SSIGL 15

NATIONAL GUIDELINES

For Small Scale Irrigation Development in Ethiopia

Surface Irrigation System Planning and Design

November 2018

Addis Ababa

MINISTRY OF AGRICULTURE

National Guidelines for Small Scale Irrigation Development in Ethiopia

SSIGL 15: Surface Irrigation System Planning and Design

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Ministry of Agriculture
Small-Scale Irrigation Development Directorate
P. O. Box 62347
Tel: +251-1-6462355
Fax: +251-1-6462355
Email: SSIDdirectorate@moa.gov.et
SSIDdirectorate@gmail.com
eDMS (intranet): MoA SSID DMS (http://172.28.1.188:8080/DMS/login.jsp)
Website: www.moa.gov.et

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FORWARD

Ministry of Agriculture, based on the national strategic directions is striving to meet its commitments in which modernizing agriculture is on top of its highest priorities to sustain the rapid, broad-based and fair economic growth and development of the country. To date, major efforts have been made to remodel several important strategies and national guidelines by its major programs and projects.

While efforts have been made to create access to irrigation water and promoting sustainable irrigation development, several barriers are still hindering the implementation process and the performance of the schemes. The major technical constrains starts from poor planning and identification, study, design, construction, operation, and maintenance. One of the main reasons behind this outstanding challenge, in addition to the capacity limitations, is that SSIPs have been studied and designed using many ad-hoc procedures and technical guidelines developed by various local and international institutions.

Despite having several guidelines and manuals developed by different entities such as MoA (IDD)-1986, ESRDF-1997, MoWIE-2002 and JICA/OIDA-2014, still the irrigation professionals follow their own public sources and expertise to fill some important gaps. A number of disparities, constraints and outstanding issues in the study and design procedures, criteria and assumptions have been causing huge variations in all vital aspects of SSI study, design and implementation from region to region and among professionals within the same region and institutions due mainly to the lack of agreed standard technical guidelines. Hence, the SSI Directorate with AGP financial support, led by Generation consultant (GIRDC) and with active involvement of national and regional stakeholders and international development partners, these new and comprehensive national guidelines have been developed.

The SSID guidelines have been developed by addressing all key features in a comprehensive and participatory manner at all levels. The guidelines are believed to be responsive to the prevalent study and design contentious issues; and efforts have been made to make the guidelines simple, flexible and adaptable to almost all regional contexts including concerned partner institution interests. The outlines of the guidelines cover all aspects of irrigation development including project initiation, planning, organizations, site identification and prioritization, feasibility studies and detail designs, contract administration and management, scheme operation, maintenance and management.

Enforceability, standardization, social and environmental safeguard mechanisms are well mainstreamed in the guidelines, hence they shall be used as a guiding framework for engineers and other experts engaged in all SSI development phases. The views and actual procedures of all relevant diverse government bodies, research and higher learning institutions, private companies and development partners has been immensely and thoroughly considered to ensure that all stakeholders are aligned and can work together towards a common goal. Appropriately, the guidelines will be familiarized to the entire stakeholders working in the irrigation development. Besides, significant number of experts in the corresponding subject matter will be effectively trained nationwide; and the guidelines will be tested practically on actual new and developing projects for due consideration of possible improvement. Hence, hereinafter, all involved stakeholders including government & non-governmental organizations, development partners, enterprises, institutions, consultants and individuals in Ethiopia have to adhere to these comprehensive national guidelines in all cases and at all level whilst if any overlooked components are found, it should be documented and communicated to MOA to bring them up-to-date.

Therefore, I congratulate all parties involved in the success of this effort, and urge partners and stakeholders to show a similar level of engagement in the implementation and stick to the guidelines over the coming years.

H.E. Dr. Kaba Urgessa
State Minister, Ministry of Agriculture

SMALL SCALE IRRIGATION DEVELOPMENT VISION

Transforming agricultural production from its dependence on rain-fed practices by creating reliable irrigation system in which smallholder farmers have access to at least one option of water source to increase production and productivity as well as enhance resilience to climate change and thereby ensure food security, maintain increasing income and sustain economic growth.

ACKNOWLEDGEMENTS

The preparation of SSIGLs required extensive inputs from all stakeholders and development partners. Accordingly many professionals from government and development partners have contributed to the realization of the guidelines. To this end MOA would like to extend sincere acknowledgement to all institutions and individuals who have been involved in the review of these SSIGLs for their comprehensive participation, invaluable inputs and encouragement to the completion of the guidelines. There are just too many collaborators involved to name exhaustively and congratulate individually, as many experts from Federal, regional states and development partners have been involved in one way or another in the preparation of the guidelines. The contribution of all of them who actively involved in the development of these SSIGLs is gratefully acknowledged. The Ministry believes that their contributions will be truly appreciated by the users for many years to come.

The Ministry would like to extend its appreciation and gratitude to the following contributors:

- Agriculture Growth Program (AGP) of the MoA for financing the development and publication of the guidelines.
- The National Agriculture Water Management Platform (NAWMP) for overseeing, guidance and playing key supervisory and quality control roles in the overall preparation process and for the devotion of its members in reviewing and providing invaluable technical inputs to enrich the guidelines.
- Federal Government and Regional States organizations and their staff for their untiring effort in reviewing the guidelines and providing constructive suggestions, recommendations and comments.
- National and international development partners for their unreserved efforts in reviewing the guidelines and providing constructive comments which invaluably improved the quality of the guidelines.
- Small-scale and Micro Irrigation Support Project (SMIS) and its team for making all efforts to have quality GLs developed as envisioned by the Ministry.

The MOA would also like to extend its high gratitude and sincere thanks to AGP's multi development partners including the International Development Association (IDA)/World Bank, the Canada Department of Foreign Affairs, Trade and Development (DFATD), the United States Agency for International Development (USAID), the Netherlands, the European Commission (EC), the Spanish Agency for International Development (AECID), the Global Agriculture and Food Security Program (GAFSP), the Italy International Development Cooperation, the Food and Agriculture Organization (FAO) and the United Nations Development Program (UNDP).

Moreover, the Ministry would like to express its gratitude to Generation Integrated Rural Development Consultant (GIRDC) and its staff whose determined efforts to the development of these SSIGLs have been invaluable. GIRDC and its team drafted and finalized all the contents of the SSIGLs as per stakeholder suggestions, recommendations and concerns. The MoA recognizes the patience, diligence, tireless, extensive and selfless dedication of the GIRDC and its staff who made this assignment possible.

Finally, we owe courtesy to all national and International source materials cited and referred but unintentionally not cited.

Ministry of Agriculture

DEDICATIONS

The National Guidelines for Small Scale Irrigation Development are dedicated to Ethiopian smallholder farmers, agro-pastoralists, pastoralists, to equip them with appropriate irrigation technology as we envision them empowered and transformed.

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ACRONYMS

a.s.l.	Above sea level
AGP	Agricultural Growth Program
avg.	Average
Ax	Canal Cross-sectional Area
B	Bank top width
b	Canal Bed Width
BC	Branch Canal
BoA	Bureau of Agriculture,
BOQ	Bill of Quantities
BSR	Balancing Storage Reservoir
CA	Command Area
CAD	Computer Aided Design
CBL	Canal Bed Level
CD	Collector Drain/ Cross Drain
CD	Collector Drains
Ctd.	Continued
CWR	Crop Water Requirement
D	Canal depth including flow depth & FB
d	Flow depth
DBL	Drain Bed Level
DEM	Digital Elevation Model
ERA	Ethiopian Road Authority
ETB	Ethiopian Birr
ETo	Reference Evapotranspiration
F	Furrow
FAO	Food and Agriculture Organization of the United Nations
FB	Free Board
FC	Field Canal or Field Capacity
FD	Field Drain
FD	Field Drains

FPE	Flood Protection Embankment
fps	foot per second
FS	Full Supply or Feasibility Study
FSD	Feasibility Study and Detail design
FSL	Full Supply Level
GIRDC	Generation Integrated Rural Development Consultant
GIS	Geographic Information System
GIWR	Gross Irrigation Water Requirement
GL	Guideline
GPS	Global Positioning System
GTZ	Gesellschaft für Technische Zusammenarbeit
ha	hectare
HFL	High Flood Level
hr	hour
I&D	Irrigation and Drainage
IC	Irrigation Cycle
ICSAA	Irrigation Construction and Scheme Administration Agency
ID	Interceptor/catch Drains
INCID	Indian National Committee on Irrigation and Drainage
IWR	Irrigation Water Requirement
l/s	Liter per second
l/s/ha	Liter per second per hectare
LB	Left Bank
LMC	Left side Main Canal
LS	Left Side
LSIDP	Large Scale Irrigation and Drainage Project
m	Canal side slope
Max.	Maximum
MC	Main Canal
MC	Main Canal
MD	Main Drain
MFL/ DFL	Maximum/Design Flood Level

Min.	Minimum
MoWR	Ministry of Water Resource
mps	meter per second
n	Manning's roughness coefficient
NIWR	Irrigation Water Requirement
Nr	Number
NSR	Night Storage Reservoir
O&M	Operation and Maintenance
OGL	Original Ground Level
OP	Option
p	Wetted perimeter
PRD	Partial Root zone Drying
PVC	Poly Venile Chloride
PWP	Permanet Wilting Point
Qd	Design Discharge
Qty.	Quantity
R	Hydraulic radius
RASM	Readily Available Soil Moisture
RB	Right Bank
RBL	River Bed Level
RCC	Reinforced Concrete
RDI	Regulated Deficit Irrigation
RMC	Right side Main Canal
RS	Right Side
S	Canal bed slope
S.F	Safety Factor
SB	Stilling Basin
SBL	Stilling Basin Level
SC	Secondary Canal
SCS	Soil Conservation Service
SLM	Sustainable Land Management
SMU	Soil Mapping Unit

SSID	Small Scale Irrigation Development
SSIGL	Small Scale Irrigation Guideline
SSIP	Small Scale Irrigation Project
SSIP	Small Scale Irrigation and Drainage Project
SSIS	Small Scale Irrigation Scheme
T	Canal top width
TC	Tertiary Canal
TD	Tertiary Drain
TD	Tertiary Drains
TEL	Total Energy Level,
TOR	Terms of Reference
TWD	Tail Water Depth
V	Flow velocity
Vs.	Versus
WB	World Bank
WCL	Weir Crest Level
WL	Water Level
WR	Water Requirement
WUA	Water Users Association

PREFACE

While irrigation development is at the top of the government's priority agendas as it is key to boost production and improve food security as well as to provide inputs for industrial development. Accordingly, irrigated land in different scales has been aggressively expanding from time to time. To this end, to enhance quality delivery of small-scale irrigation development planning, implementation and management, it has been decided to develop standard SSI guidelines that must be nationally applied. In September 2017 the Ministry of Agriculture (MoA) had entrusted Generation Integrated Rural Development Consultant (GIRDC) to prepare the National Small-scale Irrigation Development Guidelines (SSIGLs).

Preparation of the SSIGLs for enhancing development of irrigated agriculture is recognized as one of the many core initiatives of the MoA to improve its delivery system and achieve the targets in irrigated agriculture and fulfill its mission for improving agricultural productivity and production. The core objective of developing SSIGLs is to summarize present thinking, knowledge and practices to enable irrigation practitioners to properly plan, implement and manage community managed SSI schemes to develop the full irrigation potential in a sustainable manner.

As the SSIGLs are prepared based on national and international knowledge, experiences and practices, and describe current and recommended practice and set out the national standard guides and procedures for SSI development, they serve as a source of information and provide guidance. Hence, it is believed that the SSIGLs will contribute to ensuring the quality and timely delivery, operation and maintenance of SSI schemes in the country. The SSIGLs attempt to explain and illustrate the important concepts, considerations and procedures in SSI planning, implementation and management; and shall be used as a guiding framework for professionals engaged in SSI development. Illustrative examples from within the country have been added to enable the users understand the contents, methodologies presented in the SSIGLs.

The intended audiences of the SSIGLs are government organizations, NGOs, CSOs and the private sector involved in SSI development. Professionally, the SSIGLs will be beneficial for experienced and junior planners, experts, contractors, consultants, suppliers, investors, operators and managers of SSI schemes. The SSIGLs will also serve as a useful reference for academia and researchers involved and interested in SSI development. The SSIGLs will guide to ensure that; planning, implementation and management of SSI projects is formalized and set procedures and processes to be followed. As the SSIGLs provide information and guides they must be always fully considered and applied by adapting them to the local specific requirements.

In cognizance with the need for quality SSIGLs, the MoA has duly considered quality assurance and control during preparation of the guidelines. Accordingly, the outlines, contents and scope of the SSIGLs were thoroughly discussed, reviewed and modified by NAWMP members (senior professionals from public, national and international stakeholder) with key stakeholders in many consultative meetings and workshops. Moreover, at each milestone of SSIGL preparation, resource persons from all stakeholders reviewed and confirmed that SSIGLs have met the demands and expectations of users.

Moreover, the Ministry has mobilized resource persons from key Federal, National Regional States level stakeholders and international development partners for review, validation and endorsement of the SSIGLs.

Several hundreds of experienced professionals (who are very qualified experts in their respective fields) from government institutions, relevant private sector and international development partners have significantly contributed to the preparation of the SSIGLs. They have been involved in all aspects of the development of SSIGLs throughout the preparation process. The preparation process included a number of consultation meetings and workshops: (i) workshop to review inception report, (ii) workshop on findings of review of existing guidelines/manuals and proposed contents of the SSIGLs, (iii) meetings to review zero draft SSI GLs, (iv) review workshop on draft SSI GLs, (v) small group review meetings on thematic areas, (vi) small group consultation meetings on its final presentation of contents and layout, (vii) consultation mini-workshops in the National States on semi-final versions of the SSIGLs, and (viii) final write-shop for the appraisal and approval of the final versions of SSIGLs.

The deliberations, concerns, suggestions and comments received from professionals have been duly considered and incorporated by the GIRD Consultant in the final SSIGLs.

There are 34 separate guidelines which are categorized into the following five parts concurrent to SSI development phases:

Part-I. Project Initiation, Planning and Organization Guideline which deals with key considerations and procedures on planning and organization of SSI development projects.

Part-II. Site Identification and Prioritization Guideline which treats physical potential identification and prioritization of investment projects. It presents SSI site selection process and prioritization criteria.

Part-III. Feasibility Study and Detail Design Guidelines for SSID dealing with feasibility study and design concepts, approaches, considerations, requirements and procedures in the study and design of SSI systems.

Part-IV. Contract Administration and Construction Management Guidelines for SSI development presents the considerations, requirements, and procedures involved in construction of works, construction supervision and contract administration.

Part-V. SSI Scheme Management, Operation and Maintenance Guidelines which covers SSI Scheme management and operation.

Moreover, Tools for Small Scale Irrigation development are also prepared as part of SSIGLs.

It is strongly believed and expected that; the SSIGLs will be quickly applied by all stakeholders involved in SSI development and others as appropriate following the dissemination and familiarization process of the guidelines in order to ensure efficient, productive and sustainable irrigation development.

The SSIGLs are envisioned to be updated by incorporating new technologies and experiences including research findings. Therefore, any suggestions, concerns, recommendations and comments on the SSIGLs are highly appreciated and welcome for future updates as per the attached format below. Furthermore, despite efforts in making all types of editorial works, there may still errors, which similarly shall be handled in future undated versions.

UPDATING AND REVISIONS OF GUIDELINES

The GLs are intended as an up-to-date or a live document enabling revisions, to be updated periodically to incorporate improvements, when and where necessary; may be due to evolving demands, technological changes and changing policies, and regulatory frameworks. Planning, study and design of SSI development interventions is a dynamic process. Advancements in these aspects are necessary to cope up with the changing environment and advancing techniques. Also, based on observation feedbacks and experiences gained during application and implementation of the guidelines, there might be a need to update the requirements, provisions and procedures, as appropriate. Besides, day-by-day, water is becoming more and more valuable. Hence, for efficient water development, utilization and management will have to be designed, planned and constructed with a new set up of mind to keep pace with the changing needs of the time. It may, therefore, be necessary to take up the work of further revision of these GLs.

This current version of the GLs has particular reference to the prevailing conditions in Ethiopia and reflects the experience gained through activities within the sub-sector during subsequent years. This is the first version of the SSI development GLs. This version shall be used as a starting point for future update, revision and improvement. Future updating and revisions to the GLs are anticipated as part of the process of strengthening the standards for planning, study, design, construction, operation and management SSI development in the country.

Completion of the review and updating of the GLs shall be undertaken in close consultation with the federal and regional irrigation institutions and other stakeholders in the irrigation sub-sector including the contracting and consulting industry.

In summary, significant changes to criteria, procedures or any other relevant issues related to technological changes, new policies or revised laws should be incorporated into the GLs from their date of effectiveness. Other minor changes that will not significantly affect the whole nature of the GLs may be accumulated and made periodically. When changes are made and approved, new page(s) incorporating the revision, together with the revision date, will be issued and inserted into the relevant GL section.

All suggestions to improve the GLs should be made in accordance with the following procedures:

I. Users of the GLs must register on the MOA website: Website: **www.moa.gov.et**
II. Proposed changes should be outlined on the GLs Change Form and forwarded with a covering letter or email of its need and purpose to the Ministry.
III. Agreed changes will be approved by the Ministry on recommendation from the Small-scale Irrigation Directorate and/or other responsible government body.
IV. The release date of the new version will be notified to all registered users and authorities.

Users are kindly requested to present their concerns, suggestions, recommendations and comments for future updates including any omissions and/or obvious errors by completing the following revisions form and submitting it to the Ministry. The Ministry shall appraise such requests for revision and will determine if an update to the guide is justified and necessary; and when such updates will be published. Revisions may take the form of replacement or additional pages. Upon receipt, revision pages are to be incorporated in the GLs and all superseded pages removed.

Suggested Revisions Request Form (Official Letter or Email)

To: ---
From: ---
Date: ---
Description of suggested updates/changes: Include GL code and title, section title and # (heading/subheading #), and page #.

GL Code and Title	Date	Sections/ Heading/Subheading/ Pages/Table/Figure	Explanation	Comments (proposed change)

Note that be specific and include suggested language if possible and include additional sheets for comments, reference materials, charts or graphics.

GLs Change Action

Suggested Change	Recommended Action	Authorized by	Date

Director for SSI Directorate**:** ________________________**Date:** _________________

The following table helps to track initial issuance of the guidelines and subsequent Updates/Versions and Revisions (Registration of Amendments/Updates).

Revision Register

Version/Issue/Revision No	Reference/Revised Sections/Pages/topics	Description of revision (Comments)	Authorized by	Date

1 INTRODUCTION

1.1 OBJECTIVE AND SCOPE OF THIS GUIDELINE

The objective of this guideline, i.e. Guideline for Study and Design of Surface Irrigation System, is therefore aims to provide well organized and comprehensive user friendly manual tailored with the current design practice being exercised in our country on planning, study and design of surface irrigation system including surface drainage system as related to the irrigation application system.

As the purpose of the guideline is for the aid of small scale irrigation scheme design, its scope is also limited to the planning, study and design of surface irrigation system including surface drainage system for Small Scale irrigation Schemes. The guideline is supposed to assist decision-makers, planners, various experts and concerned bodies to maintain a degree of consistency and uniformity of approach in the study of irrigation projects in Ethiopia. This will in tum assure scope and levels of detail required at various stages of project development as well as facilitate project appraisals with the aid of basic concept demonstration, worked example and supplemented with design aid templates.

1.2 DEFINITIONS OF TECHNICAL TERMINOLOGIES

Base flow: is the sustained or dry weather flow of streams resulting from the outflow of permanent or perched groundwater, and from the drainage of lakes and swamps.

Base period: is the time between the first watering of a crop at the time of its sowing to its last watering before harvesting.

Canal: is a long thin stretch of artificially made network of waterways for taking water from higher area to lower area;

Catch drain/CD: it is also called interceptor drain or trench and is a drainage family used for collecting runoff coming from outside in the form of sheet flow to the command area thus located on outer side of MCs aside of access road running along main canals.

Channel: is an open natural channel system whose bed and banks confine the surface flow of a natural or artificial drainage or stream;

Crop period: Is the time period that elapses from the instant of crop sowing to the instant of its harvesting.

Crop Water Requirement (ETc=Kc*ETo): Water requirement of crop is the quantity of water regardless of source, needed for normal crop growth and yield in a period of time at a place and may be supplied by precipitation or by irrigation or by both.

Critical Depth: is a depth at which water flows over a weir; this depth being attained automatically where no backwater forces are involved. It is the depth at which the energy content of flow is a minimum;

Delta: Is total depth of water (in cm) required by a crop to come to its maturity.

Drip irrigation application: Also called **trickle irrigation** is an irrigation method using a system of perforated plastic pipes laid along the ground at the base of a row of plants. In its more advanced form, it is a micro-irrigation system in which water flow is very low, generally less than 8 l/hr. and without pressure, i.e. drop by drop. The water emerging infiltrates directly into the soil where it wets a volume of soil called bulb;

Duty of water (l/s/ha): This is the relation between areas irrigated, or to be irrigated, and the quantity of water used, or required to irrigate it for the purpose of maturing its crop;

Effective rooting depth: Soil depth from which the crop extracts most of the water needed for evapotranspiration, (also called design rooting depth);

Field Canal/FC: Also called field ditches, is the fourth order canal family and receive water from Tertiary Canal and distribute it among furrows.

Field Drain/FD: This drain family collects drainage flow which comes from the farm plots and conveys it to the corresponding Tertiary Drain or collector drain.

Free board: is a vertical distance between canal full supply level and formation level of canal bank i.e. embankment level.

Froude number: It is a ratio which is proportional to the square root of the ratio of the inertial forces over the weight of fluid. The Froude number is used generally for scaling free-surface flows, open channels and hydraulic structures;

Hydraulic jump: It is a hydraulic characteristic occurring at transition from a rapid or supercritical flow to a slow or subcritical flow motion;

Hydraulic unit: is a unit or blocks of land bounded based on water related boundaries such as secondary unit, tertiary unit, field unit, etc.

Gross Command area: is the potential land resource which is found within the proposed main canal boundary or outside of it if pumping is introduced. Gross command area includes all type of lands irrespective to land suitability to irrigated agriculture. The main canal routes determine the size of gross command area if pumping system is not inclusive;

Gross irrigation water requirement (GIWR=NIWR/Efficiency): is the amount of water to be extracted (by diversion, pumping) and applied to the irrigation scheme. It includes NIWR plus water losses.

Irrigation: It is the measured artificial application of water applied to irrigable lands to supply crop requirements that is not satisfied by rainfall. It is thus, nothing but a continuous and a reliable water supply to crops in accordance with their different needs throughout the crop period;
Irrigation Interval/Cycle: It is the time interval in days between successive irrigations of same block or crop.

Irrigation time/duration/hour: is the time required to supply the required gross depth of irrigation daily in mm.

Irrigation frequency: Irrigation frequency is defined as the frequency of applying water to a particular crop at a certain stage of growth and is expressed in days (FAO, 2002). Thus, it can be interchangeably used with irrigation interval**.**

Irrigation System Layout: is a systematic arrangement of irrigation system including command area boundaries, alignments of irrigation canals and network of natural drainage channels and related infrastructures starting from headwork up to outfall.

Irrigation Water Requirement (IWR): is the quantity of water required by a crop in a given period of time for normal growth under field conditions. It includes evaporation and other unavoidable wastes. Usually water requirement for crop is expressed in water depth per unit area.

Main Canal/MC: This is the SSIP canal families that is designed to serve for conveying irrigation water from the source to the distributary canal within the command area. It operates continuous for the whole day of the irrigation season or on a supply schedule based on amount of flow in the source of supply.

Main Drain/MD: is a drainage outlet system commonly of natural drains receiving drainage water from collector drains and pass it to the outlet such as main river course or lakes and the like.

Net Command area: is the net irrigable land identified from the gross command area based on land suitability, reduced irrigation and social infrastructure structures;

Net Irrigation Requirement: This is the crop's irrigation need excluding losses of any kind and is expressed as a layer of water in mm or cm per day, month or other period of time. It is also called Net Irrigation Water Requirement (NIWR) to express quantity of water necessary for crop growth in millimeters per year or in m^3/ha/year.

Secondary Canal/SC: This is a distributary canal which shares water among TCs. It is relatively smaller in capacity if continuous but similar size as MC if rotational at secondary canal level.

Secondary Drain/SD: Also called collector drain and is defined as an artificial or natural drain which collects drainage flow from tertiary drains and convey it to the outlet or main drain.

Sediment: Any material carried in suspension by the flow or as bed-load which would settle to the bottom of hydraulic structures in the absence of flow;

Small Scale Irrigation Project: is an irrigation project which is managed by smallholders and size of command is about 200ha. If the SSI Project development is in phases, then this limit can be exceeded.

Sprinkler irrigation application: Is a method of irrigation under pressure in which water is dispersed in the form of artificial rain through lines carrying distribution components: rotary sprinklers, diffusers with permanent water streams, perforated pipes

Stream size: It is the maximum amount of water/flow rate that can conveniently be handled by one farmer or irrigator in a furrow outlet. It is commonly in the range of 15 to 50 l/s depending on furrow slope.

Surface irrigation application: Is a method of irrigation in which water is applied to the land by allowing it to flow by simple gravity, before infiltrating. It includes various systems depending upon the relative magnitude of the surface flooding phase and infiltration phase after accumulation (submersion);

Surface irrigation design: This is the process and procedure for matching the most desirable irrigation frequency and depth of irrigation and the capacity and availability of the water supply;

Survey Area: Survey area is defined in this Guideline as the area covered for surveying the land resources to secure adequate gross command area for irrigated agriculture. In most cases, the area covered for topographic survey considered as project survey area

Tertiary Canal/TC: is the third order canal family and distribute water among field canals receiving it mostly from secondary canal and rarely from main canal.

Tertiary Drain/TD: Such drain family collects drainage flow from field drains and convey it to the secondary drain or collector drain.

2 PLANNING OF IRRIGATION AND DRAINAGE SYSTEM

2.1 BASIC QUESTIONS FOR PLANNING IRRIGATION SYSTEM

These days, application of irrigation water for improving production and productivity of crops has been becoming obligatory for survival especially in food insecure areas. To intensify such irrigation systems and optimize their application in the fields, it is essential to use proper methods and techniques. However, before we apply such irrigation, we should provide adequate answers to the following questions:

- Where to irrigate (place)?
- When to irrigate (time)?
- How much water is required to irrigate (quantity)?
- With which water we irrigate (quality)?
- How to irrigate (distribution patterns)?
- With what to irrigate (equipment)?

Such irrigation water can of-course be abstracted either from surface water (rivers, lakes, dams/reservoirs) or groundwater (springs, shallow wells or deep boreholes). It can be abstracted from these sources by different headwork structures (such as Diversion Weir, Intake structure, Pump, Spring Protection or Development and Micro Dam) and applied by different irrigation application systems. The headwork part have been treated in separate parts of this Guideline, thus this Guideline is concentrated with irrigation planning and one of the common irrigation application methods which have been practically implemented in our country: i.e. Surface, Sprinkler and Drip application methods.

These methods are considered one by one in different sections of this part of Guideline. (Note structures, drainage and flood protection design aspects are also treated in separate portions of this Guideline).

2.2 IRRIGATION SYSTEM PLANNING PROCESS

2.2.1 Needs of planning process

An irrigation system needs to comprise both canals to bring the water to the fields and natural or man-made drains to take away excess irrigation or rainfall water. Without adequate drainage, both crop growth and farm operation will be impaired. Thus, the planning process for selecting irrigation system requires an inventory of the resources available and identification of natural drainage network. The evaluation of these resources is necessary to identify the production potentials, and the physical and operational constraints, which affect the selection of viable design alternative of irrigation systems.

The analysis and comparison of these alternatives provides a basis for selecting the irrigation system design. Since reliable water resources of this country are ever changing/minimizing much over the years, the water resource planners have to make cautious decisions on optimizing the available resources for maximum benefit.

In planning irrigation systems, slope is important in determining the type of irrigation system that best suits for each project site. It is thus important in determining optimum and maximum water application rates (or stream flows). Erosion potential from excessive surface irrigation flow increases as the slope and slope length increase. Potential runoff from sprinkler systems also increases as the slope increases, thus raising the opportunity for erosion to occur. To avoid runoff from sprinklers, correction factors to infiltration rate for different slopes need to be introduced during the design process.

Irrigation project planning and preparations for a particular area include:

- Selection of a general layout for subdividing and irrigating the area in units of suitable dimensions based on topographic condition of the site;
- Adaptation of the designed distribution system that will permit deliveries of water to the different units; and
- Grading of field surfaces and construction of such features as may be needed during applications of water. All three items are more or less interrelated and should be thoroughly considered before adopting a definite plan and beginning of construction.

Irrigation planning needs the following major procedures:

- Selecting methods of delivering water to the field units;
- Determining irrigation frequencies for that season; and
- Depths of water applied per irrigation that depends principally on soil conditions, types of crops produced and amounts of existing soil moisture that may become available for plant growth as a result of rainfall.

Boundary of irrigable area need to be fixed at this stage based on relative location of headwork and command area, topographic situations/slope/, and SMU map, etc.

2.2.2 Farmers' participation in scheme planning and design

Beneficiaries need to play their own role in scheme planning through participation on following areas, but not limited to:

- Farmers could provide information on past experience with floods, point out areas with potential for flooding, and suggest to the planners locations for structures such as water abstraction from the river, hence preventing pumping station from being flooded,
- Farmers should select or participate in identification of the planned land to be irrigated and the irrigation agency should assist them by assessing suitability of those lands,
- Communities within the area to be developed should participate in Environmental Impact assessment (EIA) for the project, through contributing vital information, such as current uses of their natural resources, ecology, human health, etc.
- Farmers should provide labour for topographic, soil and socio-economic surveys. They should, through their committees, decide who should do which activity,
- Farmers should agree on selected/proposed crops & the agency should guide them only on technical matters related to suitability of such crops for climate, soils, cost of production & expected returns & marketing potential of these crops,
- Farmers shall participate on the selection of site for social infrastructure, such as cattle trough, washing basin, foot path, etc.
- The irrigation agency should facilitate exposure of farmers to various irrigation methods & enlighten them as to the advantages & disadvantages of each. Farmers then should agree on the irrigation methods they would prefer to be considered during irrigation design,
- The prospective irrigators should suggest plot sizes they would prefer to irrigate and the irrigation agency should provide information on management, labour & input costs required for different plot sizes, as well as on potential of land and water resources to satisfy the various sizes,

- After completing the designs, the irrigation agency should explain alternative designs to farmers and implications of each vis-à-vis land redistribution, water resources potential, plot sizes & total area to be irrigated, cropping programmes, labour requirements, capital costs, operation and maintenance costs, environmental aspects, land use patterns and other considerations,
- Finally, the farmers shall decide which option to adopt.

2.3 COMPONENTS OF SURFACE IRRIGATION SYSTEM TO BE CONSIDERED IN PLANNING

2.3.1 Minimum constituents of arrangements in surface irrigation system

Normally, an irrigation system layout contains irrigation distribution canals including associated hydraulic structures, and a drainage ditch and road network. Any surface irrigation system of SSI project consists of at least water sources: intake facilities; conveyance system; water storage facilities (if required), secondary canals, tertiary canals, field canal and/or pipe system; infield water use system; drainage system and accessibility and related infrastructures.

