

HARE IRRIGATION PROJECT DESIGN REPORT



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Submitted in partial fulfillment of the requirements
for the degree of Bachelor of Science in Engineering

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Arbaminch Water Technology Institute
Arbaminch, Ethiopia
1995

ABSTRACT

The Hare irrigation project consists of detailed design of an irrigation scheme that comprises the design of weir structure, channel layout, drop structures, silt excluders and other ancillary infrastructures. The existing project is located in the Northern Omo administrative region of Southern Ethiopia. It is located 10 km northwest of Arbaminch city. The project comprises of diversion of Hare river through the construction of diversion head works, canals, drop structures, culverts and turnouts to irrigate a command area of about 1000ha.

The objective of this study was to assess the design aspect of Hare irrigation project by applying the principles of weir and irrigation infrastructure design procedures. In order to design the weir, the maximum flood and the minimum dry flow of the Hare River were studied. The irrigation water requirement was estimated based on FAO estimation procedures.

The geographical location, geological formation, topography, soil type and properties, climate, the monthly diversion requirement to the proposed crops, the hydro meteorological analyses for the estimation of maximum probable flood and estimation of 1 in 5 year's dry period of flows were analysed for this study project.

Layout of primary canals, secondary canals, field size and tertiary drains are designed using both Permissible velocity criteria and Regime theory design procedures. The results of both methods were analysed and compared to come up with better estimates. Detail design of canal falls, canal off takes such as cross regulator and distributaries head regulators are designed. The design of hydraulic structures that include the diversion head work, type and site selection of the weir, design of under-sluice and other ancillary structures are implemented.

Finally, the results obtained in this study project and the information obtained from the existing Hare irrigation project compared with regard to irrigation discharge amount, weir

height and length, canal layout and number of drop structures in the channel system. From the analyses, the designs found to be similar with reasonable differences.

ACKNOWLEDGEMENTS

We are grateful to Professor Pande B.B Lal and Mr Seifu Gebre Meskel for their invaluable advice and supervision on our project work. Special thanks to Mr Seifu Gebre Meskel who sympathise and encourage us to work hard by rendering his own books and giving sensible support to finish our project in time.

Special thanks are to be paid to the librarians, particularly for their assistance in searching reference materials. We are grateful to many helpful persons and particular credit is due to the co-operation of Hare River Irrigation project site engineers in giving some valuable information.

Finally, our gratitude goes to all who contributed towards the completion of this document.

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1. INTRODUCTION

The Hare irrigation project is located near Arbaminch town in northern Omo administrative region, Ethiopia, between $6^{\circ}, 30'$ to $6^{\circ}, 08'$ N latitude and $37^{\circ}, 33'$ to $37^{\circ} 37'$ E longitudes. The average elevation of the terrain is 1200m above mean sea level (amsl), which goes down from Northwest to Southeast covering a total area of land equal to 1700ha. The irrigation area is located 10 km north of the Arbaminch town.

Since the project is located in the southern rift valley system, the geological formation is characterised by the rift valley system of the northern Omo region. The surface layers of the riverbed and flood land being 0.4-0.6 m thick; it is formed by present alluvial fine silt, sandy and sandy loam soils. Beneath it, there is unconsolidated sand and gravel layer of poor sorting. The underlying rock that is mainly basalt and small amount of volcanic debris, do have strong permeability. No cropped out bed rocks are observed at the diversion or weir site.

The topography of the irrigation area with its front edging inclined to the Abaya Lake and its back edging is located in the alluvial plain region. The plain terrain is unclosed and slopes gently. The terrain rises and falls locally. In the plain terrain, their still remains small and short gullies because of water erosion. In general, the topography of the Hare river area appears favorable for surface irrigation. Hare river originates from the Chench Mountains with a height of more than 3000m amsl, and goes through gorges towards the south in to Lake Abaya. The average gradient of the stream is 3.5 percent.

The soil type and characteristics of the out cropped stratum in the irrigation area is alluvial in the upper plains, which is commonly a combined variety of sand and gravel layer, medium fine sand, sandy loam soil. According to the soil profile survey conducted, the surface layer of the irrigation area is 0.3-0.5m thick. It is mainly formed by fine silt & sandy clay or loam soil. The layer underneath to the depth of 5m is formed by loam or sandy clay that contains a small amount of gravel. Primary consideration should be given to land suitability and capability evaluation in planning of an irrigation project. Land classification is done on the bases of the fertility and productivity of the land. Hence, a land can be called either suitable or unsuitable for the given purpose. Decision regarding land classification and evaluation requires sufficient soil data. Due to limited availability of soil data, detailed evaluation could not be carried out;

however on the basis of visit to the area and general information gathered, some general recommendation can be drawn.

The irrigation area in general presents no problem for development since variations within the area are small. The soils in the Hare irrigation project are medium textured and composed of organic matters which are moderately permeable. The infiltration rate of the soil in the project area ranges from 10-30 mm.h⁻¹. The available water holding capacity of the soil estimated to be 16.5cm.m⁻¹ on the average. Most of the soils are neutral to alkaline soils with PH ranging from 7-7.5. Generally the soil is free from salinity and other toxic substance. Hence, the soil property of the area can be taken as favorable for plant growth.

The climate information for the project area taken from the near by weather station data record that have been recorded from 1987 to 1994. The observed data indicates that the climate being influenced by the monsoon from tropical zone, the annual precipitation of the area varies from 625 mm to 1054mm. There are two rainy seasons in a year in the project area. The first season is mainly from April to May and the second from September to October. The relative humidity of the area varies from 39.9percent in January to 72 percent in May. The range of actual sunshine hours in the area is in the order of 3 hrs in July to 10 hrs in December and January. The average yearly evaporation is recorded as 1480m.

As the electrical conductivity test of the Hare river reveals, it is favorable to irrigate many crops. As the study conducted by Viak Ab. Sweden technical team indicates that the electrical conductivity of the Hare river is 820 $\mu\text{s.cm}^{-1}$. The PH of the water is 8.2, the Sodium absorption ratio (SAR) is 0.5 and the Boron concentration is 0.06 mg.l⁻¹. This class of water is considered to be good and suitable for most plants under most soil conditions.

2. QUALITY OF IRRIGATION WATER

Just as every water is not suitable for human beings, in the same way, every water is not suitable for plant life (Kumar, 1987). Water containing impurities that are injurious to plant growth are not satisfactory for irrigation. The quality of irrigation water is very much influenced by the constituents of the soil to be irrigated. Particular water may be harmful for irrigation on a particular soil but the same water may be tolerable or even useful for irrigation on some other soil (Kumar, 1987). The various type of impurities that make the water unfit for irrigation are described as follows:

2.1 Sediment Concentration

The effect of sediment concentration in the irrigation water depends upon the type of irrigated land. When fine sediment from water is deposited on sandy soils, the fertility is improved. On the other hand, if the sediment has been derived from the eroded areas, it may reduce the fertility or decrease the soil permeability. Sediment water creates troubles in irrigation canals, as they get silted up and increase the maintenance cost. Generally, ground water or surface water from reservoirs does not have sufficient sediment to cause any serious problems in irrigation (Punmia and Pande, 1981; Kumar, 1987).

2.2 Total Concentration of Soluble Salts

The presence of salts of calcium, magnesium, sodium and potassium in irrigation water may be injurious to plants when it is in excessive quantities. Salts reduce the osmotic activity of the plant and prevent adequate aeration causing injuries to plant growth (Hansen and Israelson, 1979; Kumar, 1987). The concentration of salts in water may not appear to be harmful to plants but the concentration of salts which remain in the soil after the saline water is used up by the plants is much more injurious than the first. In other words, at the beginning of irrigation no harm may be evident but with time, the salt concentration in the soil may increase to a harmful level as evaporation concentrates in the soil solution (Kumar, 1987). Hence, the effect of salt in plant growth depends largely upon the total amount of salts in the soil solution.

The salinity concentration of the soil solution after the consumptive water (C_u) extracted from the soil is given by the formula (Kumar, 1987):

$$C_s = \frac{C_Q}{(Q - (C_u - P_{\text{eff}}))} \quad (2.1)$$

Where

- C_s = concentration of salt in soil solution,
- C_Q = total salt applied to soil with Q amount of irrigation water,
- Q = amount of water applied for irrigation,
- C_u = the total amount of water used for plant growth,
- p_{eff} = useful rain water, and
- $C_u - p_{\text{eff}}$ = used up irrigation water,

The critical salt concentration in the irrigation water depends upon many factors, yet however, amounts in excess of 700ppm are harm full to some plants and more than 2000ppm are injurious to all crops (Kumar, 1987).

The salt concentration is generally estimated by measuring the electrical conductivity of water. There is a direct proportionality between the salt concentration and the electrical conductivity. Electrical conductivity is expressed in micro mhos per centimeter.

Table 2.1 Electrical conductivity of saline water (Kumar, 1987)

Salt concentration in micro mhos.cm ⁻¹ at 25 ⁰ c	Classification	Symbol
100-250	Low conductivity water	C1
250-750	Medium conductivity water	C2
750-2250	High conductivity water	C3
>2250	Very high conductivity water	C4

Table 2.2 Suitability of water for irrigation (Kumar, 1987)

Sr.no	Types of water	Use in irrigation
1	Low salinity water (C1) conductivity between 100 to 250 micro mhos/cm at 25 ⁰ .c	Can be used for irrigation for almost all crops and for almost all kinds of soils. Very little salinity may develop, which may require slight leaching; but it is permissible under normal irrigation practices except in soils of extremely low perm abilities.
2	Medium salinity water (C2) conductivity between 250-750 micro mhos/cm at 25 ⁰ c	Can be used, if a moderate amount of leaching occurs. Normal salt-tolerant plants can be grown with out much salinity control.
3	High salinity water (C3) conductivity between 750 to 2250 micro mhos/cm at 25 ⁰ c	Cannot be used on soils with restricted drainage. Special precautions and measures are undertaken for salinity control and only high -salt tolerant plants can be grown.
4	Very high salinity water (C4) conductivity more than 2250 micro mhos/cm at 25 ⁰ c	Generally not suitable for irrigation

2.3 Proportion of Sodium Ions in Soils

Most of the soil contains calcium, magnesium ions and small quantities of sodium ions. The percentage of sodium ions is generally less than 5% of the total exchangeable cations (Punmia and Pande, 1981; Kumar, 1987).

The proportion of sodium ions present in the soils is generally measured by a factor called sodium absorption ratio (SAR) and represents the sodium hazards of water. SAR is given by the formula (Kumar, 1987):

$$SAR = \frac{Na}{\sqrt{\frac{Ca+Mg}{2}}} \quad (2.2)$$

Table 2.3 SAR concentration in water its effect in irrigation (Kumar, 1987)

Sr.No	Types of water	Use in irrigation
1	Low sodium water (S1) SAR value lying between 0 - 10.	Can be used for irrigation on almost all soils and for almost all crops except those which are highly sensitive to sodium, such as stone-fruit trees and avocados, etc
2	Medium sodium water (S2) SAR value lying between 10-18	Appreciably hazardous in fine textured soils, which may require gypsum, etc. but may be used on coarse-textured or organic soils with good permeability.
3	High sodium water (S3) SAR value lying between 18 to 26	May prove harmful on almost all the soils, and do require good drainage, high leaching, gypsum addition for proper irrigation
4	Very high sodium water (S4) SAR value above 26	generally not suitable for irrigation

2.4 Concentration of Potentially Toxic Elements

As described by Kumar (1987), large number of elements such as Boron and Selenium may be toxic to plants. Traces of Boron are essential to plant growth but their concentration above 0.3 ppm is toxic to certain plants. The concentration above 0.5 ppm is dangerous to nuts, citrus fruits and deciduous fruits. Cotton and cereals are moderately tolerant to Boron, while Dates, Beets, Asparaguses are quite tolerant. Even for the most tolerant crops, Boron concentration should not exceed 4 ppm. Boron is generally present in various soaps. The wastewater containing soaps should therefore be used with great care in irrigation. Selenium in low concentration is toxic and must be avoided.

2.5 Bicarbonate Concentration

High concentration of Bicarbonate ions may result in precipitation of calcium bicarbonate and magnesium bicarbonates from the soil solution. Increasing the relative proportion of sodium ions causes sodium hazards to the plant (Kumar, 1987).

2.6 Bacterial Contamination

Bacterial contamination of irrigation water is not a serious problem unless the crops irrigated with highly contaminated water are directly eaten without being cooked. Cash crops like cotton, nursery stock that can be processed after harvesting, therefore contaminated wastewaters can be used without any trouble (Kumar, 1987).

2.7 Salinity and Ground Water Management

The level of ground water has obvious variation over the irrigation area. In the lower part, it has been found out that it is relatively higher. The ground water, if not monitored, after continuous supply of water may rise up and result in water logging, that may cause reduced crop yield. Removing obstruction in the path of natural flow and maintaining the slopes of drainage lines should improve the natural drainage of the area. Introducing the farmers with crop rotation can also minimize the problem. Those crops that require relatively less water should follow crops requiring more water.

The quality of irrigation water, though moderate to all crops, should not be ignored. Through time, the salt level may increase and reach the point beyond at which production becomes difficult. It is therefore recommended that:

- Crop selection should be based on the degree of tolerance to salinity,
- Reclamation of salt affected land should be practiced,
- The ground water level in the lower part of irrigation area should be monitored. The pattern of cropping has to be adjusted in such a way that tolerant crops like sugarcane and shallow root crops should be planted in areas where ground water is near to the surface, and
- Whenever water table is high, either drip or sprinkler irrigation system is a better option.

