 14.83\left(0.56 - 0.092\sqrt{10.95}\right)*[0.10 + 0.9*0.72]$

$7.08 - 2.83$

$4.25 mm/day$

$$Ea = 0.35\left[1 + \frac{V2}{160}\right](es - ea)\ mm/day$$

$$= 0.35\left[1 + \frac{78}{160}\right]*(17.66 - 10.95)$$

$= 3.49$

$r = 0.49$

$$PET = ETo = \frac{1.06*4.25 + 3.49*0.49}{1.06 + 0.49}$$

$$= 4.01 mm/day$$

4.5. <u>Crop coefficient (Kc)</u>.

The crop coefficient is used to relate the potential evapotranspiration (ETo) to the evapotranspiration of crop (Etc).

Information required on the crop is ‘

- The data of sowing
- the length of the total growing season including duration of initial stage.
  - the duration of the development stage ‘
  - the duration of the mid – season stage.
  - the duration of the late – season stage. .

☛ Climatic data required for the selection of kc values are wind speed and humidity.

The steps needed to arrive at the kc values for different growing stages are as follows.

1. Establish planting or sowing data from local information or from practices in similar climate zones.
2. Determine total growing seasons and lengths of crop development stages from Local information or from literature.
3. Kc for initial stage predicts irrigation and /or rain fall frequency for predetermined ETo value. Kc value may be selected form table 18 for known humidity and wind speed values (FAO 33) and plotted.
4. Kc for mid season: - for given climate (humidity and wind) select Kc value from table 21 or from table 18 and plot as straight line.
5. Late season stage: - for time of full maturity (or harvest with in few days, select Kc Values from table as above. Assume straight line between kc values at the end of midseason period and at end of growing season.

6. Development stage: - Assume straight line between Kc values at end of initial to start of mid season stage.

The actual kc value when plotted gives a smooth curve but is made a straight line for practical use, in the absence of available climate data such as humidity and wind direction application for procedures listed above seem difficult, so adopt a higher kc value from the larger so that water stress don't occur in the growing period Kc Values are on the annex B. ………..

4.6. **Net Irrigation Requirement (NIR)**

The net irrigation requirement is the depth of irrigation water, exclusive of precipitation, carry-over soil moisture or ground water contribution or other gains in soil moisture, that is required consumptively for crop production. It is the amount of irrigation water required to bring the soil moisture level in the effective root zone to field capacity. Thus, it is the difference between the field capacity and the soil moisture content

There fore:-

$$NIR = ET_{crop} – Peff – Gw - Sw + Ps + LR$$

Where:

ETc = crop evapotranspiration (mm)
Peff = Effective rainfall (mm)
Ps = Pre sowing irrigation (mm)
GW = Ground Water contribution (mm)
SW = soil water contribution and the other (mm)
LR = Leaching Requirement (mm)

Considering the existing situation for our project work, the above equation for net irrigation requirement can be reduced to the following.

- $NIR = ET_{crop} - P_{eff}$

This is due to:

☛ A depth below the root zone, the capillary properties of the soil, determines the contribution from the ground water table and soil water content in the root zone, therefore very detailed experiment will be required to determine the ground water contribution under field condition. In addition, among the selected crops, there are no any crops, which require pre sawing irrigation. Hence, both ground water contribution and pre sawing irrigation are neglected for the reasons stated.

☛ In our project, we do not have any information about ground water contribution.
☛ Snow on the soil surface contains approximately 1cm of water per 100 cm snow. (Source proj.paper, Design of irrigation system suing sille river. 2000)
Generally, winter rains, melting snow or flooding may cause the soil profile to be near at the field capacity at the start of the growing season, which may be equivalent to one full irrigation. In addition, some water may be left from the previous irrigation.

There fore this might be considered in the scheduling stage. However, it could be neglected in the planning stage as it is difficult to estimate and its contribution is generally significant (Sahsrabudhe, 1994).

☛ Leaching requirement is the minimum amount of irrigation water supplied that must be drained through the root zone to control soil salinity at the specific Level. Leaching can be practice during, before or after the crop season depending on available water supply, but provided that salt accumulation in soil doesn't exceed the crop tolerance level (FAO 24).

Since we cannot find the data for calculating the leaching requirement, we assume that percolation of water during Irrigation application would satisfy the leaching requirement

4.7 Calculation of Effective Rainfall (P_{eff})

It is that portion of precipitation, which has fallen during the base period of a crop and is available to meet the evapo-transpiration need of the crop. In a broad sense, effective rainfall is the portion of rainfall that has not lost by surface run-off or through deep percolation below the root zone. On certain problematic soils, rainwater is used in leaching of salts and accounts for p_{eff}.

Generally, rainfall effectiveness increases with high ET rates, greater allowed soil moisture depletions, and large soil water storage capacities.

The following different methods are used for calculation of effective rainfall.

1. **Fixed percentage rainfall**.

Effective rainfall is calculated according to

$$P_{eff} = a.P_{tot}$$

Where: -

a = is a fixed percentage to be given by the user to account for losses from runoff and deep percolation.

P_{tot} = the average total rainfall in ^{mm}/month.

Normally losses are accounted 10% to 30% , thus a lies between 0.7 and 0.9

An average value of 0.8 is adopted in the project.

Sample Calculation.

$$\begin{aligned} Peff &= a * P_{tot} \\ &= 0.8*32 \\ &= \underline{25.6\ ^{mm}/month} \end{aligned}$$

2. <u>Dependable rainfall</u>

Based on an analysis carried out for different Arid and sub humid climates an empirical formula developed by FAO to estimate dependable rainfall, the combined effect of dependable rainfall (80% probability of exceedence and estimated losses due to runoff and percolation. This formula may be used for design purpose where 80% probability of exceedence is required.

$P_{eff} = 0.6*P_{tot} - 10$ For $P_{tot} < 70mm$

$P_{eff} = 0.8*P_{tot} - 24$ for $P_{tot} > 70mm$

Sample Calculation

For June.

$P_{eff} = 0.6*64-10$

$= 28.4mm/month$

<u>For April</u>.

Peff = 0.8*124-24

= 72.2mm/month.

3. **<u>USDA soil conservation Service (USBR) method</u>** .

In this case effective rainfall is calculated according to the following relations given as per USNBR.

$$Pcff = Ptot\left[\frac{125-0.2*Ptot}{125}\right] --------- for\ Ptot < 250mm$$

$$Peff = 125 + 0.1Ptot ----------------- for\ Ptot. > 250mm$$

sample calcuation

for July

$$Peff = 131\left[\frac{125-0.2*131}{125}\right] = 103.5mm/menth$$

4. <u>Empirical formula</u>

The parameters involved in this relation are determined from analysis of local climatic records. Analysis of Local climatic records may allow an estimation of effective rainfall. The relation ship in most eases simplified by the following equation.

P_{eff} = a Ptot +b ------------------------ for P_{tot} < Zmm
P_{eff} = c. P_{tot}+d …………………… for P_{tot} > Zmm

Value for a, b, c, d and z are correlation coefficients. Adaptation of this empirical relation needs predetermined values of the coefficients from a long-term rainfall of a given area. But according to the crop wat for window program,

a = 0.5 b = -5

C = 0.7 d= -15

z = 50m

Sample calculation for October, P_{+0+} < Z_{mm}

P_{eff} = 0.5+Ptot + (-5)

= 0.5* 48+ (-5)

= 19mm/month

For August

P_{eff} = 0.7*P_{+0+} + (-15)

= 0.7* 128+ (-15)

= 76.6 mm/month

Adaptation of this empirical relation needs a predetermined value of the coefficient from a long-term rainfall of a given area. Hence, this method is not easily adopted.

Table 4.4

Comparison of effective rainfall by different Methods.

Station= Alaba kolito.

Month	Ptot (mm/month)	Effective rainfall (mm/month)			
		Fixed percent age	Dependable	USBR	Empirical formula
Jan	32	25.6	9.2	30.4	11.0

Feb	89	71.2	32.96	76.3	47.3
Mar	98	78.4	38.72	82.6	53.6
Apr	124	99.2	55.36	99.4	71.8
May	107	85.6	44.48	88.7	59.9
June	64	51.2	28.4	57.4	29.8
July	131	104.8	59.84	103.5	76.7
Aug	128	102.4	57.92	101.8	74.6
Sep	112	89.6	47.68	91.9	63.4
Oct	48	38.4	18.8	44.3	19
Nov	32	25.6	9.2	30.4	11.0
Dec	5	4	-	5.0	-
Total	970.0	776	402.56	811.7	518.1

We adopted the effective rainfall calculated by using USBR. Methods. This is because USBR method gives conservative value of effective rainfall data. In addition to this, it estimates P_{eff} based on the intensity, i.e as high rainfall more will be runoff and less will reach the root zone .

4.8. **Irrigation efficiency:**

It is essential that efficient use of Irrigation. Water is made, but even in the best method of irrigation because of various losses of water, the entire quantity of water applied during irrigation is not available for use. (dr.LaL and Punimia). The design of the irrigation system, the degree of land preparation, and the skill and care of the irrigator is the principal factors influencing irrigation efficiency. Loss of irrigation water occurs in the conveyance and Distribution system, non-uniform distribution of water over the field, percolation below crop root zone, and with sprinkler irrigation evaporation from the spray and retention of water on the foliage. In case of large fields loss may occur by run off at the end of irrigation borders and furrows. (A.M Michael 1997)

To account for losses of water incurred during conveyance and application to the field, an efficient factor should be included when calculating the project irrigation requirements. Project efficiency is normally subdivided in to three stages, each of which is affected by a different set of conditions.

1. **Conveyance efficiency (E.c)**

It is the ratio between water received at wet to a block of field and that released at the project headwork.

2. **Field canal efficiency (Eb)**

It is the ratio between water directly available to the crop and that received at the field inlet.

3. **field application efficiency (Ea)**

It is the ratio between water directly available to the crop and that received at the field inlet.

4. **Project efficiency (EP)**

It is the ratio between water made directly available to the crop and that release at head work, or

$$E_p = E_a * E_b * E_c$$

Conveyance and field canal efficiency are sometimes combined as distribution efficiency (Ed).

$$Ed = Ec*Eb$$

Field canal and application efficiency are same times combined as farm efficiency (Ef).

$$Ef = Eb * Ea$$

The following values have been derived from experience of system under operation and are considered suitable for use in the design of conveyance network considering the level of maintenance and management that will be available. (FAO 24).

Table of Irrigation efficiency

Conveyance Efficiency (Ec). ………………………………………………… ICID/ILR

- ❖ Continuous supply with no substantial change in flow……………………… 0.9
- ❖ Rotational supply in project of 3000 – 7000 has rotational
  Areas of 70-300 ha, with effective management………………………………. 0.8
- ❖ Rotational supply in large schemes (>10,000 ha)and small schemes (<1000 ha) with respective problematic communication and less effective management!-
  Based on predetermined schedule --- 0.7
  Based on advance request --- 0.65

Field canal efficiency (Eb)

- ❖ Block larger than 20 ha:- Unlined -- 0.8

Source:- (Alternate design of Birr Irrigation project paper, 2003 page 53)

4.9. Gross Irrigation Requirement (GIR)

The total amount of water applied through irrigation is termed as "gross irrigation water". In other words it is net irrigation requirement plus loss in water application and other losses.

The gross irrigation requirement can be determined for field, for a farm, for an outlet, command area or for an irrigation project, depending on the need, by considering the appropriate losses at various stages of the crop.

CIR= NIR/EP

Where:-

NIR= Net irrigation Requirement

EP = Project efficiency.

Other than for meeting the Net irrigation requirement (NIR) , water is need for Leaching accumulated salts from the root zone and to compensate for water losses during conveyance and Application.

Leaching Requirement (LR) and irrigation application efficiency (Ea) are included as or fraction of the NIR. From label ------------ our peak demand.

Q_{peak} = 76.5 ℓ/s

= 0.0765 m^3/sec

$E_p = E_a * E_c * E_b$

Where:- E_a = application efficiency

E_c = Conveyance efficiency

E_b = field canal efficiency

But application efficiency (Ea) is already taken in to consideration during the calculation of crop water requirement (CWR). In Cropwat 4..3 version program.

- By taking $E_c = 0.8$

  $E_b = 0.75$

  $E_p = 0.8*0.75 = 0.6$

Gross peak demand

$$= \frac{0.7565 \text{ m2/5}}{0.6} = 0.1275 \text{ m}^3\text{/sec}$$

4.10. Irrigation Scheduling.

Irrigation scheduling is fixing of depth, interval and time of Irrigation In our Project the irrigation scheduling calculation is bossed on crop wat 4 window programming

Irrigation interval (I)

The irrigation interval is the length of time avowable between successive irrigations during the peak consumptive use of the crop. It is calculated as:-

$$I = \frac{P*Sa*D}{ETc}$$

Where:- I = irrigation interval, days

P= fraction of available soil water

Sa= total available soil water, mm/m

D= rooting depth, m

E_{tc} = evapotranspiration of the crop, mm/day

Irrigation depth (dg)

It is the depth of irrigation water applied to the root zone per each irrigation application and it is given by the formula.

$$D_g = \frac{P*Sa*D}{Ea}$$

Where:- d_g = depth of irrigation , mm

E_a = application efficiency, fraction

P,Sa, D= are as defined above .

The irrigation schedule of selected crop has been calculated using cropwat 4 windows as mentioned above and presented in Annex B.

5. IRRIGATION STRUCTURE (HEAD WORK DESIGN)

5.1 GENERAL

Water is essential for human consumption and sanitation, for the production of many industrial goods, and for the production of food and fiber. Water is an important means of transport in many parts of the world and a significant factor in recreation, crops are being grown as a source of fuel such as methanol and this will place an in erasing demand for irrigation water. Even a valuable resource can be a hazard & excessive water – floods –causes sustantial damage & loss of life through out the world.

Water is unequally distributed about the earth, and its availability at any place varies greatly with time. The problem of water resources miss management & fast population growth heave long been recognized and concerned international and national authorities are attempting to bring them under controls and hope one day to be successful.

In most developing countries, especially sub-Saharan Africa experts and engineers of irrigation observed that farmers are suffering from shortage of water not because of its unavailability but mainly because of the lack of skills to make it available at the time & place required.

It was observed that farmers used river water for irrigation when it flows full of its bank, It is frequently happen once a river's water level drops to a little below its banks, farmer become desperate for emigration water & their crops fails.

Even though many rivers in our country are seasonal but have adequate discharges to develop small emigration schemes. Since farmers lack either or both of knowledge or finance to be able to divert the water to agricultural lands, the river flow is not used. Some farmers do construct temporary diversion weir in places, from free franks and stones, but these are not reliable & have to rebuilt offer each high floods.

In recent years it is realized that an effective method to tackle the problem of food shortage is by helping small-scale irrigation schemes & rehabilitant the traditional ones. In many cases, if farmers could be helped to be control the water level during the

seasons, by constructing simple but reliable diversion weirs, the food shortage and resulting hardship could be solved.

The diversion weirs, although apparently & simple structure across river, engineering work which needs careful design & through hydrological, hydraulic & structural analysis

Functionally, the diversion head work can be defined as structures which are constructed at the head of the main canal to raise water level in river and divert the required quantity of silt free water in to the canal (RAYK , LINSELY,JOSPH,B. FRANZIN,AND ROZGAR. BABAN).

5.2 <u>Site Investigations:</u>

The following are some essential preliminary investigations, which must be carried out before design of the diversion headwork commences, thus tried to be investigated

5.3 <u>Loction of the head work:.</u>

Initially, it is difficult to decide on the location of the proposed structure with out having topographic map of the project area and layout of the river course.

- By walking along a river up and d/s of the location where the existing intake or alluvial soil.

- Or where the farmers are live it is an appropriate location.

5.4 Soil investigation:

For the proposed locations of head work, soil investigation has been carried out by the geologists / soil technician, In the 1st site visit & when a few locations are proposed for the structure, the engineer should also visually test the soil & describe its physical properties. The purpose of this preliminary investigation is to recommend the type of soils or rock type of tools to be used and test required to be carried out.

Selection of type of the weir

- After the foundation soils are found to be Verti soil (alluvial soil). On this type of soil either vertical drop weirs or sloping glacis weirs can be constructed.
- In deciding the shape of the weir important factors need to be considered: -
    - Practicality (i.e. Simplicity for construction
    - Economy of the structure
    - The skill and experience of the workers available

Generally two types are very common is small-scale irrigation schemes. These are: -

1. Concrete weir with vertical u/s sloped d/s faces
2. Stepped weir, which is usually constructed from gabion boxes

There fore the stability problem of the structures which arises due to earth quake area of the project since it is in the rift vaguely system of vertical drop concrete weir vertical u/s shaped d/s face is selected. (Rozner baban 19)

5.5 Design Consideration of the weir

The behavior of the surface flow is greatly affected by the geometry of the weir and the geometry of the weir directly affects the design and economy of the structure. Some important features of the weir & their effect on the design are given below

Components of weir:

- Weir wall
- u/s and d/s cutoff
- under sluice

- u/s and d/s apron
- head regulator
    - Crest elevation
    - Length of the weir and
    - Shape of the weir

5.5.1 Hydrological Data

- Hydrological data is needed in the design of the weir to deduce from its analysis a few significant figures. The analysis is carried out is detail under the hydrology section.