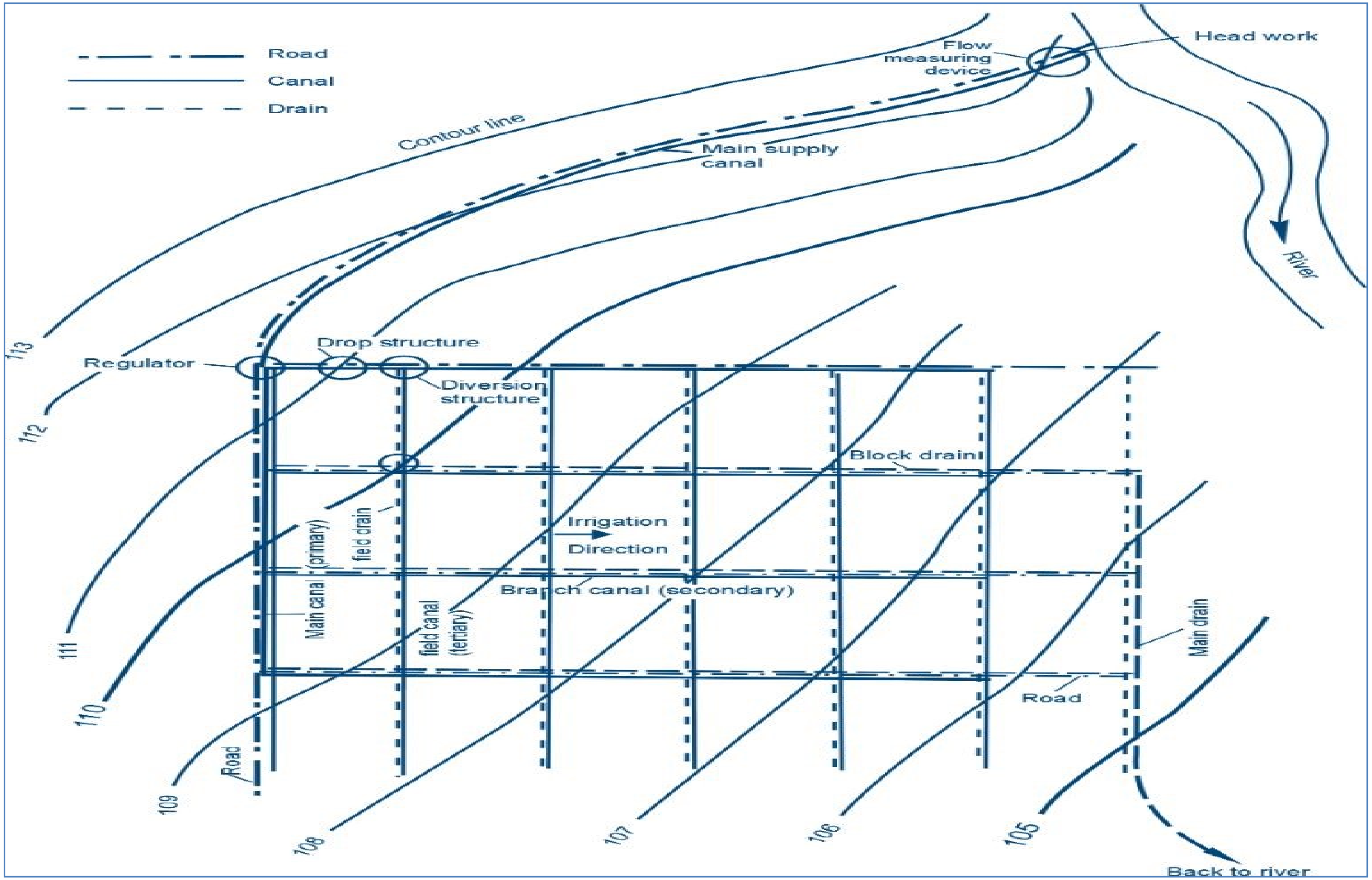


Figure 2-1: Components of typical surface irrigation system
Source: FAO, 2006

2.3.2 Water source & analysis of its availability

Water source and its availability in quantity and quality need to be collected from hydrology study report. Data which need to be gathered are monthly base/lean/dependable flows, mean monthly flows, design floods of cross drains, drainage module and sediment conditions of source of irrigation water supply river, or others.

Water planning is a wise use of water among different stakeholders in the upstream and downstream and the project area. It is simply balancing the demand and supply of water budget for safe use without causing conflicts among different beneficiaries especially during peak water demand periods.

Thus, water abstraction on the upstream and downstream of the anticipated diversion site along reaches of source of this supply river should be studied. Estimated mean monthly lean flow of a river used for supply source shall be summarized here and water balance shall be done against the available water.

In some regions with a concentrated high demand requiring major water resource development, the engineer may have a choice of sources between surface water from nearby amply watered mountainous catchments and groundwater from a thick water bearing stratum of good transmissivity. It may be technically expedient, (with regard to the reliability of yields) and economically viable to develop both sources and use them jointly to best effect.

As per FAO, 2006, conjunctive use involves the coordinated and planned utilization of both surface water and groundwater resources to meet water requirements in a manner where water is conserved. In a conjunctive scheme, during periods of above normal rainfall surface water is utilized to the maximum extent possible and, where feasible, artificially recharged (pumped into aquifers through wells known as injection wells) into the aquifer to augment groundwater storage and raise groundwater levels (care should be taken not the raise the levels to the crop root zone). Conversely, during drought periods the limited surface water resources will be supplemented by pumping groundwater, thereby lowering the water levels. However, the cost of setting up such a scheme could be prohibitive for most our country.

Box 2-1:
Worked Example-1*: Suppose we are given the following monthly cropping pattern with 48% gross irrigation efficiency. If 80% dependable/low flow of 24hr in m^3/s is expected available as presented in table below, compute monthly Water Budget and check if we can irrigate without storage requirement. Assume d/s release to be 10% of monthly lean flow.*

Table 2-1: Cropping pattern and estimated monthly water demand

Description	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation deficit (mm)												
1. Banana 1st year	98	97.9	58.8	6	0	0	0	0	0	0	4.6	65.9
2. coffee	56.4	50.8	16.2	0	0	0	0	0	0	0	0	36.7
3. Haricot, dry	125.5	47.9	0	0	0	0	0	0	0	0	0	83.7
4. Haricot, wet	0	0	0	0	0	0	1	0	0	0	0	0
5. Maize (dry season)	17.8	72.2	76.9	17.9	0	0	0	0	0	0	0	0
6. Maize (wet season)	0	0	0	0	0	0	0	0	0	2.8	0	0

Description	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
7. Onion	130	112.8	29.2	0	0	0	0	0	0	0	0	52
8. Sweet Peppers	107.2	100.1	24.1	0	0	0	0	0	0	0	2.4	52.5
9. Potato, dry	121.5	113	40.2	0	0	0	0	0	0	0	0	50.8
10. Potato, wet	0	0	0	0	0	0	0	0	0	5.3	0	0
11. Tomato, dry	120.1	112	22.1	0	0	0	0	0	0	0	0	55.6
Net scheme irrigation req.												
in mm/day	3	3.1	1.2	0.1	0	0	0	0	0	0.1	0	1.5
in mm/month	93.5	87.2	36.7	4.4	0	0	0.3	0	0	1.8	0.5	45.2
in l/s/h	0.35	0.36	0.14	0.02	0	0	0	0	0	0.01	0	0.17
Irrigated area (% of total area)	100	100	85	29	0	0	25	0	0	60	14	78
Net Irrigable area (ha)	84.1											
80% dependable/low flow, 24hr (m^3/s)	0.078	0.043	0.073	0.171	0.273	0.602	1.602	1.091	1.085	0.453	0.234	0.117

Source: Petu SSI Project Design Report, GIRDC, 2016

Solution*: The question here is to analyze monthly water available for irrigation and determine Storage requirement by comparing Demand and Supply. If this difference is negative, it indicates that available water cannot satisfy the required flow for the proposed cropping pattern thus either we need to store in the night or idle time & augment the required flow in the day time or update the cropping pattern such that supply exceeds demand.*

Table 2-2: Monthly water budgeting at project site, mm^3/month

Description	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	sum
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
Net scheme irr. req.in l/s/h	0.35	0.36	0.14	0.02	0	0	0	0	0	0.01	0	0.17	
Irrigated area (% of total area)	100	100	85	29	0	0	25	0	0	60	14	78	
Irr. req. for actual area (l/s/h)	0.35	0.36	0.16	0.06	0	0	0	0	0	0.01	0.01	0.22	
Net Irrigable area (ha)	84.1												
Supply Condition													
80% dependable/low flow, 24hr (m^3/s)	0.078	0.043	0.073	0.171	0.273	0.602	1.602	1.091	1.085	0.453	0.234	0.117	
Days of Months (Days)	31	28	31	30	31	30	31	31	30	31	30	31	
Available Low flow (Mm^3)	0.209	0.103	0.196	0.444	0.732	1.560	4.289	2.921	2.812	1.213	0.605	0.313	15.4
Demand Condition (Gross)													
Demand with 48% eff, in 24hr application (l/s/h)	0.73	0.75	0.33	0.13	0.00	0.00	0.00	0.00	0.00	0.02	0.02	0.46	
24hr Demand, (Mm^3)	0.16	0.15	0.06	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.47
D/s release=10%Lowflow (Mm^3)	0.021	0.010	0.020	0.044	0.073	0.156	0.429	0.292	0.281	0.121	0.061	0.031	1.54
Total 24hr Demand, (Mm^3)	0.19	0.16	0.08	0.05	0.07	0.16	0.43	0.29	0.28	0.12	0.06	0.11	2.01
Storage Requirement (=Row 13 – Row 8, Mm^3)	0.024	(0.060)	0.113	0.391	0.659	1.404	3.861	2.629	2.531	1.089	0.544	0.202	13.39
	(+)	(-)	(+)	(+)	(+)	(+)	(+)	(+)	(+)	(+)	(+)	(+)	(+)

Note: - From this table, it can be observed that volume of water required to be stored is 6,000m^3 in the month of February. For the remaining months, available lean flow is in excess of irrigation requirement.

- Bear in mind that without change in cropping pattern but altering the calendar it-self brings variation in demand for water and storage requirement. Thus, by undertaking similar activities repeatedly, we can present different options of cropping pattern that can suit available water in the source of supply and interest of beneficiaries.
- During planning of such irrigation project, demands other than irrigation such as livestock, domestic consumption and downstream water demand for environmental and other purposes need to be considered for the identified requirement; otherwise it may result in conflict of interest among upstream and downstream communities.

2.3.3 Intake facilities

Intake facilities are that part of headwork structure located within its wings to allow water to conveyance system. Thus, hydraulic design parameters of flow within this structure such as flow velocity, slope and bed & top levels of this structure need to be consistent with the conveyance canal.

2.3.4 Conveyance system

Conveyance System includes main canal and related structures from the intake or head regulator up to the command entrance or last secondary canal in some case. This canal and its related structures should be planned such that they perform their functions efficiently and competently with minimum maintenance, ease of operation and minimum water loss. Such conveyance system can be earthen, lined and/or piped system depending on topographic conditions and soil type along the route or the site and climatic condition of the area.

2.3.5 Water storage facilities

These facilities are planned based on need for additional irrigation water as well as need assessment of beneficiaries as it has associated environmental impacts.

2.3.6 Field canal and/or pipe system

This is on-farm network of canal or pipe distribution system that need to be planned carefully due to its direct relationship with beneficiaries. We need to answer the question of why and when to prefer pipelines over open canals.

Pipelines are preferred as low-pressure pipe line systems can lead to easier distribution and management of irrigation water. Land tenure problems can be lessened especially when a distribution system has to be routed through existing farmland having small, irregular and fragmented holdings. A pipe underground occupies no land that can be used for crops, nor does it interfere with land boundaries. Management losses are potentially close to zero with these pipelines. Flexible delivery systems, in which the farmer is encouraged to take water as and when he requires it, are achievable with a pipeline but far more difficult with open canals.

The main factor in opting for a pipeline is the availability of head. For a given discharge capacity, a pipe line needs more head or level difference to operate than does an open channel. In flat terrain, a canal may be the only possibility for conveying water. In steep terrain, there may be excess head, which an open canal system would need to dissipate by means of drop structures. A pipeline in this case could utilize this excess head and might prove cheaper than a canal if the land is steep. The more head available, the smaller the diameter and cheaper the cost of the pipe.

However, for a multi-user flexible system it is important to maintain a stable operating head, which is most easily achieved at low-pressures and large diameters, usually with the inclusion of pressure reducing valves. A pipeline also offers some built-in intermediate storage and a zero response time, which is a prerequisite for most demand-scheduled water management.

2.3.7 Infield water use system

Infield water use system is planned at operation stage depending on updated crop calendar and corresponding cropping pattern.

2.3.8 On-farm drainage system

An efficient good on-farm drainage system consisting of surface drains should be planned in order to drain away unexpected storm water and excess irrigation water (Refer chapter-8 for details).

2.3.9 Accessibility infrastructure

These networks of road infrastructure should be planned to and along the command boundary to facilitate easy access of construction material to the project site and transport production during operation.

2.4 FACTORS INFLUENCING SELECTION AND DESIGN OF IRRIGATION SYSTEM

There are different influential factors that need to be considered while selecting and designing of any irrigation system. The most important ones are discussed as follow.

Water Resource Data: Data required with respect to water are its supply location & suitability for abstraction, quantity and quality, delivery or flow rate, delivery schedule, amount and arrangement of supply ditches and/ or pipelines, and automation requirements (if required),

Soil Data: As regard to necessity of soil data for irrigation planning is concerned we need to collect its classification in terms of texture and soil mapping units to help in layout design and irrigation water requirement.

Topographic Data: This is critically used to know slope of irrigable land and hence decisive in design of layout system and irrigation structures along the canal,

Natural Drainage Data: Such data are important for planning irrigation system as its location, amount and size of drainage ditches and its network affects types of crossing structures requirement for conveying irrigation water supply.

Climate - crop- system interactions: These data are expected from agronomy study part and is used to determine irrigation water requirement in planning of irrigation system. Such data are rainfall, and other climate data to fix effective rainfall and monthly water duty based on the proposed cropping calendar and pattern.

Energy cost and availability requirements: Energy requirement of different alternatives is an important criterion in planning and selection process of irrigation system.

Labor requirements and availability of labor: While planning an irrigation system we need to consider labor requirements and its availability in the vicinity of the project as one of the factors influencing such planning. If there is more labor in the project area, then surface irrigation is preferred.

Availability of construction materials and its distance from the site: Locally available construction materials and its distance from the site can also influence selection and design of irrigation system as it has direct implication on its unit rate. For example, if clay is locally available we do not need to go for masonry or concrete lining.

Cost of system installation, operation and maintenance: surface irrigation can be managed and operated by unskilled laborers than other application methods, thus it is the first priority so long as there is no shortage of irrigation water.

Farmer's preference: This is the most crucial factor thus need to be consulted if they prefer the system proposed by the designer, i.e. the final irrigation system selection as well as options need to base beneficiaries' preference.

Environmental and health factors: Since environmental and health factors have an impact on the short- to long-term performance of irrigation such data like water quality, downstream release, existing wetland, expected soil loss/accumulation, incidence of water-related diseases, and waterlogging problems need to be gathered and analyzed and recommend safety requirement.

Therefore, the initial step in the planning process is to identify the parameters, which are needed to determine viable irrigation methods for a given system. After assessing the applicable irrigation methods for a certain farm, alternative farm irrigation systems can be designed and analyzed and appropriate system design can then be selected during planning.

2.5 BASIC DATA REQUIRED FOR IRRIGATION SYSTEM PLANNING

The first business in the study and design of any irrigation system is to identify and collect required basic design data such as water demand, land suitability map, topographic map, crops to be irrigated, existing facilities/field boundaries, socio-economics, Drainage pattern of the farm, including outlets and carryout analysis of indigenous irrigation experience (application system and hours of irrigation) as stated under chapter five in the identification of net project area and set dependable design criteria.

Besides, availability of lean flow to be used for irrigation should be analyzed during this time. Then comes crop water budget, design of corresponding layout and identification of net command area based on the available supply.

- Before designing any irrigation system land suitability (soil physical and chemical conditions) of an area has to be evaluated and shown in soil map unit. Then, based on soil suitability map unit and land evaluation, the gross irrigable area is fixed with respect to engineering criteria.
- The other important issues are the crop irrigation water requirement,
- The monthly demand and monthly low flow/supply,
- Geology for canal route and farm structure foundations: The main canal route and foundation of some basic hydraulic structures geo-technical study should be done based upon test pit opening at selected intervals and selected places. The stability of slopes, lithology, water tightness, piping at soil rock interface and foundation conditions of retaining walls (if any) are main factors while studying the canal route structures foundation. The litho-logs, location and description of test pits opened along the route should be summarized, and
- Finally, the topographic/contour maps showing all features on which system layout is prepared for the design of layout system. Relative locations of source and elevation of the water supply for the area under consideration, landscape features, such as ups and downs, existing fences, buildings, roads, and shelterbelts, are among others that influence the layout and design of the irrigation system;

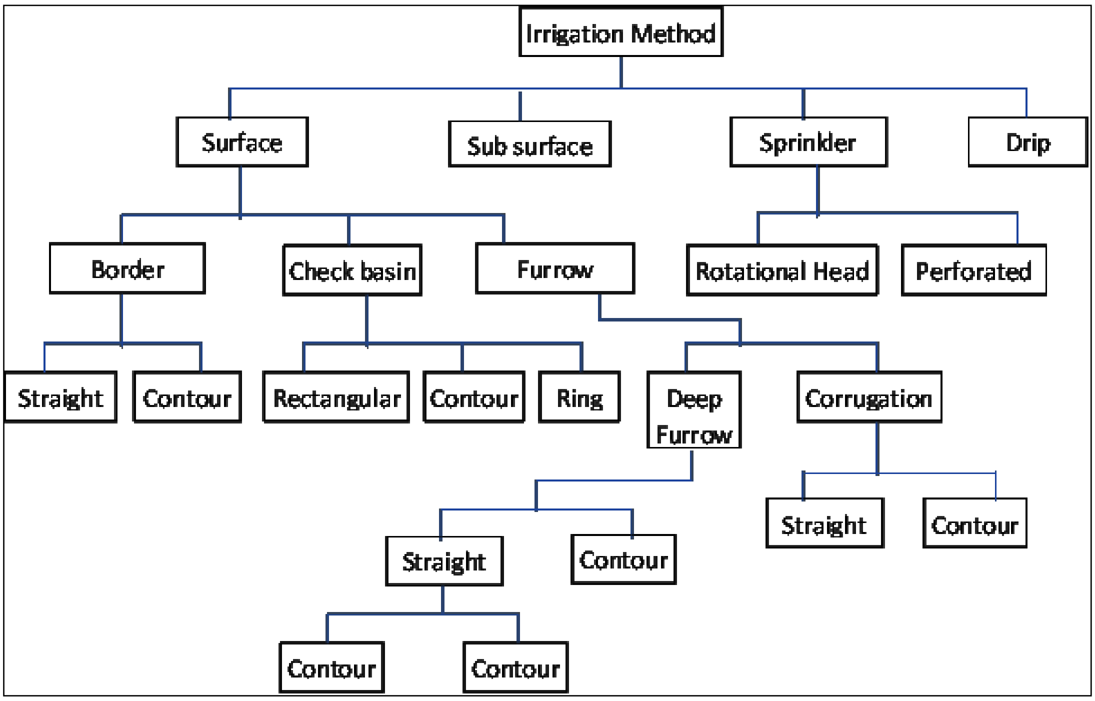
3 IRRIGATION METHODS

3.1 SELECTION OF IRRIGATION METHODS

It is indispensable to understand principles of irrigation methods before we select either of them for our purpose. The methods of application of water to the land can be broadly classified into surface, subsurface, sprinkler and drip or trickle irrigation methods. The methods of application to be selected should fulfill the following objectives:

- be such that it enables an adequate amount of water to be stored in the root zones of the plants;
- ensure uniform application of water on the land;
- not cause soil erosion problem;
- be efficient, with a minimum wastage of water
- be such that the land is not wasted for constructing field channels, borders, etc.' so that the maximum land is available for cultivation;
- be such that water is drained from the land after irrigation as far as possible;
- fit properly to the boundaries of the land to be irrigated;
- not be expensive;
- not be inconvenient and difficult;
- not cause water logging and salt problems in the irrigated land.

The choice of either method of application for a particular project is thus dependent upon a number of factors though all methods have their advantages and disadvantages. The major factors which influence the choice of appropriate method of irrigation water application are available water supply, type of soil, topography of the land, and type of crop to be irrigated (refer table 3-3: multi-criteria analysis for selection of irrigation application method).



Source: Irrigation Theory and Practice, By A.M. Michael, 1997

Figure 3-1: Flow Chart showing Families of Irrigation Application Methods

3.2 SURFACE IRRIGATION

Surface irrigation systems are based on the principle of mass movement of water over the surface of the land in order to wet it either partially or completely. All portions of land be covered with crop or not, are irrigated thus result in low efficiency. It is subdivided into furrow, border-strip, basin and free flooding or ordinary flooding or uncontrolled or wild flooding methods of irrigation.

3.3 SUB-SURFACE IRRIGATION

Sub-surface irrigation is termed as sub-surface, because in this type of irrigation, water does not wet the soil surface. But the underground water nourishes the plant roots by capillarity. Such system relies on raising or lowering of the water table in order to effect groundwater flow to the root zone. As such, they are drainage flow systems thus not commonly used method.

3.4 PRESSURIZED IRRIGATION

Pressurized irrigation system, as the name indicates, is irrigation system operating with the help of pressure. It comprises sprinkler irrigation system and drip/trickle/localized Irrigation System. As a rule, pressurized irrigation systems are composed of water lifting devices, piped networks, water delivery devices, and pressure and water control devices.

At times of steeply topography, this system can run with the pressure head difference head natural thus need no water lifting devices and water gravitates naturally into the system. However, irrespective of whether pumps are used or not, the water in the irrigation system is always under pressure. The magnitude of this pressure depends on the requirements of a particular technology (e.g. Sprinkler requires higher but drips can run with low pressure).

Generally, localized/drip irrigation systems operate at lower pressure than sprinkler irrigation systems. It is therefore necessary that during preparation of the designs and bills of quantity, the pressure requirements of the system should be clearly stated and the equipment to meet these requirements is identified.

3.5 TYPES OF SURFACE IRRIGATION SYSTEM

3.5.1 Principles of surface irrigation application

$$\boxed{\left\{\begin{matrix} Surface \\ Irrigation \\ System \end{matrix}\right\} = \left\{\begin{matrix} Water \\ Supply \end{matrix}\right\} + \left\{\begin{matrix} Water \\ Conveyance \\ or Delivery \end{matrix}\right\} + \left\{\begin{matrix} Water \\ Use \end{matrix}\right\} + \left\{\begin{matrix} Excess Water \\ Disposal \\ or Drainage \end{matrix}\right\}}$$

Figure 3-2: Schematic Representation of Surface Irrigation System

The scheme layout up to field level, such as canals, drains and related infrastructures, can be similar for each system. Low irrigation efficiencies are typical features of this method which are usually associated with poor land leveling, incorrect stream size and change in soil type along the irrigated area both vertically and horizontally.

In this application method:

- Water is applied by gravity along sloping soil surface
- Water is applied to the field in either the controlled or uncontrolled manner. It flows by gravity over the surface of the field.
- Controlled: Water is applied from the head ditch and guided by corrugations, furrows, borders, or ridges.
- Uncontrolled: Wild flooding.
- It accounts about 90% (as per FAO) of the whole irrigated area in the world. E.g. Basin, border, furrows, etc.

Some of the major advantages of surface irrigation systems over other systems are:

- Easy to operate and maintain with unskilled labour,
- Not affected by windy conditions and,
- With the exception of furrow irrigation, they are good for the leaching of the salts from the root zone.
- Low energy costs.

Disadvantages of surface irrigation systems are:

- They are less efficient in water application than sprinkler or localized irrigation systems.
- The spatial and temporal variability of soil characteristics, such as infiltration rate and texture, make water management practices difficult to define and implement.
- Difficult to apply light, frequent irrigation required early and late in the cropping season;
- High labour demand, as compared to sprinkler and localized irrigation systems.

Surface irrigation system is categorized in to three: basin, border and furrow. These methods are briefly described as follow.

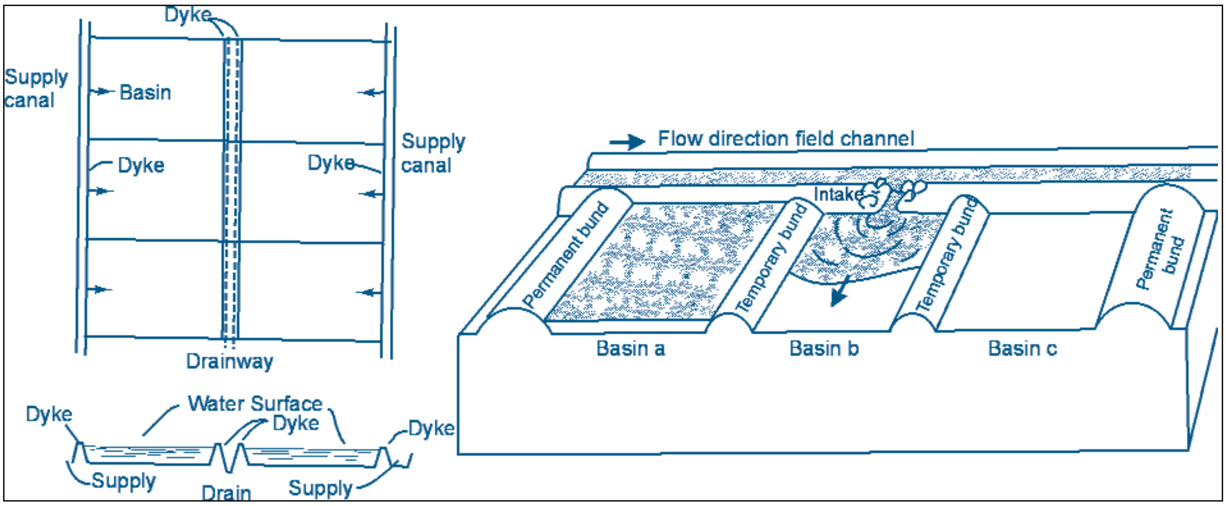
3.5.2 Basin irrigation system

Basin irrigation is the most common type of surface irrigation and is particularly used in paddy rice irrigation. A basin is a leveled area of land, surrounded by earth bunds, that does not need directed and controlled flow (FAO, 1989). Basins should be quickly filled with water during irrigation, after which the water infiltrates evenly throughout the basin, in order to achieve high application uniformity.

Basin irrigation can be a very useful way of leaching harmful salts. However, a good drainage system should also be put in place to dispose of the excess water. Basins can be adapted to suit any crop, soil or farming practices. Crops grown under basin irrigation include rice, alfalfa, row crops and orchard crops. The basins vary in size from 1-2 m^2 as in case of orchard crops up to 3-4 ha as for row crops depending on irrigation depth, land slope and farming practices.

Generally, for the same stream size and irrigation depth, basins should be smaller on light soils than on heavier soils. In cases where the land is considerably steep, terracing is necessary in order to construct basins. Typically terrace width varies from 1.5m for 4% land slopes to 150 m for 0.1% land slopes.

Basin irrigation requires less labor than the other two methods and might have to be considered if there is a critical labor shortage.



Source: FAO Irrigation & Drainage Manual, 2002

Figure 3-3: Typical Layout of Basin Irrigation Method

In general, in basin irrigation:

- The plot shape is mostly square but can exist in all sorts of irregular configurations;
- command area shall normally be flat;
- a very high stream size is introduced into the basin so that rapid movement of water is obtained;
- opportunity time difference between the upward and the downward ends are reduced;
- drainage of surface runoff is unnecessary, but exceptionally required after heavy rainfall or mistake in cut-off time;
- Generally, water is flooded in wider areas thus favoured by moderate to slow intake soils, deep-rooted and closely spaced crops like rice.
- Crops which are sensitive to flooding and soils which form a hard crust following irrigation can be basin irrigated but by adding furrowing or using raised bed planting.

Common faults in basin irrigation application are:

- Poor land preparation: reduce efficiency by 10-20%;
- Different soil types in basin: reduce efficiency by 5-10%;
- Fixed irrigation schedule (low flow rates, slow advance): reduce by 10-20%

Advantages

- Crop not wetted
- More flexible in application than furrow and border
- Feasible on range of soil infiltration rates
- Application efficiency can be as high as 90%

Disadvantages

- Relatively high flow rates required;
- Construction may be expensive;
- Reasonably labour intensive;
- Significant earthworks may increase the cost.

Basins area should be small if the:

- Slope of the land is relatively steep;
- Soil is sandy;
- Stream size to the basin is small;
- Required depth of the irrigation application is small;

- Field preparation is done by hand or animal.

Basins can be large if the:

- Slope of the land is gentle or flat;
- Soil is clay;
- Stream size to the basin is large enough, say >10l/s;
- Required depth of irrigation application is large;
- Field preparation is mechanized;

Note: Fruit trees can also be grown using basins, where one tree is usually located in the middle of a small basin.

Figure 3-4: Onion cultivated by basin irrigation system around beles river

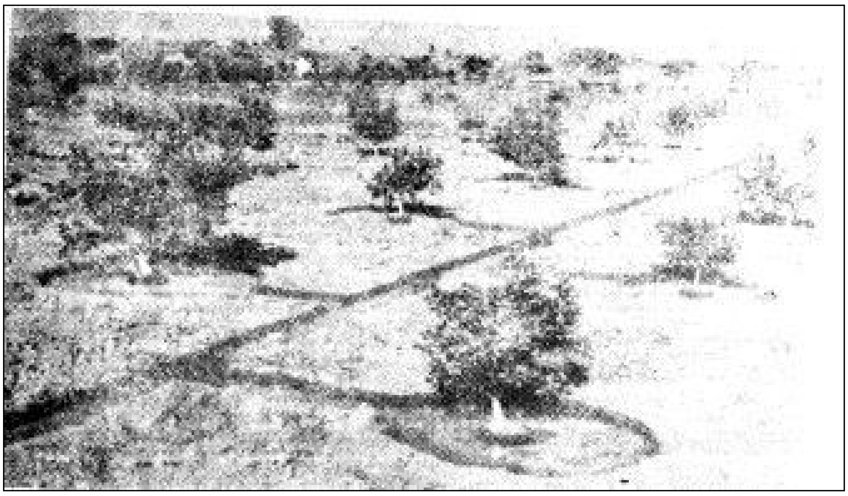


Figure 3-5: Ring method of basin irrigation in an orchard farm

3.5.3 Border-strips irrigation

Border-strips are also called border checks or strip checks and are strips of land separated by small earth bunds that guide water as it flows down the field. They can have rectangular or contoured shapes, depending on the field. Thus border irrigation can be viewed as an extension of basin irrigation to sloping, long rectangular or contoured field shapes, with free draining conditions at the lower end (unlike basin).