3. WATER DEMAND ASSESSMENT

According to FAO (1977), the crop water requirement is the total quantity of water required by a crop or diversified pattern of crop from the time of sowing to harvesting for its normal growth under field conditions. It varies from crop to crop as well as from place to place depending on climate, soil type, effective rainfall and agricultural practices.

As noted by FAO (1977), the water requirement of crop is obtained from the potential evapotranspiration (ET_o) by multiplying with the crop coefficient (K_c). Potential evapotranspiration is the evapotranspiration from an extensive surface of 8 to 15cm tall, green grass cover of uniform height, actively growing, completely shading the ground and not short of water at all times (FAO, 1977). The crop coefficient is used to relate the potential evapotranspiration with the crop evapotranspiration so as to take in to account the effect of crop characteristics and other factors on crop water requirement.

Several methods exist to estimate the potential evapo-transpiration (ET_o) of a reference crop. In this study project, considering different factors on which consumptive use depends such as wind, humidity, sunshine and elevation a modified Penman method is adopted.

Since the crop water requirement varies with the stages of crop development, crop stages considered for the design purpose as the K_c values varies with crop development stages.

The four stages of crop development are described as follows (FAO, 1977):

- Initial stage: germination and early growth stage that is when the soil surface is not or hardly cover by the crop (ground cover 10%),
- Crop development stage: from the end of initial stage to the attainment of effective full ground cover. (Ground covers approximately 70-80 %),
- Mid season stage: from attainment of effective full ground cover to the start of maturity, and
- Late season stage: from the end of mid season until full maturity of harvest.

The steps to calculate the K_c values for the different stages of crop development described as follows (FAO, 1977):

- Establishing planting or sowing date from local information,
- Determining the total growing season and length of crop development stages,
- Initial stage,
 - The irrigation frequency for predetermined ET_o values is predicted, and then the K_c value is obtained as shown in Figure 8.1,
- Mid season stage,
 - For a given climate (humidity and wind speed), select K_c value from Table 21 (FAO publication No 24) and plotting straight line would give the K_c value in the mid season stage.
- Late season stage,

For the time of full maturity to harvest with in a few days, select K_c value from Table 21 (FAO publication No. 24) for a given climate (humidity and wind) and plot values at the end of growing season or full maturity. Assume straight line between K_c value at the end of mid season period and at the end of growing season, and
- Development stage

Assume a straight-line between K_c values at the end of initial to start of mid season stage.

3.1 Net Irrigation Requirement

The net irrigation requirement (NIR) is the depth of water excluding the natural moisture supply needed to meet the evapotranspiration of the crop (FAO, 1977). The NIR can be calculated using the field water balance equation. The variables of field water balance equation described as follows:

- Crop evapotranspiration (ET_c),
- Effective rain fall (P_e),
- Ground water contribution (G_e), and
- Stored water at the beginning of each period (W_b).

Due to the variable nature of ground water table over the irrigation area, the contribution is variable from one area to the other. Based on this information, cropping pattern is adjusted in

such away that there is no harm to the crop. Since, it is believed that the soil moisture condition has no significant contribution to the irrigation requirement; G_e and W_b are not considered in the design. Hence the NIR can be calculated using Eq 3.1(FAO, 1977; Hansen and Israelson, 1979).

$$\text{NIR} = \text{ET}_c - \text{P}_e \quad (3.1)$$

Where

NIR	=	net irrigation requirement,
ET_c	=	crop evapotranspiration, and
P_e	=	Effective rainfall.

Effective rainfall (P_e) is a portion of the rainfall that can be stored in the root zone. It can be estimated by the evapotranspiration (precipitation method) (FAO, 1977). For the estimation of effective rainfall (P_e), the pertinent factors considered are: -.

- Rainfall during the growing period of the crop,
- Effective storage, and
- Crop water requirement.

3.1.1 Effective storage

Different plants have different root depth; especially young plants have much shallower rooting systems than mature plants (Hansen and Israelson, 1979). Shallow soils contain shallow root systems. Hence, root patterns depend much on local soil conditions and water availability (Hansen and Israelson, 1979).. Excessive irrigation maintaining high water table inhibits root growth. Whereas drought conditions with water availability at considerable depth would encourage deep roots. Generally, most of the water used by plants is taken from the upper half of the root zone and because of this only about half of the available water is actually used. Hence effective storage can be estimated using Eq 3.2 (FAO, 1977; Hansen and Israelson, 1979):

$$\text{Effective storage} = 50\% \text{ AMC} \times \text{rooting depth} \quad (3.2)$$

Table 3.1 Root depth and percentage of available water use (FAO, 1977)

Root zone depth	Percentage of available water used
First quarter	80%
Second quarter	60%
Third quarter	40%
Fourth quarter	20%
Average	50%

Table 3.2 Correction factor for effective storage

Effective storage	20	2.5	37.5	50	62.5	75	100	125	150	175	200
Storage factor	0.73	0.77	0.86	0.93	0.97	1	1.02	1.04	1.06	1.07	1.08

3.2 Gross Irrigation Requirement

Gross irrigation requirement is the total amount of water to be diverted at the head regulator to meet the required demand (Hansen and Israelson, 1979; Stersen HP, 1985). Other than meeting the net irrigation requirements land preparation, leaching requirements, cultural practices, channel and evaporation losses have to be considered in applying irrigation water (Hansen and Israelson, 1979; Stersen HP, 1985).

The quality of water as well as the soil of the Hare irrigation area is found to have no significant salinity problem. Hence no extra water is needed for leaching. Further, it is assumed that small quantity of salt that may tend to accumulate can be leached away by the irrigation inefficiencies (application losses). Water needs for land preparation and quality control of the harvested yield are not considered assuming adjustment of irrigation scheduling usually covers water for land preparation and quality control.

3.3 Irrigation Efficiency (E)

The performance of an irrigation scheme indicated by comparing input and output analyses. It is required to account losses of water incurred during water application.. It is also required to account losses of water incurred during conveyance, application and distribution (FAO, 1977).

Efficiencies at different levels of irrigation application and distribution described as follows:

Conveyance efficiency (E_c)	=	80%
Application efficiency (E_a)	=	70%
Field canal efficiency (E_f)	=	80%
Project efficiency (E_p)	=	$E_c * E_a * E_f$

The monthly supply requirement for a given area can be determined for each crop by Eq 3.3 (Hansen and Israelson, 1979):

$$V_i = \frac{(10A_i * I_t)}{E_p} \quad (3.3)$$

where

V_i	=	monthly volume of water required (m^3),
A_i	=	area (ha),
I_t	=	total quantity of water required to fulfill the NIR ($mm.month^{-1}$), and
E_p	=	project efficiency.

As the area of Hare irrigation project is surrounded by trees and lakes, the wind blowing over the irrigation area is almost saturated and needs no further uptake of moisture from the area. This has some effect in reducing the evapotranspiration of the crop. Hence, reduction of ET_c by 20% is done in this design to minimize an over estimation.

3.4 Drainage

In designing an irrigation project effective utilization and control of an irrigation water is the most significant step that ensures successful yield. This can be achieved through proper design of network of irrigation canals and drainage canal that disposes off the excess water from the applied irrigation water and excess rain fall as a runoff.

Adequate drainage has the following advantage (Kumar, 1987):

- improves soil structure and increases the productivity of the soil,
- leads to early ploughing and planting,
- lengthens the crop growing season,
- decrease soil erosion and gulling by facilitating water infiltration in to the soil,
- reduces the water table in an area there by:
 - providing more available plant food by increasing depth of root,
 - increases soil ventilation,
 - favors growth of soil bacteria, and
 - assures high soil temperature.

Open drains are best because of easy of construction, availability of construction materials, less inspection and maintenance requirement. The Hare irrigation area provides good ground slope with defined drainage lines so as possibly the layout of the drains can follow natural drainages.

The collector is designed in such away that it collects the water from the field and accommodates the runoff from the basin. The amount of runoff can be estimated by the Dredge or Burge formula as shown in Eq 3.4.

$$Q_p = 19.6 * A/L^{(2/3)} \quad (3.4)$$

$$A = B * L \quad (3.5)$$

Where

- Q_p = peak run off ($m^3 \cdot s^{-1}$).
- A = area of the drainage basin (km^2),
- L = length of the basin from the furthest point to the outlet (km), and

B = average width of the basin (km).

4. HYDROLOGICAL ANALYSIS

As noted by Linsley (1984), most decisions regarding the management of water resource are based on the estimate of the quantity of water to be managed or the rate of flow to be regulated. Hydrology is therefore a basic tool to the planning of water resource project and to the subsequent operation of the projects. Any water resource system must be planned for future events with no exact time of occurrence can be forecasted. Thus hydrological analysis must give the probability that a stream flow would equal or exceed a specified value with a reliable accuracy. It is important to emphasize that reliable hydrological analysis is essential to justify economic evaluation. This requires adequate and accurate relevant data. The main problem arises in the estimation of frequencies of peak flow is that the amount from the interpretation of past records of hydrological events in terms of future probability of occurrences.

For the estimation of probable maximum flood for a certain recurrence interval, the momentary peak flow of each year must be known. If the data available for the project does not have momentary peak flow for some of the years, the missed data can be calculated from Figure 4.17 of Linsley (1984) using series of curves. The curves represent lab-led ratio of instantaneous peak flow to maximum daily discharge.

4.1 Estimation of Maximum Probable Flood

The maximum probable flood is the largest flood expected while taking into account all pertinent condition of location, meteorology, hydrology and terrain. It is extremely large flood indeed, hence seldom would be used in the design of hydraulic and irrigation structures. To insure safety, it is customary to investigate the maximum expected flow in the stream while designing any structure (Linsley, 1984).

No method is available by which the exact amount of the expected flood can be predicted with absolute certainty and precision. Various method have been used for estimating peak flood, some of which are based on characteristics of the drainage basin and others based on the theory of probabilities of past data records. Some of the methods are described as follows.

4.1.1 Gumble analytical method

In Gumbel's analytical method, the maximum probable flood can be calculated by using analytical values of frequency factor K (Kumar, 1987).

The mathematical equation is given by Eq 4.1(Kumar, 1987).

$$Q_p = \frac{\bar{Q}_p}{n} + K\sigma \quad (4.1)$$

where

- Q_p = peak discharge for any given frequency ($m^3 \cdot s^{-1}$),
- \bar{Q}_p = average of observed peak discharge,
- n = number of data record years,
- K = frequency factor, given by $K=0.7797y-0.45$, and
- σ = standard deviation of momentary peaks.

y can be calculated as follows:

$$y = -\ln[-\ln(1 - \frac{1}{T})] \quad (4.2)$$

where

- T = return period (years)

4.1.2 Log-Pearson type III distribution

For the prediction of peak flows using statistical distribution thousands of samples are required. No such samples are available in our case and it is not possible to predict with certainty that a specific distribution applied to flood peaks.

The calculation procedures followed in the Log-Pearson type III distribution is to convert the data series into logarithm scale. The equations are stated as follows:

$$\text{Ave Log } x = \frac{\text{Sum of Log } x}{n} \quad (4.3)$$

$$\sigma_{of \log x} = \sqrt{\frac{(\text{Log}x_i - \text{Ave} \log x)^2 + (\text{Log}x_{ii} - \text{Ave} \log x)^2}{n-1}} \quad (4.4)$$

$$G = \frac{n \cdot \sum (\log x - \text{Ave of } \log x)^3}{(n-1)(n-2)(\sigma \log x)^3} \quad (4.5)$$

Where

- x = the maximum daily flow ($\text{m}^3 \cdot \text{s}^{-1}$),
n = number of years of records
 σ = standard deviation, and
G = skew coefficient.

The peak flood of a specified return period can be calculated using Eq 4.6 (Linsley, 1984).

$$\text{Log} (Q_T) = \text{Ave of } \log x + K\sigma \log x \quad (4.6)$$

where

- Q_T = maximum flood with a specified probability.

Where K can be selected from Table 13-4 (Linsley, 1984) for the computed value of G and desired return period (T) (Linsley, 1984).

4.1.3 Empirical formulae.

Many formulas have been derived to estimate flood peaks. Empirical formulae can be applied to areas where they are specifically derived. Some of the empirical methods are described as follows (Kumar, 1987).

➤ Fuller's formula

The formula takes in to account the flood frequency and drainage area (Kumar, 1987).

$$Q_p = C \cdot A^{0.8} (1 + 0.81 \log T) (1 + 2.67 A^{-0.3}) \quad (3.7)$$

Where

- Q_p = peak flood that come during any part of the day in T years time ($\text{m}^3 \cdot \text{s}^{-1}$),

- A = area of drainage basin in (km²), and
 C = constant varying from 0.185 to 1.3.

➤ **Regression analysis study of 42 catchments in Ethiopia that ranges in size from 200 to 9980 km² conducted by Admasu Gebeyehu. Gebeyehu derived Eq 4.8 that can be safely applied in most river basins of Ethiopia.**

$$Q_T = Q(1 + K_T * C_v) \quad (4.8)$$

where

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

- T = recurrence interval,
 Q = 2.44 (A)^{0.61},
 A = catchment area (km²),
 K_T = frequency factor, and
 C_v = 0.38 for most cases (average coefficient of variation).

Various methods have been used to estimate the peak flood. The result of applying Gumble analytical and Log Pearson Type III distribution methods depend upon the long time past records. The limitation of these methods lies in the fact that they are based on the past records (Kumar, 1987). If sufficient past records are available and no appreciable changes seen on the regime of the stream during or after the period of record, the methods would yield precise results. In this project there is no sufficient past records and hence, the peak flood obtained from both Gumble analytical and Log Pearson Type III distribution methods would be used as a preliminary guidance.

