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IRRIGATION ENGINEERING MANUAL

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Acronyms

ADB	Asian Development Bank
AWDO	Asian Water Development Organisations
BhWP	Bhutan Water Partnership
BSR	Bhutan Schedule of Rates
CA	Command area
CC	Climate Change
DG	Director General
DOA	Department of Agriculture
DWS	Drinking Water Supply
FAO	Food and Agriculture Organization
FGD	Focus Group Discussion
FMIS	Farmer Managed Irrigation System
GIS	Geographic Information System
GPS	Geographic Positioning System
GW	Groundwater
HP	Hydropower
IFAD	International Fund for Agriculture Development
IWRM	Integrated Water Resources Management
JICA	Japan International Cooperation Agency
MOAF	Ministry of Agriculture and Forest
MOWHS	Ministry of Works and Human Settlement
MOIC	Ministry of Information and Communication
NEC	National Environment Commission
NECS	National Environment Commission Secretariat
NIIS	National Irrigation Information System
NIMP	National Irrigation Master Plan
RBMP	River Basin Management Plan
RGOB	Royal Government of Bhutan
RSPN	Royal Society for Protection of Nature (Bhutan)
TA	Technical Assistance
TAC	Technical Advisory Committee
UNCDF	United Nation Capital Development Fund
USBR	United States Bureau of Reclamation
USDA	United States Department of Agriculture
WB	World Bank
WRCD	Water Resources Coordination Division
WUA	Water Users Association

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EXECUTIVE SUMMARY

The updating of the existing irrigation engineering manual 1998 covers the planning, survey, design and evaluation of an irrigation scheme. The manual mainly focuses on the hydrology, hydraulics, planning and design of hydraulic structures, canal layout and design headwork and intakes, micro irrigation and river training and flood control works. In addition, the manual covers crop planning, water demand assessment, and water balance assessment. The limitation of the manual is that it does not cover institutional and construction management aspects of irrigation project preparation. The target group of the manual is the engineering professionals of the Department of Agriculture. However, the manual will also benefit all irrigation practitioners from both the public and private sectors.

The Irrigation Engineering Manual is organized into nine chapters. The introductory chapter (Chapter 1) presents irrigation in Bhutan, the methodology adopted and the contents of the manual. Chapter two (2) of the manual describes project study and survey procedures, highlighting identification and feasibility studies, including environment assessment.

In the planning of an irrigation scheme, the water requirement and its availability at the particular location need to be assessed first in addition to the availability of suitable land for agricultural production. The water availability and requirement together decide the area to be irrigated, and the size and capacity of the conveyance system. The planning and design of an irrigation scheme must consider whether the land is suitable for irrigated agriculture, climate conditions are favorable, and water is available for the proposed cropping pattern. The identification of the scheme is the very first step of the project development process. The Dzongkhag/Gewog level office should ask the farmers through their leaders about the necessity of the rehabilitation of an existing scheme or the construction of a new scheme. In addition, local level officers should disseminate information about the plans and program of the Department of Agriculture (DOA) on irrigation development and management.

The feasibility study is the basis for project implementation and is carried out by a team of experts with engineering, agriculture, and socio-environmental backgrounds. The feasibility study covers topographical, hydrological, agricultural, geological and socio-economic surveys. In addition, the study assesses the technical feasibility, economic viability and institutional suitability of project implementation. The priority ranking of the scheme is carried out based on techno-economic, social and environmental criteria.

Chapter three of the manual addresses hydrology and agro-meteorology questions. This chapter highlights the procedures followed to assess the availability of water for irrigation, flood flows, and crop and irrigation water requirements, including water balances. According to the Food and Agriculture Organization (FAO), 80% dependable river flow is used to estimate the water available for irrigation diversion. Given the large spatial variation in run-off in Bhutan and the significant number of small catchments, it is unlikely that gauging stations will be found at the point of irrigation diversion. For gauged catchments, water availability is assessed with the help of frequency analysis; for ungauged catchments, other empirical methods need to be adopted.

The discharge adopted for the design of a hydraulic structure is referred to as design flood discharge. The estimation of design flood is essential for the design of all hydraulic structures. The return period (T) of the flood flow is the indicator which determines the relative importance of the various hydraulic structures. For gauged catchments, design flood is assessed with the help of statistical analysis; for ungauged catchments, other empirical methods are used which are briefly described.

The crop water requirement is estimated with the evapo-transpiration (ETo) of a reference crop and multiplied by the crop coefficients of proposed crops. The ETo is calculated using Cropwat-8, software developed by the FAO. The total crop water requirement is assessed with the addition of land preparation and deep percolation losses during cultivation practices. The net crop water requirement is assessed by deducting effective rainfall from the total crop water requirement. The effective rainfall is calculated from the 80% reliable rainfall of the area. The intake water requirement is calculated with consideration of the irrigation system's efficiency. The water balance at the diversion point of the source river is calculated by considering downstream environmental flow and upstream abstractions.

Chapter four (4) provides basic concepts of intakes and diversion headworks along with their design procedures and worked out examples. This chapter also addresses pump irrigation with its design procedures and a worked out example. Intake structures are hydraulic devices built at the head of an irrigation canal, whereas the structure supplying water to the off-taking canal is called a headwork. Headworks may be a storage headwork or a diversion headwork. According to the method of diverting water from the source, intakes are bank intakes, side intakes with cross weirs, bottom intakes, frontal intakes, submerged intake, and tower intakes.

A diversion headwork may be a weir or a barrage depending upon the objective of the irrigation scheme. Weirs are solid walls across the river to raise the water level and to divert it into the irrigation canal. Weirs may be provided with small shutters on the top. During flooding, there is considerable afflux on the upstream side of a weir because the entire discharge of the river has to pass over the weir's crest.

A barrage is similar to a weir but is equipped with gates to head up the water on its upstream side. The barrage crest level is on the average river bed level and has minimum afflux. From an operational point of view, a barrage is better than a weir; however, the cost of a barrage is much higher than that of a weir. The main components of a headwork are weirs, divide walls, fish ladders, under sluices, head regulators, abutments, guide bunds and canals. Based on the flow characteristics, weirs are categorized as sharp crested weirs and broad crested weirs. In addition, weirs are also categorized according to the materials used in their construction. The design of a weir involves hydraulic design and structural design. The hydraulic design involves the assessment of the length of the waterway, crest levels of the weir and under sluice, depth of cut offs in relation to both scour and exit gradients, level, length and thickness of floors, and length of protection works.

This chapter also provides design concepts on energy dissipation devices to be used in hydraulic structures which are mostly based on the hydraulic jump formation. In addition, this chapter presents the design concept, procedures and worked out example of a pumped irrigation system, including brief notes on pump characteristics, the selection and siting of pumps, and the selection of pipes and fittings.

Chapter five (5) presents the basic concept of canal design in both erodible and non-erodible beds, along with design procedures and worked out examples of a canal system. This chapter also contains concept notes on the types and layout of canals, lined and unlined canals, and piped canals. Design parameters such as canal side slope, permissible velocities, bed width to depth ratio, canal longitudinal slope, Manning's roughness coefficient, freeboard, and minimum radius of curvature are briefly presented. With regard to lining types, this chapter includes brief notes on stone masonry lining, concrete lining, soil cement lining, Ferro cement lining, plastic lining and slate lining, followed by design procedures and worked out examples. The design concept and worked out examples of pipes flowing full and pipes flowing partially full are provided in the covered canal section. In addition, this chapter covers the design of pipelines, including concept notes on hydraulic grade lines, control valves, frictional head loss pressure limits, air blocks and washouts, control valves, and break pressure tanks. It also provides formulae for the assessment of head losses in pipes and fittings. Furthermore, this chapter covers issues and concerns of hill irrigation canals in unstable areas such as those prone to landslides, and mass slips.

Chapter six (6) presents the design concept, procedures and worked out examples of canal structures, highlighting various types of structures. The canal structures are grouped into regulating structures, drop structures, cross drainage structures, and sediment control structures. For regulating structures, the manual provides the design concept, procedures and worked out examples of the head regulator, flow divider or proportional divider, and field outlets or offtakes. The drop structures section covers the design concept, procedures and worked out examples of vertical drops, chute drops, pipe drops, cascade drops, escape structure, bridges and culverts. Cross drainage structures are the main structures in an irrigation scheme, and are grouped into aqueduct, super passage, siphon, pipe crossings and level crossings. The manual provides selection criteria for cross drainage structures, design considerations, merits and demerits, design procedures and worked out examples of these structures. For the sediment control structures, the manual presents the design of gravel traps and settling basins along with worked out examples. The section on retailing walls describes common types of retaining walls based on the materials used, the design concept, the procedures for design and a worked out example.

Chapter six also provides the general design concept for water control gates, including types of gates and the standard size for small scale irrigation schemes. In addition, this chapter provides the structural design concept of hydraulic structures including loadings and structural analysis, factor of safety for stability, and bearing pressure. This section also includes the concept of seepage and uplift design, and concrete design for hydraulic structures.

Chapter seven (7) covers micro irrigation, with a focus on the design concept, procedures and worked out examples of sprinkler irrigation system, drip irrigation system and water harvesting. In addition, this chapter briefly describes the advantages and constraints of sprinkler and drip systems, types of sprinkler systems and their components. This chapter also includes the design concept and typical models of a simple drip irrigation system. The water harvesting section provides information on types of water harvesting, the design concept of ponds and tanks, design standards used elsewhere in the region, methods of sealing water harvesting ponds or tanks, and a worked out example of pond design.

Chapter eight (8) contains the design concept and design procedures of river training and flood control works. This chapter opens with general information on the rivers of Bhutan, types of floods, and flood prone areas in the country. This chapter also provides brief concept notes on types of river training works, which include embankment bunds and guide bunds, spurs and groynes, cutoffs and pitching of banks along with their design parameters and standards. This chapter illustrates the detailed design procedures of river training works. It also includes the design concept and methods of bank protection works along with the design procedure of revetment works. At the end of this chapter, brief notes on the mathematical models used for river studies -- HEC HMS, HEC RAS, MIKE BASIN, MIKE 11 and MIKE FLOOD—are provided.

The Irrigation Engineering Manual concludes with chapter nine (9), which covers project evaluation, providing the concept and procedures of agricultural benefit assessment, estimate of project costs, and economic analysis. Agricultural benefits are assessed on the basis of per unit benefits of crop yields and the agricultural inputs involved in growing particular crops. The main agricultural inputs are seeds, fertilizer, pesticide, labor, and animal power. The project cost estimate includes unit rates of the items of work, detailed quantity of the works and cost of the works. The derivation of unit rates is based on the Bhutan Schedule of Rates (BSR), which is published each year by the Ministry of Works and Human Settlement (MOWHS). The quantity of works is assessed according to the dimensions of the works as prescribed in the approved drawings. The cost of civil works comes from the multiplication of quantity and the unit rates. The total project cost may include other costs such as mobilization costs, cost for consulting services, supervision costs, cost for bank commissions etc.

CHAPTER-1

1. Introduction

1.1 Background

Development of irrigation will benefit the agriculture sector by facilitating adaptation to climate change, increasing agriculture productivity and incomes, and improving rural livelihoods (Severaju, 2014). Although Bhutan has a long history of irrigation scheme planning, design and implementation, it was felt that the existing irrigation engineering manual needed to be updated. In this context, Asian Development Bank Technical Assistance (ADB-TA-8623) on Adaptation to Climate Change through Integrated Water Resources Management (IWRM) aims to update the existing Irrigation Engineering Manual of the Department of Agriculture (DOA) as part of its strengthening governance and capacity development. This manual will benefit those involved in the planning, design, construction, operation and maintenance of irrigation schemes.

1.2 Scope of the manual

The main objective is to update the existing Irrigation Engineering Manual 1998. The scope of the manual covers irrigation project preparation, from planning and surveys to design and evaluation. The manual mainly focuses on the hydrology, hydraulics, planning and design of hydraulic structures, canal layout, concept of structural design of irrigation infrastructure, and river training and flood control works. In addition, the manual covers crop planning, water demand assessment, and water balance assessment. The limitation of the manual is that it does not cover the socio-institutional and construction management aspects of irrigation project preparation. The targets of the manual are the engineering professionals of the Department of Agriculture. However, the manual will also benefit all irrigation practitioners from both the public and private sectors. The manual also includes a separate volume for the typical drawings planned for small irrigation schemes.

1.3 Irrigation in Bhutan

Irrigation in Bhutan has been practiced since time immemorial. Farmers of Bhutan have constructed small scale irrigation schemes with their own resources and skills. These farmer-managed irrigation systems (FMIS) were constructed to provide assured irrigation to paddy fields (Chuzhing). With the introduction of planned development in Bhutan, these FMIS started to receive support from the Government (Pradhan, 1989). The Government began to develop irrigation with the first five year plan (1961-66). The Irrigation Division was established in 1967 under the Department of Agriculture to take responsibility for the planning, design and implementation of irrigation facilities.

With the request of the Royal Government of Bhutan, the involvement of bilateral and multilateral donor agencies began in the 1980s. Major donors involved in the irrigation development sector are Indian Aid, United Nation Capital Development Fund (UNCDF), International Fund for Agriculture Development (IFAD), Asian Development Bank (ADB), and the World Bank (WB). Major projects developed with financial assistance from donor agencies are the Punakha-Wangdi Valley Development Project (IFAD), Taklai Irrigation Project (UNCDF), Gelephu Lift Irrigation Project (Indian Aid), Tsirang Irrigation Project (ADB), and Decentralized Rural Development Project (WB). The Taklai irrigation scheme (1,350 ha), the largest scheme in Bhutan, was recently rehabilitated with technical and financial support from Japan International Cooperation Agency (JICA). The Gelephu Lift Irrigation Project is defunct.

According to the National Irrigation Information System (NIIS), there are about 1,100 irrigation schemes in Bhutan, of which 962 schemes have a command area larger than 15 acres. 111 of these 962 schemes are not functioning for different reasons. In addition, there are about 140 schemes with a command area less than 15 acres providing irrigation to about 2,500 acres of land. These small, medium and large irrigation schemes provide some kind of irrigation to about 64,246 acres of land. Traditionally two types of cultivated lands exist in Bhutan: wetlands (Chuzhing) and dry lands (Kamzhing). Chuzhings are recognized as irrigated lands in general while Kamzhings are un-irrigated cultivated lands. However, not all Chuzhings have irrigation facilities.

1.4 Methodology adopted

The methodology adopted to update the existing Irrigation Engineering Manual involved the following approaches:

- Review of existing engineering manuals,
- Review of irrigation design manuals of similar regions,
- Identification of gaps in the existing manuals,
- Interaction with related stakeholders such as staff of the Engineering Division in DOA and collection of expectations, and
- Preparation of the Irrigation Engineering Manual

1.4.1 Review of existing manuals

The review of existing manuals started with the collection of the manuals. The available manuals at the DOA are:

- Irrigation Engineering Manual 1990,
- Punakha-Wandi Valley Development Project Irrigation Manual-1992, and
- Feeder Road Manual

The existing Irrigation Engineering Manual 1990 was prepared by the Department of Agriculture (DOA) as a draft version in 1990 and revised subsequently in 1994 and 1998. The manual has 11 chapters and 10 appendices focusing mainly on the survey, design and construction supervision of irrigation schemes. The Punakha-Wangdi Valley Development Project Irrigation Manual was prepared by Hunting Services and Mott MacDonald Consultants in 1992 as part of the Punakha-Wangdi Valley Development Project under IFAD assistance. The manual has 6 chapters and 5 appendices. The Feeder Road Manual has three volumes covering hydrology, irrigation design and feeder road respectively. The irrigation design manual has 12 chapters and 7 appendices.

1.4.2 Interaction with stakeholders

The interaction with engineers of Engineering Division in DOA was carried out on an individual basis and their expectations regarding the updating of the Irrigation Engineering Manual were collected. The main staff consulted included the chief engineer, executive engineers, deputy executive engineers and engineers working in the Engineering Division of DAO.

After reviewing the existing manuals and interaction with the concerned DOA staff, the Consultant prepared a draft table of contents for the Manual. The proposed draft table of contents was presented in the Technical Advisory Committee (TAC) meeting on 15th December 2014 along with the basic format of the manual and was endorsed. Similarly, the draft table of contents was presented to the Engineering Division on 19th December 2014.

1.4.3 Conceptual framework of the manual

The basic format of the manual includes the introduction, design concepts , design procedure and worked out examples. The design of irrigation structures follows a farmer friendly design with simple and understandable procedures and illustrations accompanied by photos and sketches wherever possible.

1.5 Contents of the Manual

The Irrigation Engineering Manual is structured in nine chapters. The manual starts with this introductory chapter which presents irrigation in Bhutan, the methodology adopted and the contents of the manual. Chapter two describes the project study and survey procedures highlighting identification and feasibility studies. This chapter also deals with the naming and coding of irrigation schemes and scheme prioritization criteria. Chapter three deals with hydrology and agro-meteorology in irrigation project design. This chapter highlights the procedures of assessing water availability for irrigation, flood flow assessment and assessment of crop and irrigation water requirements including water balance assessment. Chapter four provides basic concepts of diversion headworks and intakes along with their design procedures and worked out examples. This chapter also covers the concept of pump irrigation with its design procedures and a worked out example. Chapter five provides the concept, design procedures and worked out examples of the canal system. This chapter also explains the types and layout of the canals, lined and unlined canals, piped canals and canals in unstable areas. Chapter six presents the design concept, procedures and worked out examples of the canal structures, and highlights various types of flow regulating structures, conveyance structures, cross-drainage structures, and sediment control structures. Chapter seven highlights the design concept and procedures of river training and flood control structures. This chapter also presents the rivers and flood prone areas of Bhutan. Chapter eight presents the concept of micro irrigation, its application and design procedures. This chapter also describes different types of micro irrigation and their advantages and disadvantages in the context of Bhutan. Chapter nine, which concludes the manual, addresses project evaluation and provides procedures for cost estimates, benefit assessments from irrigation projects and the basic economic analysis concepts involved.

CHAPTER-2

2. Project Study and Survey

2.1 Naming and Coding of Irrigation Scheme

Most irrigation schemes in Bhutan are named according to the location of their command area. For rehabilitation schemes, the existing names of the schemes should be adopted. For new irrigation schemes, the names should be based on the location of the command area or be decided by the community. For the scientific identification of the irrigation schemes, it is essential to provide codes for each scheme which should be based on the national identification system. The coding system adopted in the National Irrigation Information System (NIIS) seems appropriate and is based on geocoding standards published by the Ministry of Information and Communication (MOIC). The same coding hence should be adopted for future irrigation development and management. An example of the coding system is as follows:

01002

The first three digits represent the Gewog code while the last two digits represent the irrigation scheme code. 01002 is the Phusa irrigation scheme in Getana Gewog of Chhukha Dzongkhag.

2.2 Project Development Cycle

All irrigation schemes encompass several development cycle stages, starting with planning, followed by implementation, monitoring and evaluation. Each stage has many activities to be carried out to fulfill the specified purpose of that stage. The details of the stages and associated activities are presented in the following table (Table 2-1).

Table 2-1 Project development cycle

Project stage	Main activities	Purpose
Irrigation planning	• Assess broad elements of irrigation • Appraisal of the irrigation development	• To establish the objective and scope of irrigation development
Identification survey	• Facilitate farmers awareness • Perceive needs by farmers • Farmers request for assistance	• Ensure development is demand driven
Pre-feasibility study	• Initial field visits and PRAs • Collect existing physical and socio-economic data • Stakeholders analysis • First approval or rejection of pre-feasibility by stakeholders	• First hand assessment of irrigation potential • Identify farmers objectives, requirements, and capabilities
Feasibility study	• Detailed physical data collection and investigations • Topographical survey • Hydrological survey • Socio-environmental survey • Agricultural survey • Study on climate change impacts • Financial and institutional review	• Ensure adequate resources to meet farmers objectives • Ensure resources available for proposed development • Provide opportunities to modify design • Provide basis for loans, management and O&M

Project stage	Main activities	Purpose
	• Preliminary design and costs • Participation of farmers in design choices • Initiation of appropriate farmers organization • Prepare project feasibility report including economic appraisal	
Detailed design	• Final data assessed • Detailed design, quantities, and contract documents prepared • Funding arrangements organized • Farmers contributions clearly determined	• Finalize details and costs • Match designs with farmers capability • Enable farmers to take responsibility
Implementation	• Project approval • Tender received • Selection of contractor and contract agreement • Farmers participation in construction • Completion of the construction • Hand-over to farmers • Training to farmers on cultivation, on-farm water management, and O&M	• Enable cost-effective choice • Assure payment for works and materials • Promote sense of ownership • Promote effective use of water • Farmers assume responsibility
Monitoring and evaluation	• Regular review of performance • On-going training d extension	• Ensure targets are achieved and sustained • Encourage continued improvement

2.3 Irrigation Planning

Irrigation is the process of applying water to soil for plant growth, and involves the acquisition, conveyance, distribution and application of this water. There are significant spatial as well as temporal variations of rainfall and irrigation aims to supplement natural rainfall. When planning an irrigation scheme, the water requirement and its availability at the particular location need to be assessed first in addition to the availability of suitable land for agricultural production. The water availability and requirement together decide the area to be irrigated and the size and capacity of the conveyance system. An irrigation scheme can be planned and designed in an area when the following points are met:

- Suitability of land for agricultural production,
- Favorable climate for the growth of crops,
- Adequate availability of water.

In addition to these three points, farmers' willingness also has to be considered in planning the scheme. Small irrigation schemes can be developed relatively quickly while large irrigation schemes may take time for proper planning, design and construction. Depending upon the size of the project, the planning stage itself consists of three phases: preliminary planning, which includes pre-feasibility and feasibility studies, detailed planning of land and water use, and detailed design of irrigation structures and canals.

Survey and study works are basic requirements of any irrigation project planning. The success or failure of the implementation of a project depends on the accuracy of its study and survey. Survey works form the basis for the study of the project in which data and information are collected both from primary and secondary sources. The type and nature of the study and survey depend on the stage, size and importance of the project. For small scale irrigation schemes, it may be sufficient to carry out two levels of study: the identification study and the feasibility study. In addition, a study of climate change impacts and possible adaptation measures needs to be carried out during the planning and study stages of irrigation project development. The following sections deal with the major survey and study works of an irrigation project.

2.4 Identification Study

2.4.1 Introduction

The identification of the scheme is the very first step in the project development process. The Dzongkhag/Gewog level office should ask farmers through their leaders about the necessity of rehabilitating the existing scheme or constructing a new scheme. In addition, local level officers should disseminate information about the plans and program of the Department of Agriculture (DOA). This information dissemination should be carried out through meetings with farmer leaders and Gewog heads (Gups). The main objective of information dissemination is to seek genuine demand of the project from the farmers. A standardproject demand form should be developed and distributed to the farmers. Examples of genuine demand may be:

- The request for development must be genuine from the majority of the farmers,
- Farmers are willing to contribute to cover their part of development costs,
- Farmers are willing to organize a water users association, enter into necessary agreements with the office and undertake operation and maintenance of the system after completion,
- Soils of the command area are suitable for irrigation (only for new schemes)

Prior to starting the identification study, the request form of the farmers should be reviewed to establish whether it fulfills the minimum requirements that have been set. This process is the initial screening which aims to identify genuine requests, prioritize projects with high farmer commitment, identify the type of project (new/rehabilitation) and ensure that the set principles and criteria are met.

Criteria for selection should be prepared by the Engineering Division of DOA based on the principles and objectives of the particular program. The criteria for the selection of the project at the identification stage should include but not be limited to the following:

- At least eighty percent of the beneficiary households should sign the project request form;
- There are no potential water use disputes in the stream or river;
- The beneficiaries commit to take future O& M responsibilities;
- Sufficient water is available.

After the very first initial screening, a three-step project identification study needs to be conducted including: (a) Desk study, (b) Field visit, and (c) Analysis and reporting. The identification study should be carried out by a team comprising at least an irrigation engineer and an agriculture officer.

2.4.2 Desk study

At this step the project location should be identified on 1:50,000 scale topographic maps. The team should collect and review previous reports if any are related to the project. The team should also study the demand form submitted by the farmers. In the case of a rehabilitation project, the team should verify the information in the demand form with the National Irrigation Information System (NIIS) of DOA.

2.4.3 Field visit

After the desk study, a message should be sent to the farmers informing them about the tentative date of the field visit. The tools and equipment to be taken on the field visit are: a copy of the project request form, the project identification questionnaire, topographical maps, note books and necessary stationary, GPS, stop watch, calculator, camera, and a 3 m long measuring tape. The field visit should consist of at least three activities: initial meeting, walk through, and concluding meeting.

a) Initial meeting: local farmers should be met, discussions should be held and arrangement made for the scheme inspection. During this meeting an overview of the project should be made:
    - Is the scheme a new or improvement project?
    - Why have the farmers requested the project?
    - Where is the intake site, main canal and command area?

b) Field Inspection: visit the intake site, main canal alignment and command area. The team needs to assess the following main points in addition to filling in the questionnaire:
    - History of the project;
    - Conditions of intake site;
    - Water availability and issue of water use;
    - Suitability of land area;
    - Presence of cross-drainage works and technical complexities;
    - Length of main canal and landslide zones if any;
    - Major crops grown and cropping pattern;
    - Farmers attitude towards project implementation

c) Concluding meeting: After the field inspection and before leaving the site, the study team should hold a concluding meeting with the farmers. The team should share their experience of the scheme inspection, major findings and the possibility for further study.

2.4.4 Analysis and Reporting

Based on the data and information collected during the field visit, the team needs to analyze the project findings and finalize the identification study. The analysis should be based on technical, environmental and social aspects of project identification. Based on this analysis, the team has to prepare a report stating the project recommendation for further actions. The report shall comprise mainly the introduction to the project, the project summary sheet, the completed questionnaire, a topographical map with the project location, area and water resources calculation, and the recommendation.

2.5 Feasibility study

2.5.1 Introduction

The feasibility study is the basis for project implementation and is carried out by a team of experts with backgrounds in engineering, agriculture, and socio-environmental fields. This study normally forms the basis of financing by external funding agencies or the Government. The study assesses the technical feasibility, economic viability and institutional suitability of the project's implementation. The feasibility study is carried out in the following steps:

i. Desk study
ii. Field survey work
iii. Field data analysis and feasibility design
iv. Cost estimate
v. Economic analysis
vi. Project reporting and recommendation

2.5.2 Desk study

The desk study is a review of the work carried out at the project identification stage. This study generally needs to examine two aspects. The first is the list of outstanding matters to be studied, and any possible scheme alternatives. There may be several outstanding issues not touched during previous studies which should be stressed during the feasibility study. Scheme alternatives need to be reviewed in the feasibility study, which may include the following:

For new schemes:

- Alternative water sources from different rivers, pumped supply, ground water, supplementary rivers, etc
- Alternative intake site on the river, and
- Alternative canal alignment

For rehabilitation schemes:

- Extension of command area,
- Combining several schemes,
- Revised canal alignment, and
- Revised intake site

2.5.3 Field Survey Work

The field survey work may differ slightly based on the type of the scheme (new or rehabilitated). The following are the main activities to be carried out during the field survey work:

i. List the names, command area and water withdrawal amounts (in different seasons) of existing irrigation systems and other uses (drinking water supply) located in the vicinity of the rivers/streams. Assess the impact of the system's diversion on downstream water users, particularly where additional irrigation water is needed for the extension area. Verify the river water flows and intake withdrawal amount through flow measurements.

ii. Locate the scheme on 1:5,000 maps based on available ortho-photos and/or cadastral maps. Google Earth may also be a good source for suitable system maps. The location of the main structures should also be marked.

iii. Collect data regarding the command area, irrigation source, water rights, catchment condition, soils, land use, in a designated format.

iv. Discuss with the farmers group and confirm the project request and the list of the WUA members with individual landholding distribution within the existing and proposed extension area.

v. Basic demographic parameters will be collected such as number of households, household size, and life expectancy. Other social parameters such as education level, major source of income, and food security will also be recorded.

vi. Together with the social mobilizer undertake a "walk through" with representatives from the farmers group to determine the present and future irrigated areas and identify necessary works.

vii. Perform engineering surveys such as river long sections and cross sections. Similarly, undertake longitudinal and cross sections surveys of the main and branch canals.

viii. Conduct soil surveys and basic soils tests within the command area at a density of about one pit site for every 10 ha of command area to assess soil suitability for agriculture.

ix. Perform flow measurement to verify existing canal carrying capacities, seepage losses, and verification of water requirements and other hydraulic parameters.

x. In consultation with farmer groups, determine the location of the required major works in the canal alignment with appropriate digital photos, which will be incorporated into the infrastructure improvement plan.

xi. Collect and confirm data on the existing irrigated area; cropping pattern; input use including fertilizer, seeds, and pesticides; yields, in a systematic manner based on landholding sizes.

2.5.4 Field data analysis and design

The collected data and information has to be analyzed, on return to the office, before a final decision can be taken on the project. The major activities of the analysis are related to the estimation of the net command area and water balance at the source river. Before starting the canal design, it is important to check the water balance to confirm whether variable water is sufficient to the proposed command area in all seasons. After analyzing the data, feasibility design shall start and comprises the following:

i. System layout: Using topographic maps a plan of the system should be made showing the source, primary canal alignment, major structures and main part of the command area. A schematic line diagram of the system layout needs to be prepared;

ii. Canal design: For feasibility level design, typical canal cross sections can be related to canal discharge and ground conditions. For rehabilitation schemes canal designs, are only required for reaches where complete remodeling is needed.

iii. Structure design: It should be prepared for intake and major structures. Typical standard drawings may be used for minor structures.

iv. Standards for feasibility level design

- Head Works
  - Drawings (plan, typical section) showing location and levels and dimensions should be prepared and principal quantities estimated,
- Layout
  - A layout plan of the scheme and command area showing primary and secondary canal alignment should be prepared,
- Canals
  - Canal routes, capacities, slope, levels and dimensions should be established,
  - A longitudinal section of the canal should be prepared,
  - Typical cross sections should be drawn,
  - Earthwork quantities should be worked out,
- Structures
  - Drawings of the typical structures should be prepared,
  - Each size and type of structure should be assessed,
  - Drawings of major structures showing location, levels and dimension should be prepared,
  - Principal quantity should be calculated

2.5.5 Topographical Survey

The topographical survey is the process of determining the position both in plan and elevation of natural features of the area. The extent of the topographical survey depends upon the size, type, and level of the project study. For small scale irrigation schemes, the topographical survey includes:

- Benchmark survey,
- Traverse survey,
- Alignment survey, and
- Longitudinal and cross sectional survey

The topographical survey standard for small scale irrigation schemes may be as follows (Table 2-2).

Table 2-2 Topographical survey standards

S.N	Principal use	Details
1	Scheme identification	• Use existing maps but supplement with site inspection
2	Feasibility study	• Use existing maps, site sketches • Carry out traverse survey (GPS) • Benchmark survey at the rate of 0.50 to 1.0 km intervals • Carry out detailed site survey for structures-intake/headwork, major cross drainage structures • For command area topographical survey take 100 m interval grid for contour interval of 0.50 to 1.0 m.

S.N	Principal use	Details
		• Carry out canal alignment survey with L-section and X-sections
3	Construction stage	• Use design drawings • Check and agree levels and dimensions • Carry out additional survey for details of the site if required

2.5.6 Hydrological survey

Hydrological survey is carried out to provide data and information for the assessment of water availability for irrigation, and assessment of flood flows for the intake/headwork and cross drainage works. The data and information to be collected during the hydrological survey are:

- Catchment area of the source river/stream,
- Catchment conditions such as land use land cover, slope and length of longest channel,
- Long term rainfall and climatic data within the catchment area;
- Water flow records of source river/stream (if available),
- Flow measurement of the river/stream,
- High flood level or the source river at the point of intake or diversion weir (trash marks if any),
- Information on river bed material

2.5.7 Socio-economic survey

The socio-environmental survey is carried out to determine the social structure of the community and its livelihoods. The survey includes the collection of quantitative as well as qualitative data and information on social structure, socio-cultural institutions, and economic activities of the farmers of the scheme command area. Some of social and economic indicators of the community are as follows:

Social indicators:

- Willingness/commitment- verbal request, formation of committee, submission of request form,
- Education- literacy, school and college, awareness about irrigated agriculture, prior experience with irrigation,
- Rural organizations,
- Family size- male/female, economically active members,
- Migration- temporary, permanent, foreign/urban areas

Economic indicators:

- Land holding size- landless, marginal land (< 1 acre),large land owners> 10 acre),
- Main occupation- agriculture, service, labor, foreign service, business,
- Source of income- agriculture, service, remittance etc,
- Expenditure- food, cloth, schooling, festivals, livestock, agriculture

2.5.8 Agricultural Survey

For small irrigation schemes, the agricultural survey may include data and information regarding the soil type, land use and agriculture practices of the command area. The soil survey may include the assessment of the type of soil in the command area and its suitability for irrigated agriculture. The soils may be alluvial, sandy, gravel and boulder mixed.

The land use survey may include the general assessment of land use in the command area, which may classify the percent of agricultural land, forest land, grazing land, wetlands, National Parks and reserve forest area.

The agriculture survey includes the collection of data and information on:

- Existing cropping pattern,
- Existing crop yields,
- Inputs used and its availability,
- Marketing facility and labor situation

2.5.9 GPS Survey Methodology

The Global Positioning System (GPS) aided walk through method uses relatively recent improvements in survey aids. GPS equipment stores and downloads tracks and waypoints, digitized or digital cartography, high resolution satellite imagery, and elaboration/presentation of the survey results in GIS software.

During GPS walkthrough surveys, tracks of the main and branch canal alignments will be recorded and conveyance or structural problems in these canals will be marked on the tracks. In addition to the canal alignment surveys, the command area boundaries of the system will be recorded with the GPS equipment. The canal track and marked waypoints will be downloaded and processed in ArcGIS or compatible GIS software, using Garmin DNR software or other GPS equipment software to convert the recorded GPS file format to the GIS shape file format. In the GIS software, the recorded canal tracks and command area boundaries will be overlaid on the newly available digital topographical maps from the Survey Department and if available high resolution satellite imagery that can be downloaded commercially through a licensed version of Google Earth or the access to the Digital Globe archives through the Global mapper software. High resolution images can also be composed from the free version of Google Earth by downloading the required coverage tile by tile. There is also software available at minimal cost that processes the tile by tile downloading from Google Earth automatically.

The results of the surveys will permit the preparation of accurately geo-referenced mapping of system layouts, command area boundaries and system deficiencies. The accurate geo-referencing of the surveys and resulting maps will allow verification of the accuracy of the surveys through the overlay on the digital topographical maps and especially on the high resolution satellite imagery. Conversion of the Shape format to the Google KMZ (KML) propriety format will facilitate the display of the layout maps directly in Google Earth which allows a 3D rendering of the canal system layout and command area (Shape files can be directly converted to KMZ/KML format in ArcGIS. Verification of the survey and mapping results in Google Earth 3D further aids in assessing the correctness of the surveys and resulting system layout and facilitates the presentation to a wide audience of survey results in an easy and cost effective manner.

2.6 Environmental Impact Study

2.6.1 Legal Provisions

According to the Environment Assessment Act 2000, Environmental Clearance (EC) is mandatory for any project/activity that may have adverse impact(s) on the environment. The Regulation for the Environmental Clearance of the Projects 2002 defines responsibilities and procedures for the implementation of the Act concerning the issuance and enforcement of the environmental clearance. In addition, the Regulation under its Annex-2 mentions that all irrigation channels (irrigation schemes) to be developed need to obtain environmental clearance from the competent authority. The National Environment Commission (NEC) has revised, updated and published eight environmental assessment guidelines for the forestry, general, highway and roads, hydropower, industry, mining, tourism and transmission sectors. However, the irrigated agriculture sector has not yet been included in these sectoral guidelines and the general environmental assessment guideline applies to irrigation project preparation and implementation.

The environmental assessment process endeavors to mitigate and prevent undesirable impacts from the development of the irrigation project. It must include all of the procedures required under Bhutanese law intended to identify the means to ensure that a project is managed in an environmentally sound and sustainable way.

2.6.2 Potential socio-economic and environmental impacts

The major source of environmental effects is from infrastructure works, the impacts of which vary according to the type, size, and location. Highly significant and/or irreversible adverse environmental impacts are not expected from the implementation of the irrigation project. In addition, the rehabilitation of existing age-old community managed irrigation schemes will have negligible or no impacts on the environment. The environmental issues related to the rehabilitation of existing schemes and construction of new irrigation schemes are:

- Landslide and soil erosion,
- Loss and/or degradation of forest and vegetation cover,
- Health and safety, and sanitation issues, and
- Construction period disturbances

However, these impacts are readily manageable as mitigation procedures for such impacts are in practice in Bhutan. In addition, the irrigation development activities will have socio-economic impacts on the project site. Potential socio-economic impacts incurred from irrigation activities include:

- Livelihood impacts as a result of land acquisition for canal alignment,
- Impacts on structures requiring relocation,
- Impacts on local irrigation facilities,
- Loss of crops and trees,
- Loss or replacement of rural structures,
- Construction-related impacts, such as public health, dust and safety,
- Impacts on vulnerable and disadvantaged groups

2.6.3 Procedures for Environmental Impact Assessment

The Environmental Clearance (EC) application needs to be backed up with information on no objection certificates (NOC) and Environmental Information (EI). The EI need to include: potential adverse environment effects, compliance plan, a management plan, and environmental and other benefits of the project. The competent authority checks the EI, social clearance and NOCs, as part of environmental screening and will lead to either issuance of an EC, instructions for further study (Environmental Impact Assessment) or rejection of the application based on the situation. Consultation with affected communities (for social clearance) is expected to take place during the NOC and EA process. The EC issuing agency is responsible for monitoring compliance. The National Environment Commission and/or the competent authority are mandated to monitor compliance.

The environmental impact assessment (EIA) is a tool to identify the potential environmental and social impacts arising from the implementation of the project. According to the Environmental assessment general guideline provides six generic steps in the EIA process:

- Screening,
- Scoping,
- Baseline data generation,
- Impact assessment,
- Mitigation of impact, and
- Environmental management plan

Screening is the first step and determines whether an EIA is necessary for the project or not. The second step is scoping, which aims to establish environmental and social priorities, set the boundaries for the study and define Terms of References (TOR). The third step is to generate baseline data which provides information on the existing status of the environmental and social components of the study area. The impact assessment is the fourth step on which characteristics of the potential impacts, evaluated and predicted using baseline data. Impact predictions are normally made using common methodologies and models. The fifth step is the identification of mitigation measures of potential impacts. These may either be preventive or remedial measures. The last step is the preparation of environmental management plan which should include recommended mitigation measures in specific action plans that will be implemented by the proponent.

2.6.4 Integration of EIA into the project cycle

An irrigation project is accomplished in seven stages; the activities of each stage were briefly presented in section 2.2 Project Development Cycle. EIA plays an important role in each stage of this project cycle. Most of the activities of EIA take place during the pre-feasibility and feasibility stages of the project. Between irrigation planning and pre-feasibility, the EIA process involves site selection, screening, initial assessment and scoping of significant issues. The detailed EIA starts in the feasibility stage, which evaluates significant impacts, collection of baseline data, prediction and quantification of impacts, and review of EIA by the regulatory agency.

Following these initial steps, environmental protection measures are identified, operating conditions are determined, and environmental management is established. In the last phase of the feasibility study, monitoring needs are identified and an environmental plan and monitoring program are formulated. The environmental management plan aiming to reduce adverse environmental impacts is considered in the project cycle starting from the project appraisal stage.

The World Bank funded Remote Rural Communities Development Project (RRCDP) under the Ministry of Agriculture and Forest developed Environmental and Social Assessment Framework Implementation Guidelines and Environmental and Social Screening Procedure Guidelines in 2013.

These guidelines provide seven steps of environmental and social safeguard processing which are listed below :

Step 1 : Preliminary environmental and social information and analysis,

Step 2 : Environmental and social screening and assessment,

Step 3 : Environmental and social recommendations and preparation of sub-project Detailed Project Report (DPR),

Step 4 : Environmental clearance and social clearance,

Step 5 : Preparation of Social Action Plan (SAP),

Step 6 : Implementers site environmental and social management plan,

Step 7 : Compliance and final monitoring

2.7 Climate Change Impact Study

2.7.1 Climate change

Understanding and coping with climate change is an important global issue and serious concerns have arisen in the planning and design of water projects, including irrigation. The increase in greenhouse gas concentrations in the atmosphere has led to global warming, which causes unpredictable and extreme climate events. According to the IPCC, climate change projections to 2100 for South Asia in general and Bhutan in particular consist of increases in average temperatures with relatively warmer weather at higher altitudes and during the dry season, increase in annual average precipitation, and continued spatial variation in temperatures and precipitation due to complex topography (NEC, 2009). The number of hot days is projected to increase. Increased temperatures will be accompanied by changes in cloud cover, precipitation, air humidity, radiation, wind, and other meteorological elements (NEV, 2011). These effects will lead to changes in hydrological cycles with impacts on mass water balance, soil moisture, and evapo-transpiration. Climate change will affect agriculture through higher temperature and more variable rainfall. The following sections present the major impacts of climate change on irrigated agriculture and possible adaptation measures to cope with the potential impacts of climate change.