Border-strip slopes uniformly away from the direction from the source of the irrigation water. They should be leveled across, in order to allow for the even wetting of the whole area, covered by a border and allow free drainage at the end. Figure below shows typical layout of border-strip irrigation.

Normally, water is let onto the field from the canals through siphons. The siphoned water spreads across the width of the border when there is no cross slopes, thereby facilitating uniform water application. Uneven borders slopes and cross border slopes are some of the most common problems that result in low irrigation efficiencies.

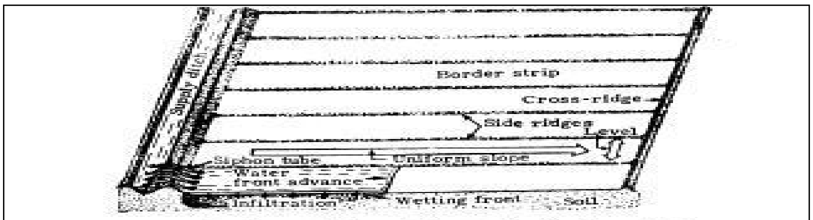


Fig. 8.1. In schematic sketch illustrating the layout of an impounding type border irrigation system.

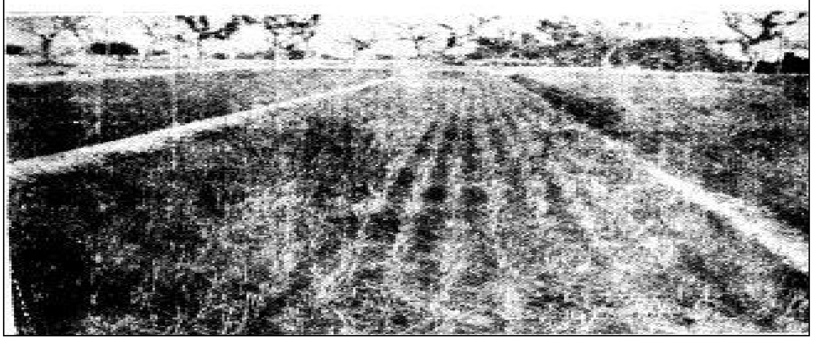


Figure 3-6: Layout of border-strip irrigation

Border-strips may vary in size from 60-800 m length and 3- 30 m width depending on the soil type, stream size, irrigation depth, slope, field size and farming practices. Generally, border width becomes smaller as the soil becomes coarser for the same unit stream size, irrigation depth, and slope, as coarse soils have a higher intake rate than fine soils and consequently less lateral water flow.

In general, in Border irrigation:

- Controlled surface flooding is practiced whereby the field is divided up into strips by parallel ridges or dykes;
- Each strip is irrigated separately by introducing water upstream and it progressively covers the entire strip

Factors affecting Border irrigation:

- Soil type: Borders gets much longer on clay than sand
- Stream size: Borders can generally be longer than when a larger unit stream size is available
- Irrigation depth: Borders can be longer when larger irrigation depth is applied
- Slope: Borders must be short on steeper sloping land to prevent erosion
- Field size and shape: Borders can be wider when larger unit stream widths are available
- Farming practice: Some runoff can be expected (<10%)
- Well designed and managed borders
- Application efficiency can be as high as 80%.

Common faults as indicated in FAO, Irrigation Guideline, 2002 are:

- Poor land preparation: reduce 10-20%
- Different soil types within border: reduce 5-10%
- Using wrong stream size: reduce 10-15%
- Fixed irrigation schedule: reduce 10-20%

Procedures in dyke construction:

- Field preparation is done by tractor or animal traction.
- It can be used for all crops provided that the system is designated to provide the needed water control for irrigation of crops.
- It is suited to soils between extremely high and very low infiltration rates.
- In border irrigation, water is applied slowly.
- The root zone is applied water gradually down the field.
- At a time, the application flow is cut-off to reduce water loses.
- The problem is that the time to cut off the inflow is difficult to determine

Advantages

- Low energy – flow by gravity
- Reduced leaf and fruit diseases
- Leaching of brackish soils
- Low capital costs if land relatively level

Disadvantages

- Low efficiency if not well designed and well maintained
- Small deviations from design specifications can significantly affect uniformity
- Not suitable for all crops
- Financial viability depends on the extent of earthworks
- Relatively labour intensive
- Very high management inputs required
- Not suitable for soils with high infiltration rates

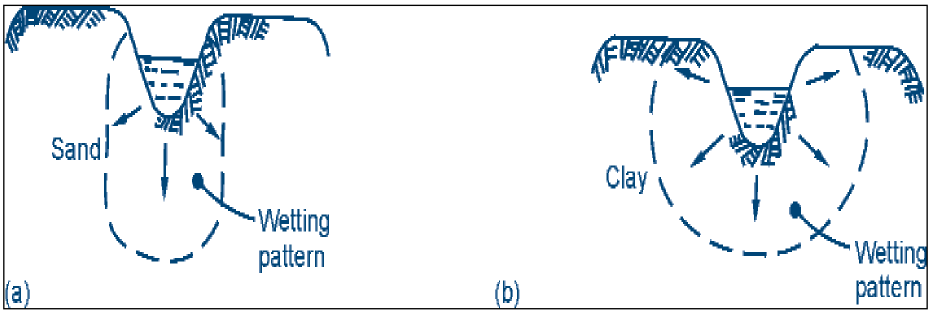
3.5.4 Furrow irrigation system

Furrow irrigation system consists of furrows and ridges, of which the shape, spacing and length depend mainly on the crops to be grown and the types of soils.

Furrow irrigation avoids flooding the entire field surface by channeling the flow along the primary direction of the field using 'furrows,' 'creases,' or 'corrugations i.e. parallel furrows'.

According to Kay (1986), the width of the furrows varies from 250-400 mm, the depth from 150-300 mm and the spacing between the furrows from 0.75-1.0 m, depending on soil type, crops and stream size to be applied to the furrow.

Coarse soils require closely-spaced furrows in order to achieve lateral water flow in the root zone. There is more lateral water flow in clay than in sand. Typical furrow lengths vary from about 60 m on coarse textured soils to 500 m on fine textured soils, depending on land slope, stream size and irrigation depth. The minimum and maximum slopes for furrows should be 0.05% and 2% respectively in areas of low rainfall intensity. In areas where there is a risk of erosion due to intensive rainfall, the maximum slope should be limited to 0.3%. With furrow irrigation there is a risk of localized salinization in the ridges (FAO, Irrigation Guideline, 2002).



Source: FAO Irrigation & Drainage Manual, 2002

Figure 3-7: Wetting patterns in coarse and fine textured soils

Figure 3-8: Furrow irrigation practice around Meki

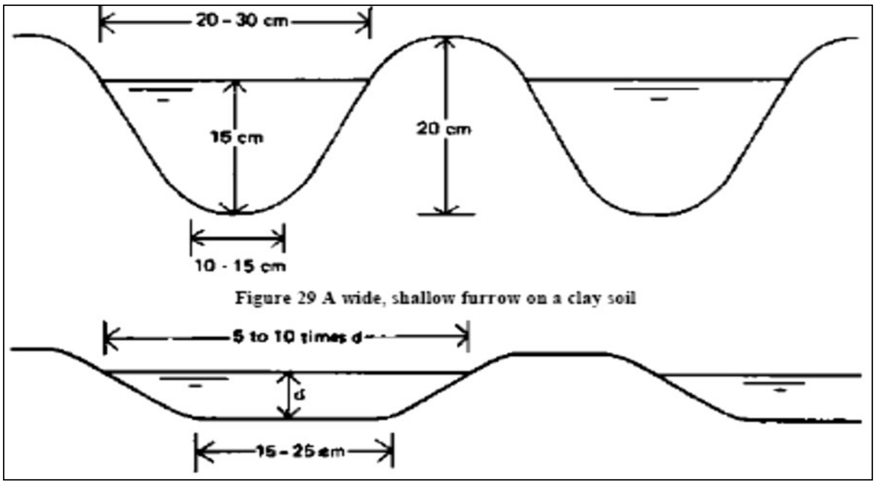


Figure 3-9: Typical furrow shapes and their hydraulic sectional parameter

In general, in furrow irrigation:

- Only a part of the land surface (the furrow) is wetted thus minimizing evaporation loss.
- Furrow irrigation is adapted for row crops like corn, banana, tobacco, and cabbage. It is also good for grains.
- Irrigation can be by corrugation using small irrigation streams.
- Furrow irrigation is adapted for irrigating on various slopes except on steep ones because of erosion and bank overflow.
- There are different ways of applying water to the furrow.
- Siphons are used to divert water from the head ditch to the furrows.
- There can also be direct gravity flow whereby water is delivered from the head ditch to the furrows by cutting the ridge or levee separating the head ditch and the furrows.

Advantages

- Relatively inexpensive
- Only roots wetted
- Low energy cost – flow by gravity
- Less sensitive to ground slope than border irrigation
- Soil surface partially wetted

Disadvantages

- Very sensitive to deviations from design specifications
- Only row crops can be irrigated
- Relatively labour intensive
- Good design and maintenance necessary for high irrigation efficiencies
- Not suitable for soils with high infiltrations rates
- Depth of irrigation water not uniform from head to tail of the furrow and
- It require high maintenance cost than other irrigation systems

Furrow Shape (width and depth) is dependent on:

- Furrows are generally V-shaped or U-shaped in cross section and are 15-30 cm deep and 25-40 cm wide at the top,
- The soil type, stream size and crop type,
- Stream size: Larger stream size, the larger the furrow channel,
- Soil type:
- Clays, slow infiltration, wider furrows to increase contact area
- Sands, rapid infiltration, use deep, narrow channels to reduce infiltration
- Crop: Can change depth of furrow to match crop root depth

Furrow spacing: The spacing between furrows depends on:

- The water movement in the soil, which is texture related;
- the crop agronomic requirements as well as;
- the type of equipment used in the construction of furrows;
- In practice a compromise often has to be reached between these factors.

Furrow length: Under mechanized agriculture, furrows should preferably be as long as possible in order to reduce labor requirements & system costs. However, they also should be short enough to retain a reasonable application efficiency & uniformity. Application efficiency & uniformity normally increase as furrow length decreases. Thus, when labor is not a constraint or inexpensive and/or water supply is limited, short furrows are most suitable. This does increase the number of field canals and overall cost of the system. For proper design of furrow length, the following factors have to be taken into account: Soil type, Stream size, Irrigation depth, Slope, Field size and shape, Cultivation practices.

Thus, furrows shall be designed to run along contour lines consisting of furrows and ridges, of which the shape, spacing and length depend mainly on the crops to be grown and the types of soils. Moreover, furrows should be put on proper gradients that allow water to flow along them and at the same time allow some water to infiltrate into the soil.

Distribution of irrigation water to these furrows could be either by siphons which are intended to take water from the field ditch or by jet of water to be released to the furrows from Lay-Flat-Tube (perforated at off-take of each furrow), or traditional diversion system from field ditch (Refer figure below).

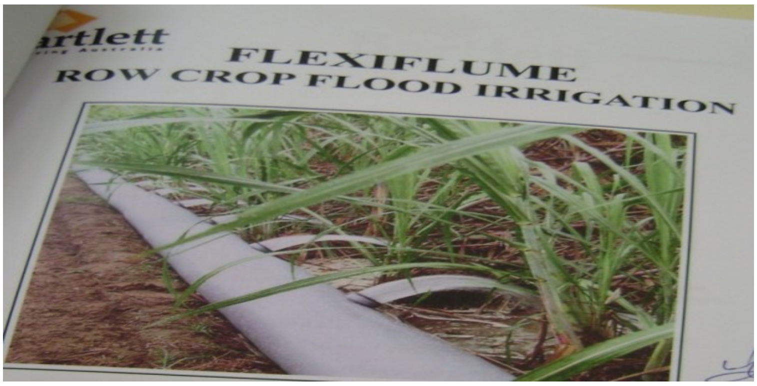


Figure 3-10: Typical flexi flume/lay-flat-tube in operation

Figure 3-11: Typical siphon tube systems

Table below summarizes the main factors affecting the furrow length and the suggested practical allowable furrow lengths according to Kay, 1986. The data given in this table are appropriate for large-scale and fully mechanized conditions.

Table 3-1: Furrow Lengths (m) as related to Soil, Slope, and Stream Size & Irrigation Depth

Soil type		Clay		Loam			Sand		
Furrow slope, %	Max. stream size (l/s)	Average irrigation depth (mm)							
		75	150	50	100	150	50	75	100
		Length of Furrow (m)							
0.05	3.0	300	400	120	270	400	60	90	150
0.10	3.0	340	440	180	340	440	90	120	190
0.20	2.5	370	470	220	370	470	120	190	250
0.30	2.0	400	500	280	400	500	150	220	280
0.50	1.2	400	500	280	370	470	120	190	250
1.00	0.6	280	400	250	300	370	90	150	220
1.50	0.5	250	340	220	280	340	80	120	190
2.00	0.3	220	270	180	250	300	60	90	150

Source: FAO Irrigation & Drainage Manual, 2002

Table 3-2: Practical values of max. Furrow lengths (m) for SSIP

Soil type		Clay		Loam		Sand	
Furrow slope, %	Max. Stream size (l/s)	Average irrigation depth (mm)					
		50	75	50	75	50	75
		Length of Furrow (m)					
0.0	3.0	100	150	60	90	30	45
0.1	3.0	120	170	90	125	45	60
0.2	2.5	130	180	110	150	60	95
0.3	2.0	150	200	130	170	75	110
0.5	1.2	150	200	130	170	75	110

Source: FAO Irrigation & Drainage Manual, 2002

3.5.5 Free/uncontrolled flooding

There are many cases where croplands are irrigated without regard to efficiency or uniformity. These are generally situations where the value of the crop is very small or the field is used for grazing or recreation purposes. Small land holdings are generally not subject to the array of surface irrigation practices of the large commercial farming systems.

In free flooding or ordinary flooding or uncontrolled or wild flooding method, ditches are excavated in the field & may be either on the contour or up & down slope. Water from these ditches flows across the field. After water leaves ditches, no attempt is made to control the flow by means of levees, etc. Since the movement of water is not restricted, it is sometimes called wild flooding. This method is not recommended except in the case of spate irrigation.

In this case, the design techniques are not generally applicable nor need they be since the irrigation practices tend to be minimally managed.

Table 3-3: Multi-criteria Analysis for Selection of Irrigation Application Method

Criteria and Implications for:		**Remarks**
Surface Irrigation	**Pressure Irrigation**	
I. Topography		
Land Slope		
Land levelling and shaping costs not too expensive for flat and almost flat land (less than 3% slopes). With steeper slopes (bench) terracing costs to form basins increase. Borders and straight furrows suited to plain lands with slopes of about 0.5% or less. Contour furrows for slopes 3% or less without terrace.	Pressure irrigation is adaptable to a range of sloping land, with control and pressure reducing valves ensuring uniformity of distribution. If gravity can be used to provide required pressure head (instead of pumping) operating costs are reduced. A practical upper limit of 8% is adopted.	There is no history of extensive terracing for irrigation in Ethiopia. This indicates a risk of poor management of terrace/contour furrow (by smallholders) leading them to failure and extensive soil loss. But if proper terracing for irrigation is practiced it is possible for up to 15% slope for surface irrigation.
Land shape and regularity		
Land levelling and shaping costs increase for irregular and broken land. Water conveyance costs also increase.	Pressure irrigation can be adapted to irregular / broken land. Though the cost for water distribution/conveyance increases, the ability of pipe lines to go up and down hill means the cost may be less than for canals.	Much of the irrigable land in our country is broken except lowland areas
II Soil Features		
Depth of soil		
Land levelling and shaping must not result in severe loss of potential for productive agriculture by exposing poor sub-soils or rock. In practice, deep soils are required where land levelling of sloping land (3% of more) is required. A minimum of 2 m soil depth is also needed if drainage and/or soil reclamation	Pressure irrigation does not require extensive land levelling: in practice only dips and bumps are smoothed over. Depth of soils is therefore not such a major constraint. However, soil depth should also be sufficient to tolerate some soil loss and allow (surface) drainage. A minimum soil depth of 1 m is suggested.	Soil depths could be sufficient to allow terracing, albeit with some loss of soil fertility.

Criteria and Implications for:		Remarks
Surface Irrigation	**Pressure Irrigation**	
works are necessary.		
Type of soil		
Surface irrigation efficiencies generally increase with heavier soils which have medium-low infiltration rates. Uniform applications are difficult on light soils: a practical upper infiltration limit of 30 mm/hr is suggested. Surface irrigation is generally considered more suited to cracking vertisols where high rates of applications are possible until soil swells and cracks seal up. After sealing irrigation should stop to prevent erosion.	Pressure irrigation is suited to a wide range of soils, but has a particular advantage over surface irrigation methods in light soils where high efficiencies are maintained. For cracking vertisols, irrigation applications should be high for short periods so that the cracks are filled and seal from the bottom up. Light applications may cause the cracks to seal at the surface without penetrating the full depth of the root zone. The greater irrigation efficiency and water control provided by pressure irrigation is beneficial for all soil types (including vertisols) which experience drainage problems.	For the heavy soils a gain in application water use efficiency of just 5-10% is considered likely with pressure (sprinkler) irrigation
Depth to Water Table		
In areas of high water table surface irrigation should not be developed without drainage to allow surplus water to be lead away.	Pressure irrigation being more efficient will contribute less to a rise in the water table. However for clay soils, including cracking vertisols once cracks seal up, infiltration rates are very slow and there is likely to be little advantage gained from pressure irrigation. As with surface irrigation, (surface) drainage is required to lead off excess water and after rainfall.	Surface drainage should be provided in flat-gently sloping areas where water logging occurs, irrespective of whether water application is by surface or sprinklers. However in other areas if pressure irrigation is adopted surface irrigation may not be necessary.
III. Water Quality and Availability		
Water Quality –sediment load		
Surface irrigation allows fine sediments to be transported to farmer fields helping to sustain fertility	Pressure irrigation methods, particularly drip, are vulnerable to blockage either due to sediments or chemical or biological growths. Settling tanks / reservoirs and /or filters are required.	Balancing / night storage reservoirs should be considered and will allow sediments to be removed.
Water Availability		
Where water is scarce and/or has high value the investment in more efficient (pressure) irrigation is likely to be justified.		Pressure irrigation will result in efficiency gains.
IV. Proposed Crops		
Surface irrigation suited to all crops	Pressure (sprinkler) irrigation is not suited to rice and results in yields about 25% lower than paddy rice grown in ponded basins.	
V. Social and Management Considerations: Small Holder & Commercial farms		
Surface irrigation methods are considered suited both small holder and commercial farms. Typical labour requirements are:	Pressure irrigation, particularly over slopes, requires technical knowledge for efficient distribution, correct operation of control valves, uniform	O&M of pumping equipment in particularly is likely to pose a risk for smallholder farms and is best avoided.

Criteria and Implications for:		Remarks
Surface Irrigation	**Pressure Irrigation**	
(i) 0.5-1.5 hrs/ha for basins; (ii) 1-3 hrs/ha for boarders; and (iii) 2-4 hrs/ha for furrows. This compares to: (i) 1.5-3 hrs/ha for sprinklers; and (ii) 0.2-0.5 hrs/ha for trickle1.	pressure distribution and periodic flushing of sediments. Smallholder pressure irrigation systems should be developed with caution and only with enthusiastic small holder interest and support.	
	Pressure irrigation for commercial estates is likely to be less of a risk as estates should have the necessary expertise for efficient operation and maintenance of pressure systems, and to meet maintenance costs.	
VI. Costs and Economic Viability		
Capital Costs		
Costs for surface irrigation application methods is about 40-60% of pressure (sprinkler) irrigation for flat or almost flat plains, but increases with land slope and terracing costs. Adopting narrow suitable terrace widths for steeper slopes reduces costs, but a practical minimum needs to be observed to facilitate farming operations.	Pressure irrigation application methods are expensive, due to costs of pumping plant (unless pressures can be built up by gravity), pipe delivery systems and application systems. The equipment also has a shorter working life than surface irrigation infrastructure.	Pressures for sprinklers could be developed by gravity where scheme command allows reducing both capital and O&M costs.
O&M Costs including Pumping Requirements and Costs		
Surface irrigation O&M costs is largely labor intensive, repairing earthen bunds, terraces and canal prism sections. O&M costs are usually less than those for pressure irrigation systems.	Pressure irrigation requires pumping with associated operation costs, unless sufficient head is available for gravity to maintain pressures in pipelines. Pressure head to rotating sprinklers should be 25-45 m.	
VII. Environmental Considerations		
Providing the characteristics of the different methods are considered and suited to the land, soil and water environment, impact will be minimized. For example: terracing of shallow soils must be avoided, and drainage provided to mitigate impact of water table and / or salinity build up.		
VIII. Government Policy		
Relevant policies / programs include: (i) The Plan for Accelerated and Sustainable Development to End Poverty (PASDEP); and (ii) Agricultural Development Led Industrialization. PASDEP expresses the view that accelerated growth will be generated through greater commercialization of agriculture, rural development and private sector development. Acquisition of advanced technology and raising the skills of the labor force is equally considered instrumental for rapid and sustainable growth. ADLI includes for specialization, diversification and commercialization of agricultural production. Government policy therefore is ambivalent concerning irrigation method, but indicates that commercial development is seen as the way forward together with measures to raise productivity of small holders.		

Source: Halcrow-GIRDC I&D Study Report for 80,000ha Net Ethiopian Nile Irrigation Project, 2010

4 IRRIGATION WATER REQUIREMENT

4.1 DUTY OF IRRIGATION WATER

Duty is stated with reference to a base period and the place of its reckoning or measurement. It is expressed in volume of water or rate of flow per unit area (l/s/ha). It is the most important tool for fixing system capacity i.e. capacity of a conveyance or main canal. It is represented by:

$q=Q/A$ ……………………………………………………………………………..………….. (4-1)

$V = Q*t$ ……………………………………………………..…………………….……………. (4-2)

Where, q is duty of water, l/s/ha
Q is discharge or capacity, l/s
A is net irrigable command area, ha
V is Volume of water, (m3) = A*d ……..………………………..……..……..…… (4-3)
t is Irrigation time, (hr)
A is the size of field to be irrigated (ha)
d is irrigation depth (mm)

4.2 ESTIMATION OF IRRIGATION WATER REQUIREMENT

Irrigation water requirement (IWR) is one of the principal parameters required for planning, design and operation of irrigation systems. It is computed from the crop water requirement which itself is estimated in agronomic section using CROPWAT8 software. But here summary of CWR shall be presented and water budget need to be analyzed based on it as indicated under section 1.4.2.

- In order to estimate irrigation water requirement /IWR/ of a project, monthly CWR need to be collected from the Agronomy report. Then IWR is computed here for the designed cropping pattern, cropping calendar and estimated monthly duty of water.
- Since plots of each beneficiary are not known independently and in case they all produce crops of high water demand at a time, we have to design our system to accommodate such worst condition (i.e. as if the peak scenario will happen all the months of the intended season).

$NIWR = ETc- (eff.\ RF + G_e + W)$ …….…………………………………………………. (4-4)

Where NIWR= Net Irrigation Water Requirement (mm)
ET_c= Crop evapotranspiration = $Kc*ET_o$ ………………………..……………….... (4-5)
Kc = Crop coefficient (Unit less)
ET_o = Potential evapotranspiration (mm)
Eff. RF = Effective rain fall (mm)
G_e= Ground water contribution (mm)
W= Residual soil moisture (mm)

However, there are cases when leaching requirement and pre-sawing or pre-planting, if any is required. Thus, we can consider these conditions as in addition to the above.

But contributions from G_e and W in most cases are insignificant thus their values are close to zero. They can also be estimated in the field by exploration.

4.3 IRRIGATION EFFICIENCY

Irrigation efficiency is a measure of canal losses such as deep percolation, seepage, evaporation (which is about 10 per cent of the quantity lost due to seepage). When we are talking of irrigation efficiency, we mean the overall irrigation efficiency, which is also known as project efficiency (E_p). It comprises conveyance efficiency (E_c), field canal efficiency (E_b) and field application efficiency (E_a).

According to FAO (1992) Conveyance efficiency (E_c) is the ratio of the water received at the inlet of a block of fields to the water released at the headwork.

Field canal efficiency (E_b) is the ratio between water received at the field inlet and that received at the inlet of the block of fields.

Field application efficiency (E_a) is the ratio between water directly available to the crop and that received at the field inlet.

Thus Project efficiency (E_p) is the ratio between water made directly available to the crop and that released from the headwork, or

$$E_p = E_c * E_b * E_a \quad (4\text{-}6)$$

Note: Conveyance and field canal efficiencies are sometimes combined and called distribution system efficiency, E_d,

$$E_d = E_c * E_b \quad (4\text{-}7)$$

Field canal and field application efficiencies are also sometimes combined and called farm efficiency, E_f,

$$E_f = E_b * E_a \quad (4\text{-}8)$$

Irrigation Efficiency is dependent on plenty of factors; among others the following can be mentioned:

- Size of Farm (i.e. operation levels in small scale or large scale)
- Climatic conditions of the project area
- Topographic conditions of the project area
- Soil factors

Table 4-1: Indicative Conveyance and Application Efficiencies

Canals/ Application	Efficiencies				
	Conveyance			Application	
	Lined canal	Unlined Canal		Clay/ Vertisols	Loams
		Clay/ Vertisols	Loams		
Main System (main & secondary canals)	95%	90%	80%		
Distribution system (tertiary& field canals)	90%	85%	75%		
Overall conveyance	85%	77%	60%		
Furrow				65%	55%

Source: Halcrow-GIRDC I&D Study Report for 80,000ha Net Ethiopian Nile Irrigation Project, 2010

GIWR =NIWR/Efficiency …………………………………………..…………….…...…. (4-9)

Where, GIWR = Gross Irrigation Water Requirement, (mm)
NIWR = Irrigation Water Requirement, (mm)
Efficiency = Project Efficiency (%)

4.4 IRRIGATION SCHEDULING

4.4.1 Required parameters

Irrespective of the mode (computerized or manual) to be adopted in preparing an irrigation schedule, which is one of the water management tool, the following parameters are required:

- Cropping pattern & calendar;
- Daily water requirements of the different crops (ETc) at the different stages of their growth;
- Root zone depth at the different growth stages of each crop;
- Total available soil moisture;
- Allowable soil moisture depletion level;
- On-site rainfall data

In general, field irrigation scheduling is based on the field water balance and is expressed in depth and interval of irrigation. To calculate irrigation intervals and depth of application rooting depth of full-grown crop, fraction of available soil water (p) and readily available soil moisture are required, which need to be collected from soil and agronomy report.

Readily Available Soil Moisture, RASM (mm) = $P*S_a$ ……………..…………………………... (4-10)

Where, P is fraction of available soil water (%);
S_a is Total available soil water, ASM (mm),

Available soil moisture, ASM (mm) = FC-PWP …………………………………………..…….. (4-11)

Where, FC is Field Capacity (%)
PWP Permanent Wilting Point (%)

4.4.2 Irrigation interval

Irrigation interval (II) also called irrigation cycle (IC) or Irrigation frequency is time interval between two successive irrigation applications of the same block. Irrigation interval is thus the time it takes a crop to deplete the soil moisture at a given depletion level.

The higher the frequency of irrigation, the interval between two irrigations decreases in a given period, whereas with lower frequency the required interval between two irrigations increase. The term, interval of irrigation indicates the time gap, usually expressed in days, between two successive irrigations. Irrigation frequency varies with stages of crop (Type, growth stage & depth of effective root zone), soil (type, texture & structure or water penetration) and climate conditions (Rainfall, temperature, humidity, sunshine hours and wind speed). Thus irrigation frequency depends on many factors but in no case should exceed the maximum permissible irrigation interval.

$$\textbf{II}\ (\text{days}) = \frac{P * Sa * D}{ET_c} \quad \ldots\ldots (4\text{-}12)$$

Where, ETc is maximum daily Crop evapotranspiration in mm/day
D is Effective root depth (cm) and
Others as indicated above.

Note: This value varies for same crop depending on its stages of development, thus may have four different interval values at initial, development, mid-season and late-season/maturity.

4.4.3 Irrigation time

Irrigation time/duration/hour, t in minute or hour is the time needed to supply the required irrigation depth in mm every day. It depends on the stream size (l/s), the required irrigation depth in mm and the size of field to be irrigated (ha). The following formula is used to determine such irrigation time.

$$\text{Irrigation time}, t\ (\text{hour}) = \frac{2.78 * d * A}{Q} \quad \ldots\ldots (4\text{-}13)$$

Where, d is the required gross irrigation depth in mm;
A is the size of field to be irrigated (ha);
Q is the stream size (l/s)

To minimize costs of developing irrigation schemes, some countries design for 22-24 irrigation hours per day, particularly for rice, requiring cultivators to irrigate at night (FAO, 2002). However, from technical and social aspects point of view, this tends to result in low irrigation efficiencies, i.e. over/under irrigation and washout of soils. Moreover, Snake varieties which only come out at night such as Russell's viper and King Cobra may bite irrigators whilst trying to irrigate at night (source: A. Laycock, 2007).

Thus, adopt 8, 10 or 12hours per day irrigation and store in the night times for the remaining hours (i.e. 16, 14 or 12 hours per day storage at heads of secondary canals respectively) to use for day times in the downstream sides of additional command area.

Note: Night storage reservoirs would be provided at intervals along the secondary canals depending on its length (Usually night storage reservoirs are provided if secondary canals are more than 3 km long so as to reduce advance time of irrigation water). These reservoirs also serve as balancing storage, facilitating efficient rotation of canal supplies, and are thus imperative in long canal systems and hence improve efficiency.

4.4.4 Depth of application (d)

Depth of Application, d in mm is given by = $\dfrac{P * Sa * D}{E_a}$ ……… (4-14)

Where, P and S_a are as defined above
E_a is Field application efficiency; usually 30-50% for smallholder based surface irrigation in Ethiopia.
D is Effective root depth (cm)

Note: If the application depth is small, FAO recommended that furrow irrigation is the most appropriate method of irrigation.