The other methods used to estimate the design flood based on the characteristic of drainage areas are the empirical formulae. Because of the fact that the magnitude of specific flood occurrences depend upon many factors, none of the empirical formula expected to yield precise results.

4.2 Selection of Dependable Flow

Analysis of hydrological records has been made to find the most severe period of records, since design is based on this single extreme period of record, probability of occurrence of the given records made on the assumption that the meteorological condition recorded in the past would be repeated. A statistical analysis is made with an arrangement in order of magnitude of records of data from which a probable frequency of a stated condition can be estimated. The monthly flow is ranked to yield 80% probable flow for the year of record, irrespective of its year of records are selected as the over all dependable flow for the demand assessment.

5. CANAL AND RELATED STRUCTURES

A direct irrigation scheme requires a network of canals to supply water to the command area. A well designed irrigation scheme delivers the required amount of water to all points of irrigation area at a required rate with out damaging the soil and causing excess water loss (Kumar, 1987).

Planning a network of canals and their layout for an irrigation system is influenced by the relative elevation of water supply, the topographic nature of the land between the source of water and intended irrigation area. Further, the size and topography of the command area also affect the canal network system.

5.1 Canal Alignment

The canal system consists of main canal, branch canals, distributaries and minors. The primary and secondary canals should be aligned along the ridges for efficient use of canal network system. Part of the irrigation project area is steep slope. Steep slop surface may need a large number of canal falls to distribute the water in to the field.

5.2 Field Size and Layout of Distributaries

The size of the field is important to ensure reasonably uniform distribution and efficient utilization of water in the field. In general, the field should be as large as possible to make use of fewer farm structures such as tertiary canals, drop structures and off takes to reduce the initial investment cost. The type of soil and method of irrigation affect field length. Generally, furrows can be adapted from 300 to 400m length, based on the slope, texture of the soil, infiltration rate and furrow spacing (FAO, 1977).

Based on the following condition, the length and capacity of field canals can be specified:

Any water delivery system may be affected by the following prevailing conditions (FAO, 1977; Kumar, 1987):

- Cost of land preparation, distribution works and farm work,

- Restrictions on use of farm machinery,
- Erosion hazards,
- Crop type and crop patterns,
- Efficiency of water application, and
- Suitability for accurate measurement and mechanisms of controlling water application.

Most of the tertiary canals, feeders to the furrows should be aligned with the contour of the surface. Aligning these canals along the contour line helps in minimizing number of canal structures.

5.3 Canal Design

Design of a canal involves determination of hydraulically efficient, stable canal section and non-silting and non-scouring velocity of flow in the channel. This is because of the fact that when a canal silts up its capacity reduces. In addition, when the bed and sides of a canal are eroded away, the cross-section increases as a result its full supply depth decrease. In both case, the canal will irrigate less area than the proposed irrigation area.

To design the canal systems two approaches can be considered: the first one is unlined section based on regime theory and the second one is lined section based on the maximum permissible velocity of 0.7m.s^{-1} to 1m.s^{-1} .

6. HEAD REGULATORS AND CROSS REGULATORS

Head regulators and cross regulators regulate the supplies of the off taking and the parent channel respectively. The distributaries head regulator is provided at the head of the distributaries and controls the supply entering the distributaries. It is a necessary link between the parent canal and the distributaries canal. The distributaries head is a regulator, measures water supply and excludes the silt from entering in to the distributaries. A cross regulator is provided on the main canal at the down stream side of the off take to head up the water level and to enable the off taking channel to draw the required supply.

Functions of distributaries head regulators:

- Regulate or controls supplies to the off taking canal,
- Serve as a meter for measuring the discharge entering in to the off taking channel,
- Control the silt entry to the off taking canal, and
- Help in shutting the supplies when no more water is needed in the off taking canal, or when the off taking channel is required to be closed for repairs.

Functions of cross regulator:

- The effective regulation of the whole canal system can be done with the help of cross regulator,
- During the periods of low discharges in the parent channel the cross-regulator raises water level of the upstream and feed the off take canal in rotation,
- Helps in closing the supply to downstream of the parent channels for the purposes of repairs, and
- Helps in absorbing fluctuation in various sections of the canal system and prevent the possibilities of breaches in the tail reaches.

6.1 Diversion Head Works

The diversion scheme comprises hydraulics structures that are provided at the head of the canal works to supply water to the off taking canal. These structure help to regulate flow in to the canal and regulates the water level in the river so that the canal would be enabled to draw the required amount of water with the available sufficient head. Especially during low flow season when the level of water in the river is very low to feed the canal, the head regulators play important role.

The design of hydraulic structures includes the design of weir portion, canal head regulator, under-sluice and other appurtenance works such as divided wall, marginal bund and other ancillary works.

6.1.1 Divide wall

The divide wall is masonry wall constructed perpendicular to the axis of the weir, and separates the weir from the under-sluice. The divide wall extends on the upstream side beyond the beginning of the canal head regulator and on the downstream side it extends up to the end loose protection of the under-sluice.

As noted by Kumar (1987), the main function served by the divided wall is described as follows:

- The floor level of the under-sluice or the pocket portion is generally kept lower than the floor level of the weir. Hence, a divide wall is essential to separate the two floors. This prevents the turbulent action,
- If the divide wall is not provided, the flow currents approaches the scouring sluice from all directions and their effectiveness is reduced. The divide wall helps in concentrating scouring action of the under-sluice for washing the silt deposited in the pocket by ensuring a strait approach through the gate,
- The divide wall prevents cross currents and parallel to the weir that leads to the formation of vortices resulting in deep scour, and

- The divide wall provides a comparatively stilling pocket in front of canal head regulator. This helps in more silt deposits in the pocket and entry of clear water in to the canal.

The divide wall extends in the upstream direction and downstream direction. The top of the divide wall is kept at the crest level of the weir. The divide wall is placed on well foundation that is well below the deepest scour depth.

6.2 Selection of Weir Site and Weir Type

Proper site selection is a basic criterion for the design of any structure. Site selection for the construction of weir should fulfill the following requirements:

- availability of suitable foundation material,
- easy of accessibility,
- minimum idle length of canal, and
- site that can facilitate easy of construction.

6.3 Design of Weir

Complete design of vertical drop weir consists of:

- hydraulic calculations for fixing various elevations,
- design of weir wall,
- design of impervious apron and other protection works, and
- design of inverted filter and downstream talus.

For the design, the following hydraulic data are required:

- maximum discharge(Q peak) in the river,
- river bed level,
- full supply level (FSL) of the off take canal from the river, and
- bed level of the off take canal.

6.4 Design of Weir Wall

The design involves estimation of both top and bottom width so that the weir would be safe under worst condition of loading. The weir should be checked when the water is at pond level, when there is no flow at the downstream side and high flood conditions.

Top Width (a)

The top width is determined from different considerations

- from consideration of stress criterion of the weir,

$$a = \frac{d}{\sqrt{G}} \quad (6.1)$$

$$d = U/s \text{ HFL -crest level}$$

where

$$G = \text{specific gravity of material of the weir (} G = 2.24 \text{ for stone masonry).}$$

- from sliding consideration,

$$a = \frac{d}{G \cdot 1/\mu} \quad (6.2)$$

where

$$\mu = \text{coefficient of friction (} \mu = 2/3 \text{ is safe value).}$$

This expression has been obtained on the consideration of no sliding criterion for determining the width of an elementary or triangular profile of a weir.

Bottom width (b)

- When water is at pond level with no tail water, the overturning moment is given by:

$$M_0 = \frac{(W*H^3)}{6} \quad (6.3)$$

where

$$\begin{aligned} M_0 &= \text{over turning moment (t-m.m}^{-1}\text{)}, \\ W &= \text{unit weight of water (=1t.m}^{-3}\text{), and} \\ H &= \text{head of water (m)}. \end{aligned}$$

The resisting moment about the outer third point of the base fulfilling the middle third rule is given by:

$$M_r = \frac{W}{12} \left[((1.5+G)*Hb^2) + (a(GH-H)b) - (0.5*a^2(H)) \right] \quad (6.4)$$

Equation 6.4 is valid when the up stream and down stream faces have the same slopes. However, if the upstream face is kept vertical the moment of resistant would increase and given by Eq 6.5 (Punmia and Pande, 1981; Kumar, 1987).

$$M_r = \left[\frac{WHG}{6} \right] * (b^2 + ba - a^2) \quad (6.5)$$

Equating M_0 and M_r solves b

When the water is flowing over the weir, the weir would be submerged and when the tail water is just at the crest of the weir, the over turning moment is given by Eq 6.6.

$$M_o = \frac{WdH^2}{2} \quad (6.6)$$

Where, d is head over the crest.

When the tail water is at the crest, d and H will be equal. For this case the value of d (head over the crest) is given by.

$$d = \left(\frac{q}{(2/3) * C_d * \sqrt{2g}} \right)^{2/3}, \quad (6.7)$$

where

$$\begin{aligned} q &= 3.07, \text{ and} \\ C_d &= 0.58. \end{aligned}$$

The moment of resistance is given by equation

$$M_r = \frac{(WH(G-1))}{12(B^2+aB)} \quad (6.8)$$

When high flood is flowing, downstream (d/s) water level after retrogression of 0.2m (assumed). Hence, the over turning moment is given by, M_0

$$M_0 = \frac{W}{6(H^3+3dh^2-z)} \quad (6.9)$$

where

$$\begin{aligned} W &= \text{unit weight of water (=1t.m}^3\text{)}, \\ H &= \text{crest level -river bed level,} \\ D &= \text{depth of water above crest level =1.43, and} \\ z &= \text{depth of water d/s of the weir (z = 2.67m (for no submergence))} \end{aligned}$$

According to the Bligh's formula, the basic section of the weir body can be determined as follows (Punmia and Pande, 1981).

$$\text{Bottom width, } L = \frac{(H+H_e)}{\sqrt{(\rho-1)}} \quad (6.10)$$

$$\text{Top width, } B = \frac{H_e}{\sqrt{(\rho-1)}} \quad (6.11)$$

Where

$$\begin{aligned} H &= \text{height of weir in meter,} \\ H_e &= \text{specific energy head (sum of over flow depth and} \\ &\quad \text{approaching velocity head (m), and} \\ \rho &= \text{specific weight of weir body (specific gravity =2-2.3).} \end{aligned}$$

If the weir body is not submerged completely by the down stream water, (ρ) should be used instead of $(\rho-1)$. The weir should be checked for various conditions under worst scenarios of loads including an earth quake forces.

6.5 Design of Under-sluice

Under-sluice is an outlets provided in the weir wall by the side of off taking canal. The main purpose of under-sluice is to provide a deep channel in front of head regulator for the dispose of heavy silt.

A comparatively less turbulent pocket of water is created near the canal head regulator. Generally under sluice serve the following functions.

- they define a clear and well defined river channel approaching head regulator,
- controls of silt entry in to the regulator,
- scouring of silt deposited in the pocket or approaching channel,
- passing of low floods with out dropping shutter (if any), and
- provide greater flood way thus lowering flood level.

6.6 Earth Quake Forces

An earthquake produces waves that are capable of shaking the earth upon which the structure is constructed. Earthquake waves may move in any direction and for the design purpose it has to be resolved into horizontal and vertical accelerations.

The worst scenarios of seismic loading occurs, when the horizontal foundation acceleration moves in upstream (u/s) direction and the vertical foundation acceleration moves down ward direction.

Hare project area located in seismic zone 3 of Ethiopia. The area has minimum to moderate damage level. For this region the values of X_h & X_v are given as (ESCP, 1983):-

Horizontal foundation acceleration, $X_h = 0.1g$

Vertical foundation acceleration, $X_v = 0.05g$,

Where, g is acceleration due to gravity.

The horizontal acceleration causes two forces; namely, the Hydrodynamic pressure and horizontal inertia forces where as the vertical acceleration causes vertical inertia forces.

Force analysis

➤ force caused by horizontal foundation acceleration:

Horizontal acceleration brings about the following two forces.

I, Hydrodynamic pressure: horizontal acceleration acting towards the reservoir causes a momentary increase in the water pressure as the foundation and dam accelerate towards the reservoir and the water resists the movement.

$$P_{emh} = X_h W \quad (6.12)$$

Where

X_h = horizontal foundation acceleration ($0.1g$),
 W = weight due to gravity, and
 P_{emh} = horizontal inertia force,

The hydrodynamic force acts at $4/3H$ of the weir $=0.424*H$

$$P_e = 0.555 X_h w H^2 \quad (6.13)$$

Where

w = unit weight of water $= 1t.m^3$, and
 H = head of water (m).

➤ Force due to vertical foundation acceleration

A vertical acceleration (X_v) may either act down ward or upward. When it is acting in the upward direction, then the foundation of the dam will be lifted upward and become closer to

the body of the weir and thus the effective weight of the weir will increase. Hence, the stress will increase.

When the vertical acceleration is acting downward, the foundation moves downward away from the dam body; thus reducing the effective weight as well as the stability of the weir. This is the worst case scenario for the design.