2.7.2 Potential impacts of climate change on irrigation

Irrigated agriculture is planned and designed based on location specific physical conditions, including the climate. Climate change induced by global warming poses significant risks to irrigated agriculture in general and water management in particular. According to the results of various research studies on the impacts of climate change, climate variability has significant impacts on crop area and crop production, especially in periods of droughts and floods. In addition, climate change will affect irrigated agriculture through increased crop evapo-transpiration, changes in the amount of rainfall, and variations in river flows. Hence, the impact of climate change on irrigated agriculture must be considered in a wider context which includes water demand, water quality and competition over water at the system as well as basin levels. The potential threats of climate change and associated impacts are presented here in tabular form (*Table **2-3***).

Table 2-3 Potential impacts of climate change on irrigation

Climate change/threats	Impacts
Increase in rainfall	• Higher stream or river flows • Benefit to in-stream ecology

Climate change/threats	Impacts
	• More water available for irrigation
Decrease in rainfall	• Lower stream or river flows • Pressure on environmental flow • Pressure on water for irrigation
Change in rainfall patterns	• Increase in pressure on water availability due to seasonal variation • Reliability of irrigation water in question
Increase in risks of droughts	• Very low flow in streams and rivers • Pressure on environmental flow • No water for irrigation abstraction
Increase in temperature and increase in hot days	• Adverse effect on stream ecology • Increase risks in weed growth • Increase invariability of water quality • Increase in irrigation water demand • Increase in maintenance cost of water infrastructure • Increase in water demand for livestock
Increased frequency of heavy rainfall events	• Increase frequency of large floods • Increase in sediment yields • Increase in erosion in catchment • Increase risk of water infrastructure damage • Increase in landslide events • Increase in susceptibility of canal alignment
Increase in windiness	• Increase in erosion of top soil • Increase in irrigation water demand

The potential threats and their impacts suggest that climate variability and change will be the main drivers of change and will necessitate specific climate change-related responses. The impacts of climate change will vary from one system to another, but the adaptation strategy should consider the context of each specific system. During the planning and study of the irrigation system, it is essential to assess the potential impacts and extent of vulnerability of the system in order to plan the mitigation as well as adaptation measures.

2.7.3 Impact and vulnerability assessment

During the pre-feasibility and feasibility study of the irrigation scheme, a vulnerability assessment needs to be carried out to ascertain the extent of climate change impacts. Vulnerability is defined as the extent to which climate change may damage or harm the physical infrastructure in general and the irrigation system in particular. It depends not only on the sensitivity of the irrigation system, but also on its ability to adapt to new climatic conditions. Vulnerability assessment plays a vital role in the design of appropriate adaptation measures targeting climate change impacts. Vulnerability assessment and adaptation planning involves several steps presented below.

i. Determining the scope and defining the assets

The scope describes the limits of the planning tasks including time, geographical area, assets to be covered, and resource availability for assessment.

ii. Baseline assessment:

The baseline describes the past and existing situations, trends, and drivers across each of the target systems and analyses the changes to these systems that will occur irrespective of climate change. The main components of the baseline assessment and their respective activities are outlined in tabular form (*Table **2-4***).

Table 2-4 Components of the baseline assessment

Components	Activities
Asset inventory and priority setting	• Provide an overall description of existing and proposed assets • Describe the condition of the target infrastructure system
Past climate variability and extreme events	• Describe the most significant past extreme events affecting the assets • Describe the impact of that event
Climate change threats and opportunities profile	• Gather all existing climate change projects relating to the area
Adaptation audit of past protection measures	• Describe measures taken by community to learn from past extreme events • Assess the effectiveness of those past adaptation measures
Socio-economic status and trends assessment	• Identify users and affected communities • Describe the socio-economic trends of relevance to the asset
Natural system status and trends	• Describe the natural systems such as river banks, catchment area • Describe the status and trends of these natural systems

Source: Derived from ADB/ICEM/GON, 2014

iii. Impact and vulnerability assessment

The method considers four important factors in assessing the vulnerability of the target system and its components to climate changes: exposure, sensitivity, impact and adaptive capacity.

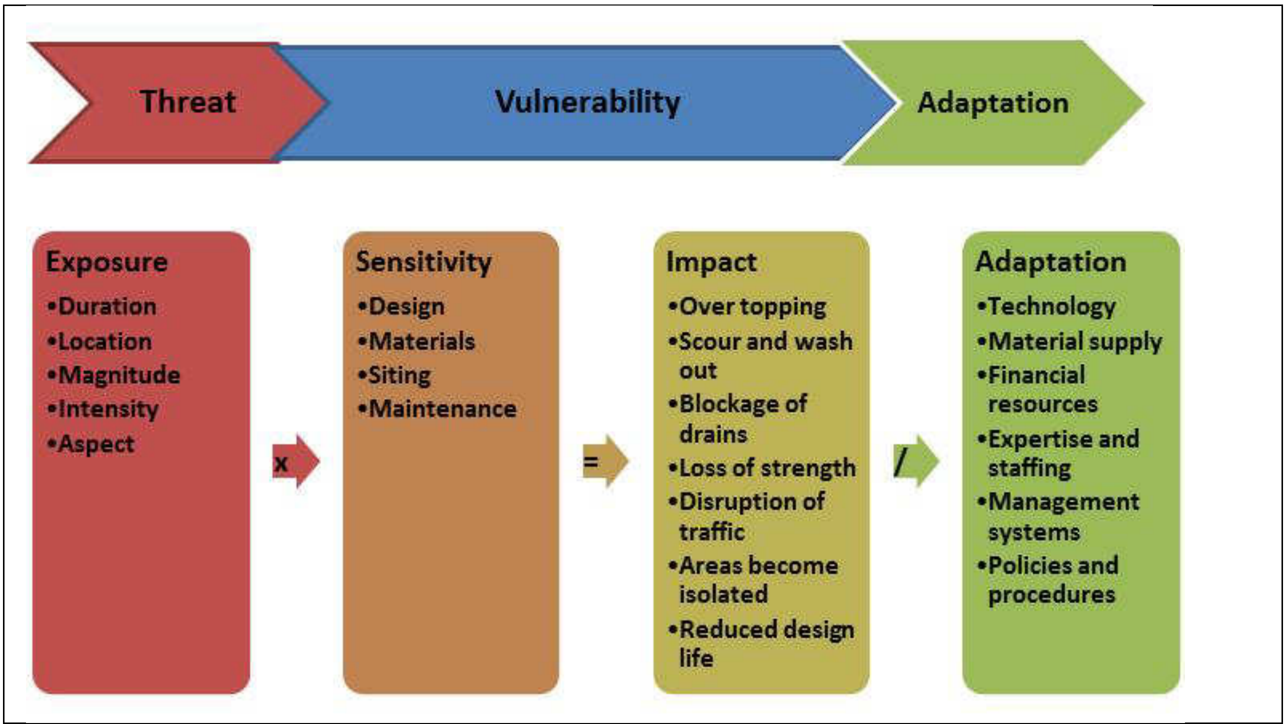
Exposure is the extent to which a system is exposed to the climate change threat. Exposure is best assessed by overlapping maps of past extremes such as floods and droughts and of projected climate changes in the area. Exposure to the threat depends upon the location of the system and threat intensity, frequency, and duration.

Sensitivity is the degree to which a system will be affected by, or responsive to, the exposure. Sensitivity for an infrastructure asset may be influenced by the specific siting, geotechnical character (bank stability, drainage), integrity of design, and integrity of materials and construction.

The potential impact is a product of exposure and sensitivity. The potential impact of the climate threat is rated based on the support tools in the vulnerability assessment matrix. Adaptive capacity is understood in terms of the ability to prepare for a future threat and ability to recover from the impact. Once the impact assessment is complete, the adaptive capacity of the community or organization to prepare for and respond to the impacts needs to be assessed.

When the impact and adaptive capacity are considered, a measure of relative vulnerability can be defined. An example of parameters and issues considered in the vulnerability assessment is presented in *Figure* ***2-1***.

Figure 2-1 Parameters and issues of vulnerability assessment



iv. Adaptation planning

Adaptation to climate change refers to the actions to be taken during the preparation of the irrigation project by the authorities concerned. Adaptation also includes taking advantage of opportunities that may arise due to climate change, as well as responding to negative impacts. Adaptation planning involves developing a range of adaptation options for each of the significant impacts of climate change and then determining priorities for implementation.

Adaptation at farm level

Climate change adaptation measures at the farm level involve crop planning, including the selection of crop varieties and crop calendars to adopt new temperatures and rainfall patterns. The use of crops requiring little water and drought-resistant crops will also be preferred. Increased agricultural diversification, including better integration of trees, crops, fish and livestock will reduce risk and increase resilience of farming systems (FAO, 2013). In addition, farmers will favor more efficient technologies to reduce evaporation losses from the field.

Adaptation at scheme level

Actions for adapting to climate change in irrigation schemes need to be considered in the overall context of irrigation modernization (FAO, 2013), which will enhance efficiency, water allocation and reliable delivery of water. Intermediate storage within the irrigation scheme and access to alternative source of water are some of the options to improve reliability of water for adaptation in irrigation planning and design. A typical example of adaptation planning is presented here (*Table **2-5***). Asset: Chisapani Naubasta Irrigation System, Banke, Nepal; Command area- 306 ha; Beneficiaries-575 HH; Main crops- paddy, wheat, potatoes, pulses and oilseeds; Main components-intake, main canal and command area

v. Mainstreaming adaptation plan

The mainstreaming adaptation plan involves the incorporation of prioritized activities into the system starting from its study through operation and maintenance.

2.7.4 Integration of climate change into project cycle

Like the EIA, the impacts of climate change also need to be integrated into the project cycle as illustrated in section 2.2 of this chapter. In addition to mainstreaming climate change adaptation, it is essential to plan, design and incorporate mitigation measures from potential adverse impacts of climate change into the project cycle.

Table 2-5 Example of climate change adaptation planning

Threats	Impacts	Significance			Adaptation options	Priority adaptation		
High and very high threats	Impacts for high and very high threats	Likelihood (chances of the impact occurring)	Seriousness of the impact	Significance of the impact	Focus on structural and bioengineering options	Feasibility	Effectiveness	Priority
Intake structure • Increased river flows • Flash floods	• Further damage to diversion weir • Unable to raise water levels to intake • Intake blocks with debris • Sediment enters into main canal	VH H VH H	H H H M	VH H VH M	• Rebuild diversion weir with CC consideration • Improve river bed protection downstream of core wall • Increase maintenance	L M M	VH H H	H H H
Main canal • Landslides	• Canals silt up, becomes blocked	M	M	M	• Increase canal clearance • Protect landslides with bioengineering	VH M	M H	VH H
Command area • Increased temperature • Increased rainfall • Storms • Draughts	• Increase ETo and then water demand • More chance of disease to winter crops • Increase rainfall reduces water demand • Storms damage crops • Draughts effects crop yields	H M H H M	VL M VL VH M	L M L VH M	• Introduce less water requiring crops • Introduce more disease resistant crops • Opportunity for alternative cropping patterns • Chose varieties less susceptible to storm damage	M M M M	M H M H	M H M H

Note: VH-very high, H-high, M-medium, L-low and VL-very low

2.8 Report preparation

For the elaboration of the feasibility study, a report needs to be prepared including the following chapters and appendices:

Executive Summary: The executive summary consists of highlights of each chapter including conclusions and recommendations.

Salient features: The salient features will be presented in a specified tabular form which contains relevant data and information on the project.

Introduction: This chapter consists of background information about the project, the objectives and scope of the project, and contents of the report.

Methodology: This chapter covers the approach and methodology of the feasibility study, analysis, design, and report preparation.

Project area: Details of the project area have to be presented in this chapter, which may include location, topography, climate, water resources, upstream and downstream water use, water rights, agricultural situation of the area covering cropping patterns, existing yields of major crops, extension service, and socio-economic conditions of the area. This chapter may also contain the condition of existing irrigation facilities, including WUAs.

Proposed project: This chapter will focus mainly on the proposed planning of irrigation infrastructure to enhance agricultural production. The chapter will contain an agricultural development plan, irrigation system plan, command area, crop-water requirement, proposed structures and their design, operation and maintenance, institutional support, environmental and climate change impacts assessment and implementation arrangement. The assessment of water availability will be calculated using this manual along with the water balance calculation. The proposed cropping pattern will be based on the farmers' preferences and the water available at the diversion point during the cropping seasons.

The potential yield will be based on the potential crop yield adjusted to the local circumstances, taking into account present crop yields under optimal irrigated conditions and average district yields. A project specific implementation schedule will have to be prepared based on specific site conditions.

Economic Analysis: For the assessment of the EIRR and B/C ration economic analysis need to be carried out for the project which is mainly based on the agricultural benefits to be accrued from the project and the cost of project implementation. The agricultural benefit will be calculated according to the crop budget of the "with" and "without" project situations. The cost of the project is assessed with the derivation of rates and quantities.

Appendices: The appendices to the Feasibility Report will include:

Appendix A: Maps, the GIS mapping of the canal alignment, command area boundaries and irrigation status overlaid on the digital topographic map layers,

Appendix B: Drawings, applicable typical drawings and drawings for headwork and major canal structures,

Appendix C: Crop water requirement, the tables generated by the water balance calculation program,

Appendix D: Agriculture data, tabular results of the FGDs,

Appendix E: Cost and quantity estimation, the tables generated by the cost calculation spreadsheets,

Appendix F: Economic analysis,

Appendix G: Project implementation plan, a summary of the proposed interventions detailed in the report,

Appendix H: Photographs.

2.9 Project Selection and Priority Ranking

2.9.1 Project Selection

The irrigation schemes to be selected for implementation should be technically feasible, institutionally acceptable and economically viable.

Technical feasibility: The scheme should be technically feasible in relation to the scheme standard which may include:

- Adequate water is available at the source to irrigate the proposed command area;
- No water rights problems;
- No major technical difficulties mainly at the intake and canal alignment;
- Soil is suitable for irrigated agriculture;
- No major landslides, erosions in the area

Institutional acceptability: The scheme should be based on the genuine demand of the beneficiary farmers in respect to:

- Majority farmers are committed the implementation of the scheme (80%);
- Written agreement exists with farmers;
- WUA exist or are in the process of formation;
- Suitability to diversify irrigated agriculture;
- Availability of market for inputs and products

Economic viability:

- Per unit cost of the scheme shall be within the defined limit;

2.9.2 Priority Ranking

The priority ranking of the scheme is carried out based on techno-economic, social and environmental criteria. The techno-economic criteria includes cost per acre or total cost of the scheme, water availability at the source, size of the command area and length of the main canal. The social criteria include the number of beneficiary households, the food security situation, and beneficiary interest in the scheme development. The environmental criteria include climate resilient scheme, focus on crops requiring less water and proximity to national parks. The details of the criteria are presented in tabular form (Table 2-6).

Table 2-6 Evaluation criteria for scheme priority ranking

S.N	Components	Weightage	Maximum score
1	Cost per acre/total cost	15 %	10
2	Water availability at source	10 %	10
3	Size of command area	15 %	10
4	Length of main canal	10 %	10
5	Number of households	10 %	10
6	Food security	10 %	10
7	Community interest/WUA	10 %	10
8	Climate resilient	5 %	10
9	Focus on crops requiring less water	5 %	10
10	Proximity to national park	10 %	10
	Total	100%	

CHAPTER-3

3. Hydrology and Agro-Meteorology

3.1 Hydrology of Bhutan

The hydrology of Bhutan correlates with the monsoon rainfall pattern from the Bay of Bengal, which has significant variation in time and space. Monsoon rainfall starts in June, is intense in July and August, and peters out in September. November, December and January are the dry months, while pre-monsoon showers occur in April and May. The mean annual rainfall ranges from 500 mm to 5,000 mm. The heaviest rainfall occurs in border areas with India, decreasing rapidly northward. All four major rivers, Amochhu, Wangchhu, Punatsangchhu, and Drangmechhu (Manas), flow to Brahmaputra River. River flows are high from June to September and start to recede from October. The flow of the rivers becomes low during the months of December to April. The long term mean annual flow of these rivers is 2,325 m^3/s, which is equivalent to 73,000 million m^3 per year (NEC, 2003).

The Department of Hydro-Met Services (DHMS) under the Ministry of Economic Affairs is maintaining a network of hydrological and meteorological stations. The hydrological network comprises 13 principal stations equipped with cableways, gauges, and pressure transducers connected to data loggers and 19 secondary stations equipped with staff gauges. Similarly, the meteorological network comprises 20 Class A stations with a full range of weather parameters and 74 Class C stations with temperature, rainfall and humidity data. Almost all of these hydrological stations are located on the major rivers and their main tributaries and seldom fulfill the requirements of the design of irrigation schemes.

3.2 Water Availability for Irrigation

3.2.1 Concept

According to the Food and Agriculture Organization (FAO), the availability of water for irrigation is assessed at 75% to 80% reliability of the full supply. The 80% reliability of the full supply is equivalent to a 1 in 5 year return period event. This means that 80% of the time (4 years out of 5 years) there is at least enough water available to meet full water demand for irrigation, and 20% of the time (1 year in 5 years) there will be undersupply. The designer has to estimate river flows for a particular location where intake or headwork can be constructed which is expected to exceed 80% of time. Given the large spatial variation in run-off in Bhutan and the significant number of small catchments it is unlikely that one will find gauging stations at the point of irrigation diversion. For gauged catchments, the assessment of water availability is carried out with the help of frequency analysis; for ungauged catchments, other methods need to be adapted. In Bhutan, such methods have not yet been developed, and this section tries to identify the best ways to estimate the available reliable flow for irrigation from ungauged catchments.

3.2.2 Frequency Analysis

Frequency analysis is a statistical method used to show that flow events of a certain magnitude may on average be expected once every *n* year. It is generally carried out to estimate the design flow from recorded flow data covering more than 10 years. From the recorded monthly flow values, 1 in 5 or 80% reliability flow can be obtained from the mean and standard deviation values for a particular month using the following formula:

$$Q_{80} = Q_{mean} - 0.8418S$$

Where, Q_{80} = 80% reliable monthly flow in m^3/s,

Q_{mean} = mean monthly flow in m^3/s,

S = standard deviation of the monthly series

The factor 0.8418 is the 1 in 5 reduced variate of the normal distribution assuming that the monthly flow series follows a normal distribution. The worked out example of frequency analysis to assess 80% reliable flow is presented hereunder (***Table 3-1***).

Table 3-1 80% Reliable Flow of Thimchhu at Lungtenphug (m3/s)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	5.19	4.22	4.50	4.60	8.02	15.67	47.59	58.92	41.05	17.69	9.84	6.61
1993	5.41	4.47	4.21	6.85	14.26	25.30	37.01	68.67	51.78	23.42	11.53	7.20
1994	5.51	4.74	5.09	6.13	10.82	10.82	30.78	50.41	42.39	14.91	8.50	6.00
1995	4.75	4.12	5.83	7.18	13.17	30.92	66.40	63.14	54.11	29.35	16.93	10.10
1996	7.87	5.85	6.49	9.87	12.35	26.58	61.09	60.11	63.56	39.27	11.74	8.58
1997	6.05	5.62	4.44	5.36	12.55	25.43	57.93	67.15	51.69	21.40	10.90	7.90
1998	5.38	3.75	3.88	6.77	12.78	22.35	74.06	80.83	45.90	28.37	13.02	7.58
1999	5.43	4.06	3.43	4.14	14.70	34.18	61.32	75.84	55.47	29.93	15.22	7.54
2000	5.93	4.54	4.79	10.13	19.49	37.12	63.66	76.50	62.35	18.13	9.50	6.09
2001	6.50	6.17	5.11	5.82	9.86	28.26	36.62	73.05	52.02	32.55	11.81	6.64
2002	4.96	4.23	3.60	5.68	10.28	34.18	76.91	69.95	37.94	19.43	9.92	6.16
2003	4.61	3.98	3.56	6.47	8.74	31.61	62.41	56.52	53.58	24.39	12.69	7.31
2004	5.17	4.44	5.11	5.32	10.49	32.09	60.52	61.88	34.98	27.51	11.12	6.53
2005	4.71	4.06	3.78	4.22	8.34	11.45	45.86	51.72	29.90	27.31	11.99	6.71
2006	4.72	3.81	3.20	4.12	13.13	29.98	52.61	53.22	49.67	23.22	10.66	6.73
2007	5.07	4.44	5.26	8.49	12.13	16.18	48.45	56.70	58.17	23.17	10.76	6.18
2008	4.39	3.37	3.34	4.98	8.46	36.84	57.85	70.33	40.17	21.01	11.14	7.32
Mean	5.39	4.46	4.45	6.24	11.74	26.41	55.36	64.41	48.51	24.77	11.60	7.13

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
SD	0.85	0.76	0.95	1.84	2.90	8.46	12.90	9.26	9.56	6.11	2.05	1.04
80% Reliable	4.68	3.83	3.65	4.69	9.30	19.28	44.50	56.61	40.47	19.62	9.88	6.25

3.2.3 Similar catchment transformation method

When the diversion site is located near the gauged site, the monthly flows at the diversion site is calculated using an area ratio.

$$Q_d = Q_g\left(\frac{A_d}{A_g}\right)$$

Where, Q_d = monthly discharge at diversion site in m^3/s,

Q_g = monthly discharge at gauged site in m^3/s,

A_d = catchment area at diversion site in km^2,

A_g = catchment area at gauged site in km^2

*Worked out example (Table **3-2**):*

Catchment area of the stream at intake site (Ad) = 7.50 km2

Catchment area of Thimpu Chhu at Lungtenphug (Ag) = 663.0 km2

Table 3-2 Worked out example of 80 % reliable flow

Description	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Q80 at Thimpu chhu	4.68	3.83	3.65	4.69	9.30	19.28	44.50	56.61	40.47	19.62	9.88	6.25
Q80 at intake site	0.05	0.04	0.04	0.05	0.11	0.22	0.50	0.64	0.46	0.22	0.11	0.07

3.2.4 Empirical Methods for Ungauged Catchment

A method to assess water availability from ungauged catchments does not exist in Bhutan. The Water and Energy Commission Secretariat of Nepal has developed a method (WECS method) for the assessment of river flows from ungauged catchment areas larger than 100 km^2. This method is based on a regression analysis of past data. Although it was developed for Nepal, and considers the country as one unit, it would be useful for Bhutan, which has similar topography and hydrological characteristics. For long term average monthly flows, all catchment areas below 5,000 m are assumed to contribute flows equally per km^2 area. The average monthly flows can be calculated by the equation:

Q_{mean} (month) = C x (Area of Basin)^{A1} x (Area below 5000m +1)^{A2} x (Mean Monsoon precipitation)^{A3}

Where Q_{mean} (month) is the mean flow for a particular month in m^3/s,

C, A_1, A_2 and A_3 are coefficients of the different months (Table 3-3).

Table 3-3 Values of coefficients for WECS method

Month	C	A_1	A_2	A_3
January	0.01423	0	0.9777	0
February	0.01219	0	0.9766	0
March	0.009988	0	0.9948	0
April	0.007974	0	1.0435	0
May	0.008434	0	1.0898	0
June	0.006943	0.9968	0	0.2610
July	0.02123	0	1.0093	0.2523
August	0.02548	0	0.9963	0.2620
September	0.01677	0	0.9894	0.2878
October	0.009724	0	0.9880	0.2508
November	0.001760	0.9605	0	0.3910
December	0.001485	0.9536	0	0.3607

Note: The units of flow are in m^3/s and a power of 0 indicates that the particular parameter does not enter into the equation for that month.

Example: If the catchment area is 12.54 km^2 and monsoon precipitation is 980 mm, the mean monthly flow is calculated as follows (*Table **3-4***). The equation for the month of July is

Q_{mean} July = 0.02123 (basin area below 5,000 m + 1)$^{1.0093}$ X (mean monsoon precipitation)$^{0.2523}$

Q_{mean} July = 0.02123 * (12.54+1)^1.0093 *980^0.2523 = 1.76 m^3/s

Table 3-4 Example of mean monthly flow

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Flow, m^3/s	0.18	0.16	0.13	0.12	0.14	0.52	1.67	2.08	1.60	0.72	0.29	0.20

3.2.5 Climate change consideration in water availability

Climate change has significant impacts on water availability for irrigation, which is one of the key characteristics of irrigation scheme design. Assessment of the accurate reliable flow at the diversion site is the major requirement of the design. In general practice, 80% reliability of available water is considered adequate for irrigation, which means that in 4 out of 5 years the assessed flow shall meet the irrigation demand. In the context of climate change, 9 out of 10 years reliability can be considered for local water availability. The methods described above for ungauged catchments are based on regional experience and do not perfectly represent Bhutanese hydro-meteorological conditions. Hence, the designer also needs to explore additional methods and compare with local conditions.

3.3 Estimation of Design Flood Discharge

3.3.1 Concept of Design Flood

The discharge adopted for the design of a hydraulic structure is referred as design flood discharge or design flood. The estimation of design flood is essential for the design of all hydraulic structures such as barrages, weirs, intakes, bridges, causeways, culverts, cross drainage structures, and river training structures. The magnitude of flood a structure is required to pass without getting damaged depends on its importance. The return period (T) of the flood flow is the indicator which determines the relative importance of the various hydraulic structures.

The return period is that period of occurrence flood (Q_T) which will be equaled or exceeded once in T-year and the recurrence interval is the average number of years within which a given flood event is equaled or exceeded. For example, if the 10 year return period flood of a river is 100 m^3/s, it is assumed that 100 m^3/s will be equaled or exceeded once in 10 years. It is expressed as Q_{10} = 100 m^3/s. The criteria for the selection of the return period are based on technical and economic considerations which are summarized as follows:

- Importance of structure to be constructed,
- Effect of overtopping of the structure,
- Potential loss of life and downstream damage, and
- Cost of the structure

Reference books on hydraulic structures and manuals have suggested return periods as presented below (***Table 3-5***).

Table 3-5 Suggested Return Period

S.N	Type of structure	Return Period in years
1	Irrigation supply reliability	5
2	Road culverts	5-10 years depending upon the type of crossings
3	Highway bridge	10-50 years depending upon the type of rivers
4	Irrigation intake	10-25 years
5	Irrigation weir/barrage	50 -100 depending upon its size
6	Cross drainage structures	10-25 years

There are various methods of estimating design flood discharge, some of these are illustrated below.

3.3.2 Flood Frequency Analysis

Flood frequency analysis is a statistical method to show that flood events of certain magnitude may on average be expected once every *n* year. It is generally carried out to estimate the design flood from the recorded flow data of more than 10 years. The most commonly used methods for frequency analysis are:

- Normal distribution,

- Log normal distribution, and
- Gumbel's distribution

i. Normal Distribution: The general quantile estimation equation is as follows:

$$X_T = x_{av} + K_T \times \sigma_x$$

Where, K_T is the frequency factor, obtained from standard table corresponding to probability of execeedence, p (= 1/T) and Cs;

x_{av} is the mean of the original time series,

σ_x is the standard deviation of the original time series;

Cs is the coefficient of Skewness and in Normal distribution Cs equals to zero

ii. Log Normal Distribution: The general quantile estimation equation is as follows:

$$X_T = e(Y_{av} + K_T \times \sigma_y)$$

Where, K_T is the frequency factor, obtained from standard table corresponding to probability of execeedence, p (= 1/T) and coefficient of Skewness, Cs

Yav is the mean of the log-transformed time series,

σ_y is the standard deviation of the log-transformed time series,

C_{sy} is the coefficient of Skewness and in Log Normal distribution Cs equal to zero

iii. Gumble's Distribution or Extreme Value Type-I Distribution (EVI): The general quantile estimation equation is as follows:

$$X_T = u + \alpha Y_T$$

Where, $u = \mu - 0.5772 \times \alpha$

μ = xav is the mean of original time series,

$$\alpha = \left(\sigma_x \times \sqrt{6}\right) / \pi$$

σ_x is the standard deviation of original time series,

Y_T = (-ln(-ln(1-1/T))) and (1-1/T) is the probability of non exceedence and

T is the return period

The details of these methods are available in the Handbook of Applied Hydrology by Vent Te Chow, 1964.

3.3.3 Design Example of Flood Frequency Analysis

Calculate the design flood discharge of Thimpu River for a 50 year return period from the following data (*Table 3-6*).

Table 3-6 Annual Maximum Flow of Thimpu Chhu (m3/s)

S.N	Year	Max flow (m^3/s)	Date
1	1991	143.40	17-Jul
2	1992	82.00	27-Aug
3	1993	106.70	24-Aug
4	1994	62.16	31-Aug

S.N	Year	Max flow (m^3/s)	Date
5	1995	93.04	15-Aug
6	1996	83.81	30-Aug
7	1997	105.80	13-Aug
8	1998	129.94	8-Jul
9	1999	118.07	26-Aug
10	2000	111.55	1-Sep
11	2001	137.00	19-Aug
12	2002	132.64	23-Aug
13	2003	104.54	1-Jul
14	2004	100.03	21-Jul
15	2005	78.88	15-Aug
16	2006	84.32	26-Sep
17	2007	155.09	7-Sep
18	2008	86.68	20-Jul

i. Normal distribution

Mean of the sample size (x_{av}) = 106.42 m^3/s
Standard deviation of sample size (Ϭx) = 25.44
Reduced variate is provided based on return period.

Return period (years)	2	5	10	20	50	100
Reduced variate (K_T)	0.00	0.84	1.28	1.64	2.05	2.33

$$X_T = x_{av} + K_T \times \sigma_x$$

For 50 years return period flood (Q_{50}) = 106.42 + 2.05 * 25.44 = 158.57 m^3/s

ii. Gumble distribution (type-I)

Mean of the sample size (x_{av}) = 106.42 m^3/s
Standard deviation of sample size (Ϭx) = 25.44
Scale factor (α) = Ϭx * 0.78 = 25.44 * 0.78 = 19.84
Location factor (u) = Xav – α * 0.5772 = 106.42 – 19.84 * 0.5772 = 94.97
Reduced variate is provided based on return period.

Return period (years)	2	5	10	20	50	100
Reduced variate (Y_T)	0.37	1.50	2.25	2.97	3.90	4.60

$$X_T = u + \alpha Y_T$$

For 50 years return period flood (Q_{50}) = 94.97 + 19.84 * 3.90 = 172.36 m^3/s

In summary the calculated design floods are:
Normal distribution, Q_{50} = 158.57 m^3/s
Gumbel distribution, Q_{50} = 172.36 m^3/s

3.3.4 Rational Method

The Rational Method is a widely used method for design flood estimation of small catchments. This method considers the entire catchment area as a single unit assuming uniform rainfall distribution. This method is suitable for small catchments with an area of less than 25 km^2. The rational method formula is as follows:

$$Q = 0.278\,C\,I\,A$$

Where, Q is the peak discharge in m^3/s,

C is the runoff coefficient (roughly defined as ratio of runoff to rainfall),

I is the rainfall intensity in mm/h,

A is the catchment area in km^2

The Rational formula adapts following assumptions:

- The predicted peak discharge has same probability of occurrence as that of rainfall intensity,
- The run-off coefficient is constant during rain storms, and
- The recession time is equal to the time of rise

The maximum run-off rate in the catchment is reached when all parts of the watershed contribute to the outflow. This happens when the time of concentration (Tc) is reached, which is regarded as the time taken for the run-off at the most remote part of the catchment to reach the point of outflow. There are many methods available to calculate the time of concentration. Each method has its own parameters to consider. The most widely used method for the calculation of the time of concentration is the Ramser-Kirpich method, which is expressed as follows:

$$T_c = 0.0019 \times L^{0.77} \times S^{-0.385}$$

Where, L is the length of the main stream in m and S is the weighted slope of the main stream.

Rainfall intensity (I) is computed from the rainfall intensity-duration curves of a known location based on the time of concentration (Tc) of the stream in hours. The average rainfall intensity (I) has duration equal to the critical storm duration, normally taken as a time of concentration (Tc). The Feeder Road Manual, Volume I, Hydrology has prepared rainfall intensity curves for 10 selected stations of Bhutan and these curves can be used to determine the rainfall intensity of a particular area for the design return period. For natural catchments, the values of C vary from 0.2 to 0.4.

3.3.5 Empirical Methods

Empirical relations are based on the statistical correlation between the observed peak flow and the catchment area in a given region and are therefore region specific. The method used in central and northern parts of India is the Modified Dickens Method.

(i) Modified Dickens Method

$$Q_f = CA^{2/3}$$

$$C = 2.342\log(0.6T)\log(1185/P) + 4$$
$$P = 100(A_S + 6)/A$$

Where,

A is the catchment area in km^2,

A_s is the snow covered area within the catchment area in km^2,

T is the return period in years

The most widely used empirical methods for calculating flood flow in Nepal are the WECS and DHM Method and the Sharma and Adhikari Method. The Sharma and Adhikari Method is the updated version of the WECS and DHM Method. These methods are based on regression analysis of the available data.

(ii) WECS and DHM Method

Water and Energy Commission Secretariat (WECS) of Nepal developed the method in 1990 based on the regression analysis of available Department of Hydrology and Meteorology (DHM) data of Nepal. The equations used for the calculation of 2-year and 100-year return period floods are:

$$Q_2 = 1.8767(A_{3000})^{0.8783}$$
$$Q_{100} = 14.639(A_{3000})^{0.7342}$$

Where, Q_2 is 2-year return period flood,

Q_{100} is 100-year return period flood,

A_{3000} is catchment area below 3000m altitude,

Based on the algebraic evaluations used for lognormal distribution, the following relationship can be used to estimate floods at other return periods.

$$Q_f = \exp(\ln Q_2 + S\sigma_l)$$

Where,

$$\sigma_l = \ln(Q_{100}/Q_2)/2.326$$

S is the standard normal variate (***Table 3-7***).

Table 3-7 Standard Normal Variate

Return Period (T) in years	Standard Normal Variate (S)
2	0
5	0.842
10	1.282
20	1.645
50	2.054
100	2.326

Hence, the 50-year return period flood (Q_{50}) is estimated as follows:

$$Q_{50} = \exp[\ln Q_2 + 2.054(\ln(Q_{100}/Q_2)/2.326)]$$

(iii) Sharma and Adhikari Method

$$Q_2 = 2.29\left(A_{3000}\right)^{0.86}$$

$$Q_{100} = 20.7\left(A_{3000}\right)^{0.72}$$

Where,

Q_2 is 2-year flood

Q_{100} is 100-year flood

A_{3000} is the catchment area below 3000 m altitude

For estimating the floods of other return periods, the relationships shown in the WECS and DHM method can be used. Hence, the 50-year return period flood (Q_{50}) is estimated as follows:

$$Q_{50} = \exp\left[\ln Q_2 + 2.054\left(\ln(Q_{100}/Q_2)/2.326\right)\right]$$

(iv) SCS Method

The U.S. Soil Conservation Service (SCS) method is applicable in small catchments and is widely used in irrigation projects for catchments below 100 km^2. The runoff from natural catchments may be estimated using the following formula:

$$Q = \frac{\left(P - I_a\right)^2}{P + 4I_a}$$

Where, Q is the runoff in mm,

P is the design rainfall in mm, and

Ia is the initial loss due to infiltration, interception and surface storage

The initial loss (Ia) is defined by a curve number (CN) which is equal to:

Ia = 5.08 (1000/CN-10) mm

CN represents the hydrological soil group, the type of land use and the antecedent moisture condition. A range of CN values appropriate to normal antecedent moisture conditions and applicable Himalayan conditions is presented in ***Table 3-8***. A high value of CN reflects low initial losses and correspondingly high runoff, whereas a low value indicates diminished runoff.

The surface runoff categories can be described as:

Rapid - Early growth stage of crops

- Poor plant density
- Poor ground cover
- Over grazed areas

Slow - Later growth stage of crops

- Dense plant intensity

- Good ground cover
- Lush grazing areas

The mean slope may be calculated from

$$Slope = \frac{\delta E}{L} \times 100$$

Where, δE is the difference in elevation (in m) between the highest point on the watershed nearest to the stream head and its outfall point and L is the length of the main stream. The slope categories are given in Table 3-8.

Table 3-8 Hydrological Soil Groups and CN Values

<table>
<tr><th colspan="3">Type of Soil</th><th colspan="2">Infiltration Rate</th><th>Class</th></tr>
<tr><td colspan="3">Deep Sandy</td><td colspan="2">High</td><td>A</td></tr>
<tr><td colspan="3">Shallow Sandy and Medium Textured</td><td colspan="2">Moderate</td><td>B</td></tr>
<tr><td colspan="3">Shallow with Medium/Heavy Texture</td><td colspan="2">Slow</td><td>C</td></tr>
<tr><td colspan="3">Clay and Shallow/ hard pans</td><td colspan="2">Very Slow</td><td>D</td></tr>
<tr><td rowspan="2">Land use or Cover</td><td rowspan="2">Surface Runoff</td><td colspan="4">Hydrological Soil Class</td></tr>
<tr><td>A</td><td>B</td><td>C</td><td>D</td></tr>
<tr><td rowspan="3">Fallow
Row Crops e.g. Maize</td><td>Rapid</td><td>77</td><td>86</td><td>91</td><td>94</td></tr>
<tr><td>Moderate</td><td>72</td><td>81</td><td>88</td><td>91</td></tr>
<tr><td>Slow</td><td>67</td><td>78</td><td>85</td><td>89</td></tr>
<tr><td rowspan="2">Broadcast Crops e.g. Upland Rice</td><td>Rapid</td><td>65</td><td>76</td><td>84</td><td>88</td></tr>
<tr><td>Slow</td><td>63</td><td>75</td><td>83</td><td>87</td></tr>
<tr><td rowspan="3">Pasture or range</td><td>Rapid</td><td>68</td><td>79</td><td>86</td><td>89</td></tr>
<tr><td>Moderate</td><td>49</td><td>69</td><td>79</td><td>84</td></tr>
<tr><td>Slow</td><td>39</td><td>61</td><td>74</td><td>80</td></tr>
<tr><td rowspan="2">Woods</td><td>Moderate</td><td>36</td><td>60</td><td>73</td><td>79</td></tr>
<tr><td>Slow</td><td>25</td><td>55</td><td>70</td><td>77</td></tr>
</table>

Table 3-9 Slope Categories

Category	Range of Slope
Flat	0 - 5
Moderate	6 - 10
Steep	> 10

To choose the design rainfall a range of growth factors corresponding to various return periods is presented in ***Table 3-10*** .

Table 3-10 Growth factors for maximum rainfall

Return Period (Years)	Growth Factor (G)
5	1.3
10	1.5
25	1.8
50	2.0
100	2.3
150	2.4
200	2.5

Note: $G = 0.32 \ln T + 0.78$

Where, G is the growth factor for Gumbel distribution,

T is the return period and

ln is the natural logarithm

The growth factors are used to adjust the mean annual maximum 24 hour rainfall values. As the rainfall data represents a single location, they must be reduced according to the areal reduction factor, which depends on the size of the catchment area in question. Factors for a range of catchment areas are given in Table 3-11 .

Table 3-11 Areal Reduction Factor for 24 Hours Point Rainfall

Catchment Area km^2	Areal Reduction Factor
10	1.00
50	0.99
100	0.98
200	0.96
500	0.91
1000	0.86
5000	0.76

The steps to determine the flood flow using the SCS method is summarized as follows:

Step 1: Choose a curve number (CN) from *Table 3-8*, taking into account hydrological soil class, cover and runoff category.

Step 2: Find the catchment area (*A*) in km^2, main stream length (*L*) and the slope in percentage (δE/L x 100).

Step 3: Choose a slope category from Table

Step 4 : Choose a design 24-hour rainfall adjusted according to ***Table 3-10*** to give the required return period and adjust according to ***Table 3-11*** for the catchment area.

Curve numbers (CN) of 60, 70 and 80 have been chosen and for each there are three slope categories: flat, moderate and steep.

Once CN value and slope category have been chosen, the remaining task is to locate the intersection of the catchment area ordinate with the design rainfall curve in order to read off the design peak flood.