From hydrology section the design discharge Qd = 186.32 m^3/sec

5.5.2 Weir Design

The following data's are required for weir design

I. Design discharge Qd = 186.32m^3/sec (From hydrology)
II River bed laved = RBL = 1890m (from top mop)
III. Lacey silt factor f = 1.0, assume (Arora)

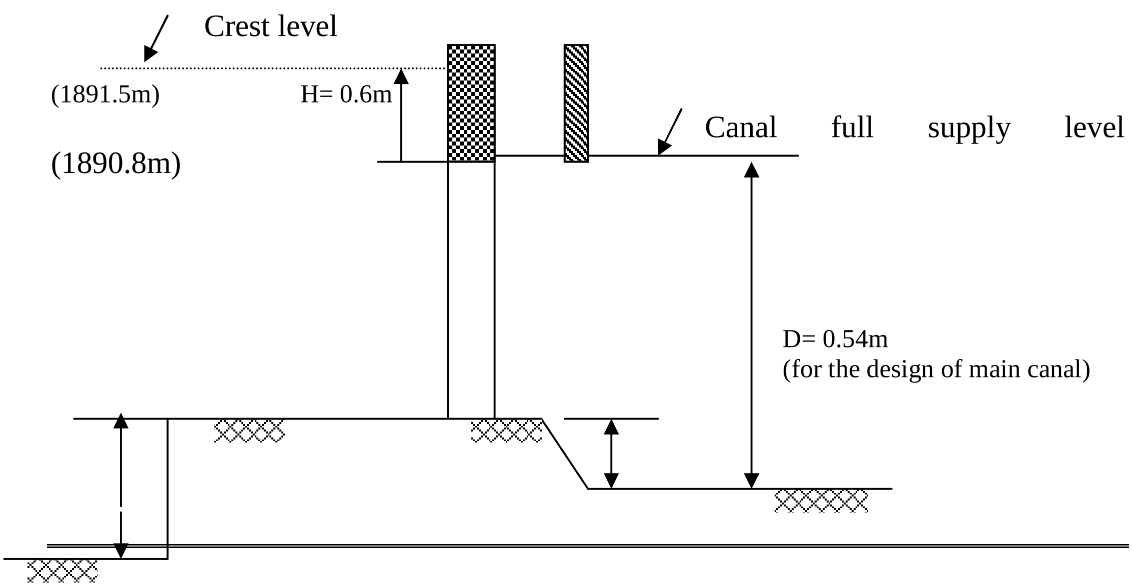
5.5.3 **Hydraulic design**

I) Fixation of the weir crest level

Since the crest elevation of the weir affects the water profile in the following two ways.

1. Height of the crest affects the discharge coefficient & consequently the water shade above the weir & the backwater curve.
2. The height of the weir affect the shape & location of the jump and the design basin. The height of the weir and in turn the crest level is decided with the requirement of the canals intakes in mind. (Rozger Baban)

The crest level of the weir will be



d = 0.15m

P= 0.5m

River bed loved = 1890m

Where:

H = head loss through the regulator for (0.5 to 1.0m) - (ARORA)

P = Intake canal height from river bed level (0.5-1.5m) (modi and Baban)

D = main candy depression from canal intake (0.15-0.25) (humor Gripe)

-pond level = canal full supply + Head loss through regulator
= 1890.8 + 0.6 = 1891.5m

Since no shutter is provided over the crest level of the weir fixing crest level

At the pond level.

Crest level of the weir = pond level = 1891.5m

Weir height = crest level of the weir – River bed level =1891.5 – 1890.0 = 1.5m

For weirs with out shutter

Head of the ponded water = weir height = 1.5m

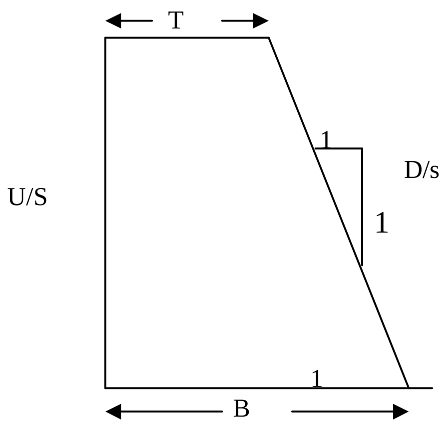
u/s TEL =crest level +Hd

II) <u>Design of weir wall</u>

- Since the selected weir is a vertical chop concrete weir of vertical u/s face & the sloping down stream face (with slope 1;1) . The weir wall has trapezoidal cross. Section

A/ <u>Top width (T)</u>

The max of the following value should be considered :



1. $from\, stress\, criterion of\ the\, weir$

$T = d/\sqrt{rc}\ \ where$

$= d = u/s\, HFL - crest\, leval$

$= \dfrac{1.364}{\sqrt{2.4}} \qquad = 1892.864 - 1891.5 = 1.364m$

$= 0.88m \qquad = specitic\, gravity\, of\ the\, matarial\ \ for\, concrete\, r = 2.4$

2. $from\, sliding\, considerations$

$T = \dfrac{d}{rc} * \dfrac{1}{r} \qquad where$

$r = \dfrac{2}{3} = or\, 0.7$

$T = \dfrac{1.364}{2.4} * \dfrac{1}{0.7} = 0.82m$

3. $considaringg\ Hd = total\, head\ 0ver\, the\, weir\, crest\ \left(1.42m\right)$

$$T = \frac{Hd}{\sqrt{rc-1}} = \frac{1.42}{\sqrt{2.4-1}} = 1.20m$$

$This,\ faving\, the\, mox =, top\, wir\, dth\, of\ the\, weir\, wial\, be$

$T = 1.20m$

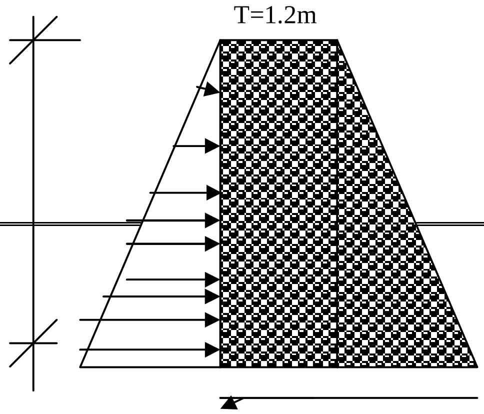
B) **Bottom Width**

The bottom width of the weir should be sufficient so that the maxim compressive stresses are with in the allowable limits is with in the allowable limit and the tension does not develop.

Determined by the following approaches & the max of them should be considered.

<u>Condition (I) (No flow condition)</u> when the heed water on the up stream is at the top of weir wall & there no flow .

The over turning moment at the base due to the water pressure is given by



H=1.5m

B

$$Mo = \frac{rwHs^3}{6} \quad but$$

$$Hs = total\ stafic\ head$$

$$= \frac{(1)(1.5)^3}{6} \qquad Hs = h + hs, h = 1.5, hs = 0$$

$$= 0.5625 kN/m = \qquad = 1.5m \qquad rw = 1000N/m = 1kN/m$$

The resisting moment is due to the weight of concrete and the weight of water on the u/s slope. It the u/s face is vertical the moment of resistance in given by

$$Mr = \frac{r_w h r_c}{6}\left(B^2 + TB - T^2\right) \qquad where$$

$$= \frac{1*1.5x2.4}{6}\left(B^2 + 1.2B - 1.44\right) \qquad T = top\ width\ of\ the\ weir\ (1.2m)$$

$$= 0.6\left(B^2 + 1.2B - 1.44\right) \qquad r_C = specific\ gravity\ of\ concrete\ r_c = 2.4$$

$$equating\ equotion\ [Mo = M\ r]\ we\ have\ Mo = Mr$$

$$0.5625 = 0.6\left(B^2 + 1.2B - 1.44\right)$$

$$B^2 + 1.2B - 2.3775 = 0$$

$$solving\ the\ quadratic\ equation\ we\ have\ B = 1.06m$$

Condition (II) (High flood condition) , the weir submerged when the flood is passing over the crest & the weir is submerged .
The over turning moment is obtained from the water pressure

$$Mo = \frac{r_w h a}{2} = \frac{1.0*(1.5)}{2}*1.364 = \qquad a = 1.42 - 0.056$$

$$= 1.5345 KN - m \qquad = 1.364m$$

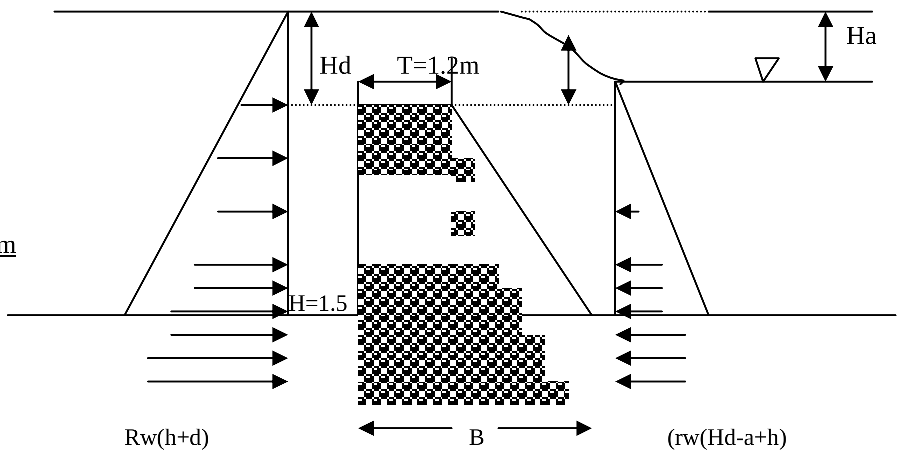
Resisting moment , with fail water at weir crest level considering full floatation

$$Mr=\frac{r_w h}{6}(rc-1)\left(B^2+TB-T^2\right)=\frac{1*1.5}{6}(2.4-1)\left(B^2+1.2B-1.44\right)$$
$$Mr=0.35\left(B^2+1.2B-1.44\right)$$
$$\textit{equating equation}\left[Mo=Mr\right]\textit{we have}$$
$$1.5345=0.35\left(B^2+1.2B-1.44\right)$$
$$\Rightarrow B+1.2B-5.824=0$$
$$\textit{solving the quadratic equation} \quad B=1.886=1.89m$$

H_a

0.056

1.364m

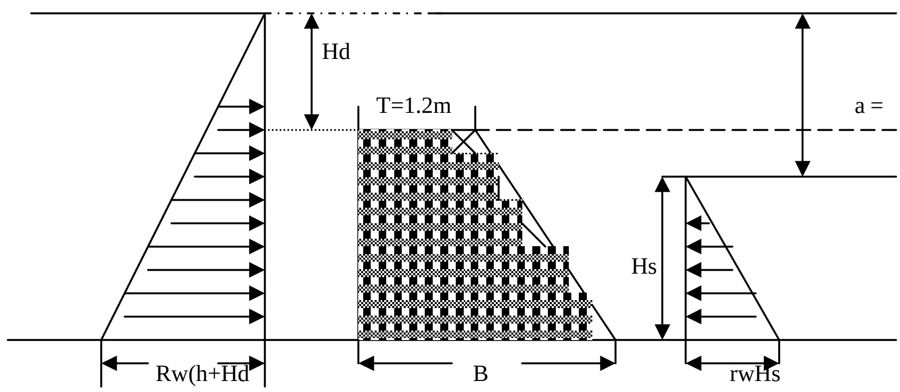


A= Hd-

= 1.42-

=

Condition III (high flood condition) the weir dies changing free. If the fail water remains below the crest level churning floods, the weir dis charge free.



III) Design of Impervious floor

$Hs = 0.41(from\, hydroulic\ jump\, calculetion)$

$Hs = h(K) \Rightarrow \left(\frac{Hs}{h}\right) = k \qquad k = is\, cons\tan t, whose\, volve\, can\, be\, \det er \min ed\ if\ the\, rives$

$k = \left(\frac{0.41}{1.5}\right) = 0.075 \qquad dis\, change\, at\ amone\, depth\, d\ is\, knon$

$over\ forning\, moment$

$$o = \frac{rwh}{6}(1 + 2k\,3/2) = \frac{1*1.5}{6}(1 + 2*0.075)$$

$$= 0.5855$$

$The\, resisting\, moment$

$$r = \frac{rwh}{6}(rc - 1)(B + TB - T)$$

$$= 0.42(B + 1.2B - 1.44)$$

$soheing\, o = r = 1.3940 = B + 1.2B - 1.44$

$;- \ B = 1.86m \approx 1.2m$

$condition(IV)\, mning\, the\ formula$

$$B = \frac{Hel + weir\, haipat}{\sqrt{rc - 1}} = \frac{1.42 + 1.5}{\sqrt{2.4 - 1}} = 2.4678m$$

$$\approx 2.5m$$

$Taking\, the\, bottom\, width\, weil\, be\, B = 3.0m$

- For under seepage the worst condition occurs when the water on the upstream is at the pond leusl & there is no tail water

The maximum seepage head

H_S = pond level –D/s bedrails offer introgression

= 1891.5 – (1890-0.5) Assuming

= 2.0m retrogression 0.5m (AROVA)

Depth of sheet pile

The sheet piles at u/s & d/s ands of impervious floor are provide;-

RL of bottom of u/s sheet pile = u/s HFL – 1.5*R

= 1892.864 – 1.5*2.73 (3)

Depth of u/s sleet pile = 1.7m 2.0m (d_1)

RL of bottoms d/s sheet pile = d/s HFL offer retrogression - 2R

Assuming Afflux = 1.0 (ARORA) = (1891.36 –2*3) = 1885.36m
D/s TEL = U/S TEL – Afflux
= 1891.92 * Depth of d/s pile (d_r) = 1890 – 1885.36 = 4.6m

Total length of impervious floor

By Bight's theory the total creep length (L) is given by
L= CH where C = Blights coefficient for the type of the soil at the sife (moderate to heavy textual soil C = 10 (Aroma Rood)
L = 2*10 = 20.0m

i) length of d/s aspersions floor
The length of d/s caparisons floor (LD) in determined by the following formula (por a weir with out shutter)
The following formula (For a weir with out shutter

ii) Length of u/s cipervious floor (Lu)

$$Ld = 2.21C\sqrt{Hs/10 = 2.21*10\sqrt{\frac{2.0}{10}}} = 9.88$$

$$= 10m$$

$$ii)\ length\ of\ u/s\ impervious\ floor\,(lu)$$

$$Lu = b - (ld - B) \qquad where$$

$$= 6.8 - (10 - 3) \qquad b = L - 2(d + d)$$

$$= 0 \qquad = 20 - 2(2 + 4.6) = 6.8m$$

$$Lu = 0 \qquad B = bottom\ width\ of\ the\ weir$$

$$B = 3.0m$$

How ever , a length of 1m with 0.6m nominal this cues coercions floor may be re commended four safety. There fore the footlight of impervious floor would be 16m and the total crag length wowed be <u>27.2m</u>

For the satiety of the hydraulic structures on pervious foundation the following two criteria should be satisfied

I) The sub-soil hydraulic gradient showld be less than permissible vaue to prevant piping failure
II) The floor should be sufficiently frich to prevent its venture due to up lift pressure
  - pipe failure accorder to Blight will not cower if the hydraulic gradient is equal for less than a late value.

$$i \le \frac{1}{C} \quad where \quad i = \frac{Hs}{L} \qquad hydraulic\ gradient$$

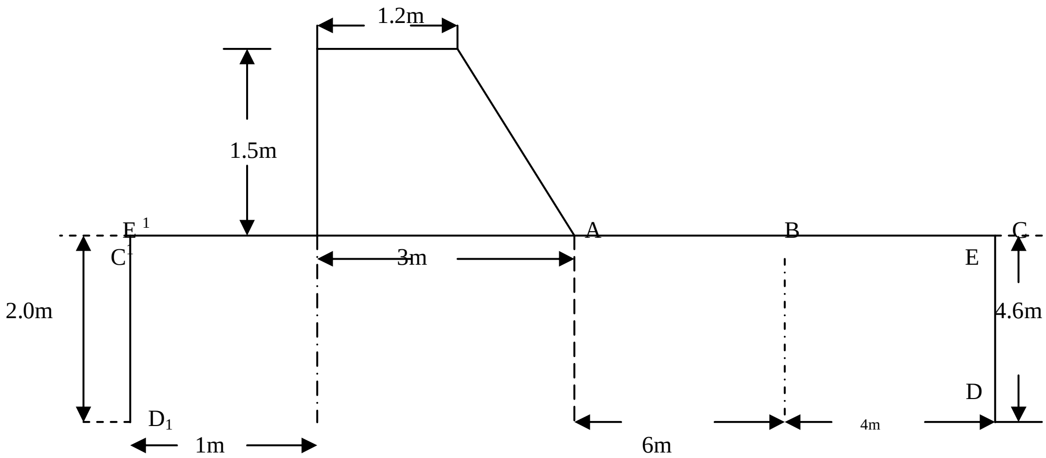
$$Hs = seepage \quad head$$

$$L = Creep \quad Length$$

$$i = \frac{Hs}{L} = \frac{2}{27.2} \le \frac{1}{10}$$

$$= \frac{1}{13.6} \le \frac{1}{9} (safe)$$

According to Bligh's , the uplift pressure and floor thickness at different pt's are calculated as