The acceleration, exert an inertia force given by.

$$\frac{W}{g} X_v \text{ (That is force = mass * acceleration)}$$

Weight can be expressed by mass and acceleration due to gravity as:

$$W = mg$$

Hence, the equation becomes $\frac{mg}{g} X_v = mX_v = \text{force}$

Where

$$W = \text{the total weight of the weir}$$

Therefore the net effective weight of the weir = $W - W/g * X_v$

$$X_v = 0.05g$$

Then, the net effective weight of the weir is given by

$$= W - W/g * X_v * g = W [1 - X_v]$$

$$= W - W/g * 0.05g = W [1 - 0.05]$$

The vertical acceleration reduces the unit weight of the weir material and that of water 0.95 times their original unit weights.

$$P_{emv} = X_v * W = 0.05W$$

where

$$P_{emv} = \text{vertical inertia force (N), and}$$

$$X_v = \text{vertical foundation acceleration (m.s}^{-2}\text{)}.$$

6.7 Hydropower Generation

In water resource development, multipurpose project is of high economic benefit than single purpose projects. Combining hydroelectric power plant with irrigation scheme results in much saving of the construction cost and increases project benefit. Besides water for power generation being non-consumptive, the two uses are quite compatible with each other.

The Hare river irrigation project site has steep topographic nature, especially in the upper reach of the area near the diversion head works. On the other hand, the canal bed slope is flat in accordance with the requirement of the permissible velocity in the canal. Because of these, the canal come in fill after short distances, but keeping canal in fill for long distances bears difficulty in construction and maintenance. Due to this reason canal falls should be provided in very short distances to adjust the difference and keep the canal in cut and fill.

An advantage design is recommended to utilize the available head for hydroelectric power generation. Combining a number of small falls can be provided. Here it is recommended that seven 1.5m height falls can be replaced by one big falls producing a potential head of H equals to 0.5m.

The potential power available is given by

$$P = \gamma QH \quad (6.14)$$

Where

- P = power (kw),
- γ = unit weight of water (=1000kg.m⁻³),
- Q = discharge (m³.s⁻¹), and
- H = available head (m).

To generate the power, a suitable type of turbine on the bases of the available head and quantity of water has to be selected. The head available i.e. 10.5m (low head range), is sufficient for small hydropower generation. Kaplan or propeller type of turbine is recommended for use for this head

7. DATA ANALYSES

Data records and analysed data are contained in Table 7.1 to 7.14.

Table 7.1 Mean monthly flow of Hare River

year	Mean monthly flow of Hare river (m^3s^{-1})											
	Jan.	Feb.	March	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1980	0.649	0.471	0.407	4.261	5.315	2.9	2.595	5.54	2.835	2.662	0.711	0.362
1981	0.169	0.429	1.724	2.96	4.36	1.226	3.136	2.07	3.58	2.815	1.741	0.649
1982	0.407	0.834	1.188	2.308	4.513	3.542	2.818	2.675	3.56	4.525	3.015	1.709
1983	0.794	0.593	0.579	3.015	5.865	6.045	4.315	5.595	5.285	6.195	4.025	2.171
1984	0.285	0.12	0.166	0.287	4.179	4.515	1.416	2.17	2.74	2.275	1.449	0.987
1985	0.478	0.906	0.883	3.264	5.421	2.09	6.27	2.485	5.19	1.398	2.172	0.2
1986	0.055	0.222	1.464	1.674	3.54	3.95	2.2	2.012	5.25	5.11	2.497	3.128
1987	0.938	0.712	6.272	7.19	6.645	5.065	1.92	1.829	2.13	4.65	2.175	0.959
1988	1.348	1.503	0.502	2.369	5.905	3.165	9.535	8.21	8.929	5.03	1.85	0.773
1989	0.917	3.437	4.297	3.755	5.32	2.11	3.995	2.865	5.99	25.29	1.938	3.714
1990	3.335	3.700	4.615	4.385	2.99	2.215	2.015	3.88	2.26	2.155	1.500	1.371

Table 7.2 Momentary peak flows of Hare river

year	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990
Q ($m^3.s^{-1}$)	19.14	11.7	10.1	14.01	10.95	41.11	18	14.45	19.6	41.45	16.83

Table 7.3 Monthly 1 in 5 dry year rainfall (mm)

Month	Jan	Feb	Mar.	Apr.	May	June	July	Aug	Sept	Oct	Nov	Dec
Rainfall	13.2	10.1	65.4	144.2	163.1	49.3	18.3	6.82	53.9	92.4	45	17.4

Table 7.4 Monthly flows 1 in 5 years dry period

Month	Jan	Feb	Mar	April	May	June	July	Aug	sep	Oct	Nov	Dec
Discharge($m^3.s^{-1}$)	0.285	0.429	0.502	2.308	4.179	2.110	2.015	2.170	2.740	2.275	1.5	0.649

Table 7.5 Growth stage and total growing period of selected crops in the Hare Irrigation project

Crop	Area (ha)	Planting date	Growth stage (I,II,III,IV)	Total growing period
Maize I	227	01-Mar	25/40/45/31	140
Ground Nuts	227	01-Sep	30/40/50/30	150
Sugar Beet	246	01-Jun	25/35/50/55	165
Pulses	246	01-Jan	20/30/40/20	110
Maize II	118	01-Aug	20/40/45/30	140
Tobacco	118	01-Feb	10/45/45/50	150
Cotton	91	01-Apr	30/50/55/45	180
Soybean	91	01-Des	15/15/40/15	85
Sugarcane	46	-----	-----	perennial
Banana	109	-----	-----	perennial
Fruits And Vegetables	73	-----	-----	perennial

Table 7.6 Monthly reference crop evapotranspiration (ET_o)

Month	Jan.	Feb.	March	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
ET _o (mm)	171	177	189	165	156	153	180	183	180	168	165	171

Table 7.7 Crop evapotranspiration and reference evapotranspiration

Month	Jan.	Feb.	March	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
ET _o (mm)	171	177	189	165	156	153	180	183	180	168	165	171
ET _c (mm)	102.6	106.2	113.4	99	93.6	91.1	108	109.8	108	100.8	99	102.6

Table 7.8 Longitudinal profile of primary canal

Dist. to fall (m)	G. level at fall (m)	U/S bed level(m)	D/S bed level (m)	U/S fsll (m)	D/S fsll (m)	Depth of flow(m)	Fall Height (m)
200	1229.5	1228.84	1227.64	1229.75	1228.55	0.91	1.2
300	1228.5	1227.54	1226.04	1228.45	1226.95	0.91	1.5
360	1227	1225.98	1224.48	1226.89	1225.39	0.91	1.5
430	1226.1	1224.43	1222.93	1225.34	1223.84	0.91	1.5
580	1225	1222.78	1221.28	1223.69	1222.19	0.91	1.5
670	1224	1221.19	1219.69	1222.1	1220.6	0.91	1.5
760	1223.6	1219.6	1218.1	1220.51	1219.01	0.91	1.5
860	1221.6	1218	1216.7	1218.91	1217.61	0.91	1.3
1080	1219.5	1216.4	1215.1	1217.31	1216.01	0.91	1.3
1180	1218	1215	1213.7	1215.91	1214.61	0.91	1.3
1250	1217	1213.6	1212.3	1214.51	1213.21	0.91	1.3
1320	1216	1212.2	1210.9	1213.11	1211.81	0.91	1.3
1390	1215	1210.8	1209.5	1211.71	1210.41	0.91	1.3
1490	1214	1209.4	1208.1	1210.31	1209.01	0.91	1.3
1590	1212	1208.1	1206.8	1209.01	1207.71	0.91	1.3
1700	1210	1206.68	1205.38	1207.59	1206.29	0.91	1.3
1850	1207.3	1205.24	1203.94	1206.15	1204.85	0.91	1.3
2070	1204	1203.74	1202.44	1204.65	1203.35	0.91	1.3
2220	1203.3	1202.3	1201	1203.21	1201.91	0.91	1.3
2360	1201.8	1200.9	1199.6	1201.81	1200.51	0.91	1.3
3460	1199.3	1198.6	1197.1	1199.51	1198.01	0.91	1.5
3760	1197.6	1196.8	1195.3	1197.71	1196.21	0.91	1.5
4060	1195	1195					

Table 7.9 longitudinal profile of secondary canal 1

Dist. to fall (m)	G. level at fall(m)	u/s bed level(m)	d/s bed level(m)	u/sFSL(m)	D/sFSL(m)	depth of flow	Fall height (m)
1140	1217.2	1216.47	1214.97	1217.2	1215.7	0.73	1.5
1290	1215.4	1214.83	1213.33	1215.56	1214.06	0.73	1.5
1410	1214	1213.22	1211.72	1213.95	1212.45	0.73	1.5
1610	1212.1	1211.54	1210.04	1212.27	1210.77	0.73	1.5
1780	1210.5	1209.85	1208.35	1210.58	1209.08	0.73	1.5
1930	1208.8	1208.2	1206.71	1208.93	1207.44	0.73	1.5
2080	1207.3	1206.57	1205	1207.3	1205.73	0.73	1.5
2200	1206.2	1204.89	1203.4	1205.62	1204.13	0.73	1.5
2380	1203.8	1203.24	1201.74	1203.97	1202.47	0.73	1.5
2470	1202.2	1201.66	1200.16	1202.39	1200.89	0.73	1.5
2630	1200.5	1200	1298.5	1200.73	1299.23	0.73	1.5
2830	1199	1198.3	1196.8	1199.03	1197.53	0.73	1.5
3080	1197.27	1196.57	1195.07	1197.3	1195.8	0.73	1.5

Table 7.10 Longitudinal profile of secondary canal 2

distance fall(m)	to ground level at fall(m)	u/s level(m)	bed/d/s level(m)	bed/u/s FSL(m)	D/s FSL(m)	depth of flow	fall height (m)
2180	1201.3	1201	1199.5	1201.69	1200.19	0.69	1.5
2260	1200	1199.4	1197.9	1200.09	1198.59	0.69	1.5
2400	1198	1197.7	1196.2	1198.39	1196.89	0.69	1.5
2520	1197.6	1196.07	1194.59	1196.76	1195.28	0.69	1.5
2830	1195	1194.3	1192.8	1194.99	1193.49	0.69	1.5
3250	1192.9	1192.4	1190.9	1193.09	1191.59	0.69	1.5
3450	1190.9	1190.7	1189.8	1191.39	1190.49	0.69	1.5
3560	1190	1189.7	1188.2	1190.39	1188.89	0.69	1.5
3710	1188.5	1188	1186.5	1188.69	1187.19	0.69	1.5
3960	1186.6	1186.2	1184.7	1186.89	1185.39	0.69	1.5
4260	1184.5	1184.1	1182.6	1184.79	1183.29	0.69	1.5
4420	1182.5	1181.9	1180.4	1182.59	1181.09	0.69	1.5
4570	1180.7	1180.2	1178.7	1180.89	1179.39	0.69	1.5
4800	1179	1178.5	1177	1179.19	1177.69	0.69	1.5
5000	1176.6	1176	1174.5	1176.69	1175.19	0.69	1.5

Table 7.11 Longitudinal profile of secondary canal 3

Dist. to fall (m)	G.level at fall (m)	u/s bed level (m)	d/s bed level (m)	u/sFSL (m)	D/sFSL(m)	depth of flow	fall height (m)
3460	1198	1197.23	1195.73	1198	1196.5	0.77	1.5
3650	1196.4	1195.56	1194.06	1196.33	1194.83	0.77	1.5
3840	1194.6	1193.88	1192.38	1194.65	1193.15	0.77	1.5
3990	1193	1192.14	1190.64	1192.91	1191.51	0.77	1.5
4240	1190.9	1190.5	1189	1191.27	1189.78	0.77	1.5
4370	1189.5	1188.88	1187.38	1189.65	1188.15	0.77	1.5
4620	1187.6	1187.1	1185.6	1187.87	1186.67	0.77	1.5
4770	1186.2	1185.76	1184.26	1186.53	1185.03	0.77	1.5
4970	1184.5	1184	1182.5	1184.77	1183.27	0.77	1.5
5070	1183.5	1182.4	1180.9	1183.17	1181.68	0.77	1.5
5330	1181.3	1180.66	1179.16	1181.43	1179.93	0.77	1.5
5580	1179.5	1178.9	1177.4	1179.67	1178.17	0.77	1.5
5880	1178	1177.12	1175.62	1177.89	1176.39	0.77	1.5
6130	1176.3	1175.39	1173.89	1176.16	1174.96	0.77	1.5

Table 7.12 Longitudinal profile of secondary canal 4

Dist. to fall (m)	G level at fall(m)	u/s bed level(m)	d/s bed level(m)	u/sFSL(m)	D/sFSL(m)	depth of flow	fall height (m)
4570	1190.8	1190.1	1188.6	1190.81	1189.31	0.71	1.5
5070	1188.7	1188.16	1186.66	1188.87	1187.37	0.71	1.5
5270	1187	1186.48	1184.98	1187.19	1185.69	0.71	1.5
5370	1185.4	1184.89	1183.39	1185.6	1184.1	0.71	1.5
5770	1183.5	1183	1181.5	1183.71	1182.21	0.71	1.5
5940	1181.5	1181.1	1179.6	1181.81	1180.31	0.71	1.5
6140	1180	1179.4	1177.9	1180.11	1178.61	0.71	1.5
6470	1178	1177.6	1176.1	1178.31	1176.81	0.71	1.5
6770	1176.5	1175.83	1174.33	1176.54	1175.04	0.71	1.5

Table 7.13 Longitudinal profile of secondary canal 5

Dist. to fall (m)	G level at fall(m)	u/s bed level (m)	d/s bed level(m)	u/sFSL(m)	D/sFSL(m)	depth of flow	fall height (m)
4530	1191.5	1190.78	1189.28	1191.5	1190	0.72	1.5
4790	1190	1189	1187.5	1189.72	1188.22	0.72	1.5
5090	1188	1187.2	1185.7	1187.92	1186.42	0.72	1.5
5660	1185.5	1185.1	1183.6	1185.82	1184.32	0.72	1.5
5880	1184	1183.4	1181.9	1184.12	1182.62	0.72	1.5
6130	1182.5	1181.67	1180.17	1182.39	1180.89	0.72	1.5

Table 7.14 Hydraulic characteristics of all the channels

Canal	command area (ha)	discharge($m^3.s^{-1}$)	Velocity ($m.s^{-1}$)	canal top width (m)	canal total length (km)
primary	683	1.55	0.89	4.2	4.51
secondary 1	178.91	0.85	0.77	3.53	2.4
secondary 2	117	0.744	0.79	3.4	3
secondary 3	334.64	1	0.8	3.68	3
secondary 4	135.45	0.8	0.757	3.46	2.5
secondary 5	144	0.82	0.762	3.5	1.82

8. DESIGN METHODOLOGY

The design procedures such as irrigation scheduling, canal layout and drop structures, the weir and other ancillary structures are described in this chapter.