3.3.6 Flood Estimation from Trash Marks

The Trash mark method is one of the conventional methods of estimating flood flow which is used when other data and information are unavailable. This method is based on the slope area survey of the particular river stretch where past flood levels can be recorded with the help of old inhabitants and/or local informants. It may be possible to find debris indicating a past flood level or to have a level on a tree or building pointed out. Trash mark levels are more accurate than remembered levels. A river cross sectional survey is needed at two or three sites where flood levels can be obtained and the hydraulic calculation is carried out using Manning's formula:

$$Q = \frac{A \times R^{2/3} \times S^{1/2}}{n}$$

Where, A is the area of river cross section up to trash level in m^2,

R is the hydraulic radius = A/P

S is the water surface slope

n is the Manning's roughness coefficient and is given in a standard table (Ven Te Chow, 1959)

P is the wetted meter in m,

The water surface slope is that obtained from the trash marks at three different sites. The flood flow can be estimated for two or three sites and then averaged.

3.3.7 Assessment of Flood Flow Considering Climate Change

Climate change may increase the intensity of extreme rainfall and flow rates. Irrigation structures are designed for a certain level of exceedence of rainfall, flow rates and water levels. These all are based not only on the accurate assessment of the data quality but also adequate knowledge of the consequences of an exceedence. Hence, adequate data and knowledge is essential during the design of hydraulic structures such as intakes and cross drainage works. One suggested approach is to select a higher return period for design flood flow assessment, which is not always economical. In addition, the computed design flood discharge can be increased by adding 10 to 20% in order to assess design flood with climate change considerations.

3.4 Water Requirement Assessment

3.4.1 Crop Water Requirements

The crop water requirement is defined as the quantity of water utilized by the plant during its life time. It is estimated with the evapotranspiration (ETo) of a reference crop. The steps used to calculate the crop water requirement are as follows:

i. Collect climatic data from nearby meteorological station,
ii. Decide cropping pattern,
iii. Calculate reference crop evapotranspiration (ETo) (from FAO methods and tables),
iv. Use crop coefficients (from FAO),
v. Calculate evapotranspiration of crops ETcrop (mm/day)
vi. Allow for land preparation loss (rice and wheat only) (Lp),
vii. Allow for deep percolation (rice only)(dp),
viii. Calculate total crop water requirements (CWR),
ix. Calculate effective rainfall (Pe),
x. Calculate net crop water requirements (net CWR),

3.4.2 Climatic data

In order to calculate the reference crop evapotranspiration (ETo), climatic data shall be collected from the nearest and most representative meteorological stations. The required climatic data are long term monthly minimum and maximum air temperatures, monthly rainfall, relative humidity or vapour pressure, wind speed and sunshine duration. In addition, locational information such as latitude, longitude and altitude are necessary.

3.4.3 Cropping Pattern

The proposed cropping pattern for irrigation schemes should be based on agro-climatic conditions and farmers' preferences. The possible cropping patterns practiced in Bhutan are presented in the table below (***Table 3-12***).

Table 3-12 Possible Cropping Patterns of Bhutan

S.	Agro-ecological Zones	Dates		
N	**Crops**	**Seedbed**	**Transplant/Plant**	**Harvest**
1	**Cool temperate (2600 m to 3600 m amsl)**			
i	Potato		March	August
ii	Maize		March	October
iii	Wheat		November	June
iv	Buckwheat		September	June
2	**Warm temperate (1800 m to 2600 m amsl)**			
i	Paddy	April/May	May/June	October
	Fallow			
ii	Paddy	April/May	May/June	October
	Legumes/Oilseeds		December	April
iii	Paddy	April/May	May/June	October
	Wheat		Nov/December	May

S.	Agro-ecological Zones	Dates		
N	Crops	Seedbed	Transplant/Plant	Harvest
iv	Potato		December	April
	Vegetables		April	August
V	Orchard	Round the year		
3	**Dry Sub-tropical (1200 m to 1800 m amsl)**			
i	Paddy	April/May	May/June	October
	Fallow			
ii	Paddy	April/May	May/June	October
	Wheat		Nov/December	May
iii	Paddy	April/May	May/June	October
	Vegetables/Legumes		November	February/March
iv	Paddy	April/May	May/June	October
	Potato		November	March/April
v	Orchard		Round the year	
4	**Humid sub-tropical (600 m to 1200 m amsl)**			
i	Paddy	April/May	May/June	October
	Fallow			
ii	Paddy	April/May	May/June	October
	Wheat		December	April/May
iii	Paddy	April/May	May/June	October
	Potato		November	March
iv	Paddy	April/May	May/June	October
	Vegetables/Legumes		January	April
v	Spring Maize		February	June
	Paddy	May/June	June	October
vi	Orchard		Round the year	
5	**Wet sub-tropical (150 m to 600 m amsl)**			
i	Paddy	April/May	May/June	October
	Fallow			
ii	Paddy	April/May	May/June	October
	Paddy	January	February	May/June
iii	Paddy	April/May	May/June	October
	Wheat		Nov/December	April
iv	Spring maize		February	June
	Paddy	April/May	May/June	October
v	Paddy	April/May	May/June	October
	Potato		December	April
vi	Paddy	April/May	May/June	October
	Vegetables/Legume		November	February/March

Source: Derived from DOA information, 2015

3.4.4 Reference Crop Evapotranspiration (ETo)

The Reference Evapotranspiration (ETo) represents the potential evaporation of a well-watered grass crop known as the reference crop. Depending upon the availability of climatic data, there are four methods for calculating ETo of which the most widely used method is the Penman-Monteith method. The FAO has developed software to calculate ETo, the latest version is Cropwat-8. This software is user- friendly and easily available on FAO websites. A worked out example of Crowat-8 is presented here (***Table 3-13***).

Table 3-13 Worked out example of Cropwat-8

Monthly ETo Penman-Monteith - C:\ProgramData\CROPWAT\data\climate...

Country: Bhutan Station: Bhur

Altitude: 375 m. Latitude: 26.90 °N Longitude: 90.43 °E

Month	Min Temp	Max Temp	Humidity	Wind	Sun	Rad	ETo
	°C	°C	%	m/s	hours	MJ/m²/day	mm/day
January	12.5	21.6	84	0.6	7.8	14.4	1.94
February	15.3	23.4	77	1.1	8.2	16.9	2.69
March	17.7	25.6	66	1.3	8.5	19.9	3.75
April	19.3	26.4	75	1.7	8.2	21.4	4.18
May	21.6	28.1	100	1.6	6.0	18.9	3.44
June	23.0	28.3	99	1.3	6.0	19.1	3.65
July	23.3	28.4	98	1.2	5.8	18.7	3.63
August	23.7	29.2	97	1.1	5.6	17.8	3.53
September	22.9	29.1	100	1.1	6.0	16.9	3.28
October	20.9	27.9	99	1.1	6.9	15.9	2.89
November	17.8	25.6	93	0.8	7.9	14.9	2.43
December	14.4	23.0	86	0.8	8.3	14.2	2.04
Average	**19.4**	**26.4**	**90**	**1.1**	**7.1**	**17.4**	**3.12**

3.4.5 Crop Coefficient (Kc)

The crop coefficient (Kc) is the experimentally determined ratio of crop evapotranspiration to reference crop evapotranspiration (ETc/ETo). The crop coefficient considers the leaf anatomy, stomata characteristics, aerodynamic properties, and albedo of the crop under the same climatic conditions. In addition, the crop coefficient factor (Kc) serves as an aggregation of the physical and physiological differences between crops and the reference definition (FAO, 1998). The factors that determine the Kc are crop type, climate, soil evaporation, and crop growth stages. The Kc values vary with crop growing stages, which are:

- Initial stage,
- Crop development stage,
- Mid-season stage, and
- Late season stage

Crop coefficients (Kc) of various crops proposed in this manual are derived from the FAO Irrigation and Drainage Manual no 56 and are presented below (***Table 3-14***).

Table 3-14 Crop Coefficients of Selected Crops

Crops	Initial stage	Crop development stage	Mid-season stage	Late season stage
Maize	0.45	0.80	1.05	0.80
Rice	1.10	1.10	1.05	0.95
Wheat	0.43	1.05	1.15	0.40
Potato	0.42	0.80	1.15	0.77
Mustard	0.45	0.85	1.00	0.72
Legumes	0.40	0.95	1.05	0.95
Vegetable (winter)	0.28	0.54	0.95	0.89
Vegetable (summer)	0.34	0.93	1.05	0.91
Orange	0.85	0.85	0.85	0.85
Apple	0.80	1.00	1.10	0.80

Source: Derived from FAO, 1998

3.4.6 Crop Evapotranspiration (ETcrop)

Crop evapotranspiration is calculated by multiplying the reference crop evapotranspiration (ETo) by the crop coefficients (Kc) of the particular crop. All of the calculations are tabulated on Excel sheets on 15 days basis of the year.

$$ETcrop = ETo \times Kc$$

Where ET_{crop} is the crop evapotranspiration in mm/day,

ETo is the reference crop evapo-transpiration in mm/day, and

Kc is the crop coefficients of the particular crop for given a stage of crop development

3.4.7 Land Preparation Requirement

Since land preparation on paddy fields is done during the dry period, some water is required for that work. For non-paddy crops, much lower land preparation requirements are assumed, and can be met by soil moisture storage, except in the case of wheat, for which 60 mm is applied to improve germination. Land preparation requirements for paddy over a 15-day period are presented below (***Table 3-15***).

Table 3-15 Land Preparation Requirements

15-day periods	1^{st}	2^{nd}	3^{rd}	4^{th}
Pre-monsoon paddy	75	75	50	50
Follow paddy	55	50	50	-

15-day periods	1^{st}	2^{nd}	3^{rd}	4^{th}
Monsoon paddy	55	55	50	50
Wheat	60	-	-	-

3.4.8 Deep Percolation Losses

Deep percolation losses are considered only for paddy rice and depend on soil types (***Table 3-16***). In addition, evaporation losses from the paddy field are also considered based on pan evaporation data of each location.

Table 3-16 Estimated Deep Percolation Losses (mm/day)

Soil Texture	Newly Irrigated	Long term Irrigated
Sand, loamy sand	> 20	>20
Sandy loam	20	10
Very fine sandy loam, loam silty loam, sandy clay loam	20	5
Silty clay loam, clay loam silty clay, clay	5	2

3.4.9 Total Crop Water Requirements (mm/15 day)

The Total Crop Water Requirement (CWR) is the sum of crop evapotranspiration (*ETcrop)*, water requirement for land preparation (Lp), losses due to percolation (D*p*) and loses due to evaporation (Ep).

$CWR = ET_{crop} + L_p + D_p + Ep$

3.4.10 Reliable Rainfall and Effective Rainfall

Rainfall contributes to a greater extent in satisfying crop water requirement depending upon the location. How much water will come from rainfall and how much will be covered by irrigation is unfortunately difficult to predict as rainfall varies from season to season and year to year. In addition, not all of the rain that falls on the field is used by the plant. The amount of rainfall used effectively for plant growth is termed effective rainfall, and is calculated from the 80% reliable rainfall of the area.

80% Reliable Rainfall: The amount of rainfall which can be depended on in 1 out of 5 years, corresponding to the 80% probability of exceedence, is termed **Reliable Rainfall**. The FAO suggests using 80% reliable rainfall for irrigation system design. Rainfalls with 20%, 50% and 80% probability of exceedence are defined as wet year, normal year and dry year rainfall respectively. To compute dry year rainfall (80% reliable) from the time series meteorological data, the following procedures have been adopted:

i. Tabulate yearly rainfall totals of the given period,
ii. Arrange data in descending order of magnitude,
iii. Tabulate plotting positions according to the following:

$$Fa = \frac{100 \times m}{N + 1}$$

Where, Fa is the plotting position,

m is the rank number,

N is number of records

iv. Plot values in log-normal scale and obtain logarithmic regression equation (Figure 3-1),

v. Calculate year values of 80% probability (P80),

vi. Determine monthly values of dry year according to the following relationship:

$$P_{idry} = P_{iav} \times \frac{P_{dry}}{Pav}$$

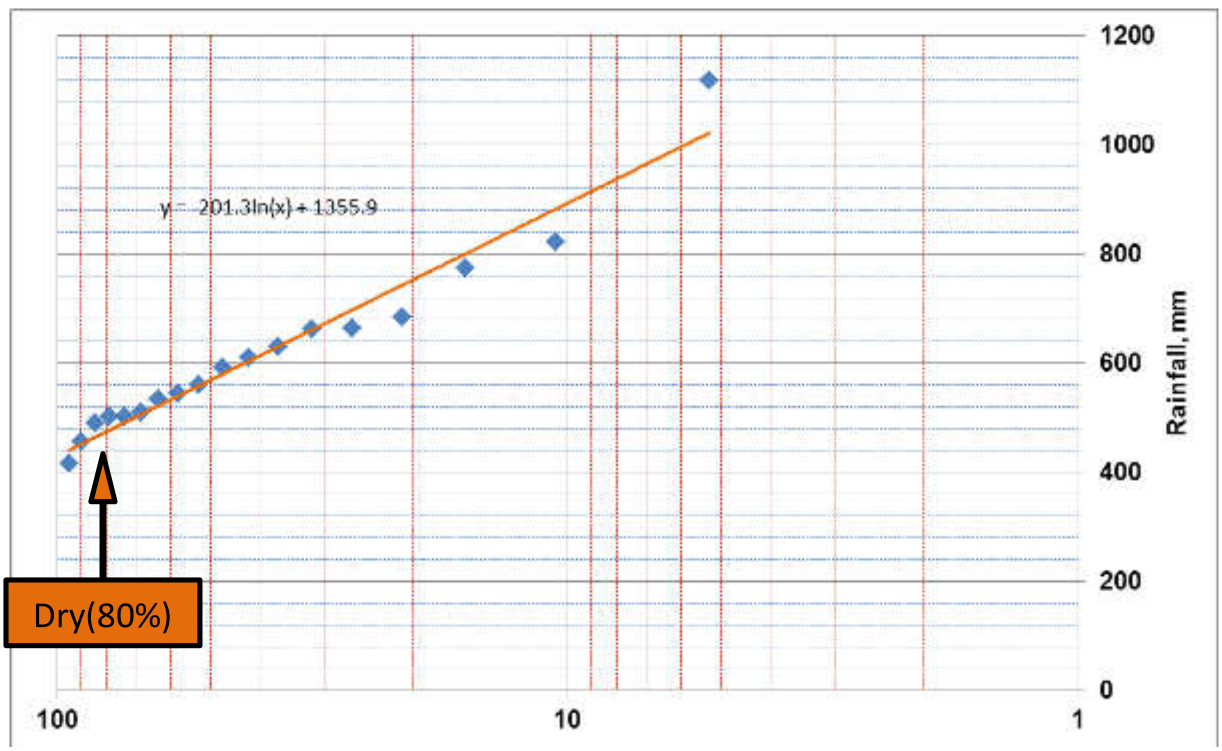
Where, P_{idry} is the monthly rainfall dry year for month i,

P_{iav} is the average monthly rainfall for month i,

P_{dry} is the yearly rainfall at 80% of probability of exceedence

P_{av} is the average yearly rainfall,

Figure 3-1 Worked out Example of Reliable Rainfall



The part of the rainfall which is effectively used by the crop after rainfall losses due to surface runoff and deep percolation is termed as **Effective Rainfall** (P_{eff}). The FAO developed software, Cropwat-8, has different options for computing effective rainfall. The most widely used method is the USDA Soil Conservation Service method. A worked out example of 80% reliable rainfall and effective rainfall of the Paro DCS station is presented here (*Table* ***3-17***).

Table 3-17 Worked out example of reliable and effective rainfall (mm)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average Rainfall	7.4	15.1	20.0	32.1	55.6	83.3	136.9	118.6	89.1	52.2	1.7	2.8
P80 Reliable	5.7	11.6	15.4	24.7	42.8	64.2	105.5	91.4	68.7	40.2	1.3	2.2
Effective Rainfall	5.6	11.4	15.1	23.7	39.9	57.6	87.7	78.0	61.1	37.7	1.3	2.2

3.4.11 Net Crop Water Requirements (mm/15 day)

The net crop water requirement (*Net CWR*) is calculated as the difference between total crop water requirements and effective rainfall (*Pe*).

Net CWR = CWR – Pe

Where, Net CWR is the net crop water requirement in mm/15 day and Pe is the effective rainfall in mm.

Hence, the net crop water requirement for paddy crops is crop evapotranspiration plus land preparation requirements (including evaporation from land preparation) plus deep percolation minus effective rainfall. The net crop water requirement for dry root crops is the crop evapotranspiration plus, in the case of wheat, land preparation requirements minus effective rainfall.

3.4.12 Canal System Efficiencies

During the irrigation process, considerable water loss occurs through evaporation, seepage and deep percolation. The amount lost depends upon the efficiency of the system. In order to calculate the intake water requirements, field irrigation losses, distribution canal losses, and conveyance canal losses must be taken into account.

(i) **Field efficiency (Fe)**: Irrigation field application efficiency is expressed as the ratio of water stored in the root zone to the water applied. When irrigating dry root crops, it is difficult to apply the water evenly over the whole field. To allow for this, a field efficiency of 60% is assumed. For rice, as the irrigation is based on flooding field efficiency can be assumed to be 75% to 85% to allow for limited run-off and overspill.

(ii) **Distribution efficiency (De)**: The distribution efficiency is the efficiency of the water distribution canals and conduits supplying water from the conveyance system to individual fields. Irrigation water is lost along the field channels from the off-take structures due to seepage, percolation, evapotranspiration, evaporation, and/or leakage. Distribution efficiency is considered to be 70% to 80% depending upon the type of distribution system.

(iii) **Conveyance or main canal efficiency (Ce)**: Irrigation water is also lost along delivery canals such as the main canal and the secondary/tertiary canal from the intake to the off-takes of distribution system. It is caused by seepage, percolation, evapotranspiration, evaporation, leakage and/or over diversion of irrigation water. Conveyance efficiency is considered to be 70% to 80%.

Overall irrigation system efficiency ranges from 29% for small scale hill irrigation schemes to 54% for well managed large irrigation systems.

3.4.13 Intake Water Requirement

The intake water requirement is the maximum field crop water requirement divided by the system efficiencies.

The Field Crop Water Requirement (l/s/ha) = Net CWR (mm/15day)/(Fe*129.6)

Where, 129.6 is the conversion factor to convert mm/15day into l/s/ha, since 8.64 mm/day = 1 l/s/ha and Fe is field efficiency.

Intake Water Requirement (IWR) = Field CWR/ (Cse), l/s/ha

Where, Cse is the canal system efficiency, which may be taken as 30% to 50%.

The detailed worked out example of the crop water requirement is presented hereunder (*Table **3-19***).

3.4.14 Consideration of climate change in crop water requirement

Consideration of the potential impacts of climate change on crop water requirements suggest that farm operations should be suited to the local conditions in order to reduce the risk of crop damage. This may require adjusting the timing of crop planting and harvesting dates, changing the cropping intensity, and proposing crop varieties with low water requirements and which can tolerate heat and drought. In addition, irrigation water requirements could be minimized by increasing irrigation system efficiency and using crop coefficients on the lower side of the given range. Furthermore, deficit irrigation shall also be introduced in case of acute climate change conditions.

3.4.15 Irrigation Water Duty

The duty of irrigation water is the amount of water required for irrigation of a unit of land for specific crop at a specific time. The irrigation water duty depends upon:

- Type of soil,
- Percolation rate of soil,
- Rainfall, and
- Cropping practice

In Bhutan, the peak water requirement occurs mostly during the land preparation period in June and ranges from 1.6 l/s/ha to 4.3 l/s/ha (*Table 3-18*).

Table 3-18 Irrigation water duty

Soil type		Clay	Loam	Sandy loam
Percolation rate (mm/day)		0-5	5-10	10-15
Rainfall <2,500 mm/year	Irrigation duty (l/s/ha)	2.4	3.3	4.3
Rainfall > 2,500 mm/year		1.6	1.9	2.2

Source: Irrigation Engineering Manual, 1998

Table 3-19 Worked out example of crop water requirement

Total Water Requirement Computation																									
Project:		**Irrigation Master Plan**									**Location:**		**Paro DCS**				**Cropping pattern:**			**Rice**	**Potato**				
Month		Jan		Feb		Mar		Apr		May		Jun		Jul		Aug		Sep		Oct		Nov		Dec	
Half Month		1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Cropping Pattern			**Potato**												**Rice**									**Potato**	
Rice---Potato	20%																								
ET0 mm/day		1.46	1.46	1.96	1.96	2.53	2.53	3.02	3.02	3.34	3.34	3.20	3.20	3.14	3.14	3.14	3.14	2.83	2.83	2.54	2.54	1.84	1.84	1.37	1.37
Crop coefficient (Kc)		1.01	1.13	1.13	1.08	0.94	0.77				1.10	1.10	1.10	1.10	1.05	1.05	1.05	0.95	0.95	0.95			0.42	0.55	0.79
ETcrop mm/day		1.47	1.65	2.21	2.12	2.38	1.95	0.00	0.00	0.00	3.67	3.52	3.52	3.45	3.30	3.30	3.30	2.69	2.69	2.41	0.00	0.00	0.77	0.75	1.08
ETcrop (mm/half month)		22.86	25.57	31.01	29.64	36.86	30.20	0.00	0.00	0.00	56.95	52.80	52.80	53.54	51.10	51.10	51.10	40.33	40.33	37.40	0.00	0.00	11.59	11.68	16.78
Land preparation (mm)											55.00	55.00	50.00	50.00											
Deep percolation (mm)											75.00	75.00	75.00	75.00	75.00	75.00	75.00	75.00	75.00	75.00					
Pan evaporation (mm/day)											5.00	5.00													
Pan evaporation (mm/15day)											75.00	75.00													
Total crop water requirement		22.86	25.57	31.01	29.64	36.86	30.20	0.00	0.00	0.00	261.95	257.80	177.80	178.54	126.10	126.10	126.10	115.33	115.33	112.40	0.00	0.00	11.59	11.68	16.78
(mm/half month)																									
80% Reliable Rainfall		2.57	3.32	4.03	5.09	6.49	8.18	10.16	13.41	17.93	22.44	26.97	36.99	52.49	55.10	44.81	37.52	33.22	27.75	21.08	14.10	6.82	2.73	1.83	1.77
Effective Rainfall		0.00	0.00	0.00	0.00	0.00	0.43	1.28	3.05	5.75	8.46	11.19	18.46	30.29	32.08	23.83	18.19	15.16	11.65	7.65	4.24	1.41	0.00	0.00	0.00
Net irrigation water requirement																									
mm/half month		22.86	25.57	31.01	29.64	36.86	29.77	0.00	0.00	0.00	253.48	246.61	159.34	148.25	94.03	102.28	107.92	100.17	103.68	104.75	0.00	0.00	11.59	11.68	16.78
Overall system efficiency		0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
Total water requirement																									
(mm/half month)		76.2	85.2	103.4	98.8	122.9	99.2	0.0	0.0	0.0	844.9	822.0	531.1	494.2	313.4	340.9	359.7	333.9	345.6	349.2	0.0	0.0	38.6	38.9	55.9
Water requirement of the																									
system (m3 per ha)		762	852	1034	988	1229	992	0	0	0	8449	8220	5311	4942	3134	3409	3597	3339	3456	3492	0	0	386	389	559
Water requirement (l/s/ha)		0.59	0.66	0.80	0.76	0.95	0.77	0.00	0.00	0.00	6.52	6.34	4.10	3.81	2.42	2.63	2.78	2.58	2.67	2.69	0.00	0.00	0.30	0.30	0.43
Monthly water requirement																									
(l/s/ha)		0.62		0.78		0.86		0.00		3.26		5.22		3.12		2.70		2.62		1.35		0.15		0.37	

3.5 Water Balance Assessment

3.5.1 General

Water balance is the difference of inflows and outflows at the particular point of river diversion. Inflows are rainfall and runoff while outflows are water demand for irrigation, household water supply, evaporation and evapotranspiration from vegetation. The water balance is calculated at each diversion point in order to ascertain the amount of available water that could be used in the proposed irrigation scheme without impacting the existing uses of water. The main components of the water balance are available river flow at the intake site, the water requirement for irrigation as per the proposed cropping pattern and intensity, and downstream mandatory releases for household supply and environmental flow.

3.5.2 Available Flow at the Intake Site

The available river flow at the intake site is assessed according to the methodology described in section 3.2 of this chapter (Chapter-3). From the available flow, the 80% reliable flow is assessed for diversion to the irrigation scheme under consideration. The available flow at the intake site is the balance of the available 80% reliable flow minus mandatory releases for upstream and downstream existing uses, including environmental flow.

3.5.3 Existing Downstream Requirements

Existing water uses need to be considered while designing the irrigation scheme. The users and farmers using the water for irrigation and/or drinking water in the vicinity of the proposed intake site should first agree to the development of the proposed irrigation scheme. The designer/planner needs to assess all of the withdrawals of water upstream and downstream of the proposed intake site. The existing water withdrawals can be assessed by measuring the flow at its intake site and by consulting the community. For the mandatory environmental flow, at least 10% of the non-monsoon flow shall be released from the proposed intake site.

3.5.4 Water Balance

The available water at the source stream/river should meet the requirements of all existing water uses, proposed intake water requirements, and the mandatory downstream flow. If the water balance is positive in terms of meeting these requirements, the scheme is technically feasible from a water availability point of view. If the water balance is negative, it is essential to change the cropping patterns and/or reduce the command area of the proposed scheme as it is not possible to change the existing water requirements or the mandatory downstream flow.

3.6 Discharge Measurement

3.6.1 General

The amount of water passing at a point on the stream or channel during a given time is a function of the velocity and cross-sectional area and is expressed by the Continuity equation: $Q=AV$, where Q is the stream discharge, A the cross-sectional area, and V the flow velocity. The discharge measurement concerns the measurement of the flow velocity and cross sectional area. For small irrigation schemes, three methods are appropriate for the discharge measurement: float method, bucket and watch method, and current meter method.

3.6.2 Float method

The float method measures the surface velocity of flowing water, and the mean velocity is obtained using a correction factor. The basic idea is to measure the time taken to float an object over a specified distance.

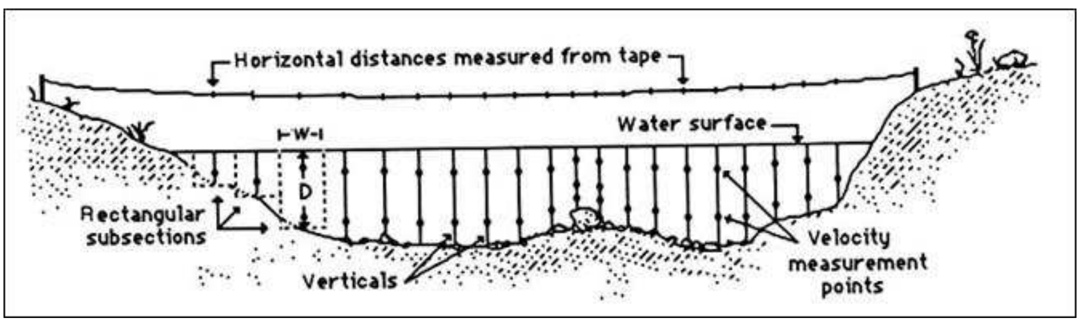
$V_{surface}$ = travel distance/ travel time = L/t

Because surface velocities are typically higher than mean or average velocities, $V_{mean} = k\ V_{surface}$ where k is a coefficient that generally ranges from 0.65 for rough beds to 0.75 for smooth beds. The procedures for discharge measurement are as follows:

- Step 1 Choose a suitable straight reach with minimum turbulence (minimum distance should be 20 m);
- Step 2 Mark the start and end points of the chosen reach;
- Step 3 If possible, travel time should exceed 20 seconds but be at least 10 seconds;
- Step 4 Drop your object into the stream upstream of the upstream marker;
- Step 5 Start the watch when the object crosses the upstream marker and stop the watch when it crosses the downstream marker;
- Step 6 Repeat the measurement at least 3 times and use the average in further calculations;
- Step 7 If you only do the float method, you need to measure the cross-sectional areas at the start and end point of your reach;
- Step 8 Average cross-sectional areas;
- Step 9 Using the average area and corrected velocity, you can now compute discharge

$$Q = AV_{mean}$$

Figure 3-2 Discharge measurement with float area method



3.6.3 Bucket and Watch method

The bucket and watch method is a simple volumetric measurement of water at the given time period. This method is applicable for spring sources with a discharge less than 1 lps. The volume of water can be measured by diverting the entire flow into a measuring vessel of known capacity (for example, a plastic bucket). The time taken to fill the vessel is noted. At least three to five readings are necessary for the measurement of the water and average value is computed. The discharge calculation may be carried out using the following format (***Table 3-20***).

Table 3-20 Discharge Measurement with Bucket-Watch Method

Project name:					Location:		
Source name:					Date of measurement:		
Description		Measurement number					
A	Vessel capacity (liter)	1	2	3	4	5
B	Time to fill the vessel (sec)					
C	Discharge (l/s) = A/B					
D	Average discharge Q (l/s)	Q = (C1 +C2 + C3 +C4 +C5)/5				

3.6.4 Current Meter

A current meter is a device used to measure the velocity of flowing water. It has a propeller mounted on a shaft which allows the device to move freely regardless of the depth of the water . The speed of rotation is the function of the velocity of water. The manufacturer of the current meter provides the correlation between the number of rotations and water velocity. The measurement of velocity is more accurate and reliable and the current meter is extensively used in streams with uniform cross sections. This method is useful to measure velocities in the range of 0.2 m/s to 5 m/s. The procedure to measure the velocity using a current meter is as follows:

- Choose a suitable site for the measurement;
- Secure a measuring tape across the section;
- Measure the chainage at both banks and calculate the stream width;
- Divide the stream width into sections;
- If there is staff gauge, take a gauge reading at the start of the flow measurement. Check at the end that the measurement is the same;
- Measure the flow in the center of each section:
    - Measure the water depth,
    - If the depth is less than or equal to 0.3 m measure the velocity at 0.6 d,
    - If the depth of water is more than 0.3 m measure the velocity at 0.2 d and 0.8 d,
- Calculate the velocity from the machine manufacturers table;
- Calculate the flow area of each section;
- Calculate the discharge in each section and sum discharges to give the totals,
- The total stream discharge $Q = a_1v_1 + a_2v_2 + a_3v_3 + \ldots\ldots + a_nv_n$

CHAPTER-4

4. Intakes and Headworks

4.1 Introduction

Intake structures are hydraulic devices built at the head of the irrigation canals. The irrigation canals in the context of the small scale irrigation include only main canals. The intakes for medium and large irrigation schemes are different and beyond the scope of this manual. The main purpose of an intake is to allow the abstraction of water from the source with a minimum sediment entry. There are a number of different types of intakes; the selection of the most appropriate type depends on the location, scale and function of the irrigation scheme. Intakes can be from rivers, reservoirs, and lakes.

Any hydraulic structure which supplies water to the off-taking canal is called a headwork. A headwork may be a storage headwork or a diversion headwork. A storage headwork comprises the construction of a dam on the river, and stores water when there is excess supply and releases water according to irrigation demands. A diversion headwork serves to divert the required amount of water to the canal from the river. This manual covers the design of side intakes and diversion headworks.

4.2 Types of Intakes

Types of intake structures are mainly distinguished by the method used to divert water from the source river:

- Bank intakes,
- Side intakes with cross weirs,
- Canal diversion works,
- Bottom intakes,
- Frontal intakes,
- Submerged intakes, and
- Tower intakes

4.2.1 Bank Intake

Bank intakes are located on the banks of the river and their faces are aligned with the banks. Bank intakes are appropriate for irrigation where water fluctuation in the river is low and a small portion of the river flow is abstracted to the canal. The inflow into the intake structure depends upon the water level in the river. In existing community managed schemes, temporary arrangements are used to raise the water level. The intakes, which are at the river bed level, have coarse screens, control gates and settling chambers to trap coarse material. Bank intakes are referred as side intakes or lateral intakes.

4.2.2 Side intake with cross weir

Side intakes with cross weir are essential for rivers and streams where a substantial portion of the flow is to be diverted. The weir is situated in the river which dams up the water level to ensure a constant minimum depth of water upstream of the weir and allows water to be diverted to the canal system. Based on the material used for their construction, weirs are masonry weirs, concrete weirs, or wooden weirs.

4.2.3 Canal diversion works

Canal diversion works are weirs or barrages constructed across the rivers to maintain adequate levels to abstract water towards the main canal. Barrages are constructed across the main river at the headwork of the new canals. Barrages and associated headworks consist of an under sluice for sluicing sediments and a head regulator to control the flow of water to the main canal.

4.2.4 Bottom intake

Bottom intakes are located on the river bed and the water to be diverted is taken through a collection chamber built in the river bottom covered with trash screens. The bars of the screen are laid in the direction of river flow and inclined in a downstream direction. The coarse sediment is kept out of the collection chamber and transported further downstream. Particles which are smaller than the openings of the screen bars are introduced into the collection chamber together with the water, and are excluded from the settling basin through a flushing device constructed a short distance downstream. The bottom intake can be constructed at the same level as the river bed or in the form of a sill.

4.2.5 Frontal intake

Frontal intakes are designed to abstract clear water from mountain rivers. These intakes are successfully utilized in Turkey. The intake has two layers: the upper layer draws clear water for irrigation and the lower layer continuously flushes the sediment in front of the intake.

4.2.6 Submerged intake

A submerged intake is located at the bottom of the river or reservoir. It comprises a bell mouth and bend set in a concrete block with a concrete draw-off pipe. The inlet is protected from the bar screen and set high enough above the bed to allow sediments to pass on either side of the inlet. This type of intake is often used in drainage outlet of small reservoir.

4.2.7 Tower intake

Tower intakes are free standing structures, set in deep water connected by an access bridge to the bank. Tower intakes are used in outlet works of the reservoirs with the arrangements of gates or valves.

Irrespective of whether the intake structures are selected with or without damming the river, the structures should be designed so that:

- At times of lowest discharge, the required amount of water can always be diverted,
- All floods can be evacuated without damage of the structures, and
- The amount of water flowing into the canal is limited to the amount of water to be diverted.

The selection of the type of intakes depends mainly of the objective of the project, specific site conditions, and the requirements of the irrigation scheme. The general criteria that affect the selection of the intake/headwork are:

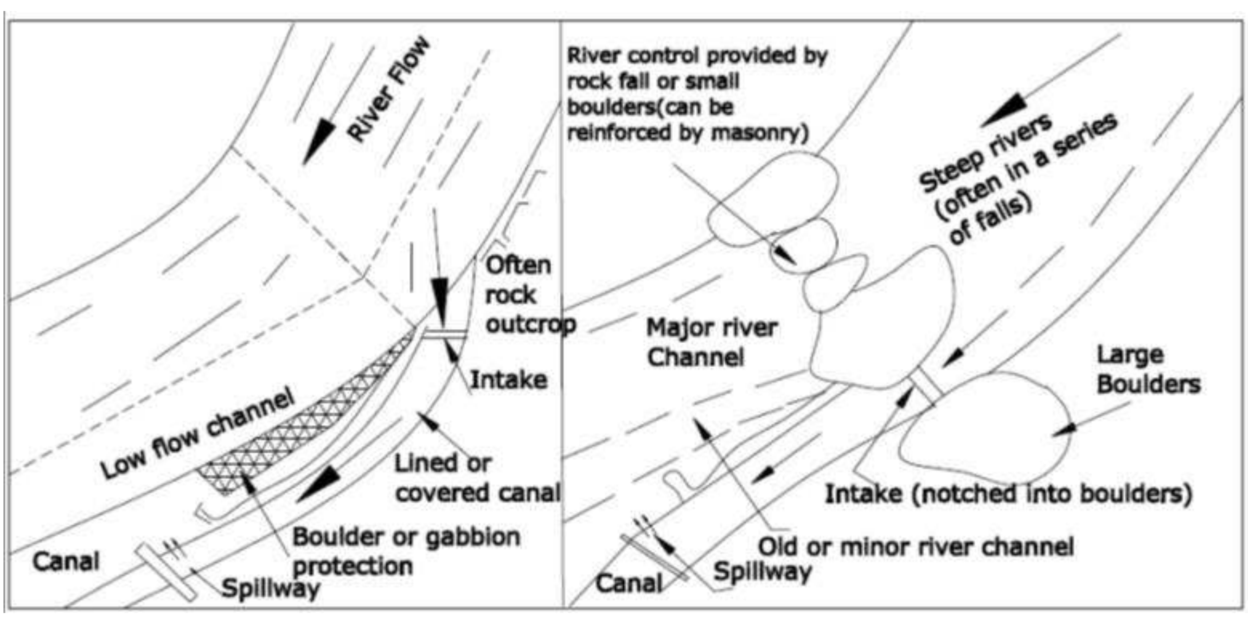
- Effectiveness in excluding sediment entry into the canal,
- Ease and relative cost of structure,
- Effect of floods in terms of damage risk,
- Effect of floods on sediment entry, and
- Flow control to the canal system

4.3 Selection of Site for Intakes and headworks

The site for the headwork or intake should be selected to ensure that the required flow can be diverted to the canal at all stages of the river. The main considerations in the selection of the headwork or intake site are as follows:

- Channel dimensions- narrow floodway with a relatively straight approach reach, enabling the river to develop linear flow,
- Thalweg deflection- the main channel with low flow is referred as the thalweg. The stability of the thalweg is of prime importance in selecting suitable intake sites,
- Outer side of a bend- secondary currents develop on the outer side of bends which reduce the entry of bed load into the intake and deflect the bed currents away from the intake (***Figure 4-1***),
- Confluence of two rivers- the most suitable location for an intake is downstream of the confluence of two rivers,
- Boulder deposits- the location of big boulder deposit is stable in steep rivers,
- Bank stability-banks with rock outcrops, or armored boulder banks protecting the intake from erosion,
- River slope-intake location is stable at pool reach of the river slope,
- Stable geological section- intake is stable where geology is stable.

Figure 4-1 Layout plans of side intakes



4.4 Side Intake

4.4.1 Design Concept

A side intake is an opening on the bank of a river which extracts water from the river for the irrigation canal. Side intakes are less expensive than other types of intakes and are less complicated. Side intakes must be correctly located for maximum safety and effectiveness. They must be safe from boulder impact and flood water entry. The most appropriate location for a side intake is the outer bend of the river or near the natural pools (***Figure 4-2***). Side intakes are sensitive to river flood water levels.

Figure 4-2 Idle Locations for Side Intakes

4.4.2 Design Procedure

In its simplest form, a side intake consists of an ungated orifice in a masonry wall. The design of a side intake involves the sizing of an orifice and the design of a link canal. The design procedures are as follows:

Step-1 Select a suitable opening size in relation to the required discharge. For rectangular opening, the size width to depth ratio should be 2:1. The size of the orifice is determined considering the low flow of the river and needs to be checked to ensure safety during high flow periods. The opening size must be more or less similar to the size of the link canal that connects the intake to the settling basin. A trash rack is also necessary to prevent debris from entering into the canal.

Step-2 Compute discharge using formulae:

Free flow case: Free flow can occur only when the downstream flow depth in the canal is equal to or less than subsequent depth, h2 (*Figure **4-3***).

$$Q = C_d \times C_v \times b \times a^{3/2} \sqrt{2g\left(\frac{h_1}{a} - 0.63\right)}$$

Where, C_d is the discharge coefficient = 0.60,

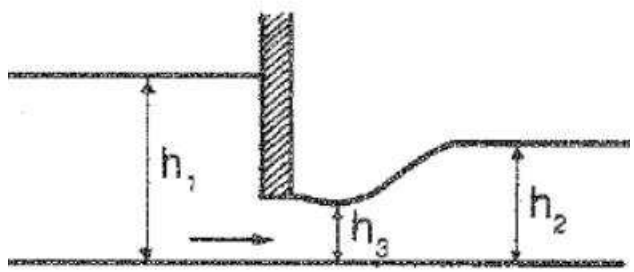
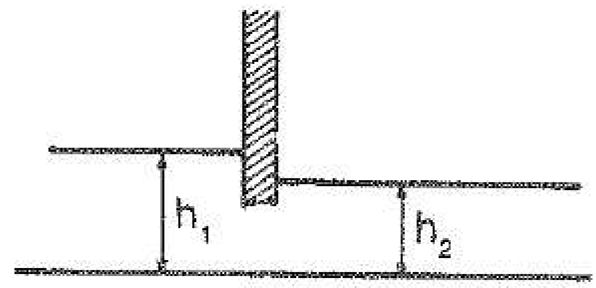
C_v is the velocity coefficient = 1,

b is the width of the orifice,

a is the height of the orifice,

h_1 is the depth of water upstream of the orifice,

Figure 4-3 Free flow and submersed flow orifice

Free flow	Submersed flow
	

Subsequent depth, h_2 is a function of Froude number, F,

$$F = \frac{V_3}{gh_3}$$

$$\frac{h_2}{h_3} = \frac{1}{2}\left[\sqrt{1+8F^2} - 1\right]$$

When the downstream water depth in the canal is greater than subsequent depth, the orifice will become submersed and submerged flow equation will apply.