Uplift pressure head

At point A = hA= 2-((2*2)+4)*1/13.6 =1.412m
At point B= hB =-2-((2*2)+4+6)*1/13.6=0.970
At point C=hC=2-((2*2)+14)*1/13.6=0.676

Check Uplift pressure head at point C

1/13.6*(2*4.6) =.6767

Thickness of the floor at each points ($\gamma_{c=2.24}$)

At point A = t A=4/3*(h A/($\gamma_c - 1$)) =1.6m

At point B = t B=4/3(hB/(γ_c -1)) =1.0m

At point C= tC=4/3(Hc/(γc -1))=0.8m

$For\ u/s\ cut\ off\ d, 3m\quad h = 1.5m\ \&\ b = 10m$

$$\propto = \frac{b}{d} = \frac{10}{3} = 3.33$$

$Pe = 1.5m$ $$\lambda = \frac{1+\sqrt{1+\alpha}}{2} = \frac{1+\sqrt{1+(3.33)}}{2} = 2.24$$

$$\phi\ \frac{H}{\amalg}\cos-1\left(\frac{1-\lambda}{\lambda}\right)\quad \frac{1.5}{\amalg}\cos-1\left(\frac{1-2.24}{2.24}\right) = \Rightarrow 1.03m$$

$$\phi = \frac{1.03}{1.5}*100 = 68.67\% \approx 69\%$$

$$\phi = \frac{H}{\amalg}\cos-1\left(\frac{2-\lambda}{\lambda}\right) = \frac{1.5}{\amalg}\cos-1\left(\frac{2-2.24}{2.24}\right) = \Rightarrow 0.80m$$

$$Pc = \frac{0.8}{1.5}*100 = 53.33\% \approx 53\%$$

$for\ d/s\ cut\ off\ d = 4m\qquad b = 10m\qquad h = 1.5m$

$$\alpha = \frac{b}{d} = \frac{10}{4} = 2.5m\qquad \lambda = \frac{1+\sqrt{1+\alpha}}{2}$$

$$= \frac{1+\sqrt{1+2.5}}{2} = 1.85$$

$$\frac{H}{\amalg}\cos-1\left(\frac{\lambda-2}{\lambda}\right)$$

$$= \frac{1.5}{\amalg}\cos-1\left(\frac{1.85-2}{1.85}\right) = 0.788 \approx 0.8$$

$$Pe = \frac{0.8}{1.5}*100 = 53\%$$

$$\phi = \frac{H}{\amalg}\cos-1\left(\frac{\lambda-1}{\lambda}\right) = \frac{1.5}{\amalg}\cos-1\left(\frac{1.85-1}{1.85}\right) = 0.522m$$

$$Pd = \frac{0.522*100}{1.5} = 34.8\% \approx 35\%$$

corretion for interference

$D = 3m,\qquad D = 4m\qquad b = b = 10ma\ \ at\ the\ D/s$

$$C = 19\sqrt{\frac{D}{b}}\left(\frac{d+D}{b}\right) = 19\sqrt{\frac{4}{12}}\left(\frac{3+4}{12}\right) = 6.39 \approx 6.4\%$$

$d = 4m\qquad D = 3m = b = b = 10m\ at\ u/s$

$$C = -19\sqrt{\frac{D}{b}}\left(\frac{d+D}{b}\right) = 19\sqrt{\frac{3}{12}}\left(\frac{3+4}{12}\right) = 5.54\%(-)$$

correction for floor thickness

correction for floor thickness for ϕc

correction $$\frac{\phi D-\phi c}{d}*t = \frac{69-53*0.6}{3} = 3.2\%(+)$$

$check\ thckness\ of\ floor\ by\ Khosla's\ theory$

$l)\ Exit\ gradient, GE$

$Total\ length\ of\ inprevious\ floor, b = 16m$

$depth\ of\ olls\ sheet\ pile, d_2 = 4.6m$

$$\propto = \frac{16}{4.6} = 3.478 \qquad \lambda = \frac{1+\sqrt{1+x^2}}{2} = \frac{1+\sqrt{1+3.478^2}}{2} = 2.3/m$$

$$Gs = \frac{Hs}{d_2} * \frac{1}{\amalg\sqrt{\lambda}} = \frac{2.0}{4.6} * \frac{1}{\amalg * \sqrt{2.31}}$$

$$G_E = \frac{1}{10.98} \qquad but\ GE\ for\ \text{mod}\,erte\ texture\ soil\ for\ the\ project\ area$$

$$\frac{1}{10-98} < \frac{1}{5}(safe) \qquad = 1/5$$

$ii)\ uplift\ pressure$

$correction\ for\ floor\ thickness\ for\ \phi c$

$$correction \qquad \frac{\phi e - \phi c}{d} * t \qquad \frac{53-35}{4} * 0.6 = 2.7\%\ (-)$$

$corrected\ \phi c = 53 + 6.4 + 3.2 = 62.6\%$

$$\Rightarrow Pc = \frac{\phi c}{100} * Hs = \frac{62.6}{100} * 1.5 = 0.939m \qquad \approx 1.0m$$

$corrected\ \phi e = 53 - 5.54 - 2.55 = 44.91\%$

$$correced\ \ PE = \frac{\phi e}{100} * Hs = \frac{44.91}{100} * 1.5 = 0.674m \qquad \approx 0.7m$$

$pressure\ at\ different\ po\text{int}$

$$P\ \ at\ \ the\ \ toe = Pe + \frac{Pc - Pe}{b} * b$$

$$= 0.7 + \frac{1-0.7}{10} * 7 = 0.91m$$

5.6. **Design of protection work**

In order to safe guard further, the impervious floor against the failure due to piping protection work have to provided at both u/s and D/c end of impervious floor . these are inverted filter lock protection and launching apron.

► Black protection & launching apron are provided in u/s end of impervious floor where as inverted filter & launching apron are provided in d/s end.

D/S protection work.

- By Bligh's the combined length of the d/s impervious floor and protection work (ℓ) is give as.

$$\ell = 18C\sqrt{\left(\frac{H_s}{10}\right)\left(\frac{q}{75}\right)}$$

$$= 18*10\sqrt{\left(\frac{2}{10}\right)\left(\frac{2.87}{75}\right)}$$

$$= 15.75 \approx 16m$$

<u>Therefore the length (lp) of d/s protection is given by</u>

ℓp = ℓ - ℓd ⟹ 16-10 = <u>6m</u>

i) ⟹ Length of inverted filter = 1.5 *d_2 (d/s sheet pile)
= 1.5* 4.6 = 6.9m = 7.0m

Provide with 1m thickness stone or concrete block on 0.5m thick graded.

ii) Minimum Length of d/s Launching apron
= 2.5*d_2 =2.5*4.6 = 115m= <u>12.0m</u>
Provide with <u>1.5m</u> thickness.

D/s protection work

i) Concrete block

Minimum length of block protection
= Depth of u/s sheet pile = D_1*1 = 2.0m Provide with from thick stone or concrete block laid on 0.5 thick loosely packed stone.

ii) minimum length of u/s launching apron .
= 2*d_1=2*2 = 4m
Provide with 1.5 thickness.

5.7 <u>Flow through sluice gates.</u>

Design Criteria

The function of the weir sluice bay is to prevent high sediment loads entering the off take canal . There is no standard technical criteria for measuring the dimensions of the sluice up on which engineers and investigator have agreed (Rozgar.B)

Scouring sluice

- Scoring sluices are gate controlled openings Provided in the weir with crest at a low level. It is located on the same sides of the off – take canal.

The sluices are used to :-

- Remove or scoured the silt, which is deposited in the front of the head regulator.
- To lower the highest flood level by providing greater discharge per meter length than the weir

► Discharges Capacity of the sluices provided in higher of the following.

i) Two times the max = discharge in the off taking canal
Max discharge in the off taking canal.
capacity of sluice = 2*0.780m^3/ses =1.56m^3/see

ii) 10-20% of the max flood discharges .
Qd= 186.32 m^3/see
➔ sluice capacity = 0.1*186.32 = 18.632m^3/see

iii) Max$^{\underline{m}}$ wintedr discharge
➔0.22m^3/see

There fore the discharges capacity of the sluices 18.63m^3/see .

► In order to have adequate removal of silt, a minimum velocity of 2 to 4.5 m/sec should be maintained through the tunnels.

A high velocity of 4 to 4.5 m/sec is adopted for boulders rivers (This is suitable for our project area.) (ARORA)

There for the cross-section area (A) can be calculated as

A=Q/V= 18.63 m^2/sec ?

► considering the height (H) of the tunnel is equal to weir height, the width of the sluice will be:

B =A/h =4.14/1.5 =2.76m

Hence, provide 1.5mx2.76m sluice on both side of the weir near the off-taking canal

5.8 STABILITY ANALYSIS’S

for Structure to remain stable, the moment, which tends to topple it must be equal to the moment which balance it. Usually a safety factory of about 1.5 to 2.0 is applied (Baban,1995)

$$\frac{\sum M\,balance}{\sum M\,topple} > 1.5$$

- In order to avoid lifting up of the structure at the heel and occurrence of tension at the base, the forces must pass through the middle third of the structure base

$$i.e\ \ eccentricity\, e < B/6\ \ or\ \ e = \left|\frac{B}{2} - x\right| < \frac{B}{6}$$

$$where\ \ x = \frac{\sum M}{\sum Vf}$$

$$where \qquad \sum M\ = sumation\, of\ \ all\ moments\ about\ the\ strucure\ toe.$$

$$\sum vf\ = sumation\ vertical forces\ excluding the base\ reaction$$

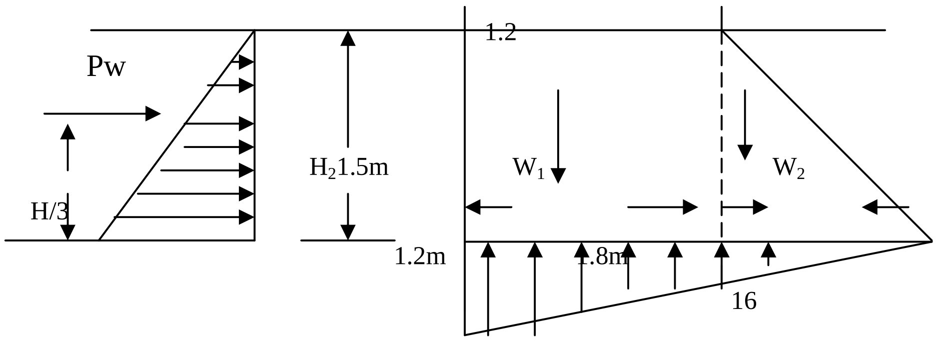
$$B = width\, of\ \ weir\ \ base$$

5.81 Critical cases to be considered

For weirs constructed non-monolithically with the foundation two general cases are considered for a stepped & a sloped face weir:-

- The base of the weir is not sealed i.e water seeps from the up stream side freely through the base to the down stream sid
- No water is flowing over the weir and no water ponding is in its down stream

i) The critical condition considered for stability analysis is the maximum pool level on the u/s and no water in the d/s side of the weir (static stability)



Since r_w= 1KN/m^3 , $r_{c =}$ 2.4 KN/m

(I) vertical forces

1. weight of concrete /masonry (W_s)

W_2 = W_1 = 2.4*1.5*1.2 = 4.32KN/m acting at moment arm = 2.4m

½*1.5*1.8*2.4 = 3.24KN/m acting at moment arm =1.2m

2.up lift pressure

u=1/2rw*H*B= ½*1*1.5*3=2.25KN/m acting at moment arm =2.0m

(II) Horizontal force (p_w=p_H)

P_w=1/2*1*1.5^2 = 1.125KN/m acting at moment arm = 1.5/3=0.5m

Check for stability

a) safety against over forming

i) Over turning moment, (M_o)

M_{o1}=p_w*0.5 = 1.125*0.5 =0.5625KN-m/m

M_{o2} = U*2 = 2.25*2 = 4.50KN-m/m

Σ M_o = M_{o1}+M_{o2} = 0.5025+4.50 = 5.0625 KN-m/m

ii)Resisting moment (M_r)

M_r =4.32*2.4+3.24*1.2 = 14.256KN-m/m

F.S = M_r/M_o = 14.256/5.0625 > 1.5

= 2.816>1.5 (ok!)

b) safety against sliding

$\Sigma H = p_w = 1.125KN/m$

The sum of vertical forces (Σv) = $w_1+w_2 = 4.32+3.2 = 7.52KN/m$

F.S = $\Sigma H/ \Sigma v < 0.65$

$=0.15 < 0.61$ hence (ok!)

Check for the structure may be slide in flow direction if there is no enough grip b/n base & the foundation. To prevent this happening the following condition should be fulfilled

where f= frictional factor (varsheny 1992) =(0.65)

$$\frac{Horizontal\ external\ force}{vertical\ external\ force} < f$$

c) Safety against tension

$X = M_n/ \Sigma_V$ where X = a distance from d/s and impervious floor which the resultant force pass

M_n = net moment

$M_n = M_r - M_o$

$= 14.256-5.0625$

$= 9.1935KN\text{-}m/m$

$X = 9.1935/7.52 = 1.22m$

Eccentricity, e = |B/2-X|

$=|3/2-1.22| = 0.28m$

For the structure to be safe against tension

$E<B/6 \Rightarrow e <3/6$

$\Rightarrow e <0.5$ but $e = 0.28<0.5$ (ok!)

<u>contact pressure</u>

$$P= \frac{\sum Vf}{B} + \frac{6e\sum Vf}{B^2}$$

$P_{max} = 7.52/3+6*0.28*7.52/3^2$

$=2.5067\pm1.4037=3.9104$

$<$ allowable bearing capacity of the soil assume($10KN/m^2$)

$$P_{min} = \frac{\Sigma vf}{B} - \frac{6e\,\Sigma vf}{B^2}$$

$$= \frac{7.52}{3} - \frac{6*0.28*7.52}{3^2}$$

= 1.1030 < allowable comp Stress of masonry's assume.

:. The above dimensions are adopted for the weir.

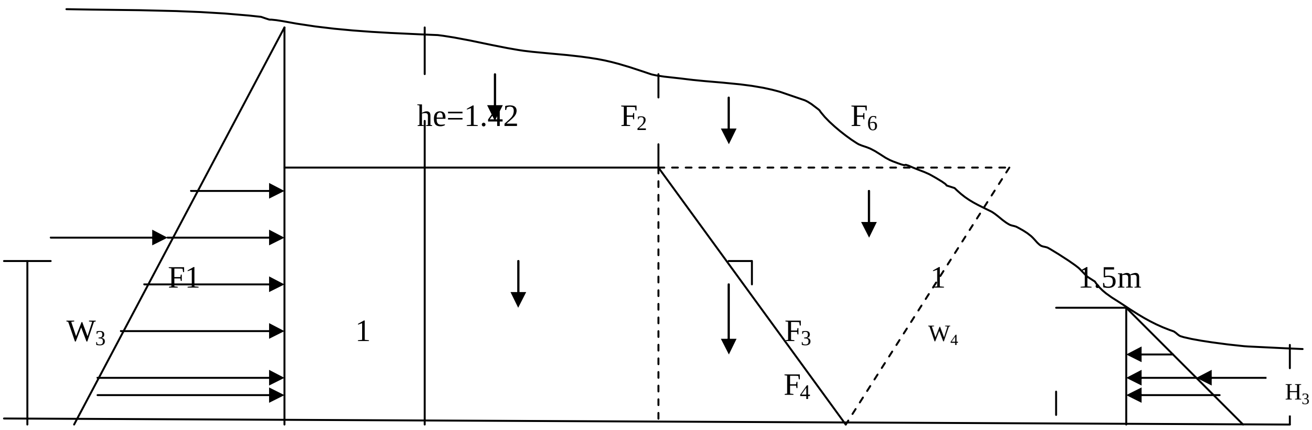
T = 1.20m

B = 3.0m

5.8.2.**Stability analysis of the weir (Dynamic stability)**

The critical loading condition for the stability analysis is the case when the weir is just submerged. The loading diagram will be;

☛ Considering per unit length of the weir axis



Hs=0.41m

Calculation of forces

$F_1 = ½ \quad \gamma w(1.5+1.42) = 4.263KN$

F2=1/2 $\gamma w(1.2*1.42) = 1.704KN$

F3= $\gamma c(1.2*1.5)*1 = 4.32KN$

F4=1/2 $\gamma c(1.8*1.8) = 3.88KN$

F5=1/2 $\gamma w(1.8*1.8) = 1.62KN$

F6=1/2 $\gamma w(1.42+1.5*1) = 1.065KN$

F7=1/2($\gamma w(0.41)^2 *1 = 0.1KN$

Summation of horizontal forms

1. $\Sigma H = F_1 - F_7$ = 4.263km – 0.1km = 4.163KN ≈ 4.2KN

Summation of vertical forces

2 $\Sigma V = F_2+F_3+F_4+F_5+F_6$

= 1.704+4.32+3.88+1.62+1.065 = 12.58KN

3 Inertia force (P amh)

Pamh = $\alpha hw = 0.1w$

where $\alpha h = 0.1(horizontal acceleration)$

= 0.1* (8.2)

W=weight due to gravity

=0.82KN

W=w3+w4=4.32+3.88=8.2KN

4. hydrodynamic load (p_e)