To obtain crop coefficient for initial stage (K_{c1}), the initial frequency of irrigation (irrigation interval) has to be determined as follows:

Average available moisture content (AMC) $b = 16.5 \text{ cm.m}^{-1}$

Readily available moisture (RAM) assuming 75% depletion,

$$\text{RAM} = 0.75 * 16.5 = 12.4 \text{ cm.m}^{-1}$$

Average rooting depth at the initial stage considered as 40cm

$$\text{RAM} = 12.4 * 0.4 = 4.96 \text{ cm}$$

Assuming $ET_o = ET_{\text{peak}}$

$$\text{Irrigation interval (I.F)} = \frac{\text{RAM}}{ET_{\text{peak}}}$$

Sample calculation for the month of March as contained in Table 8.1.

$$ET_o = 6.3 \text{ mm.day}^{-1}$$

$$\text{I.F} = 4.96 / 0.63 = 7.87 \sim 8 \text{ days}$$

Similarly, it can be calculated for all the months as shown in Table 8.1

Table 8.1 K_c values for the initial stage

Month	ET_o (mm.day ⁻¹)	IF(days)	K_c
January	5.7	8.70	0.38
Feb.	5.9	8.41	0.37
March	6.3	7.87	0.35
April	5.5	9.02	0.39
May	5.2	9.54	0.41
June	5.1	9.73	0.42
July	6	8.27	0.36
August	6.1	8.13	0.35
Sept	6	8.27	0.36
Oct	5.6	8.86	0.38
Nov	5.5	9.02	0.39
Dec	5.7	8.70	0.38

The Kc values for mid-season stages are taken for relative humidity less than 20% and the wind speed between 0-5m/sec as indicated in FAO (1977) manual. The Kc value is contained in Table 8.2.

Table 8.2 Kc values for mid and late season

Crop	Maize	Ground nuts	Sugar beet	Pulses	Tobacco	Cotton
Mid-season	1.15	1.05	1.15	1.15	1.1	1.2
Late-Season	0.6	0.6	1.0	0.25	0.58	0.65

Using the above Kc values, the crop stage curve is plotted and correspondingly the values of Kc for each month are read from the curve. Crop water requirement is then calculated and contained in tables. The spreadsheet is annexed.

Kc value curve for Maize-I is shown as in Figure 8.1.

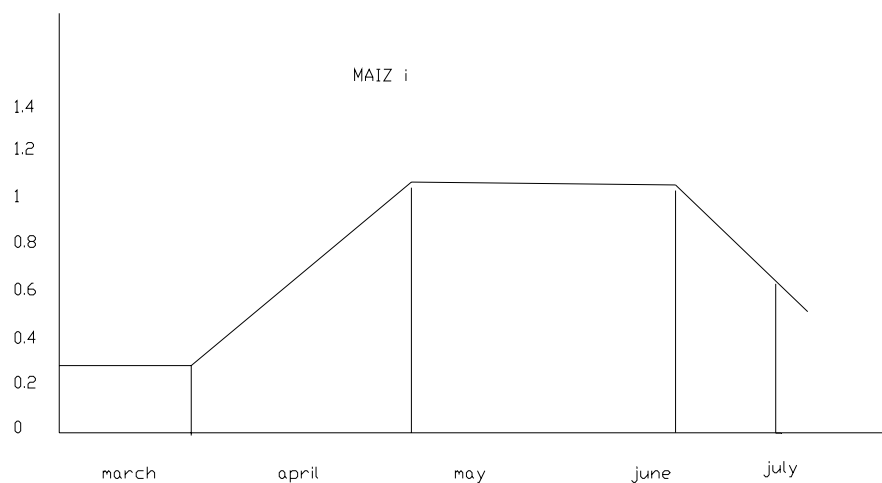


Figure 8.1 Kc value curve

Depending on the crop type and climate, root depth varies from 30 to 40 cm per month of active growth, it is assumed that the active growth period excludes the late season (maturity).

Sample calculation:

Soybean, total growing period = 85 days

Late season = 15 days

Active growth period = $(85 - 15) / 30 = 70 / 30 = 7/3$ months

Average root depth = $(30 + 40) / 2 = 35 \text{ cm.month}^{-1}$

Total root depth = $7 / 3 \times 35 = 0.82\text{m}$

Similarly the root depths for other crops are calculated and contained in Table 8.3.

Table 8.3 Root depth of different crops

Crops	Depth of Rooting (m)
Maize I& II	1.28
Ground nuts	1.45
Sugar beets	1.28
Pulses	1.05
Cotton	1.57
Banana	0.70
Soybean	0.82
Sugarcane	1.70
Tobacco	1.16
Vegetables	0.50

The effective storage required for the selected crops with AMC of 16.5cm/m is contained in Table 8.4.

Table 8.4 Effective storage for different crops

Crops	Maize I & Maize II	Ground nuts	Soybeans	Sugar beets	Pulses	Cotton	Tobacco	Banana	Sugarcane	Vegetables
Effective storage (mm)	105.60	115.5	67.65	105.60	88.63	129.53	95.70	57.80	140.30	41.30

Crop water requirement is the weighted water requirement of a given crop obtained by multiplying the specific water requirement of the crop to the crop growth coverage area and dividing the sum by the total area.

$$\text{Crop water req} = \frac{\text{Etc} * \text{area of given crop}}{\text{Area}} + \frac{\text{Etc} * \text{area of given crop}}{\text{Area}} + \dots$$

As shown in Table 8.4, it can be observed that the weighted effective storage of the crops is greater than 75mm and more than 50% of the crops have effective storage between 100 and 125mm. Hence, for all the crops an average storage factor of 1.03 is taken.

Several vegetable and fruits can be grown in the area satisfactorily as confirmed from information obtained from Arbaminch Water Technology Institute demonstration plot. However, some important data such as crop coefficient (Kc) for the area is not available. Hence, a suitable crop coefficient of 0.6 is assumed and the corresponding crop water requirement is obtained for each month as shown.

Assuming all months to be 30 days for the calculation purpose:

E.g. Feb. = $ET_o = 5.9 \text{ mm} \cdot \text{day}^{-1}$

$$5.9 * 30 = 177 \text{ mm/month}$$

$$ET_c = 177 * 0.6 = 106.2 \text{ mm} \cdot \text{month}^{-1}$$

The effective rainfall can be found and the net irrigation requirement is then estimated as:

$$\text{NIR} = ET_o - (\text{Pe} \times \text{Storage factor})$$

The probable maximum flow calculation of Hare River are contained in Table 8.5

Table 8.5 Probable maximum flow calculation

year	Mflow(m ³ .s ⁻¹)	logx	(logx-average of logx) ² /(10 ⁻³)	(logx-average of logx) ³ /(10 ⁵)	(lox-average logx) ³
1980	19.14	1.28	1.27	4.55	4.54656E-05
1981	11.7	1.07	31.71	564.59	-0.005645921
1982	10.1	1.00	58.53	- 1,416.01	-0.014160082
1983	14.1	1.15	9.42	91.36	-0.000913566
1984	10.95	1.04	42.78	884.88	-0.008848758
1985	41.11	1.61	135.20	4,971.29	0.049712925
1986	18.03	1.26	0.09	0.09	9.2543E-07
1987	14.45	1.16	7.46	64.46	-0.00064459
1988	19.6	1.29	2.12	9.74	9.73698E-05
1989	41.45	1.62	137.84	5,117.79	0.051177949
1990	16.83	1.23	0.41	0.82	-8.20162E-06
	sum	13.71			

sum 0.070813516

average of logx 1.25
 standard devia of log 0.21 skew coefficient G 0.98
 x
 N 11

G can be taken as 1 and the K values for various recurrence intervals are obtained from (Linsley, 1984) Table 13-4 for calculated values of skew coefficient, then the peak flood is computes using Eq 4.6. Further extrapolation is carried out using a semi log paper graph that yields the probable maximum flood for 100 years recurrence interval. K=3.022

$$\text{Log } Q_p = \text{Ave Log } X + K \sigma \log X$$

Then, the antilog of Log Q_p is:

$$Q_p = \text{Antilog } (1.25 + 3.002 * 0.21) = 74.39 \text{m}^3 \cdot \text{s}^{-1}$$

Table 8.6 Probable maximum flow when G equals to 1

Recurrence Interval (Years)	Value of K	Probable maximum flood(PMF)in (m ³ .s ⁻¹)
5	0.758	25.2889
10	1.3	33.372
25	2.043	46.655
50	2.542	59.183
100	3.022	74.397

From the water requirement calculation it is known that the channel discharge is calculated to be $1.44 \text{ m}^3/\text{s}$ but 8% is added for unidentified losses hence it becomes $1.55 \text{ m}^3 \cdot \text{s}^{-1}$.

8.1 Design of Canal Regulation Works

Canal falls

The natural slope of the ground surface is steeper than the designed bed slope of the canal. This necessitated construction of canal falls (drops) at suitable intervals to adjust the differences. This is shown in plan on the lay out system design.

Design of vertical drop fall

Sample design calculation for a typical canal fall along the primary canal located 50m upstream (u/s) of the start of secondary canal.

Bed level upstream = 1202.3

Full supply level upstream = 1203.21

Full supply level downstream = 1201.95

Full supply depth upstream = 0.91m

Full supply depth downstream = 0.91m

Down stream bed level = 1201.04m

Discharge ($\text{m}^3 \cdot \text{s}^{-1}$) = 1.55

Calculation of Energy head (H) and depth of fall [drop] (d)

Since the discharge is less than $14 \text{ m}^3 \cdot \text{s}^{-1}$, rectangular crest is considered.

$$Q = 1.835 * LH^{3/2} \left(\frac{H}{W} \right)^{1/6}$$

where

Q	=	discharge ($\text{m}^3 \cdot \text{s}^{-1}$),
W	=	crest width (m),
H	=	energy above crest level (m), and
L	=	crest length (m).

Assuming $W = 0.5\text{m}$ and taking $L = 1.0\text{m}$, then H can be calculated as:

$$1.55 = \frac{(1.835 * 1 * H^{10.6})}{(0.5)^{1/6}}$$

Then, $H = 0.84\text{m}$

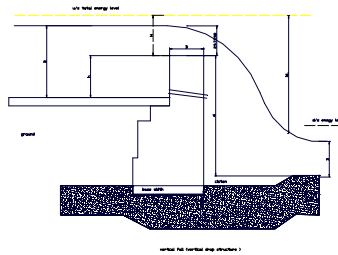


Figure 8.2 Sarda fall

$$\text{Velocity head} = \frac{V^2}{2g}$$

$$\text{Velocity head} = \frac{(0.89)^2}{2 * 9.81} = 0.04\text{m}$$

Upstream total elevation level (U/S-TEL) = Upstream full supply level (U/S- FSL). +
Velocity head

$$\text{U/S-TEL} = 1203.21 + 0.04 = 1203.25\text{m}$$

$$\text{Crest level} = \text{U/S-TEL} - H$$

$$\text{Crest level} = 1203.25 - 0.84 = 1202.41\text{ amsl}$$

Height of crest above U/S floor = $1202.41 - 1202.3 = 0.11\text{m}$

There fore

$$\begin{aligned} \text{Drop height (d)} &= (\text{Crest level}) - (\text{D/S bed level}) \\ &= 1202.41 - 1201.04 = 1.37\text{m} \end{aligned}$$

$$\begin{aligned} W &= (0.55) * (\sqrt{d}) = (0.55) * (\sqrt{1.37}) \\ &= 0.64 \end{aligned}$$

$$1.55 = \frac{(1.835 * 1 * H^{10.6})}{(0.64)^{1/6}}$$

$$H = 0.86$$

Now taking $W = 0.5\text{m}$, $L = 1.0\text{m}$, and $H = 0.8\text{m}$, Then

$$\text{Crest level} = 1203.25 - 0.8 = 1202.45$$

Height of crest above U/s floor = $1202.45 - 1202.30 = 0.15\text{m}$

$$d = \text{crest level} - \text{D/s bed level} = 1202.45 - 1201.04 = 1.41\text{m}$$

$$T_1 = \frac{(H+d)}{\rho} = \frac{(0.8+1.14)}{2} = 1.11$$

Where

T_1 = thickness of base (m)

ρ = density ($\rho = 2$, for masonry)

Design of cistern

For energy dissipation

$$L_c = 5 * \sqrt{(E * H_L)} = 5 * \sqrt{(0.8 * 1.3)} = 5.30\text{m}$$

Where

$$E = \text{U/s TEL} - \text{crest level} = 1203.25 - 1202.45 = 0.8$$

$$H_L = \text{U/s TEL} - \text{D/s TEL} = 1203.25 - (1201.04 + 0.91 + \text{velocity head } 0.04\text{m}) = 1.26\text{m}$$

Depth of cistern below D/s bed level

$$X = \frac{1}{4} (E * H_L)^{2/3} = 0.3\text{m}$$

Design of Wing Walls

The U/s wing wall is splayed at an angle of 45° from u/s edge of crest and goes upward and embedded to U/s berm. The d/s wings are kept vertical for the length of $6 * \sqrt{(E * H_L)} = 6.3\text{m}$.