Submersed flow case:

$$Q = C_d \times C_v \times b \times a^{3/2}\sqrt{2g(h_1 - h_2)}$$

Step-3 Check the maximum intake flow at high flood level. The link canal and automatic spill of the settling basin must be adequate to cope with the expected intake flow at the high flood level of the river. If these conditions are not satisfied, the following options should be considered to re-design the intake.

Option-1 Try different combinations of orifice size, link canal and auto spilling capacity of the settling basin;

Option-2 Provide a control gate at the orifice;

Option-3 Provide a double orifice intake

The basic hydraulic design for a side intake will depend on the size of sediment and hence on the desirable nominal trap velocity. The basic design parameters of the side intake are presented below (*Table 4-1*).

Table 4-1 Velocity and head loss across gravel trap

Description	**Coarse Sand**	**Fine/medium gravel**	**Medium/coarse gravel**
Velocity (m/s)			
Nominal trap velocity	0.15	0.25	0.50

Description	Coarse Sand	Fine/medium gravel	Medium/coarse gravel
Sill velocity	0.15	0.25	0.50
First orifice velocity	0.25	0.42	0.83
Outlet gate velocity	0.60	0.60	0.60
Head Loss (mm)			
At sill	3	8	32
At first orifice	8	22	88
At outlet gate	46	46	46
Total head loss	57	76	166
Take total head loss	60	80	170

Source: PDSP Manual, 1990

4.4.3 Design Example of Side Intake

Design a bank intake structure for a canal with design discharge of 50 lps. Assume that the sediment to be handled is mainly fine/medium gravel.
Design data:

- Canal design water level 101.00 m
- River minimum level 101.20 m
- River maximum level 103.00 m

i) Calculate net trap cross-section
From the Table 4-1, nominal trap velocity = 0.25 m/s
Trap cross sectional area (A_{te}) = 0.05/0.25 = 0.20 m^2
Adopting 1 m wide trap, the depth of trap = 0.20/1.0 = 0.20 m
A further 0.30 m depth is provided to allow for storage
Trap length L = 10 x Ate = 10 * 0.20 = 2.00 m
ii) Design of upstream orifice
From Table 4-1, for fine/medium gravel, upstream orifice velocity = 0.42 m/s
Orifice area (A) = Q/V = 0.050/0.42 = 0.119 m^2
Let the orifice be 0.60 m wide and 0.20 m deep, hence the area (A) = 0.12 m^2
Using the orifice formula the loss of head at the orifice is calculated.

$$Q = 0.7BD\sqrt{2g\Delta h}$$

$$\Delta h = \frac{Q^2}{\left(0.49 \times B^2 \times D^2 \times 2g\right)}$$

Where, Q = 50 l/s, B = 0.60 m, and D = 0.20 m
Then, Δh = 0.018 m, say Δh = 0.02 m
iii) Design downstream orifice
Let outlet gate velocity be 0.50 m/s
Then, orifice area = 0.05/0.50 = 0.10 m^2

Let orifice be 0.5 m wide and 0.20 m deep,

$$\Delta h = \frac{Q^2}{\left(0.49 \times B^2 \times D^2 \times 2g\right)}$$

Where, B = 0.50 m and D = 0.20 m

Then, Δh = 0.0025/0.096138 = 0.026, say Δh = 0.03 m

iv) Design of sill

From the table sill velocity = 0.25 m/s,

Area across the sill = 0.050/0.25 = 0.20 m2

If sill width = 1.0 m, then the depth of water over the sill = 0.20 / 1.0 = 0.20 m

Head loss across the sill = $\frac{KV^2}{2g}$

Where, K is coefficient = 2.5, and V is velocity = 0.25 m/s

Then, head loss = 0.008 m

v) Check minimum river level

Total head loss = 0.02 + 0.03 + 0.01 = 0.06 m

Minimum river water level = 101.00 + 0.06 = 101.06 m

Actual minimum river water level = 101.20 m

Therefore the structure level corresponding water levels are acceptable.

vi) Check structure for maximum river levels

Assume canal can take 50 % extra flow, that is 75 lps

For a 50 lps canal with z = 1:1 and n = 0.025 and bed slope 1: 500, B = 0.30m, D = 0.25m

Flow depth in the canal for 75 lps is 0.32 m

Therefore maximum water level in the canal = 101.00 -0.25 + 0.32 = 101.07 m

Head loss through the downstream orifice $\Delta h = \frac{Q^2}{\left(0.49 \times B^2 \times D^2 \times 2g\right)}$ = 0.0.08 m

Head loss through the upstream orifice $\Delta h = \frac{Q^2}{\left(0.49 \times B^2 \times D^2 \times 2g\right)}$ = 0.02 m

Head loss across sill (nearly) = 0.02m

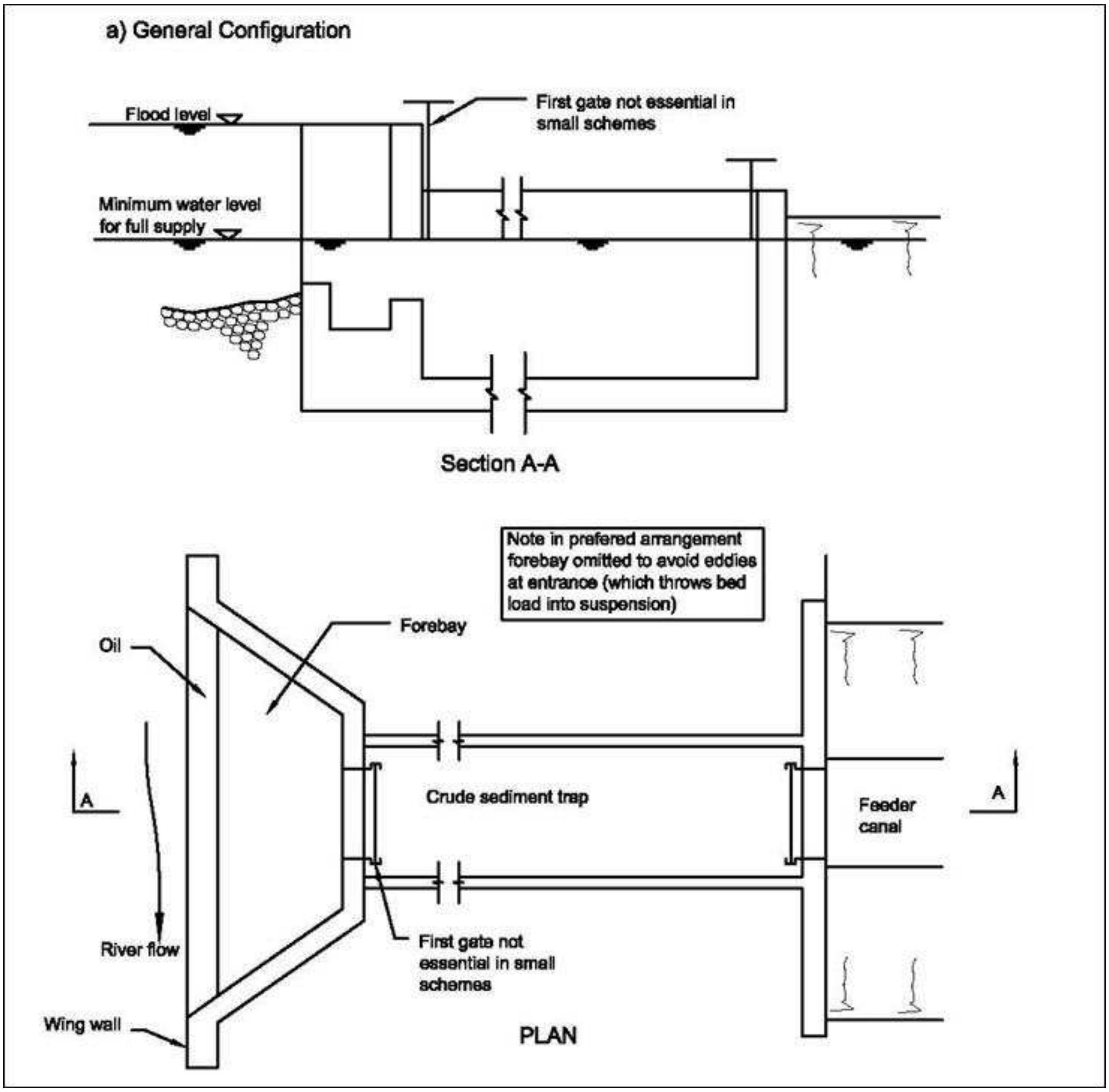
Total head loss = 0.08 + 0.02 + 0.02 = 0.12 m

Maximum river water level = 101.07 + 0.12 = 101.19 m

Actual maximum river water level = 103.00m

As the head over the orifice is high, a control gate on the canal orifice is necessary to prevent flood flow from entering the canal.

Figure 4-4 Side intake design

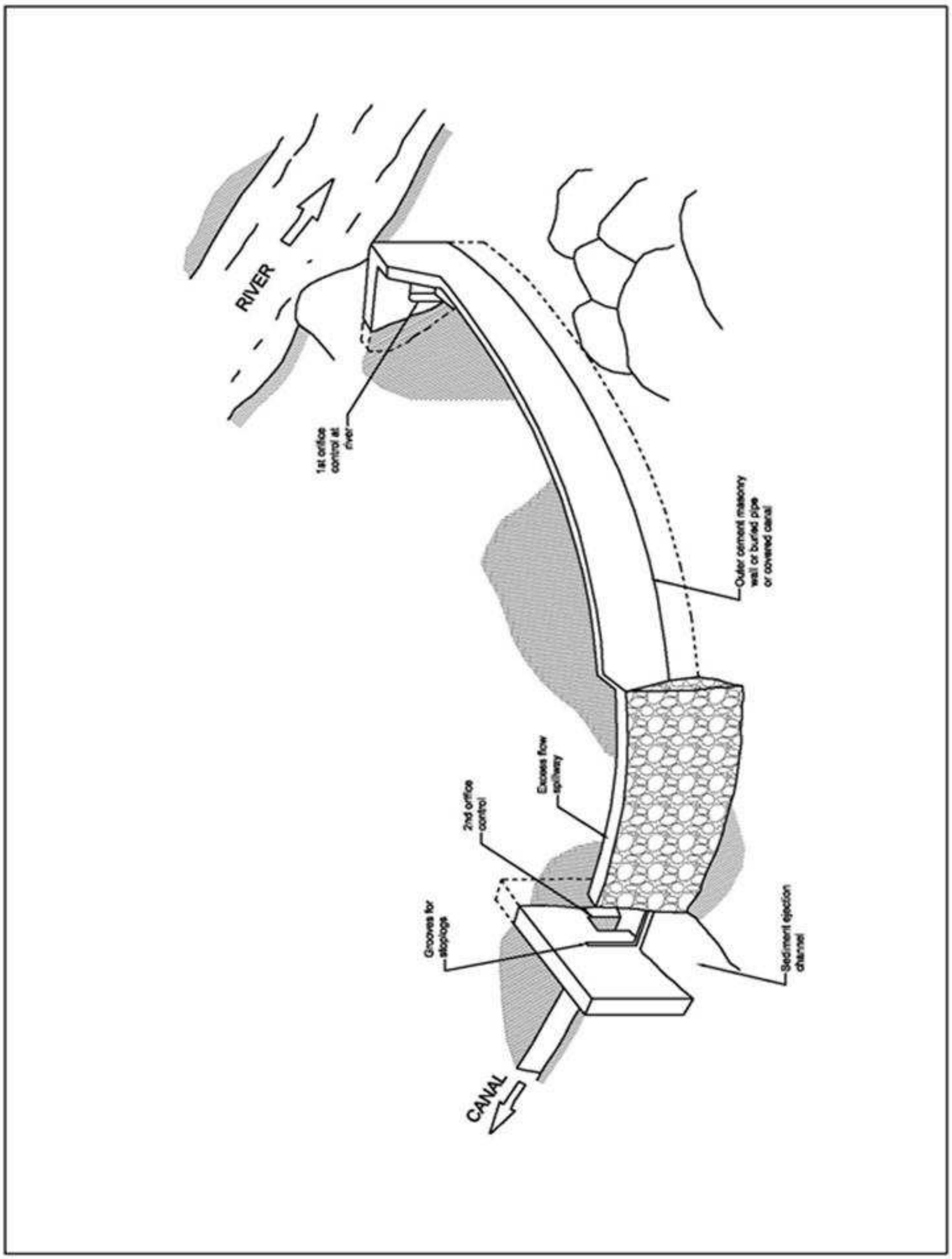


4.5 Double Orifice Intake

4.5.1 Design Concept

A double orifice intake has two orifices, one at the river intake proper and a second orifice at the end of the link canal or settling basin with a side spillway. The first orifice is the first to control the flow. During high stage of the river the first orifice delivers excess flow to the link canal. This excess flow would be spilled from a side-spillway towards the downstream end of a section of the link canal and sluices sediment from time to time. At low river levels, there will be a relatively small sediment supply; the link canal, if properly sized, will act as a sediment trap which may be cleared out by removing the stop-logs from the sediment ejection channel. At higher stages, the continuous operation of the side spillway will lead to a significant proportion of the bed material being transported through the link canal. A typical sketch of a double orifice intake is presented in *Figure **4-5***.

Figure 4-5 Typical sketch of a double orifice intake



4.5.2 Design procedure

(i) Assess the design with maximum allowable flows Q_1 and Q_2 to enter the canal downstream of the second orifice, with corresponding water levels at the canal entrance, WL_1 and WL_2. Let the energy heads, h_1 and h_2, be the same as these water levels,

(ii) Choose a size for the second orifice, A_2, to give a maximum velocity, say 2 m/s i.e. for the maximum allowable flow to enter the canal, Q_2,

(iii) Using A_2, calculate orifice head loss from graph at design canal flow, Q_1; add to h_1 to give energy head, h_3 in the structure,

(iv) Set the excess flow spillway crest level allowing 0.05 m as free board during normal conditions,

(v) As there is no friction loss in the channel, the first orifice downstream energy level can be taken as h_3. Calculate the head loss at the first orifice (it is suggested that both orifices are made the same size), and calculate the energy head, h_4, upstream of the first orifice (***Figure 4-6***),

(vi) Compare this energy head loss, h_4, with the minimum river level during the irrigation season. If there is excess available head, the system can deliver more than the required flow,

(vii) Now consider maximum canal flow, Q_2, with second orifice downstream energy level, h_2, assuming the link canal has scoured out,

(viii) Calculate the head loss at the second orifice and add to h_2 to give h_3, the energy head in the structure,

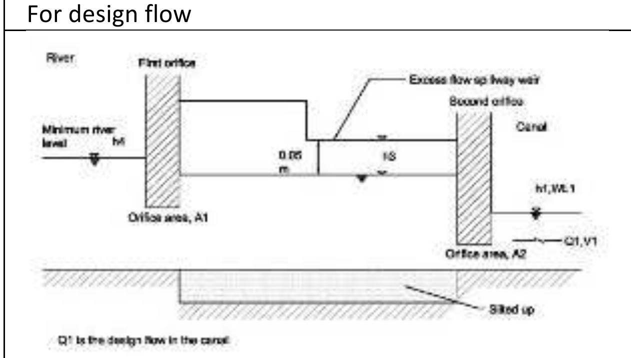
(ix) Calculate the head over the crest of excess spillway using broad crest weir formula,
The downstream energy level at the first orifice can be taken as h_3,

(x) Calculate the orifice head loss with flow rate Qmax, add to h_3 to give h_4 and compare with maximum river level. If the river level is higher, there will be a greater inflow and the side-weir is too short. If the maximum river level is lower, the crest length of the weir calculated above is more than adequate.

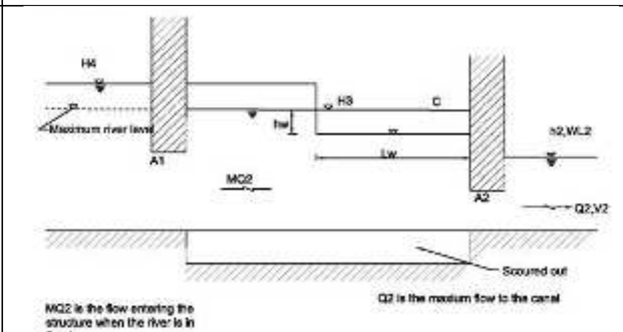
(xi) Amend the assumed value of spill water above accordingly and repeat until the head upstream of the first orifice agrees with maximum water level, or it is obvious that the design needs to be reconsidered, i.e. by changing one or both orifice sizes, or the ratio Q_2/Q_1, or the relative canal and river levels, etc.

Figure 4-6 Double orifice intake

For design flow



For maximum flow



4.5.3 Design Example of Double Orifice Intake

Design flow in canal (Q_1)	= 250 l/s
Maximum allowable flow in canal (Q_2)	= 350 l/s
Canal design water level (h_1)	= 502.00 m
Canal maximum allowable water level (h_2)	= 502.11m (see below)
Structure length	= 30.00 m
Minimum river level	= 502.50 m
Maximum river level	= 504.00 m
Maximum flow entering structure	= 700 l/s

Calculations

For a canal with a design flow of 250 l/s, water slope of 1:500, n = 0.025,
Z = 1:1, b = 0.45 and d = 0.45.
For maximum allowable flow in canal of 350 l/s using formula:

$$\left[\frac{Q_a}{Q_b}\right]^{0.60} = \frac{d_a}{d_b} \qquad d_a = d_b \times \left[\frac{Q_a}{Q_b}\right]^{0.60}$$ = 0.45 *(0.35/0.25)^0.60 = 0.55 m

Maximum canal water level = 502.00 + (0.55 – 0.45) = 502.10 m
Area of flow at design flow A1 = b*d1+m*d1^2= 0.45*0.45+1*0.45^2 = 0.41 m2
Velocity at design flow Vdes = Qd/A1 = 0.25/0.41 = 0.62 m/s
Area of flow at maximum flow A2 = b*d2+m*d2^2= 0.55*0.45+1*0.55^2 = 0.55 m2
Velocity at maximum flow Vmax = Qmax/A2 = 0.35/0.55 = 0.64 m/s
For design canal flows
h1 = WL1 = 502.00 m
For maximum allowable canal flows
H2 = WL2 = 502.10 m
Assume maximum velocity of 2 m/s through orifice (i.e. for maximum allowable flow in canal).
For maximum allowable flow, Q2 = 0.35 m3/s
A2 = 0.35/2.0 =0.175 m2
Let orifice dimension: Width be 0.45 m and the depth be 0.40 m
The orifice width is same as that of canal bed width and the depth allows adequate submergence of the orifice at all flows.
For design canal flows
For Q1 = 250 l/s, B = 0.45 m, D = 0.40 m
Head loss,

$$\Delta h = \frac{Q^2}{(0.70 \times B \times D)^2 \times 2g}$$ = 0.20 m

Water level d/s of first orifice, h3 = h1 + Δh = 502.00 + 0.20 = 502.20 m
To allow a freeboard of 0.05 m, make crest level of excess flow spillway 502.25 m
Assume inlet orifice is same size as outlet orifice B = 0.45 m and D = 0.40 m
Head loss

$$\Delta h = \frac{Q^2}{(0.70 \times B \times D)^2 \times 2g}$$ = 0.20 m

Water level u/s of first orifice, h4 = h3 = Δh = 502.20 + 0.20 = 502.40 m
Check for minimum river level, h4 is less than min river level of 502.50 m (h4<NWL)
River level is adequate
For maximum allowable canal flow, Q2 of 350 l/s
H2 = 502.10 (see step (i) above)
For maximum canal flows head loss

$$\Delta h = \frac{Q^2}{(0.70 \times B \times D)^2 \times 2g}$$ = 0.39 m

Water level d/s of orifice at maximum flow
h3 = h2 + Δh = 502.10 + 0.39 = 502.49 m
Head across excess flow spillway = water d/s of first orifice – spillway crest level
hw = 502.49 – 502.25 = 0.24 m
Assume flow entering from the river in flood,
Qf = 2 x Q2 = 2*350 = 700 l/s
Amount of flow to be spilled Qspill = 700- 350 = 350 l/s
Length of spillway crest, LW = Qspill/(1.4*h3/2) = 0.35/1.4 x (0.24)1.5
= 2.08 m say 2.20 m
For maximum flow entering from river
For A = 700 l/s, B = 0.45 m, D = 0.40 m head loss, Δh

$$\Delta h = \frac{Q^2}{(0.70 \times B \times D)^2 \times 2g}$$ = 1.57 m

Water level u/s of first orifice at maximum flow = h3 + Δh = 502.49 + 1.57 = 504.07 m
Maximum river level = 504.00 m which is less than 504.07 m
Therefore structure design is acceptable.

4.6 Bottom Intake or Trench Intake

4.6.1 Design Concept

A bottom intake or trench intake is located in the river bed that draws off water through racks into the trench which conveys the flow to the main canal. The main characteristic of the bottom intake is that it has minimum impacts on river levels. This type of intake is mostly suitable in hilly regions where rivers have low sediment concentration. The main components of a bottom intake are:

- Weir to control minimum river flows and to maintain the flow towards the rack,
- Steel rack to extract water from the river,
- Bottom trench immediately below the rack to lead the water extracted by the rack into the canal,
- Flood control gate immediately downstream of the trench,
- Feeder canal to carry all sediment entering the intake to the settling basin,
- Settling basin with a side spill and sediment flushing arrangement,

4.6.2 Design Procedure

Data Required

- Maximum diversion discharge required (Qc),
- Design flood discharge (Qd),
- Existing width of the river (P),
- Bar diameter (a),
- Bar spacing (b)(as per the desirable sediment size required to be eliminated),
- Approach flow depth (ho),

The design procedure of the bottom intake is related to the design of the size of the trench and collection chamber including sediment flushing arrangements (*Figure **4-7***).

i. Select the height of the weir (P) above the river bed

ii. Fix the length of the weir based on the existing width of the river,

$$P = 4.75\sqrt{Qd}$$

Where P is the Lacey's waterway in m and

Qd is the design flood discharge in m^3/s,

iii. Calculate the head over the rack crest using the formula,

$$h = \frac{2}{3} k h_o$$

Where ho is the approach flow depth in m, and k is constant depending upon the inclination of the trash rack (β),

β	0	2	4	6	8	10	12	14	16	18	20	22	24	26
k	1.00	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.87	0.85	0.84	0.83	0.81	0.80

iv. Calculate coefficient of discharge through trash rack using the formula,

$$C = 0.60 \times \frac{a}{b} \times \cos^{3/2} \beta$$

Where a is the width of the bar section,

b is the center to center distance between bars

β is the inclination of the trash rack

v. Calculate width and length of the trench opening or trash rack using the formula,

$$Q_A = \frac{2}{3} \times C \times \mu \times b \times L \times \sqrt{2gh}$$

Where C is the discharge coefficient,

μ is the contraction coefficient of the trash rack and depends upon the type of bars,

b is the width of the trash rack

L is the length of the trash rack and

h is the head over the rack crest

The calculation of the width and length of the trash rack should be based on a trial and error method and the final dimensions should be determined by whether the structure is easy to operate and the required flow is ensured. For practical purposes, the length of the trash rack is increased by 20%.

vi. Calculate the size of the collection canal - along the trash rack width (weir width) the amount of water for irrigation falling through the rack opening increases linearly and reaches its maximum value at the end of the cross section. For reasons of simplicity, this end cross-section is used for the dimensioning of the collection canal.
The water depth in the collection canal is calculated using a continuity equation along with Manning's formula,

$$Q = AV = B \times d \times \frac{1}{n}\left(\frac{B \times d}{B + 2d}\right)^{2/3} \times S^{1/2}$$

Where B is the width of the canal and is the same as the length of the trash rack,
d is the water depth in the collection canal to be assessed,
n is the roughness coefficient, and
S is the slope of collection canal

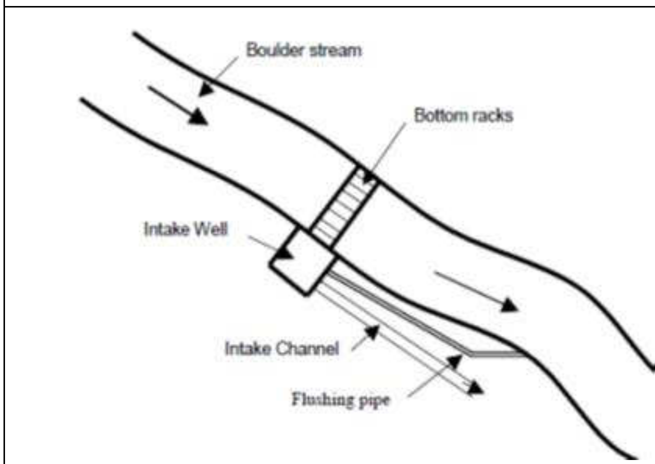
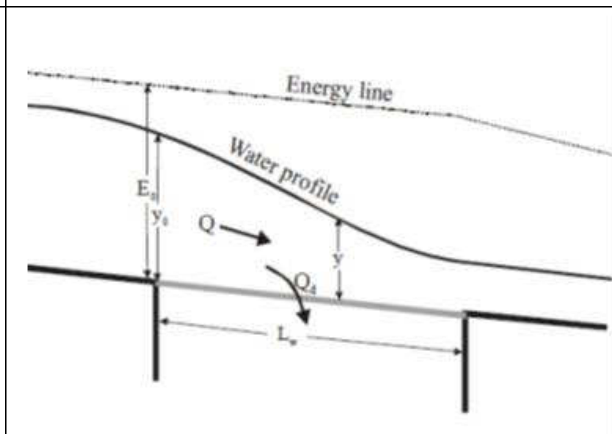
Water depth can be calculated using trial and error.

vii. Calculate the free board of the canal from

$F_b = 0.25d$ where d is the dater depth in m,

viii. Fix the total depth of collection canal

Figure 4-7 Bottom intake design

<table>
<tr><td>Layout of weir</td><td>Flow through racks</td></tr>
<tr><td>
</td><td>
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4.6.3 Design Example of Bottom Intake

Data

Canal discharge (Q) = 400 l/s

Design flood discharge (Qd) = 12 m3/s

Average water depth at low flow (ho) = 0.30 m

Existing width of the river (W) = 15 m

Calculations

Calculate the length of weir across the river from P= 4.75 sqrt (Qd)

P = 4.75*sqrt(12) = 16.45 m adopt weir length as the existing width of river as 15 m

Adopt height of weir as 0.30 m above the river bed level,

Adopt slope of trash rack as 8o (β)

Adopt flow for flushing (Qf) = 1.5* Q = 1.5* 400 = 600 l/s

Design flow considering clogging of 40% (QA)=1.4*Qf = 1.4* 600 = 840 l/s

Calculate head over the u/s edge of the trash rack from

h = 2/3*k*ho where k contraction coefficient of rack; for 8o k = 0.927 (from above table)

ho is the average depth of water at low flow (approach flow depth) = 0.30 m

h= 2/3*0.927*0.30 = 0.19 m

Calculate coefficient of discharge, $C = 0.60 \times \frac{a}{b} \times \cos^{3/2} \beta$

Where a is the width of bar section = 0.02 m (adopt)

b is the center to center distance between bars = 0.04 m (adopt)

β is the inclination of trash rack = 8o

C = 0.60 *0.02/0.04* cost^3/2 8 = 0.30

Calculate size of trash rack from formula, $Q_A = \frac{2}{3} \times C \times \mu \times b \times L \times \sqrt{2gh}$

Where C is the discharge coefficient = 0.30 from calculation,

µ is the contraction coefficient of trash rack and depends upon type of bars, for ordinary rack µ = 0.85

b is the width of the trash rack

L is the length of the trash rack and

h is the head over the rack crest = 0.19 m from calculation

Substituting values we get 0.84 =2/3*0.3*0.85*b*L*√(2*9.81*0.19)

0.84 = 0.32 b*L

Solving this equation we get

b (m)	2	4	6	8	10
L (m)	1.30	0.65	0.43	0.32	0.26

Select width of trash rack, b = 4 m, and length of trash rack is 0.65 m
From operational point of view the length of trash rack is taken (Lt) =1.2 L
Lt= 1.2* 0.65 = 0.78 m and hence adopt as 0.80 m

Calculate the dimensions of collection canal from,

$$Q = AV = B \times d \times \frac{1}{n}\left(\frac{B \times d}{B + 2d}\right)^{2/3} \times S^{1/2}$$

Where B is the width of the canal and is same as the length of the trash rack= 0.80 m,
d is the water depth in the collection canal to be assessed,
n is the roughness coefficient = 0.02 for concrete and
S is the slope of collection canal = 0.03 (S is taken as 30%)
For Q = 0.84 m3/s we get, 0.84 = 0.80*d*1/0.02*(0.03)^1/2*(0.80*d/(0.80+2*d)^2/3
From trial and error, d= 0.355 m
Calculate free board as 0.25 * d = 0.25* 0.355 = 0.09 m
Total height of canal at collection chamber = 0.355 +0.09 = 0.44 m
Adopt D= 0.45 m

Hence, design is acceptable.

4.7 Diversion Headwork

4.7.1 Introduction

A diversion headwork may be a weir or a barrage depending upon the objective of the irrigation scheme. Weirs are solid walls across the river to raise the water level and to divert water into the irrigation canal. Weirs may be provided with small shutters on the top. During floods, as the entire discharge of the river has to pass over the crest, there is considerable afflux on the upstream due to the solid wall across the river.
A barrage has a similar structure as a weir but the gate alone affects the heading of the water on the upstream. The barrage crest level is on the average river bed level and has minimum afflux. From an operational point of view, a barrage is better than a weir; however, the cost of a barrage is much higher than that of a weir.

4.7.2 Ideal site for diversion headwork

An ideal site for diversion headworks should have the following characteristics:

- Narrow and well defined river section,
- River should have high and non-submergible banks in order to minimize the cost of river training,
- Maximum canal command with moderate earth works,
- Suitability of river diversion during construction,
- The site should be such that the weir or barrage can be aligned right angled to the river flow,
- Suitable location for all components of the headworks,
- Diversion headworks should not submerge valuable property upstream,
- Good foundation should be available at the site,
- The site should be accessible and construction material available,

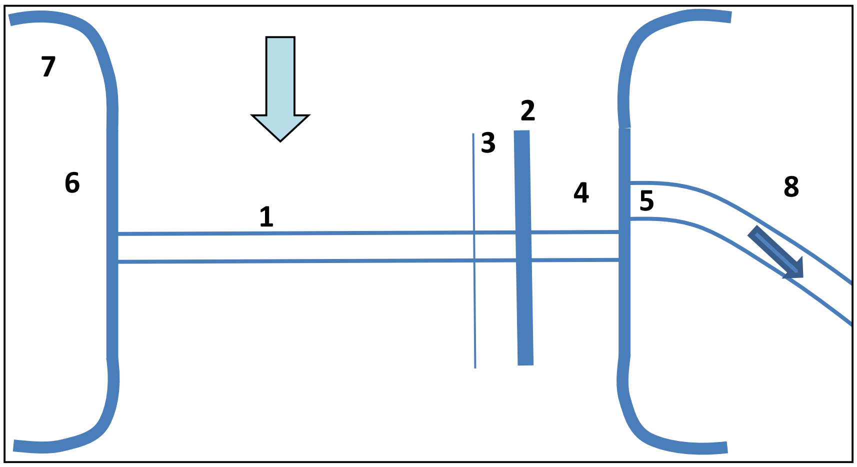
4.7.3 Components of a Headwork

The diversion weir is a hydraulic structure across the river which maintains the pond level at its upstream so that water can be diverted to the off-taking canal. Generally, a diversion headwork is constructed in perennial rivers which have adequate flow throughout the year. A diversion headwork may either be a weir or a barrage depending upon the type and objectives of the scheme.

For small scale irrigation schemes, weirs are suitable. According to the construction material, weirs may be classified as masonry weir, concrete weir and gabion weir. According to the shape of the crest, weirs are classified as vertical drop weir, Ogee weir, broad crested weir, and sharp crested weir. The layout of the diversion weir and its components are presented below (***Figure 4-8***).

1. Weir 2. Divide wall 3. Fish ladder for large weir 4. Under sluice 5. Head regulator 6. Abutments 7. Guide bund 8. Canal

Figure 4-8 Definition sketch of weir and its components

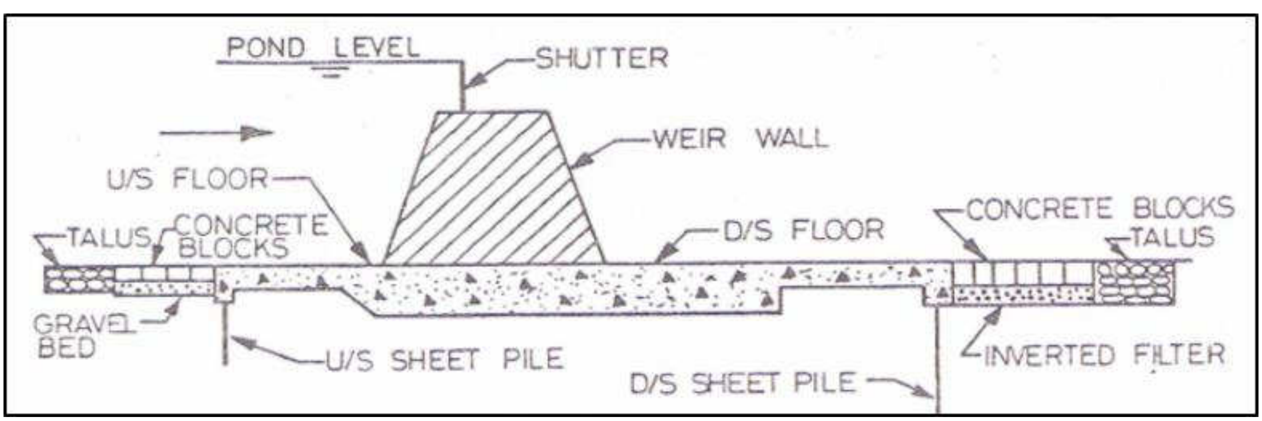


4.7.4 Types of weirs

Based on the flow characteristics, weirs are grouped into two types: sharp crested weir and broad crested weir. Sharp crested weirs are used as flow measuring devices in irrigation while broad crested weirs are used for diversion works. There are three categories of weirs, distinguished by how they are constructed and the materials used: (i) Vertical drop weirs, (ii) Rockfill weirs, and (iii) Concrete weirs with sloping glacis.

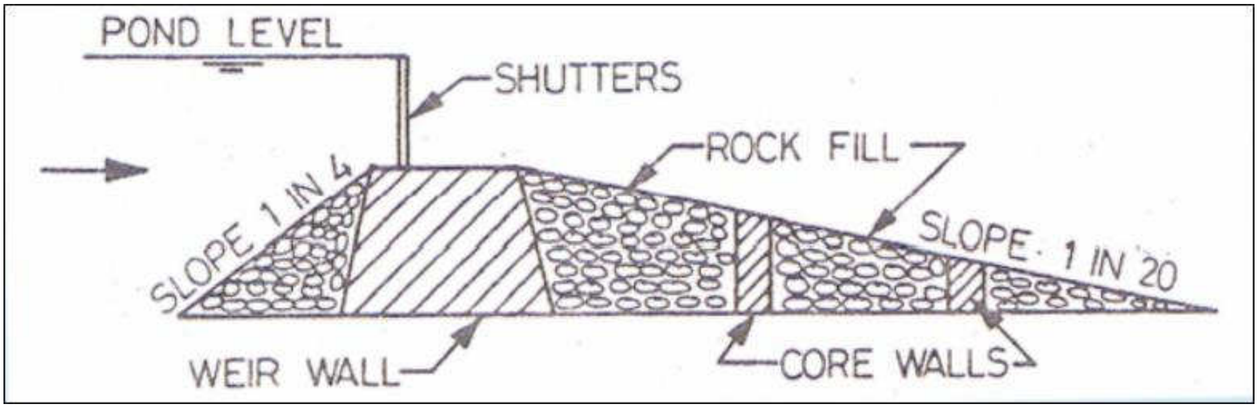
(i) Vertical drop weirs are wall type structures on a horizontal concrete floor. Shutters may also be provided on the crests which are dropped during the flood. Water is ponded up to the top of the shutters during non-flood periods. Vertical drop weirs are suitable for small irrigation diversions, a typical sketch of one is presented here (*Figure **4-9***).

Figure 4-9 Vertical drop weir



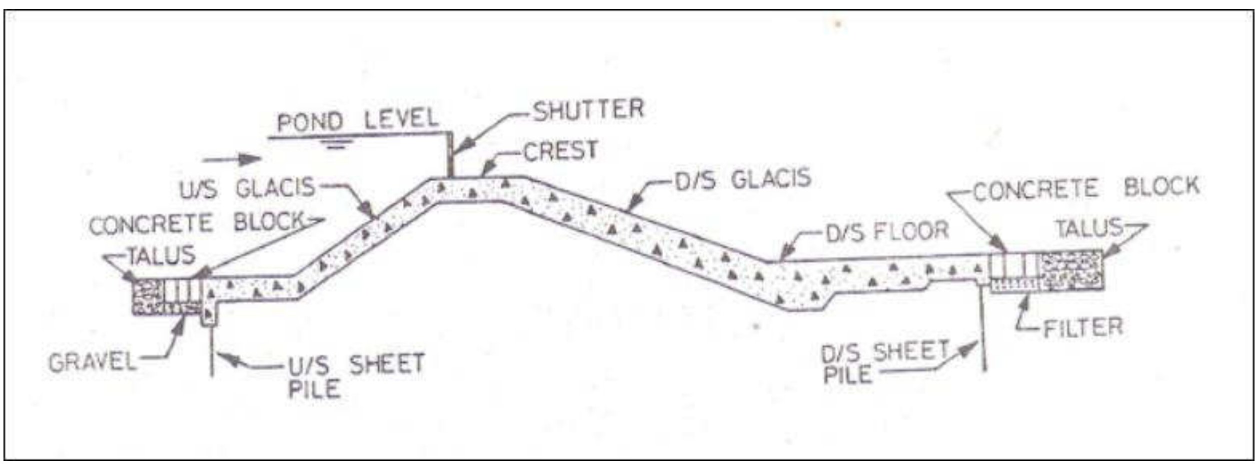
(ii) Rockfill weirs consist of a number of core walls across the river, with the space between these walls filled in by rocks. These weirs are economical in areas where rocks are available. A typical sketch of a rockfill weir is presented here (*Figure **4-10***). For small irrigation schemes, gabion weirs with core walls are also used which are economical but susceptible to flood damage.

Figure 4-10 Sketch of rockfill weir



(iii) Concrete weirs with sloping glacis are common for both small and large irrigation diversions. The crest has sloping glacis both on the upstream and downstream sides. Energy dissipation is achieved with great deal at sloping glacis and thus concrete weirs are extensively used in irrigation diversions and in large drops. A typical sketch of a concrete weir is presented here (*Figure **4-11***).

Figure 4-11 Sketch of concrete weir



4.7.5 Design concept of weir

The design of a weir consists mainly of two parts: hydraulic design and structural design. The hydraulic design shall decide the following parameters:

- Length of waterway, discharge intensity and afflux;
- Safe exit gradient;
- Lacey's silt factor, $f = 1.76\sqrt{Dm}$ where, D_m- average size of sediment particle in mm;
- Depths of cutoffs in relation to both scour depth and exit gradient;
- Level and length of downstream horizontal floor;
- Thickness of downstream horizontal floor with reference to uplift pressure and hydraulic jump;
- Length and thickness of protection works

In addition to the above parameters, the hydraulic design of weir involves the following concepts:

Crest level of weir and undersluice

Fixation of the crest level of the weir is governed by two factors: the required command level of the canal and available low flow of the river during the dry season. If the minimum flow of the river exceeds the design discharge of the canal, the crest level of the weir is set lower than the water level of the river. The crest level of the undersluice is fixed at the lowest bed level of the river in the deepest channel and is generally kept 1 m below the crest level of the weir in small and medium rivers (*Figure **4-12***).