Pe = 0.555*$\alpha h\gamma_w H^2$, H= height of the water

= 1.5+1.42 = 2.92m

= 0.555*0.1*2.92 α_h = 0.1

= 0.162 KN

Pe is hydrodynamic load which acts at $4*H/3\Pi$ = 0.424H

= 0.424*2.92

= 1.239m

5) Force due to vertical acceleration (Pamv)

$P_{amv} = \alpha_v * w =$ 0.05w

= 0.05*8.2

where

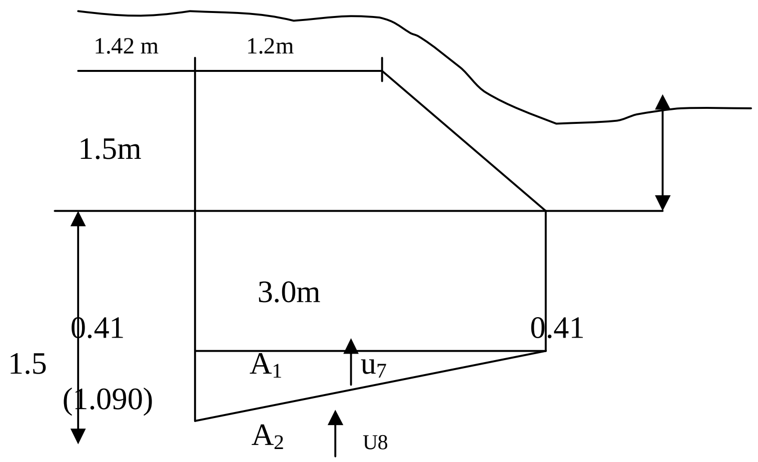
α_V = vertical acceleration =0.05

W = weight due to gravity

=8.2KN

6) Up lift Force

0.41



U_7= 0.41*0.3*10*1=1.23KN
Moment arm at the toe (=1.5m)
U_8 =0.5*1.09*3*1=1.64KN
Moment arm about the toe (2m)

7) silt pressure

$F^{st} = 0.18h^{2,}$ these is used for large hydraulic structure, but for small hydraulic structures like weir , barrages, dykes etc, the effect of silt pressure can be neglected. These is due to structures like weir a barrages can be provided with silt ejectors so that the effect due to silt pressure can be minimized (Arora)

Taking moment about the weir (toe of the weir)

Counter clock wise (+)
Clock wise (-)

Type of loads (Forces)	force in (KN/m)	moment arm (m)	moment about toe (KN-m/m)
vertical fore			
1) self weight			
F_3	4.32	2.4	+ 10.368
F_4	3.88	1.2	+ 4.656
2) Water load			
F_2	1.704	2.4	+ 4.090
F_5	1.62	0.6	+ 0.972

F_6 1.065 1.2 + 1.278

3) seismic load (sine it is fond is the rite valley)

$The\,sum\,of\,reristing\,marat(+Ve)$

$\sum Mr2 \quad 22.129Cm-m/n$

$The\,sun\,of\,over\,turring\,moments(-Ve)$

$\sum Mo=4.868m-m/n=9.78m-m/n$

$;-Factor\,of\,satety\,apainst\,overturnig=\sum Mr(+Ve)$

$\frac{22.129}{7.93}1.1=2.79\;1.5$

$sefety\,ogainst\,sliding\,in\,grile\,by=\frac{\sum H<0.75}{\sum V}$

$Fs=\frac{\sum H}{\sum V}=\frac{4.2cm}{12.589}0.334$

$*cheat\,for\,the\,strocture\,may\,be\,shieinin\,flow\,direetion\,hence\,if\,there\,is\,no\,eroghgrip\,b/n\,base\,and\,the\,foundation\,poprevnt\,OK$

$this\,happening\,the\,folloing\,conditions\,haual\,be$

$whare\,\underline{frictianal\,tactor(vorsteny,1992)}$

$F=0.65$

KNÅSðç NHN[kKð ðSkK[Ñk

$ß\;\frac{14.20}{12.589}=1.13m$

$Mn\;net\,momant$

$Mn=Mr-mo$

$22.129-7.93$

$=14.20Cm.m/n$

X_VW_3	2.4	+ 0.518	0.216
X_VW_4	1.2	+ 0.233	0.194

4) Horizontal

F_1	0.973	- 4.149	4.263
F_y	0.14	+ 0.014	0.1
XhW_3	0.750	- 0.324	0.432
XhW_4	0.50	- 0.194	0.388
Pe	1.239	- 0.201	0.162

Upsize pressure

U_7	1.5	- 1.85	1.23
U_8	2	- 3.27	1.64

$Eccentrircity=e=\left|\frac{B}{2}-X\right|$

$=\left|\frac{3}{2}-1.130\right|=0.37$

For stability aginst over turning for factor of safety should be greater than 1.5 (Arora)

$For\,the\,stroctire\,to\,be\,sate\,opainst\,tensis$

$e<\frac{B}{6}\quad e<\frac{3}{6}$

$e<0.5\,but\,e=0.37\;0.5(Ok)$

$contect\,pressur$

$p=\frac{\sum Vf}{B}+\frac{6e\sum Vf}{B}$

$Pmx=\frac{12.589}{3}+\frac{6x0.37x12.589}{3}$

$4.1963+3.1053=>7.3016Kc/m$

$<accowable\,beaing\,capauity\,of\,the\,soil$

$Assume$

$prnic=\frac{\sum Vf}{B}-\frac{6e\sum Vf}{B}$

$\frac{12.589}{3}-\frac{6x0.37x12.589}{3}$

$=4.1963-3.1053=1.0910kw$

$<accowable\,comp\,struss\,of\,masray$

$Assume$

$:-The\,above\,dimensions\,are\,edopted\,for\,the\,weir$

$T=1.20m$

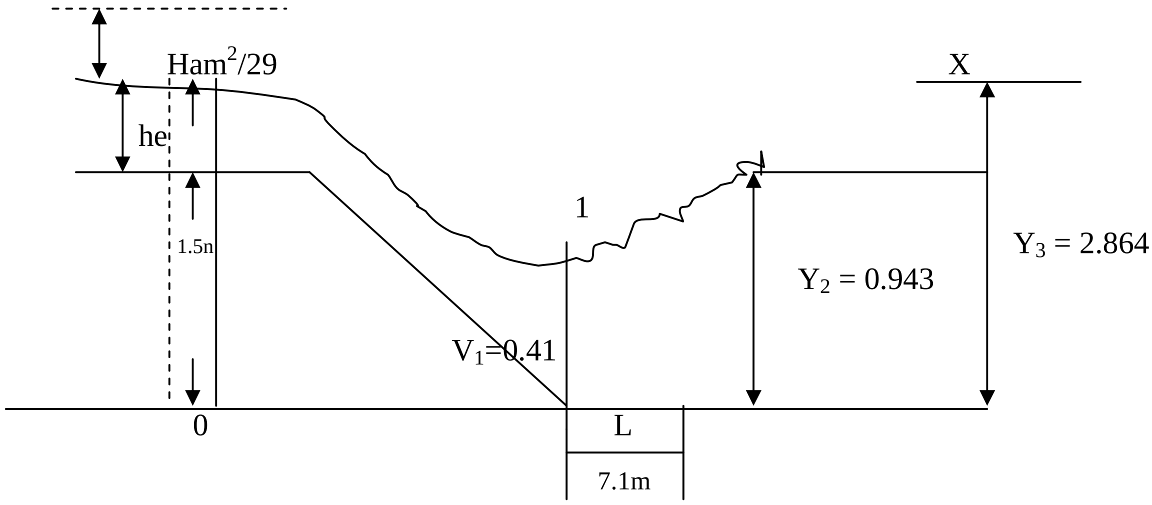
$B=3.0m$

5.9.Determination of hydraulic jump

Hydraulic jump is a sudden and turbulent rise of water, which occurs, in an open channel when the flow changes from the sup critical state to the sub critical state. It is accompanied by the formation of extremely turbulent roller and considerable dissipation energy.

Thus hydraulic jump is a very efficient means of dessipation energy below hydraulic structures like weir and spill ways.

Calculation of hydraulic jamp



Considering section b/n O&1 using Bernoulli's eqn.

$$H+he+ha = y_1 + \frac{V_1^2}{2g} + \text{head loss}$$ Given he = 1.42

Neglect the head loss on the weir & velocity heed at the u/s head

Using continuity egn. Q = AV

$$V = \frac{Q}{L*Y = (A)}$$ But Q/L =q

Therefore $V_1 = q/y$

q = 2.87 m^3/sec/m = from weir design

H+he+ha = $y_1 + V_1{}^2$ /2g+head loss

$$+ 1.42 = y_1 + * \frac{2.87}{} 2..872/$$

$$* 2*9.81 \quad 2.92 = y_1 + \frac{0.420}{y_1^2}$$

$$y_1^3 + 0.420 = 2.92 y_1^2$$

solving the equation trial and error we have

y_1= 0.41m

Critical depth (Y_c) = $y_C = \left(q^2 / g \right)^{1/3} = \left(\frac{2.87^2}{9.81} \right)^{1/3} = 0.943$

0.41 <0.943

Since Y_1 is less than the critical depth (it is ok)

$Y2 = \frac{Y1\left(-1+\sqrt{1+8F1}\right)}{2}$ But $F1 = V1/\sqrt{g*Y1}$

V1= g/Y1=2.87/0.41
=7m/s

$= \frac{0.41\left(-1+\sqrt{8*3.49}\right)}{2} = 1.829 \approx 1.83\,\text{m}$

Length of the jump (Lj)=5*(Y2-Y1)
=5*(1.83-0.41)
=7.1m

tail water depth (Y3)=HFL-river bed level
=1892.864-1890= 2.864m

Since the tail water depth after construction (2.864) is greeter than Y2. The tail water depth before constriction may be less then Y2.

If Y2.>Y3, the jump will recede d/s and its location depends on the slope and condition of the flow in the river. To avoid this condition dissipating structures in the apron should be made so that the jump will occur in the protected apron or the length of the d/s floor e should be such that hydraulic jump will be fully formed on it and energy is dissipated.

Length of hydraulic jump (Lj) = 5 (Y2-Y1)
= 5 (1.83 – 0.41)
= 7.1m

- The length of d/s impervious floor (LD = 10m) it is sufficient for the jump to occur and hence it is ok

5.10. <u>Design of Retaining weir</u>

This is a structure placed at some angle normal to the dissection of the file of the weir to retain the ends of approach and to support the leads pf the structure built

In order to prevent out flanking of water towards the main canal and safely over pass the discharge during high flood time .A gravity retaining wall is provided at both banks. In the design of Retaining wall the following points should be considered.

- The soil depth of the soil is up to the top level of the wall (Assumed)
- The wall face on the side of the soil is vertical.
- The soil is homogeneous (Assumed)
- The top width and bottom width of the wall is analysis by the timbrule (i.e. Assuming b/n the rule).

Data available :

- River bed level = 1890m
- Flood level at u/s = 1892.92m
- Pond level = 1891.5m
- Specific weight of masonry (γ_w) = 21KN/m^3
- Specific weight of masonry concrete (γ_c) = 2KN/m^3
- Angle of response (ϕ) = 30^0 (Assumed)
- Specific weight of soil = 19 kw/m^3 (γ_S)
- Assume the free board =0.6m
- As per Bligh, top width of the wall =60cm (Murty 1990)

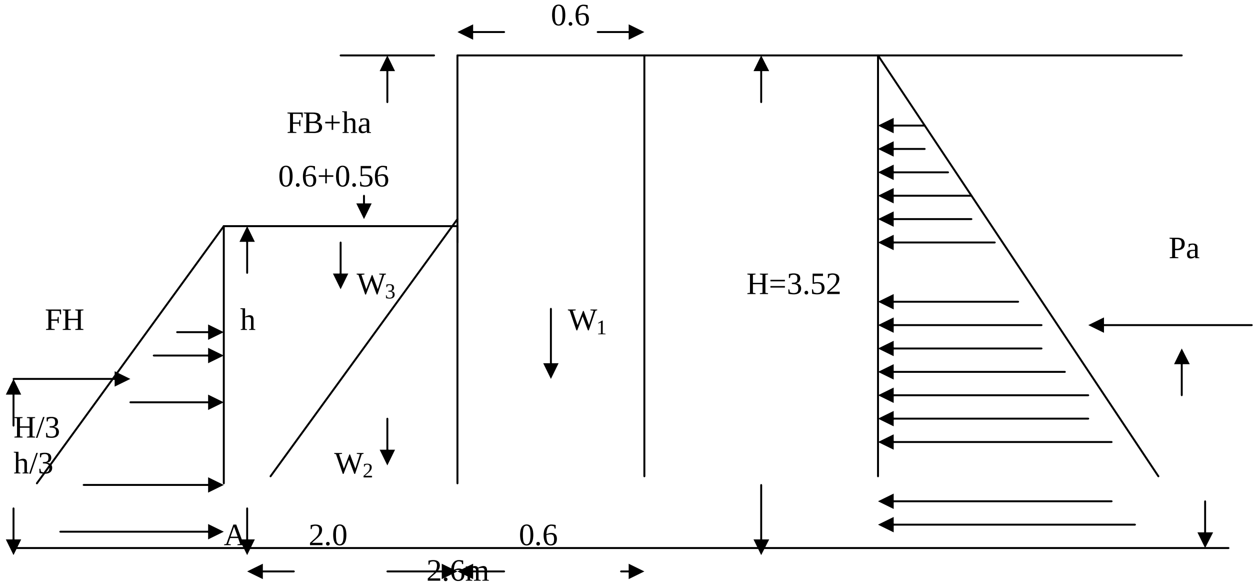


Fig. Cross-section of retaining wail

- Top level the wall=U/S HFL+FB where Fb=free board (Assume =0.6

  Top level =1892.92+0.6

  = <u>1893.52m</u>

  h=pond level –Bed level of the river

=1891.5-1890 =1.5m

H=1893.52-1890 =3.52 if it is very large or height of the weir

+FB+allow Forces Acting on the wall

ii) <u>horizontal force</u>

a) <u>Active earth pressure (Pa)</u>

The active earth pressure at depth (H). against a retaining wall is a homogeneous soil is

$P=K_a \gamma_s H$ H= taking the height of retaining wall

= height of the weir +FB+ allowance

- Represents a triangle stress distribution, the total force against the wall will be;

$P_a = \frac{K_e}{2} * \gamma_s H^2$ where P = active force

γ_s= specific weight of the soil.

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^0}{1+\sin 30^0} = \frac{1}{3}$$

$$P_a = \frac{1}{3} * 19KN/m^3 * \frac{3.52^2}{2}\left(m^2\right) = 39.24KN/m$$

P_a will act at a distance of 1.17m from the bed level of the river

b) <u>water pressure</u>

$$F_H \frac{1}{2}\gamma_w h^2 = \frac{1}{2} * 9.81 * 1.5^2 \qquad \gamma_w = specific\ weight\ of\ water$$

$$= 11.04KN/m \qquad \left(\gamma_w = 9.81kN/m^3\right)$$

F_H will act at a distance of 0.5m from the bed of the river

II. <u>Vertical Forces</u>

Weight of the wall, W_1=0.6*3.52*21KN/m^3=44.4KN/m

acts at (2.0+0.3=2.30m

$$W_2 = \frac{1}{2} * 2.0 * 1.5 * 21KN/m^3 = 31.50KN/m$$

$$= acts\ at \frac{2}{3} * \gamma = 1.33m$$

weight of the water on the sloping surface of the wail

$$w_3 = \frac{1}{2}\gamma_w * 2.0 * 1.5 * 9.81 = 14.72 kN/m$$

it is acting at a distance of $0.67m$ *from point A*

.

Stability analysis for retaining wall

- Taking all moment a bout point A
- Safety against over turning (F.S)
- Over turning moment (Mo)

Mo = Pa*1.17

= 39.24*1.17 = 45 .9 KN-m/m

Resisting moment (Mr)

Mr = FH*0.5 + w1 *2.3 + w_2 *1.33 + w_3 *0.67

= 11.04*0.5 +44.4*2.3 +31.50*1.33+14.72*0.67

= 5.52 +102.12+41.90+9.86 = 159.40 KN-m/m

$$F.s = \frac{Mr}{Mo} > 1.5 = \quad \frac{159.40}{45-91} = 3.53 > 1.5 \qquad (OK!)$$

$\Rightarrow$ *safety against sliding*

$$\sum H = 39.24 - 11 - 04 = \quad 28.20 kN/m$$

$$\sum V = W_1 + W_2 = \ 44.4 + 31.50 + 14.72 = 90.62 kN/m$$

$$\frac{\sum H}{\sum V} < \mu(f) = 0.65$$

$$= \frac{28.20}{90.62} \Rightarrow < 0.65 \ \Rightarrow 0.31 < 0.65 \qquad (OK!)$$

$\Rightarrow$ *safety against tension*

$$\bar{X} = \frac{Mn}{\sum V} = \qquad where\ Mn = Net\ moment$$

$$= 159.40 - 45.91 = 113.49 kN - m/m$$

$$\bar{X} = \frac{113.49}{90.62} = 1.25$$

$$Eccentricity, e = \left|\frac{B}{2} - \overline{X}\right|,$$

$$\left(B = \quad bed\ width\ of\ the\ wail\ B = 2.6\right)$$

$$e = \left|\frac{2.6}{2} - 1.25\right| = 0.05 \approx 0.1$$

$$= 0.1$$

for the structure to be safe against tension

$$e < \frac{B}{6} \qquad \Rightarrow 0.1 \leq \frac{2.6}{6} = 0.1 < 0.43 \quad (OK!)$$

$: -$ *adopt all the above* dim *ensions for u / s wail*

For d/s for portion, since the d/s HFL is below the u/s TEL it can be adopt e smaller height of Retaining

Wall compared to the u/s part besides that considering the topography of the areas.