Then the wings warped to 1:1 slope until the u/s canal section begins.

Height of d/s wings above bed = water depth + free board (FB) = $0.91 + 0.3 = 1.21\text{m}$.

U/s Protection

Brick pitching is laid on the U/s bed, sloping down towards the crest in 1:10 gradient. Two 7cm diameter drainpipes are provided at the U/s bed level. These pipes drain out the U/s side whenever the canal is closed.

Curtain Wall

$$\begin{aligned}\text{Maximum depth of U/s curtain wall} &= 1/2 * (\text{water depth U/s}) \\ &= 1/2 * (0.91) = 0.5\end{aligned}$$

D/s Protection

This consists of bed protection and d/s wings. The bed protection consists of dry brick pitching about 20cm thick

Impervious Floor

As the canal floor is lined, seepage is minimum but there is uplift pressure imposed on the floor itself. Hence uplift pressure is considered in this design.

$$\begin{aligned}\text{Maximum static head} &= \text{crest level} - \text{D/s bed level} \\ d &= 1202.45 - 1201.04 \\ &= 1.41\text{m}\end{aligned}$$

Total Floor length given by $c * d$

Where

$$\begin{aligned}c &= \text{Bligh's coefficient} \\ d &= \text{maximum static head (m)}\end{aligned}$$

Assuming $c = 6$

$$\text{Floor Length} = 6 * 1.41 = 8.46 \sim 8.5\text{m}$$

Minimum d/s floor length required, L_{\min}

$$L_{\min} = 2 * (\text{U/s Full supply depth} + 1.2) + H_L$$

$$H_L(\text{head loss}) = U/s \text{ full supply level} - D/s \text{ full supply level} = 1203.21 - 1201.95 = 1.26\text{m}$$

$$L_{\min} = 2 * (0.91 + 1.2) + 1.3 = 5.52\text{m}$$

$$U/s \text{ floor length} = 8.5 - 5.52 = 2.98\text{m} \sim 3\text{m}$$

Floor Thickness

- Maximum unbalanced up lift at the D/s toe of the fall

$$h = \left[\text{depth of cister} + \frac{(\text{crest level} - D/s \text{ bed level})}{8.5} \right] * 5.52 = 1.09\text{m}$$

$$\text{Thickness required} = \frac{h}{G-1} = \frac{1.09\text{m}}{2.4-1} = 0.78\text{m}$$

Where G = specific gravity

0.6m thick concrete overlain with 0.2 thick brick pitching is provided

- **Unbalanced head at 2m from the toe.**

$$= \{ [0.27 + (1202.45 - 1201.04)] / 8.5 \} * 3.52 = 0.7\text{m}$$

$$\text{Thickness required} = 0.7 / 1.4 = 0.5\text{m}$$

0.5m thick floor is provided

0.3m concrete overlain with 0.2m thick brick pitching is provided.

- **Unbalanced head at 4.0 from the toe**

$= [(0.27 + 1.41) / 8.5] * 1.52 = 0.3\text{m}$, 0.2m thick concrete, overlain with 0.1m thick brick pitching is provided

0.2 * 0.5m curtain wall is provided in the u/s and a 0.2 * 1m curtain wall in the d/s.

The detail is shown in the drawing.

Design of cross regulator

The design parameters of cross regulators are contained in Table 8.7.

Table 8.7 Design parameters of cross regulators

Discharge of parent canal	1.55m ³ /s
Depth of water in canal U/s	0.91m
Depth of water in canal D/s	0.91m
F.S.L. of canal ; U/s	1199.61m
F.S.L. of canal ; D/s	1199.50m
Assumed safe exit gradient	1/5
U/s bed level	1198.7m
Provide crest at R.L.	1198.7m

$Q(\text{water way}) = B \sqrt{[h(1.69h + 3.54h)]}$, which is obtained from,

$$Q = \frac{2}{3} * c_{d1} \sqrt{2g} * [B(ch + hv)^{3/2} - hv^{3/2}] + c_{d2} * B * h_1 \sqrt{2g(h_1 + hv)}$$

Ignoring velocity head

Where

B = clear water way required

$h = \text{u/s FSL} - \text{D/s FSL}$

$$= 1199.61 - 1199.5 = 0.11\text{m}$$

hv = head due to velocity of approach

$$C_{d1} = 0.577$$

$$C_{d2} = 0.8$$

$h_1 = \text{d/s FSL} - \text{crest level}$

$$= 1199.5 - 1198.7 = 0.8\text{m}$$

$$1.55 = B \sqrt{0.11[(1.69 * 0.11) + (3.5 * 0.8)]}$$

$$B = 1.5\text{m}$$

In this design, two inlets, each equals to 0.8m and separated by 0.4m thick piers are provided.

Total length is equals to 2m.

D/S floor level or cistern level

Discharge intensity $q = 1.55/1.6 = 0.97 \text{ m}^3 \cdot \text{s}^{-1} \cdot \text{m}^{-1}$

$H_L = 0.11 \text{ m}$

EF2 is taken 0.89 from graphs (blench curve)

Floor level = D/s FSL - EF2

Neglecting velocity head, d/s TEL = d/s FSL

$$\text{D/s floor level} = 1199.5 - 0.89 = 1198.8 \text{ m}$$

D/s bed level = $1199.5 - 0.91 = 1198.59$ which is lower than the floor level

Hence, the cistern or d/s floor is adopted at R.L 1198.59m

➤ **Vertical cut off**

Provided u/s cut off for a depth = $Du/3 + 0.6$

$$= 0.91/3 + 0.6 = 0.9 \text{ m}$$

➤ **Total floor length from exit gradient consideration**

$$GE = H/d * 1/\pi * 1/\sqrt{\lambda}$$

Where

H= maximum static head, which is caused when there is full water on u/s and there is no water at the d/s side.

$$= 1199.61 - 1198.59 = 1.02 \text{ m}$$

d= depth of d/s cut-off = 1.06 given by $(Yd/2 + 0.6)$

where Yd=depth of water = 0.91

GE= 1/5 (assumed)

$$1/5 = 1.02/1.06 * 1/\pi * 1/\sqrt{\lambda} \quad \lambda = \{1 + \sqrt{1 + \alpha^2}\}/2$$

where $\alpha = b/d$

$$1/\pi * 1/\sqrt{\lambda} = 0.208$$

From Koshla's curves $\alpha = b/d = 4.0$

Therefore $b = \alpha * d = 4 * 1.06 = 4.24 \text{ m}$, say 4.3m

Minimum length of d/s floor required

$$= (2/3)*b=2/3*4.3=2.9\text{m}$$

There fore u/s balancing length provided

$$= (4.3-2.9) = 1.4\text{m}$$

Calculation of floor thickness

$$\text{Unbalanced head at d/s toe} = 1.02*2.9/4.3 = 0.6879\text{m}$$

$$\text{Thickness required} = h/G-1 = 0.6879/24-1 = 0.5\text{m}$$

$$\text{Unbalanced head at 1.45m from the toe} = (1.02*1.45) / 4.3 = 0.344\text{m}$$

$$\text{Thickness required} = 0.344/1.4 = 0.25\text{m}$$

U/s floor

Theoretically no floor thickness is required under the upstream floor, since the uplift can be counter balanced by the weight of water standing over it. But a nominal thickness of 0.2m is provided.

Transition

To bring the canal to its normal original section a 1:1 splay with warped fall is provided. Both u/s and d/s sides of the regulator are shown in the drawing.

Design distributaries head regulator.

$$\text{Discharge of distributaries} = 1 \text{ m}^3 \cdot \text{s}^{-1}$$

$$\text{FSL of distributaries} = 1199.4\text{m}$$

$$\text{Bed level of distributor} = \text{FSL} - \text{depth of flow}$$

$$= 1199.4 - 0.77 = 1198.63\text{m}$$

Crest level

The crest level of distributor head is generally kept 0.3 to 1.0m higher than the bed level of the parent canal. Keep crest level 0.3m higher.

$$\text{Hence the crest level of head regulator} = 1198.7 + 0.3 = 1199\text{amsl}$$

D/s bed level = 1198.63m keep bed level at 1198.65 amsl.

Water way

$$Q = B\sqrt{h}[1.69h + 3.554h_1]$$

where

$$\begin{aligned} h &= \text{FSL of parent canal - FSL of distributaries} \\ &= 1199.61 - 1199.4 = 0.21\text{m} \end{aligned}$$

$$\begin{aligned} h_1 &= \text{FSL of distributaries - crest level} \\ &= 1199.4 - 1199 = 0.4\text{m} \end{aligned}$$

$$Q = 1\text{m}^3 \cdot \text{s}^{-1}$$

$$B = \text{clear water way required}$$

$$1 = B \cdot 0.21 (1.69(0.21) + 3.54 \cdot 0.4)$$

$$B = 1.2\text{m}$$

Hence, 1.2 m inlet is provided

Cistern

A fall of $d=1.4$ is combined. The fall is vertical drop type and the design procedure used in the above calculations can be adopted here for d/s of the regulator

Vertical cut off

Depth of u/s cut off below floor

$$= du/3 + 0.6$$

$$= 0.77/3 + 0.6 = 0.9\text{m}$$

The bottom level of u/s cut off = $1198.65 - 0.9 = 1197.75\text{m}$

Provide 0.3×0.9 cut off.

The design of the combined fall is calculated as follows:

$$Q = 1.835 LH^{10/6} (1/B)^{1/6}$$

Where the parameters are as

By trial and error, $B=0.5\text{m}$

$L=1.2\text{m}$, the bay width

$$l=1.835*1.2 *H^{10/6}(1/B)^{1/6,1}$$

$h=0.58\text{m}$

Thickness of base = $(H+d)/\rho$

$$= (0.58 + 1.4)/2$$

Length of cistern $L_c=5\sqrt{HL} * e$

$E= H + \text{velocity}$

$$= 0.58 + (0.8)^{2/2} * 9.81 = 0.61$$

$H_L = U/s \text{ TEL} - D/s \text{ TEL}$

$$= 1.1\text{m}$$

There fore $L_c = \sqrt{1.1 * 0.61} = 4.1\text{m}$

Depth of cistern below d/s bed level

$$X = 1/4 (H_L E)^{2/3} = (1/4) * (1.1 * 0.61)^{2/3} = 0.2\text{m}$$

Impervious floor: assuming Bligh's coef=6

Total floor length = CH

H is maximum unbalanced uplift at the d/s toe of the fall occurs when the regulator is closed and no water at the d/s side.

$$H = 1.87\text{m}$$

There fore total floor length = $6 * 1.87 = 11.22\text{m}$

Minimum d/s floor length = $2 * (0.77 + 1.2) + 1.1 = 5.1\text{m}$

Keep the cistern length equal to 4.5m

Floor thickness

Maximum unbalanced uplift at the d/s toe

$$h = [0.2 + 1.87]/11.22 * 4.5 = 0.95$$

Thickness required = $h/G - 1$ G unit weight of concrete

$$t = h/G - 1 = 0.95/2.4 - 1 = 0.70\text{m}$$

0.5m thick concrete over lain with 0.2m thick Brick pitching is provided.

Unbalanced head at 2m from the toe

$$h = [0.2 + 1.87] / 11.22 * 2.5 = 0.6$$

$$t = 0.6 / 1.4 = 0.44\text{m}$$

0.24m thick concrete overlain with 0.2m thick brick pitching up to the end of the cistern is provided.

$$D/s \text{ cut off} = D_d / 2 + 0.6$$

$$= 0.77 / 2 + 0.6 = 1.0\text{m}$$

0.3*1m cut off is provided

Bottom level of d/s cut off = crest level -1.4-1

$$= 1199 - 2.4 = 1196.6 \text{ m}$$

The detail is shown in the drawings

The moment about the toe calculations are contained in Table 8.8.

Table 8.8 Load (moment about the toe)

Type of load	Force(t/m)	Momentum(m)	Moment about the toe(t-m/m)
Vertical			
<i>1.self wt.</i>			
W ₁	3.36	2.83	+9.51
W ₂	10.08	1.75	+17.64
W ₃	3.36	0.67	+2.25
<i>2 water load</i>			
VW ₁	1.50	3.167	+4.75
VW ₂	1.49	3.0	+4.47
VW ₃	1.50	0.33	+0.50
<i>3.seismic load</i>			
X _v W ₁	0.168	2.83	-0.475
X _v W ₂	0.504	1.75	-0.882
X _v W ₃	0.168	0.670	-0.113
Horizontal			
HW ₁	10.08	1.50	-15.086

Table 8.8 continued....

HW ₂	4.09	0.53	+3.898
X _h W ₁	0.336	1	-0.336
X _h W ₂	1.008	1.5	-1.512
X _h W ₃	0.336	1	-0.336
Pe	1.089	1.9	-2.069

For stability against over turning the factor of safety should be greater than 1.5

From the Table 8.8, the sum of resisting moment (M +ve) =43.018tm.m⁻¹

The sum of over turning moment (M-ve) =20.80gt-m.m⁻¹

Factor of safety against over turning =[43.018/20.80g]-2.1=1.5

Since the factor of safety is greater than 1.5 the weir is safe against sliding which is given by

Fs=H/v and should not exceed 0.75

Fs=8.759/20.45 =0.43 and since this is less than 0.75, factor of safety against sliding is very much less than the allowable limit, hence the weir is safe against sliding.