Figure 4-12 Typical weirs

Concrete weir	Gabion weir
	

Length of weir and afflux

The length of a weir is directly correlated with the afflux and discharge intensity of the weir. The longer the length the smaller is the discharge intensity. The length of the weir also depends upon the topography of the weir site, river characteristics, and provision of energy dissipation at downstream end. Lacey has provided formula for waterway:

$P = 4.83\sqrt{Q}$ Where, Q is the high flood discharge in m^3/s

For boulder stage rivers, Lacey's formula may not work properly and the designer has to judge considering the following main points:

- Average wetted width of the river during flood;
- Formation of shoals upstream of the weir;
- Energy dissipation devices downstream of the weir;

The discharging capacity of the undersluice is determined with the consideration of 20% of the design flood or twice the canal discharge and calculated as:

$$Q = 1.705(L - 0.1nH)H^{3/2}$$

Where, L is total clear waterway in m,
n is number of end contraction,
H is head over the crest in m

Effect of retrogression

Retrogression is the lowering of the river bed downstream of the weir after its construction. In designing the weir, the effect of retrogression needs to be considered. For small rivers the retrogression is taken to be 0.30 m to 0.50 m.

Design of hydraulic jump basin

Hydraulic jump occurs when a super critical flow of water changes into a sub-critical flow. This is the most efficient method of dissipating the kinetic energy of flowing water. Hydraulic jump is stable in sloping glacis and hence it is essential to provide sloping glacis at the weir surface with the slope ranging from 1:3 to 1:5. With the analysis of hydraulic jump, the following parameters are designed:

- Downstream floor level,
- Length of horizontal floor,
- Downstream protection works, and
- Uplift pressure

4.7.6 Design Procedure of Weir

The following data are essential for the design of diversion weirs:

- Maximum flood discharge (Q_d)- based on design return period flood,
- Stage discharge curve of the river,
- Minimum water level of the river,
- Cross-section of the river at weir site
- Average sediment size, and silt factor

Step 1 Fixation of crest levels of weir and undersluice;
Weir crest level = minimum water level of the river
Undersluice crest level = deepest river bed level

Step 2 Calculate required waterways of weir and undersluice,
Lacey waterway, $P_w = 4.83\sqrt{Q}$
Divide total water ways into weir and undersluice with consideration of divide wall and piers,

Step 3 Calculate discharge intensity of weir, scour depth and velocity head therein,
q = Q/L, $R = 1.35\sqrt[3]{\left(\frac{q^2}{f}\right)}$ and approach velocity, V= q/R

Step 4 Determine the head over the crest of weir and undersluice = u/s TEL – crest level

Step 5 Calculate the discharges passing through the undersluice (Q_1) and weir (Q_2) for both flood flow and pond level flow and compare with design flood discharge,

$$Q_1 = 1.705\left(L_1 - 0.10nH_1\right)H_1^{3/2}$$

$$Q_2 = 1.84\left(L_2 - 0.10nH_2\right)H_2^{3/2}$$

Where L_1 is the length of undersluice,

L_2 is the length of weir,

H_1 is the head over the crest of undersluice,

H_2 is the head over the crest of weir

Step 6 Determine head loss for different flow conditions (high flood flow and pond flow); H_L = u/s TEL – d/s TEL

Step 7 Determine parameters of hydraulic jump of under sluice for flood flow and pond level conditions considering with and without concentration and retrogression

Hydraulic jump calculations are carried out either using Blench curves or specific energy curves or by trial and error method.

Step 8 In trial and error provide trial value of post jump depth and calculate pre jump depth from formula:

$$D_1 = -\frac{1}{2}D_2 + \sqrt{\left(\frac{2/g \times q^2}{D_2} + \frac{1}{4}D_2^{\ 2}\right)}$$

Step 9 Calculate specific energy of u/s and d/s of jump from formula:

$$Ef_2 = D_2 + q^2/\left(2g \times D^2{}_2\right)$$

$$Ef_1 = D_1 + q^2/\left(2g \times D^2{}_1\right)$$

Step 10 Calculate head loss, Hl= E_{f1}-E_{f2} and equal with calculated values of head loss in step 6,

Step 11 Calculate level of jump formation, length of concrete floor and Froude number for each flow condition,

Level of jump = d/s TEL - E_{f2}

Length of concrete floor, L = 5 (D_2-D_1)

Froude number, $F = \frac{q}{\sqrt{gD_1}}$

Step 12 Adopt d/s floor level as minimum of jump levels and adopt d/s floor length as maximum of concrete floor lengths for different flow conditions,

Step 13 Determine the u/s and d/s scour depths from the formula of scour consideration, $R = 1.35\left(\frac{q^2}{f}\right)^{1/2}$; determine depth of u/s sheet pile or cutoff as 1 to1.25 R and d/s sheet pile or cutoff as 1.25 to 1.5 R;

Step 14 Work out total floor length and exit gradient (GE), determine value of $\frac{1}{\pi\sqrt{\lambda}}$ from the equation $\frac{1}{\pi\sqrt{\lambda}} = G_E\frac{d}{H}$ for a given value of GE and known values of d and H. From corresponding value of $\frac{1}{\pi\sqrt{\lambda}}$ read the value of α from Khosla's exit gradient curve.

Step 15 Calculate the total length of floor (b) = α x d; disposition of total floor length may be as follows:

Cistern length = 5 to 6 (D_2-D_1)

Glacis length = 3 to 5 times (crest level – cistern level) for 1:3 to 1:5 slope of glacis;

Balance for u/s floor length,

Step 16 Determine uplift pressure acting on the floor; % pressures at u/s and d/s sheet pile lines are worked out as:

% pressure at upstream sheet pile line

For known value of b and d_1, $\frac{1}{\alpha} = \frac{d_1}{b}$, having known α read out values of Φ_D and Φ_E from Khosla's pressure curve.

% pressure at the bottom of sheet pile = 100 - Φ_D

% pressure at the bottom of floor = 100 - Φ_D

% pressure at downstream sheet pile line

For known value of b and d_2, $\frac{1}{\alpha} = \frac{d_2}{b}$, read values of Φ_D and Φ_E corresponding to $\frac{1}{\alpha}$ from Khosla's pressure curve which would be % pressure at the bottom of the floor and sheet pile respectively,

Step 17 Correction due to floor thickness

The thickness of the floor at the location of the sheet piles are tentatively assumed for correcting the values of Φ_C in the u/s and Φ_E in the d/s. If t_1 is the floor thickness at u/s sheet pile of depth d1, correction due to floor thickness = $\frac{t_1}{d_1}(\phi_D - \phi_C)$which is positive. If t_2 is the floor thickness at d/s sheet pile of depth d2, correction due to floor thickness = $\frac{t_2}{d_2}(\phi_E - \phi_D)$which is negative,

Step 18 Correction due to mutual interference of sheet piles is worked out by the formula:

$$C = 19\sqrt{\frac{D}{b'}} \times \frac{d+D}{b}$$

Step 19 After knowing the corrected % pressures at key points, depth of sheet piles or cutoffs, thickness of floor is calculated at different points from the d/s end of the cutoff wall.

4.7.7 Design Example of Concrete Weir

Design data:

High flood discharge, Q	35 m^3/s
Average bed level of river	303.8 m
HFL before construction	305.75 m
Permissible Afflux	0.5 m
Pond Level	305.25 m
Silt factor	1
Safe exit gradient	0.20 (1/5)
Concentration	20%
Retrogression	0.5 m

Fixation of crest level and waterway

Crest level of undersluice is taken at 303.80 m

Crest level of weir is taken at 305.25 m

Lacy's waterway Pw = 4.83x√Q = 4.83 x√35 = 28.63 m

Waterways for undersluice and weir

Undersluice portion both side

1 bay of 1m each L1 = 1m

Overall waterway 1m

Weir portion

1 bay of 17 m each L2 = 17 m

Overall waterway 17 m

Assume 2 divide walls 0.5 m thickness = 1 m

Overall waterway, L = 2x1+17+1= 20 m

U/S HFL = d/s HFL +Afflux =305.75+0.50 = 306.25 m

Discharge intensity, q = Q/L = 35/20 = 1.75 m3/s

Scour depth, R = 1.35x (q2/f) 1/3 = 1.35 x (1.75^2/1)^1/3 = 1.96 m

Approach velocity, V = q/R = 1.75/1.96 = 0.89 m/s

Velocity head, hv =V2/2g = 0.89^2/2x9.81 = 0.04 m

u.s. TEL = H.F.L. + velocity head = 306.25 + 0.04 = 306.29 m

Head over the undersluice crest, H1 = u/s TEL – crest level = 306.29 – 303.80 = 2.49 m

Head over the weir crest, H2 = u/s TEL – crest level = 306.29 – 305.25 = 1.04 m

Discharge passing through undersluice and weir are Q1 and Q2 respectively,

Q1 = 1.705 (L1-0.1nH1) H13/2 = 1.705*(1-0.1*0*2.49)*2.49^3/2 = 6.70 m3/s

Q2 = 1.84 (L2-0.1nH2) H23/2 = 1.84* (17 – 0.1*0*1.04) 1.04^3/2 = 33.17 m3/s

Total discharge passing through headwork Qd = 2x Q1 + Q2 = 2*6.70 + 33.17 = 46.57 m3/s

Since Qd > Q the crest level and waterway are OK

Looseness factor = L/Pw = 20/28.63 = 0.70

Flow at pond level

Head over the undersluice crest, H1 = Pond level – crest level = 305.25 – 303.80 = 1.45 m

Head over the weir crest, H2 = Pond level – crest level = 305.25- 305.25 = 0.00 m

Discharge passing through undersluice, Q1

Q1 = 1.705(L1-0.1nH1) H13/2 = 1.705*(1-0.1*0*1.45)*1.45^3/2 = 2.98 m3/s

Q2 = 1.84(L2-0.1nH2) H23/2 = 1.84* (17 – 0.1*0*0) 0 = 0.00 m3/s

Total discharge passing through headwork Qd = Q1 + Q2 = 2.98 m3/s

Discharge intensity, q = Q1/L1 = 2.98/1 = 2.98 m3/s/m

Normal scour depth, R = 1.35 x (q2/f)1/3 = 1.35* (2.98^2/1)^1/3 = 2.80 m

Velocity approach, V = q/R = 2.98/2.80 = 1.07 m

Velocity head, v =V2/2g = 1.07^2/2*9.81 = 0.06 m

u/s TEL = pond level + velocity head = 305.25 + 0.06 = 305.31 m d/s water level for Q = 2.98 m3/s = 304.75 m from stage discharge curve

d/s TEL = d/s water level + velocity head = 304.75 + 0.06 = 304.81 m

Head loss = u/sTEL – d/s TEL = 305.31 – 304.81 = 0.50 m

Further design is carried out in tabular form (Table 4-2 and *Table **4-3***).

Table 4-2 Design of undersluice

Descriptions	For maximum flood		For pond level	
	Without concentration and retrogression	With 20% concentration and 0.50m retrogression	Without concentration and retrogression	With 20% concentration and 0.50m retrogression
Discharge intensity, $q = 1.7 \times H^{3/2}$	6.68	8.02	3.15	3.78
Head required for q	2.49	2.81	1.45	1.70
d/s HFL	305.75	305.25	304.75	304.25
d/s TEL (d/s HFL + velocity head)	305.79	305.29	304.81	304.31
u/s HFL (d/s HFL + afflux)	306.25	306.25	305.25	305.25
u/s TEL (u/s HFL + velocity head)	306.29	306.61	305.31	305.50
Head loss	0.50	1.32	0.50	1.19
Post jump depth, D_2-trial value	2.66	3.42	1.74	2.21
Pre- jump depth, $D_1 = -\frac{1}{2}D_2 + \sqrt{\left(\frac{2/g \times q^2}{D_2} + \frac{1}{4}D^2{}_2\right)}$	0.95	0.89	0.52	0.49
d/s specific energy, $Ef_2 = D_2 + q^2/\left(2g \times D^2{}_2\right)$	2.98	3.70	1.90	2.36
u/s specific energy, $Ef_1 = D_1 + q^2/\left(2g \times D^2{}_1\right)$	3.48	5.02	2.41	3.55
E_{f1}-E_{f2} = Head loss (should be equal to above value)	0.50	1.32	0.50	1.19
Level of jump = d/s TEL - E_{f2}	302.81	301.59	302.91	301.95
Length of concrete floor L =5 (D_2 - D_1)	8.56	12.62	6.09	8.62
Froude no $F = q/(g*D^3{}_1)^{0.5}$	2.31	3.04	2.70	3.55

d/s floor level is provided at (minimum of jump level) 301.59m

d/s floor length (maximum of length of concrete floor) 12.62 m roundup to 13 m

Depth of Cut-off from scour consideration

Discharge passing through undersluice 6.68 m^3/s

Overall waterway of undersluice 1.00 m

Discharge intensity 6.68 $m^3/s/m$

Scour depth, R = 1.35x (q2/f) 1/3 = 1.35 *(6.68^2/1)^1/3 = 4.80m

u/s cut-off

On u/s side allow cut-off depth as 1.1 R = 1.1*4.80 = 5.28 m

RL of bottom of scour hole = u/s HFL – cut-off depth = 306.25 – 5.28 = 300.97 m Say 300.90 m

d/s cut-off

On d/s side allow cut-off depth as 1.25 R = 1.25 * 4.80 = 6.00 m

RL of bottom of scour hole = d/s HFL – cut-off depth = 305.25 – 6.00 = 299.25 m

Say 299.00 m

Total floor length and exit gradient

The exit gradient should be checked for the condition that the canal is completely closed when high flood is passing in the river; this provides worst static condition.

Maximum static head = Pond level – d/s floor level = 305.25-301.59 = 3.65m

Depth of d/s cutoff = d/s floor level – RL of bottom of scour hole = 301.60-99.00 = 2.60 m

Exit gradient, $\frac{1}{\pi\sqrt{\lambda}} = G_E \frac{d}{H}$

Hence, $\frac{1}{\pi\sqrt{\lambda}}$ = 1/5*2.60/3.65 = 0.142

From Khosla's exit gradient curve, α (alfa) = 8.9

Required total floor length, b= α x d = 8.9*2.6= 23.14 m

Adopt total floor length = 24.00 m

The floor length shall be provided as per following proportion:

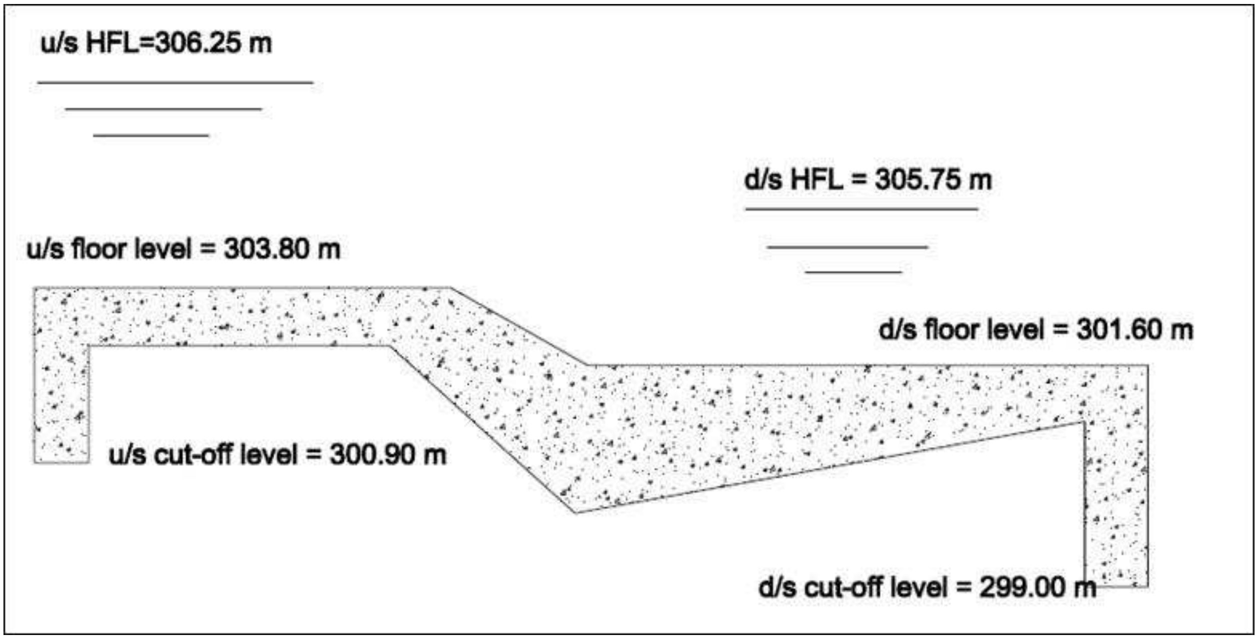
d/s horizontal floor length, = 13.00 m

d/s glacis length with 3:1 slope, 3* u/s floor – d/s floor = 3 (303.80-301.60) = 6.60 m

Balance should be provided as upstream floor = 4.40 m

Total floor length = 13.00 + 6.60 + 4.40 = 24.00 m

Figure 4-13 Design sketch of undersluice



Pressure calculations

Let the floor thickness in the u/s be 0.50 m and near the d/s cut-off be 0.50 m

Determine the uplift pressure acting on the floor and % pressures at u/s and d/s sheet piles:

% pressure at u/s sheet pile line

Upstream cutoff, d1 = 303.80-300.90 = 2.90 m

Length of floor, b = 24.00 m and u/s cutoff depth, d1 = 2.90 m

Then, 1/α =d1/b= 2.90/24.00 = 0.121

From Khosla's curves, ϕD = 88.5 %

ϕC = 83.4 %

ϕD - ϕC = 88.50 – 83.40 = 5.10 %

Apply correction to the above values,

Thickness of u/s floor, t1= 0.50 m

ϕC correction for depth = t1/d1 (ϕD –ϕC) = 0.50/2.90*5.10 = 0.88 (+ ve)

ϕC correction for interference of d/s cutoff

Khosla's formula, $C = 19\left(\frac{D}{b_1}\right)^{0.50} \times \left(\frac{D+d_1}{b}\right)$

d1=303.80-0.50-300.90 = 2.40 m

D = 303.80-0.5-299.00 = 4.30 m

b1 = 23.50 m

b = 24.00 m

C = 19*(4.30/23.50)^0.5*(2.40+4.30)/24.00 = 2.25 % (+ve)

ϕC corrected = 83.40+0.88+2.25 = 86.52 % say 86.50 %

Downstream cutoff, d2 = 301.60-299.00 = 2.60 m

Length of floor, b = 24.00 m and d/s cutoff depth, d1 = 2.60 m

Then, 1/α =d2/b= 2.60/24.00 = 0.108

From Khosla's curves, ϕE1 =15.8 %

ϕD1 = 11.0 %

ϕE1 - ϕD1 = 15.80 – 11.00 = 4.8 %

ϕE1 correction for depth = 0.60*4.80/2.60 = 1.11(- ve)

ϕE1 correction for interference due to u/s cutoff

d =301.60-0.60-299.00 = 2.00 m

D =301.60-0.60-300.90= 0.10 m

$C = 19\left(\frac{D}{b_1}\right)^{0.50} \times \left(\frac{D+d_1}{b}\right)$ =19(0.10/24.00)^0.5*(2.00+0.10/24.00) = 0.11 % (-ve)

ϕE corrected =15.80 -1.11-0.11 = 14.58 % say 14.60 %

ϕC = 86.50 % and ϕE = 14.60 %

Floor thickness

The maximum static head will occur on the floor when there is no flow in the canal but maximum flood is passing down river. In this case the maximum static head = 3.65 m

Downstream floor

At 1 m from d/s end

Unbalanced head = (86.50-14.60)/100*3.65*1.00/24.00+14.60*3.65/100 = 0.64 m

Floor thickness = 0.64 /(2.24-1) = 0.52 m

Provide floor thickness = 0.6 m at 1 m from d/s end of cut-off

Up lift pressure = 1000 * 0.64 = 642.25 kg/m2

Slab weight = 2500*0.70 = 1750.00 kg/m2

Net uplift pressure = 642.25 – 1750.00 = -1107.75 kg/m2

Calculate moment Mr = $w*l^2/12$ = -194087396 Nmm

d = (Mr/Rb*b)0.5 = 546 mm, say 600 mm

Provide 50 mm cover both side, so that total thickness of slab is = 700 mm

Floor thickness provided = 700 mm

At 13.00 m from d/s end

Unbalanced head = (86.50-14.60)/100*3.65*13.00/24.00+14.60*3.65/100 = 1.95m

Floor thickness, t = 1.95/(2.24-1) = 1.58 m

Provide floor thickness as 1.60 m at 13 m from d/s end of cut-off

Up lift pressure = 1000 * 1.95 = 1954.42 kg/m2

Slab weight, = 2500*0.30 = 750.00 kg/m2

Net uplift pressure, = 1954.42 – 750 = 1204.42 kg/m2

Mr = $w*l^2/12$ = 25092144 Nmm

d = (Mr/Rb*b)0.5 = 196 mm say 200 mm

Provide 50 mm cover both side, so total thickness of slab is = 300 mm

Floor thickness required = 300 mm at end of floor

Protection works beyond impervious floor

Upstream protection

Block protection

Scour depth, R =1.35(q2/f)1/3 = 1.35(4.80^2/1)^1/3 = 4.80 m

Anticipated scour = 1.50 * R = 1.5*4.80 = 7.20 m

Upstream scour level = u/s HFL-anticipated scour level = 306.25 – 7.20 = 299.05m

Scour depth, D below u/s floor = crest level of undersluice – scour level

D = 303.80- 299.05 = 4.75 m

Volume of block protection to be given (D) = 4.75 m3/m

Provide 1.5 m x 1.5 m x 1.0 m concrete block over 0.50 m hick gravel

The length required = 4.75/(1.0+0.50) = 3.16 m

Provide three rows of the above block in a length of 3.00 m

Launching apron

Quantity of launching apron should be 2.25 D m3/m

Thickness of launching apron = 1.50 m

The length required = 2.25*4.75/1.50 = 7.12 m

Provide launching apron as 7.00 m length

Downstream protection

Scour depth, R = 4.80 m

Anticipated scour = 2 * R = 9.60 m

Downstream scour level = HFL – retrogression – anticipated scour

= 305.75 – 0.50- 9.60 = 295.65 m

Scour depth, D below d/s floor = d/s floor level – scour level = 301.60 - 295.65 = 5.95 m, Say 6.00 m

Inverted filter

The length of inverted filter = D m and thickness equal to the d/s launching apron

Provide row of 1.50 m x 1.50 m x 1m concrete block with 10 cm gap filled with pea gravel over 0.5m thick graded filter

The length required = 5.95/(1.0+0.50) = 3.96 m = 4 m

Launching apron

Quantity of launching apron should be 2.25 D m3/m

Thickness of launching apron = 1.50 m

The length required = 2.25*5.95 /1.50 = 8.92 m

Provide launching apron as 9.00 m length

Table 4-3 Design example of weir

Descriptions	For maximum flood	
	Without concentration and retrogression	With 20% concentration and 0.5m retrogression
Discharge intensity, $q = 1.84 \times H^{3/2}$	1.95	2.34
Head required for q	1.04	1.17
d/s HFL	305.75	305.25
d/s TEL (d/s HFL + velocity head)	305.79	305.29
u/s HFL (d/s HFL + afflux)	306.25	306.25
u/s TEL (u/s HFL + velocity head)	306.29	306.42
Head loss	0.50	1.13
Post jump depth, D_2-trial value	1.325	1.687
Pre- jump depth, $D_1 = -\frac{1}{2}D_2 + \sqrt{\left(\frac{2/g \times q^2}{D_2} + \frac{1}{4}D^2{}_2\right)}$	0.35	0.33
d/s specific energy, $Ef_2 = D_2 + q^2/\left(2g \times D^2{}_2\right)$	1.44	1.79
u/s specific energy, $Ef_1 = D_1 + q^2/\left(2g \times D^2{}_1\right)$	1.94	2.92
E_{f1}-E_{f2} = Head loss (should be equal to above value)	0.50	1.13
Level of jump = d/s TEL - E_{f2}	304.36	303.51
Length of concrete floor L=5 (D_2 - D_1)	4.88	6.79
Froude no $F = q/(g*D^3{}_1)^{0.5}$	3.01	3.97

Downstream floor level is provided at the minimum level of jump formation or average bed level minus afflux = 303.30 m

d/s floor length is provided as maximum of length of floor = 6.79 m round up to 7.00 m

Depth of Cut-off from scour consideration

Discharge passing through the weir (Qd) = 33.17 m^3/s

Total waterway of weir = 17.00 m

Discharge intensity = 1.95 $m^3/s/m$

Scour depth, R = 1.35 x $(q^2/f)^{1/3}$ = 1.35*(1.95^2/1)^1/3 = 2.14 m

u/s cut-off

On u/s side allow anticipated scour as 1.1 R = 1.1*2.14 = 2.36 m

RL of bottom of scour hole = u/s HFL – anticipated scour depth = 306.25 – 2.36 = 303.89 m

Adopt RL of scour hole = 302.00 m

d/s cut-off

On d/s side allow anticipated scour as 1.25 R = 1.25* 2.14 = 2.68 m

RL of bottom of scour hole = d/s HFL- anticipated scour depth = 305.25 – 2.68 = 302.57 m

Adopt RL of d/s scour hole = 301.50 m

Total floor length and exit gradient

The exit gradient should be checked for the condition that the canal is completely closed when high flood is passing in the river; this provides worst static condition.

Maximum static head = pond level – d/s floor level =305.25-303.30 = 1.95m

Depth of d/s cutoff = 303.30-301.50 = 1.80 m

Exit gradient, $\frac{1}{\pi\sqrt{\lambda}} = G_E \frac{d}{H}$

Hence, $\frac{1}{\pi\sqrt{\lambda}}$ = 1/5*1.80/1.95 = 0.185

From Khosla's exit gradient curve, α (alfa) = 10.38

Required total floor length, b= α x d = 10.38*1.80= 18.68 m

Adopt total floor length = 18.00 m

The floor length shall be provided as per following proportion:

d/s horizontal floor length, = 7.00 m

u/s glacis length with 1:1 slope = 1* (crest level- u/s floor) = 305.25 – 303.80 = 1.45 m

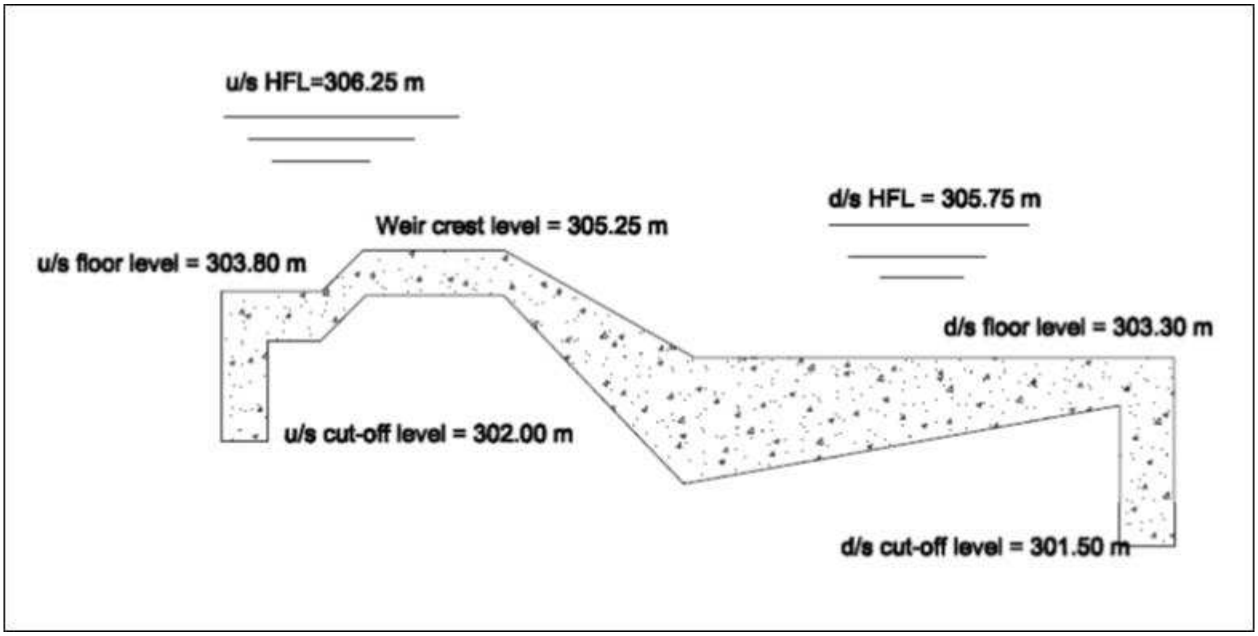
Crest width = 1.50 m

d/s glacis length with 3:1 slope, 3* u/s crest level – d/s floor = 3*(305.25-303.30) = 5.85 m

Balance should be provided as upstream floor = 2.20 m

Total floor length = 7.00 + 1.45 + 1.50 + 5.85 + 2.20 = 18.00 m

Figure 4-14 Design sketch of weir



Pressure calculations

Let the floor thickness in the upstream be 0.4 m and near the d/s cut-off be 0.40 m

Upstream cutoff, d_1 = 303.80-302.00 = 1.80m

Length of floor, b = 18.00 m and u/s cutoff depth, d_1 = 1.80 m

Then, $1/\alpha = d_1/b$ = 1.80/18.00 = 0.10

From Khosla's curve, ϕD = 88.8 %

ϕC = 84.0 %

ϕD - ϕC = 88.80 – 84.0 = 4.80 %

Apply correction to the above values,

Thickness of u/s floor, t_1= 0.40 m

ϕC correction for depth = t_1/d_1 (ϕD –ϕC) = 0.40/1.80*4.80 = 1.07 (+ ve)

ϕC correction for interference of d/s cutoff

Khosla's formula, $C = 19\left(\frac{D}{b_1}\right)^{0.50} \times \left(\frac{D+d_1}{b}\right)$

d_1=303.80-0.40-302.00 = 1.40 m

D = 303.80-0.40 - 301.50 = 1.90 m

b_1 = 17.50 m

b = 18.00 m

C = 19*(1.90/17.50)^0.5*(1.40+1.90)/18.00 = 1.13 % (+ve)

ϕC corrected = 84.0+1.07+1.13 = 86.20 %

Downstream cutoff, d2 = 303.30 – 301.50 = 1.80 m

Length of floor, b = 18.00 m and d/s cutoff depth, d1 = 1.80 m

Then, 1/α =d2/b= 1.8/18.00 = 0.100

From Khosla's curves, ϕE1 =14.70 %

ϕD1 = 10.20 %

ϕE1 - ϕD1 = 14.70 – 10.20 = 4.50 %

ϕE1 correction for depth = 0.40*4.50/1.80 = 1.00 (- ve)

ϕE1 correction for interference due to u/s cutoff

d =303.30 - 0.40-301.50 = 1.40 m

D =303.30 - 0.40-302.00= 0.90 m

$C = 19\left(\frac{D}{b_1}\right)^{0.50} \times \left(\frac{D + d_1}{b}\right)$ =19(0.90/17.50)^0.5*(1.40+0.90/18.00) = 0.54 % (-ve)

ϕE corrected =14.70 -1.00-0.54 = 13.16 % say 13.20 %

ϕC = 86.20 % and ϕE = 13.20 %

Floor thickness

The maximum static head will occur on the floor when there is no flow in the canal but maximum flood is passing down river. In this case the maximum static head = 305.25 – 303.30 = 1.95 m

Downstream floor

At 1 m from d/s end

Unbalanced head = (86.20-13.20)/100*1.95*1.00/18.00+13.20*1.95/100 = 0.34 m

Floor thickness = 0.34/(2.24-1) = 0.27 m

Provide floor thickness = 0.40 m at 1 m from d/s end of cut-off

Up lift pressure = 1000 * 0.34 = 335.70 kg/m^2

Slab weight = 2500*0.70 = 1750.00 kg/m^2

Net uplift pressure = 335.70 – 1750.00 = -1414.30 kg/m^2

Calculate moment Mr = w*l^2/12 = -247798530 Nmm

d = (Mr/Rb*b)^0.5 = 617 mm, say 600 mm

Provide 50 mm cover both side, so that total thickness of slab is = 700 mm

Floor thickness provided = 700 mm

At 7.00 m from d/s end

Unbalanced head = (86.20-13.20)/100*1.95*7.00/18.00+13.20*1.95/100 = 0.81 m

Floor thickness, t = 0.81/(2.24-1) = 0.65 m

Provide floor thickness as 0.80 m at 7.00 m from d/s end of cut-off

Up lift pressure = 1000 * 0.81 = 810.46 kg/m2

Slab weight, = 2500*0.30 = 750.00 kg/m2

Net uplift pressure, = 810.46 – 750.00 = 60.46 kg/m2

Mr = w*l^2/12 = 1259587 Nmm

d = (Mr/Rb*b)^0.5 = 44 mm say 200 mm

Provide 50 mm cover both side, so total thickness of slab is = 300 mm

Floor thickness required = 300 mm at end of floor

Protection works beyond impervious floor

i. *Upstream block protection*

Scour depth, R =$1.35(q^2/f)^{1/3}$ = 1.35(2.14^2/1)^1/3 = 2.14 m

Anticipated scour = 1.5*R = 1.50* 2.14 = 3.21 m

u/s scour level = u/s HFL – anticipated scour depth = 306.25 – 3.21 = 303.04 m

Scour depth, D below u/s floor = u/s floor level- scour level = 303.80 – 303.04 = 0.76 m

Volume of block protection to be given = D m^3/m = 0.76 m^3/m

Provide 1.0 m x1.0 mx 0.30 m block over 0.20 m thick gravel

The length required = D/(block thickness +gravel thickness) = 0.76/0.50= 1.53 m

Provide two rows of the concrete block in a length of 2.00 m

ii. *U/s launching apron*

Quantity of launching apron for 2:1 slope be 2.25 D m^3/m

Thickness of launching apron = block thickness + gravel thickness = 0.50 m

The length required = 2.25*0.76 / 0.50 = 3.44 m

Provide launching apron of 3.00 m length

iii. *D/s block protection*

Scour depth, R = 2.14 m

Anticipated scour depth = 2*R = 4.29 m

Downstream scour level = HFL- retrogression – anticipated scour depth

= 305.75 - 0.50 - 4.29 = 300.96 m

Scour depth, D below d/s floor = d/s floor level – scour level = 303.30 - 300.96 = 2.34 m

iv. *D/s Inverted filter*

The length of inverted filter = D meter and thickness equal to the d/s launching apron

Provide in row of 1.00 m x 1.00 m x 0.60 m concrete block with 10 cm gap filled with pea gravel over 0.40 m thick graded filter in a length of 2.00 m

v. *D/s launching apron*

Quantity of launching apron should be 2.25 D m^3/m

Thickness of launching apron = 1.00 m

The length required = 2.25 * D /1.00 = 2.25* 2.34 = 5.26 m

Provide launching apron 6.00 m in length

4.8 Energy Dissipation in Hydraulic Structure

4.8.1 Energy dissipation

The energy of water flowing over weirs/spillways needs to be absorbed or dissipated before it is returned to the river or canal where unprotected or soft foundations exist. The objective of energy dissipation is thus to destroy the energy of a mass of water so that it does not create serious erosion. The devices used for dissipating the energy of flowing water are called energy dissipaters. The dissipation of energy is performed by transforming super-critical flow into sub-critical flow through hydraulic jumps, turbulence, and impacts. The most widely used energy dissipaters in irrigation works are (i) USBR basins (type-I to type-IV), (ii) Vlugter basin, (iii) SAF Basin, (iv) Impact block (straight drop) basin, (v) Slotted grating dissipater, (vi) Impact type stilling basin, (vii) Plunge pool, (viii) Stilling well, (ix) Baffled spillway, and (x) Deflector bucket. Among these energy dissipaters (i) to (v) and (x) are mostly used in spillways, whereas types (vi) to (ix) are used in outlets. The deflector buckets are suitable for use in headwork structures, but not generally used in canal works. The detailed sketches of these energy dissipaters are presented in *Figure **4-15*** . In this section, only the hydraulic jump basins are discussed further.

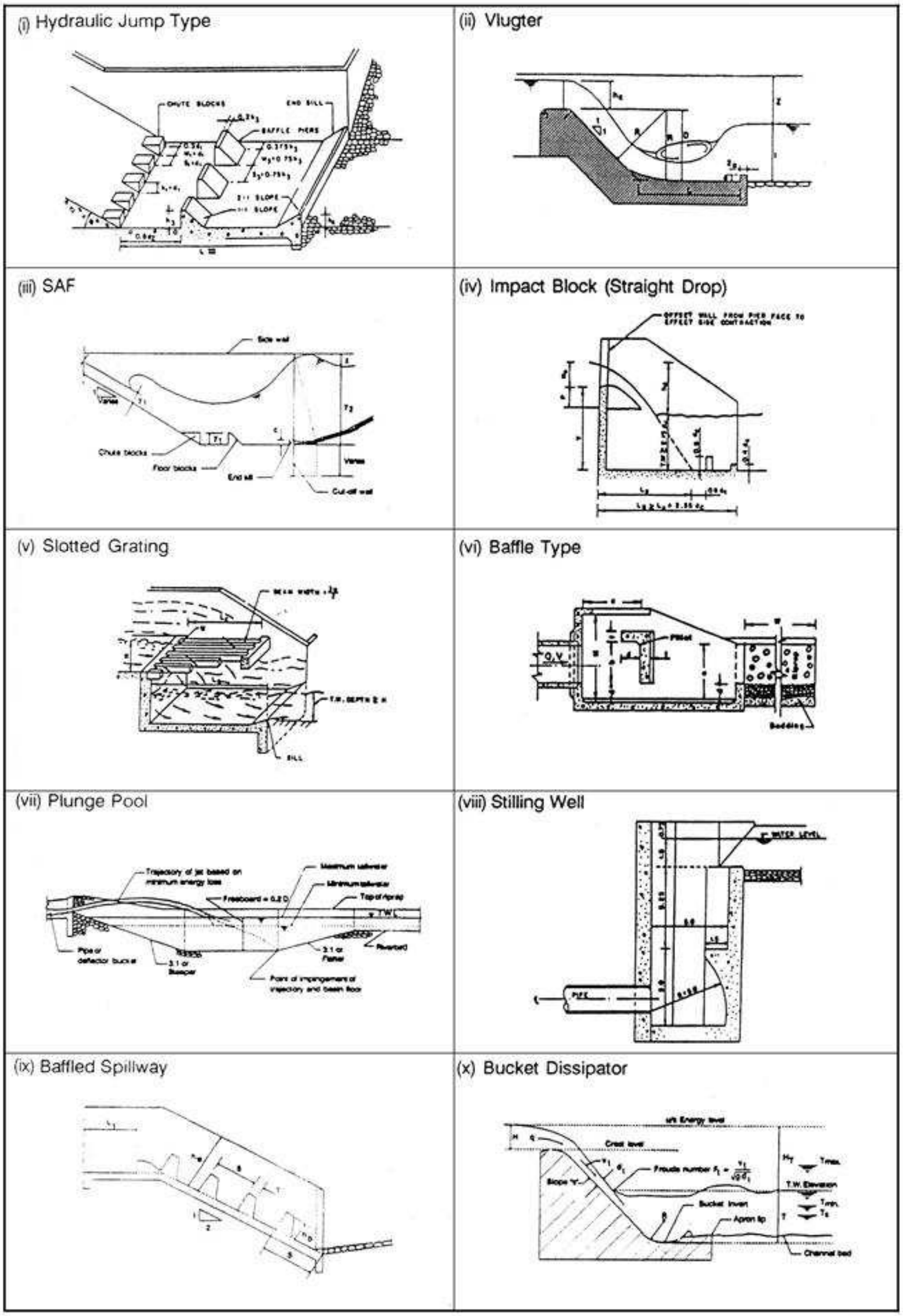
4.8.2 Hydraulic jump basins

Hydraulic jump basins are the most effective devices to dissipate the energy of flowing water and are widely used in all types of hydraulic structures. The form and flow characteristics of the hydraulic jump which occurs in a stilling basin can be related to the Froude number ($F = V/ (gd)^{0.5}$) of the flow entering the basin. The selection of particular energy dissipater depends on the Froude number, type of structure and economy, which is illustrated in tabular form (*Table **4-4***).