Box 4-1:
Worked Example-2: *Consider two net irrigable block areas maize crop to be, A_b=2.4ha each. Corresponding field data are given in table 4-2 below. Based on these given data, compute Net depth of water, d_{net} for both unites; Gross depth of water, d_{gross} for both unites; Identify which block requires more frequent irrigation; Volume of water abstracted at entrance to each block; Capacity, Q of this system; and abstracted volume of water, V per day for this system,*

Table 4-2: Given Net Irrigable Block Areas

Design Parameters	Unit	Block-A	Block-B	Remark
Total Area to be irrigated, A_b=	ha	2.4	2.4	To be taken from designed layout
Soil	Type	Clay loam	Sandy mixture	To be taken from SMU
Field capacity (FC) % weight	%	180	120	From soil report/Lab result
Wilting point (WP) % weight	%	50	40	"
Design root zone depth (RZD) of maize	m	1.0	1.0	From standard references/this GL
Allowable soil moisture depletion, (P)	%	0.55	0.55	From soil report/Lab result
Field application efficiency (Ea)	%	0.60	0.50	From FAO, 2002
Field canal efficiency (Eb)	%	0.90	0.80	"
Conveyance efficiency (Ec)	%	0.90	0.80	"
Peak ETc	mm/d	6.0	6.2	From agronomy or hydrology report or our analysis of climate data
Irrigation time/duration per day	hr	12	12	Fixed by the designer/agronomist

Required:

i. Net depth of water for both unites, d_{net}
ii. Gross depth of water for both unites, d_{gross}
iii. Identify w/h block requires more frequent irrigation
iv. Volume of water abstracted at entrance to each block
v. Capacity of the system, Q
vi. Abstracted volume per day, V

Solution: *The following table shows summary of computed values of irrigation water requirement & related parameters.*

Table 4-3: Exercise on Computation of Irrigation Water Requirement

Design Parameters	Unit	Block-A	Block-B	Remark
Farm irrigation efficiency (E_f) = E_b*E_a	%	0.54	0.40	
Distribution system efficiency (E_d)=E_c*E_b	%	0.81	0.64	
Overall irrigation efficiency, (E_p= E_c * E_b * E_a)	%	0.49	0.32	
Available soil moisture (= FC-PWP)	mm/m	130.0	80.0	
d_{net} = (FC - PWP) x D x P	mm	71.5	44.0	
Irrigation Frequency, IF = d_{net}/ETc	days	11.9	7.1	
Thus, for practical purposes the system should be designed to provide d_{net} in	days	11	7	Decimals are not used for IF
Now, d_{net} should be revised for this IF, i.e. revised d_{net} =	mm	66	43	
d_{gross} = d_{net}/Ep	mm	135.8	135.6	
Adjusted allowable depletion for revised d_{net}, P =	%	51%	54%	

Design Parameters	Unit	Block-A	Block-B	Remark
Volume of water abstracted, V = 10 x A x d_{gross}	m^3/d	3,259	3,255	
If irrigation cycle/Interval is say, IC or II =	days	7	5	Can be any Nr ≤ IF
Now, area irrigated per day, A=A_b/IC	ha	0.34	0.48	i.e. if IF is reduced to IC
Now, volume of water to be abstracted/day, V=	m^3/d	466	651	
Thus, system capacity for 12 hrs/day, Q=	m^3/hr	39	54	

4.4.5 *Types of irrigation scheduling*

Irrigation can be scheduled using a variety of different methods based on observations or measurements of plants, soil, the weather or a combination of these (i.e. water balance method). All methods aim to determine when to irrigate to avoid water stress and how much water to apply to refill the soil.

Once the three parameters i.e. daily water requirements, available soil moisture and effective root zone depths are known, an irrigation operation schedule can be established easily. While estimated values of ETc, based on climatic data, are sufficient for planning and designing purposes, for more accurate scheduling more accurate field data are necessary. For this purpose class-A pan or tensiometers are required.

Irrigation scheduling can be computed based on either:

- Measurement of daily crop water use (by Pan-A or Tensiometer)
- Crop water requirement calculations

As per FAO, 2006, scheme irrigation scheduling is subdivided in to Rigid, Rotational, Flexible or On-demand. Description of these types are given in the following sections.

4.4.5.1 Rigid schedules

This schedule is usually predetermined by the scheme bylaws, scheme policy, or other means. The schedule is often determined before the start of the irrigation season-based on historical crop water requirements, or simply by allocating expected water supplies proportionally to land ownership or other criteria. Some kind of rotational schedule is usually implied. Capital costs are the least with this type of schedule, as canals and structures are designed for continuous supply at peak demand periods.

4.4.5.2 Rotational schedules

This is the most commonly adopted schedule especially in cases when there is shortage of water in the source. This schedule can be either Fixed Rotation or Varied Rate Rotation.

Fixed Rotation: implies a fixed flow rate, fixed irrigation frequency and fixed duration. It is a type of fixed interval fixed amount schedule. Intervals are, for example, weekly, bi-weekly or monthly. The irrigation interval and amount are often determined by the peak use period on a scheme. The average allowable depletion (P) at peak use periods, along with application and distribution efficiencies, determines the amount of water delivery. This type of schedule is easy to administer from a schematic point of view.

Varied Rate Rotation: In this type of fixed interval-variable amount scheduling method, irrigation frequency and duration are fixed and the flow rate is varied to approximate seasonal demands. Monocrop or perennial crop areas with deep uniform soils are best suited for this schedule. As with the varied frequency system, this method may result in greater efficiencies than with fixed rotations, as over-applications early and late in the season are minimized. However, small stream sizes are often difficult to manage in farm and scheme canals. Flow control structures must be capable of adjustment to the required rates. As surface irrigation systems are most efficiently operated for fixed application depths, this may also present a problem for farm-level management. The farmer must generally become a better water manager to deal with the efficient application of variable rate and amounts. Again, communication from the irrigation management committee down to farm-level must be adequate.

4.4.5.3 Flexible schedules

In this type of irrigation schedule, the farmer has control of one or more of the three scheduling components. The degree of flexibility is dependent on the system design and the management capabilities at scheme-level. Compromises between the farmers' needs and capabilities of the delivery system are generally required. On the systems with greatest flexibility, over-sizing of canals, offline reservoirs, and automation may be required to meet demand and to avoid spillage and overtopping. On the less flexible systems (for example, restricted/arranged), the main requirements are adequate system capacities and control, along with good communication between farmers and water authorities.

4.4.5.4 On-demand irrigation

This type of irrigation schedule imposes no limits on rate, frequency, or duration of water delivery. This type of schedule implies that the water authorities impose no external controls on the water use. The system capacity is designed based on certain assumptions, for example the probability that maximum 85% of the farmers irrigate at the same time. Although this system is often ideal from the farmer's point of view, sometimes the economics of scheme implementation cannot justify such a system.

4.5 IRRIGATION DUTY

Duty of a canal or pipeline is a measure of its design capacity. Thus, it should be the first task to fix duty for designing canal capacity. It can be expressed in several ways:
Flow duty: The duty of water in hectares /cumec is convenient in the case of flow irrigation from canals. In this case, duty and command area i.e. land to be irrigated are known, thus the required discharge in the canal can be determined.

$$Q = A / q \quad \text{(4-15)}$$

Where, Q is Discharge, (m^3/s),
A is Area, (ha), and
q is unit discharge (ha/cumec) are as defined in chapter-3

Quantity of Duty: For Tank /pond irrigation, the duty is usually expressed as the total area of land which can be irrigated per million m^3 of water stored in the tank. If the duty and the area to be irrigated are known, the volume of water to be stored in the tank can be determined.

Volume of water, $V = A / q$ …………………………………………..…………………………… (4-16)

Where, V (Mm^3), A (ha), and q (ha/Mm^3)

Duty in the form of Total Depth (or Delta): It can be expressed in terms of the total depth (i.e. delta) of water required for a crop. It is another form of the quantity duty because the total depth is equal to the volume divided by the area of land.

Delta = Volume (ha-m)/ Area (ha) ……………………………………...……………………..… (4-17)

Delta is the quantity of water actually supplied to the crop. Delta included not only consumptive use of water for a crop but also the water lost by evapotranspiration and seepage from canals, and deep percolation in the field.

Note: To get the overall irrigation duty at the system head, take the calculated peak crop water requirements and divide by the overall efficiency.

4.6 STREAM SIZE

Unit stream size is flow in l/sec to furrows that can apply an adequate depth to a unit field in a day of 8-12 hours or intended application periods per day. Normally stream sizes up to 2.5 l/sec will provide an adequate irrigation provided the furrows are not too long. It is advised not to use stream sizes larger than 5.0 l/sec to prevent erosion in SSIP.

5 IRRIGATION SYSTEM LAYOUT AND THE COMMAND AREA

5.1 SYSTEM LAYOUT DESIGN

5.1.1 General arrangement

Irrigation system layout is a systematic arrangement of irrigation network system including command area boundaries, alignments of irrigation canals and network of natural drainage channels and related infrastructures starting from headwork up to outfall. The first and foremost approaches for a proper layout design are that drainage networks have to be identified by joining the lowest relevant points in the project area. Existing water rights must also be respected and included in designing the layout. These days such delineation can be done using different software like AutoCAD, Global Mapper and ArcGIS.

A primary concern in the layout of irrigation system is that it serves the purpose of conveying and distributing water to key locations in the delineated area. In addition to this, excavation and earth fill volumes not excessive, otherwise cost of the project can increase tremendously.

Irrigation canals can generally be aligned in any of the following three ways:

- As contour canal: Where we are interested to increase command area as in case of MC layout and reduce flow velocity as in case of TC and Furrow.
- As ridge canal or watershed canal: These are canals running on dividing ridge line i.e. between the catchment areas to allow supply on both sides e.g. Secondary canals are commonly laid in this manner.
- As side slope canal: A side-slope canal is that which is aligned at right angles to the contours; i.e. along the side slopes. Since such canals run parallel to the natural drainage flow, they usually do not intercept drainage channels, thus, avoid the construction of cross-drainage structures.

For designing such irrigation distribution system layout, a topographic map of scale of commonly 1:1,000 or 1:2,000 of the irrigation command area that extends back up to the proposed headwork site is essential. In case of extended ideal length between headwork site and initial command block, only strip survey along the proposed main canal route is required between these two extremes.

A primary concern in the layout of irrigation system is that it serves the purpose of conveying and distributing water to key locations in the area of service. Moreover, the layout shall be designed such that earth excavation and fill volumes are not be excessive as such imbalances may result in excessive project cost.

For surface irrigation application system, appropriate location of the diversion weir and/or storage dam including the irrigation outlet is essential to ensure adequate command over the proposed irrigation area. Then the main conveyance canals are positioned relative to this headwork site such that we make use of the natural ground slope and water flows downhill through the canals and enters the fields by gravity.

The main conveyance canal generally follows a contour but with slight gradient, and for steeper topography will be a one bank balanced cut and fill channel. Usually the secondary distribution canals are laid out at right angles to the main canal and across the contours thus results in a series of drops to convey the irrigation water to the lower irrigation plots. The flow and associated size of the main canal is reduced at each turnout to a secondary canal, if continuous supply otherwise uniform section for rotational supply system. Sometimes small tertiary canals are required to provide water to all the irrigation plots. A separate head ditch may be required for the irrigated farm plots. At the farm turnouts, the canal water surface must be high enough to permit irrigation of the land located at lower end of the plot (usually an allowance of 0.2 m is added to get the water on to the farm plot).

The recommended size of irrigation farm plots varies for each project and location in Ethiopia, depending on the availability of land and what each individual farmer and his family can manage. Generally for small scale irrigation projects, 0.2 to 2.0 ha irrigation farm plot sizes are recommended depending on nature of topography of the land.

One of the major concerns for the layout of the main canal, and particularly in areas with steep topography, is the number of drainage courses intercepted by the channel. Of more concern than the water flow in the drainage course is the sediment load. If flows are allowed to enter the canal, the channel can be over sized to handle the additional flow, or waste ways can be added to extract additional flow; however, in all probability the sediment entering the canal will remain in the channel and will restrict the capacity of the canal.

Sand excluders and canal sediment ejectors around cross drains for small scale irrigation projects are difficult to operate properly and requires high maintenance and will generally not be properly functional, thus such facility need to be avoided if possible. Alternately, drainage flow can be passed over or under the canal with a drainage crossing structure. Designing these structures to ensure that the sediment load is not deposited at the structure is difficult. Probably the only way to ensure that the drainage course is not restricted with associated sediment build-up is to provide a flume for the main canal flows. However, flumes are costly and are generally only used for major drainage courses.

Each surface irrigation and drainage system layout is dependent on the local situations of the project area. However, designing either system requires consideration of both horizontal and vertical alignment of canals.

5.1.2 Alignment of canals

In order to design irrigation system layout, manageable size of hydraulic units should be predetermined based on traditional irrigation water management, if any.

Command hydraulic units are the smallest building blocks for irrigation system design, operation and maintenance, which consequently defines the smallest irrigation and hence drainage boundaries. Size of these units depends on flow sizes that can be safely handled by an irrigator i.e. ranges of flow sizes that can be safely handled by an irrigator depending on the experience of the farmer, soil permeability, topography and method of watering. This value is estimated to be between 15 - 30 l/s.

Irrigation system layout must consider topographic factors and geographic features while selecting method of irrigation. For the suitability of each method (surface, sprinkler and drip) and even within surface irrigation methods, specific site conditions must be considered.

Factors that should be considered by the designer while selecting irrigation system layout include:

- Topographic features such as surface irregularities, steepness of slope, change in slope direction, soil depth;
- Geographic features such as field shape, natural drains, buildings, utilities or obstructions;
- Location and elevation of the water delivery and drainage systems, water use, location of field boundaries, field roads and row direction of crops to be grown for each field,
- Method of surface irrigation to be used and the regular farming operations should be considered by the designer in planning the overall irrigation system design. Generally, topographic features are adjusted through Land grading activities especially required for installation of surface irrigation systems.
- Where topography allows, field canals should effectively irrigate the fields located on both sides of the canal which is known as the herringbone layout.
- Furrows should be designed to run along contour line but slightly running away from them to create some gradient for water flow.

For a proper layout design of an irrigation network as per the so called MoWR guideline, it is crucial to undertake the layout design in three stages. These are:

- **Stage-1:** Contour maps should be indicated on the maps with intervals of 0.5 meter for flat areas and intervals of 1.0 meter for areas with terrain slopes of more than 2 percent. The preliminary layout prepared will serve as the basis for the preliminary canal design. Layout adjustments often will be necessary to obtain a better (more economic) canal design solution. Alternative layouts need to be reviewed before the best preliminary layout is obtained.
- **Stage-2:** Canal alignment (particularly for medium/large scale projects) as indicated on this layout needs to be set out and surveyed in the field, yielding an alignment and longitudinal section with levels. In addition main features, gullies and crossings' cross sections at an interval of 100 to 200 meters (based on the total length and topography of the canal route) shall be done for engineering quantity estimation and reconsideration possibilities of the layout if it proves uneconomical. The irrigation engineer together with topographic surveyor and geo-technical engineer need to check the physical alignment in the field to ascertain compatibility of the basic topographical map through ground truth verification. If all this is ok, then the final layout can be prepared. Setting out may not be justified for preliminary designs where accuracy requirement is relatively low; it can thus be done at the detailed design stage. Note: main canal profile surveying shall be undertaken at change in terrain and/or 20m interval allowing 50-100 meters from the centre to let for optimum design option.
- **Stage-3**: Back in the office, adjustment of the preliminary layout to best fitting and economical layout needs to be done on the basis of data obtained during ground truth inspections in Stage-2 and additional topographic data collected on canal alignment, as necessary. After the alignment of canals and drains is drawn in plan, areas served by different canals shall be calculated. Based on the layout map the location and type of structures needs to be determined.

The smallest irrigation development unit in the project i.e. field unit /FU/ and tertiary unit /TU/ boundaries shall be designed depending on different factors. The FU/TU size (usually 1 to 10ha for SSIP and up to 60 for medium and large SIP) in addition to slope/topography and soil mapping units are taken as the basic dividing blocks for designing layout of irrigation and corresponding drainage and other related infrastructures system.

The tertiary unit is meant here the irrigation area supplied by one tertiary off-take (if it supplies one side only). It consists of tertiary canals and field canals with their regulating structures. Thus the average tertiary unit size needs to be about 100-200m length by 100-500 m width for full-flagged block and half or less of this for partial blocks/units and even less on marginal areas. In general, this irrigation system layout needs to be planned and designed keeping in mind that:

- It fits best into existing topography;
- It serves the requirement of proposed crop;
- Irrigation water is able to reach every part of command area by gravity flow (if surface);
- Geological conditions along the alignment of the canal;
- Type of canal distribution system;
- Cost of construction and operation is minimum;
- It operates efficiently free from trouble i.e. as per plan.

For this purpose, a network of irrigation canals like main, secondary, tertiary and field canals and corresponding drainage canals like field, tertiary, collector and main drains or natural drains and other related infrastructures system (such as farm roads, Flood protection works) and on-farm irrigation structures, comprising various components starting from main conveyance to field canals needs to be systematically designed. This infrastructure Layout should also consists of bench marks, natural and artificial features, and legends for each feature. Note that the smallest units vary in size from location to location depending on landscape and marginality of the blocks. The project layout shall be prepared for different options and the best one should be selected from the point of view of technical, economic, management and convenience, etc.

In general, canal networks should be designed such that conveyance canals follow contour at a 1:1,000 or a 1:2,000 longitudinal slope or gradient of the canal route; secondary canals are aligned across contour on ridges so that they can feed tertiary canals (running parallel to contour) on its both sides or on single side as topography favors and field canals are aligned across contour to feed furrows that should run along contour.

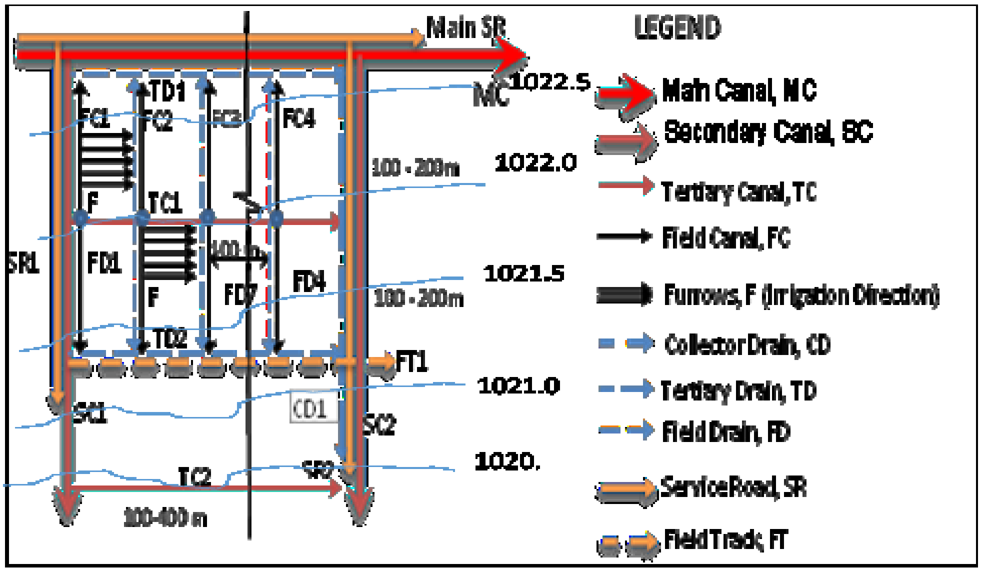


Figure 5-1: Typical schematic view of farm unit layout

While designing horizontal alignment of main canals in the system layout, we need to consider radius of curvature based on canal flow regime and its top width at the water surface as shown in table 5-1.

Table 5-1: Limiting radius of curvature for irrigation canal

Canal Flow Regime	Canal Type	Minimum Radius
Sub-critical velocity	Unlined	$7 \times T$ to $10 \times T$
	Lined	$3 \times T$
Super-critical velocity	Lined	Not permitted

Note: *T*-is width at the water surface, Source: USBR, 1967

5.1.3 Longitudinal profile of canals

The best way to present canal and drain design data for construction purpose is to draw a longitudinal profile of the canal route and to tabulate and present necessary data on it. The longitudinal profile shows the Chainage or distance along the canal at the horizontal or x-axis and the elevations of the natural ground, the canal bed and the FSL at the vertical or y-axis. These data are tabulated under the graph, showing the elevation of ground and canal bed in figures at each reach or distance. The Chainage need to be plotted on 1:1,000 on horizontal scale and 1:20 (small canal with low gradient) or 1:100 (long canal with high gradient) on vertical scale. The Chainage on this profile, starts from a reference point or the beginning of the canal and terminates at endpoint of that canal.

5.1.3.1 Alignement of Canals

General principles that need to be given attention in working the vertical alignment of canal are the following:

- The water level in supply canals should be sufficiently high enough to irrigate the highest level of the land for which irrigation is envisaged;
- A balance between cut and fill is considered as economical for construction of supply canal. Canals in high fill are more difficult to construct and would in general lose more water by seepage. This would be more certain if the bottom of the canal is above the original ground surface.
- The supply or distribution canal should have sufficient working head required at the off take or turnout. In the case of off take canal, the supply canal should maintain the minimum working head fixed at the planning stage. In case of turnout, it shall not be elevated above excessively or buried much below the original ground level significantly which would make the water supply to the turnout impossible to some part or the entire irrigated field or quaternary unit. The requirement of the agreement between the ground level and the bed level of the turnout shall be taken in to account.
- Drops are inevitable in canal design especially if the ground or natural slope is steeper than the canal bed slope. Attention should be given to avoid construction of drops on fill or embankment as it has less stability than on partly excavated material. The size of drop structures shall be made small say between 0.5m to 2m, if possible otherwise “Drop Design Manual by J Skutch TDR, 1997” recommends up to a maximum height of 5m under good workmanship).

5.1.3.2 Fixing full supply level of canal

While planning and designing the canal system layout, fixing the design water levels which are commonly called full supply level (FSL) at various points of canal reach as well as at bifurcation or offtake need to be defined. This will help to ensure that the desired flow of water from supply canal to offtake canal and finally onto the land surface is achieved. Generally, FSL and working head of a canal system is calculated starting from the field ground level, back through the system to all canals and structures, up to the source location such as river diversion point or vice versa. In medium and large scale irrigation schemes, this task is commonly called the command statement.

The full supply level and minimum working head requirements at the off taking points of different canal system components can be fixed based on the following:

i) Critical point: it is the spot, which requires highest water level due to the combined impact of spot level in terms of elevation and its distance from the irrigation channel/outlet, and

ii) Head over the field: it has been assumed that the depth of water should be a minimum range of 100 to 200 mm over the critical spot level.

Head loss from the quaternary/field canal to the critical point will be estimated along the proposed path of water flow after identifying the critical point and assuming appropriate slope for water surface.

5.1.3.3 Working head

Working head is the difference in the FSL of the parent channel and that of the off taking channel. It is provided in order to facilitate the flow of the design discharge. For preliminary purpose the minimum working heads provided in Table 4-2 shall be used as guideline. However, these values need to be checked with the actual required working head during the detail design study. The actual working head at the off-take will be the difference of heads between full supply level in the supply and offtake canal. However, it is customary to assume a minimum working head of 100mm to 200mm in Small scale canal network.

Table 5-2: Working head for different canals

Parent canal	Off-taking canal	Working head (mm)		
		For medium & large scale		For small scale
		Preferable	Minimum	Minimum
Main canal	Secondary canal	500	300	150
Secondary canal	Tertiary canal	300	150	100
Tertiary canal	Field canal	150	100	100

5.1.3.4 Working head at turnout

The minimum working head required at the turnout/off take can be fixed based on the maximum water depth (*d*) required at the critical point of the field supplied, the elevation or level of the critical point in the tertiary unit above project datum (*Z*), the sum of head loss ($h_{l\text{-}inlet}$) over the farm inlet. The starting point should be the critical point of the field.

$$h' = Z + d + h_{l-inlet} \quad \text{(5-1)}$$

Where *h'* is the required head in the supply canal (m).

5.1.3.5 Working head at secondary or branch off-take

The minimum working head required at the off take of secondary or branch canal can be fixed based on the sum of the working head requirement at the turnout plus the head loss as a result of canal bed slope over the length of the supply canal reach up to that offtake plus all the losses on structures along the reach up to that offtake.

5.1.3.6 Working head at main or primary canal intake

The main system should not only be able to allocate a peak flow to the irrigation unit according to the operational objectives, but also this peak flow should have sufficient head to irrigate the entire field. In general the required heading the main system at the intake of the headwork to irrigate the critical point (the spot in the field which requires highest water level due to the combined impact of spot level in terms of elevation and its distance from the distribution or outlet channel) is calculated as

$$h = Z + d + h_{l-inlet} + \sum(L \times S_{canal}) + (j \times h_{l-box}) + h_{l-offtake} \quad \text{(5-2)}$$

Where *h* is the required head in the main system at the intake above project datum (m), *L* is the canal length to reach the critical point (m), S_{canal} is gradient (when the highest or critical point is further away from the off take), h_{l-box} is the head loss over each of the *j* division boxes, and $h_{l-offtake}$ is the head loss over the tertiary off take in m.

It should be noted that the calculated required head in the main system using the above formula is a *minimum value*. Therefore, the water level in the water source shall be higher so that more head loss is possible over the system.

In general, the full supply level of the subsequent canal need to be fixed based on the requirements that sufficient head-loss is allocated at each structure location. Table 4-3 presents values used in medium and large scale irrigation scheme and can also be used as preliminary guideline. However, guidance on the minimum working head required at the different off-taking needs to be set at the beginning. During the detailed design phase, the actual design for water level need to be computed based on the types of out let structures to be provided. The full supply level fixed need to be checked against this minimum requirement.

Table 5-3: Ranges of canal head losses

No.	Description	Head Loss (mm)
1	Cross Regulators and Control Structures (Main Canal)	200 - 400
2	Secondary Canal Control Structures	300 - 400
3	Tertiary Canal Off take Structures	150 - 200
4	Flow Measurement Structures	100 - 200
5	Culverts (Pipe/Box)	50 - 100
6	Inverted Siphons and other crossing structures for carrying canal over/above drainage canals	300 - 500

Source: I&D System Design Training Material, GIRDC, 2015

5.2 FIXING THE COMMAND AREA

5.2.1 *Fixing boundary of potential command area*

Boundary of potential command area can be delineated based on different factors. Some of these factors are:

- Governing contour lines and hence the MC for surface application, but the potential resource for pumping system;
- Natural drainage network;
- Available net irrigable area;
- Available water resource in the river which is supposed to supply the area;
- Nature of Topographic Feature of the Command Area i.e. slope classes and brokenness

5.2.2 Contour or topographic map

A contour is an imaginary line connecting the same elevation on the surface of the earth. Numbers on the break of the line usually indicates elevation of the contour line. A contour map also called a topographic map is the simplest method of showing elevation on a two-dimensional sheet of paper. Thus, it is very important tool to show features of the command area.

Minor contours should be generated at 0.5m vertical intervals i.e. contour intervals for representing of flat topography, but 1m is enough for representing steep command areas. Major contours can be at 5m interval in both case, if visible but 10m can be used for the steeper topography case.

Table 5-4: Topographic features based on slope classes

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

Source: Guidelines for Irrigation Systems Design in Hills and Valleys, MoWR, Nepal, 2006

5.2.3 Contour map reading

To make irrigation system design, knowledge of reading maps is highly essential. Characteristics of these contour lines are:

- Evenly spaced contours show a uniform slope.
- The distance between contours indicates the steepness of the slope. Wide spacing denotes flat slopes: close spacing, steep slopes.
- Contours, which increase in elevation, represent hills whereas those which decrease in elevation represent valleys.
- Irregular contours signify rough and rugged topography.
- Contour lines tend to lay in parallel to each other on uniform slopes.
- Contours never meet except on a vertical surface such as a wall or cliff.
- Valleys are usually characterized by n-shaped contours, ridges by U-shaped contours.
- The V's formed by contours crossing a stream point upstream.
- The U's made by contours crossing a ridgeline, point down the ridge.
- All contour lines must close upon themselves either within or outside the borders of the map.

In general, the topographic map should show contour lines at a minimum at 0.5 m intervals on very flat, but 1 m increments might be desirable on irregular topography. When the slope exceeds 1%, the planner may desire a contour interval of 0.5 m or greater in order to have a more readable map.

All topographic maps show physical features of the irrigable area such as location and elevation of bench marks, location and source of irrigation water, existing field boundaries, drainage patterns and outlets, farmstead, farm roads, location of both buried and above ground utilities and any other physical features that may affect the planning of the system and the design of irrigation system design.

5.2.4 Identification of net command area

The command area is area which is identified as irrigable by the fixed intake level for surface and potential flow for pump system. It includes both suitable and unsuitable/gross areas within the boundary. Thus, the net command area is deduced from the gross suitable command area that needs to be used for design even excluding some percentage (say 3 to 5%) for areas occupied by infrastructure.

Indicative planning norms for land use of other projects' command area shows that a typical value of 3-6% of irrigation and drainage development is taken up by associated infrastructures from the productive or suitable command area but about 0.5 - 1% is enough for pressurized irrigation as the network is buried. In addition, 2-3% is expected to be occupied by roads and other infrastructures from the same land, if farm roads are introduced. In general, 3-4% is acceptable for the whole deduction on surface irrigation of SSI Project.

5.2.5 Recommended irrigable land slope ranges

This country is endowed with ample water and land resources. Unfortunately, where there are plenty of water resources, land resources may be steep thus difficult to use it for irrigation and where there are wide ranges of flat land resources, surface water resources may not exit. On the other hand, due to unreliability of rainfall distribution, these days it is becoming questionable thus investing as much energy as possible and using these resources is inevitable.

Consequently, sloppy land agriculture gives opportunities to use our resources effectively by integrating irrigation with various land management technologies to cultivate crops without affecting the soil resource. The followings are techniques used to develop such land by irrigation.

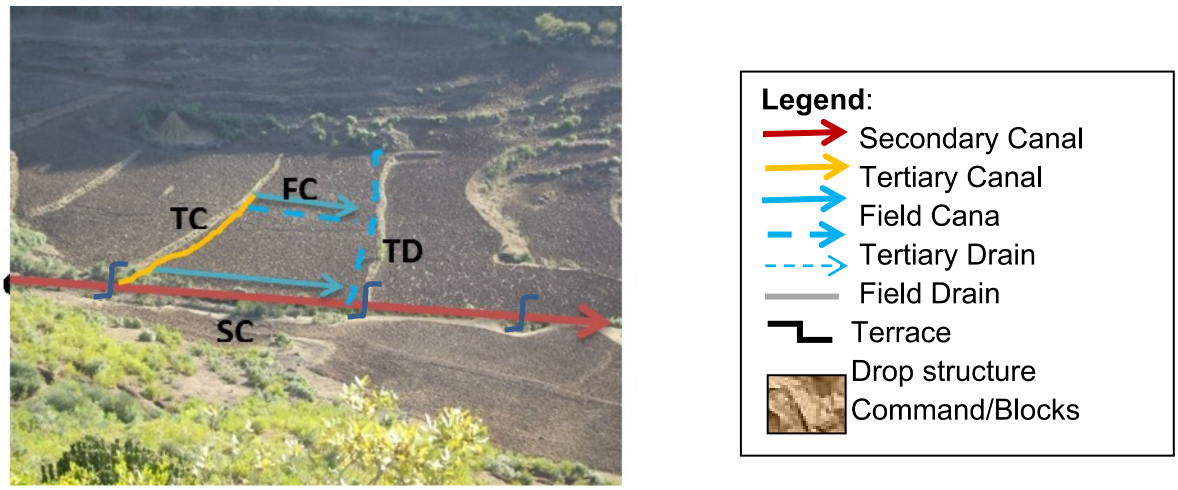


Figure 5-2: Arrangement of soil bund on land with steep slope (TNRS)

5.2.6 Bench terracing

Terracing is the levelling of the slopes along the contour lines in combination with the planting of crops. It refers to building a mechanical structure of a channel and a bank or a single terrace wall, such as an earthen ridge or a stone wall to protect erosion as it reduces slope steepness and divides the slope into short gently sloping sections of hillsides. Terraces are designed to intercept irrigation and surface runoff during heavy rainfall, encourage it to infiltrate, evaporate or be diverted towards a predetermined and protected safe outlet at a controlled velocity to avoid soil erosion.