Moment about the center is contained in Table 8.9.

Table 8.9 Moment about the centre

Type of load	force (t/m)	moment arm(m)	Moment about the centre(t-m/m)
Vertical			
1 self weight			
W1	3.36	1.083	+3.36
W2	10.08	0	0
W3	3.36	1.083	-3.36
2,water load			
VW1	1.5	1.417	+2.126
VW2	1.49	1.25	+1.863
VW3	1.5	1.417	+2.126
3,seismic load			
XvW1	0.168	1.08	-0.182
XvW2	0.504	0	0

Table 8.9 continued.....

XvW3	0.168	1.08	+0.182
Horizontal			
HW1	10.08	1.5	-15.086
HW2	4.09	0.953	+3.898
XhW1	0.336	1	-0.336
XhW2	1.008	1.5	-1.512
XhW3	0.336	1	-0.336
Pe	1.089	1.9	-2.07

Stress analysis

To avoid over stress (tension) the resultant of all the loads acting under worst condition of loading should pass through the middle third of the weir base.

From the above Table 2.28 the sum of moments of the forces about the centre, $m = -13.579 \text{ t} \cdot \text{m} \cdot \text{m}^{-1}$

The sum of the vertical forces, $V = 20.45 \text{ t} \cdot \text{m}^{-1}$

$$x = M/v = (-0.66) =$$

$$\text{Eccentricity} = b/2 - x = 3.5/2 - 0.66 = 1.09 \text{ which is } < 1.75$$

The negative sign shows the resultant force passes through the d/s centre. For no tension to develop,

$e < b/6$ but $x < b/6$, that is 1.09 meaning the resultant of all the forces pass out of the middle third by 0.7m

However, this happens under severe loading conditions.

This may be permitted because of the fact that such worst loading conditions shall occur only momentarily for little time and would neither last long nor occur frequently .

$$P_{\max}/\min = V/b (1+6e)/b$$

$$= 20.45/3.5 [(1+6 \cdot 1.09)/3.5]$$

$$P_{\max} = 16.76 \text{ t/m}^2 \text{ (at the heel) which is } < 300 \text{ t/m}^2$$

$$P_{\min} = -5.07 \text{ t/m}^2 \text{ (at the toe) which is } < 42 \text{ t/m}^2$$

Both the stress at the heel and toe are within the permissible limits hence the weir is safe for all conditions, so the dimension can be adopted.

Hence provide $a=1.5\text{m}$

$b=3.5\text{m}$

Design of Cistern

The vertical weir dissipates energy by means of impact rather than hydraulic jump formation and their sudden deflection of the flow velocity from vertical to horizontal. This requires provision of depressed floor at the toe so as to reduce the impact of falling jet and to save the down stream floor from scouring.

Design of cistern involves.

Determination of cistern length

Depths of fall below the d/s bed level

Length of cistern, L_c

$L_c = 6HL$

where

HL = head loss b/n u/s and d/s energy level,

E = k , energy above the crest level,

$L_c = 6(EHL)^{0.5}$,

$L_c = 6(1.49 * 1.77)^{0.5} = 9.74\text{m}$, and

$L_c = 10\text{m}$

The length of the glacis below d/s floor level, x

$x = 1/4(HL * E)^{2/3} = 0.48$

$x = 0.5\text{m}$

Design of impervious floor

The design of impervious floor comprises determination of length of the floor and thickness of the floor

I, Length of impervious floor

The length of the impervious floor is designed based on Bligh's creep theory. According to this theory the total creep length of the floor is equal to CH , where c is Bligh's creep coefficient which can be assumed 12-25 for coarse grained sands (Kumar, 1987).

H is the available water head, which is equal to crest level minus bed, level. Therefore total creep length $= 12 \times 3 = 36\text{m}$

D/s impervious apron for the weir with no crest shutter is given by $L_1 = 2.21cH/10$

$$2.21 \times 3 \times 3 / 10 = 14.5\text{m}$$

U/s length of the impervious $= L - b - L_1 - (2d_1 - 2d_2)$

Where d_1 and d_2 are depth of pile sheet at u/s and d/s respectively

Bottom level of u/s pile $= \text{U/s HFL} - 2R$

$$= 1232.43 - 2 \times 2.86 = 1226.71\text{m}$$

Keep the bottom at 1227m

$$d_1 = 1228 - 1227 = 1.0\text{m}$$

Bottom level of d/s pile $= \text{d/s HFL} - 2R$

$$= 1230.76 - 2(2.86) = 1225.04\text{m}$$

$d_2 = 1227.9 - 1225.04 = 2.9\text{m}$ therefore the u/s length impervious floor length is $= 36 - 3.5 - 14.5 - 2(2.9 + 1) = 10.2\text{m}$

II, Thickness of the impervious floor

The thickness of the impervious floor has been checked by both Bligh's and Koshla's theories and values obtained are similar, hence it can be adopted.

Residual pressure at toe of the weir

$$h = (3 - 3/36)(15.7) = 1.69\text{m}$$

Thickness, t required $= 4/3(h)/p - 1$

Therefore $t = 4/3(1.69)/2.24 - 1 = 1.8\text{m}$

Residual pressure at 5m from the toe

$$h = 3 - 3/36 \times 20.7 = 2\text{m}$$

Thickness required, $t = 4/3(1.275)/2.24-1=1.4\text{m}$

Residual pressure at 10m from the toe

$$h=3-3/36*25.7=0.86\text{m}$$

Thickness required, $t= 4/3 *(0.86)/2.24-1=0.9\text{m}$

Hence 1.8m thick concrete floor for 5m from the toe is provided. 1.4 thick concrete floor 5m far from the toe up to 10m distance is provided. Finally 0.9m thick concrete floor for the rest of the floor length up to 14.5m from the toe is provided.

Since there is negative residual pressure at the u/s of the weir, theoretically no floor is needed. But a nominal thickness of 0.5m can be provided.

A nominal thickness of 1m below the weir wall is provided.

Design of inverted filter and down stream talus

The inverted filter is a protection work provided to relief any residual pressure. Talus or launching apron is a loose pervious floor provided after the inverted filter to protect d/s pile from scour hole progressing in u/s direction. The design of the above protection work involves determination of the length and thickness.

The total length of d/s apron:

$$L3 = 18c * \sqrt{\{(h/10)*(q/75)\}}$$

$$L3 = (18*12*\sqrt{(3/10)*(3.077/75)})=24\text{m}$$

$$\begin{aligned} \text{Length of filter +Launching apron} &= L3-L1 \\ &= 24-14.5=9.5\text{m} \end{aligned}$$

Minimum length of inverted filter= $1.5d_2=1.5*2.9= 7.25\text{m}$

where d_2 is d/s sheet pile depth.

Total minimum length of lose protection work= $4.35 +7.25=11.6\text{m}$, Hence 4.35m of inverted filter and 7.25m length of launching apron is provided. Make filter of 0.50m thick block stone placed on 0.5m thick graded filter.

Length of launching apron on 3:1 slope = $10d_2=10*2.9 =9.17\text{m}$

Keeping 0.8m thick in launching position, the thickness in horizontal position is 1m. Hence u/s talus of 1m thick after the inverted filter is provided

U/s talus = $2d_1 = 2 * 1 = 2.0\text{m}$ is provided

where d_1 is u/s sheet pile depth and u/s brick

Protection = $1.5d_1 = 1.5 * 1 = 1.5\text{m}$, thickness of both is taken to be 0.5m the detail of the design are shown on the drawing.

8.2 Capacity of Under-sluices

The capacity of the under sluices is determined on the basis of the following consideration

I, to insure proper scouring, the discharge capacity be twice the full supply discharge of the main canal, that is $Q = 2 * 1.55 = 3.1\text{m}^3/\text{s}$

II, the discharge capacity should be higher than discharge of winter floods with out necessity of dropping shutter (if any)

III, during flood season, the capacity should be 10-15 % of high flood discharge i.e. in our case (20-30 m^3/s)

The discharge capacity should be higher of the three. Based on this, the capacity is fixed to discharge 10-15% of high flood discharge.

A design discharge of $20\text{ m}^3 .\text{s}^{-1}$ has been taken to come up with economical design.

Design consideration

To lead the bottom layer of water, which is highly charged with silt and sediment, through rectangular tunnels a silt excluder is provided.

I, Crest level of Under-sluice = crest level of head regulator- 1.8m

Therefore crest level of head regulator = crest level of under sluice + 1.8m

Fixing the crest level of under sluice at river bed level, the crest level of canal head regulator = $1228 + 1.8 = 1229.8\text{m}$

Keeping the level of the roof slab of the excluder tunnels at the same level as that of the head regulator crest, a roof slab of thickness that equals to 0.5m is provided so that the height of tunnels is 1.30m that is nearly in the allowable range (0.8 to 1.2m) for boulder stage rivers.

The discharge through the tunnel can be determined by orifice equation

$$Q = C_{d_0} * A * \sqrt{2gH}$$

Where

- H = head during high flood flows=3.1m, and
 C_{d₀} = coefficient of discharge assumed to be 0.62.

$$A = Q / (C_{d_0} * \sqrt{2gH}) = 20 / \{(0.62) * (\sqrt{2 * 9.81 * 3.1})\} = 4.14 \text{m}^2$$

$$B = \text{area} / \text{height of tunnel} = 4.14 / 1.3 = 3.20 \text{m}$$

Two bays, 1.6m separated by reinforced pier of 0.4m are provided.

Provide a glacis of 1 in t and keep the stilling basin at 1227.5m, the d/s floor is kept at 1227.8m

II, Length of talus below the under sluice, Lt

Lt is determined using the equation

$$L_t = 27 * c * H_s / 13 * q / 75$$

$$q = 20 / 3.2 = 6.25$$

$$L_t = 27 * 12 * \sqrt{(3/13 * 6.25/75)} = 45 \text{m}$$

But the impervious floor provided d/s of the weir can be sufficient. Hence provide 26 m long talus below the under sluice.

Floor thickness is the same as the floor thickness of the impervious floor of the weir.

III, The impervious floor u/s of the sluice gate

$$= 3.9c \sqrt{H_s / 13} = 3.9 * 12 * \sqrt{3/13} = 22.5 \text{m}$$

But the impervious floor that is provided u/s of the weir can be sufficient.

IV, Provide floor thickness which is the same as the floor thickness of the impervious floor of the weir.

8.3 Design of Canal Head Regulator

The head regulator is a structure constructed at the head of the off take from the reservoir behind the weir.

The main function of the canal head regulator is (Kumar, 1987):

- it regulates the supply of water entering the canal,
- it controls the entry of silt in to the canal, and
- it prevents the river flood from entering the canal.

To design the canal head regulator the following data's are necessary

Full supply discharge of canal	= 1.55 m ³ .s ⁻¹
Bed level of the canal	=1229.04m
FSL of canal	= 1229.95m
Pond level	= 1231m
Crest level of head regulator	= 1229.8m

Water way

The waterway of the regulator is determined using the following equations.

$$Q_1 = \frac{2}{3} * c * d_1 * B * \sqrt{2g} * [(h+hv)^{3/2} - hv^{3/2}] \text{---free weir equation}$$

$$Q_2 = c d_2 B h_1 * \sqrt{2g} (h+hv) \text{----- drowned weir}$$

where

$$h_1 = \text{canal FSL -regulator crest level,}$$

$$h = \text{pond level - canal FSL,}$$

h_v = head due to velocity of approach,
 B = clear width of water way, and
 G = acceleration due to gravity.

Combining the above two equations the total discharge, $Q=Q_1+Q_2$

$$Q = \frac{2}{3} c_{d1} \sqrt{2g} B [(h+h_v)^{3/2}] + c_{d2} B h_1 \sqrt{2g} (h+h_v)$$

c_{d1} and c_{d2} are taken 0.577 and 0.8 respectively

$$h = 1231 - 1229.95 = 1.05 \text{ m}$$

$$h_1 = 1229.95 - 1229.8 = 0.15 \text{ m}$$

$$Q = 1.55 \text{ m}^3 \cdot \text{s}^{-1}, \text{ assuming } h_v = 0$$

$$1.55 = \left\{ \frac{2}{3} * 0.577 * B * \sqrt{2 * 9.81 * [1.15^{3/2}]} \right\} + \left\{ 0.8 B (0.15) \sqrt{2 * 9.81 * 1.05} \right\}$$

$$1.55 = 2.4B$$

$$B = 1.55 / 2.4 = 0.65 = 0.7 \text{ m}$$

Allowing a velocity of 1.7m/s in the off taking canal and maintaining the d/s regulator to the depth of full supply level of 0.91m, bed width of the canal calculated as 1m. Length of the waterway, B can be taken as 1 m also.

8.3.1 Hydraulic calculation for various flow condition

I, full supply discharge during high floods

Let G be get the opening

Applying submerged orifice flow formula

$$Q = C_d * A * \sqrt{2gH}$$

where $h = \text{HFL} - \text{canal FSL}$

$$1232.43 - 1229.95 = 2.48 \text{ m}$$

Assuming $C_d = 0.62$

$$1.55 = 0.62 * G * 1 \sqrt{2 * 9.81 * 2.48}$$

$$G = 1.55 / 4.325 = 0.36 \text{ m}$$

Velocity through the opening $v = Q/A = 1.55 / 4.325 = 4.3 \text{ m/s}$

Loss of head at entry $= 0.5 V^2 / 2g = 0.5 * (4.3)^2 / [2 * 9.81] = 0.47 \text{ m}$

TEL u/s of the gate $= 1232.43 + 0.06$ (velocity head calculated before) $= 1232.49 \text{ m}$

TEL just d/s of the gate $= 1232.49 - 0.47 = 1232.02 \text{ m}$

TEL D/s water level = canal FSL = 1229.95m

Head loss, $h=1232.02- 1229.95 =2.07\text{m}$

Discharge intensity, $q=1.55/1=1.55\text{m}^3 \cdot \text{s}^{-1} \cdot \text{m}^{-1}$

II, full supply discharge passing down the regulator at pond level, $H_2=\text{pond level} -\text{canal FSL}$

Hydraulic jump calculation for the two flow conditions is contained in Table 8.10.