6.0 CANAL DESIGN

6.1 General

To take away water from the canal head works such as weir, barrage, storage reservoir or storage dam to the field a well designed distribution system consisting of a network canal is required.

Based on the water requirements of the crops on the area to be irrigated the entire system of main canal, secondary canal, tertiary canal and field distributaries should be designed properly for a certain realistic value of peak discharge that must pass through them, so as to provide sufficient irrigation to the commands.

There are two main canals, which are located on the left side and on the right side of the river.

The design of canal is based on the Irrigation water Requirement. From low flow analysis the minimum flow of the furfuro stream is 220l/s which can satisfy our peak crop water requirement after having 30% of the base flow is released to the down stream ecosystem.

Factors that affect the canal alignment

While fixing the canal alignment the following factors should be kept in to consideration.

► A canal should be aligned on a water shed (ridge) as far as possible because it ensures irrigation on both sides of the canal and avoids cross drainage works.

► The main canal mounts the ridge is as smaller length as possible from the point of off take.

►The main canal should run straight even when the water shod makes a sharp loop.

► As for as possible, the canal should run through the center of the command area to keep the cost of distribution system to & minimum.

► The length of the canal should be as small as possible. The smaller the canal length, the less are the absorption and seepage lessed and the lower is the maintenance cost.

► The canal should cross the stream where it is straight and has a minimum Discharge.

From crop = Wat program (computer program) , We have a duty of = $0.3^{L}/s/_{ha}$, at application efficiency = 70%

$\Rightarrow Design capacity of the canal$

$= 0.3L/S/ha * 150ha * \frac{85}{100} * \frac{1}{0.8*0.75} * \frac{24}{12}$

$= 127.5 \, {}^{L}\!/\!{}_{s}$

$Where :- 85\% - Lntensity\ of crop$

$E_b = 75\%$

$E_c = 80\%$

$Total\ Aommand\ Area = 150ha$

$Assu\min g 12hr\ working\ hours.$

$\Rightarrow We have two main canals to the Right and Laft of the river.$

$\Rightarrow Main\ canal\ capacily = \frac{127.5}{2} = {}^{\ell}\!/\!{}_{s} = 63.75 \, {}^{\ell}\!/\!{}_{s}$

$This\ is\ becouse\ of\ equal\ command\ area\ on\ both\ sides\ (75ha\ each)$

$\Rightarrow Q = 0.06375 \, {}^{m3}\!/\!{}_{\sec}$

$say = 0.0638 \, {}^{m3}\!/\!{}_{\sec}$

6.2.1 $DESIGN\ OF\ RIGHT\ SIDE\ CANAL\ (Bilwanjacanal)$

$Main\ canal$

$- Lined$

$\rightarrow The lined\ main\ canal\ has\ a\ rec\tan gular\ cross - \sec tion$

$Given\ Data$

$Disch\arg e, Q = 0.0638 \, {}^{m3}\!/\!{}_{\sec}$

$But\ to\ be\ safe, the\ de\sin g\ disch\arg e, Qd\ inerealed\ by\ 20\%\ [Bharat\sin gh, 1988]$

$\Rightarrow Qd = 1.2 * 0.0638 \, {}^{m3}\!/\!{}_{\sec}$

$Qd = 0.075 \, {}^{m3}\!/\!{}_{\sec}$

$Bed\ stope = {}^{1}\!/\!{}_{1000}\ [\mathrm{Re} commended]$

$Roughness\ coefficient, n = 0.017\ [ARORA, 2000]$

$Permissible Velocity\ , V = 2.7 \, {}^{m}\!/\!{}_{\sec}\ [ARORA, 2000]$

$\Rightarrow$ *This Canal is Locted just ofter the intake structure.*

Area of flowm $, A = B * D$

Perimeter of flow $, R = B + 20$

HydraulicRadius $, R = A/P$

$$= m \frac{BD}{B + 20}$$

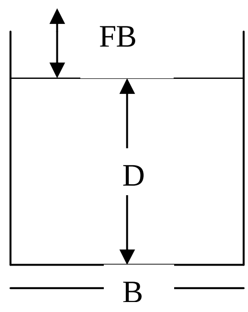


Fig. Cross – section of Rectangular section.

Mannigg's Eguation

$$V = \tfrac{1}{n} R^{2/3} S^{1/2}$$

$$= \tfrac{1}{0.017} * \left[\frac{BD}{B+2D}\right]^{2/3} \left[\tfrac{1}{1000}\right]^{1/2} \quad ....(*)$$

Continuity equation

$$Q = AV \quad ....(**)$$

$$V = Q/A \quad ....(***)$$

Equating equation $(*) \& (**)$

$$\Rightarrow \frac{0.075}{B*D} = \frac{1}{0.017} * \left[\frac{BD}{B+2D}\right]^{2/3} * \left[\tfrac{1}{1000}\right]^{1/2}$$

Assuming $\frac{B}{D}$ 2 [*for econonical section*]

$$\Rightarrow \frac{0.075}{2D} = \frac{1}{0.017} * \left[\frac{2D^2}{4D}\right]^{2/3} * \left[\tfrac{1}{1000}\right]^{1/2}$$

$\mapsto$ *Sorving for* $, D = 0.275$

$$= 0.28m$$

$$B = 0.56m$$

Check for Velocity, $V = \frac{Q}{A}$

$$= \frac{0.075}{(0.28*0.56)m^2} \; m^3/s$$

$$v = 0.478 \, m/\sec \angle 2.7 \, m/s$$

Ok!

$\Rightarrow B = 0.56m$

$D = 0.28m$

Free board

- It is the margin between the full supply level (FSL) of canal and the bank level.
- Lacey recommended the following formula for free board:-
- FB = 0.20 + 0.15* (Q) ⅓

Where = FB = free board

Q = discharge in m^3/sec

➔ FB = 0.20+ 0.15 (0.075)⅓

= 0.26m

Unlined

The unlined main canal is trapezoidal in cross section and can be designed by the following four methods depending up on the geologic and soil nature of the line where the canal lined.

i) Tractive force approach
ii) Based on Maximum permissible velocity
iii) Based on kennedy's theory
iv) Based on Lacey's theory

➔ The geology and soil type of the line that the canal going to be lied Punic rock and clay, clay loam, where these are characterized by the stable and water tight Nature, so that the design of unlined trapezoidal channel boundary surface can resist with out scouring.

- Based on Maximum permissible approach
- Given data:-

Q = 0.075 m^3 /sec

Bed slope = 1/1000

N= 0.025 (ARORA, 2000)

Side slope = 1:1

A= (B+D) D

$$P = B + 2\sqrt{2}\,D$$

$$R = A/P = \frac{(B+D)D}{B+2\sqrt{2}\,D}$$

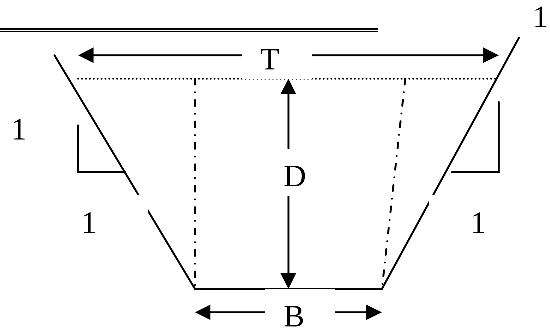


Fig. Cross – section of trapezoidal section. (Unlined)

$$u\sin g\ Manning\ 's\ equation$$

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

$$= \frac{1}{0.025} * \left[\frac{(B+D)}{B+2\sqrt{2}\ D} D\right]^{2/3} * [1/1000]^{1/2} \text{.......... ......} \qquad *$$

$$Continuity\ \ eqution$$

$$Q = AV$$

$$V = Q/A \text{.......... ....} \qquad (**)$$

$$equating\ \ equation\ (*)\ \&\ (**)$$

$$\Rightarrow \frac{0.075}{(B+D)D} = 1/0.025 * \left[\frac{(B+D)}{B+2\sqrt{2}\ D} D\right]^{2/3} * \left[\frac{1}{1000}\right]^{1/2} \text{.......... .} (***)$$

$$\Rightarrow\ for\ economical\ \ trapezoida\ l\sec tion$$

$$Top\ width\ = Two\ times\ the\ length\ of\ the\ Side$$

$$T = 2*\left(D\sqrt{2}\right)$$

$$\Rightarrow B + 2D = 2\sqrt{2}\ D$$

$$\Rightarrow B = 0.83D$$

$$insert\ the\ value\ of\ B\ in\ equation\ (***)$$

$$\frac{0.075}{1.83D^2} = \frac{1}{0.025} * \left[\frac{1.83D^2}{3.658D}\right]^{2/3} * \left(\frac{1}{1000}\right)^{1/2}$$

$$\Rightarrow D = 0.33m$$

$$\Rightarrow B = 0.274m$$

$$Check\ \ for\ velocity$$

$$V = Q/A = \frac{0.075\ m^3/S}{[0.274+0.33]*0.33}$$

$$= 0.376\ m/\sec$$

$$Take\ \ free\ board$$

$$FB = 0.26m$$

DESIGN OF LEFT SIDE MAIN CANAL (Temeda canal)

Lined

➔ The lined main canal has a rectangular cross-section and it starts just after the intake structure.

Given data

Qd = 0.075 m3/sec

$$Bed\ slope = 1/1000$$

$$n = 0.017$$

$$permissible\ Velocity, V = 2.7 m/\sec$$

$$Area\ of\ flow, A = B * D$$

$$Perimeter\ of\ flow, p = B + 2D$$

$$Hydraulic\ Radius, R = A/p$$

$$= \frac{BD}{B+2D}$$

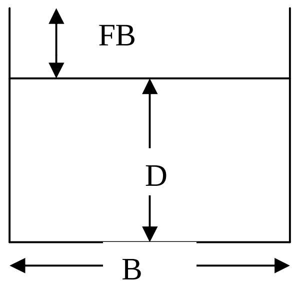


Fig. Cross – section of rectangular section.

$$Manning's\ Equation$$

$$V = 1/n * R^{2/3} S^{1/2}$$

$$= 1/0.017 * \left[\frac{BD}{B+2D}\right]^{2/3} * \left(1/1000\right)^{1/2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(*)$$

$$Continuity\ equation$$

$$Q = AV$$

$$V = Q/A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(**)$$

$$equating\ equation\ (*) \& (**)$$

$$\frac{0.075}{BD} = \frac{1}{0.017 * \left[\frac{BD}{B+2D}\right]^{2/3}} * \left(1/1000\right)^{1/2}$$

$$Assu\min g\ \frac{B}{D} = 2\ [for\ economical\ section]$$

$$B = 2D]$$

$$= 0.075 = 1/0.017 * \left[\frac{2D^2}{4D}\right]^{2/3} * \left(1/1000\right)^{1/2}$$

$$selving\ for\ D = 0.275$$

$$= 0.28m$$

$$B = 0.56m$$

Check for velocity

$$V = Q / A = \frac{0.075m^3 / s}{0.56 * 0.28}$$

$$- \; 0.478m / s < 2.7m / s \quad OK!$$

$$\Rightarrow B = 0.56m$$

$$D = 0.28m$$

$$Free\,boord.$$

$$FB = 0.28m$$

Unlined

The unlined main canal is trapezoidal in cross section and can be designed by the Maximum permissible Velocity.

Based on maximum permissible velocity.

Given data

$Q = 0.075 m^3 / \sec$

$Bed\ siope = 1/1000$

$n = 0.025 [ARORA, 2000]$

$side\ siope = 1:1$

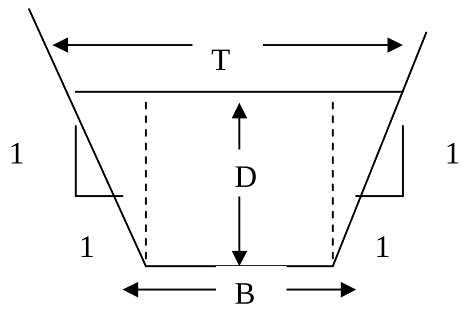


Fig. Cross – section of rectangular section.

$A = (B+D)D$

$P = B + 2\sqrt{2}\ D$

$R = A/P = \frac{(B+D)D}{B+2\sqrt{2}\ D}$

$U\sin g\ \ Manning's\, equation$

$V = 1/n\, R^{2/3} S^{1/2}$

$= 1/0.025 * \left[\frac{(B+D)*D}{B+2\sqrt{2}\ D}\right]^{2/3} * \left[1/1000\right]^{1/2}$ ....(*)

$Continuity\ equation$

$Q = AV$

$V = Q/A$...(**)

$equating\ equation\ (*)\ \&\ (**)$

$$\frac{0.075}{(B+D)D} = \frac{1}{0.025} * \left[\frac{(B+D)*D}{B+2\sqrt{2}\ D}\right]^{2/3} * \left[\frac{1}{1000}\right]^{1/2} \ldots\ldots\ldots\ldots\ldots\ldots(***)$$

$for\ economical\ trapezoidal\ section.$

$Top\ width = Two\ times\ the\ length\ of\ the\ side.$

$$T = 2*\left(D\sqrt{2}\right)$$
$$B + 2D = 2\sqrt{2}\ D$$
$$B = 0.83D$$

$Insert\ the\ value\ of\ B\ in\ equation\ (***)$

$$\frac{0.075}{1.83D^2} = \frac{1}{0.025} * \left(\frac{1.83D^2}{3.658D}\right)^{2/3} * \left(\frac{1}{1000}\right)^{1/2}$$
$$\Rightarrow D = 0.33m$$
$$B = 0.274m$$

$Check\ for\ Velocity.$

$$V = Q/A = \frac{0.075\ m^3/\sec}{(0.274+0.33)*0.33}$$
$$= 0.376\ m/\sec$$

$Free\ board.$

$$FB = 0.26m$$

6.3 Design of canal structures

6.3.1 Design of the intake structure.

The intake structure is a hydraulic device constructed at the head of each main canal off taking from the diversion head works. It should be eloigned in such away that no back flow and stagnant zone in the river pocket is formed. The direction of flow of river makes an angle of to 60^{0-} witch the direction of flow canal.

The proposes of the intake structure are:

- ➔ It regulates supply of water in to the canal.

- ➔ It controls the entry of silt in to the canal
- ➔ It prevents the river floods from entering the canal .
- ➔ It can be used to stop the canal supplied when the silt charge in the river water

  exceeds a curtain limit.

A circular concrete pipe of 3m length is selected for its hydraulic efficiency and simplicity in clearing debits along the length of the pipe. The capacity of the intake structure is determined using the orifice formula

$Q = CdA\sqrt{2gh}$

$where, Q = discharqethroughtheorifice$

$= 0.075\, m^3/\sec$

$Cd = coefficent\ of\ disch\arg e$

$(cd\ 0.7\,(R.Baban))$

$A = Area\ of\ qate\ opening\ in\ m^2$

$h = difference\, b/nu/s7d/s\ water\ level$

$= pond\ level - FSL$

$= (1891.5 - 1890.9)m$

$= 0.6m$

$\Rightarrow 0.075 = 0.70 * \frac{\pi d^2}{4}\sqrt{2*9.81*0.6}$

$\Rightarrow d = 0.1994m$

$sayd = 0.2m$

Bu applying Bernoulli’s equation at the in let & outlet points

H= he + hf +ho

Where:-

H= Total energy head

Ha = entrance loss

Ho = loss at out let

Hf = friction loss

$$he = ke\frac{V^2}{28}, (ke\ is\ coefficent\ of\ entrance\ loss = 0.1)$$

$$ho = ko\frac{V^2}{28}, (ko\ is\ coefficent\ of\ out\ let\ loss = 1.0\ )$$

$$hf = \frac{n^3 v^2 L}{R^{4/3}}, n = 0.017(ARORA)$$

$$L = 3.0m, (length\ of\ pipe)$$

$$R = \frac{d}{4} = \frac{0.2}{4}$$

$$= 0.05$$

$$\therefore H = \frac{V^2}{2g}[0.5 + 1.0 + 0.923] = \frac{V^2}{2g}[2.023]$$

$$\Rightarrow V = \sqrt{\frac{H * 2g}{2.023}} = \sqrt{\frac{0.6 * 2 * 9.8}{2.023}1}$$

$$= 2.412\, m/s$$

$$\Rightarrow Q = V.A$$

$$= 2.412 * \frac{\Pi(0.2)^2}{4}$$

$$= 0.07578\, m^3/\sec > 0.075\, m^3/\sec\ hence\ OK!$$

At the opening a steel gate is provided to regulate the discharge passing through the pipe.

Note:- If 0.2m diameter of concrete pipe is not available on the market use the nest larger size of concrete pipe, which are available on the market.

6.4 <u>Design of pipe culvert</u> .

Conveyance culvert is structure built in conveyance system at the intersection of irrigation canal and roads, high ways, drainage canals and so on . The fundamental objective of the hydraulic design of culvert is to determine the most economic diameter at which the design discharge is flow safely.