The critical runoff velocity, at which soil particles that have been detached from soil aggregates begin to be transported over the surface, is 5 m/s in sandy soils and 8 m/s in clay soils (Rufino, 1989; FAO, 2000).

In some countries like in Jamaica, research has shown that a 90-95% soil erosion has been reduced by implementing bench terrace on steep slope land. Similarly in Sierra Leone, research results has shown that soil loss from 31% sloped land was reduced by bench terracing from 41-55 tons/ha/yr to 7- 5 tons/ha/yr (Millington 1982, Lal 2001).

So, a bench terrace is considered to be among the most effective structural erosion control and moisture retaining measures recommended to be used in this Guideline for a land slope up to 15% by surface irrigation. Thus, in areas characterized predominantly by steep terrain topography and dense populated settlement on uplands, conservation based irrigated agriculture is an alternative development opportunity. It is a question of time in highland of the country to start such type of agriculture to ensure sustainable livelihood.

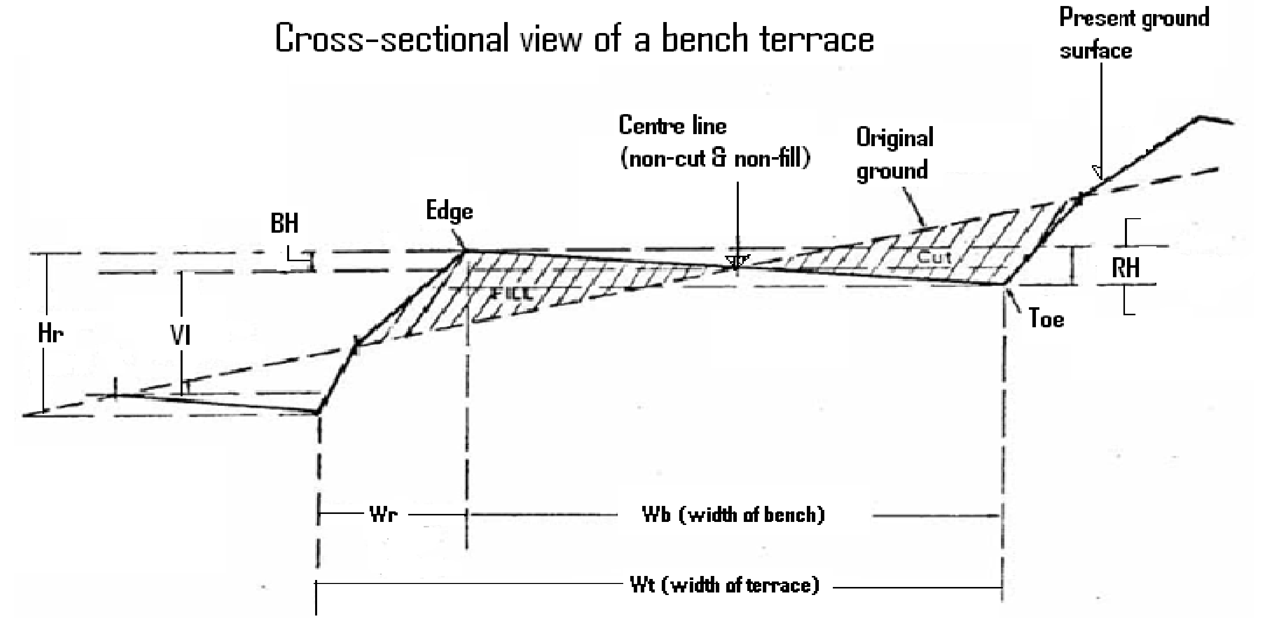
There are lots of experiences in the country in implementing irrigated agriculture with water harvesting technologies on bench terraced land. Bench terrace technology integrated with surface irrigation system is being practiced in SLM project weredas of TNRS like in Endmoheni Woreda, Embahasity micro-watershed to expand irrigated agriculture on sloppy areas. Such technique shall initially be exercised in areas where flat lands are limited but demand for irrigation is higher and then replicated in all areas of the country. The recurrent climate change like Illinoi phenomenon and intermittent rainfall in the country is enforcing factor to intensify irrigated agriculture in such sloppy areas.

In general, it is recommended to exercise two types of terraces: the first is soil bund terraces which is suitable for lands with the slope gradient ranging from 5-12% and bench terrace for sloppy areas where the gradient ranges between 12 – 15%. But still, such land needs to be levelled to recommended slope for surface irrigation of less than 5%.

Table 5-5: Recommended bund size per hectare for respective slope

Slope %	Bund size per ha		Work Norm PD/Km
	Bund width, m/ha	length m/ha	
0-5	-	-	-
5-8	1.25 m	450m	150
8-12	1.35m	700 m	150
12-15	1.35 m	700 m	150
15-20	1.45 m	1000 m	500
20-30	1.45 m	1000m	500
>30	1.5m	1500 m	500

Source: Petu SSI Project Design Report, GIRDC, 2016



$D = \frac{Wb}{2} \tan \emptyset$ (for level terraces)

$D = \frac{Wb}{2} \tan \emptyset + \frac{RH}{2}$ (for reverse – sloped terraces)

$D = \frac{Wb}{2} \tan \emptyset - \frac{OH}{2}$ (for outward – sloped terraces)

Where; D: depth of cut in m; Wb: width of bench in m; : tangent of the slope angle.

RH : reverse height; OH: Outward height

(viii) **Net Area**: This is the area in benches or flat strips which is used for cultivation. The net area can be calculated by using the following formula: Where A is net area of benches per ha in m^2; Wt: width of terraces (the sum of the width of the bench and the width of the riser), in m; Wb: width of the bench, in m; When calculating the net area of level terraces, the dyke width should be subtracted.

(ix) **Cross section**: The cross-section can be computed by the following formula:

$C_A = \frac{Wb*Hr}{8}$ Where, C_A: Cross-sectional area of the cut triangle, in m^2; Wb: Width of bench, in m;Hr: Height of riser, in m

The linear length of terraces per hectare can be calculated by the following equation: $L = \frac{10,000}{Wt}$

The linear length of terraces per acre can be calculated by the following equation: $L = \frac{43,560}{Wt}$

L: Linear length of terraces in one hectare, in m; Wt: Width of terrace, in m (where Wt = Wb + Wr).

The volume (V) can be calculated by multiplying the linear length (L) by the cross-sectional area (C_A); V = L * C_A ; When calculating the volume to be cut and filled it should be noted that only one cross-section is used. This is because the same cross-section is moving down slope to form a terrace.

For level terraces, the following formulas should be used for computing cross-sectional area:

$C_A = \frac{Wb*VI}{8} + DC$ Where, C : cross-section, in square m; Wb : width of bench, in m; VI : vertical interval, in m; DC: Dyke cross-section, in square m (or m^2)

For outward-sloped terrace a modification of the riser height (Hr) is required for calculating cross-section and volume as follows:

Hr = VI - OH; Hr: Height of riser; VI: Vertical interval; OH: Outward height (equals width of bench multiplied by 0.03)

Figure 5-3: Technical Design Considerations of Bench Terrace

5.2.7 Soil bund

Where a command area is endowed with deep soil on steep lands, soil bund is preferable than stone bund. Soil bund need to be supported with biological conservation technologies to stabilize the bund and control soil erosion more effectively. The width of walkways need to be wider to accommodate annual crop cultivation with furrow irrigation and the land need to be graded with slight inclination of 0.5% gradient towards the drainage or water way system of the irrigable land. The soil bund gives opportunity to develop suitable land with maximum width of 20meter which is convenient to use oxen ploughing on space between bunds.

In reference with GIZ-SLM manual the following width of cultivated land by slope and by potential soil erodibility is suggested and presented in Table 4-5. The command area has moderately erodible soils, the spacing between bund or the potential irrigable land width is an average of 15m.

Table 5-6: Recommended spacing between bunds

Slope (%)	Spacing (m)		
	Sandy soils (easily erodible)	Silt loam soils (moderately erodible)	Clay soils(less erodible)
3 – 8	**15 – 40**	**20 – 25**	**25 – 60**
9 – 20	8 – 14	8 – 19	10 – 24
21 – 40	4 – 7	5 – 7	5 – 10
41 – 50	3 – 4	3 – 4.5	4.6 – 5.8

Source: SWC training manual (unpublished), GIZ-SLM, Dire Dawa, (Hailu H., 2012)

5.3 OPTIMIZING THE NET COMMAND AREA

5.3.1 General

All available potential irrigation command area cannot be developed at a time as it is dependent mainly on available water potential in the source. However, it can be optimized by either one or more of the following factors even for the case when there is water stress:

- Introduction of On-farm Structures;
- Optimizing Crop Calendar and Cropping Pattern; and
- Improve efficiency;
- Use of deficit Irrigation.

5.3.2 Introduction of on-farm structures

This strategy works for the case when there is limited irrigation times a day. It can be managed by introducing night storage reservoirs just in the middle of the command area and irrigate area downstream of it. For design of this structure refer GL-B18: Canal Related Structures, which has been separately prepared for canal related structures.

5.3.3 Optimizing crop calendar and cropping pattern

There could be several options of crop calendar and cropping pattern for a single irrigation system. However, one can optimize it based on the available water supply and irrigation water management capacities or practices of beneficiary farmers.

5.3.4 Improving efficiency

This is another strategy to optimize the net irrigable command area by improving irrigation efficiency by lining the conveyance and distribution canal and/or use of pipe line conveyance.

5.3.5 Deficit irrigation

The scope for further irrigation development to meet food requirements in the coming years has been strongly diminished as a result of decreasing water resources and growing competition for fresh water. The great challenge for the future will be the task of increasing food production with less water, particularly in countries with limited water and land resources. In the context of improving water productivity, there is a growing interest in "deficit irrigation" – an irrigation practice whereby water supply is reduced below maximum levels and mild stress is allowed with minimal effects on yield. This irrigation technique is exercised where there is limited water resource but high demand of irrigation. This method takes into account maximum and actual crop yields as influenced by water deficits using yield response functions relating relative yield decrease and relative evapotranspiration deficits. It involves two methods as depicted in following sections.

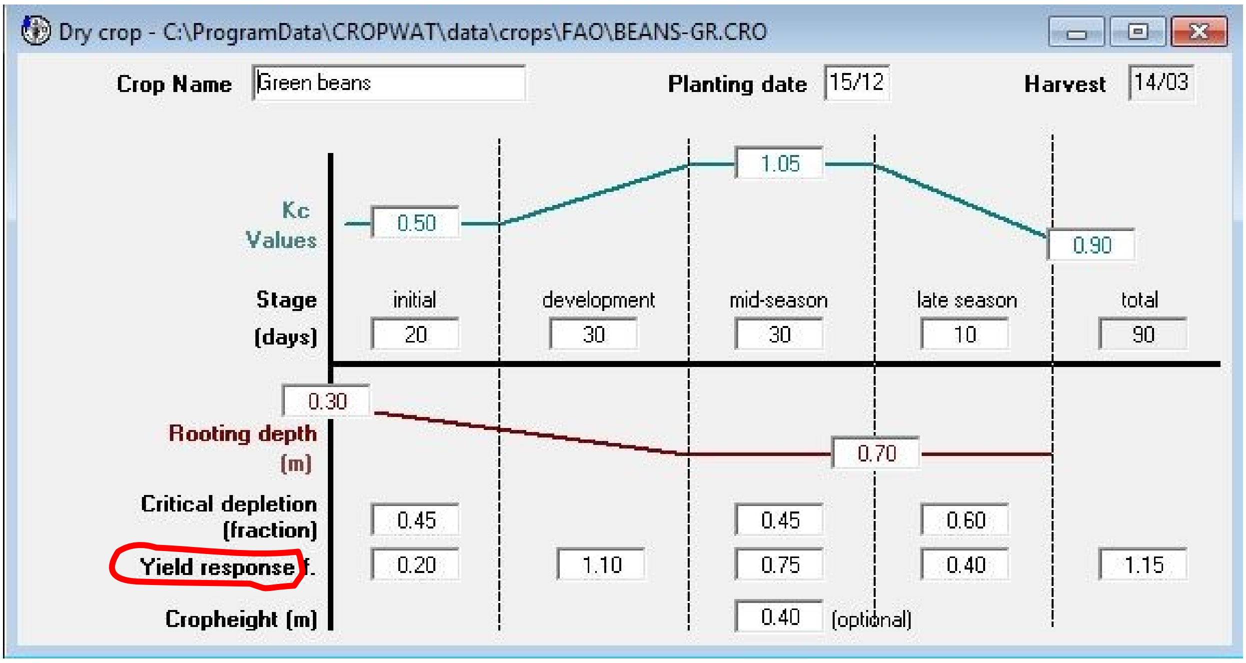


Figure 5-4: Yield Response as one function for regulating yield

In general, with increasing municipal and industrial demands for water, its allocation for agriculture is decreasing steadily. The major agricultural use of water is for irrigation, which, thus, is affected by decreased supply. Therefore, innovations are needed to increase the efficiency of use of the water that is available. For this purpose, there are several possible approaches and irrigation technologies and irrigation scheduling which can be adapted for more-effective and rational uses of limited supplies of water as mentioned above. Deficit (or regulated deficit) irrigation is one way of maximizing water use efficiency (WUE, which was achieved up to 1.2 times that under normal irrigation practices.) for higher yields per unit of irrigation water applied: the crop is exposed to a certain level of water stress either during a particular period or throughout the whole growing season. The expectation is that any yield reduction will be insignificant compared with the benefits gained through diverting the saved water to irrigate other crops. Note that, the grower must have prior knowledge of crop yield responses to apply deficit irrigation. Moreover, Irrigation scheduling plays decisive role on deficit irrigation as it requires careful evaluation to ensure enhanced efficiency of use of increasingly scarce supplies of irrigation water.

5.3.5.1 Regulated deficit irrigation

The latest research concepts and various practices involved for deficit irrigation has shown that, both annual and perennial crops were exposed to different levels of water stress, either during a particular growth phase, throughout the whole growing season or in a combination of growth stages. The overall finding, based on the synthesis of the different contributions, is that deficit or regulated-deficit irrigation can be beneficial where appropriately applied. Substantial savings of water can be achieved with little impact on the quality and quantity of the harvested yield. However, to be successful, an intimate knowledge of crop behavior is required, as crop response to water stress varies considerably.

The use of models can be an important tool to simulate crop water behavior under different conditions of water supply. The yield-response-to-water functions as developed by Doorenbos and Kassam, FAO Irrigation and Drainage Paper No. 33) were tested with the FAO CROPWAT model and applied successfully to evaluate and predict the impact of deficit irrigation on crop yield. The crop parameters used in the model include the crop response factor, which estimates relative yield reductions based on the measured reduction in crop transpiration. The factor is a useful indicator for the sensitivity and tolerance of crop and crop stage to water stress.

Analyses has shown that crops less sensitive to stress such as cotton, maize, groundnut, wheat, sunflower and sugar beet can adapt well to deficit irrigation practices provided good management practices can be secured. For more sensitive crops such as potatoes deficit irrigation proved less economic (Water Reports #22; Deficit Irrigation Practices, FAO, 2002).

In southeastern Australia, regulated deficit irrigation (RDI) of peach and pear trees increased WUE by 60 percent, with no loss in yield or reduction in vegetative vigor. In Washington State, United States of America, RDI of grapevines prior to fruit set (veraison) was effective in controlling shoot growth and pruning weights, with no significant reduction in yield. RDI applied after veraison to vines with large canopies resulted in greater water deficit stress. Wine quality improved with pre-veraison RDI applied as compared to post-veraison RDI. RDI applied at any time resulted in better early season lignification of canes and cold hardening of buds.

The response of yield to water supply is quantified through the Yield response factor (Ky) which relates relative yield decrease to relative evapotranspiration deficit. In general, for the total growing period, the decrease in yield is proportionally less with the increase in water deficit (Ky < 1) for crops such as alfalfa, groundnut, safflower and sugar beet while it is proportionally greater (Ky >1) for crops such as banana, maize and sugarcane. For the individual growth periods the decrease in yield due to water deficit during that growth period is relatively small for the vegetative and ripening period and relatively large for the flowering and yield formation period.

The relationship between crop yield and water supply can be determined when crop water requirements and crop water deficits, on the one hand, and maximum and actual crop yield on the other can be quantified. Water stress in the plant can be quantified by the rate of actual evapotranspiration (ETa) in relation to the rate of maximum evapotranspiration (ETm). When crop water requirements are fully met from available water supply then ETa = ETm; when water supply is insufficient,. ETa < ETm. For most crops and climates ETm and 'ETa can be quantified. Crop yield response data from deficit irrigation were fitted to the following linear equation used earlier by Stewart *et al*. (1977):

$$\frac{Y}{Y_m} = 1 - k_y\left[1 - \frac{ET_a}{ET_m}\right] \quad \text{(3-1)}$$

Where, Y is expected crop yields, corresponding to ETa, actual evapotranspiration,

Y_m is maximum crop yields, corresponding to ETm, max. Evapotranspiration,

ky is a crop yield response factor that varies depending on species, variety, irrigation method and management, and growth stage when deficit evapotranspiration is imposed. The crop yield response factor gives an indication of whether the crop is tolerant of water stress. A response factor greater than unity indicates that the expected relative yield decrease for a given evapotranspiration deficit is proportionately greater than the relative decrease in evapotranspiration (Kirda *et al.*, 1999a).

Box 5-1:

Worked Example-3: A farmer has ploughed 2 hectares of maize farm by irrigation and obtained maximum crop yields of 90qt/ha corresponding to maximum Evapotranspiration of 504mm peak Evapotranspiration per season and 40qt/ha corresponding to actual Evapotranspiration of 504mm peak Evapotranspiration per season. Climatic conditions of the study area has the following features. How much is the monthly crop yield response factor?

Table 5-7: Mean data from long-term climatic data

Month	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual/ Avg.
Rainfall (mm)	50.6	49.5	115.8	166.3	222.3	250.4	243.1	321.9	217.9	162.4	142.6	74	2016.8
Min. Temp. (^{0}c)	17.2	18	18.1	17.1	16.2	15.3	14.6	15	15.7	15.9	16.2	16.5	16.32
Max. Temp. (^{0}c)	23.1	24	24.1	22.8	21.8	20.5	19.5	20	21	21.4	21.9	22.2	21.86
Rel. Hum. (%)	52	53	58	67	72	73	71	71	66	72	64	60	65
Wind speed (km/d)	130	138	147	147	147	112	112	104	130	130	112	104	126
Sunshine (hr)	7.0	6.4	6.3	6.6	6.2	5.2	3.6	4.1	5.1	6.7	7.2	7.5	6

Note: Calculation Procedures has been presented in appendix-iv

Solution: *First of all we have to compute monthly potential ETo from FAO CROPWAT8 as follow.*

Given:		
Irrigation Area	Unit production	A = 2.0 ha
Max. production	90.0 qt/ha	Ym = 180qt
Actual production	40.0 qt/ha	Y = 80qt

ETC = Kc * ETo and $K_y = \left(1 - \frac{Y}{Y_m}\right) \Big/ \left(1 - \frac{ET_a}{ET_m}\right)$

Table 5-8: Computed maize yield response factors for the given condition

Month	Rainfall	Min. Temp.	Max. Temp.	Rel. Hum.	Wind speed	Sun	Rad	Peak ETo			Kc Maize	Actual ETc Maize		Yield response, Ky
	(mm)	(0c)	(0c)	%	km/day	Hrs	MJ/m²/day	mm/day	mm/month	mm/season		mm/month	mm/season	
Jan.	50.6	17.2	23.1	52	130	7	20.5	4.36	135.2	504.0	0.30	40.6	326.8	0.79
Feb.	49.5	18	24	53	138	6.4	19.8	4.44	124.3		0.70	87.0		1.85
Mar.	115.8	18.1	24.1	58	147	6.3	19.4	4.32	129.6		1.05	136.1		(11.11)
Apr.	166.3	17.1	22.8	67	147	6.6	18.7	3.83	114.9		0.55	63.2		1.23
May	222.3	16.2	21.8	72	147	6.2	16.7	3.28	101.7					
Jun.	250.4	15.3	20.5	73	112	5.2	14.6	2.77	83.1					
Jul.	243.1	14.6	19.5	71	112	3.6	12.7	2.54	78.7					
Aug.	321.9	15	20	71	104	4.1	14.4	2.79	86.5					
Sept.	217.9	15.7	21	66	130	5.1	17	3.43	102.9					
Oct.	162.4	15.9	21.4	72	130	6.7	20.1	3.78	117.2					
Nov.	142.6	16.2	21.9	64	112	7.2	20.8	4.01	120.3					
Dec.	74	16.5	22.2	60	104	7.5	21.1	4.09	126.8					
Total/Avg.	2016.8	16.32	21.86	65	126	6	18	3.64	1321.2					

Note: Water deficits from onset of flowering to peak flowering may cause a more negative effect on yield as compared to when occurring after peak flowering.

5.3.5.2 Partial root zone drying

In addition to RDI, partial root zone drying (PRD) is also a promising practice for inducing stress tolerance in fruit trees. PRD is a new irrigation technique that subjects one-half of the root system to a dry or drying phase while the other half is irrigated. The wetted and dried sides of the root system alternate on a 10-14-day cycle. Both RDI and PRD systems require high management skills. Close monitoring of soil water content is recommended. Both practices improve the WUE of wine grape production.

6 DESIGN OF CANALS

6.1 CANAL TYPES

Canals can be categorized in to different types based on different factors.

6.1.1 Classification based on canal cross-section

According to the shape of their cross-section, canals can be classified in to rectangular, triangular, trapezoidal, Semi-circular, parabolic and irregular or natural canal. Selection of these sections depends on nature of topography along canal route, discharge capacity, efficiency, etc.

Table 6-1: Indicative parameters for selecting types of canal cross-section

Parameters	Canal Cross-Section			
	Triangular	Rectangular	Trapezoidal	Parabolic/Semi-circular
Land slope	Used on flatter slope	Preferred on steeper slope	Commonly Used on flatter slope	Can be used anywhere
Discharge	For on-farm distribution	For lesser discharge	Preferred for higher capacity	Preferred for higher capacity
Efficiency	Less efficient	Efficient since it is used as lined canal	Less efficient as it is commonly unlined	The most efficient
Land take	Less land take	Less land take	More land take	Less land take
Cost	Less costly	More costly as it is unlined	Less costly as it is unlined	The most costly as it is lined
Easiness of construction	Easy	Relatively easy	Relatively difficult	The most difficult
Practice	Not common & used for unlined canals	Common for MC running on hills & SC only as lined	Commonly used for unlined canals	Not common for SSIP

Source: From own experience

6.1.2 Classification based on size of discharge

Canals can be classified as MC, SC, TC and FC based on their carrying capacity; however, depending on the project nature, all canal types and infrastructures may not be found in one system. For example, in most small scale irrigation schemes, tertiary canals takeoff water from the secondary or main canals.

6.1.3 Classification based on proneness of water surface to air

Based on condition of proneness of water surface to air, canal can be categorized in to open canal and closed canal. An open canal, channel, or ditch, is an open waterway to carry water from one place to another by gravity i.e. due to head difference. Whereas, closed canal is a buried canal in which flow is driven by pump or gravity pressure. Closed canals can be by Pipe flow or canal covered by concrete) and are selected when topographic condition does not allow construction of open canal and/or there could be fear of land slide.

6.1.4 Classification based on lining condition

Canal linings can be broadly classed into three groups: hard surface, membrane and earth linings. Hard Surface Linings include linings with concrete, shotcrete (In shotcreting, concrete is sprayed under pneumatic pressure rather than placed between forms), soil-cement, asphaltic concrete, brick and masonry linings. Membrane linings fall into two categories - exposed membranes and buried membranes. Burying membranes results in a longer life for most linings since nearly all the materials are subject to ultra violet, ozone and biological attack. Where suitable materials are locally available, earth is the cheapest form of lining. Earth linings are usually compacted and are classified as thick linings (0.3 - 1.0 m) or thin linings (0.15 - 0.3 m). Thick compacted earth linings are preferred for several reasons including easier construction, better erosion resistance and better resistance to damage during maintenance. In practice the USSR have shown that thick linings are more economical than thin linings in the long term due to the difference in cost of maintenance. Thin linings are also susceptible to damage caused by wetting and drying (e.g. in canals operating on rotation).

Thus, in such classification, canals can in general be earthen or unlined canal and lined canal. Earthen canals are simply dug in the ground and the bank is made up from the removed earth. The disadvantages of earthen canals are the risk of the side slopes collapsing and the water loss due to seepage. They also require continuous maintenance in order to control weed growth, erosion and to repair damage done by livestock and rodents. In case of lined canals, earthen canals are lined with impermeable materials to prevent excessive seepage and growth of weeds (For details refer section 6.4.12).

6.1.5 Classification based on service

Canal classification based on their service can be subdivided in to irrigation canal (for transporting/conveying irrigation water), power canal (for hydroelectric power generation) and ship canal (for transportation). However, in this guideline, we concentrate on irrigation canal only.

6.1.6 Classification based on nature of soil

Depending upon nature of soil through which they pass, canals can also be categorized as alluvial canals or non-alluvial canals. Alluvial Soil is the soli which is so formed by the continuous deposition of silt from the water flowing through a given area. The canals when excavated through such soils are called Alluvial Canals. On the other hand, canals, passing through regions of hard and rocky plain areas are stable and are called non-alluvial canals.

6.1.7 Selection of canal type

The canal type must be determined on the basis of the designed/peak discharge and designed level in consideration of the natural and social environments of the route, economy, water use, water requirement, operation and maintenance and other conditions, so that the purpose and function of the entire canal system may be fully achieved. The selection of the canal type greatly affects the function of the entire canal system, and significantly affects the construction costs of canals. It is, therefore, necessary to consider the conditions of the costs and future water management and maintenance system in the selection of the canal type, aiming at the entire fulfillment of its purpose and function.

6.2 HYDRAULIC DESIGN CRITERIA AND CONSIDERATIONS

6.2.1 General

Hydraulic design of canal system is the first task for the designer, once a layout has been fixed. The main hydraulic considerations are: Water Levels, Canal Capacity or Discharge, Sizes of Canals and Structures, Safety (Energy Dissipation & Escapes), Distribution (control and management), longitudinal slope, Cost (Construction and O&M), environmental conservation, Right-of-way (easements) along the canal path, Cross-channel surface drainage requirement, Need for emergency spill requirement, Secondary uses (such as clothes washing & taking bath, livestock watering) and Aesthetics. Structural failure considerations like slope stability are also mandatory for designing canal sections and profiles, especially those in fill section.

As a general design criteria, flow rate capacity and construction cost are the dominant design criteria, though it is necessary to consider all the mentioned criteria.

6.2.2 Water levels

For successful irrigation system requirement, the headwork should be sited so that it can irrigate the command area intended to be developed. In doing so, adequate allowances must be made for canal slopes, head losses through structures and for the head required to extract water from the source. In many cases, it is desirable to locate the headworks as far downstream as possible to provide the maximum catchment area, and hence the maximum amount of water. In such cases however, accurate leveling can be critical to ensure that the command area can be supplied at its potential.

6.2.3 Canal capacity or discharge

This parameter is an important consideration and is determined by the field water requirements and efficiency of the conveyance and distribution systems. In carrying out a preliminary design, it is usual to assume certain crops and cropping pattern, and a system efficiency, and then determine the water requirements on a monthly basis taking into account the infiltration rates and rainfall. If the resulting water requirements exceed the water availability, then the cropping pattern has to be revised and/ or the system efficiency improved. If there is sufficient water, then the cropping pattern may be revised to increase the cropped area or the intensity, if this is practical. Actual required delivery system capacity depends on:

- Size of the irrigated area;
- Cropping patterns (Crop type, planting and rotation schedules);
- Climatologic conditions;
- Conveyance efficiencies;
- On-farm efficiencies;
- Availability and exploitation of other water resources (conjunctive use);
- Type of delivery schedule (Continuous, rotation, on-demand); and
- Non-agricultural water needs.

It is often recommendable to allow for a safety factor by increasing capacities by 10 to 20% in case crops change, an expansion in irrigated area occurs, conveyance losses increase, and other possible factors.

6.2.4 Sizes of canals and structures

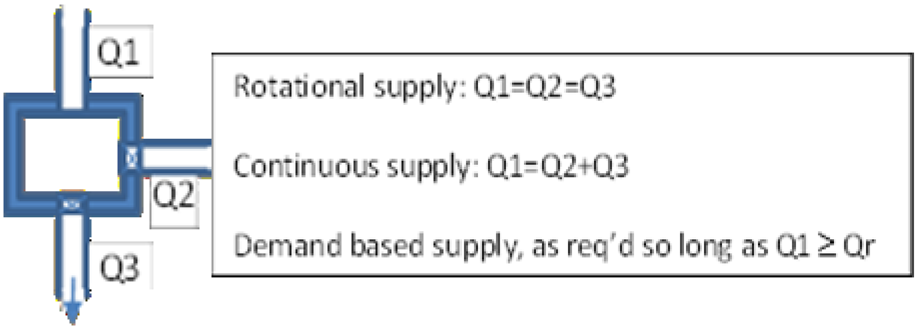
Canals are usually designed to carry peak design discharge with provision of free board for some surcharging or siltation. However, canals those running along hill slopes or have surface water inlets need to be oversized to allow for excess flows. Structures are also designed to pass more than the design flow (commonly 25% or more) to allow for any surcharging. Such size also depends on longitudinal slope as its selection is essentially a compromise between the various design factors, i.e. a steeper canal has a smaller section, takes less land and has a lower excavation requirement, but may cause erosion, have problems from fluctuating water levels and not be able to command all the potential area. A flatter canal on the other hand, can command the maximum area, have little water level fluctuation and be easier to control but may suffer from siltation, encourage weed growth and mosquito breeding, take more land and require greater excavation.

6.2.5 Safety (energy dissipation, escapes)

This involves provision of emergency escapes to dispose of surplus water brought into the canal from surface flows, inlets or flood waters. Each canal will have a deferent requirement depending on the amount of water that is likely to enter the canal. Escapes should be automatic, requiring no operation, to deal with unexpected surplus flows. (Gated escapes may also be provided to assist with draining the canal for maintenance purposes.) Emergency escapes should be located at frequent intervals along the canal, particularly on hill canals where a lot of surface water inflow is unavoidable, and can be combined with cross-drainage structures.

6.2.6 Distribution (control and management)

Planning of such system should be based on the proposed distribution system. To simplify control, a proportional distribution system is recommended whereby the water is delivered through each tertiary head regulator in proportion to the land area. During times of low flows, some form of rotation should be operated, so that the secondary canals are either running at near design discharge, or are closed.