Table 8.10 Hydraulic jump calculations for canal head regulator

Sr.no	Item	High flood flow(m)	pond level flow(m)
1	$q (\text{m}^3/\text{s}/\text{m})$	1.55	1.55
2	U/s water level	1232.43	1231
3	d/s water level	1229.95	1229.95
4	u/s total energy level	1232.49	1231
5	d/s total energy level	1230.85	1230.85
6	Head loss HL	2.07	1.05
7	D/s specific energy (Ef2 from blench curves)	1.5	1.25
8	Level at which jump will form(d/s TEL -Ef2)	1228.35	1228.7
9	u/s specific energy(Ef1=Ef2+HL)	3.67	2.4
10	pre jump depth corresponding to Ef1 from specific energy curve (SEC)	0.18	0.22
11	post jump depth D2 corresponding to Ef2	1.75	1.62
12	height of jump ,D2-D1	1.57	1.4
13	length of concrete floor required beyond the jump =5(D2-D1)	7.9	7

As shown in Table 8.10 the length of floor required beyond the jump level is 8m. A d/s glacis of 4:1 (H:V) is provided

III, Calculation for floor thickness

Unbalanced head at high flood flow condition is:-

HFL -D/s canal bed level = 1232.43- 1229.04 =3.39m

Since the design head is greater than that of a high flood conditions, the floor is designed $2/3$ of the head. That is, $2/3 * 3.39 = 2.26\text{m}$

Therefore, thickness, $t = 2.26/2.24 - 1 = 2.26/1.24 = 1.82$

Hence, thickness of 1.9m at the d/s of the glacis is provided

Unbalanced head for high flood flow condition about crest level of head regulator

Crest level of regulator = HFL-regulator crest level

$$= 1232.43 - 1229.8 = 2.63\text{m}$$

The floor is designed for $2/3 * 2.63 = 1.75\text{m}$

Hence, thickness, $t = 1.75/1.24 = 1.4\text{m}$

Hence, a thickness of 1.4m provided at the end of the crest and increasing the thickness to 1.9m towards the d/s of glacis.

For pond level flow condition, the level of the hydraulic jump formation is at 1228.7m below d/s canal bed level, which is, = (1229.04m)

Therefore to let the hydraulic jump-starts at the end of d/s glacis a stilling basin of depth $1229.04 - 1228.7 = 0.34\text{m}$ is required. Thickness required at the end of the floor may be taken as 1m thick.

Assuming Bligh's creep coefficient $C = 6$

$$L = 6 * 2.26 = 13.56\text{m}$$

$$L = 14\text{m}$$

Horizontal length of glacis = $(1229.8 - 1228.7) * \text{slope} = 1.1 * 4 = 4.4\text{m}$ total length of horizontal floor = $14 = 4.4 + 8 + 2d$

Where d is depth of d/s cut off below the bottom of the impervious floor

$$2d = 14 - 4.4 - 8$$

$$d = 1.6/2 = 0.8\text{m}$$

8.3.2 The non over flow sections

The non over flow section is structure constructed at both ends of the over flow section to prevent over spilling of water sideways owing to the raise of water lever upstream of the over flow section. These structures consist of marginal bund and guide bank. The crest level of the guide bank, providing a free board of 0.5m, is taken to be at 1233m. A masonry wall with

vertical face and sloping back face is used. The top width is taken as 1.0m. The back of the wall is filled with earthen material.

9. SUMMARY AND GENERAL RECOMMENDATION

The system of irrigation has been designed on 80% probability bases; water or river supply would be critical in some months, i.e. in January, February, march. To overcome this water deficiency, the following measures are recommended: -

- The time of irrigation should be increased from 13hr to 16 hr so that the water demand can be met by allowing small flows into the canal for long time,
- From the area where the water level is relatively higher, irrigation by pumping should be taken as the option to supplement the direct irrigation, and
- The area on the u/s right of the diversion work can be utilized for additional storage so that significant amount of water can be drawn to supplement during periods of low flows in the river.

The project should be managed properly in order to ensure safe and reliable operation after the completion of the project. To enhance the project benefits and for continuous operation of the project, it is vital to establish project administrative organization. The project may be administered through both specific and mass management. It is suggested to set up a committee from the three farmers associations in the irrigation district at the basic level for irrigation water management.

The objective of the committee is to carry out the following tasks:

- To work out plans for continuous project operation,
- To carry out the plan of farm irrigation during the period of water cut off,
- To take responsibility for maintenance of canals and structures,
- For deciding timely distribution of water among each other,
- For clearing away obstacles and uprooting weeds in time,
- -For repairing the gate at regular interval to keep water flowing as per requirements, and
- To make rules and regulations for water use.

Administrative station should be established near the diversion site for the administrative group with the provision of necessary infrastructure for the project management and protection.

Finally, irrigation development in the area may contribute in various ways to environmental degradation such as creating colder and dumper climates that might lead to the out break of diseased like malaria and the rise of water table causing water logging as well as salinity as a result of evapotranspiration.

10. BILL OF QUANTITIES

The table of bill of quantity gives the estimated quantity of material required to carry out the construction. It is calculated based on the dimension of the structures obtained from the detail design of the structure. Bills of quantities are contained in Table 10.1.

Table 10.1 Bill of quantities

Sr.no	Material	Item	Description	Quantity in m ³	
1	Concrete	Canal	Primary	1607	
			Secondary 1	729	
			Secondary 2	876	
			Secondary 3	947	
			Secondary 4	745	
			Secondary 5	548	
			Impervious floor	U/s impervious f	331.5
				D/s impervious floor	1332.5
			Cut off	U/s cut off	19.5
				D/s cut off	39
			2	Stone	Masonry wall
Divide wall	48				
Guide bank and marginal bund	240.5				
Fall structure	134.5				
Talus	520				
3	Sand (course and fine)	Fill material	Inverted filter	348	

11. DRAWINGS AND TABLES

Crop water requirement for each crop is given in Tables 11.1 to 11.8.

Table 11.1 Irrigation water requirement for ground nut (Planting date Sep-1)

I/II/III/IV stages

Area of planting 227 ha 30/40/50/30days

Development stage	month (period)	growing period in the month (days)	ETo(mm/day)	K _c	ETc(mm/day)	ETc(mm/day*no of days in the stage	ETc(mm/each month)
initial stage	sept	30	6	0.3	2.16	64.8	64.8
development stage	oct	30	5.6	0.61	3.42	102.6	102.6
	nov	10	5.5	0.95	5.23	52.3	
mid season	nov	20	5.5	1.05	5.78	115.6	167.9
	dec	30	5.7	1.05	5.99	179.7	179.7
late season	Jan	30	5.7	0.83	4.7	141	141.1
sum							656.1

Table 11.2 Irrigation water requirement for Maize-I (Planting date March-1)

type of crop:-

maize I

date of planting

:-March 1

area of planting 227ha

development

stage I/II/III/IV stages

or 25/40/45/30 days

development stage	month (period)	growing period in the month (days)	ETo(mm/day)	Kc	ETc(mm/day)	ETc(mm/day*no of days in the stage)	ETc(mm/each monthstage)
initial stage	march	25	6.3	0.35	2.21	55.13	
					-	-	
development stage	march	5	6.3	0.45	2.84	14.18	69.30
	april	30	5.5	0.75	4.13	123.75	123.75
	may	5	5.2	1.1	5.72	28.60	
mid season	may	25	5.2	1.15	5.98	149.50	178.10
	jun	20	5.1	1.15	5.87	117.30	
late season	jun	10	5.1	0.98	5.00	49.98	167.28
	july	20	6	0.78	4.68	93.60	93.60
sum							632.03

Table 11.3 Irrigation water requirement for Sugar-beet (Planting date June-1)

type of crop:-sugar beet

date of planting :-June 1

area of planting 246ha

development stage I/II/III/IV stages

or 25/35/50/55 days

development stage	month (period)	growing period in the month (days)	ETo(mm/day)	Kc	ETc(mm/day)	ETc(mm/day*no of days in the stage)	ETc(mm/each month)
initial stage	june	25	5.1	0.42	2.14	53.55	
					-	-	
development stage	june	5	5.1	0.49	2.50	12.50	66.05
	july	30	6	0.85	5.10	153.00	153.00
					-		
mid season	aug	30	6.1	1.15	7.02	210.45	210.45
	sept	20	6	1.15	6.90	138.00	
late season	sept	10	6	1.13	6.78	67.80	205.80
	oct	30	5.6	1.07	5.99	179.76	179.76
	nov	15	5.5	1.02	5.61	84.15	84.15
sum							899.21

Table 11.4 Irrigation water requirement for Pulses (Planting date Jan-1)

type of crop:-pulses

date of planting :-January 1

area of planting 246ha

development stage I/II/III/IV stages

or 20/30/40/20 days

development stage	month (period)	growing period in the month (days)	ETo(mm/day)	Kc	ETc(mm/day)	ETc(mm/day*no of days in the stage)	ETc(mm/each month)
initial stage	jan	20	5.7	0.38	2.17	43.32	
					-	-	
development stage	janu	10	5.7	0.5	2.85	28.50	71.82
	feb	20	5.6	0.89	4.98	99.68	
mid season	feb	10	5.6	1.15	6.44	64.40	164.08
	march	30	6.3	1.15	7.25	217.35	217.35
late season	april	20	5.5	0.69	3.80	75.90	75.90
sum							529.15

Table 11.5 Irrigation water requirement for Maize-II (Planting date Aug-1)

type of crop:-maize II

date of planting :-August 1

area of planting 118ha

development stage I/II/III/IV stages

or 25/40/45/30days

development stage	month (period)	growing period in the month (days)	ETo(mm/day)	Kc	ETc(mm/day)	ETc(mm/day*no of days in the stage)	ETc(mm/each month)
initial stage	augest	25	6.1	0.35	2.14	53.38	
					-	-	
development stage	augest	5	6.1	0.4	2.44	12.20	65.58
	sep	30	6	0.78	4.68	140.40	140.40
	oct	5	5.6	1.13	6.33	31.64	
mid season	oct	25	5.6	1.15	6.44	161.00	192.64
	nov	20	5.5	1.15	6.33	126.50	
late season	nov	10	5.5	1.12	6.16	61.60	188.10
	dec	20	5.7	1.07	6.10	121.98	121.98

sum

708.70

Table 11.6 Irrigation water requirement for Tobacco (Planting date Aug-1)

type of crop:-tobacco

date of planting :-August 1 Feb-01

area of planting 118ha

development stage I/II/III/IV stages

or 10/45/45/50days

development stage	month (period)	growing period in the month (days)	ETo(mm/day)	Kc	ETc(mm/day)	ETc(mm/day*no of days in the stage)	ETc(mm/stage)
initial stage	feb	10	5.9	0.42	2.48	24.78	
development stage	feb	20	5.9	0.56	3.30	66.08	90.86
	march	25	6.3	0.9	5.67	141.75	
mid season	march	5	6.3	1.1	6.93	34.65	176.40
	april	30	5.5	1.1	6.05	181.50	181.50
	may	10	5.2	1.1	5.72	57.20	
late season	may	20	5.2	0.98	5.10	101.92	159.12
	june	30	5.1	0.71	3.62	108.63	108.63

sum

716.51

Table 11.7 Irrigation water requirement for Cotton (Planting date Aug-1)

type of crop:-cotton

date of planting :-August 1 april

area of planting 91ha

development stage I/II/III/IV stages

or 30/50/55/45days

development stage	month (period)	growing period in the month (days)	ETo(mm/day)	Kc	ETc(mm/day)	ETc(mm/day*no of days in the stage)	ETc(mm/stage)
initial stage	april	30	5.5	0.39	2.15	64.35	64.35
development stage	may	30	5.2	0.63	3.28	98.28	98.28
	june	20	5.1	1.04	5.30	106.08	
mid season	june	10	5.1	1.2	6.12	61.20	167.28
	july	30	6	1.2	7.20	216.00	216.00
	aug	15	6.1	1.2	7.32	109.80	
late season	aug	15	6.1	1.15	7.02	105.23	215.03
	sept	30	6	0.83	4.98	149.40	149.40

sum

910.34

Table 11.8 Irrigation water requirement for Soyabean (Planting date Dec-1)

type of crop:-soybean

date of planting :-Dec1

area of planting 91ha

development stage I/II/III/IV stages

or 15/15/40/15days

development stage	month (period)	growing period in the month (days)	ETo(mm/day)	Kc	ETc(mm/day)	ETc(mm/day*no of days in the stage)	ETc(mm/stage)
initial stage	dec	15	5.7	0.39	2.22	33.35	
development stage	dec	15	5.7	0.74	4.22	63.27	96.62
mid season	jan	30	5.7	1.1	6.27	188.10	188.10
	feb	10	5.9	1.1	6.49	64.90	
late season	feb	15	5.9	0.78	4.60	69.03	133.93

sum

418.65

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