Table 4-4 Selection of energy dissipaters

Type	Consideration/Application limits
USBR Type I	Wide range of Froude number and velocity (V< 15 m/s) Long basin and simple construction, specially suitable for Fr < 2.5
USBR Type II	For V > 15 m/s Long basins with chute blocks
USBR Type III	For Fr > 4.5, V < 15 m/s Short basin, but complicated by floor and chute blocks
USBR Type IV	For Fr between 2.5 - 4.5, V < 15m/s Short basin, but complicated by floor and chute blocks
Vlugter	For canal falls where drop < 4.0 m Simple to design and construction, suitable for masonry structure
SAF	For small low head structures Short basin, but complicated by floor and chute blocks
Impact block	For drop < 2.0 m Generally economic, suitable for canal falls
Slotted grating	For Fr between 2.5 - 4.5 Small structure, but complicated by grating
Baffled spillway	No tail water requirement Complicated by blocks, economic
Bucket	Needs more tail water than jump type basins Good foundation and protection required in view of scour hole formation

Figure 4-15 Sketches of different types of energy dissipaters



4.9 Pumped Irrigation System

4.9.1 Introduction

Small scale pumped irrigation systems are made up of the following components:

- Water source,
- Pump and power unit,
- Distribution system and
- Method of irrigation

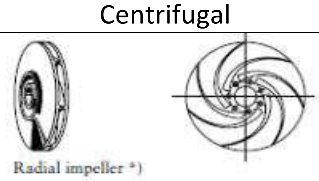
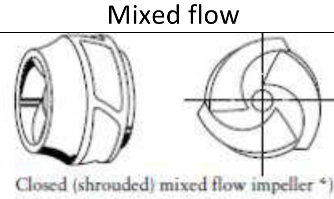
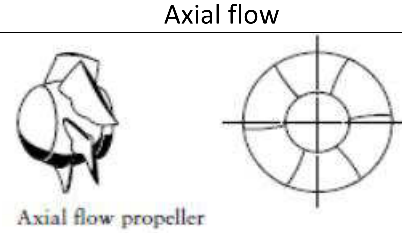
The water source, distribution system and irrigation method determine the energy demand, while the pump and power unit supply the power. The water source may be a river, lake or groundwater. The amount of water abstracted and the height to which it must be lifted from the river or borehole add to the energy demand. Pumps are mostly driven by electric motors in Bhutan.
The distribution system conveys water from the pump to the farmer's field and consists of open channels or pipes. Some systems are a combination of open channel and pipes. The choice of distribution system has a significant effect on energy demand. The irrigation method may be surface irrigation, sprinkler or drip irrigation. Surface irrigation may be supplied either by a pipe or open channel. Sprinkler or drip irrigation systems would normally use piped distribution systems.
The most common combinations of the components for an irrigation system are:

- Pump open channel surface irrigation
- Pump pipe supply surface irrigation
- Pump pipe supply sprinkler or drip irrigation

4.9.2 Type of pumps

A pump is a device to lift water to a higher level or to transport it from one point to another. Pumps require some power to perform these works which is supplied by a driving device (electric motor, internal combustion engine) through a medium of a transmission. Hence, a pumping set consists of a power supplier, a transmission and a pump.
Among various types of pumps to lift water, Turbo type pumps comprising centrifugal, mixed flow and axial flow pumps are most frequently used in pumping works for water supply, irrigation, and drainage services. These pumps all function on the principle that the energy represented by head is imparted to the water pumped through the rotational motion of an impeller. The principle of a pump's functioning is represented by Euler's equation.

Figure 4-16 Sketches of Pumps

Centrifugal	Mixed flow	Axial flow
		

In a radial flow (centrifugal) pump, the water leaves the impeller in planes perpendicular to the shaft axis due to centrifugal force. The water then is guided through a volute casing into the discharge nozzle, while a part of the velocity head is converted into the static head. In some cases diffusers are arranged around the impeller.

In a mixed flow pump, the water leaves the impeller in a direction diagonal to the shaft axis on conical surfaces. The water leaving the impeller is either collected by a volute casing or received by a diffuser casing.

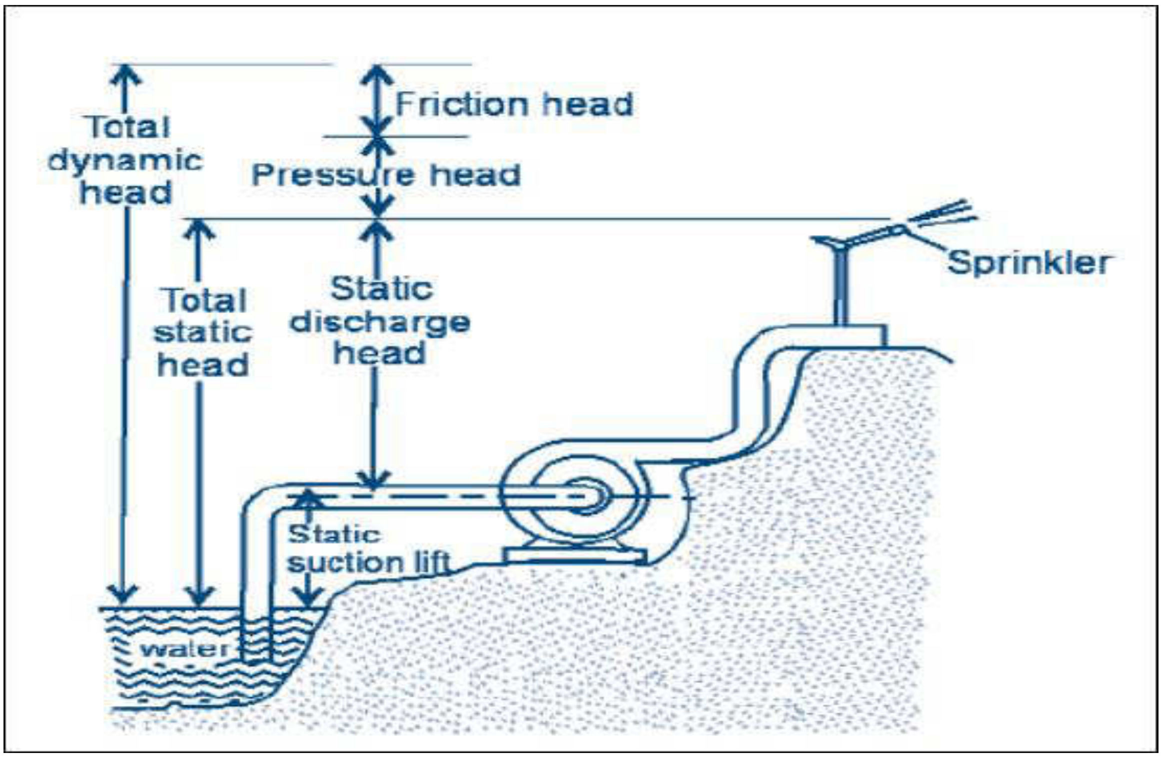
In an axial flow pump, the water flows through a propeller type impeller on cylindrical surfaces and a diffuser casing is exclusively used for restoring pressure.

Most of the pumps are coupled with electric motors or engines being driven from the pump shaft end. Submersible motor pumps are driven by motors integrated into the pump casing and immersed in the suction sump or deep wells. These pumps are exclusively vertical shaft types featuring ease in handling when used for portable applications.

4.9.3 Total Dynamic Head

A total dynamic head or total pumping head is the head that the pump is required to impart to water in order to the head requirement of the particular system. The total head is made up of static suction head, static discharge head, required pressure head, and friction head (*Figure **4-17***).

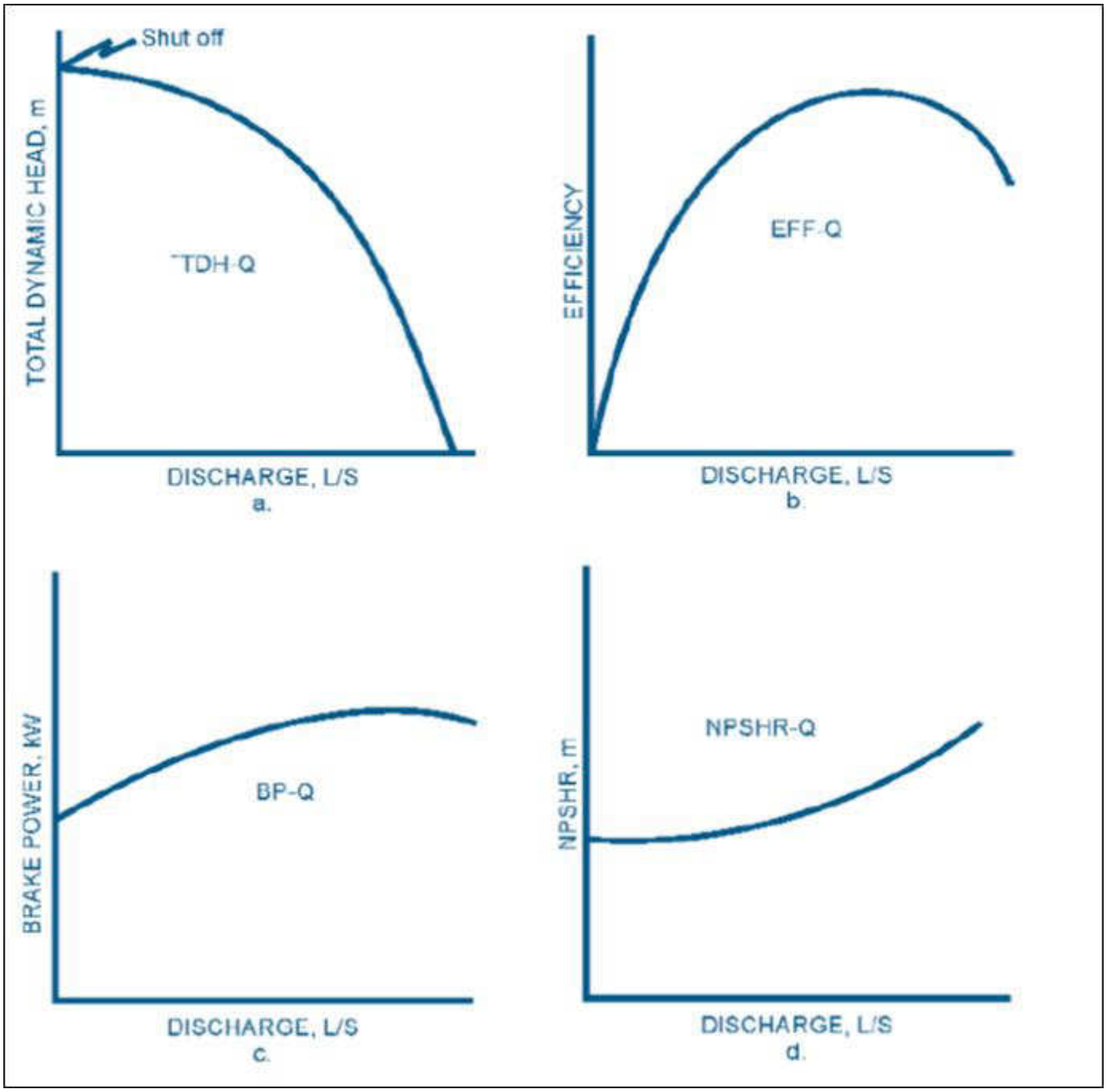
Figure 4-17 Sketch of total dynamic head



4.9.4 Pump Characteristic Curves

Most manufacturers provide four different characteristic curves for every pump: the total dynamic head versus discharge (TDH- Q) curve, the efficiency versus discharge (Eff-Q) curve, the break power versus discharge (BP-Q) curve, and net positive suction head required versus discharge (NPDH-Q) curve. All four curves are discharge related (*Figure **4-18***).

Figure 4-18 Sketches of pump characteristic curves



Total dynamic head versus discharge curve relates the head to the discharge of the pump. This curve shows that the pump can provide different combinations of discharge and head. When the head increases the discharge decreases or vice versa.

Efficiency versus discharge curve relates the pump efficiency to the discharge. The efficiency is defined as output work over input work.

$$E_{pump} = \frac{Output\ work}{Input\ work} = \frac{WP}{BP} = \frac{Q \times TDH}{C \times BP}$$

Where E_{pump} is the pump efficiency,

BP is the break power in kw or HP (1kW=1.36 HP) the energy imparted by the prime mover to the pump,

WP is the water power in kw, the energy imparted by the pump to the water,

Q is the discharge in m^3/s hour or l/s,

TDH is the total dynamic head in m, and

C is the coefficient to convert work to energy unit and equals to 102 if Q is measured in l/s and 367 if Q is measured in m^3/ hour.

Break power versus discharge curve relates the input power required to drive the pump to the discharge.

The net suction head required versus discharge: Some of the atmospheric pressure at water level is lost in the vertical distance to the eye of the impeller, some to frictional losses, and some to the velocity head. The total energy that is left at the eye of the impeller is called the Net Positive Suction Head (NPSH) and the required energy to move water into the eye of the impeller is called the Net Positive Suction Head Required (NPSHR). It is the function of the pump speed, the shape of the impeller and the discharge. In order to avoid cavitation of the pump, available NPSH should be greater than NPSHR.

Manufacturers also provide the combined characteristic curves for total head, shaft power and efficiency plotted on the ordinates against capacity on the abscissa. The forms of characteristic curves are closely related to the values of the specific speed in which respective characteristics are given in percentage of the value at the best efficiency point.

4.9.5 Power and Energy for Pumping

In general, the power for the pump set is electricity obtained either from the central power grid system or generated from its own source. Nominal operating condition of the power supply is 400 Volt, 3 Phase A.C. with 50 Hz frequency. The amount of energy required to lift the water depends on the volume to be lifted and the head required:

$$E = \frac{V_w \times H}{367}$$

Where E is the energy in kilowatt hour (kWh), V_w is the volume of water in m^3, and H is the head in m,

Power is the rate of using energy and is commonly measured in kilowatt or horsepower (1kW =1.36 HP). Power is calculated as:

$$P = \frac{E}{t}$$

Where P is the power in watt, E is the energy in watt hour and t is the time in hour. From the energy and power equations discharge relation is obtained as:

$$P = 9.81QH$$

Where Q is the discharge in liter per sec, and H is the total head in m

4.9.6 Pumps in series

Connecting pumps in series applies to the cases where the same discharge is required but more head is needed than that one pump can produce. For two pumps operating in series, the combined head equals the sum of the individual heads at a certain discharge.

4.9.7 Pumps in parallel

Pumps are operated in parallel when for the same head, variation in discharge is required. A typical example would be a smallholder pressurized irrigation system with many users.

4.9.8 Selection of pumps

A wide range of pumps are available on the market. The selection of the right pump for the right use depends on a series of inter-related factors. Centrifugal pumps have the largest application in irrigation compared to other type of pumps. The most commonly used centrifugal types of pumps are horizontal end suction, vertical submersible and vertical turbine pumps. Mixed or axial flow impellers are sometimes used in low lift, high capacity operations like lifting water from a canal. In general practice, the selection of the pumps requires the use of manufacturers' characteristics curves.

Prior to selecting the pumping system, it is essential to calculate total head over a number of stages and finalize the complete layout of the system. All components of the system including valves, bends, manifolds, reducers, etc. may be incorporated to evaluate total head for each stage of pumping, summing up static head and friction head. A market survey for various types of pumps and their availability will help to identify the cost-effective pump for a given site. The selection of the prime mover governs the selection of a pump, whether it is mechanically driven or electrically driven. In our case, we use electrically driven pumps. In the case of electrically driven pumps, directly coupled AC motors are generally available in the market. While selecting a pump, try to ensure that the power is adequate for the required discharge and head.

Centrifugal pumps are the most commonly used engine-powered pumps for small-scale irrigation. They are mainly designed to provide relatively high head and low discharge for a given power. Some sort of compromise has to be accepted between required discharge, total head and geographical location.

4.9.9 Siting of pumping station

The pumping station is a building which provides a foundation for the pumping set and its accessories and protects them from adverse weather conditions. It also houses the electrical and other auxiliary equipment and supports suction and discharge piping. The selection of a suitable location for a pumping station is very important in irrigation development. Several factors have to be considered while selecting the pumping site.

- Reliability of the water to match with the proposed cropping pattern of the command area,
- Outside the flood level in case of river water abstraction,
- Adequate water depth for suction pipe

4.9.10 Selection of Pipes and Fittings

Pipes are an integrated part of a pumping system. In any lift scheme, the selection of proper pipes is as important as that of the pump. Not only do pipes represent a major part of the initial investment, their improper selection will adversely affect operation and maintenance costs. Thecomplete failure of a scheme can result from improper pipe selection.

In selecting pipes, consideration must be given to their diameter, material and cost, keeping in view the working condition and availability of such pipes on the market. The basic considerations that finally lead to the selection of pipes are discussed below:

- Working pressure,
- Water hammer - introduce break pressure tank or surge tank,
- Internal pressure - adequate pipe thickness,
- External pressure - consider traffic load, earth pressure,
- Friction in pipes - small friction factor,
- Quality of water- silt free water,
- Cost of pipe - low cost,
- Availability

4.9.11 Main civil works for pumped irrigation

Intake

Intake is necessary to guide the water towards the jack well or sump well. It is also necessary to maintain the water level fluctuations in the river.

Jack well/Sump well

The water diverted from the river is collected in a jack well or sump well from where it is pumped to the irrigation command area. The term jack well is used when the pump house is located directly over the well. In such cases, vertical turbine pumps are mostly used. When the pump house is located on the ground close to the well the term sump well is used. For permanent lift irrigation schemes, properly designed pumping wells are provided for installing the pumping sets.

Sump wells are constructed on the banks of the river to protect pumping units from flood water. They are usually used to pump water using centrifugal pumps from rivers and streams in which the flood levels are not high and the water level in the river is within the suction lift of centrifugal pumps.

Pump house

The permanent pump house can be provided if the water level in the river is more or less constant throughout the year. Generally the pump house is rectangular in shape. The pumps, motors, panel box for electrical components, starters and other accessories are located in the pump house separately. The dimensions of the pump house are based on the specific requirements of the installation. This may vary from site to site depending upon the stream hydrology also.

Delivery Chambers/discharge tank

The delivery chamber or discharge tank is necessary at the head of the distribution system. Generally it can be designed for one or two minute retention capacity of the incoming flow.

4.9.12 Design procedure of pumped irrigation

i. Assess the design discharge of the system based on peak water requirements and the command area,
ii. Ascertain the water availability at the source, which should have adequate flow to meet the crop water requirements,
iii. Draw the proposed pumped irrigation layout mentioning intake, sump well, pump house, pipeline alignment, discharge tank, etc,
iv. Draw the longitudinal profile of the system based on the layout plan mentioning bed level of the source, high flood level, pump house level, critical points along pipe alignment, and discharge tank,
v. Calculate the length of the pumping mainline and level difference between the water level at source and the discharge tank,
vi. Determine the number of pumping units, operating hours per day and design discharge of each pump,
vii. Determine the stages of the pumping system based on the available head and capacity of the pump,
viii. Calculate economic size of pumping main from Lea's formula,

$$D = 1.22 \times \sqrt{Q}$$

Where Q is the design discharge in m^3/s,

ix. Adapt nearest pipe diameter available in the market,

x. Calculate velocity of flow in the pipe, $V = \dfrac{4Q}{\pi D^2}$

Where Q is the design discharge and D is the pipe diameter,

xi. Adopt economic diameter of pipe considering permissible velocity for the particular type of pipe,

xii. Calculate total head loss in pipe flow,

Total head loss = friction head loss + bend loss + entrance loss + exit loss

Friction head loss from Darcy-Weisbach formula, $h_f = f\dfrac{L \times V^2}{D2g}$

Where hf is the fiction head loss in m,

f is the friction coefficient and can be calculated using various formulas,

L is the pipe length in m,

D is the pipe inside diameter in m,

V is the velocity of water in m/s, and

g is the acceleration due to gravity = 9.81 m/s^2

Colebrook White equation for friction factor, $\dfrac{1}{\sqrt{f}} = -\log\left(\dfrac{e/D}{3.70} + \dfrac{2.51}{N_R\sqrt{f}}\right)$

Swamee-Jain equation for friction factor, $f = \dfrac{0.25}{\left(\log\dfrac{e/D}{3.70} + \dfrac{5.74}{N_R^{0.90}}\right)^2}$

Where e is the roughness height of pipe material,

D is the diameter of the pipe, and

N_R is the Reynolds number, $N_R = \dfrac{DV}{\lambda}$, where λ is the viscosity of the water

xiii. Calculate total dynamic head for pumping,

Total dynamic head = Level difference between source and discharge tank + Head loss + Suction head,

xiv. Select the type of pump to be installed in the system (centrifugal, submersible),

xv. Select the pump capacity based on the manufacturer's characteristic curves,

xvi. Based on the pump model and its power calculate the efficiency of the pump,

$E_{pump} = \dfrac{WP}{BP}$

Where WP is the theoretical power,

BP is the break horse power based on the pump model,

xvii. Calculate the water hammer pressure at the delivery pipe,

$$P_h = \frac{V}{\sqrt{\dfrac{g}{w}\left[\dfrac{1}{K} + \dfrac{d}{t.E}\left(1 - \dfrac{1}{2m}\right)\right]}}$$

Where V is the velocity of flow in pipe in m/s,

g is the acceleration due to gravity in m /s^2,

w is the specific weight of the water in $N/m^3 = 9810\ N/m^3$,
K is thee Bulk modulus of water in $N/m^2 = 2060\ N/m^2$,
d is the diameter of pipe in m,
t is the thickness of the pipe in m,
E is the Young's modulus of elasticity of pipe in kN/m^2,
1/m is the Poisson ratio,

xviii. Compare the water hammer pressure with the allowable pressure of pipe material,

4.9.13 Design example of Pumped irrigation system

Design data:

Command area	10 ha
Source of water	Perennial River
Minimum water level at the source	505.00 m
Maximum water level at the command area	640.00 m
Length of mainline profile	320 m

Calculations

The peak water requirement be 1.25 l/s per ha
The total water requirement be 10* 1.25 = 12.50 l/s
Assume the pump shall run for 12 hours, then peak discharge = 12.50*2 =25 l/s
Let us provide three pumps in parallel; two for operation and one for reserve, then discharge of a single pump unit be, q = 12.50 l/s
Calculate economic size of pumping main from Lea's formula, $D = 1.20\sqrt{Q}$ = 1.2*√0.0125 = 136.4 mm
Available size of GI pipe = 125 mm
Calculate head loss for pipe diameter 125 mm and length of 320 m
Velocity in pipe, $V = \frac{Q}{A} = \frac{4Q}{\pi D^2}$ = 4*1000/ (3.14*125^2) = 1.020 m/s
Using Darcy-Weisbach equation head loss, $hf = \frac{f\,L}{D} \times \frac{V^2}{2g}$
Where f is coefficient of friction which is calculated from Moody's diagram
For GI pipe absolute roughness, e= 0.15 mm
L is the length of pipe, L= 320 m
D is the diameter of pipe, D= 125 mm
V is the velocity in pipe, V= 1.02 m/s
From Moody's diagram f= 0.036
Then, hf = 0.036*320/0.0125 *1.02^2/2*9.81 = 4.88 m
Take bend and join losses as 10 % of the frictional loss, the total loss, h_f= 4.88*1.1= 5.36 m
Total head for pumping = level difference + head loss + suction head
Let the depth of sump well be at 2.0 m and minimum water level below the sump well be 1.20 m
Then total head = (640.00-505.00) + 5.36 + 2.00+ 1.20 = 143.56 m
Based on the discharge to be pumped and dynamic head, select pump from manufacturer's characteristic curve,
Select KSB or equivalent pump with model no: UPHA 293/8 + HBC
Power of pump = 30 kW = 40 HP,

Discharge 45 m^3/hour for total head of 144 m,
Total length of pump = 2.36 m,

Calculate the efficiency of the pump, $\eta = \frac{Q\rho g H}{P}$ = 0.0125*9.81*143.56/30 = 58.60%

Calculate the water hammer pressure at the delivery pipe,

$$P_h = \frac{V}{\sqrt{\frac{g}{w}\left[\frac{1}{K} + \frac{d}{t.E}\left(1 - \frac{1}{2m}\right)\right]}}$$

Where V is the velocity of flow in pipe = 1.02 m/s,
g is the acceleration due to gravity = 9.81 m /s^2,
w is the specific weight of the water = 9810 N/m^3,
K is thee Bulk modulus of water in N/m^2 = 2060 N/m^2,
d is the diameter of pipe = 125 mm,
t is the thickness of the pipe = 5.4 mm,
E is the Young's modulus of elasticity of pipe = 210 kN/m^2,
1/m is the Poisson ratio = 0.27

Ph = 1.02/sqrt(9.81/9810(1/(2060*1000000)+0.0125/0.0054*(210*1000000*1000)(1-0.27*0.50))/1000*1000 = 1.34 N/mm^2, = 13.62 kg/cm^2

Total pressure = static pressure + water hammer pressure,
Static pressure =(head difference+ depth of sump well +water level below sump well)/ acceleration due to gravity =(135+2.0+1.2)/9.81 = 138.20/9.81 =14.08 kg/cm^2
Total pressure = 14.08 + 13.62 = 27.70 kg/cm^2
GI pipe with heavy duty can withstand this pressure,
Hence, the design is acceptable.

CHAPTER-5

5. Canal System Design

5.1 Introduction

Before the start of the survey and design, the canal system has to be planned. Canal system planning includes deciding what types of canals and drains are required and what type of system to use. It also comprises deciding how the system will be operated and what control structures will be required to distribute water up to the farmer's field. In addition, system planning involves laying out the system of canals and drains on suitable base maps. Hence, an irrigation system comprises the canals to bring water to the fields and the drains to take away the excess irrigation or rainfall water.

5.2 Type of Canals

Based on their hierarchy, canals are named the main canal, branch canal, tertiary canal and field canals. The main canal takes off water from the headwork or intake while a branch canal takes off water from the main canal. Similarly, a tertiary canal takes water from the branch canal and feeds field canals.

Based on the alignment, the canals are categorized as a contour canal and a ridge canal. A contour canal is aligned along the contours and provides irrigation on the valley side only. A ridge canal is aligned along the ridge and provides irrigation on both sides. Generally, main canals are contour canals and branch canals are ridge canals.

Based on the shape of the cross sectional area, the canals are named as trapezoidal canal, rectangular canal, triangular canal, and circular canal. The shape of the canal depends upon the capacity, nature, topography, and purpose of the canal.

Based on the materials in the section boundary, the canals are lined canal and unlined canals. The choice of lined or unlined canal depends upon the type, nature, purpose and development cost of the scheme. Generally lined canals are designed to reduce the seepage from the canal section along its conveyance. The types of lined canals and their relevance are presented in separate sections.

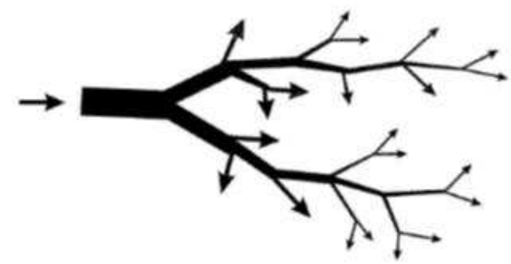
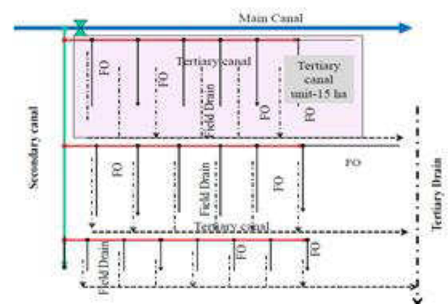
5.3 Layout planning

The layout plan is the arrangement of canals, drains, and structures required to achieve water distribution and remove excess water from irrigation fields. The layout planning establishes the optimum arrangements, positions, levels of canals and structures. It also considers physical, technical, and social constraints of the specific site. Generally two types of canal layouts exist (***Figure 5-1***):

Bifurcating layout: In bifurcating system water is divided among two or three large group of canals, subdivided into two or three small group of canals and so on. This type of layout system exists in community managed systems in valleys and plains.

Hierarchical layout: In hierarchical system water is divided into large blocks and then after sub-divided into smaller units. Generally, this type of layout is existed in modern irrigation systems. The layout of canals and drains fallow ridges and valleys respectively.

Figure 5-1 Canal Layouts

Bifurcating layout	Hierarchical layout
	

In small hill irrigation schemes like in Bhutan, the following two types of layout systems are suitable for planning purpose (*Figure 5-2*):

a. The first system has a primary or main canal with a series of direct off-takes along the main canal. This type of canal layout existed in the community managed irrigation systems;

b. The second type of system has small and medium sized off takes along the main and secondary canals. The number of secondary canals depends upon the size of the irrigation scheme.

Figure 5-2 Small Scale Canal Layouts

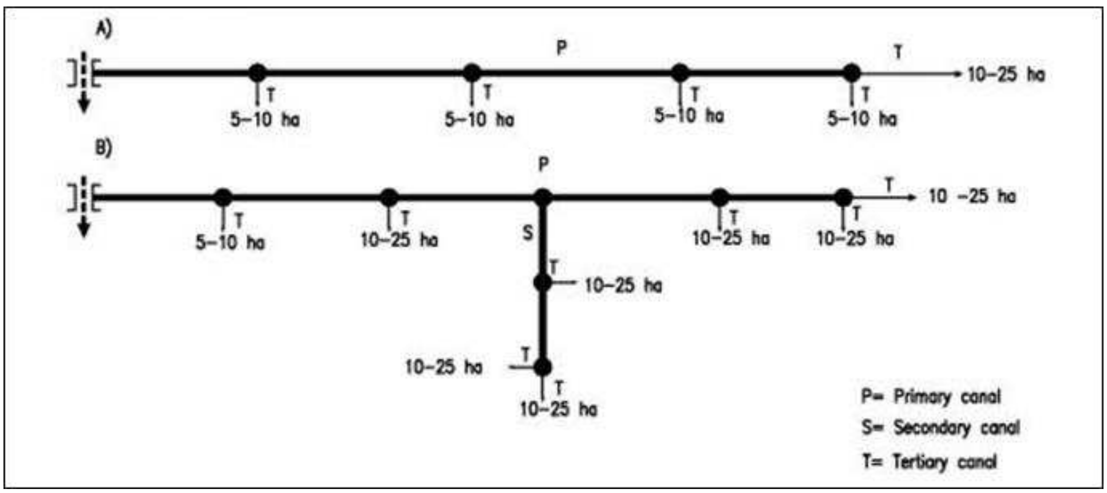


Figure 5-2 shows that in the type-A layout, all of the off takes are located along the main canal while in the type-B layout, secondary canal or branch canals also exist. The layout planning of a canal system involves three key parameters:

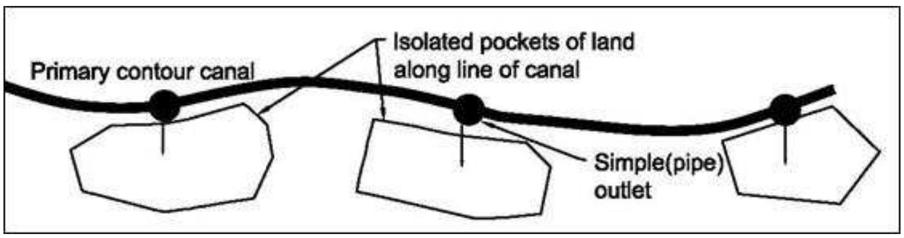
- Linkage of command area and intake,
- Alignment of the primary canal, and
- Type of canal layout

The location of the intake is the key determinant of the command area and in many cases will set the limit of the area to be irrigated. Establishing the relationships between the intake/headwork site and the command area in terms of levels and water quantities involves a trial and error approach.

Route selection of a canal depends on topography, geological conditions along the alignment, canal type, type and size of irrigation project. Primary canals in the hills follow the contours and pass through ridges, valleys, deep gullies, natural drains, land slide zones and vertical to moderately sloped terrain. In selecting the alignment of the primary canal, it is essential to establish the geological features along the alignment, and define the stable and unstable portions. The route should be modified if unstable and difficult features are observed. If the alignment has to pass such features, appropriate solutions need to be adopted.

Type of canal layout also decides the system planning. The major layout systems being adopted in hilly environment are discussed above. However, there may be an alternative layout system applicable in hilly terrain like Bhutan (***Figure 5-3***). This type of layout feeds the isolated pockets of the command area along the primary canal.

Figure 5-3 Alternative Layout for Small Scale Canals



5.4 Design Concept of Irrigation Canals

In general principle, irrigation canals lie along ridges and drains lie in valleys. The design of irrigation canals involves the following steps:

- Draw the canal layout,
- Measure scheme command area,
- Determine canal capacity or design flow,
- Prepare schematic diagram of canal system,
- Design canal parameters for each reach of the canal system,
- Work out canal water levels at critical locations,
- Draw the preliminary canal long profile,
- From structures design finalize canal water levels,
- Draw up final canal long profile,
- Draw up canal cross sections

For large irrigation systems, canal capacity design starts from the field level to main canal and up to the intake. This design approach involves calculations of water requirements at the field level and capacities of their feeder canals summing up to the main canal. For small scale irrigation, the design of canal capacity is carried out by:

Q = NCWR x CCA/8.64

Where, Q – design capacity of canal at the intake in lps,

NCWR – net crop water requirement in mm/day,

CCA – cultural command area in ha,

If crop water requirement is not assessed, the capacity of a canal can also be determined by the duty of irrigation water or depth of irrigation water for the main crops to be planted.

Q = Duty x CCA

Where, duty or water requirement in lps/ha,

Example:

Duty = 3 l/s/ha and command area is 20 ha,

The capacity of canal (Qd) = 3 x 20 = 60 l/s

Prior to starting the canal system design along with the structure design, a schematic diagram of the canal scheme is drawn indicating canal segments and lengths, major structures, and the command area of the main branch canals and direct off-takes. Based on the schematic diagram of the proposed canal system (*Figure **5-4***) canal parameters are designed for each reach. The canals cross sectional parameters are bed width, water depth, side slope, roughness coefficient, and free board. The longitudinal slope of the canal is determined based on the size of the canal and topography of the alignment.

5.5 Design of Unlined Canals

5.5.1 Design Procedure

The design of unlined canals is carried out using various methods. For small irrigation schemes in the hills, it is appropriate to use Manning's formula. The design procedure of the canal follows a series of steps:

i. Determine the design discharge, Q,

ii. Determine the design side slope, m (1 vertical: m horizontal),

iii. Determine canal longitudinal slope, S,

iv. Determine bed width to depth ratio, r (r = B/D; B being bed width and D being water depth),

v. The calculation shall be carried out by trial and error method,

vi. Assume bed width of the canal B,

vii. Calculate depth of water depth, $D = r \times B$

viii. Calculate cross sectional area of flow, $A = BD + mD^2$

ix. Calculate wetted perimeter of flow, $P = B + 2D\sqrt{(1 + m^2)}$

x. Calculate hydraulic radius, $R = \frac{A}{P}$

xi. Calculate velocity of flow using Manning's formula, $V = \frac{1}{n} R^{2/3} \times S^{1/2}$

xii. Calculate discharge of canal using continuity equation, $Q = A \times V$

xiii. Check the calculated discharge and compare with designed one, Qc>Qd

xiv. Compare the calculated velocity with limiting velocity, $V_c < V_{per}$

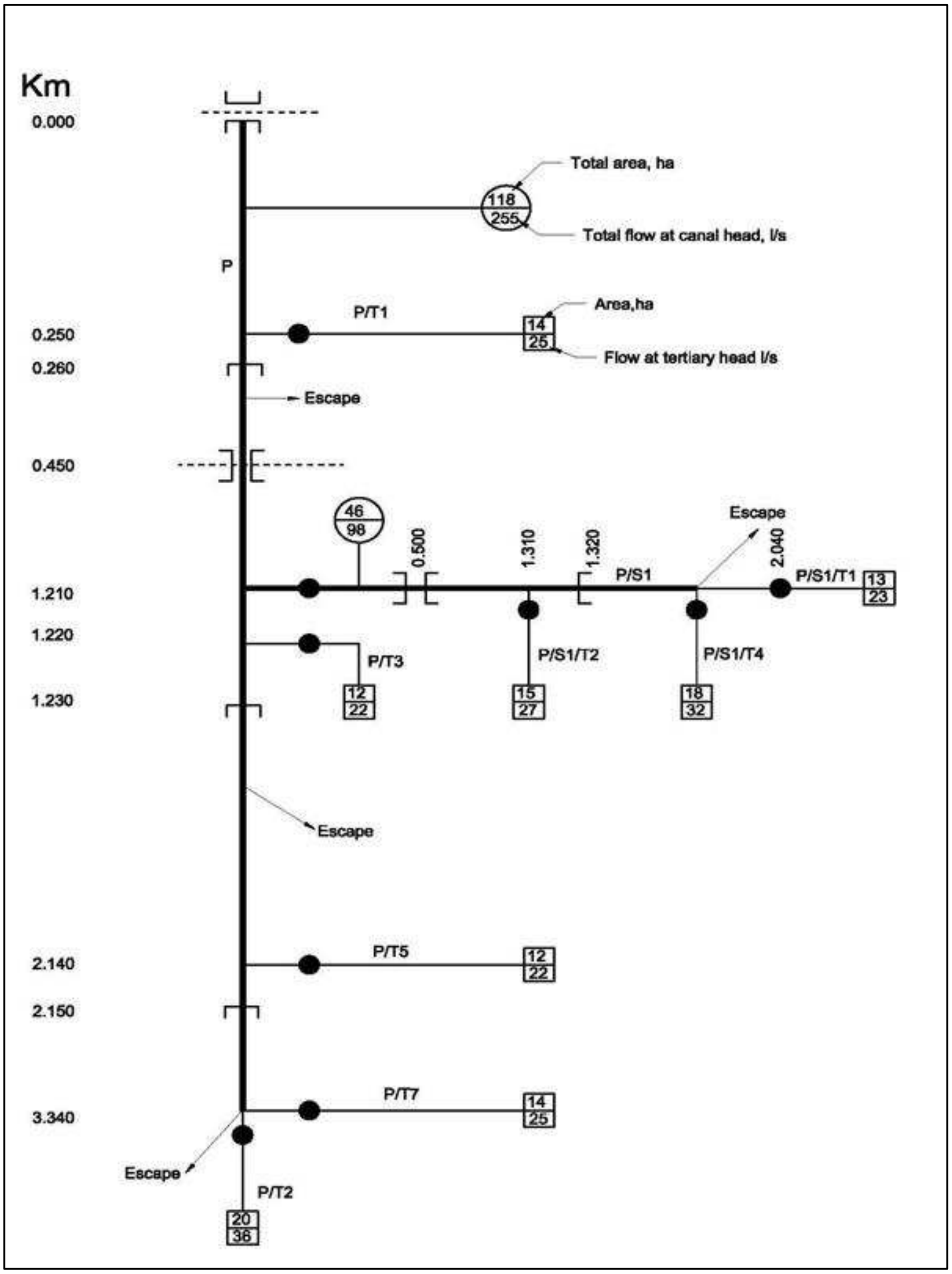
xv. Calculate tractive force, $T = C \times W \times R \times S$

Where, C is the coefficient equal to 1 for bed and 0.76 for sides, W is the specific weight of water in N/m^3, R is the hydraulic radius of the canal, and S is the slope of the canal,

xvi. Compare calculated tractive force with permissible values according to their material,

Excel sheets are required to design various sections of the canal. The design parameters of a canal are presented in the sections below.

Figure 5-4 Schematic Diagram of Canal System



5.5.2 Side Slopes

The side slopes for the trapezoidal canal section is determined on the basis of the slope stability of soil. The side slope should be as low as possible. The side slopes should be slightly flatter in filling sections than in cut sections. The recommended side slopes of the canal are presented below (***Table 5-1***).