……………..…. (6-1)

Figure 6-1: Schematic Representation of Flow Distribution Options at Division Boxes

6.2.7 Criteria for canals in hills

The following general principles of design are as adopted while designing irrigation projects in the hills and valleys of Nepal:

Right:

- Keep shortest possible length of canal in the hills
- Use local skills whenever possible ·

- Use local materials whenever possible
- Use designs so that farmers can maintain and repair
- Preserve vegetation on hill slopes above and below canal
- Cut slopes to uniform gentle gradient (<35° in soils) and vegetate where possible
- Design canal to meet crop-water requirement and other domestic needs so ensuring minimum cross- section excavation
- Construct sufficient automatic escapes onto hard rock or into natural drainage lines
- Line all canals with serious seepage and slumping problems, maximizing use of local materials
- Cover all canal passing through areas prone to toppling or spalling slope failure
- Minimize blasting and use tunnelling and appropriate bench cutting where possible
- Use temporary channel structures on unconsolidated debris
- Protect bare, eroded and unstable hill slopes with appropriate vegetative measures
- Place and compact fill carefully

Wrong

- Cut slopes in mountainside are too steep and irregular
- Loose rock pieces are not removed from cut-slopes during excavation
- Cross sections are too large involving more excavation than is necessary
- Fill is not placed and compacted properly and is therefore easily eroded or quickly slides down the hillside
- Insufficient automatic escapes are provided
- Lining and covering of the canal is insufficient
- Spoil is disposed-off in an uncontrolled manner
- Vegetation is destroyed and not protected or replaced
- New fill areas are not planted with vegetation
- Existing landslide areas are not recognized and expensive structures (like rigid lining) are soon damaged by subsequent sliding
- Inappropriate design does not allow farmers to maintain or repair the canal after construction.

6.3 CANAL FAMILIES, NOMENCLATURE & DESIGN CONSIDERATIONS

6.3.1 Description of canal categories

The following irrigation canal categories have been used in this Guideline.

- Main Canal– MC (that can be a pipe or a canal);
- Branch Canal – BC (required only in case of larger areas)
- Secondary Canal – SC;
- Tertiary Canal – TC
- Field Canal/Ditch – FC
- Furrows - F

Canals should be numbered to the standard, starting from the headwork side and working toward the end of the command area (i.e. from upstream to downstream). The left and right banks can also be included just to identify their location. Thus the suffix L or R, need to be used to indicate off-taking side as Left or Right respectively. For example, RSC1 is to mean the first secondary canal on the right side; similarly, TC1-2 is to mean the second tertiary canal being feed from secondary canal-1; moreover, FC1-2-3 is meant the third field canal on tertiary canal-2 and first secondary canal.

6.3.2 Conveyance system

This is the main supply system to either branch systems or secondary systems or even tertiary and field canals depending on their location. It can be either Canal (lined and/or earthen) or pipe supply system. Thus, it has to be studied in detail before deciding on type of conveyance as indicated in the following comparison table.

Table 6-2: Comparison of pipe and canal conveyance options

Nr	Consideration	Pipeline Conveyance	Canal Conveyance
1	Conveyance efficiency (water loss)	Water losses in a pipe conveyance system are negligible	Water losses from canals varies largely depending on whether lined or unlined; type and condition of lining; soil material in which the canal is constructed; whether in cut or fill and depth to water table; etc.
2	Layout /alignment	Pipelines are not constrained by topography, and layouts can be optimized following direct routes to outlet / hydrant locations.	Canals need to be aligned down-slope, and depending on topography may follow meandering alignments to around ridges and valleys.
3	Land slope	Velocity limits in pipelines need to be observed, but excess head may be burnt off using pressure reducing valves inserted in the pipeline. These are typically cheaper than drop structures for canals giving pipelines a comparative advantage for steeply sloping land.	Velocity limits to canals must be observed, even for lined canals. For steeply sloping land, the required drop structures can greatly increase project costs.
4	Land take	Pipelines are generally buried with negligible land take	Canals take up wider land
5	Sediment	Pipelines with slow velocities carrying sediment are likely to blockages	Removal of sediment from canals is not usually a problem
6	Health and safety /H&S	Pipeline conveyance reduces chance of contamination, and there are little H&S issues	Canal flows can pose H&S issue, particularly large, fast flowing canals and ponded water also.
7	Construction	Pipeline laying, bedding and jointing needs to be done carefully	Unlined canals are cheaply constructed and may be upgraded by lining later. Labour often poses a significant proportion of the cost.
8	Design life and maintenance	The design life of pipes varies from 15-50 years depending on pipe type, soil conditions, etc., assuming proper construction.	Canals can be maintained in-definitely. Design life for concrete lining depends on quality of construction particularly embank-ment compaction and concrete quality. Water loses through cracks can be high. Design life varies from 5-30 years.
9	Operation	Control devices enable easy operation. Procedures for starting -up and closing-down the system must be observed to avoid water hammer and pipe bursts.	Appropriate choice of regulating and flow measuring (gated) structures facilitate relaxed operations.
10	Cost	Pipes are expensive and may need to be imported. However pipeline conveyance has a comparative advantage on steeper slopes and where more direct alignment is possible than by canal.	

Source: Halcrow-GIRDC I&D Study Report for 80,000ha Net Ethiopian Nile Irrigation Project, 2010

If pipes are our preferred option, then the following table should be considered for choosing the best pipe material. As a general criterion, buried GRP pipes are suggested for larger flows where pipe diameters of 400 mm or larger are required. For smaller flows buried uPVC or HDP pipes can be considered.

Table 6-3: Analysis of conveyance pipe options

Nr	Material	Typical Diameters (mm)	Jointing	Nominal Working Pressure, PN (bar)	Average Life (years)	Suggested Use
1	Steel : spiral welded (bitumen painted)	500 – 2,500	Welding	As required (depends on class of pipe, steel wall thickness)	15-20 years on surface; 10-30 years buried depending on soils & pipe protection	Due to high cost usually only used for crossing gullies, etc.
2	Steel: threaded (6 m lengths; galvanized)	½" to 3"	Screw-type threaded joints			Due to high cost used only as fittings / connections at structures.
3	GRP	300 to 2,000 mm	Couplings with elastomeric seals	10 & 16 bar (other pressure classes possible)	50 years buried with proper trenching / bedding	Irrigation water conveyance where larger diameters are required. Careful handing necessary to avoid damage.
4	Aluminium (6, 9, 12 m lengths)	2,3,4,5 & 6"	-	10 or 16 bars	15 years with good management to avoid denting/ kinking.	With quick coupling used as hand-move laterals for sprinkler systems, etc.
5	Rigid uPVC (6 m lengths)	50 to 225 mm	Solvent (small);spigot & socket (large)	4, 6, 10 & 16 bars	50 years buried with proper trenching/ bedding	Ideal for irrigation water conveyance for smaller pipes. Maximum flow velocity, 1.5 m/s
6	PE (50-400 m length coils)	Range from 12-110 mm	Range of fitting available depending on pipe size	2,4,6,10 & 16	10-15 years on the surface; much longer buried with proper trenching/ bedding	LDPE – flexible: may be used as hoses / laterals. HDPE – rigid: may be used as laterals as well as buried pipelines. Maximum flow velocity 1.5m/s

Source: Halcrow-GIRDC Study Report for 80,000ha Net Ethiopian Nile Irrigation Project, 2010
Note: Hoses used for drag-hose sprinkler systems are plastic hoses that are textile reinforced. In such cases ratio of minimum burst pressure to design working pressure shall be 3.

6.3.3 Advantages of using HDPE as compared to Cast iron

- It has no reaction with sewage and seawater and does not suffer from the corrosion problem associated with cast iron;
- No protective coating is required and it is almost maintenance-free;
- HDPE flap valves require very low opening pressure in operation (like 5mm water level difference). For cast iron flap valves, due to its own heavy self-weight, the required opening pressure of cast iron flap valves is higher than that of HDPE flap valves. This criterion is essential for dry weather flow conditions;
- However, the pressure resistance of HDPE flap valves is not as good as cast iron flap valves. For instance, a typical 450mm wide HDPE flap valve can only withstand about 5m water column.

6.3.4 Differences in installing cast iron and ductile iron manhole covers

Traditionally, manholes covers are made of cast iron. However, in the viewpoint of pipe maintenance, frequent opening of manhole covers has to be carried out. Therefore, it poses potential safety hazard to the workers during the lifting-up process of manhole covers because cast iron manhole covers are very heavy to normal workers. Consequently, research has indicated that ductile iron is considered as a better choice than cast iron because it can resist the same traffic loads with lower self-weight.

Moreover, as ductile iron is less brittle than cast iron, the traditional cast iron manhole covers are more susceptible to damage and thus requires higher maintenance cost. However, ductile iron manhole covers do suffer from some demerits. For instance, owing to their relative low self-weight, vehicles passing over these manhole covers would lead to the movement of covers and generate unpleasant noises. To solve this problem, instead of increasing the self-weight of ductile iron manhole covers which similarly causes safety problems to workers during regular maintenance, the covers can be designed to be attached to the manhole frames which hold them in firm position.

6.3.5 Design considerations of main canals

The layout of this canal is the most important and decisive component of the entire irrigation planning work, that call for most careful consideration of all the factors governing the alignment: such as topography, natural drainage pattern, land suitability, etc. thus, main canal is aligned nearly along contour lines expecting to minimize loss of head and maximizing command area. Consequently, main canal layout need to be designed in partial cut and partial fill at most except for few meters of localized depressions, which requires full fill work.

When sizing the MC, its provided capacity should be checked to ensure that it is sufficient to meet peak demand. Moreover, following head losses shall be considered.

- Canal slopes of 1-2m/km or more for hill medium and small schemes
- Head losses of 0.20m at cross drainage sites
- Head losses of 0.25m at cross-regulator sites
- Head losses of 0.10m at road crossings (assuming culverts)
- Head losses of ±1.0m at intake (for sediment control and management)
- Safety arrangements to facilitate safe disposal of excess canal flow
- Interceptor/catch drains located adjacent to access road along MCs need to be designed to intercept surface or seepage flows and increase stability of the MCs

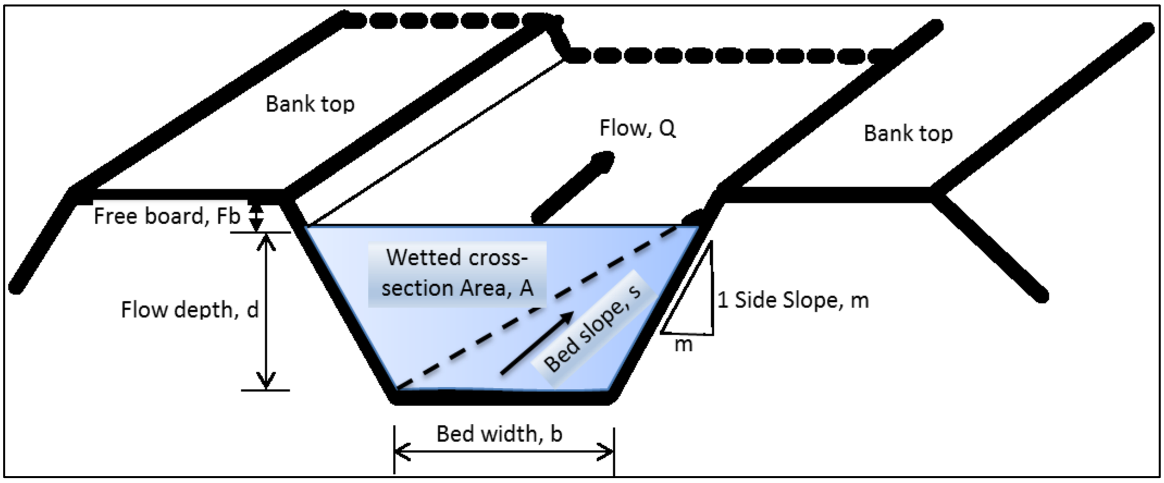


Figure 6-2: Typical trapezoidal main canal cross-section

6.3.6 Design considerations of secondary canals

These are canals receiving water from MC and are designed such that they are aligned across contour directions so that they can feed tertiary canals running parallel to contour. Thus, they comprise breaking erosive nature of flow by introduction of vertical drops/falls and turnouts to tertiary canals. In case of large scale irrigation projects they involve head-regulators at their beginning.

The command level in the secondary canal is calculated from the level at the tertiary head plus the head loss in the tertiary head regulator (H_L). This head loss will depend on the type of structure used, but will typically be 0.20 to 0.30 m. For planning purposes, where the types of structure have yet to be determined, an allowance of 0.25 m is appropriate.

6.3.7 Design considerations of tertiary canals

These canals are the smallest unit next to field canals from which command area need to be planned for irrigation in the project boundary. These tertiary canals take-off from secondary or sometimes from main canals and supplies irrigation water to field canals. They are usually designed of trapezoidal cross section from unlined i.e. earthen material. Off-takes are provided on such canals to control water level being diverted to FCs.

In case fall of WL in SC encounters, check structures shall be used to raise & divert to corresponding TC. Carrying capacity of these canals is determined based on beneficiaries preferred application times assuming rotation will be considered within this unit (unlike secondary canals which are commonly designed for 12 hours of application and MCs for 24 hours). Thus, water shall be stored for the remaining 12 hours in the night storage reservoir to irrigate additional area by the stored water, but this need to be confirmed if beneficiaries support the idea.

Manageable size of Tertiary units vary between 2 to 10 ha depending on its location (i.e. if it is on marginal area then it will be of the actual size found, otherwise the designed size holds true). The tertiary/farm unit in small scale irrigation project is the irrigation area supplied by one tertiary off-take. It consists of tertiary canals and field canals with their structures. Thus the average tertiary size shall be about 100-150 by 200-2000 m for full-flagged block and lower than these values for partial blocks/units.

6.4 DESIGN CONSIDERATIONS OF CANAL PARAMETERS

6.4.1 General approaches to canal design

Canal design can be approached in two ways: stable canals and alluvial canals. An alluvial canal is defined as a canal in which flow transports sediment having the same characteristics as that of the material in the canal and bottom. Such a canal is said to be stable if the sediment inflow into a canal reach is less or equal to the sediment outflow. Thus, the canal cross section and the bottom slope do not change due to erosion or deposition in stable canal case.
Processes of stable canal design can be approached in one or more of the following different directions. However two different approaches have been used for the design of stable alluvial channels: tractive force method and regime theory (which are discussed in sections 5.4.5 and 6.5.7).

1) Discharge or capacity & longitudinal slope are known, and object is to determine normal flow depth and permissible velocity;
2) Discharge is known, and the object is to examine a range of canal slopes to see effect on depth and velocity (This is what we commonly do for new SSI Projects);
3) Longitudinal slope & depth are known, & object is to find discharge capacity (This is a common situation in rehabilitation of existing schemes, or as a rough guide to estimating drainage flows in ditches or natural channels);
4) Discharge & slope are known but velocity needs to be restricted within ranges of non-silting or non- scouring. The object is to test out several different bed widths or profile shapes;
5) Canal slope is known and object is to know increase in depth and velocity in the event of a flood surcharge;
6) Object is to compare cost of several different canal profiles and shapes having same conveyance;
7) Object is to know area of critical flow conditions to avoid or accommodate supercritical flow;
8) Object is to test effects of low flow or flood surcharge on depth and velocity.

6.4.2 Roughness coefficient

Based on the findings of the geology report for the requirement of lining of canals, we need to select relevant roughness coefficient, unless it is obligatory like the case of conveyance and/or crossing structures & steep slopes which require lining unquestionably.

Table 6-4: Manning's coefficient of roughness

Type of lining	Condition	n
Glazed coating of enamel Timber	In perfect Order a) Planed boards carefully laid b) Planed boards inferior workmanship or aged c) Un-planed boards carefully laid d) Un-planed boards inferior workmanship or aged	0.010 0.014 0.016 0.016 0.018
Masonry	a) Net cement plaster b) Sand & cement plaster c) Concrete, Steel troweled d) Concrete, wood troweled e) Brick in good condition f) Brick in rough condition g) Masonry in bad condition	0.013 0.015 0.014 0.015 0.015 0.017 0.020
Stone Work	a) Smooth dressed ashlar b) Rubble set in cement c) Fine, well packet gravel	0.015 0.017 0.020
Earth	a) Regular surface in good condition b) In ordinary condition c) With stones & weeds d) In poor condition e) Partially obstructed with debris or weeds	0.020 0.025 0.030 0.035 0.050
Steel	a) Welded b) Riveted c) Slightly tuberculated	0.013 0.017 0.020
Cast Iron		0.013
Asbestos Cement		0.012
Plastic (Smooth)		0.011

Source: INCID, Pipe distribution system for irrigation, 1998

Roughness coefficient of 0.014 for reinforced concrete lining as in case of chute structure, 0.018 for masonry lining and 0.025 for earthen canal sections are practically adopted in designing of canal sections.

6.4.3 Recommended canal side and longitudinal slopes and velocity

For clay soil texture, main canal side slope is proposed to be 1V:1.5H and for other soil types it is kept 1V:1H. Likewise, its longitudinal slope needs to be designed for 0.1%. For the same soil texture, non-scouring and non-silting velocity needs to be kept to ±0.75 m/sec.

Unlined canals are to be designed so that the velocity is low thus the bed and sides are not eroded by the water. For this reason, unlined canals tend to be wide and shallow, spreading the flow over a large area to reduce the erosive influences of irrigation water.

Lined canals are expensive to construct. For this reason they tend to be narrow and deep which ensures the minimum area of lining for a given canal carrying capacity. The velocity also tends to be high, but this is not usually a problem as the canal is protected from erosion by the lining (up to 1.5m/s is allowable).

Table 6-5: Maximum allowable Flow Velocities in Earth Canals

Soil type	Maximum Flow Velocity (m/s)
Sand	0.3-0.7
Sandy loam	0.5-0.7
Clayish loam	0.6-0.9
Clay	0.9-1.5
Gravel	0.9-1.5
Rock	1.2-1.8

Source: I&D System Design Training Material, GIRDC, 2015

Lined canals can manage a range of velocities, as erosion is not an issue. However, for easy management of water, this permissible velocity should be critical or subcritical.

Fortier and Scobey have also recommended the following maximum permissible velocity in earth canals.

Table 6-6: Maximum permissible velocity in earth canals, by fortier and scobey

Material	Clear water		Water with colloidal silt	
	f/s	m/s	f/s	m/s
Fine sand, colloidal	1.5	0.46	2.5	0.76
Sandy loam, non-colloidal	1.75	0.53	2.5	0.76
Silt loam, non-colloidal	2	0.61	3	0.91
Alluvial silt, non-colloidal	2	0.61	3.5	1.07
Firm loam soil	2.5	0.76	3.5	1.07
Volcanic ash	2.5	0.76	3.5	1.07
Stiff clay, highly colloidal	3.75	1.14	5	1.52
Alluvial silt, colloidal	3.75	1.14	5	1.52
Shales and hard "pans"	6	1.83	6	1.83
Fine gravel	2.5	0.76	5	1.52
Coarse gravel	4	1.22	6	1.83
Cobble and shingle	5	1.52	5.5	1.68

Source: I&D System Design Training Material, GIRDC, 2015

USBR has also recommended the following Permissible Velocities for Non-cohesive Soils.

Table 6-7: Permissible velocities for non-cohesive soils, USBR

Material	Particle diameter	Mean velocity	
	mm	f/s	m/s
Silt	0.005-0.05	0.49	0.15
Fine sand	0.05-0.25	0.66	0.20
Medium sand	0.25	0.98	0.30
Coarse sand	1.00-2.50	1.8	0.55
Fine gravel	2.50-5.00	2.13	0.65
Medium gravel	2.50-5.00	2.62	0.80
Coarse gravel	10.00-15.00	3.28	1.00
Fine pebbles	5.00-20.00	3.94	1.20
Medium pebbles	25.00	4.59	1.40
Coarse pebbles	40.00-75.00	5.91	1.80
Large pebbles	75.00-200.00	7.87-12.8	2.40

Source: I&D System Design Training Material, GIRDC, 2015

6.4.4 Canal side slope and b/d ratio

On top of the above recommendation slope of a canal is normally decided at an early stage of design depending on discharge capacity and topographic conditions of that canal. It can be made as flat as possible to keep command size, or steeper to reduce canal size or increase velocity.

Table 6-8: Indicative canal side slope and corresponding b/d ratio

Discharge (m3/s)	Canal Side Slope (1V:mH)	b/d ratio
0.03 - 0.15	0.5	1.3 – 2.2
0.15 – 0.3	1.0	0.8 – 1.2
0.3 – 1.0	1.0	1.2 – 2.5
1.0 – 5.0	1.5	1.6 – 3.0

Source: Halcrow-GIRDC Study Report for 80,000ha Net Ethiopian Nile Irrigation Project, 2010

The recommended bed width to water depth (b/d) ratios for earthen trapezoidal canals are also given in Table 5-7 as related to flow depth.

Table 6-9: Recommended b/d ratios for earthen trapezoidal canals

Water Depth	(b/d) Ratios	Remark
Small (d < 0.75 m)	1 (clay) - 2 (sand)	Topography also matters
Medium (d = 0.75-1.50 m)	2 (clay) - 3 (sand)	
Large (d > 1.50 m)	> 3	

Source: FAO Irrigation & Drainage Manual, 2002

6.4.5 Canal geometry

Canal cross-section: A canal cross-section can be any shape. But it is sensible to choose a profile that is easy to construct and does the job of carrying water for the least cost and with the best practical hydraulic efficiency. This limits the choice of canal section to a few standard sections. For this purpose, a trapezoidal canal section shall be adopted as it is the commonest section and is still universally used though a parabolic section is the best hydraulic section but difficult in practice to construct curves than straight lines. In case of canals running on cliffs/hills, rectangular section (typical of which is shown in Figure 5-4) shall be used so as to avoid extended embankment width and reduce land slide. For designing cross-sections of canals, the most commonly used formula of Manning's and others shall be adopted.

Table 6-10: Indicative canal dimensions (cm) as related to its capacity

Canal Capacity, l/s	Trapezoidal Canals				Rectangular Canals	
	Unlined Canals		Lined Canals		Only Lined Canals	
	b	h	b	h	b	h
25	20 - 25	15 - 25	15 - 20	20 - 25	20 - 25	25 - 30
50	20 - 30	20 - 30	25 - 30	20 - 25	30 - 35	30 - 35
75	25 - 35	25 - 35	25 - 35	25 - 30	35 - 45	35 - 40
100	30 - 35	25 - 40	30 - 35	30 - 35	40 - 45	35 - 45
125	30 - 40	30 - 45	30 - 35	30 - 40	45 - 50	40 - 50
150	30 - 45	30 - 45	35 - 40	35 - 40	45 - 50	45 - 55
175	35 - 45	35 - 50	35 - 40	35 - 45	50 - 55	45 - 60
200	35 - 50	35 - 55	40 - 45	35 - 45	50 - 60	50 - 60

Source: Source: FAO Irrigation & Drainage Manual, 2002

Note: From our country's practical point of view, a minimum bed width of 20cm for lined and 30cm for earthen canals shall be used.

Free board: Free board is a safety reservation which is designed to accommodate waves, a flood surcharge or a surge flow caused by faulty operation of the canal. It can be regarded as an ignorance factor to allow for inaccurate estimates in roughness or slope, or to accommodate the effects of poor construction tolerances on the same parameters. Table below gives indicative guidelines for bank and lining free board which have been found workable in practice.

Table 6-11: Indicative guidelines for bank and lining free board

Capacity, (m3/s)	Free board (mm)		
	Lining	bank	Total
0 - 0.5	50	150	200
0.5 - 1	100	300	400
1 - 10	200	500	700

Source: Halcrow-GIRDC Study Report for 80,000ha Net Ethiopian Nile Irrigation Project, 2010

Guidelines for Irrigation Systems Design in Hills and Valleys, MoWR, Nepal, 2006 has given the following empirical formula to estimate free board of canals.

$F_b = 0.20 + 0.253Q^{1/3}$ …………………………………………………………….………………. (6-2)

Where, F_b = Free board (m), and Q = design discharge (m^3/s)

Table 6-12: Free board in lined and earthen canals

Capacity,(m3/s)	Lined canal (m)	Earthen Canal (m)
< 0.1	0.10	0.30
0.1 – 0.5	0.15	0.30
0.5 – 1.0	0.20	0.40
1.0 - 2.0	0.20	0.50
2.0 - 3.0	0.25	0.50
3.0 - 5.0	0.30	0.60

Source: Guidelines for Irrigation Systems Design in Hills and Valleys, MoWR, Nepal, 2006

6.4.6 Canal bends

When a canal changes direction, it is to be diverted in semicircular direction so as to maintain flow condition. Sharp bends in a canal can cause head loss or erosion because centripetal force creates a dominant secondary current which tends to erode the outer bank and leads to head loss. So with a subcritical lined canal the aim in designing a bend is merely to avoid creation of a disturbance which may be transmitted downstream as surface waves, any head loss is likely to be negligible in practical terms. With an unlined canal the aim is to avoid erosion damage, but if the route dictates a tighter bend then the outside of the bend can be lined at minimal cost. For small canals the following guide can be used for the minimum radius of curvature measured to the canal centerline, with the radius expressed in terms of the canal water surface top width, T:

Table 6-13: Canal bends centerline radius

SN	Canal Capacity, (m3/s)	Bend Centerline Radius	
		Concrete Lined	Unlined
1	0 - 1	3T	5T
2	1 - 10	5T	7T

Source: Halcrow-GIRDC Study Report for 80,000ha Net Ethiopian Nile Irrigation Project, 2010
Note: T-is width at the water surface

6.4.7 Canal banks

Canal banks are used to hold water within the water section of a channel. Suitable bank dimensions of an earth canal depend on size of canals, height of water surface above natural ground, amount and nature of excavated earth available for bank construction, and need of inspection roads along the canal. Bank widths at all elevations must provide stability against water pressure at the sides of the canal section. They should also keep percolating water below ground level outside the banks and prevent piping of bank materials.

Thus, bank widths of SSI Projects are often determined by local design rules. In areas of restricted land take, it may be preferable to adopt a parabolic or rectangular flume with almost zero bank width. In other cases the bank may perform one or more of several functions: supports the canal lining, restricts seepage, serves as an access road and contains temporary flood surcharges.

A reasonable minimum bank top width that can be handled for canals of about 1 m^3/s capacity is 1m such that a small vibrating roller is used for compaction. However, this can be reduced if the canal has a self-supporting lining (such as precast parabolic segments) and soil compaction is not critical.

Table 6-14: Recommended canal bank top width

SN	Discharge (m3/s)	Top width of bank (m)
1	< 0.10	0.5-0.6
2	0.10 to 0.28	0.92
3	0.28 to 1.4	1.22
4	1.4 to 4.2	1.50

Source: Irrigation and Water Resources Engineering, G.L. Asawa, 2005

6.4.8 Canal curves

As far as possible, curves should be avoided in the alignment of canals as it leads to disturbance of flow and a tendency to silt on the inner bend and scour the toe of the outer (concave) bend. Thus if curves have to be provided, they should be as gentle as possible to avoid scour damage. Lacey suggested the following formula for the curve radius, R, measured from the centerline of the canal.

$$R = 128 * Q^{0.5} \text{ [m]} \quad \ldots\ldots (6\text{-}3)$$

Where, Q is canal design discharge in m^3/s; ranges of values as presented in table below.

Table 6-15: Minimum canal radii

Discharge (m^3/s)	Minimum Radii to the Channel Centerline (m)
< 0.5	100
0.5-3.0	150
3.0-15	300
15-30	600
30-80	1,000
> 80	1,500

For a width factor, e, of 1.0, this formula can be reduced to:

$$R = 26\, W_s \text{ [m]} \quad \ldots\ldots (6\text{-}4)$$

Where, W_s is the water surface width of the channel [m].

The use of this formula leads to large radii not always desirable where space is limited. Also, for colloidal soils with some cohesion sharper bends are possible without any danger of scour.
Other relationships commonly used are:

$$R = 8 \text{ to } 10 * W_s \quad \ldots\ldots (6\text{-}5)$$

$$R = 15 * b \quad \ldots\ldots (6\text{-}6)$$

Where, b is canal bed width (m)

These formulae may be used where the soils in which the channel is to be constructed have some cohesion, such as for most rice growing areas in Indonesia.

For the case of lined canals; the minimum radius, R, as recommended by USBR is:

$$R = 3 * W_s \quad \ldots\ldots (6\text{-}7)$$

It should be noted that the minimum practical radius would be much larger if slip forming machines are being used. If an unlined canal is lined at bends to allow sharper curves, then the lining should extend at least 4times the depth of flow downstream of the canal curve.

6.4.9 Canal Berms

Berms are usually provided to allow for lateral channel instability of medium and large unlined canals which may either be eroded or further deposited over time, reducing risk of embankment breaching. They shall be provided to unlined canals where depth of flow, is greater than about 0.8m. As breaches in fill sections are more likely than in cut sections, more damaging and difficult to plug, berm widths are greater for canals in fill than in cut. For SSI projects this will not be considered unless compulsory for example, the case if deep cut across hills encounter.

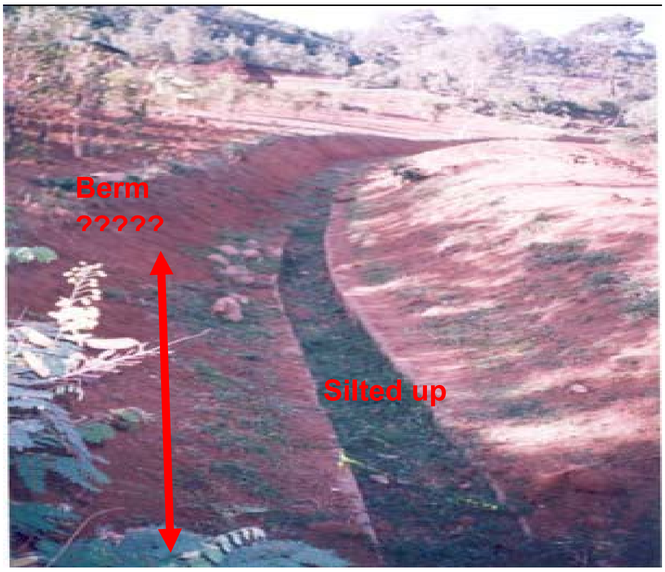


Figure 6-3: Silted-up main canal of satame irrigation project (due to lack of berm)

Proposed berm widths related to depth of flow (d) in alluvial soils are:

- 1.5d or 2d where the OGL is above the full supply level (cut section).
- 2d when OGL is below the full supply level but above the bed level (cut & fill section).
- 3d where the OGL is below the full supply level as well as the bed level (fill section).

In lined canals, berms are clearly not required to accommodate change in canal prism or additional discharge. They may be provided, in exceptional cases, to facilitate access to inspect and maintain the lining and if there is deep cut. Berms should be provided in hill drains running along MC, which cut across ground slopes steeper than 1:3 and excavations deeper than 1.5 m. The width of the berm is commonly 0.5 to 1.0 m and side slope is 1: to 1:1.5 depending on soil type.