The design of a pipe a pipe culvert on the left side canal is as follows.

Operating heed = velocity head + Entrance loss + friction loss.

Where:-

Operating heed, HL = 0.20 (Assumed)

Velocity head, = $V^2/2g$

Entrance loss = ke * $V^2/2g$

(ke is an entrance loss coefficient taken as 0.05 for bell mouth Entrance)

$$Friction\,loss = \frac{n^2V^2L}{R^{4/3}}$$

$$H_2 = \frac{v^2}{2g} + ke\frac{V^2}{2g} + \frac{n^2V^2L}{R^{4/3}} = \frac{V^2}{2g}\left(1 + ke + kf\right)$$

$Wrere:-$

$$kf = \frac{n^2L*2g}{R^{4/3}}$$

$L = length\,of\ the\,culvert\,(= 5m)$

$n = roughness\,coefficient\,of\ the\,concrete\ pipe\,(n = 0.014)$

$R = Hydroulic\,mean\,redius, R = \frac{A}{P}$

$g = Acceleration\,due\,to\,gravity\left(= 9.81 m^2/s\right)$

$$\Rightarrow V = \left[\frac{2gH_2}{1 + ke + kf}\right]^{1/2}$$

$multiplying\,both\,sides\,by\,A$

$$V*A = A*\left(\frac{2gHL}{1 + ke + k_f}\right)^{1/2}$$

$$Q = A*k\sqrt{2gh_l}$$

Where;-

Q = Discharge – through the culvert

= 0.075 m^3/s +0.075 m^3/sec =0.15m^3/sec

(because mostly the culvert design discharge is taken twice the canal design, i.e taking in to consideration the likely of runoff entering in to the canal.)

$$K = \sqrt{\frac{1}{1+ke+kf}}$$

$$Kf = \frac{n^2 * L * 2g}{R^{4/3}} = \frac{0.014^2 * 5 * 2 * 9.81 * 4^{4/3}}{d^{1/4}} = \frac{0.122}{d^{4/3}}$$

Substituting all the values

$$0.15 = \frac{1}{4}\prod d^2 * \left[\frac{1}{1+0.05+\frac{0.122}{d^{4/3}}}\right]^{1/2} * \sqrt{2*9.81*0.2}$$

By trial and error

$$d = 0.345m$$

But in order to allow inspection of conduit, the circular pipe should have a minimum diameter of 0.6m

d = 0.6m

6.5 Fluming of the canal

A flume is a structure constructed to take the canal(s) crossing depression or river. The contraction in the water way of the canal will reduce the length of the barrel or width of aqueduct. The maximum fluming is generally governed by the extent that the velocity in the trough should remain sub critical (of the order of 3m/sec), the transition has to be designed so as to provide a smooth change from one stage to the other so as to avoid sudden transition and the formation of eddies etc. Generally, the normal earth earth channel section is trapezoids, while the flumed canal is rectal is trapezoidal, while the flumed canal is rectangular. The following method may be used for designing the channel transition.

i) mitra's method of design of transition (when water depth remains constant)

ii) chaturved's method of design of transition .

iii) Hind's method of design of transition (when water depth may or may not vary) .

Since the depth is constant, for our project mitre's method is appropriate.

Design of flume.

i) Design of canal water way

Flumed width =0.2m

Normal bed width of the canal = 0.274m

Provide a splay of 2:1 in contraction, the length of contraction transition. .

$$=\frac{0.274-0.2}{2}*2=0.074$$

Provide a splay of 3:1 in expansion, the length of expansion transition .

$$=\frac{0.274-0.2}{2}*3=0.111$$

Length of flumed rectangular portion of the canal b/n column is provided.

i) Design of transition

(a) Contraction transition

$$B_x=\frac{BnB_f L_f}{B_n L_f - x(B_n - B_f)}$$

where:-

Bn = Bed width of the normal channel section .

Bf = Bed width of the flumed channel section.

Lf = Length of transition = 0.074m

$$\Rightarrow Bx = \frac{0.274 * 0.2 * 0.074}{0.274 * 0.2 - *(0.274 - 0.2)} = \frac{0.0041}{0.0203 - x(0.0074)} = \frac{0.0554}{0.2743 - x}$$

Table 6.1 --computed values for the design of contraction transition.

X(m)	0	0.015	0.03	0.045	0.06	0.074	0
0.0554	0.20	0.2137	0.2268	0.2416	0.2585	0.2766	
0.2743-x							

(b) Expansion transition

Bn = 0.274

Bf = 0.2

Lf = 0.111

$$B_x = \frac{Bn' B_f \; L_f}{Bn \, L_f - x(Bn - B_f)} = \frac{0.0061}{0.0304 - x(0.074)} = \frac{0.0824}{0.4108 - x}$$

Table 6.2--- Computed values for the design of expansion transition.

X(m)	0	0.015	0.03	0.045	0.06	0.074	0.1	0.111
0.0824	0.2	0.2082	0.2164	0.2253	0.2349	0.2454	0.2651	0.2748
0.4108-x								

The depth of water in the flume section.

Y = 0.33m

(c) Design of column for flume section down wand water load acting when there is no uplift.

0.2*0.33*1*1000kg/m = 66kg/m

= 0.6775 KN m

Self-weight of flume per m^2 for 3.15mm thickness (black sheet) = 24.70kg/m.

Self weight load per meter

=2.92*1*24.7

= 72.124kg/m

= 0.7075 KN/m

Total load perimeter = 0.475 KN/m + 0.7075K/m

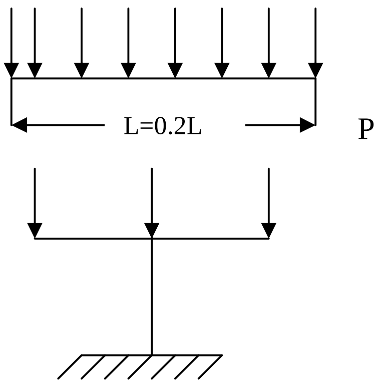
= 1.3550KN/m

Take = 1.5KN/m

Number of columns

Take 5 columns. With 5m c/c distance, length of column, L= 7m

Analyses for one column



$$P = \frac{1.5*1.5}{7} = 0.321KN$$

$$P^1 = 2.p = 2*0.321$$
$$= 0.642KN$$

$$P_{cr} = \frac{\lambda^2 EI}{\ell^2} = \frac{\lambda^2 * 200*10^6 * 3.65*10^{-8} = 1.470KN}{7^2}$$

$P^1 < P < \Rightarrow Ok!$

Rolledsteeltubes

$No\min el\, bore = 25mm$

$Out\, sidediemeter = 33.7mm$

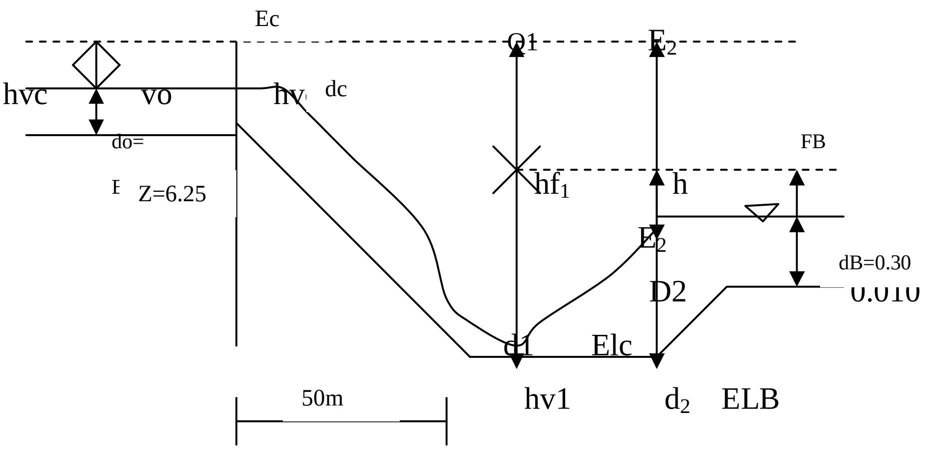
$I = 3.65cm^4$

$L = 7m$

6.6 Design of chute structure

Chutes are control structures required at suitable locations in canals so as to reduce the velocity of flow by dissipating energy.

The structure is used when an appreciable fall of water is necessary due to the surrounding topography (i.e. relatively steep slopes.



do = 0.329

Vo = 0.007

EL.A = 1887.500

EL.B = 1881.250

1 **Critical flow hydraulics** .

Discharge, Q = 0.075 lit/sec

$$Notch\,width\ b_c\ \frac{0.734Q}{d_o^{3/2}} = \frac{0.734*0.075}{(0.329)^{1.5}} = 0.29$$

$$Use = 0.30m$$

$$Unit\,disch\arg\ e\,q = \frac{Q}{Bc} = \frac{(0.075)}{0.30} = 0.25m^3/\sec/m$$

$$Critical\,depth, d_c = \frac{(q^2)^{1/3}}{q_c} = \frac{(0.25^2)^{1/3}}{9.81} = 0.185m$$

$$Critical\,velocity, Vc = \frac{q}{d_c} = \frac{0.25}{0.185} = 1.351m/\sec$$

$$Velocit\ head, hv_c = \frac{V_c^{\ 2}}{2g} = \frac{(1.351)^2}{2*9.81} = 0.093m$$

$$Welocity\ Area; A_c = bcdc = 0.30*0.185 = 0.056m^2$$

$$Water\ perimeter, pc = bc + 2dc = 0.30 + 2*0.185 = 0.67m$$

$$Hydraulic\,radius,\ Rc = \frac{A_c}{p_c} = \frac{0.056}{0.670} = 0.084m$$

$$Water\ Surface\,Slope,\ I_c = \frac{(nVc)^2}{(Rc^{2/3})} = \frac{(0.017*1.351)^2}{[(0.084)^{2/3}]}$$

$$= 0.01431$$

2. **Energy at critical section (c)**

$Z = EL_A - ELB$

$= 1887.500 - 1881.230$

$= 6.25m$

$Ec = Z + d_c + {}_hV_c$

$= 6.25 + 0.185 + 0.003$

$= 6.528m$

Table 6.3 Energy at section 1 (E_1

Designation	Result of the			
Trial Number	1	2	3	4
Assumed depth, d_1	0.055	0.101	0.082	0.070
$b_1 = bc$	0.300	0.300	0.300	0.300
$A_1 = b_1d_1$	0.0165	0.0303	0.0246	0.0209

$V_1 = Q/A_1$	4.545	2.475	3.0488	3.5885
$hv_1 = V1^{2/2g}$	1.053	0.3122	0.474	0.6563
$p_1 = b_1+2d_1$	0.410	0.502	0.464	0.4394
$R_1 = A_1/p_1$	0.0402	0.0604	0.0530	0.476
$1_1 = \frac{({}_nV_1)^2}{(R1^{2/3})^2}$	0.4335	0.0747	0.1349	0.2159
$I_m = \frac{(I_c+I_1)}{2}$	0.2239	0.0445	0.0746	0.1151
$hf_1 = I_mL$	11.195	2.225	3.730	5.755
$E_1 = d_1+hv_1+hf_1$	12.303	2.638		6.481
$E_9 = E_c = 6.528$	too high	too small		OK!

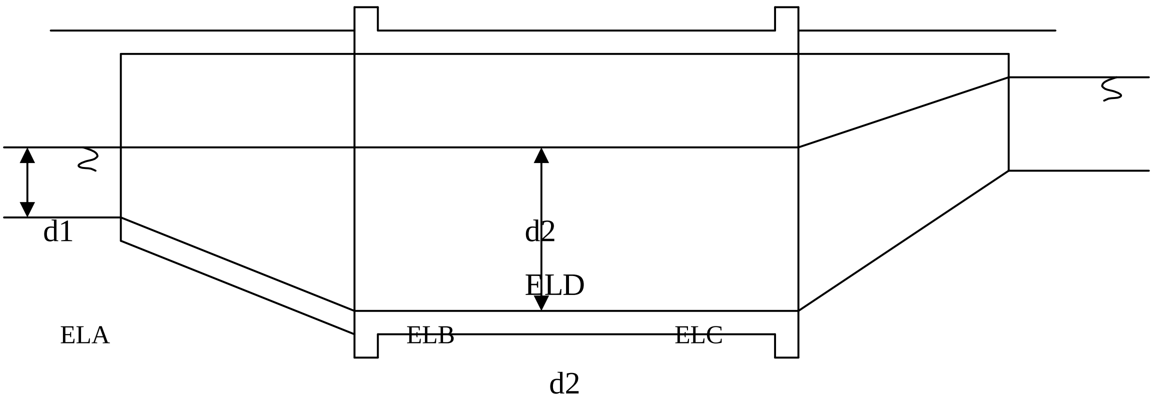
4. **Conjugate depth after jump**.

$$froude number, F_r = \frac{V_1}{gd1} = \frac{3.5886}{9.81*0.070} = 4.33$$

$$Conjugate depth, d_2 = \frac{d_1}{2}\left(1+8F_r^2-1\right)$$

$$= \frac{0.70}{2}\left(1+8(4.33)^2-1\right)$$

$$= 0.395m$$



5**. Stilling basin**

a. Length, $L = 4d_2 = 4*0.395 = 1.58m$

Adopt L = 1.58m

$$b.\,Wodth\,B = \frac{18.46Q}{Q+0.01} = \frac{18.460.075 = 0.50m}{0.075 + 9.91}$$

$$C.\,Bottom\,Elevation(ELC)$$

$$V_2\,\frac{q}{d_2} = \frac{0.25}{0.395} = 0.633\,mm/\sec$$

$$hv_2 = \frac{V^2{}_2}{2g} = \frac{0.6332^2}{2*9.81} = \quad 0.020$$

$$E_2 = d_{2+}\,h_{UL} = \quad 0.395 + 0.02 = 0.415$$

$$E_B \; = d_3 + h_{v2} = 0.30 + 0.010 = 0.310$$

$$a = E_2 - E_B = \quad 0.415 - 0.310 = \quad 0.105$$

$$adopt = \quad 0.12m$$

$$;- EL_c = \quad EL.B - a$$

$$= \quad 1881.250 - 012$$

$$= 1881.130$$

6.7 Farm Turnouts

An out let or a farm turn out is a structure at the head of a water course, which connects it with a supply canal. There are more farm out lets than other structures in an irrigation system and there fore they have a decisive influence on the functioning of system.

<u>Canal out lets</u> :- may be divided in to three classes as follows

a) **<u>Non- modular</u>**:- out lets are out lets in which the discharge depends on the difference in level between the water surface

b) **<u>Semi – modules or flexible modules:-</u>** are out lets the discharge through which are independent of the water level in the water course, but vary with the water level of the distributor

c) **Rigid modules**:- are those which maintain a constant discharge irrespective of the fluctuation in the water level of the supplying channels as well as the water channels.

Due to easiness to be installed and relative cheapness, the concrete pipes of different diameters are adopted for the required position of the canals.

Sample Calculation

Design of off-take to Tc-1 on Mc – 1

Data available:-

A = 2.9ha

$Q_{mc\text{-}1}$ = 0.075m^3/sec

$Q_{Tc1\text{-}1}$ = Q_{off} –take = 0.003m$^{3/}$s

Q d/s = Qmc –1 –QTc 1-1 = 0.072 m^3/s

Bmc-1 = 0.274 m

Dmc-1 = 0.33 m

Use, s = 0.1, C=1.71, m=1

Bc =2ms+b

Bc = 2*1*0.1+0.23 = 0.43m

$$Q_{d/s} = CBcH^{3/2} \Rightarrow H = \left[\frac{Q_{d/s}}{CBc}\right]^{2/3} = \left(\frac{0.072}{1.71*0.43}\right)^{2/3} = 0.212m$$

$$H_t = H + s$$
$$= 0.12 + 0.1 = 0.312m$$

1st trial

$$h = Ht - \phi/2, Assume\,\phi = 3^{11} = 7.62cm$$

$$h = 0.312 - \frac{0.0762}{2} = 0.274m$$

$$A = \frac{\Pi\phi^2}{4} = 0.0046m^2$$

$$Qp = CA(2gh)^{1/2}$$
$$= 0.6 * 0.0046(2 * 9.81 * 0.274)^{1/2}$$
$$Qp = 0.0064 m^3/s > Q_{off-take} = 0.003 m^3/s \quad OK!$$
$$Therefore,\ 3''\ pipe\ size\ is\ selected.$$

A= 8.24 ha

Q_{mc-1} =0.075 m^3/s

Q_{Tc1-2} = $Q_{off-take}$ = 0.008 m^3/s

$Q_{d/s}$ = Q_{mc-1} - Q_{Tc1-2} = 0.067 m^3/s

B_{mc-1} = 0.274 m

D_{mc-1} = 0.33 m

$$Qd/s = cBc\ H^{3/2} \Rightarrow H = \left[\frac{Qd/s}{cBc}\right]^{2/3} = \left[\frac{0.067}{1.7 * 0.43}\right]^{2/3} = 0.203m$$

$$Ht = H + 5$$
$$= 0.203 + 0.1 = 0.303m$$

$\mathbf{2^{nd}}$ $\mathbf{T}rial$

$$h = ht - \varphi/2,\ Assume\ \varphi = 4 = 10.16cm$$
$$h = 0.303 - \frac{0.102}{2}$$
$$h = 0.252m$$
$$A = \frac{\prod \varphi^2}{4} = 0.0081m^2$$
$$Q_p = c\,A(2gh)^{1/2}$$
$$= 0.6 * 0.0081(2 * 9.81 * 0.252)^{1/2}$$
$$= 0.0108m^{3/5} > 0.008m^{3/5}$$
$$therfore, 4''\ pipe\ size\ is\ selected$$

The design of farm outlets on other canals are done on the same procedure as above.