Table 5-1 Recommended side slopes of canal

Soil type	Hard rock	Soft rock	Heavy clay	Loam	Sandy loam
Side slope (Vertical to horizontal)	1:0.25	1:0.50	1:0.50 to 1:1	1:1 to 1: 1.50	1:1.5 to 1:2

5.5.3 Permissible Velocity

The maximum flow velocity, above which scouring will occur in the canal, is termed permissible velocity. This permissible design velocity is chosen from experience and knowledge of the soil conditions. A lower velocity allows weeds to grow in the canal section, consequently decreasing the canal capacity, while a higher velocity tends to erode the bed of an unlined canal. Hence, the velocity should be within the permissible limits based on the type of soil and canal capacity (***Table 5-2***). The velocity shall be checked with permissible tractive force of the canal material, which is given by the equation:

$$T = CWRS$$

Where, C is the coefficient taken as 1 for bed and 0.76 for sides,

W is the specific weight of water, 1000 kg/m^3,

R is the hydraulic radius of the canal section, m

S is the water surface slope of the canal,

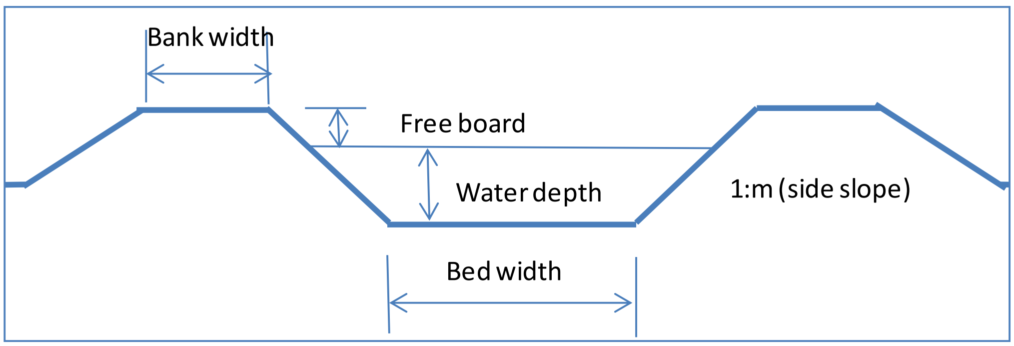
Table 5-2 Maximum permissible velocities and tractive forces

S.N	Material	Maximum permissible velocity (m/s)	Tractive Force (kg/m^2)	
			Clear water	Silty water
1	Fine sand	0.45 – 0.75	0.13	0.37
2	Silt loam	0.60 – 0.75	0.23	0.54
3	Alluvial silt	0.60 – 0.75	0.23	0.73
4	Ordinary loam	0.75 – 1.05	0.37	0.73
5	Fine gravel	0.75 – 1.05	0.37	1.56
6	Coarse gravel	1.00 - 1.25	1.47	3.28
7	Cobbles and shingles	1.25 - 1.50	4.45	5.39

5.5.4 Bed Width to Depth Ratio

The bed width to depth ratio depends upon the capacity of the canal and determines the hydraulic efficiency of the section. For small irrigation canals, when the discharge is less than 0.50 m^3/s, the ratio of bed width to depth ranges from 1 to 2. For discharges higher than 0.50 m^3/s, wider canals should be provided if necessary. In a hilly terrain, a wider canal section requires more hill cutting. In addition, in rocky terrain, narrow sections are preferable. Hence, bed width to depth ratio is fixed based on the scheme's site conditions. A typical canal section of a trapezoidal canal is presented below (***Figure 5-5***).

Figure 5-5 Typical section of trapezoidal canal



5.5.5 Canal Slope

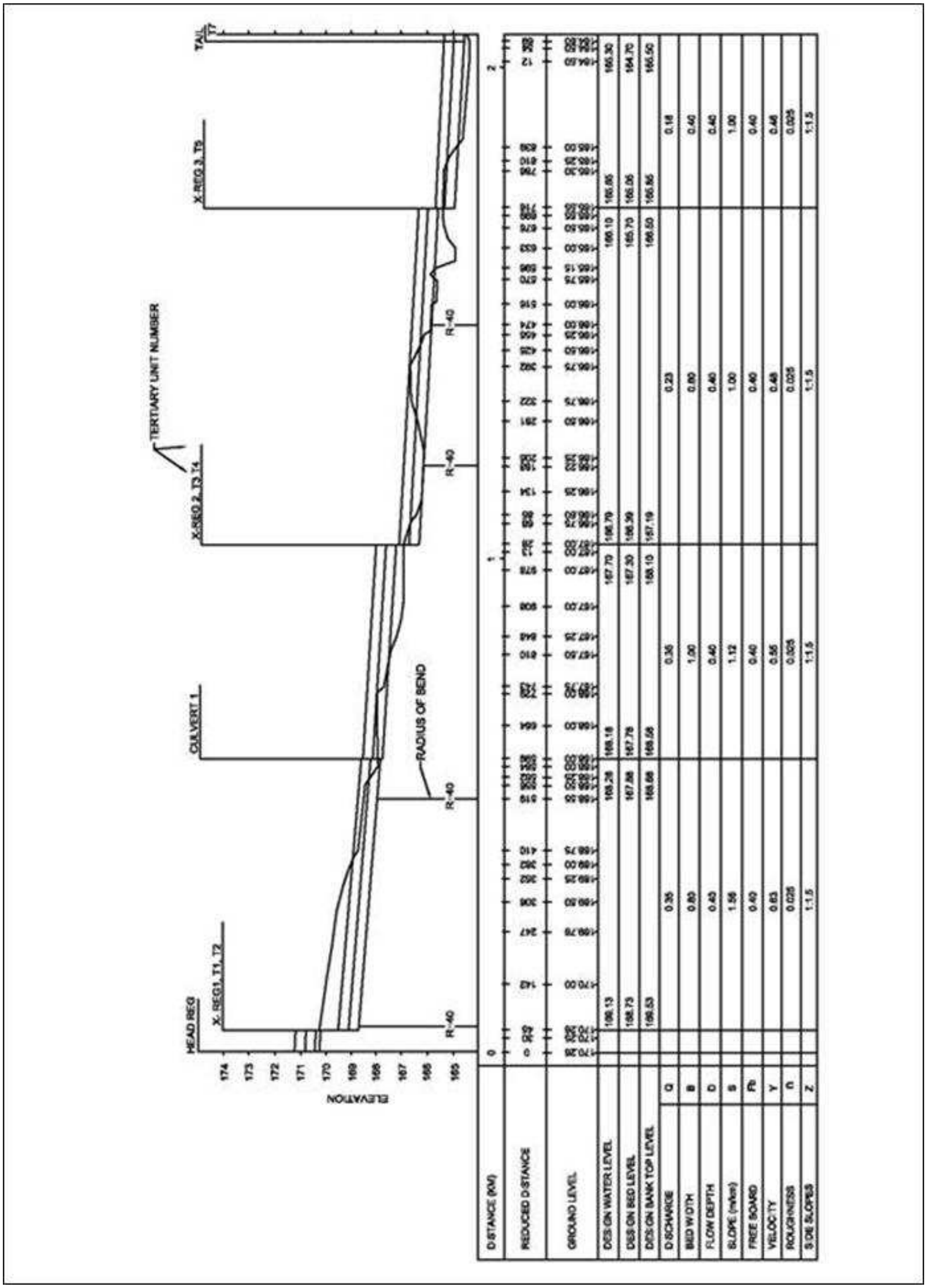
The canal slope is designed for non-silting and non-scouring velocity and is calculated using Manning's Formula:

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

Where, R is hydraulic radius = A/P, S is canal slope and n is roughness coefficient,

For small irrigation schemes, the slope of the canal may be up to 1:500 in the hills and 1:1000 in plain areas. However, for existing community canals, the slopes may be steeper ranging from 1:50 to 1:500 depending upon the canal size, soil type, and topography. A typical longitudinal profile of the canal is presented below (***Figure 5-6***).

Figure 5-6 Typical longitudinal profile of canal



5.5.6 Manning's Roughness Coefficient

The value of Manning's roughness coefficient (n) depends upon the type of bed materials, condition of the canal and its geometry. The recommended values of 'n' are presented below (***Table 5-3***).

Table 5-3 Values of roughness coefficient

S.N	Description	Roughness coefficient, n
1	Earthen canal recently excavated	0.016 to 0.022
2	Gravel bed canal	0.022 to 0.030
3	Canal with short grass	0.022 to 0.033
4	Canal with dense weeds	0.030 to 0.040
5	Canals with earth bed and rubble sides	0.028 to 0.035
6	Canal with stony bottom and weedy banks	0.025 to 0.040

Source: Derived from Ven Te Chow, 1959

5.5.7 Free Board

Free board is the difference between bank top level and full supply level of the canal. It is necessary to accommodate surplus discharge of a canal coming from uphill reaches or from other operations. The free board depends upon the capacity of the canal and its recommended values are given below.

For Q less than 0.10 m^3/s Fb=0.20 m,

For Q between 0.10 to 0.50 m^3/s Fb=0.30 m,

For Q larger than 0.50 m^3/s Fb = 0.50 m

5.5.8 Minimum Radius of Curvature

The irrigation canal should be as straight as possible from a hydraulic point of view. However, in practice it is not possible to align straight canal and canals need to bend to provide an appropriate radius of curvature, which depends upon the canal capacity and topography. . For small irrigation schemes, the acceptable radius for an unlined canal is taken to be from three to seven times the water surface width. In rocky canal sections, the radius of curvature shall be only three times the width, while for general soil, it is up to seven times the width.

5.5.9 Design Example of Unlined Canal

Data:

Command area = 20 ha,

Water requirement in hills = 3pls/ha

Soil type = Ordinary soil

Manning's roughness coefficient = 0.025

Calculation:

Determine design discharge (Q) = command area x water requirement = 20*3 = 60 lps = 0.06 m^3/s

For ordinary soil take side slope (m) = 1:1

Determine bed width to depth ratio (r) = 1

Assume bed width, B = 0.20 m

Adopt slope of canal as 1:100 as per existing conditions,

Calculate depth of flow, D = r*B= 0.20 m

Calculate cross sectional area, $A = BD + mD^2$ = 0.20*0.20 + 1*0.20^2 = 0.08 m^2,

Calculate wetted perimeter of flow, $P = B + 2D\sqrt{(1+m^2)}$=0.20+2*0.20*sqrt(1+1^2) = 0.77 m

Calculate hydraulic radius, R= A/P= 0.08/0.77 = 0.10 m

Calculate velocity of flow, $V = \frac{1}{n} R^{2/3} \times S^{1/2}$ = 1/0.025*0.10^2/3*(1/100)^1/2 = 0.89 m/s

Calculate discharge, Q = A x V= 0.08 * 0.89 = 0.071 m^3/s which is greater than designed discharge of 0.060 m^3/s,

The velocity of flow is slightly higher than the permissible for earthen canal, however it may be suitable for existing site condition having hard clay or rocky bed,

Calculate tractive force, T =CWRS = 1*1000*0.10*1/100 = 1.04 kg/m^2 which is in permissible limit,

Hence, design of unlined canal is acceptable and parameters are:

Bed width = 0.20 m,

Water depth = 0.20 m,

Free board = 0.10 m,

Canal depth = 0.30 m

Side slope (V:H) = 1:1

Longitudinal slope = 1/100

The design of canal section is usually carried out in Excel sheet with trial and error method.

5.6 Lining of Canals

5.6.1 Purpose of Lining

Canal lining is the method of sealing the canal surface to prevent seepage and leakage from the section. In addition, canal lining supports unstable banks and prevents erosion in steep sections of the canal. Lining is often a major demand of farmers in the rehabilitation of existing schemes or in the construction of new schemes. The need for and the type of canal lining should be carefully considered during the planning and design stages and wherever practical lower cost alternatives should be seriously considered.

5.6.2 Type of Canal Lining

i. Dry Stone Lining

Dry stone lining is common in hilly areas where stones are easily available and water is abundant at the source. This lining is mainly to protect against erosion and is not effective for seepage control.

The thickness of the lining depends upon the size of the available stones and ranges from 20 cm to 50 cm. At the downhill side of the canal, dry stone retaining walls are used as part of canal lining to stabilize the bank and protect soil erosion.

ii. Stone Masonry Lining

Stone masonry lining is the most commonly used lining in the hills and foothill areas. Due to its effectiveness in preventing seepage and erosion, it is widely used where stones are available. The recommended thickness of the lining ranges from 30 cm to 50 cm in 1:4 cement sand mortar. For small schemes, lining can often be made with dressed stone, but selected undressed river stones will be cheaper in most cases. The inner sides of the masonry lining may be plastered with sand and cement mortar, but this is not recommended for small schemes. In some cases, the inner bank of the masonry lining is only made with dry stones as this allows drainage into the canal from uphill. A stone masonry lining with the concrete bed is the most common form of canal lining. However, due to quality of works in the hills, the sustainability of stone masonry lining is in question.

Figure 5-7 Typical stone masonry and concrete lining

Stone masonry lining	Concrete lining
	

iii. Concrete Lining

Cement concrete lining is the most effective means of canal lining as it prevents canal seepage and leakage. Depending upon the workmanship, the roughness coefficient of a concrete lining is significantly low in comparison to stone masonry lining, which allows higher discharge in a similar section. Concrete lining may be plain cement concrete lining or reinforced cement concrete lining depending upon the type of canal to be lined and the nature of the project. The thickness of the concrete lining varies from 7.50 cm to 15 cm depending upon the capacity of the canal and the objective of the lining. For large canals, trapezoidal section is suitable for concrete lining while for small canals, rectangular sections are popularly used in the hills. In addition, concrete lining with nominal reinforcement or with GI wires is recommended as low cost canal lining solutions for small irrigation schemes.

iv. Soil Cement Lining

Soil cement lining is a low cost canal lining technology and is rarely used in irrigation canals. It is a mixture of red soil and cement with 7 parts soil and one part cement. The mortar of the soil cement is applied to the surface of the canal as cement plaster in two to three layers. The thickness of one layer of plaster may be 2 to 3 cm. After plastering, curing is also important and the lining should be covered and kept moist for a few days. This type of lining is effective in seepage control but can be damaged by cattle movements and other erosion. Soil cement lining is the most economical lining in an irrigation project.

Figure 5-8 Ferro cement and soil-cement linings

Ferro cement lining	Soil cement lining
	

v. Ferro Cement Lining

Ferro cement is a thin wall reinforced concrete made up of wire mesh, sand, and cement which possesses strength and serviceability. The mixture of cement with pie gravel is placed over the thoroughly compacted surface of canal laid with plastic sheet and chicken mesh wire. Nominal reinforcement is also provided if necessary. This type of lining is costlier than masonry lining, but is durable and effective to control seepage and leakage.

vi. Plastic Lining

Plastic lining is used where the alignment of the canal falls in unstable areas like those prone to landslides. A membrane consisting of a sufficiently thick plastic sheet is laid on the properly leveled surface of the canal and a cover of dry stone or soil is provided to protect it from minor puncture and tear. Plastic lining is quite effective for seepage control, is economical and easy to construct. The thickness of the plastic sheet may be 200 to 300 microns or equivalent (0.25 mm to 0.50 mm). Plastic lining is prone to damage and temporary in nature.

vii. Slate Lining

Slate lining is a special type of canal lining used where slates are available. This type of lining is suitable in mountains where soils are erosive in canal bed. This lining is effective for seepage when used with cement mortar pointing and cheaper than other types of linings. The thickness of the lining may be 10 cm to 50 cm depending upon the thickness of slates.

In short, cement concrete lining is the most sustainable type of lining compared to other types of lining, and hence it is recommended to use even in small canals.

5.6.3 Design Procedure of Lined Canal

Design of lined canal is carried out using continuity equation and Manning's formula considering as rigid boundary canal section. The design procedure of the canal follows a series of steps:

i. Determine the design discharge, Q,
ii. Determine the side slope, m (1 vertical : m horizontal),
iii. Determine canal longitudinal slope, S,
iv. Determine bed width to depth ratio, r (r = B/D; B being bed width and D being water depth),
v. Determine the roughness coefficient based on the lining material (***Table 5-4***),

   The design calculation shall be carried out by trial and error method.

vi. Assume bed width of the canal B,
vii. Calculate water depth, $D = r \times B$
viii. Calculate cross sectional area of flow, $A = BD + mD^2$
ix. Calculate wetted perimeter of flow, $P = B + 2D\sqrt{(1 + m^2)}$
x. Calculate hydraulic radius, $R = \frac{A}{P}$
xi. Calculate velocity of flow using Manning's formula, $V = \frac{1}{n} R^{2/3} \times S^{1/2}$
xii. Calculate discharge of canal using continuity equation, $Q = A \times V$
xiii. Check the calculated discharge and compare with designed one, Qc>Qd
xiv. Compare the calculated velocity with limiting velocity, $V_c < V_{per}$

Table 5-4 Recommended roughness coefficients for lined canal

S. N	Lining Type	Maximum Velocity (m/s)	Roughness Coefficient 'n'
1	Dry stone	1.0	0.025 – 0.028
2	Dry Brick (without mortar)	1.0	0.020 – 0.025
3	Dressed stone masonry	2.0	0.018 – 0.020
4	Brick masonry	1.5	0.017 – 0.020
5	Random rubble masonry (stone)	1.5	0.020 – 0.015
6	Plain cement concrete or RCC	2.5	0.015 – 0.017

Source: PDSP Manual Volume M 8, 1990

The longitudinal slope for small canals depends upon the topography and existing condition of the canal. For rectangular section of canal the bed width to water depth ratio ranges from 0.5 to 2. The side slope of the lined canal is zero in case of rectangular section while it is 1:0.50 to 1:1.50 for trapezoidal canal.

5.6.4 Design Example of Lined Canal

Data:

Command area = 20 ha,

Water requirement in hills = 3 lps/ha

Type of lining = stone masonry

Section type = rectangular section

Manning's roughness coefficient = 0.02

Calculations:

Determine design discharge (Q) = 20*3 = 60 lps = 0.06 m^3/s

For rectangular section side slope (m) = 0

Determine bed width to depth ratio (r) = 1

Assume bed width, B = 0.25 m

Adopt slope of canal as 1:100 as per existing conditions,

Calculate depth of flow, D = r*B= 0.25 m

Calculate cross sectional area, $A = BD + mD^2$ = 0.25*0.25 + 0*0.25^2 = 0.0625 m^2,

Calculate wetted perimeter of flow, $P = B + 2D\sqrt{(1+m^2)}$=0.20+2*0.20 = 0.75 m

Calculate hydraulic radius, R= A/P= 0.04/0.6 = 0.083 m

Calculate velocity of flow, $V = \frac{1}{n} R^{2/3} \times S^{1/2}$ = 1/0.02*0.083^(2/3)*(1/100)^1/2 = 0.95 m/s

Calculate discharge, Q = A x V= 0.0625 * 0.95 = 0.60 m^3/s which is equal to the designed discharge of 0.060 m^3/s,

The velocity of flow is less than the permissible velocity for stone masonry canal. Hence, design of lined canal is acceptable and parameters are:

Bed width = 0.25 m,

Water depth = 0.25 m,

Free board = 0.15 m,

Canal depth = 0.40 m

Side slope (V:H) = 0

Longitudinal slope = 1:100

The design of canal section is usually carried out using excel sheets following a trial and error method.

5.7 Covered Canals

5.7.1 Introduction

Covered canals are used where local slips occur or are likely to occur, which could block the canals. Sometimes they are also necessary in deep cut sections and in the head reach of a canal to protect the canal from floods. The cross sectional size should be made large enough for desilting purposes and access should be provided on long lengths. If rock falls are likely, the structure should be covered with earth to dampen the fall. Covered canals are generally not suitable for deep slide areas. There are various types of covered canals (***Figure 5-9***), some of these are presented below.

i. Concrete or Stone Slab Cover

In rocky areas where canal section is perfectly cut into the size and concrete or stone slabs can be laid directly over the canal section. The slabs should be of manageable size so that, if necessary, they can be removed during canal maintenance.

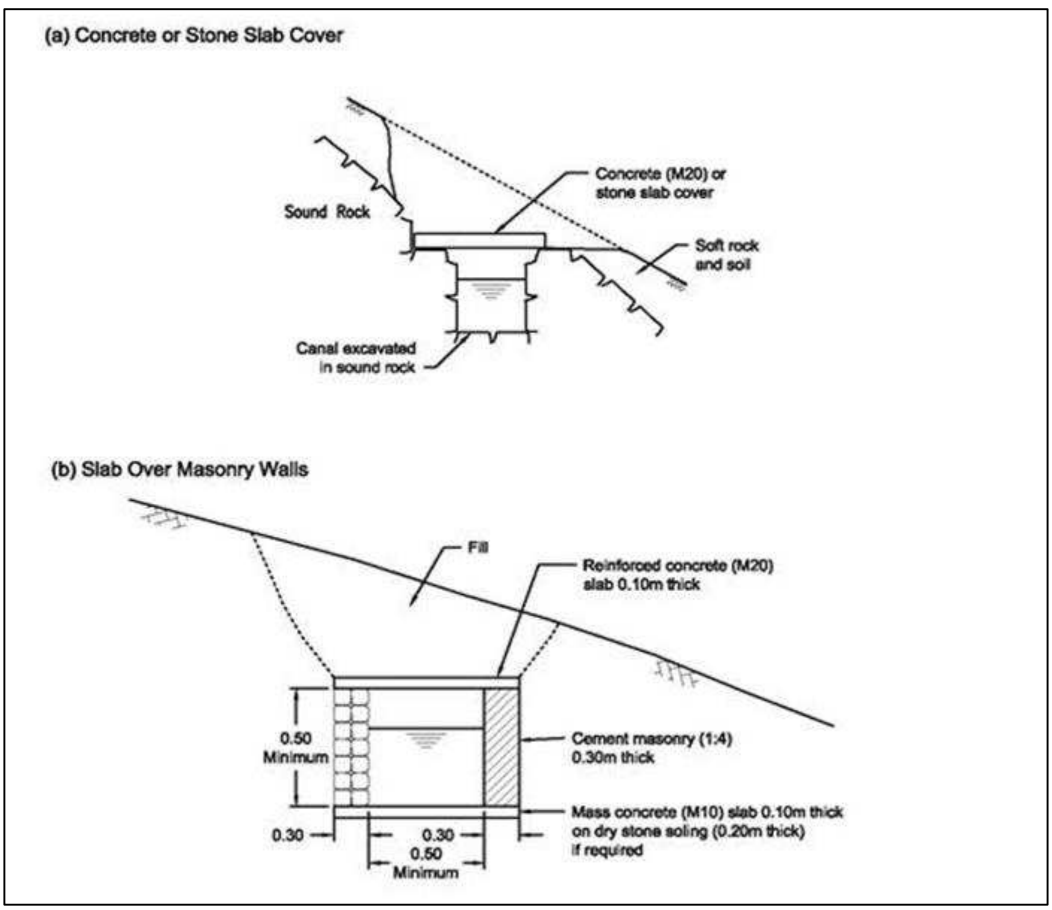
ii. Slabs over stone masonry lining

For lengths where small slips are likely to occur, slabs are laid on masonry walls. Generally cover slabs can be made of reinforced concrete. If a slip occurs on an uncovered section of a lined canal, repairs using slabs would be the most economical solution. On long reaches, for maintenance access, one or more slabs can be temporarily removed.

iii. Pipes

HDP pipes can be used as a cheap form of the covered canal for small schemes, but care must be taken to ensure that the pipes are laid on an adequate slope to prevent siltation and blockage. In addition, adequate pipe joints must be provided to prevent seepage, which could lead to further slips. For larger canals, reinforced concrete pipes of adequate diameter can be used for covered canals. Access manholes should also be provided at 20 to 30 m intervals of covered canal.

Figure 5-9 Typical covered canals



5.7.2 Design Concept

Covered canals are generally designed to be free flowing open channels like lined canal sections. For the hydraulic computation, Manning's formula, which was already described in previous sections, is used. The velocity of water should be kept approximately 1 m/s and free board should be sufficient (at least 0.20 m). The water depth in the pipes should be kept as 2/3 of the diameter of the pipe. For small scale schemes, HDP pipes may run full. Head losses in the transition between the open canal and the covered canal need to be considered, which are calculated as:

$$Hi = \frac{Ci\left(V_p^{\,2} - V^2\right)}{2g}$$

$$Ho = \frac{Co\left(V_p^{\,2} - V^2\right)}{2g}$$

Where, H_i, H_o are the head losses at the inlet and outlet respectively in m,

C_i, C_o are loss coefficients for the inlet and outlet;

V_p is the velocity in the covered section in m/s,

V is the velocity in the open canal in m/s,

g is the acceleration due to gravity = 9.81 m/s^2

The value of Ci and Co is different for different types of transitions. The values for some of the transitions are as presented here (***Table 5-5***). The most practical values of Ci and Co are 0.50 and 1.00 respectively.

Table 5-5 Coefficients for inlet and outlet of different transitions

S.N	Type of transition	Ci	Co
1	Culvert pipe terminates in headwall across channel	0.50	1.00
2	Headwall with rounded transition of which the radius exceeds 0.1 times the depth of water	0.25	0.50
3	Broken back transition with flare angle of about 1:5	0.20	0.40

Large diameter pipes are generally flow part full in covered canals. The following table presents the hydraulic properties of the pipes flowing part full (***Table 5-6***).

Table 5-6 Hydraulic properties of part full flowing pipe

Water depth from invert	Wetted area (A)	Wetted perimeter (P)	Hydraulic radius (R)	Water surface width (Ws)
0.10 d	0.04 d^2	0.64 d	0.063 d	0.6000 d
0.20 d	0.11 d^2	0.93 d	0.118 d	0.8000 d
0.30 d	0.20 d^2	1.16 d	0.172 d	0.9165 d
0.40 d	0.29 d^2	1.37 d	0.212 d	0.9798 d
0.50 d	0.39 d^2	1.57 d	0.250 d	1.0000 d
0.60 d	0.49 d^2	1.77 d	0.277 d	0.9798 d
0.70 d	0.58 d^2	1.98 d	0.293 d	0.9165 d

Water depth from invert	Wetted area (A)	Wetted perimeter (P)	Hydraulic radius (R)	Water surface width (Ws)
0.80 d	0.67 d^2	2.21 d	0.303 d	0.8000 d

5.7.3 Design Examples of Covered Canal (Piped Canal)

i. Case-I Pipe flowing full

Data:

Discharge, Q = 100 l/s

Velocity in canal = 0.50 m/sec.

Pipe type = HDP

Length = 70 m

2 bends at 45^o radius = 2* diameter

Available head difference = 2.0 m

Calculations:

Based on the design discharge and pipe type select size of the pipe,

From HDP pipe friction chart (***Figure 5-11***) select 250 mm diameter pipe,

Calculate the cross section area of the pipe A = $\pi D^2/4$ = 3.14*0.25^2/4 = 0.049 m^2

Calculate the velocity of flow, V=Q/A= 0.1/0.049 = 2.05 m/s, acceptable for HDP pipe

Determine friction loss from graph (***Figure 5-11***), friction loss is 1.10 m for 100 m length,

Calculate friction loss for 70 m length of pipe, friction head loss h_f= 70 x 1.1/100 = 0.77 m

Head loss at entry and exit, $h_e = \dfrac{1.5\left(V^2{}_2 - V^2{}_1\right)^2}{2g}$ = 1.5 [2.05^2 – 0.5^2] / 19.62 = 0.32 m

Head loss at pipe bends, $h_b = f_b \dfrac{V^2}{2g}$ = 0.09 x 2.05^2 / 19.62 = 0.02 m

For two bends head loss, h_b = 0.02 x 2 = 0.04 m

Therefore, total head loss = 0.77 + 0.32 + 0.04 = 1.13 m < 2 m

The total head loss is less than available level difference and hence design is acceptable.

ii. Case-II Pipe flowing part full

Data:

Discharge = 200 l/s

Depth = 0.5 m

Pipe type = concrete

Length = 150 m

Velocity in canal = 0.5 m/s

Manning's n = 0.015

Simple head wall for entrance and exit of the pipe

Calculations:

Velocity is to be kept below 3.0 m/s

The pipe will flow with minimum of 0.20 diameter freeboard.

From Table 5-6 area for water depth of 0.8 diameter = $0.67d^2$

Calculate minimum flow are in the pipe with the velocity limit, A = Q/V_{max}

Q= 200 l/s = 0.20 m^3/s and V_{max} = 3 m/s; then A = 0.2/3.0 = 0.067 m^2

And, hence, $0.67d^2$ = 0.067

The diameter of pipe, d = 0.315 m

Required diameter = 0.315 m

Use 350 mm diameter pipe as next available size,

Calculate the actual velocity of flow in pipe, V = Q/A = 0.2/(0.67 X d^2) = 0.2/(0.67 X 0.35^2)

V = 2.44 m/s

Before proceeding with the calculation of losses, check the pipe inlet setting in relation to the canal depth,

Canal depth = 0.5 m

Pipe diameter = 0.35 m

If pipe invert is set at canal bed level, then water depth above invert = 0.5 m. This is about 1.4 times the pipe diameter. The pipe shall submerge when water depth is greater than 1.2 times the pipe diameter. To avoid submergence, the pipe invert would have to be set above canal bed level which is not satisfactory for sediment transport. Therefore, increase pipe diameter to 0.6 m in order to match the canal depth and set invert of pipe at bed level.

Calculate the slope of the pipe from formula:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \text{ or } S = \frac{n^2 V^2}{R^{4/3}}$$

For depth of flow = 0.8 x diameter, hydraulic depth R = 0.303 d

Area of flow, A = 0.67 d^2 = 0.67 x 0.60^2 = 0.24 m^2

Hydraulic radius, R = 0.303 d = 0.303 x 0.60 =0.182 m

Velocity of flow, V = Q/A = 0.20/0.24 = 0.83 m/s

Slope of the pipe, S = (0.0152 x 0.832)/ (0.1824/3) = 0.0015

Head loss = S x length = 0.0015 X 150 = 0.23 m

For entry and exit losses, Ci = 0.50 and Co = 1.0

Losses = [1.5(Vp2 – V2)]/2g = [1.5(0.83^2 – 0.5^2)]/19.62 = 0.03 m

Total head loss = 0.23 + 0.03 = 0.26 m

Set outlet to ensure that the pipe is not submerged. Allow 0.2 x diameters as freeboard i.e., set pipe invert at (downstream water level – 0.8 x diameters).

The Froude number in pipe flowing part full (and covered canals) should be kept below 0.55 to avoid standing waves. The Froude number is calculated from formula:

F = [αV2/(g . dm)]0.5

Where, dm = flow area in sq m/water surface width in m, Here V = 0.83 m/s

dm = $0.67d^2/0.8d = 0.536 \times d = 0.32$ m

And then, F = 0.83^2/(9.81x0.32)^1/2 = 0.39 < 0.55

Hence, design is acceptable.

Figure 5-10 Design chart for concrete pipes

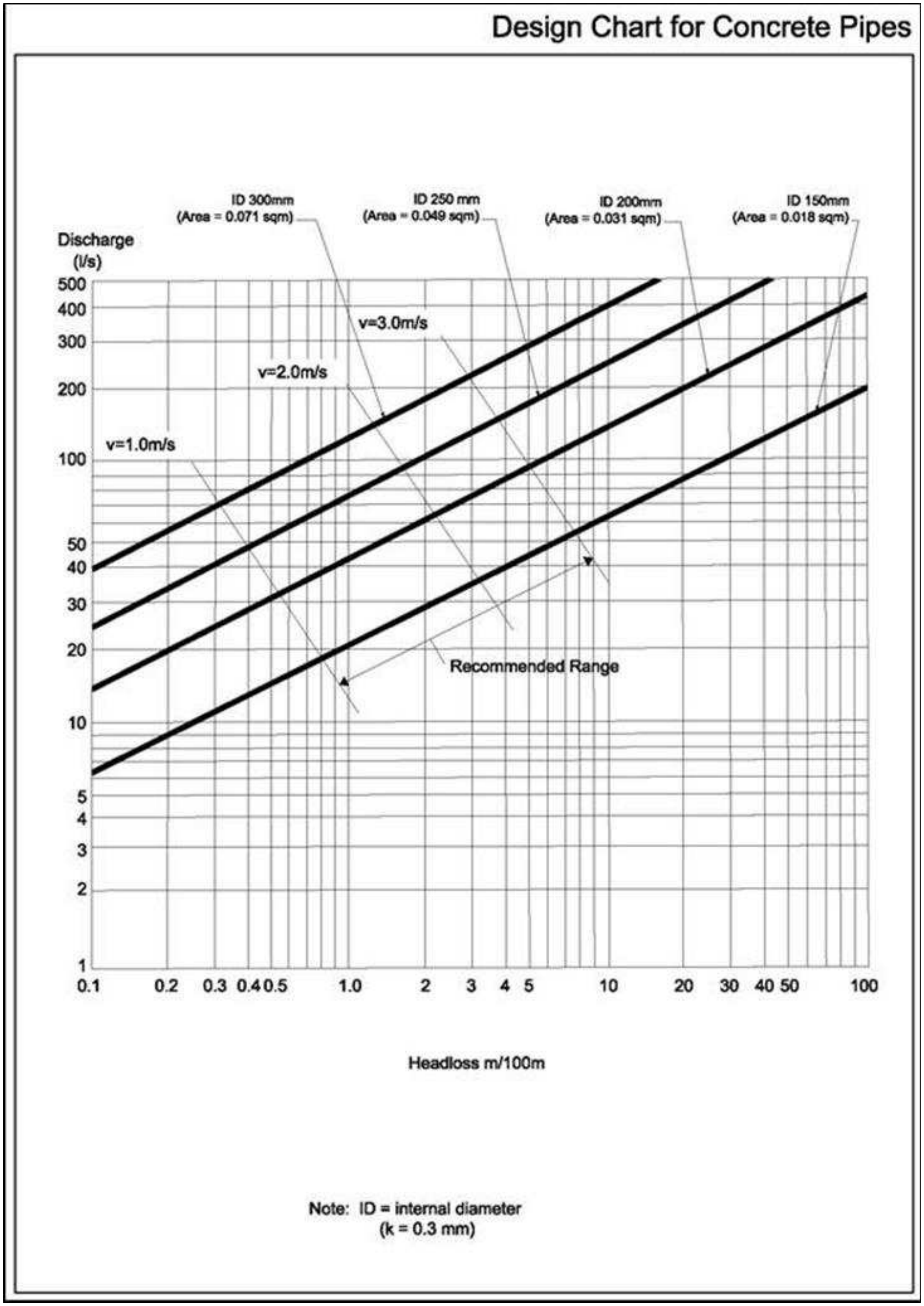
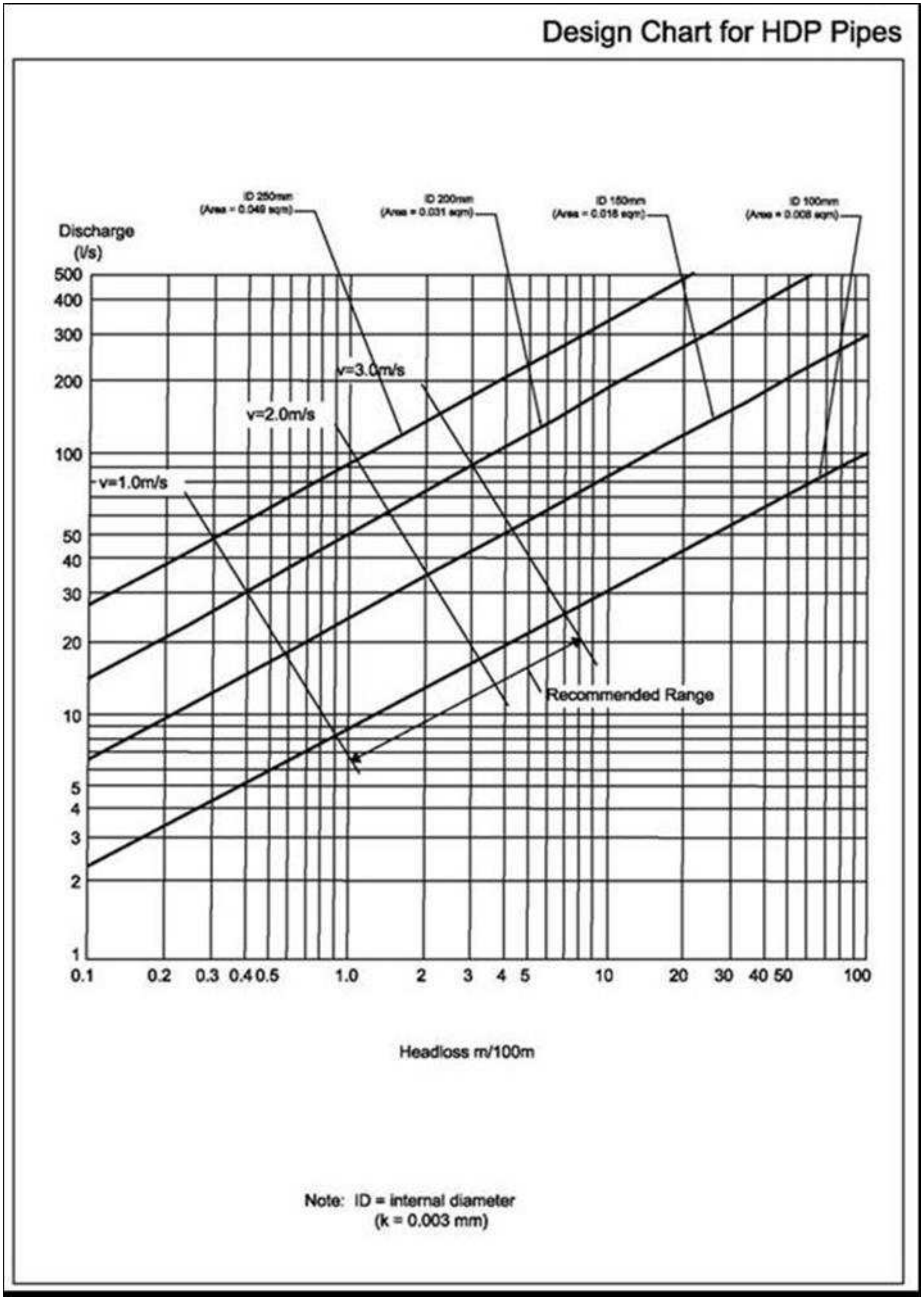


Figure 5-11 Design chart for HDPE pipes



5.8 Design of Pipelines

5.8.1 Basic concept on hydraulic design

Energy is necessary for the movement of water from one place to another. A gravity-flow water system is powered by gravitational energy which is determined by the relative elevations of the system. As water flows through pipes, some energy is lost forever, dissipated by friction. Due to the changing topographic profile of the system, some points have minimal energy (low pressure) while some points have excessive energy (high pressure). The purpose of pipeline design, therefore, is to manage properly frictional energy losses in order to deliver the desired amount of flow to the desired locations. This is accomplished by the careful selection of pipe sizes, strategic location of control points, break pressure tanks, reservoirs and outlets.

i. Relation between head and water pressure

Head is the measure of energy and represents the amount of gravitational energy contained in the water. In metric system of units, head is always measured in meters. The water pressure at any point is determined by the depth of water at that point (***Table 5-7***).

Table 5-7 Water pressure according to the head

Head (m)	10	20	30	50	60	100
Water pressure (kg/cm^2)	1	2	3	5	6	10

In a pipeline where water is not moving, the level of water surface is termed as static level and the pressures are reported as static heads. When water is moving in the pipeline with a constant flow the system is in dynamic equilibrium.

ii. Hydraulic grade line (HGL)

Hydraulic grade line represents the energy level at each point along the pipeline. The vertical distance from the pipeline to the HGL is the measure of pressure head and the difference between HGL and static level is the amount of head lost by the friction of the flow. The water pressure at atmosphere is zero and HGL always come to zero wherever comes into contact with the atmosphere. Since frictional losses are never recovered, the HGL always slopes down along the direction of the flow.

iii. Control valves

An excessive amount of energy can cause the pipe to burst. One method to control an excessive amount of the energy is to install control valves at strategic points throughout the pipe line system. A valve is a device which can be adjusted to create greater frictional losses as water flows through it. There are two types of control valves: gate valves and globe valves (*Figure* ***5-12***) used in pipe line.

A gate valve serves as an on/off control valve for the purpose of completely cutting off the flow. Generally, it is located at the outlets of the intake, reservoir, break pressure tank, and at major branch points. Globe valves are designed to regulate flow through the system and are best located near the discharge points.

Figure 5-12 Gate and globe valves

Gate Valve	Globe Valve
	

iv. Frictional head loss

Any obstruction to the flow, partial or otherwise, causes frictional losses of the energy. The magnitude of the energy lost due to the friction against some obstacle is determined by several factors. The major factors would be the roughness of the obstacle and the velocity of the flow while minor factors would include water temperature, suspended particles etc. The diameter of the pipe and the amount of flow through it determines the velocity of flow. The greater the flow, the faster is the velocity, and the greater the frictional losses. Likewise, the rougher the surface of the obstacle, the greater is the frictional losses. Frictional head losses are provided by pipe manufacturers for both HDP and GI pipes.

Frictional head losses of flow through fittings such as elbows, reducers, tees, valves etc are calculated using various formulas and coefficients. In small scale pipeline design, these frictional head losses are available as equivalent pipe lengths. The amount of head loss in the fittings depends upon the shape of the fitting and the flow through it. Head losses are computed with the determination of the equivalent length of pipe necessary to create the same amount of head loss. The lengths to diameter (L/D) ratio for various fittings are given here (*Table **5-8***).