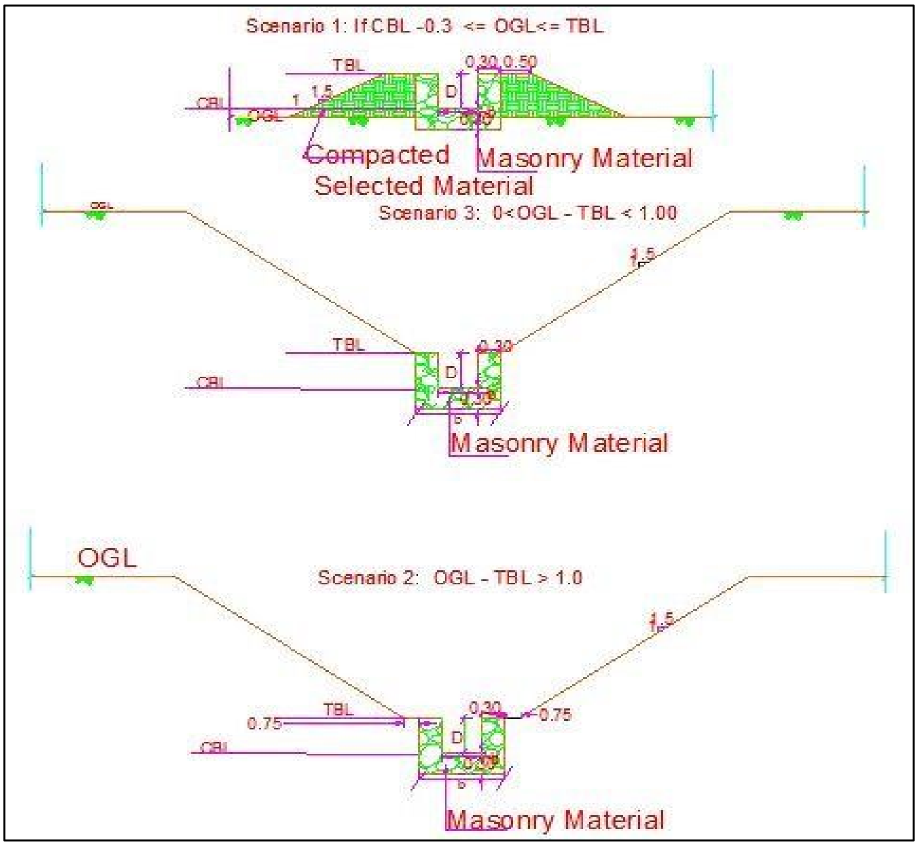


Figure 6-4: Typical canal berm provision under different scenarios

6.4.10 Canal transitions

Canal transitions are structures which are part of canal sections and designed to join two different canal cross sections and/or fit structures' inlets and outlets to incoming and outgoing canals respectively. They are commonly constructed from dry or wet riprap/stone pitching. They are designed to allow smooth flow from one section to the other and to prevent erosion effects on structures. Separation of flow is also avoided by use of suitable transition walls. Transitions should be properly designed so as to avoid accumulation of silt jetty as well as sudden transition resulting in erosion and formation of eddies.

For example, earthen canal section is normally of trapezoidal and the flumed canal section is rectangular thus, they need to be connected by transitions which are not steeper than 2:1 on the upstream and 3:1 on the downstream. It is also not necessary to keep the same depth in the normal and flumed sections. Rather, it may sometimes be economical to increase the depth and still further reduce the canal width in cases where it encounters a reach of rocky terrain and has to be flumed to curtail rock excavation. If the parent canal is lined rectangular section then transition may not be required but fluming may be required based on hydraulic requirements.

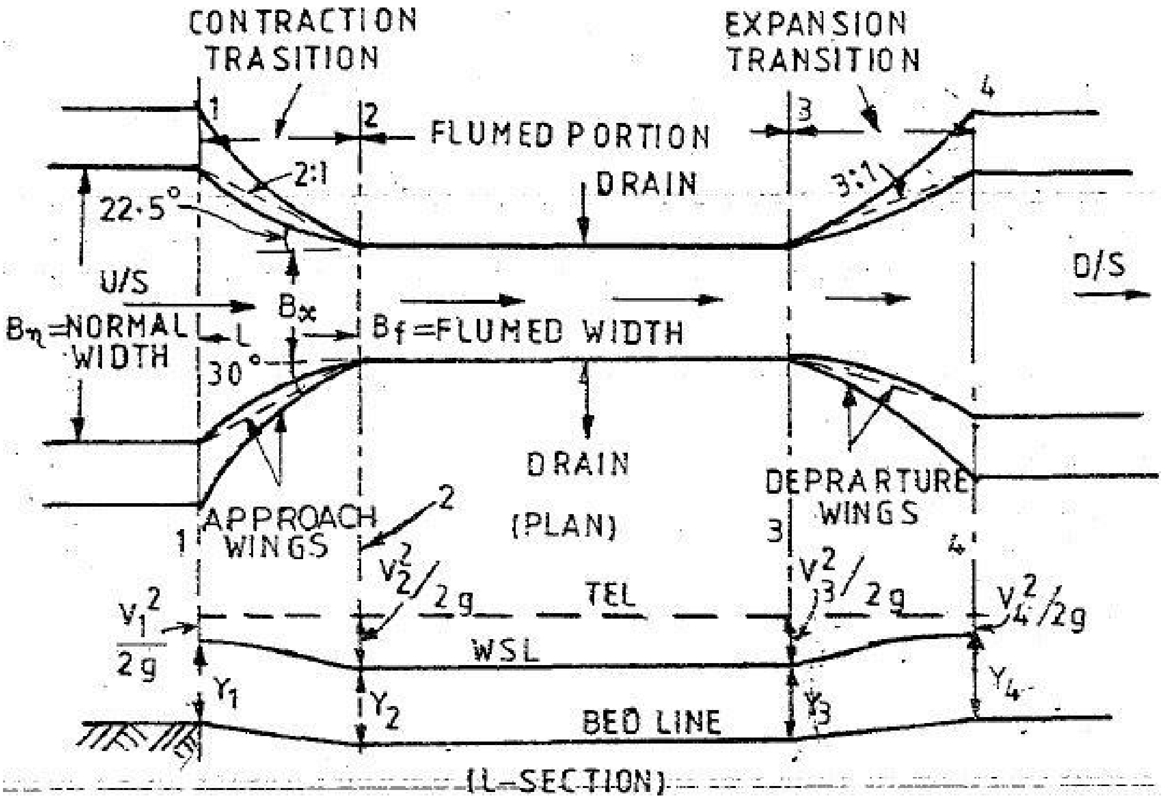


Figure 6-5: Typical canal transition for canal-flume-canal combinations

In general, wherever a structure like bench flume is used with a canal, transitioning from the canal section to the flume section is usually required to provide a relatively smooth water surface and to conserve energy. This is usually also true with transitioning from flume section to canal section, especially if the canal section is unlined.

6.4.11 Best hydraulic section

It is well known that, conveyance of a channel section increases with increase in the hydraulic radius or with decrease in the wetted perimeter. Therefore, from the point of view of hydraulic aspects, the canal section having the least wetted perimeter for a given area has the maximum conveyance; such a section is known as hydraulically efficient canal or best hydraulic section.

Thus, for any shape of canal section, there is a 'best' hydraulic section, meaning the theoretical most economical combination of depth and width for a given discharge. It has the shortest wetted perimeter, the least cross-sectional area, the least frictional resistance to flow and the smallest surface area of lining material, thus the most economical section.

Best hydraulic section is the section which carry maximum discharge for a given excavation, i.e. a section with maximum hydraulic radius, R or minimum wetted perimeter, P because, Q is proportional to section factor, $AR^{2/3}$ for a given canal constants (i.e., n and S are specified) and R = A/P. It is a section that gives maximum $AR^{2/3}$ for a specified flow area, A.

The section of minimum excavation is possible only if the water surface is at the level of the top of the bank. When the water surface is below the bank top of the bank (which is very common in practice), channels smaller than those of the best hydraulic section will give minimum excavation. If the water surface overtops the banks and these are even with the ground level, wider channels will provide minimum excavation. Generally, hydraulically efficient channel is adopted for lined canals. It may also be noted that hydraulically efficient channel need not be economical channel (least cost). Mathematically,

$\partial P/\partial d = 0$ …………………………………………………………………………………..…… (6-8)

Where, P is wetted perimeter, (m)
d is flow depth or water depth (m)

For any shape of canal section, there is a 'best' hydraulic section; meaning the theoretical most economical combination of depth and width for a given discharge at some combination of b & d. It has the shortest wetted perimeter, the least cross-sectional area, the least frictional resistance to flow and the smallest surface area of lining material, thus the most economical section. The ideal best hydraulic section is a semi-circle, and for any other profile the best section is the one which most closely approximates to a semi-circle.

However, this is never the most appropriate section in practice due to its difficulties in construction. Theoretically, the best trapezoidal section will have a side slope of 60 degrees, which is too steep for easy construction of earth or in situ concrete. Similarly in large parabolic canals the theoretical best section has sides that are too steep to be easily made in in situ concrete. Steep sides are a safety hazard for both people and animals that may accidentally fall into the canal.

The need for Free board and the possibility of a fluctuating discharge, both above and below the design level, will further modify the optimum economic section away from the best hydraulic section. In practice, the cost implications of deviating even quite substantially from the best hydraulic section are often not great, and should always be over-ridden by considerations of safety, structural strength and ease or practicality of construction.

Table 6-16: Geometric elements of best hydraulically efficient section

Cross Section	Area, A	Perimeter, P	Hydraulic rad., R	Top width, T	Flow depth, d	Z=A√d
Rectangular	$2d^2$	4d	0.5d	2d	d	$2d^{2.5}$
Trapezoidal	$\sqrt{3}d^2(1.732d^2)$	$2\sqrt{3}d(3.464)$	0.5d	$(4\sqrt{3}/3)d(2.3094d)$	3/4d(0.75d)	$3/2d^{2.5}(1.5y^{2.5})$
Semi Circular	$\Pi/2\ d^2$	Πd	0.5d	2d	$\Pi/4$ d	$\Pi/4\ d^{2.5}0.25\Pi d^{2.5}$

In general, a channel section should be designed for the best hydraulic efficiency however it should be modified such that it is practicable.

Box 6-1:
Worked Example-4*: Consider the following two schematic diagram of rectangular canal cross section having same sectional area. Which one do you think is the best hydraulic* section?

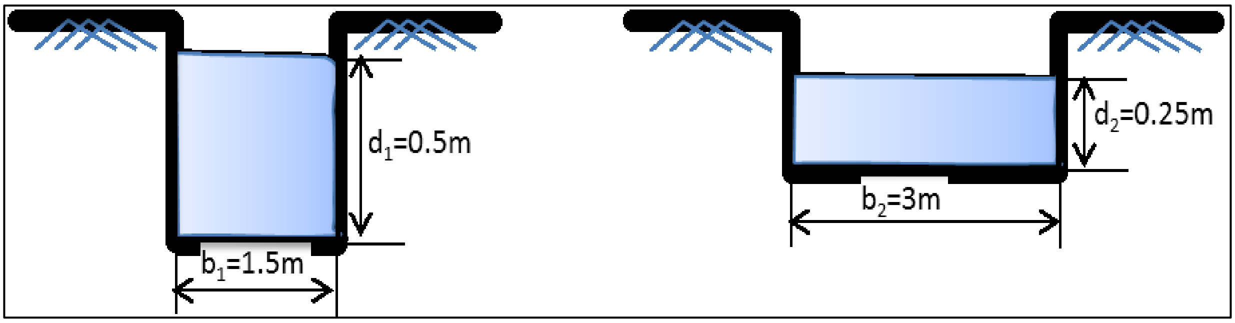


Figure 6-6: Exercise on best hydraulic section of rectangular canal

***Solution**: Both of these cross sections are analyzed as follow:*

$A_1 = b_1*d_1 = 1.5*0.5 = 0.75m^2$ $A_2 = b_2*d_2 = 3*0.25 = 0.75m^2$

$P_1 = b_1+2d_1 = 1.5+2*0.5 = 2.5m$ $P_2 = b_2+2d_2 = 3+2*0.25 = 3.5m$

$R_1=A_1/P_1 = 0.30$m and $R_2=A_2/P_2 = 0.21$m

This indicates, the best efficient geometry of the canal is when the wetted perimeter is minimum or the hydraulic radius is higher. If we maintain same roughness coefficient in both cases, discharge will be greater for the first case than the second section. Thus, we can say the first section is of better hydraulic section than the second one.

$A = b*d \Rightarrow b=A/d$ …………………………………………………………………………………… (6-9)

$P = b+2d \Rightarrow P=A/d + 2d$ …………………………………………………………………………… (6-10)

Now from equations 29 & 30, $\partial P/\partial d \Rightarrow -A/d^2 +2 = 0$ or $A=2d^2$ ……………………………………… (6-11)

Therefore, from $A=bd=2d^2$ or $b=2d$……………………………………………………………… (6-12)

In general, the best hydraulic section of a rectangular canal is when it is made half a square. Similarly, the best hydraulic section of a trapezoidal canal is when it is made half a regular hexagon with side slope 60 degrees, m=0.58, where m is the horizontal ratio of the slope 1V:mH, which gives R=d/2. However, for all but the smallest canals, this is too steep slope to construct insitu. Thus, a practical maximum recommended side slope is 1:1 except for small precast segments.

6.4.12 Canal lining

Seepage always occurs, even if canals are constructed with clay soils. If there is abundant water available that can be diverted under gravity, one might accept the water losses without resorting to lining. In fact, worldwide, unlined canals are the most common as they are the cheapest and easiest type of canal to construct. However, if water has to be used more efficiently, due to its scarcity or if it has to be pumped, it usually becomes economical to line the canals. Another consideration in analyzing the economics is the health-related cost (i.e. cost of medicines and time lost by smallholders due to poor health).

Generally, canal lining need to be done in order to reduce seepage losses and thus increase irrigation efficiencies. It also substantially reduces drainage problems and canal maintenance as well as water ponding, thus reducing the occurrence of vector-borne diseases. Also, smooth surface linings reduce frictional losses, thereby increasing the carrying capacity of canals.

Thus, lining canals has numerous advantages though it incurs more cost to a project. Canal lining is required for preventing seepage out and seepage in, fast response time, reduce land slide on canals running on cliffs/hills, reduce pumping costs (i.e. loss will be minimized), reduce land take, maintain integrity of cross section, prevent animal damage, control encroachment, reduce health risks, ease maintenance, ease management, limit siltation, prevent erosion, limit damage of canals by beneficiaries, structures simplified, reduce decrease in discharge and improve bank stability.

Lining materials such as concrete, masonry, blocks, slabs or bricks of stone and clay can be used wherever lining is found necessary. But, masonry lining which is the commonest form of construction in many countries especially for small canals is recommended in this Guideline. However, it needs great attention as it is extremely difficult to make it watertight or crack free especially on expansive soils. It always leaks and easily disrupted by swelling soils, roots, wild animals activity and differential thermal expansion. In such hydraulic structures, it should be used with removal of expansive soils underneath and replacement of selected materials all-round. A minimum of 0.5m working space is required for rectangular canal.

Note: Canals running on cliffs/hills should be of rectangular in section as trapezoidal may result in broadening of side slopes for stabilization purpose and even can go for the requirement of retaining walls.

Table 6-17: Indicative concrete lining thickness for ranges of discharges

SN	Indicative Discharge (m3/s)	Lining Thickness (mm)		Likely Cost effective b/d Ratio	
		Type A	Type B	Type A	Type B & C
1	0.015 - 0.03	-	-	0.5 - 2.0	2.0 - 3.5
2	0.03 - 0.15	60	100		
3	0.10 - 1.0	75	100		
4	1.0 - 3.0	75	120		
5	1.5 - 5.0	75 - 100	150		
6	5.0 - 15.0	75 - 100	150		
7	15.0 - 30.0	100	200		
8	>30.0	100	200		

Source: Planning & Design Criteria by Halcrow-GIRDC, March 2010

Typical Canal lining thicknesses for different canal sections are shown below.

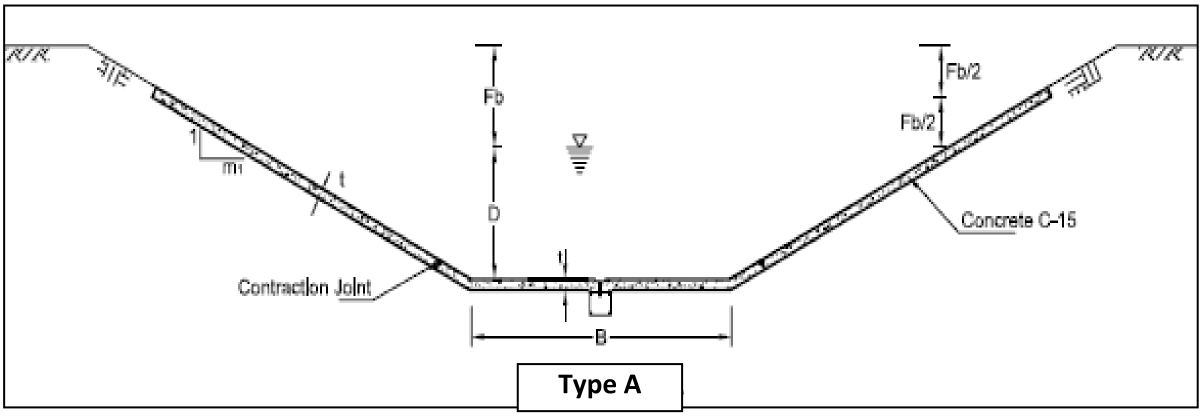


Figure 6-7: Concrete lining thickness for typical trapezoidal canal section (Type-A)

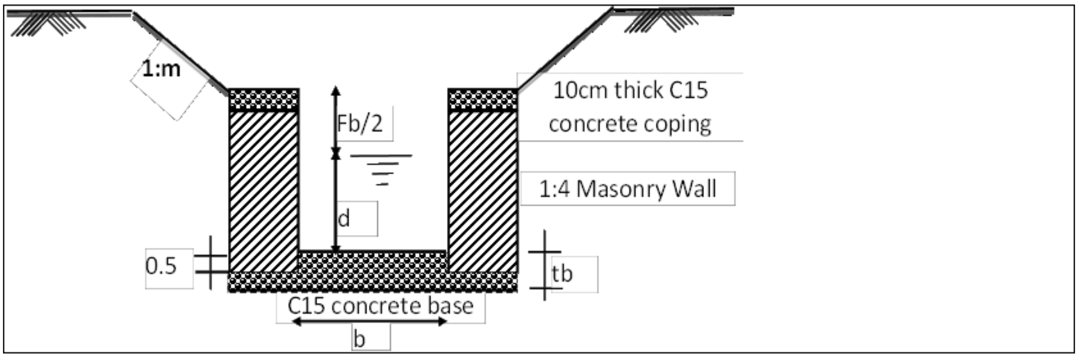


Figure 6-8: Lining thickness for rectangular canal section (Type-B, on SC & Others)

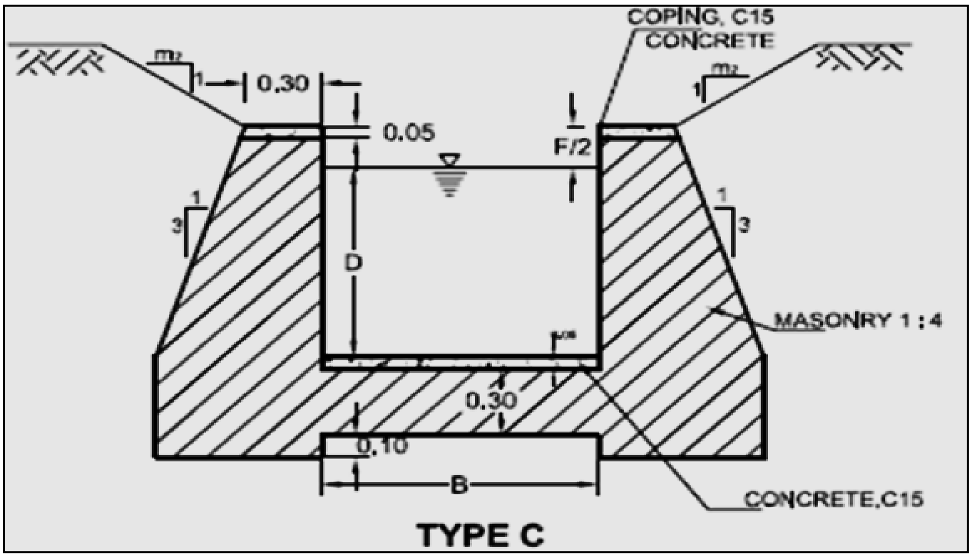


Figure 6-9: Masonry lining thickness for rectangular canal section (Type-C, on MC)

6.4.13 Water level

The objective of setting canal water levels is to ensure that water can be delivered to the fields at the required rate and to the required depth. Establishing canal levels should thus start at the field channel level in the tertiary units and at the head (intake). It is usually adequate to use the tertiary unit as the starting point for level setting.

- Water level in field canals shall be a minimum of 15cm above maximum ground level in crop field to be commanded. This allows for a 10cm depth of water in this field;
- A minimum head loss for all check/regulator structures of 10cm;
- A minimum head loss for all culverts not more than about 8.0m long of 10cm;
- Thus canal design shall be started from bottom-up i.e. from downstream or command to upstream or headwork side.

6.5 CANAL DESIGN

6.5.1 Canal design methods

Unlined canals are classified into two classes based on the stability of the boundaries of the canal for design purposes: Canals with stable (non-erodible) bed and Canals with erodible bed (Alluvial) with significant amount of sediments flowing in it. Design of these two types of canal requires different considerations and approaches.

Canals carrying sediment are generally designed using Lacey's regime equations. Then canal erosion is checked using Tractive Force theory and bed material carrying capacity using Colby, Engelund, Hansen or Ackers and White methods. On the other hand, canals carrying clear water or hard surface lined canals are normally designed by Manning's equation which is used for uniform flow condition in open canals.

In most of small scale irrigation projects, irrigation is carried out in the dry season when flow in the source of irrigation water supply river is clear thus Manning's equation need to be used. Nevertheless, for spate irrigation system where conveyance canal is expected to carry huge amount of sediment, other methods should be adopted. If the canal bottom or sides are erodible, then the design requires that the canal size and bottom slope are selected so that canal is not eroded. Two methods have been used for the design of such canals: the permissible velocity method and the tractive force method.

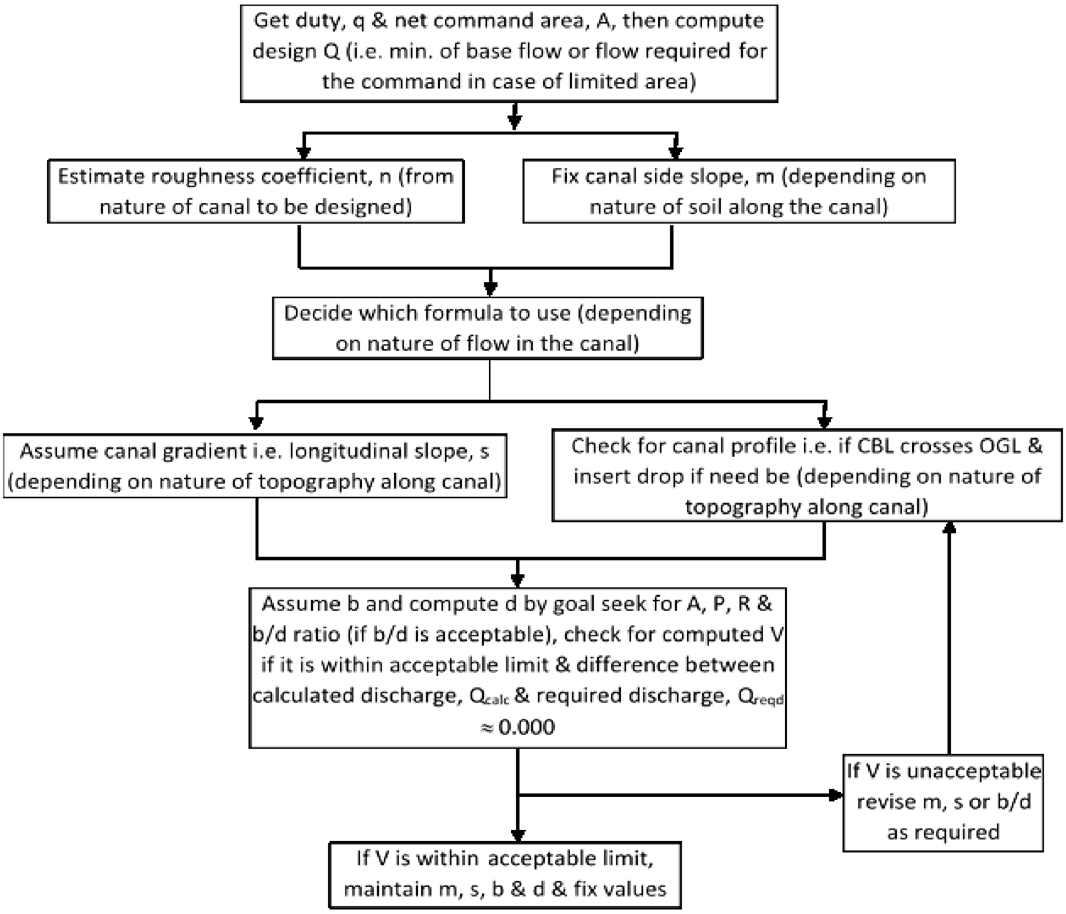


Figure 6-10: Flowchart for canal design procedures using manning's equation

Note: As it is indicated in the first box of this chart, design of main canal can be approached in two ways:

i. When there is sufficient base flow in the river: In this case there is no worry of supply, thus governing condition is our demand i.e. peak duty, q or availability of suitable irrigable area. Consequently, we should simply multiply peak duty with net area and then fix canal capacity based on this value and other hydraulic design parameters of canal including longitudinal slope;
ii. When there is limited base flow in the river: Under such circumstances, there is inadequate flow in the river thus we need to worry of supply even if we have plenty of suitable irrigable area. Thus, governing condition here is flow in the river thus our command area should be fixed based on this flow plus night storage flow if irrigation duration is less than 24hours per day and beneficiaries' are interested.

This chart also indicates that we need to fix both cross section of canal and its profile simultaneously, since flow velocity and longitudinal profile have direct relationship. In general, design of canals involve the selection of canal alignment, shape, size, and bottom slope and whether the canal should be lined to reduce seepage and/or to prevent erosion of canal sides and its bottom.

Based on the available base flow, the amount of usable water is calculated as:

$W_u = Q_b \times K - Q_{dw}$ …………………….…………………………….…………………..…… (6-13)

Where, W_u is the amount of usable water (m3/s),

Q_b is base flow (m3/s),

K is a coefficient of released flow for downstream ecology (0.7 to 0.9),

Q_{dw} is the existing design demand in the downstream of the project (downstream water demanded).

6.5.2 Manning's equation

This method is used for lined or other non-erodible canals, or canals conveying clear water. Generally it should not be used for earth canals conveying river water with a sediment load. The equation is:

$$Q = {}^{1}/_{n} * A * R^{2/3} * S^{1/2}$$ …………………….………………………………… (6-14)

Where, A is cross sectional area (m2)

R is hydraulic radius (= A/P, m)

P is wetted perimeter of cross section (m)

S = Longitudinal /friction slope of canal, S=∆H /L, (%)

∆H = Head difference between reference points, (m) and

L = Distance between reference points (m)

n = Roughness coefficient, with the following commonly used values:

- Concrete lined canals 0.012 – 0.015
- Masonry lined canals 0.018 – 0.022
- Earthen canals 0.025 – 0.030
- Natural rivers/drains 0.030 – 0.040

Box 6-2:

Worked Example-5: Consider the longitudinal profile data of LSC2 from Melka Lola SSI Project (Refer Appendix-I). As per geology report of this project, this canal needs to be lined. There are four tertiary units feed by this secondary canal. Traditional irrigation experience of the beneficiaries has shown that irrigation duration is 12 hours per day. This canal is supposed to have two reaches (based on its layout design): its downstream reach irrigates 16ha and its upper reach supplies irrigation water to 30ha including the downstream. If the 24 hour peak duty of the project site is computed out to be 0.67l/s/ha and supply system is rotational at tertiary unit level, then design appropriate cross section and profile of this canal.

Solution: *Based on the design procedures presented in the above flowchart, canal section has been computed such that flow velocity is within non-silting and non-scouring range for lined canal or Fraud Number, Fr is less than or equal to unity, bed width is about double of flow depth, and CBL is within the ground by adding drop structure as shown in following tables. Corresponding longitudinal profile of this canal has also been designed and presented in Appendix-II.*

Table 6-18: Designed hydraulic parameters of LSC_2 (Melka Lola SSIP)

Chainage (m)	A_{net} (ha)	12hr Duty (l/s/ha)	Q (m3/s)	n	s	b	m	d	b/d	AX (m2)	P (m)	R (m)	V (m/s)	Fr=V/ √(gd)	Q_{cal} (m3/s)	$Q_{cal} - Q_{reqd}$	F_b (m)	D (m)	T (m)	Remark
0+000 to 0+110	30.0	1.34	0.04	0.018	0.0040	0.35	0	0.18	1.94	0.06	0.71	0.09	0.70	0.53	0.044	0.004	0.20	0.38	0.35	Masonry Lined Rectangular
0+110 to 0+466.7	16.0	1.34	0.021	0.018	0.0045	0.25	0	0.15	1.67	0.04	0.55	0.07	0.62	0.51	0.023	0.002	0.20	0.35	0.25	

Note: In designing canal section using this table we should recognize that: (i) This canal section is lined i.e. n=0.0018; (ii) This canal has two reaches/sections: 0+000 to 0+110 and 0+110 to 0+466.7; (iii) b/d ratio is within acceptable ranges; (iv) Flow is subcritical, i.e. Fr<1; (vi) Flow velocity is within acceptable ranges (v) carrying capacity of designed canal (Q_{calc}) is a bit higher than the required (Q_{reqd}) and at the same time economical as the difference is not significant; (vii) This design is done by assuming bed slope, s and flow depth, d and then checking computed b/d, V, Fr, and $Q_{calc} - Q_{reqd}$ (For detailed procedure, refer Flowchart shown in figure 5-8). Thus, we should consider these parameters while fixing/designing size of canals. In-fact we can also design using any one of the approaches mentioned under 5.4.1.