6.8 <u>Canal Drop</u>

A canal drop is a structure constructed a cross a canal to lower down its water level and destroy the surplus energy.

In our case a rectangular weir with raised crest that is the category of vertical drop has been adopted which is the cheapest and simplest to construct.

In our case a rectangular weir with raised crest that is the category of vertical drop has been adopted which is the cheapest and simplest to construct.

Energy- dissipation by a vertical drop is usually resorted to where the drop is small. Although the interpretation of “small” differs in various parts of the world, according to the USBR standard a small vertical drop which doesn’t exceed 1m except where the canal is lined with a hard surface d/s of the structure, when the drop may be up to 2m. (FaO, 1986).

Design of drops on Sc-1

Sample Calculation

Available Data:-

$Q = 0.064 m^3/sec = 64$ L/sec

$B_1 = B_2 = 0.00$m,where $B_1$and B_2 are the base width of canal in u/s and d/s respectively

$D_1 = D_2 = 0.25$m

Design details

i) volume of basin ,$V_B = (Q*H_L) /150$

$$V_B = 64*1/150 = 0.43\ m^3$$

where H_L=drop head in m

Q = Discharge in L/sec

ii) Length of basin , $L = 1.5*H_L$

$$= 1.5*1 = 1.5m$$

iii) Cross sectional area of the basin

$$A = L\ (D_2+H_b)$$

Where , H_b = the depth of basin (0.1 to 0.3m)

Let’s take H_b=0.1m

A= 1.5(0.25+0.1) = 0.525m^3

$$iv)\ width\,of\,basin, \mathbf{B}_b = \frac{volume\ of\ basin}{Area\ of\ basin} = \frac{0.43}{0.525} = 0.82m$$

$v)\ crest\ width, B_t$

$$B_t = B_b - 0.1\left(for\ trapeziodal - canal\right)$$
$$= 0.82 - 0.1$$
$$= 0.72m$$

$vi)\ Depth\ of\ flow\ above\ crest\ level\left(H\right)$

$it\ is\ determined\ by$

$$Q = C_d * B_t * \sqrt{2g} * H^{3/2} \qquad where\ H = 0.36\ for\ vertical\ u/s\ face\ crest\ wall$$

$$0.064 = 0.36 * 0.72\sqrt{2*9.81}\ H^{3/2}$$

$$H = 0.15m$$

$vii,\ Height\ of\ canal\ over\ the\ u/s\ bed - level, h$

$$h = D_1 - H$$
$$= 0.25 - 0.15$$
$$h = 0.1m$$

$viii)\ u/s\ \ protection\,work$

$a,\ stone-pitching\ \ is\ provided\ on\,the\,u/s\,bed$

$$length\left(L_1\right)=1.5D_1$$

$$=1.5*0.25=0.375m$$

$provide\,u/s\,stone\ pithing\ \ for\ lelengthOf\,0.375m\,and\ a\,thickness\,of\ \ 0.10m$

$b,\ side\ protection\ \ for\ 0.1m\,thickness\,and\ length\,of\ \ 0.375m\,has\,been\ provided$

$ix)\ \ D/s\ protection\,work$

$a,\ Bed\ \ protection\,of\ \ stone-pitching$

$$length\left(L_2\right)\ =3D_2$$

$$=3*0.25=0.75m$$

$provide\,a\,stone\,dry\ pitching\ of\ \ 0.1m\,thickness\ \ for\,length\,of\ 1.2m$

$b,\ \ side\ protection$

$The\,same\,length\,as\,that\,of\ \ bed\ \ protection\,i.e\,0.75m$

$length\ \ is\ provided\ as\,side\ protection\ \ for\,thikness\,of\ \ \ \ 0.1m$

The results of important parameters of the other vertical drops on available canals is designed in the same procedure as the above sample calculation.

7.0 SELECTION AND DESIGN OF IRRIGATION SYSTEM

7.1 GENERAL

In the surface methods of irrigation, water is applied directly to the soil surface from a channel located at the upper reach of the field .

Two general requirements of prime important to obtain high efficiency in surface me thods of irrigation are properly constructed water dis tribution system to provide adequate control of water to the fields & land preparation to permit uniform distribution of water over the field.

7.2 Furrow Irrigation

Irrigation water may be applied to crops by flooding it on the field surface by applying it beneath the soil surface by spraying it under pressure or by applying it in drops.

- The water supply, the type of soil, topography of the land & the crop to be irrigated determined the correct method of irrigation to be used for this project furrow irrigation is selected for the following reason : (as given & explained by Michael, 1997)

- Both large & small irrigation streams can be used by adjusting the number of for rows irrigated at any one time to suit the available flow.
- Furrows irrigation can be used to irrigational all cultivated crops planted inI rows, there fore suitable for those crops chosen for the project.
- Furrows are particularly will adopted to irrigating which are subject to fungi root not (sweet potato), an well as those crops subjected to injury from ponded surface water.
- Furrow irrigation is suitable to most soils except sands that have a very high infiltration rate.
- Furrows irrigation has some distinct advantages compared to the surface irrigation in that :-
    - i) water in furrows contacts only one half to fifth of the land surface, there by reducing pudding, and cresting of the soil & evaporation loss
    - ii) Earlier cultivation is possible which is a distinct advantages in heavy soils.
    - iii) The method reduces lab our requirements in land preparation & irrigation so economical.
- Furrow method can be adapted to use, with out erosion ,on wide range of natural sloped by carrying out the furrows a cross a sloping field rather than down the slopes
- Furrows can be spaced to fit the crops grown & the standard machines used for cultivation.
- Furrows should be spaced close enough to ensure that water spread to the side's into the ridge and root zone of the crop before it moves down below the root zone.

7.3 Furrow Irrigation Design Consideration

Efficient water application in furrow irrigation depends on the knowledge of the hydraulic of flow in furrow. The dominant variables influencing the rate of flow in f arrows are the entrance stream size, infiltration rate, size and shape of wetted section of furrow. Furrow slope and hydraulic resistance, which is the combination roughness offered by the wetted surface of the furrow and the resistance offer by the crop.

A) FURROW SPACING:

Furrows can be spaced to fit the crops grown and the type of the machines used for planting and cultivation.

- crops live potties maize and cotter are plated 60 to 90 cm
- Vegetable crops like lettuce, carrots and onions are spaced 30 to 40 cm.

Furrows should be spaced close enough to ensure that water spreads to the sides into the ridge and root zone of the crop to replenish the soil moisture uniformly.

B) <u>FURROW LENGTH:</u>

The optimum length of a furrow is usually the longest furrow that can be & safely and efficiently irrigated. Proper furrow length depends largly on hydraulic conductivity of the soil.

The length of furrow which can be efficiently irrigated may be as short as 45m on soils which take up water rapidly, or as much as 300m or longer on soils with low infiltration rates.

The length of furrow may offend be limited by the size and shape of the field.

<u>C) FURROW SLOPE:</u>

The slope or grade of the furrow is important because it controls the speed at which water flows down the furrow . A minimum furrow graded of 0.05% is needed to ensure surface drainage.

As the furrow grade increase, the rate of infiltration slows down and the side spread of water into the crop ridge decreases, so that wastage may occur at the end of the furrow.

As we know uniform wetting of the soil and maximum efficiency of irrigation are impossible unless the grade is uniform. There for there is deviation from perfection is tolerable with large furrows and the efficiency of water use may frequently be in proved by use of furrow irrigation

D) FURROW STREAM:

The size of the furrow stream is the one factor, which can be varied after the furrow irrigation system has been instolled. The size of the furrow stream usually varies from 0.5 to 2.5 L/s.

To obtain the most uniform irrigation, the largest stream of water that will not cause erosion is used in each furrow at the beginning of irrigation.

7.4 Design parameters for fixed inflow graded furrow system

⇒ INTAKE FAMILIES (infiltration families) : soils with similar infiltration characteristics can be grouped in to infiltration families. The classification is based on one dimension infiltration

(for boarder, basin irrigation) but can also be applied to two dimensional infiltration (furrow irrigation) by taking into account the wetted perimeter of furrow & furrow spacing .

- Advance time (T) : is the time at which the advance wave front reach a particular point .
- Opportunity time (To): It is the difference b/n the time the water front reaches a particular point along the furrow and the time at which the tail H_2o receded from the same point
- Recession time (TR): The time for out flow of water to stop after inflow at the head of the furrow has ended

- Cut off time (t_{co}): The time at which the supply is turned off, measured from the onset of irriga

8.0 Drainage System

Water logging of the land occurs when the water table table rises and the soil in the root zone of the plants gets saturated and the air circulation is stopped. This will happen due to intensive irrigation and inadequate drainage of the irrigated land so; a properly designed drainage system is quite effective for prevention of water logging. Since our project is small scale irrigation project and the non availability of data for the Ground water condition we have selected a surface drainage system parallel to the secondary canal.

The capacity of the drainage canal is fixed as drainage coefficient times drainage area

i.e $Q_{ar} = DC * A$

where Dc = drainage coefficient

A = drainage area

8.1 Design of surface drainage canals

Data available

. Side slope of drain

The side slope of drain obtained from table below:

Table 8.1 Maximum side slope for drainage canal

Soil type	Side slope (H:V)
Sand, soft clay	3:1
Sandy clay, silt 10am	2:1
Fine clay, clay 10am	0.5:1

- Side slope of 0.5 :1 is taken for this project since the soil is clay loam type

1 M = 0.5

0.5

8.1.1 Drainage coefficient (DC)

It is the amount of depth of water that must be removed in 2.4 hours. The DC can be calculated as follows:-

DC = 10mm/ day f or MAR < 1000mm

$= \frac{MAR}{100}$ mm/day, for MAR > 1000mm (Hundson,1983)

8.1.2 Mean Annual Rainfall (MAR)

Mean annual rainfall is usually calculated by taking the simple average of total rainfall of several consecutive years.

Sir Alexander Bennie has shown that if the available rainfall records are fora period lesser than 35 years, there will be error in the computed mean rainfall , as shown

Table 8.2

Period in years	5	10	15	20	25	30	35
% age error in mean rainfall	14.9	8.0	4.8	3.3	2.8	2.3	1.8

From Table 8.3 Data from wulbareg meteor logical station
In our case for 26 years the % age error is = 2.7%.

$$MAR = \left(\frac{41.3+30+25.4+48.7+33.6+33.3+\text{-------------}+37.2+53.3}{26} \right)$$

= 39.8346 cm + 2.7% * 39.8346 mm
= 39.8346 + 1.0755
MAR = 38.8mm or 40.9 m

Table 8.3

Year Yearly max	1972 41.3	1973 30	1974 25.4	1975 48.7	1976 33.6	1977 33.3	1978 38.6	1979 35.1	1980 41.8	1981 36.5
Year Yearly max	1982 29.1	1983 32.5	1984 52.7	1985 43.4	1986 47.6	1987 67.7	1988 58.6	1989 59.4	1990 34.8	
Year Yearly max	1991 42.3	1992 29.7	1993 23.5	1994 24.3	1995 35.3	1996 37.2	1997 53.3			

In this case since MAR = 39.8mm < 1000 mm, the drainage coefficient,

Design DC = 10mm/day

A Properly designed drain should have:

I) A scouring and non silting velocity of flow,
II) Sufficient capacity to carry the design runoff,
III) Adequate depth to drain the land , and
IV) Stable side slopes, which will not cause of slide in to the drain.

Data available

- Drainage coefficient (DC) = 10 mm/day
- Bed slope of the surface drain. $S = \frac{1}{100}$ to $\frac{1}{1000}$ (According the to Topography)

In this case, S = 1/800 is adopted   S= 0.00125

- side slope m = 0.5
- Manning's roughness coefficient (for open drain $\geq$ 0.03)
  Take n =0.03 since our channel is earth canal, unlined.

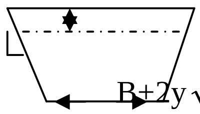
From Manning's equation

$Q = \frac{1}{N} AR^{2/3} S^{1/2}$ - For Drainage area (A) = 2.963ha

- Drainage discharge (Qdr)

$Dc*A = 0.00343 m^3/S$

$AR^{2/3} = \frac{Qn}{S^{1}/2} = \frac{0.00343*0.03}{\sqrt{0.00125}} = \underline{0.00291}$ -----------(1)



F.B   T= B+2my , A = my^2 + By , p = $B+2y\sqrt{m^2+1}$ 1

B

For most efficient trapezoidal section ,

$\frac{R}{2} = y,$

$A = \sqrt{3}\, y^2$ and

From Q = AV (continuity eqn)

V = Q/

According to lacey F.B = 0.20 + 0.15 Q $^{1/}$

Table 8.4

Drainage discharges of the canals on the left command

Drainaage cana	Area (h	Q_{dr} = D.c*Area (m^3/sce)	$Q_{dr(l/sec)}$

Dc 1	2.9	0.00336	3.36
Dc 2	8.242	0.00954	9.54
Dc 3	3.348	0.00388	3.88
Dc 4	2.841	0.00329	3.29
Dc 5	3.075	0.00356	3.56
Dc 6	3.164	0.00366	3.66
Dc 7	6.75	0.00777	7.77
Dc 8	2.363	0.00273	2.73
Dc 9	3.333	0.00385	3.85
Dc 10	2.667	0.00309	3.09
Dc 11	2.287	0.00265	2.65
Dc 12	2.963	0.00343	3.43
Dc 13	2.216	0.00257	2.57
Dc 14	3.643	0.004213	4.213
Dc 15	2.463	0.00285	2.85
Dc 16	2.613	0.00302	3.02
Dc 17	2.068	0.0024	2.396
Dc 18	2.4	0…00278	2.78
Dc 19	2.734	0.00316	3.16
Dc 20	2.629	0.00304	3.04
Dc 21	3.654	0.00422	4.22
Dc 22	2.575	0.00298	2.98
Dc 23	4.124	0.00477	4.77

For simplicity drainage canals are grouped according to their area (or discharge capacity)

Area range (ha)	Drainage canal	Area (ha)	Drainage discharge ($Q_{dr\ in\ m}{}^{3}$/sec
2.068 – 2.963	Dc_a	2.263	0.00343
3.075 – 3.654	Dc_b	3.654	0.00423
4.124	Dc_c	4.124	0.00477
6.71	Dc_d	6.71	0.00777
8.242	Dc_e	8.242	0.00954

Table 8.5 The calculated dimensions of left side drainage canals

Drain type	Q_{dr} $(_{m}{}^{3}$/sec)	slope	m	n	B(m)	D(m)	A(m)	P(m)	V(m/s)	(m) F.B
DC_a	0.00343	0.00125	0.5	0.03	0.13	0.108	0.02	0.38	0.172	0.22

DCb	0.00423	0.00125	0.5	0.03	0.145	0.117	0.024	0.41	0.176	0.22
DCc	0.00477	0.00125	0.5	0.03	0.153	0.123	0.026	0.43	0.183	0.22
DCd	0.00777	0.00125	0.5	0.03	0.182	0.182	0.147	0.037	051	0.23
DCe	0.00954	0.00125	0.5	0.03	0.197	0.159	0.044	0.044	0.55	0.23

- The same procedure is used to design the drainaaage system for the right side command.

9.0 ECONOMIC ANALYSIS9.1 General:-

The main aim of economic analysis is to estimate the amount and relative financial Profitability and attractiveness of the proposed.

9.2 Methodology and Estimates Used.

Benefits and costs are estimated for the both with and without the project case to obtain the incremental net return from the project. Based on this cash flow is made over the project life (20 year). Under cost with the project case investment cost, cost of operation and maintenance, replacement cost for major structures (weir) and production cost are included while production cost is the main cost component for the with out project case.

Direct Benefits obtained from agricultural product are used on benefit stream for both with and with out project.

A project life of 20 years is taken for the Analysis. Interest of 10.5% is used as discount rate.

9.3 Project Benefit Estimation:-

The purpose of implementing irrigation project is to produce different crops from which the benefit of the Project is obtained. For the determination of the project benefit the cash flow for financial cost benefit analysis for both with the project and without the project is used.

le 9.1 Volume of production (Qt) and Economic value (Br)

(Suppl + Compl.)