Table 5-8 Length to diameter ratio of different fittings

S.N	Fittings	L/D ratio
1	Tee (run-side)	68
2	Tee (run-run)	27
3	Elbow (90^o)	33
4	Union	7
5	Gate valve	7
6	Free entrance	29
7	Screened entrance	150

v. Maximum pressure limits

Maximum pressure limits are considered when selecting pipe materials and type to withstand the highest possible pressure than occurs in the pipeline. For a gravity system, the worst scenario is for pressure during shut-off conditions. Class III HDP pipes have maximum pressure limits of 6 kg/cm^2 which mean 60 meters of head. Class IV HDP pipes have maximum pressure limits of 10 kg/cm^2 for the use in 100 meters of head. Class IV pipes have higher wall thickness which withstands greater pressures. Galvanized Iron (GI) pipes can withstand 25 kg/cm^2 of pressure, meaning 250 meters of head. To limit the maximum pressure, break pressure tanks could be constructed along the main pipeline.

vi. Minimum pressure limits

The pressure in the pipe along the section where HGL is underground is a negative pressure. Negative pressure in the pipe needs to be avoided for the proper design of the system. It is recommended to align HGL 10 meters above the ground level to have the minimum pressure limit. In any case, HGL is not allowed to go underground at all.

vii. Velocity limits

The velocity of flow through the pipeline is also an important consideration in the design of a pipeline. If the velocity is too high, the flow can cause excessive erosion of the pipe, while if it is too low, it can create clogging of the pipeline. The recommended velocity limits are 3.0 m/s (maximum) to 0.70 m/s (minimum).

viii. Air-blocks and washouts

An air-block is a bubble of air trapped in the pipeline whose size is such that it interferes with the flow of water through the section. When water is allowed to fill or refill the pipeline, air cannot escape from certain section and is trapped. The proper design can minimize the trapped air and potential air-blocks by:

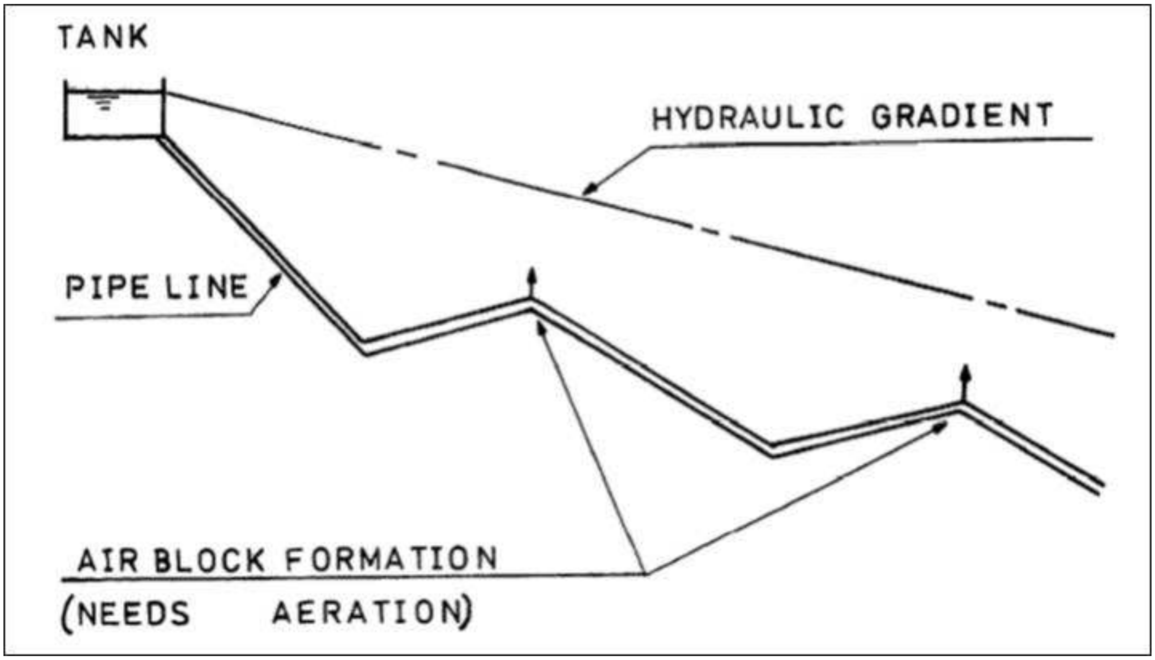
- Arranging pipe sizes to minimize frictional head loss between the source and the first air-block,
- Using larger size pipe at the top and smaller size at the bottom of the critical sections,
- Eliminating or minimizing higher air-blocks which are closer to the static level,
- Providing air valves,

A typical layout of pipeline is presented for reference (*Figure 5-13*)

ix. Air valves

Air valves are automatic air release devices, which releases accumulated air pressure of the pipeline. In mains, air valves are mostly placed at critical places. A structure constructed to locate the valve is referred as valve chamber. The typical examples of air valves are presented in *Figure **5-14*** and typical plan of the valve chamber are presented in ***Figure** 5-16*.

Figure 5-13 Typical layout of pipeline



x. Washouts

Washouts are located at the bottom points of the major U-profiles, especially those upstream from the reservoir tank. The washouts are the gate valve mounted on the Tee branch to a main line. The number of washouts in the system depends upon the type of source, whether, or not there is sedimentation tank or reservoir, and the velocity of flow through the pipe line. The size of the washout pipes should be same as that of the pipeline at that point. These washouts are regularly opened to clean the pipe.

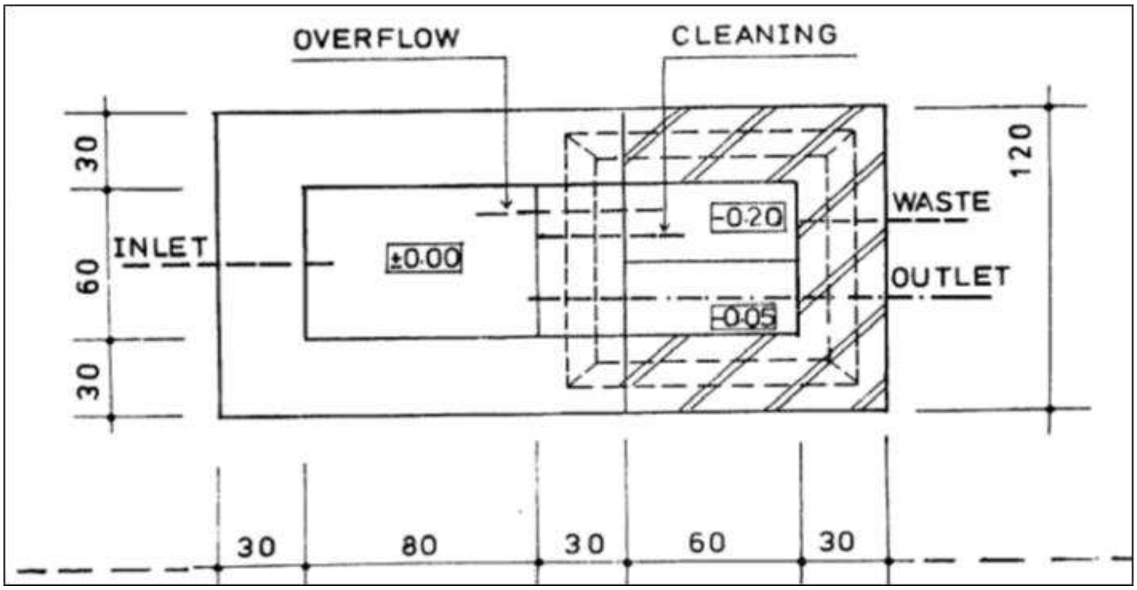
Figure 5-14 Typical air release valves

xi. Break pressure tank

A break pressure tank (BPT) is a structure to allow the flow to discharge into the atmosphere, thereby reducing the hydrostatic pressure to zero and establishing a new static level. A BPT is built along the main pipeline at strategic locations. The flowing water is discharged in the tank and the energy of the flowing water is dissipated with appropriate devices and water is back to atmospheric pressure. The energy dissipation is carried out with baffle walls constructed specially in the BPT.

Break pressure tanks are constructed with cement concrete or stone masonry with or without float valves. An overflow system with the capacity to remove the maximum quantity of water which is likely to overflow should be included (*Figure **5-15***). The overflow water should be disposed in such a way that it will not cause soil erosion.

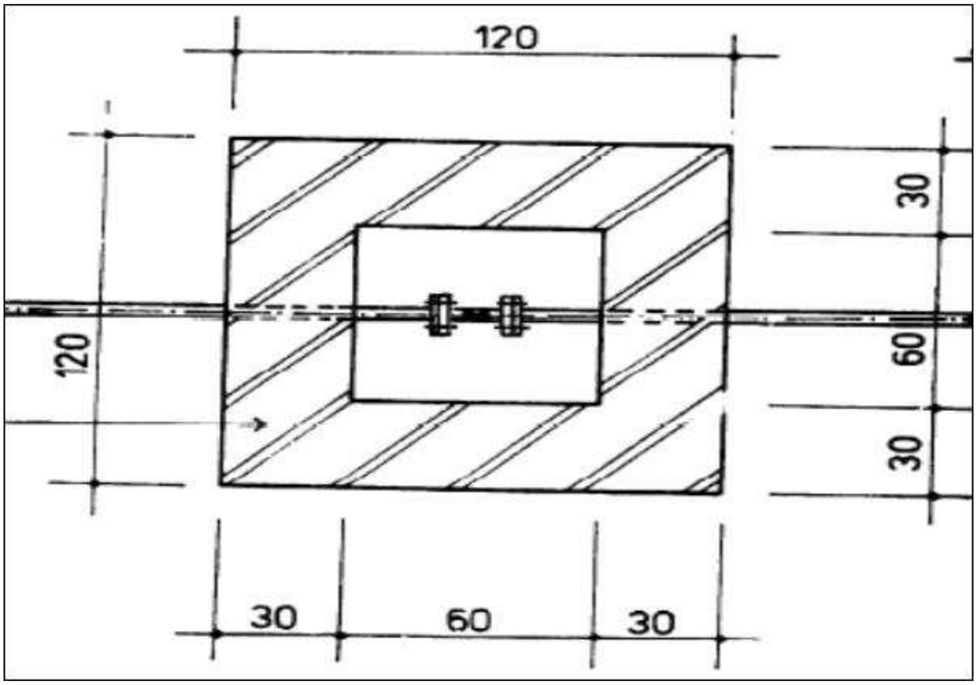
Figure 5-15 Typical plan of break pressure tank



5.8.2 Design Concept of Main Pipeline

The basic concept of the main pipeline is to calculate the head losses in different reaches of the pipeline. For each reach, it is necessary to determine the desired amount of head to be burned off and the length of the pipeline, calculate the desired frictional head loss factor, and select the pipe size from the table. When designing the pipe, the designer can begin from the source or from the end of the pipeline. It is also necessary to assess the requirement of the reservoir based on the balance study of supply and demand.

Figure 5-16 Typical valve chamber



5.8.3 Head losses in pipes and fittings

The calculation of head losses in the pipes and their fittings is the most important parameter of pipe line design. Head losses in the pipe line occur due to friction, bends, and sudden expansion and contraction of pipes. The commonly used formulas for the computation of head loss due to friction are:

- Darcy-Weisbach formula,
- Hazen Williams formula,
- Manning's formula, and
- Colebrook-White equation

i. Darcy-Weisbach formula

Darcy-Weisbach is the most widely used formula in pipeline design for short length pipes.

$$h_f = \lambda \frac{4L \times V^2}{D2g}$$

Where hf is the fiction head loss in m,

λ is the friction coefficient,

L is the pipe length in m,

D is the pipe inside diameter in m,

V is the velocity of water in m/s, and

G is the acceleration due to gravity = 9.81 m/s^2

For cast iron pipes λ = [0.02+1/(2,000D)]x1.5

For steel pipes λ = [0.0144+9.5/(1000√V)]x1.5

For exact calculation of λ in terms of Reynolds number and relative roughness Moody's diagram is used (*Figure* ***5-17***). For the use of Moody's diagram it is necessary to calculate the Reynolds number from formula:

$$R_e = \frac{V\,D}{\gamma}$$

Where V is the average velocity of flow in m/s,

Diameter of pipe in m, and

γ is the kinematic viscosity of water in m^2/s

ii. Hazen-William's Formula

Hazen-William's formula is widely used for long distance pipelines.

$$\frac{h_f}{L} = \frac{10.67 \times Q^{1.85}}{C^{1.85} \times D^{4.87}}$$

Where h_f is the friction head loss in m,

L is the pipe length in m,

Q is the flow rate in m^3/s,

C is the flow coefficient based on type of pipes and is provided in *Table* ***5-9***,

D is pipe inside diameter in m

Table 5-9 Hazen-William's flow coefficients

Type of pipe	Flow coefficient (C)	Type of pipe	Flow coefficient (C)
New welded steel pipe	140	Cement line pipe	140
Steel pipe after 20 years use	100	Asbestos cement pipe	130
New cast iron pipe	130	Pre-stressed concrete pipe	130
Cast iron pipe after 20 year use	100	Plastic pipe	150

iii. Manning's formula

Manning's formula is generally used in open channel flow and in culverts and is given by:

$$\frac{h_f}{L} = \frac{n^2 \times V^2}{R^{4/3}}$$

Where h_f is the friction head loss in m,

L is the pipe length in m,

n is the roughness coefficient and is given in *Table* ***5-10***, V is the velocity of flow in m/s, and

R is the hydraulic radius of the pipe and is equal to cross sectional area divided by wetted perimeter of the pipe in m,

Table 5-10 Values of pipe roughness

Pipe material	Roughness k, (mm)
Glass, drawn copper	0.003
Steel	0.05
Asphalted cast iron	0.12
Galvanized cast iron	0.15
Cast iron	0.25
Concrete	0.3 to 3.0
Plastic	0.03

iv. Colebrook White Equation

The Colebrook-White equation provides a mathematical method for calculation of the friction factor.

$$\frac{1}{\sqrt{f}} = 1.14 - 2Log_{10}\left(\frac{e}{D} + \frac{9.35}{R_e\sqrt{f}}\right)$$

Where f is the friction factor,

e is the internal roughness of the pipe,

D is the inner diameter of pipe and

Re is the Reynolds number

v. Head loss in pipe fittings

Pipeline fittings and valves produce head losses depending on their configurations which state that value of head loss is proportional to the square of flow velocity.

$$h_f = f\frac{V^2}{2g}$$

Where hf is the head loss in m,

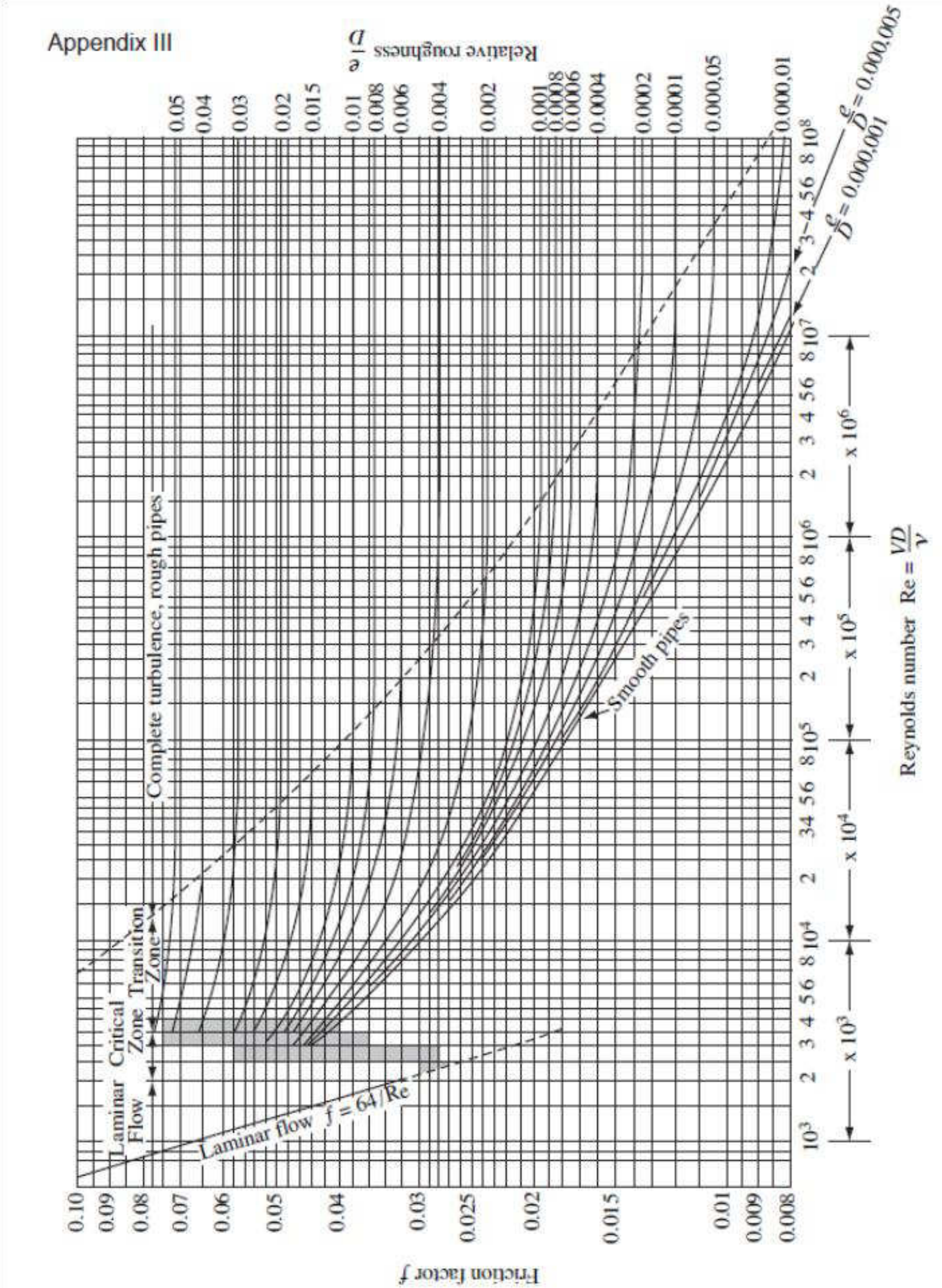
f is the head loss coefficient,

V is the velocity of flow in m/s, and

g is the acceleration due to gravity in m/s^2

For small scale pipeline design, the head loss in pipe fittings including other losses en route is taken as the percent of friction head loss which ranges between 10 to 20 percent. In that case, other losses do not need to be calculated separately.

Figure 5-17 Moody's diagram for friction loss



From L. F. Moody, "Friction factors for Pipe Flow," *Trans.* A.S.M.E., Vol. 66, 1944

The Moody diagram for the Darcy-Weisbach friction factor f

vi. Head loss at pipe inlet

The loss of head occurs at the inlet of the pipeline which is based on the shape and configuration of the inlet structure and is calculated as follows:

$$h_i = f_i \frac{V^2}{2g}$$

Where hi is the head loss at inlet in m,

fi is the inlet loss coefficient which depends upon the shape of the inlet structure and provided in *Table* ***5-11***,

V is the velocity of flow after inlet in m/s, and

g is the acceleration due to gravity in m/s^2

Table 5-11 Head loss coefficients at pipe inlet

Inlet shape	Straight	Chambered	Rounded	Bell mouth
Coefficient (fi)	0.50	0.25	Circular = 0.10 Rectangular= 0.20	0.06

vii. Head loss at pipe outlet

The loss of head also occurs at the outlet of the pipeline which is based on the velocity of flow before the outlet and is calculated as follows:

$$ho = f_o \frac{V^2}{2g}$$

Where h_o is the head loss at outlet in m,

f_o is the outlet loss coefficient and is equal to 1,

V is the velocity of flow before outlet in m/s, and

g is the acceleration due to gravity in m/s^2

viii. Head loss at sudden expansion

The loss of head also occurs with the sudden expansion of the pipeline (*Figure* ***5-18***) which is based on the sizes of the pipes and their corresponding velocities and is calculated as follows:

$$h_{se} = \frac{(V_1 - V_2)^2}{2g} = f_{se} \frac{V^2{}_1}{2g}$$

Where f_{se} is the loss coefficient for sudden expansion (*Table* ***5-12***),

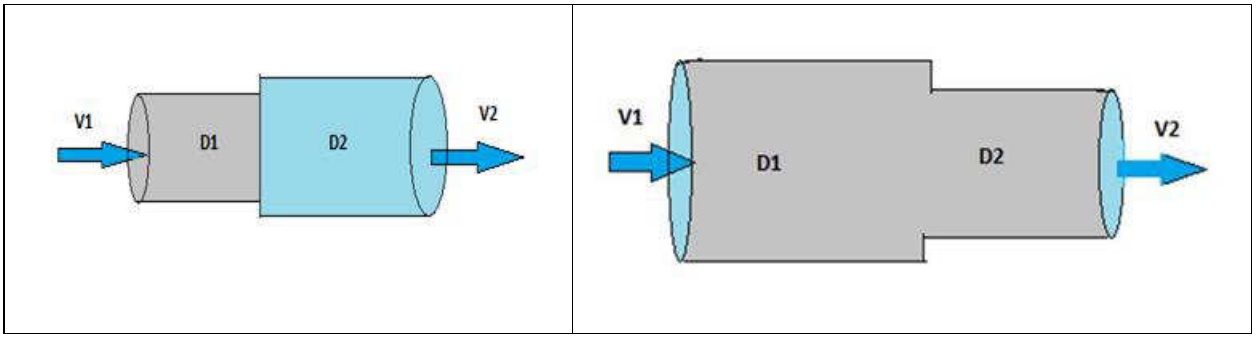
V_1 is the velocity before expansion in m/s,

V_2 is the velocity after expansion in m/s

Table 5-12 Head loss coefficients for sudden pipe expansion

D_1/D_2	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90
f_{se}	0.98	0.92	0.82	0.70	0.56	0.41	0.26	0.13	0.04

Figure 5-18 Sudden expansion and contraction of pipes



ix. Head loss at sudden contraction

The loss of head also occurs at the sudden contractionof the pipeline which is based on the sizes of the pipes and their corresponding velocities and is calculated as follows:

$$h_{sc} = f_{sc}\frac{V^2{}_2}{2g}$$

Where f_{sc} is the head loss coefficient for sudden contraction (*Table* ***5-13***),

V_2 is the velocity after contraction in m/s,

Table 5-13 Head loss coefficient for sudden contraction of pipe

D_2 / D_1	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90
f_{sc}	0.50	0.45	0.43	0.40	0.36	0.31	0.24	0.17	0.09

x. Head loss at gradual expansion

The loss of head also occurs with the gradual expansion of the pipeline (*Figure* ***5-19***), which is based on the angle of the expansion and is calculated as follows:

$$h_{ge} = f_{ge}\frac{(V_1 - V_2)^2}{2g}$$

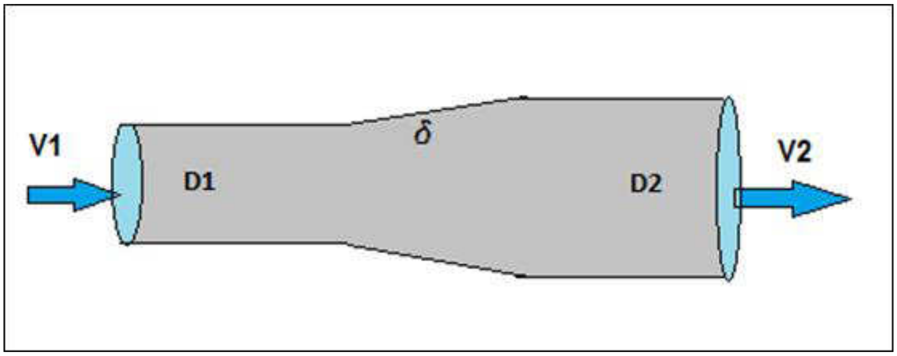
Where f_{ge} is the head loss coefficient for gradual expansion and is equal to $0.011\theta^{1.22}$,

V_1 is the velocity before expansion in m/s, V_2 is the velocity after expansion in m/s, and θ is the expansion angle in degrees (*Table 5-14*).

Table 5-14 Head loss coefficient for gradual expansion

θ	7.50	10.00	12.50	15.00	17.50	20.00	22.50
F_{ge}	0.126	0.183	0.240	0.299	0.361	0.425	0.491

Figure 5-19 Typical gradual expansion of pipeline



xi. Head loss at elbow

Head loss occurs at bends of the pipeline which is based on the turning angle, ratio of radius of curvature to the diameter of pipe and is calculated as follows:

$$h_b = f_b \frac{V^2}{2g}$$

Where f_b is the loss coefficient for elbow,

V is the velocity of flow in m/s,

The values of f_b are given by the following experimental equation for smooth steel pipe, which are function of ratio of radius of curvature (R) to the diameter (D) and turning angle (θ) in the range of 0.50 to 2.5 (*Table **5-15***).

Table 5-15 Head loss coefficients for elbows of pipeline

R/D\θ	22.50°	30°	45°	60°	90°
0.50	0.989	1.420	1.399	1.615	1.978
0.75	0.289	0.334	0.409	0.472	0.578
1.00	0.147	0.17o	0.208	0.240	0.294
1.50	0.085	0.121	0.121	0.139	0.170
2.00	0.073	0.084	0.103	0.119	0.145
2.50	0.069	0.079	0.097	0.112	0.138

xii. Head loss at Tee

A loss of head also occurs at the Tee of the pipeline (*Figure **5-20***) which is based on the flow direction and velocity in the pipe, and is calculated as follows:

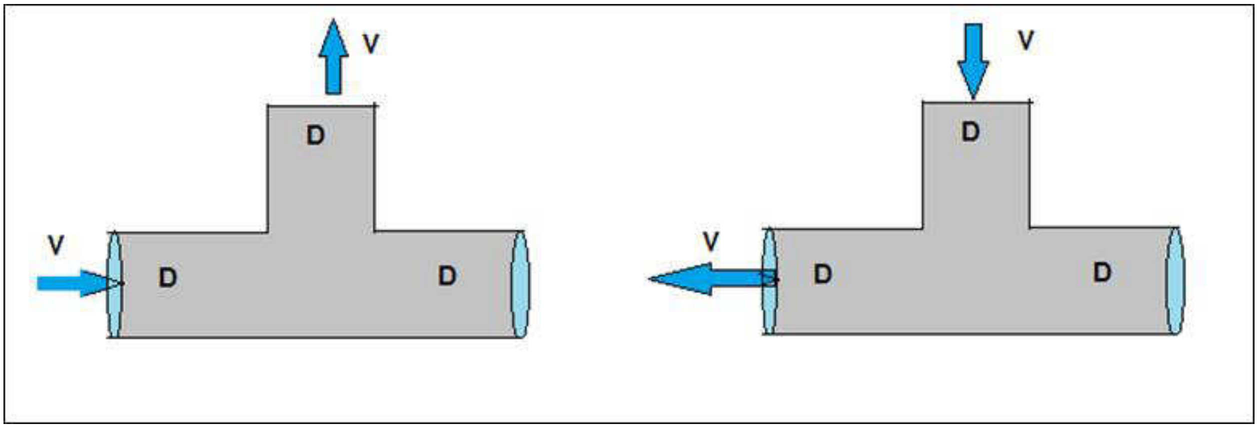
$$h_t = f_t \frac{V^2}{2g}$$

Where f_t is the head loss coefficient for Tee,

V is the velocity of flow in m/s,

The values of f_t depend upon the flow ratios of respective flow directions and corner chamfers. For flow from straight pipe towards tee f_t is 0.97 and from tee to straight pipe f_t is 1.5. The loss coefficients for branches are also similar to that of tee.

Figure 5-20 Sketch of flow from straight pipe to tee and tee to straight pipe



xiii. Head loss at Orifice

Head loss occurs at the orifice of the pipeline which is based on the throttling ratio and velocity in pipe and is calculated as follows:

$$h_{or} = f_{or}\frac{V^2}{2g}$$

Where f_{or} is the head loss coefficient for orifice,

V is the velocity of flow in m/s,

The head loss at an orifice depends on the throttling ratio as given in *Table **5-16***.

Table 5-16 Head loss coefficients for orifice

$(D_2/D_1)^2$	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90
f_{or}	226	47.80	17.50	7.80	3.75	1.80	0.80	0.29	0.06

xiv. Head loss at Valves

At valves connected with pumps and installed on the pipeline, the head losses are produced according to the valve types used even though they are fully opened. The head loss is expressed as:

$$h_v = f_v\frac{V^2}{2g}$$

Where f_v is the loss coefficient for valves and depend upon the type of valves used. These coefficients are given in *Table **5-17***.

V is the velocity of flow in m/s,

Table 5-17 Head loss coefficients for valves

Nominal size (mm)	100	150	200	250	300	350	400	500	600
Gate valve	0.16	0.15	0.11	0.05	Negligible				
Butterfly valve	-	-	2.40	1.80	1.40	1.09	0.86	0.60	0.45
Check valve	1.32	1.27	1.21	1.16	1.11	1.05	1.00	0.98	0.96

xv. Head losses at screens

In case of inlets with trash screens (*Figure **5-21***), the head loss across the screen shall be calculated by the following formula:

$$h_{ts} = 3\sin\delta \times \left(\frac{s}{b}\right)^{4/3} \times \frac{V^2}{2g}$$

Where h_{ts} is the head loss across the screen in m,

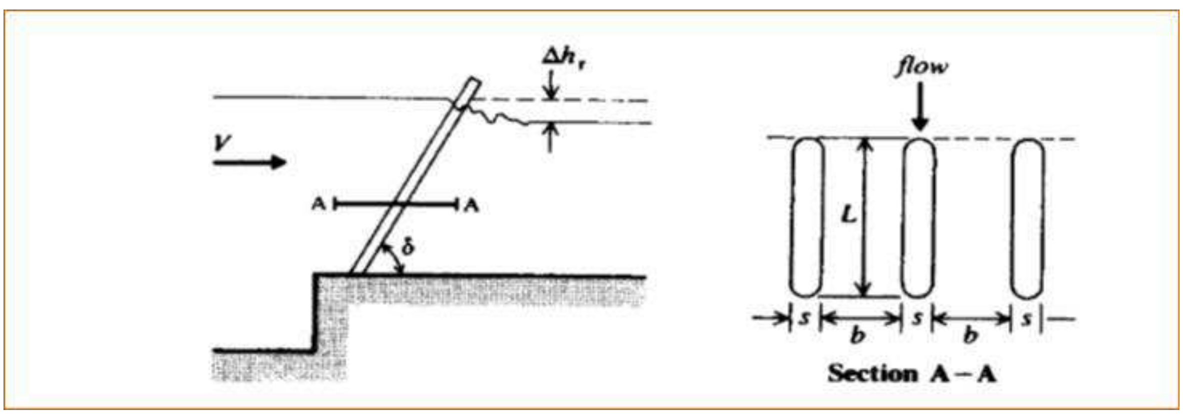
δ is the inclination of the screen,

s is the thickness of bar in mm,

b is the opening between bars in mm, and

V is the velocity upstream of the screen

Figure 5-21 Typical sketch of trash screen bars



5.8.4 Design Procedure of Piped Canal

i. Calculate the design flow of the pipeline as 25% greater than required flow considering partial blocking,

$$Qd = 1.25 \times Qr$$

Where Qd is the designed flow and Qr is the required flow,

ii. Determine the elevation difference of the pipeline between start and end points,

iii. Divide the pipeline length into strategic segments, the first segment will be from source to the first break pressure tank, second segment from first break pressure tank to second break pressure tank, and the third segment will be from second break pressure tank to discharge tank,

iv. If the demand of water is greater than the available flow at the source a reservoir will be necessary to provide to balance the flow. If the demand of water is less than the flow at the source storage reservoir will not be necessary. The design of main pipeline will be carried out based on the maximum water demand for irrigation,

v. Calculate the diameter of the pipe considering the limiting velocity and flow in the pipe for each segment,

vi. Prepare line diagram of the pipeline system with details of levels and pipe diameters,

vii. The example of pipeline works:
    a. Intake,
    b. Supply main pipeline,
        i. Intake to first break pressure tank,
        ii. First BPT to second BPT, and
        iii. Second BPT to discharge tank
    c. Distribution system

viii. A sample excel sheet of pipeline design is presented in *Table **5-18*** .

5.9 Canals along unstable areas

By virtue of the unique topography, fragile geology and monsoon hydrology, Bhutan's hill slopes are often unstable due to debris flow, landslides and river bank erosion. The irrigation canals passing along the slopes of the hills and mountains in many cases are susceptible to instability of the alignment. There are three types of instability that can affect the canal, which are also expressed in *Figure **5-22*** .

5.9.1 Deep slide across the canal alignment

Landslides and overburden creep affect the whole hillside for large areas above and below the canal, and more or less independently of the influence of canal construction. In such areas, trees have often slipped, and the upper part of the tree trunk grows vertically while the lower part is sloping. Sometimes the ground will be fairly broken with little dense vegetation.

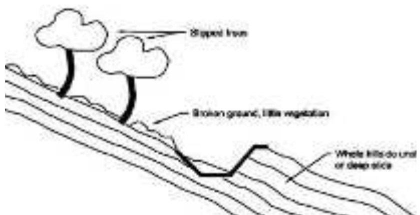
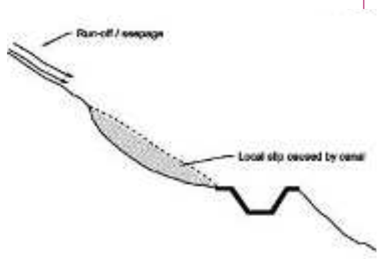
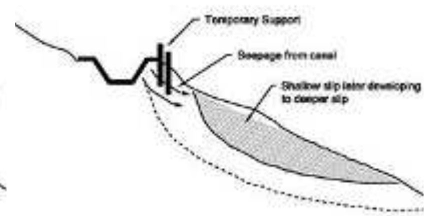
5.9.2 Shallow slide above the canal

A shallow slide above the canal is due to the effect of the canal cutting causing its own instability. For existing canals, the ground will have often slumped into the canal and there will sometimes be groundwater seepage.

5.9.3 Shallow slide below canal

The possibility of instability below the canal is due to steepness and particularly excess pore water from the canal itself. In existing canals, such areas are usually fairly obvious; the canal size is usually restricted and has been temporarily stabilized by bamboo or boulders, while continuing seepage through or over the canal banks usually only worsen the situation.

Figure 5-22 Three types of canal instability

<table>
<tr><th>Deep slide across the canal</th><th>Shallow slide above the canal</th><th>Shallow slide below the canal</th></tr>
<tr><td>
</td><td>
</td><td>
</td></tr>
</table>

The general solutions for unstable areas of the canal alignment are grouped into five categories:

- Lining of the canal,
- Covered canal,
- Retaining walls on uphill side as well as downhill side,
- Temporary measures, and
- Reforestation and grassing

These solution measures are applicable to the specific site conditions of the above mentioned canal instabilities. The typical instability problems and their solutions are given in *Figure* ***5-23***.

Figure 5-23 Canal instability problems and their solutions

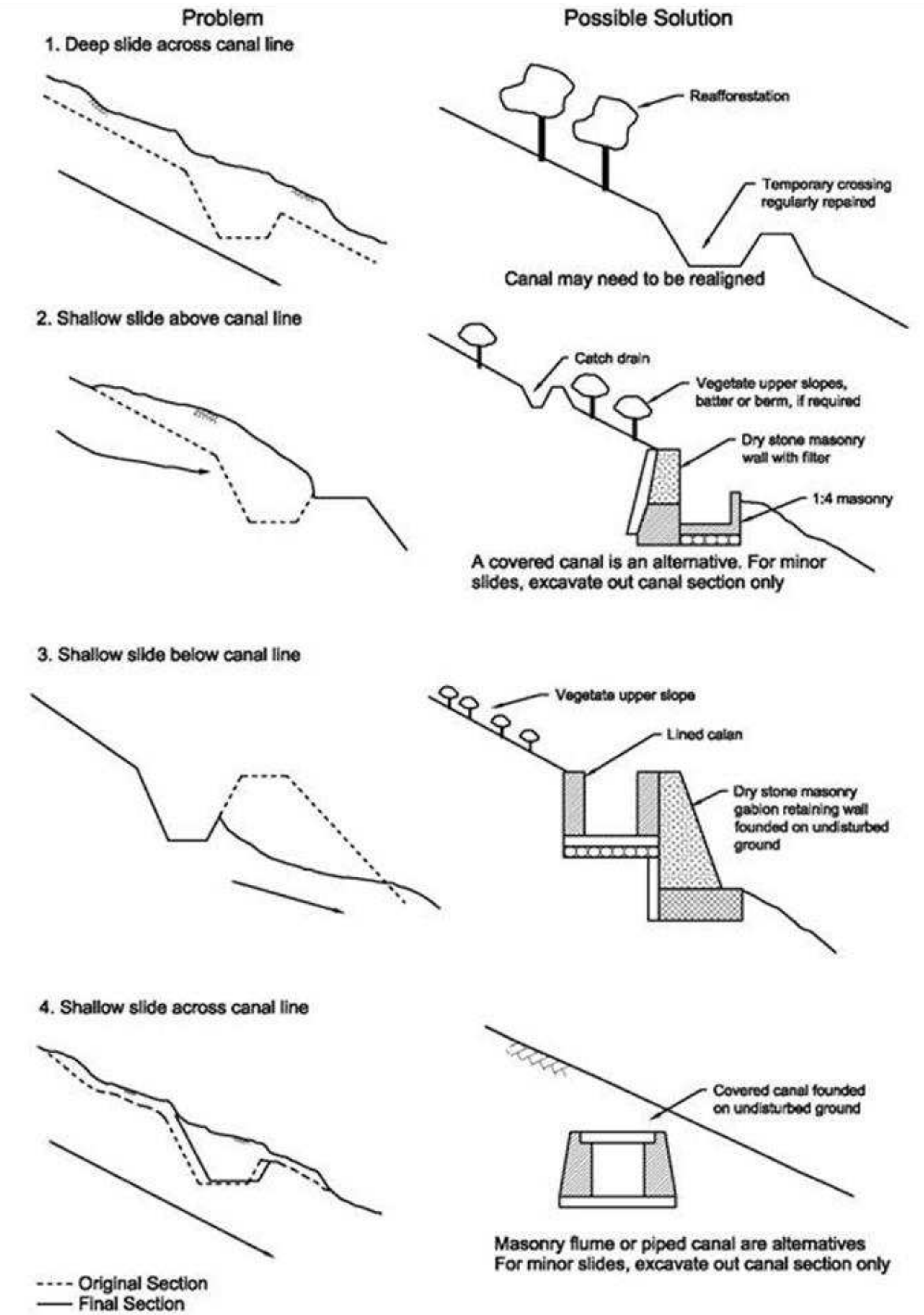


Table 5-18 Sample design sheet of pipeline works

Project:

Scheme name: Source name: Safe yield:

S.N	Pipe line		Length		Design discharge	Reduced level		Level Diff	New static level	Total avail head	Max static pressure	Pipe used HDPE			Friction factor	Head loss	Residual head	Flow velocity	Hydraulic grade line	
	From	To	Actual	Design		From	To					OD	ID	Class					From	To
			(m)	(m)	(lps)	(m)	(m)	(m)	(m)	(m)	(m)	(mm)	(mm)	(kg/cm2)	%	(m)	(m)	(m/s)	(m)	(m)
(0)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(17)	(18)	(19)	(20)	(21)	(22)
1	CC	P1	250	275	4	1000	993	7	1000	7.00	7	90	76.3	6	1.15	3.16	3.838	0.88	1000	996.8375
2	P1	P2	250	275	4	993	947	46	1000	49.84	53	90	76.3	6	1.15	3.16	46.675	0.88	996.8375	993.675
3	P2	P3	220	242	4	947	955	-8	1000	38.68	45	90	76.3	6	1.15	2.78	35.892	0.88	993.675	990.892
4	P3	P4	100	110	4	955	956	-1	1000	34.89	44	90	76.3	6	1.15	1.27	33.627	0.88	990.892	989.627
5	P3	P4	200	220	4	956	947	9	1000	42.63	53	90	76.3	6	1.15	2.53	40.097	0.88	989.627	987.097
6	P4	P5	200	220	4	947	943	4	1000	44.10	57	90	76.3	6	1.15	2.53	41.567	0.88	987.097	984.567
7	P5	FRC	200	220	4	943	942	1	1000	42.57	58	90	76.3	6	1.15	2.53	40.037	0.88	984.567	982.037