No	Crops	Y_1		Y_2		Y_3		Y_4		Y_5		Y_6	
		Prod^{n}	Value	Prod^{n}	Value	Prod^{n}	Value	Prod^{n}	Value	Prod^{n}	Value	Prod^{n}	V
1	Maize	66	36630	1034	5680	2052	112860	2880	158400	3528	194040	4016	2
2	Sorghum	60	3900	112	7280	320	20800	520	33800	672	43680	840	5
3	H/bean	118	15980	310	26350	640	54400	882	74970	1060	90100	1232	1
4	S/potato	350	17500	644	32200	1500	75000	2340	117000	3220	161000	3640	1
5	Pepper	84	16800	144	28800	288	57600	448	89600	558	116600	660	1
6	Tomato	-	-	378	37800	880	88000	1300	130000	1800	180000	2210	2

7	Cabbage	-	-	318	12720	640	25600	1000	40000	1440	57600	1820	7
8	Onion	-	-	315	42525	700	94500	1080	145800	1430	193050	1820	2
	Total	-	90810		244545		528760		789570		1036070		1

Table 9.2 Specification and bill of quantity

Item	Description	Unit	Qty	Rate	Sub total	Total
1.	**Head work**					
1.1.0.	Weir Foundation					
1.1.1.0.	Excavation and Earth work					
1.1.1.	Clearing the site form boulders	M^2	2000	5	10,000.00	
1.1.1.2.	Construction of temporary diversion channel .	M^3			10,000.00	
1.1.1.3.	Excavation of hard rock for foundation of the diversion weir.	M^3	46.80	100	4,580.00	
	Total Carried to Summary				24,680.00	24,680.00
1.1.2.0.	Concert work					
1.1.2.1.	Plane concrete works as specified mix proportion by the designer. Form work are measured as hole	M^3	147.40	600	88,440.00	
	Total Carried to Summary				88,440.00	113120.00
2	**Retaining wall**					
2.1.0.	Excavation and earthwork					
2.1.1.	Excavation of compacted soil for retaining wall.	M^3	123.851	15	1,857.765	
2.1.2.	Excavation of hard rock retaining wall foundation.	M^3	36.17	100	3,917.00	
2.1.3.	Filling at the back of the retaining wall.	M^3	69.56	30	2,086.80	
2.1.4.	Cart away of surplus excavated material.	M^3	93.461	10	934.610	
	Total Carried to Summary				8,796.175	121,916.175
2.2.0.	Masonry work					
2.2.1.	40cm thick basaltic.	M^3	184.743	350	64,660.05	
2.2.2.	Rip rap	M^2	60.00	150	9,000.00	
2.2.3.	Point of the external side of the wall specified mortar mix proportion.	M^2	127.002	30	3,810.060	
2.2.4.	Plaster the required part of wall.	M^2	161.500	30	4,845.000	
	Total Carried to Summary				82,315.110	204,231.285
2.3.0.	**Intake Gate**					
2.3.1.	R.C pipe preparation & installation	nr/pcs	5	350	1,750.00	
2.3.2.	Provide, Prepare and fix in position sheet metal or equivalent to it for pates.	nr	2	10,000	20,000	
	Total Carried to Summary				21,750.00	213,981.285

2.4. 2.4.1. 2.4.2.	**Under – sluice** R.C work Include form work and curing. Provide, prepare and fix in position sheet metal or equivalent to it for gate.	 M^3 nr	 2.610 2	 800 10,000	 2,088.00 20,000.00	
	Total Carried to Summary				22,088.00	248,069.285

Item	Description	Unit	Qty	Rate	Sub total	Total
2.5	Steel bar for fencing of the plat form of the under sluice	kg	76.098	50	760.980	
	Total Carried to Summary				760.980	248,830.265
3.0	**Main canal**					
3.1	Excavation & earth work					
3.1.1.	Excavation of compacted soil for the main canal shaping.	M^3	1,773.709	15	26,605.635	
3.1.2.	Excavation of soft rock for the main canal shaping.	M^3	1,700.890	25	42,522.250	
3.1.3.	Excavation of hard rock for the main canal shaping	M^3	405.385	100	40,538.50	
3.1.4.	Filling of selected material for main canal shaping.	M^3	65.450	30	1,963.500	
3.1.5.	Cart away the surplus excavated material.	M^3	3,814.534	10	38,145.34	
	Total Carried to Summary				149,775.225	398,605.490
3.1.2.	Masonry Work (Canal Lining)					
3.1.2.1.	40cm thick basaltic	M^3	649.800	360	227,430.00	
1.3.1.2.	Plaster the required part of the main canal 1:3 mortar	M^3	1976.000	30	59,280.00	
	Total Carried to Summary				286,710.00	685,315.490
3.1.3.	**<u>Structures</u>**					
3.1.3.1.	Drops Excavation and earth work					
3.1.3.1.1. a.	Excavation of compacted soil for structural formation.	M^3	7.849	15	117.735	
b.	Cart away the surplus excavations material	M^3	7.849	10	78.490	
	Total Carried to Summary				196.225	685,511.175
3.1.3.1.2.	**<u>Masonry Work</u>**					
a.	40cm thick basaltic	M^3	7.849	350	2,747.150	
b.	Plaster the required part of the structure with 1:3 mortar	M^3	32.400	30	972.00	
	Total Carried to Summary				3,719.15	689,230.865
3.1.3.2	**Chute**					
3.1.3.2.1.	Excavation and earth work					
a.	Excavation of compacted soil for structural foundation	M^3	28.500	15	427.50	
b.	Cart away the surplus excavations material.	M^3	28.500	10	285.000	
	Total Carried to Summary				712.500	689,943.365
3.1.3.2.2..	**Masonry work**					
	40cm thick basaltic	M^3	32.633	350	11,421.550	
	Plaster the required part of the chute	M^3	103.834	30	3,115.020	
	Total Carried to Summary				14,536.570	704,479.935

	Description	Unit	Qty	Rate	Sub total	Total
3.1.3.3. 3.1.3.3.1 a b c d.	Elevated Flume Excavation & earth work Excavation of soft rock for structure foundation. Excavation of hard rock for the structure foundation Compacted back fill from previous excavated Cart away the surplus excavated material	 M^3 M^3 M^3 M^3	 1.800 5.880 1.560 6.120	 25 100 30 10	 45.00 588.00 46.800 61.200	
	Total Carried to Summary				741.000.	705,220.935
3.1.3.3.2. a. b.	Masonry Work 40cm thick basaltic Plaster the required part of the structure.	 M^3 M^2	 1.800 6.60	 350 30	 630.000 198.00	
	Total Carried to Summary				828.000	706 ,048.935
3.1.3.3. a b.	**Concert work** Reinforced concrete work Includes formwork & curing. Plaster the required part of the structure.	 M^3 M^2	 13.758 110.278	 800 30	 10,006.40 3308.340	
	Total Carried to Summary				14,314.74	720,363.675
3.1.3.4. 3.1.3.4.1. a b. c.	**Road/Drainage Culverts** Excavation & earth work Excavation of Compacted soil for structure foundation. Compacted back fill from previous Excavated around the structure. Cart away the surplus excavated material.	 M^3 M^3 M^3	 37.406 16.560 20.846	 15 30 10	 561.060 461.090 208.460	
	Total Carried to Summary				1,266.350	721,630.025
3.1.3.4.2 a. b.	**Masonry Work** 40cm thick basaltic Plaster the required part of the structure with 1:3 mortars.	 M^3 M^3	 39.509 103.108	 350 30	 13,828.15 3,093.24	
	Total Carried to Summary				16,921.39	738,551.415
3.1.3.4.3. a.	Concrete work Plane concrete works Includes form work & cunning.	 M^3	 0.502	 600	 301.200	
	Total Carried to Summary				301.200	738,852.615
3.1.3.4.4. a.	**Pipe work** Plane concrete pipe(with 0.4@).preparation & installation works as specified	 Nrs	 17	 300	 600	

	Total Carried to Summary				600	744,552.615

Item	Description	Unit	Qty	Rate	Sub total	Total
	Access road	km	10	4000	40,000	
	Total Carried to Summary				40,000	784,552.615
	Mobilization & camping	Ls			50,000	
	Total Carried to Summary				50,000	834,552.615
					10% contingency Project Cost	83455.262 918,007.877

Table 9.3 **Farm Budget Economic**

No	Description	Y_1	Y_2	Y_3	Y_4	Y_5	Y_6	Y_{7-20}

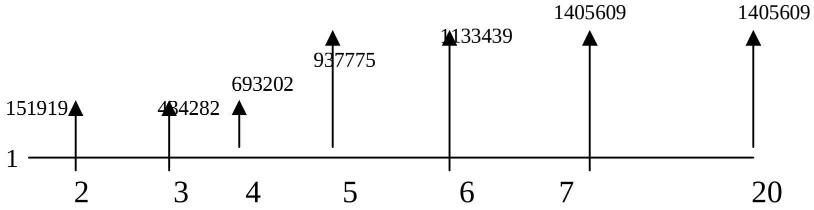
1.	Value of Production 'with project '	90,810	244,545	528,760	789,570	1,233.070	1,233,700	1,507,8
	Net value of Production 'without		94,656	94,478	96,368	100,261	100,261	102,26
2.	project'	90,810	151,919	434,282	693,202	937,775	1,133,439	1,405,6
	Incremental value of production							
3.	Costs	0						
	4.1. **Investments**							
4.	- Headwork							
	- Main canal							
	- Farm system	347,630						
	- Access road	481185.8						
	Camping & Mobilization	900,000						
	S/Total	40,000						
		50,000						
	4.2. **Operating costs**	1,818,815.8						
	- Seed							
	- Fertilizer							
	- Chemical	-	32,652	43,188	55,048	61,716	65,218	72,280
	- Lab our	-	57,280	73,892	93,598	105,184	111,070	123,54
	- Oxen	-	22,191	28,459	36,465	41,313	44,101	49,295
	- Repair &	-	3,427	4,452	5,654	6,365	6,750	7,561
	Maintenance	-	21,210	27,200	34,400	38,450	40,510	44,820
	4.3. Training	-	-	50,256	50,256	50,256	50,256	50,256
	4.4. Land use & income tax							
	4.5. Water charge	30,000	20,000	10,000	-	-	-	-
	Total cost	5,500	5,500	5,500	5,500	5,500	5,500	5,500
		-	6,000	6,000	6,000	6,000	6,000	6,000
5.		1854315.8	168,260	248,947	286,921	314,784	329,405	389,25

In order to decide weather the project is Visible or not we have used the benefit/cost ratio techniques and net present value evaluation criteria. And the results are as follows:

From table of Economic cash flow, the total cost and incremental value of production for each year is stated.

A) Present worth of benefit

$$P = \left[1 \big/ (1+i)^N \right] *F$$



Cash flow diagram

$\Rightarrow P = present\ Value$

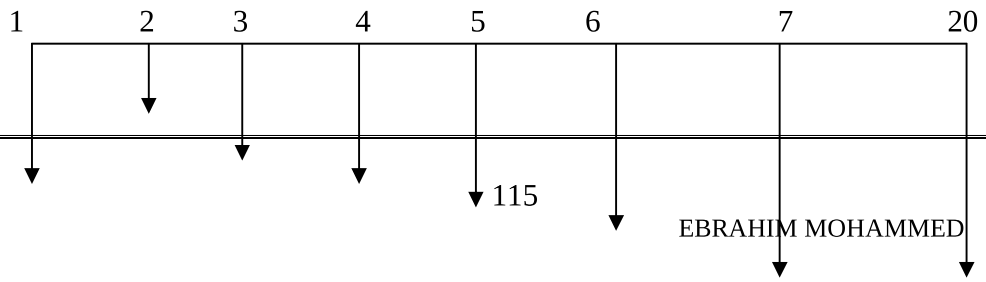
$$P=\left[\left(\frac{1}{1.105}\right)*151{,}919\right]+\left[\frac{1}{(1.105)^2}*434{,}282\right]+\left[\frac{1}{(1.105)^3}*693{,}202\right]+\left[\frac{1}{(1.105)^4}*937{,}775\right]+$$

$$\left[\frac{1}{(1.105)^5}*1{,}133{,}439\right]+1{,}405{,}609\left[\frac{(1.105)}{0.105(1.105)^{14}}{}^{14}-1\right]\left[\frac{1}{(1.105)^5}\right]$$

, $P = 137{,}483.26 + 355{,}670.0313 + 513{,}775.01 + 139{,}8130.678 + 1687{,}997.344 + 6117626.63$

$P = 9{,}210{,}682.94\ birr \qquad (benefit)$

B) Present worth of investment and operating costs.



168260

1818815.8 248947 286921 314784 329405

359252

359252

$$P = -[1,818,815.8 + [\frac{1}{(1.105)} * 168,260] + \left[\frac{1}{(1.105)^2} * 248{}^2947\right] + \left[\frac{1}{(1.105)^3} * 286{}^2$$

$$\left[\frac{1}{(1.105)^4} * 314,784\right] +$$

$$\left[\frac{1}{(1.105)^5} * 329,405\right] + 359,282 * \left[\frac{(1.105)^{14} - 1}{0.105 * (1.105)^{14}}\right]\left[\frac{1}{(1.105)^5}\right]$$

$$P = 1,818,815.8 + 152271.4932 + 203883.6224 + 212654.9527 + 211136.6068 + 1$$

$$P_{T = -4307,653.082\ birr} \qquad (\cos t)$$

$$\Rightarrow\ Benfit / \cos t$$

$$= \frac{9,210,682.94}{4,307,653.082} = 2.1 > 1 - - - - - OK$$

$$There\ \ fore, the\ project\ is\,Visible$$

Net present Value E vaiuation

- This technigue is nothing ,but comparing the present worth of the benefits with the present worth of investment and operation

$$\Rightarrow P = -$$

$$P = 9,210682.914 - 4,343,152.37$$

$$= 4,867,530.54\ birr$$

$$There\ fore,\ the\ project\,is\,accep\,table$$

10.0 ENVIRONMENTAL IMPACT ASSESSMENT

10.1 GENERAL:-

An environment is a complex system consisting of physical biological and socio – economic sub systems and the construction of water resource development project results in a variety of impacts, both beneficial and adverse on environment.

Environmental impact assessment (ELA) is an examination, analysis , prediction, evaluation and assessment of planned action (activities) with a view to ensure environmentally sound and sustainable development. This is no matter, how carefully planned, designed built commissioned, operated.

Every major project has an impact on the environments positively or negatively, which ever the case may be. It is there fore, important and their environments consequences and incorporate an Environmental impact assessment into decision making as an integral component in the design of projects, rather than some thing utilized after the design phases is complete

Thus, it is in height of effects on the environment and development areas that it would be critical to undertake environmental impact assessment of projects in a comprehensive and adequate manner.

10.2 **Purposes of EIA**

EIA is a process with several important purpose. The purpose of EIA is to deter mine and present the environmental impacts of a proposed project, plan or policy in such away that rational decision can be made about its implementation. There fore it is useful to

- Determine and present the Environmental impact of a proposed project
- Generate alternative projects to reduce or mitigate adverse impacts.
- To optimize the project design by early identification of alternatives.

10.3 DESERIPTION OF POTENTIAL IMPACTS

10.3.1 Positive impacts

The following may be considered as positive impacts of the project under consideration

SOCIO – ECONOMIC Impacts

This includes such benefits as

- The increase in crop production and therefore, self sufficiency in food production.
- Advantage of job, created due to construction and operation activities.

- Better social interaction among farmers and sharing of experiences among each other
- Improvements in the living standards of farmers. Protection from famine.

Physical environmental impacts

Some of the positive physical environmental impacts are:-

- protection from drought
- Hydrological change in the river system
- change (improvement) in the micro – climate of the irrigation

10.3.2 Negative Impacts

In contrast of the positive impacts, the following are some of the negative impacts considered.

<u>Social impacts</u>

Some social problems may be arises due to the implementation of the project. i.e Land allocation and water distribution if not properly handled, can create conflict among the farmers. Moreover, Malaria and other related diseases can attack people if in any case, water is collected and the area becomes marshy.

<u>Physical environmental impacts</u>

Water logging

During Irrigation some amount of water losses due to percolation.

The excess water that percolated in to the ground may raise the water table, which in turn reaches to the ground surface and can cause salinity and water logging in the soil.

Deforestation

Trees were cutted from the project site. I.e for access road, construction material, etc

- Reduction of down stream flow
- Loss of Land

Mitigation Measures required

It is difficult to eliminate the negative impacts of the project. However, there has to be some measures to reduce these adverse effects. There fore, the following mitigation measures can minimize the adverse effects.

- Creating awareness among farmers so that no conflict may raise
- Providing appropriate surface drainage to prevent salinity and water logging
- Provision of public health centers and pesticide chemicals to minimize the spread of water borne diseases and other related diseases.
- Discouraging irrigation during rainy season and properly applying the irrigation water
- Planting trees in place of cut down trees.

11.0 CONCLUSION AND RECOMMENDATION

Based up on the study and the results obtained; the following conclusions and recommendations have been made about the project.

- ❖ The geology of the head work area needs a treatment, since it has a significant transmit of Water
- ❖ The river called furfuro is un gauged so that to determine the peak discharge of the river, the soil conservation techniques is adopted.
- ❖ According to the feasibility report, the type of weir adopted is ogee weir but from the Economic point of view as well as easiness for the construction this project paper adopted a broad crested weir.
- ❖ The necessary metrological data are not fully available and hence metrological data of Alaba. Kolito is used.
- ❖ The computation of ETo of the crop is done by penman - monteith computer program since it is more accurate.
- ❖ From financial Analysis, the project is beneficial to be implemented.
- ❖ It is better to have full information about the down stream eco- system in order to mitigate environmental impact.
- ❖ The implementation of furfuro Irrigation project is essential to over come the adverse effects of erratic rainfall distribution on farming by increased food production.
- ❖ The diverted water from the river is adequate enough to satisfy the crop water requirement with out having any storage structure.
- ❖ The design of the tertiary canals gives the smaller dimension which is not physically practical to construct canal; so that it is better to leave it for the users.
- ❖ If necessary data were available, it was possible to work more than what is done.