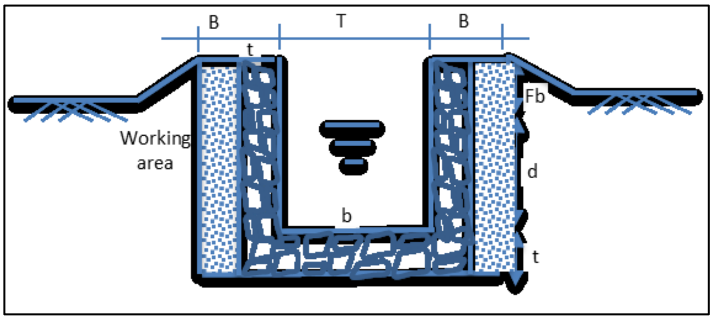


Figure 6-11: Schematic diagram designed for cross section of LSC_2

6.5.3 Lacey's regime equation

In this method the principle is 'canal is considered to be in regime if over a hydrological cycle, neither net erosion nor deposition of materials occurs'. Canals are designed to have a trapezoidal cross section with a bed width to water surface width ratio of 0.8, although in practice side slopes of 1:2 are normally specified so that the actual water surface width is greater than the design value. This allows the rapid formation of natural regime shape through erosion and deposition.

The water surface width equation is: $Ws = 4.83 * e * Q^{1/2}$ ………………………….………. (6-15)

Where, Ws is the design water surface width (m)
e is the width factor which varies between 0.7 and 1.10, and
Q is the dominant discharge, (m^3/s)

The choice of an 'e' value depends up on the nature of the soil through which the canal is excavated, the nature of the sediment carried and the need to restrict full width development where land is available. A low 'e' value is adopted if the soils and sediment are tenacious i.e. have a clay fraction. When they are non-cohesive and fine a high 'e' value is recommended. Two values commonly used are 0.83 and 1.0 corresponding to tenacious and friable soils.

The Lacey uniform flow formula may be expressed generally as:

$Q = CR^{(n+1)/2}S^n$ ………………………………………………………….……..…..….. (6-16)

Where, v is the mean velocity
R is the hydraulic radius
S is the slope (Longitudinal), and
C & n are coefficients.

The coefficients C & n vary according to median grain size (mm) of the material. In metric units the formulae are:

$V = 4500RS$, for m < 0.2mm and n =1
$V = 46.4\ R^{3/4}S^{1/2}$, for m 0.2<m< 0.6mm and n =1/2
$V = 10.77R^{2/3}S^{1/3}$, for m 0.6<m <2mm and n =1/3
$V = 5.98R^{5/8}S^{1/4}$, for m > 2mm and n =1/4

Combining equations of WS and Q,

$Q = Ke^2R^{(n+3)}S^{2n}$ ………………………………………………….…….…..…….. (6-17)

Thus, for a given discharge, soil type and sediment size there is a range of combinations of R and S which can satisfy the equation. To define a unique solution, an additional equation is required. The equation which has been adopted is:

$f = 2.46\ V^2/R$ …………………………………………………………………………… (6-18)

Thus Lacey regime equations can be summarized as follows (valid for all values of median grain size, m):

$W_s = 4.83 * e * Q^{1/2}$ ……………………………………………………………… (6-19)

$f = 2.46\ V^2/D_m$ ……………………………………………………………………… (6-20)

Where, D_m is hydraulic mean depth or hydraulic radius = A / P = A / W_s

$D_m = (0.4725Q^{1/3}) / (e^{2/3}f^{1/3})$ ………………………………………………… (6-21)

$S = 0.000206\ (e^{1/3}f^{2/3})(E/Q^{1/6})$ for m < 0.2mm

$S = 0.000274(e^{1/3}f^{2/3})(E/Q^{1/6})$ for 0.2 <m< 0.6mm

$S = 0.000303(e^{1/3}f^{2/3})(E/Q^{1/6})$ for 0.6 < m < 2mm

$S = 0.000188(e^{1/3}f^{2/3})(E/Q^{1/6})$ for m > 2mm

Where, E is shape factor and E = (Wetted perimeter / W_s)
S is water surface slope (m/m), and
f is Lacey silt factor

6.5.4 Permissible velocity method

In this method, the canal size is selected such that the mean flow velocity for the design discharge under uniform flow conditions is less than the permissible flow velocity. The permissible velocity is defined as the mean velocity at or below which the canal bottom and sides are not eroded. This velocity depends primarily upon the type of soil and the size of particles even though it has been recognized that it should depend upon the flow depth as well as whether the canal is straight or not. This is because, for the same value of mean velocity, the flow velocity at the canal bottom is higher for low depths than that at large depth.

Similarly, a curved alignment induces secondary currents. These produce higher flow velocities near the canal sides, which may cause erosion. A trapezoidal canal section is usually used for erodible canals. To design these canals, first an appropriate value for the side slope is selected so that the sides are stable under all conditions.

6.5.5 Tractive force method

The term Tractive Force also referred to as "shear force" or "drag force." It is the pull of the water on the wetted area of a channel. This force is produced when water flows in a channel and acts in the direction of flow. It has been presented in detain under chapter-6.

$\tau_o = \rho gRS$ ……………………………………………………………………………… (6-22)

Where, τ_o is average shear/tractive stress, N/m2 (refer details in chapter-6)
ρ is average density of material, kg/m3
g acceleration due to gravity, m/s2
R mean hydraulic radius, m
S slope of channel, %

$$\tau_c = 0.155 + \frac{0.409\ d^2}{\left(1 + 0.177\ d^2\right)^{1/2}} \qquad \text{(6-23)}$$

Where, τ_c is critical tractive force, N/m2
D is mean participle diameter, mm

6.6 CANAL RELATED STRUCTURES

6.6.1 General

Canal related structures include irrigation structures like division box drop, turnout, culvert, chute, flume, cross drain structure, and the like. They are separately dealt in GL-B18: Canal related structures. Thus, reader is advised to refer this guideline complete consultation.

6.6.2 Differences in selecting pipe culverts and box culverts

Basically, a culvert means a covered hydraulic structure which conveys fluid. Therefore in a broad sense, pipe culverts in a small scale represent normal pipes like precast concrete pipes.
In terms of hydraulic performance, circular section is the best geometrical sections among all. Therefore, for relative small discharge, precast concrete pipes and ductile iron pipes are normally used which are circular in shape. But for applications of very large flow, precast concrete pipes and ductile iron pipes may not be available in current market.

In this regard, cast-in-situ construction has to be employed. It is beyond doubt that the fabrication of formwork for circular shape is difficult when compared with normal box culvert structures. However, circular shape is the most hydraulic efficient structure which means for a given discharge, the area of flow is minimum.

Therefore, it helps to save the cost of extra linings required for the choice of box culverts. However, box culverts do possess some advantages. For example, they can cope with large flow situation where headroom is limited because the height of box culverts can be reduced while the size of pipe culverts is fixed. Secondly, for some difficult site conditions, e.g. excavation of structure in rock, for the same equivalent cross-sectional area, the width of box culverts can be designed to be smaller than that of pipe culverts and this enhances smaller amount of excavation and backfilling.

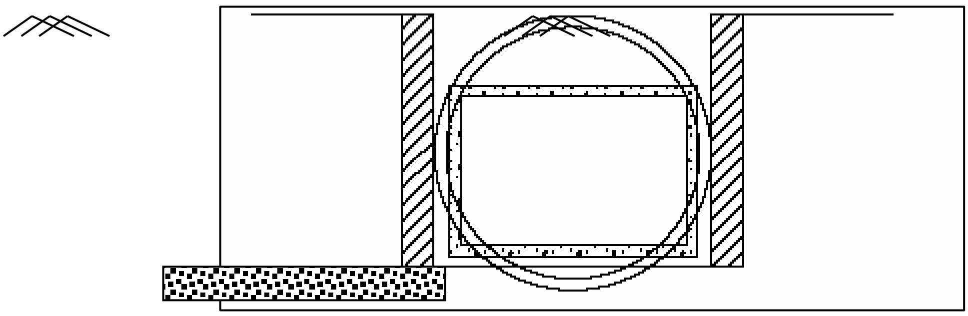


Figure 6-12: Relative arrangements of both culvert types

7 PIPE CONVEYANCE SYSTEM PLANNING AND DESIGN

7.1 GENERAL

There are situations where a buried pipe (or covered canal section) must be considered for designing the main canal such as in the cases of deep cut sections (for example, downstream of the diversion weir or dam outlet) where the maintenance of siltation because of steep side slopes could be a problem. In some cases inspection/maintenance manholes are periodically spaced along the buried pipe/covered canal section at every 20m interval.

7.2 NEED FOR PIPE CONVEYANCE SYSTEMS

Pipelines are slowly gaining acceptance as a viable alternative to open canals as a means of distributing irrigation water as low-pressure pipe line systems can lead to easier distribution and management. Thus, with the increasingly greater demand on a limited water supply, there is an urgent need for its efficient utilization by reducing losses at various points in the irrigation system. Within the farm area, water losses can be greatly reduced by having proper system for conveyance and distribution of irrigation water. A pipeline distribution system offers such a possibility.

7.3 ADVANTAGES OF PIPE CONVEYANCE SYSTEMS

There are many advantages of pipe conveyance system compared with other modes of conveyance. The advantages are:

- Pipelines are often the most economic form of transport (considering either capital costs, running costs or overall costs).
- Pipeline costs are not very susceptible to fluctuations in prices since the major cost is the capital outlay and subsequent operating costs are relatively small.
- Operations are not susceptible to labour disputes as little attendance is required. Many advanced systems operate automatically.
- Being hidden beneath the ground, a pipeline will not mar the natural environment.
- A buried pipeline is reasonably secure against sabotage.
- A pipeline is independent of external influences such as traffic congestion & weather.
- There is normally no problem of returning empty containers to the source.
- It is relatively easy to increase the capacity of a pipeline installing a booster pump.
- A buried pipeline will not disturb surface traffic and services.
- Way for laying the pipelines are usually easier to obtain than roads and railways.
- The accident rate per ton - km. is considerably lower than for other forms of transport.
- A pipeline can cross rugged terrain which could be difficult for vehicles to cross.
- Little loss of farm land, as almost the entire system .is buried - as a result, no significant amount of productive farm area is lost to crop production, as is the case in an open channel network.
- Virtually no water loss - a pipeline system is essentially water tight, with no evaporation and seepage losses during transmission as a result there are water savings and less pumping cost.
- Less seepage losses also reduces drainage problems.
- Labour saving- the control of water is simple and usually requires 25-50% less labor per unit area of irrigation than that required with open channel.
- Permanence - a pipeline properly designed, made of good quality materials, and well-constructed has a long life span.

- Ease of conveyance - one can take the most direct route from water supply to outlet points. This is particularly important in undulating land.
- Water can be transported across a depression or boosted uphill, which is not possible with open channels, unless an elaborate structure is built.
- Low maintenance cost - generally maintenance costs are low, as the system is buried, on the other hand earthen channels have to be maintained continuously.
- No channel block problems - there are no channels to become choked with weeds to hinder flows. Weeds can also harbour harmful insects. In addition, weed seeds which can be transported to fields in open channel are eliminated.
- Better control - better and easier control of the flow of water means that more efficient irrigation is possible.
- No hindrance to equipment - there are few obstacles to hinder the movement of agricultural equipment and farm transport. This is an important feature where fields are small.
- Full and effective control of irrigation water resulting into taking up of crop diversification such as horticulture, vegetables and other cash crops such as groundnut etc.
- No evaporation losses.
- Long durability of system.
- A relatively permanent, trouble free system with long term benefits on investment.
- Possibility of introduction of pressure irrigation methods such as sprinkler and drip. These new concepts have also made the provision of pipe distribution network, a necessity.
- No culverts or other structures such as falls are required. Buried pipelines are taken in straight line so considerable saving in length results in considerable economy.
- Easy to install and lay and therefore construction period is reduced.
- Excavation limited to one meter plus diameter of the pipe, hence less earthwork.
- High-returns because of assured and controlled water supply.
- Crop diversification to derive maximum benefits from irrigated farming practices.
- Scheduling of irrigation based on crop water requirement can be efficiently implemented.
- Farmers at tail end reaches can get enough assured irrigation water.
- As time can be saved the realization of the benefits from the capital investments are materialized quickly and this boosts the economy.

7.4 LIMITATIONS OF PIPE CONVEYANCE SYSTEMS

Of-course there are disadvantages associated with pipeline systems which include:

- Initial capital expenditure is often large, so if there is any uncertainty in the demand, some degree of speculation may be necessary.
- Pipelines cannot be used for more than one material at a time (although there are multiproduct pipelines operating on batch basis).
- There are operating problems associated with the pumping of solids, such as blockages on stoppage.
- It is often difficult to locate leaks or blockages
- Less advantage with large flow, as the cost of pipe lines increases faster with capacity than does the cost of open channels. The net economy of pipeline varies with the value of land, frequency of irrigation and cost of irrigation labor. Thus, it is impossible to set flow limits above which ditches might have some advantages over pipelines.
- Greater initial investment in pipelines, but in the long run pipelines are economical, because of savings in water, labor, maintenance, land and permanence of installation.
- Saline conditions require selection of the proper type of material, as concrete pipes are subject to deterioration.

- Earthquake damage - although the risk is very low, there are a few examples of the underground pipelines getting damaged.
- Pipelines are not desirable for irrigation water transmission if the irrigation supply contains large amounts of sediments and the flow conditions in the line allow the sediments to settle out and reduce the carrying capacity of the line.

7.5 IRRIGATION PIPE LINES

Three types of on-farm irrigation pipelines are generally used.

- Completely portable surface system, where water enters the line at the supply a well, reservoir or open channel turn out and the water is applied to the field from the open end of the pipeline, or from gated outlets distributed along the line.
- A combination of buried and surface pipeline through one or more risers, and
- Completely buried system, generally used for border or basin irrigation, where the need for surface pipe is eliminated. Water is released into the portion of the field to be irrigated form risers on the buried pipeline.

7.6 PIPE CONVEYANCE SYSTEM QUALIFYING CRITERIA

The pipeline distribution system shall be adopted when there is:

- Limited water availability and extensive command.
- The steep topography where canal system is very expensive.
- Heavy and uncontrolled seepage losses.
- Uneven ground and undulated terrain.
- Adoption of modern techniques for future development.
- Farmers' responses and acceptability.
- Need for crop diversification.
- Availability of adequate fund.
- When high returns are expected.
- Timely availability of material, technology, labour, maintenance techniques, construction technology etc.
- The pipeline (gravity flow) is generally feasible in the case of sloping topography having ground slopes steeper than 1 to 500 or 0.2%.

7.7 DESIGN CONSIDERATIONS

7.7.1 Soil data for pipe design

Information regarding existing soil and its characteristic is essential. This information includes types of soils, chemical and physical properties of soil. The depth of pipe and cover over top of pipe are determined by soil load. The cost of pipe line can be influenced by existing high groundwater table. Soil PH can also affect the choice of material of pipe. PH < 5 would preclude use of iron and concrete pipe.

The basic data related to nature and extent of existing soil should be obtained. The information such as Texture of soil, soil PH, existing groundwater table, soil specific characteristics, drain ability, shear strength, compressibility, infiltration rate, soil classification etc. regarding existing strata should be collected.

7.7.2 Pipe materials

Pipe material shall be judiciously selected from the point of view of durability, life and overall cost which includes, besides the pipe cost, the installation and maintenance costs necessary to ensure the required function and performance of the pipeline throughout its designed life time design capacity. All fittings such as couplings, reducers, bend tees and crossings should be made of material that is recommended for use with the pipe and should be installed in accordance with the recommendations of the manufacturer. Where fittings made of steel or other metals subject to corrosion are used in the line, they should be adequately protected by wrapping with plastic tape or coating with high quality corrosion preventives. Where plastic tape is used, all surfaces to be wrapped should be thoroughly cleaned and then coated with a primer compatible with the tape.

7.7.3 Working pressure or head

Different types of pipes such as RCC, PVC, uPVC, HDPE, steel, Galvanized iron or cast iron can be used for conveyance systems in irrigation. The first design consideration is the working head or pressure. The design pressures in irrigation systems are normally in the range from 5m to 25m. Therefore, in general, PVC PN2.5 pipes can be used conveniently. However in hilly terrain, operating pressures in excess of 2.5 kg/cm^2 or 25m head are not unusual.

Gravity drip systems operate with head of 2m and above but pressurized drip irrigation systems usually require minimum of 1bar or 10m head at the emitters for proper operations while most sprinkler systems require about 2 to 3.5bar or 20 to 35m head at sprinkler head for better operation. However it is usually advisable to follow specific recommendations set by the manufacturer for all pipe conveyance systems.

The pressures in the main pipe line of a minor irrigation scheme is generally up to 10 m Since the main line generally follows the alignment of the contour, head drop along the main line is less (about 1 to 5 m loss of head). However, quite frequently there are a number of falls along the canal alignment and hence the main line can be subjected to large static pressures. The allowable heads under each class of pipe may be seen from nominal pressure (PN) of the manufacturer to be used for specific site conditions. These pipe specifications are generally available in diameters ranges.

7.7.4 Low head systems

In low head pipe line system water is taken from the water source and directly distributed to basins, borders, and furrows. These low head pipeline works satisfactorily on non-uniform grades, and also at uphill and downhill the land slopes. Such pipeline consists of an inlet, one or more outlets, with head control devices and surge protection structures, air relief valve, flow meter and debris and sand removal devices. Pressure relief, air release, and vacuum relief valves that are used for pressurized pipelines are also used with low-head pipelines. Pipelines permit the conveyance of water on uphill or downhill slopes. These systems are also suitable to undulating topography and can supply water at any part of the farm. The pipe line systems can be buried or on the surface. Surface pipe lines portable and these are brought back after irrigation. The buried pipe lines placed below the ground surface are permanent and called as permanent underground pipeline. Underground pipe line conveyance system is preferred over surface pipe lines as the cultivation can be done on the land above pipeline and it does not affect farming operation.

Pressure relief valves may be used as alternative to serve the pressure relief functions of vents and stands (manholes) open to the atmosphere. They do not function as air release valves and should not be substituted for such valves where release of entrapped air is required. Pressure relief valves shall be marked with the pressure at which the valve starts to open. Adjustable valves shall be installed in such a manner to prevent changing of the adjustment marked on the valves.

In Low Head Pipe lines-Pipelines using low head pipe shall be designed such that the maximum static or working pressure of the system, including free board, does not exceed 15.2 m (50ft) of water.

7.7.5 High head Systems

The pipeline shall have a pressure class rating greater than the State of Working pressure plus surge at any point. If the surge is not known, the working pressure shall not be exceeding maximum allowable working pressure given by manufacturer for the particular pipe used.

In High Head Pipe lines- Pressure relief valves shall be large enough to pass the full discharge with a pipe line pressure no greater than 50% above the permissible working head of the pipe and shall be set to open at a pressure no greater than 0.345 kg/cm^2 (3.45m) above the pressure rating of the pipe.

7.7.6 Pipe flow velocity

The continuity equation governs flow in pipe systems. The pipe system is liable to clogging/choking due to the sedimentation. Hence minimum permissible velocity should be fixed to safeguard the system against silting this is usually not less than 1m/s. Economic optimum velocity is typically in range of 1.8 to 2.4 m/s. Different maximum permissible flow velocity are stated for different pipe materials and thus it is advisable to follow manufacturers recommendations for the specific pipe type. Special consideration must be given to assure that proper pressure and/or air relief valves are used with all velocities. The Continuity equation is given by:

$$Q=V*A \qquad (7\text{-}1)$$

Where, Q= Discharge in m^3/s
V= Velocity of flow in pipe in m/s
A= flow area in m^2

7.8 PIPE HYDRAULICS AND DESIGN FORMULAE

The design of supply conduits is dependent on resistance to flow, available pressure or head, allowable velocities of flow scour, sediment transport, quality of water and relative cost.
There are a number of formulae available for use in calculating the velocity of flow. However, Hazen and Williams’s formula for pressure conduits and Manning's formula for free flow pipe lines are popularly used:

7.8.1 Hazen- William's formulae

The Hazen - Williams' formula is expressed as:

$V = 0.849\ CR^{0.63}\ s^{0.4}$ …………………………………………………………………. (7-2)

For Circular conduits, the expression becomes

$v = 4.567 \times 10^{-3}\ Cd^{0.63}\ S^{054}$ ………………………………………………………. (7-3)

$Q = 1.292 \times 10^{-5}\ Cd^{2.63}\ s^{0.54}$ ……………………………………………………... (7-4)

Where, Q =discharge in m^3 per hour
d = Diameter of pipe in mm.
V = Velocity in m/s and
R = Hydraulic radius in m.
S = Slope of hydraulic gradient and
C = Hazen and Williams' Coefficient

The values of Hazen and Williams' Coefficient "C" (Coefficient of Roughness) for various materials are given in Table 7-1.

Table 7-1: Values of Hazen-Williams coefficient 'c' for various conduit materials

Conduit Material	Recommended values for	
	New Pipes	Design Purposes
Cast Iron	130	100
Galvanized iron 50 mm	120	100
Galvanized iron 50 mm and below used for house service connections	120	55
Steel, reverted joints	110	95
Steel Welded joints lined with cement for bituminous enamel	140	110
Steel Welded Joints	140	100
Concrete	140	110
Asbestos cement	150	120
Plastic Pipes	150	120

Source: INCID, Pipe distribution system for irrigation, 1998

7.8.2 Manning's formula

$V = (1/n)\ R^{2/3}\ S^{1/2}$ ……………………………………………………………………. (7-5)

For circular conduits

$V = 3.968 \times 10^{-3} \times (1/n)\ d^{2/3}\ S^{1/2}$ and ……………………………………………. (7-6)

$Q = 8.661 \times 10^{-7} \times (1/n)\ d^{2/3} \times S^{1/2}$ ………………………………………………. (7-7)

Where, Q = discharge in cubic meter per hour
S = Slope of hydraulic gradient
d = diameter of pipe in mm.
R = Hydraulic radius in meters
V = Velocity in m/s., and
n = Manning's coefficient of roughness

The co-efficient of roughness for use in Manning's formula for different materials as presented in Table 6-4 may be adopted generally for design purposes unless local experimental results on other considerations warrant the adoption of any other lower value for the coefficient. For general design purposes, however, the value of 'n' for all sizes may be taken as 0.013 for plastic pipes and 0.015 for other pipes.

7.8.3 Darcy Weisbach formula

Darcy and Weisbach suggested the first dimensionless equation for pipe flow problem as:

$$S = H/L = f\, v^2/(2gD) \qquad (7\text{-}8)$$

Where S = Slope of hydraulic gradient
H = Head loss due to friction over length L in m
f = Dimensionless friction factor and
g = acceleration due to gravity in m/s^2
D= diameter of pipe in m

The friction factor values in practice for commonly used pipe materials are given in table below.

Table 7-2: Recommended friction factor 'f' in Darcy and Weisbach formula

SN	Pipe material	Diameter (mm)		Friction factor	
		From	To	New	Design period of 30 years
1	R.C.C. pipe	100	2000	0.01 to 0.02	0.01 to 0.02
2	A.C (Asbestos Cement)	100	600	0.01 to 0.02	0.01 to 0.02
3	HDPE/PVC	20	100	0.01 to 0.02	0.01 to 0.02
4	SGSW pipe	100	600	0.01 to 0.02	0.01 to 0.02
5	C.I (For Corrosive waters)	100	1000	0.01 to 0.02	0.053 to 0.03
6	C.I (for non- corrosive water)	100	1000	0.01 to 0.02	0.034 to 0.07
7	Steel	100	2000	0.01 to 0.02	0.01 to 0.04
8	G.I	15	100	0.14 to 0.30	0.315 to 0.06

Source: INCID, Pipe distribution system for irrigation, 1998

7.8.4 Limitations in using Hazen-Williams' formula

The followings are some of the limitations in using Hazen-Williams' Formula:

- Commonly used Hazen-Williams formula has following inherent limitations. The numerical constant of Hazen-Williams formula (1.318 on fps units or 0. 85 in mps units) has been calculated for an assumed hydraulic radius of 1 foot and friction slope of l/1000. However, the formula is used for all ranges of diameter & friction slopes. This practice may result in an error of up to +/-30% in the evaluation of velocity and +/- 55% in estimation of frictional resistance head loss.
- The Darcy-Weisbach formula is dimensionally consistent. Hazen-Williams coefficient 'C' is usually considered independent of pipe diameter, velocity of flow and viscosity. However, to be dimensionally consistent and to be representative of friction conditions it must depend on relative roughness of pipe and Reynold's number. A comparison between estimates of Darcy, Weisbach friction factor f, and its equivalent value computed from Hazen - Williams C for different pipe materials brings out the error in estimation of 'f' as, up to +/- 45% in using Hazen Williams formula. It has been observed that for higher C values (new and smooth pipes) and larger diameters error is less whereas, it is appreciable for lower 'C' values (old and rough pipes) and lower diameter at higher velocities.
- The Hazen-Williams formula is dimensionally inconsistent, since the Hazen Williams 'C' has the dimension of $L^{-0.37}\ T^{-1}$ and therefore is dependent on units employed.

7.8.5 Modified Hazen-Williams formula

The modified Hazen Williams formula has been derived from Darcy-Weisbach and Colebrook-White equations and obviates these limitations of Hazen-Williams Formula:

$$V = \frac{3.83\ \mathrm{Cr}\ d^{0.6575}\ (gs)^{0.5525}}{u^{0.105}} \quad \text{(7-9)}$$

Where, V = Velocity in m/s.
d = Pipe diameter
s = Friction slope
Cr = Coefficient of roughness
g =Acceleration due to gravity (i.e.9.81 m/s^2)
u = Viscosity of liquid, for circular conduits, value of u at 20°c for water is 10^{-6}m^2/s

The modified Hazen-Williams formula is derived as:

$$V = 143.534 C_r r^{0.6575} S^{0.5525} \quad \text{(7-10)}$$

$$h = \frac{L[\frac{Q}{C_r}]^{1.81}}{994.62\ D^{4.81}} \quad \text{(7-11)}$$

Where, V = Velocity of Flow in m/s
Cr = Pipe roughness coefficient, (1 for smooth pipes, < 1 for rough pipes)
r = hydraulic radius in m = flow area/wetted perimeter.
s = friction slope
D = internal diameter of pipe in m.
h = friction head loss in m
L = Length of pipe in m and
Q = Flow in pipe, in m^3/s.

7.8.6 Experimental estimation of Cr values

The coefficient of roughness in various pipe flow formulae are based on experiments conducted over a century ago. The value of Hazen-Williams C, Manning's n, and roughness values in Moody's Diagram have also been based on experimental data collected in early nineteenth century. There have since been major advances in pipeline technology. Both the manufacturing processes and jointing methods have improved substantially over the years and newer pipe materials have come into use. Continued usage of roughness coefficients estimated without recognition of these advances in bound to result in conservative designs of water system.

Accordingly, Cr values of commonly used commercial pipe materials have been experimentally determined in a study conducted at Hyderabad. This study covered pipe diameters of 100 to 1500 mm over a wide range of Reynolds numbers (3 x 10^4 to 1.62 x 10^6) encountered in practice. The results bring out that centrifugally spun CI, RCC, AC and HDPE Pipes behave as hydraulically smooth when new and hence Cr =1 for these pipes.

The use of Hazen-Williams 'C', as per Table 6-1, results in underutilization of above pipe materials when new. The extent of underutilization varies from 13 to 40 percent for CI Pipes, 23 percent for RCC and AC pipes and 8.4 percent for HDPE/PVC pipes.

7.8.7 Design recommendations for use of modified Hazen-Williams formula

Following design recommendations need to be ensured effective utilization of pipe carrying capacity:

- New CI, RCC, AC and HDPE, pipes behave hydraulically smooth and hence Cr of 1 is recommended for design purpose.
- For design period of 30 years, no reduction in Cr needs to be effected for RCC, AC and HDPE pipes irrespective of the quality of water. However, care must be taken to ensure self-cleansing velocity to prevent formation of slimes and consequent reduction in carrying capacity of these pipes with age.
- For design period of 30 years, 15% reduction is required for CI pipes if non-corrosive water is to be transported. The design must also ensure self-cleansing velocity.
- While carrying corrosive water, CI and steel pipes will lose 47 and 27 percent of their capacity respective over a design period of 30 years. Hence, a cost trade-off analysis must be carried out between chemical and bio-chemical correction of water quality, provision of a protective lining to the pipe interiors and design at reduced Cr value for ascertaining the utility of CI and steel pipe material in the transmission of corrosive waters.

Recommended Cr values are presented in Table 7-4. The use of recommended Cr values in conjunction with Modified Hazen Williams formula or the nomograph will ensure fuller utilization of pipe materials.

Table 7-3: Recommended Cr Values in Modified Hazen-Williams Formula (At 20^{o} C)

SN	Pipe material	Diameter (mm)		Velocity (m/s)		Cr value when new	Cr value for design period of 30 years
		From	To	From	To		
1	R.C.C.	100	2000	0.30	1.80	1.00	1.00
2	A.C	100	600	0.30	6.00	1.00	1.00
3	HDPE/PVC	20	100	0.30	1.80	1.00	1.00
4	C.I (For water with positive Langelier's Index**)	100	1000	0.30	1.80	1.00	0.85*
5	C.I (For water with negative Langelier's Index)	100	1000	0.30	1.80	1.00	0.53*
6	Steel (For water with negative Langelier's Index)	100	2000	0.30	2.10	1.00	0.73*
7	SGSW	100	600	0.30	2.10	1.00	1.00
8	G.I (For water with positive Langelier's Index)	15	100	0.30	1.50	0.87*	0.74

Source: INCID, Pipe distribution system for irrigation, 1998
*These are average Cr values which result in a maximum error of +/- 50% in estimation of surface resistance.
**The Langelier Index is an approximate indicator of the degree of saturation of calcium carbonate in water.

7.8.8 Effect of temperature on coefficient of roughness

Analysis carried out to evaluate effect of temperature (viscosity) on value of Cr reveals that the maximum variation of CR for a temperature range of 10^{o}C to 30°C is 4.5% for a diameter of 2000 mm at a velocity of 3.0 m/s. In the light of this revelation, Cr values above are presented for average temperature of 20°C.

7.8.9 Reduction in carrying capacity of pipe with age

The values of Hazen-Williams "C" are at present arbitrarily reduced by about 20 to 23% to cater reduction in carrying capacity of pipes, with age. A recent study has revealed that chemical and bacteriological quality of water and flow velocity affect the reduction in carrying capacity of pipes with age.

The data on existing system in some cities has been analyzed along with the experimental information gathered during the study to bring out a rational approach to the reduction in carrying capacity of pipe with age.

The 'Cr' values obtained in this analysis has shown that, except in the case of CI and Steel pipes while carrying corrosive water, the current practice of arbitrary reduction in 'C' values "coefficient of roughness" results in underutilization of pipe material to the extent of 38 to 71 percent for CI Pipes for non-corrosive water, 46 to 93 percent for RCC pipes and 25 to 64 percent for AC and HDPE pipes.

7.8.10 Resistance due to specials and appurtenances

Pipeline transitions and appurtenances add to the head losses, which are expressed as velocity heads as $K\ V^2/2g$ where V and g are in m/s and m/s^2 respectively or equivalent length of straight pipe. The values of K to be adopted for the different fittings are given in Table below.

Table 7-4: 'K' Values for different fittings

Type of fitting	Value of K
Sudden Contractions	*0.30-0.50
Entrance shape well rounded	0.50
Elbow 90^o 45^o 22^o	0.50-1.00 0.40-0.75 0.25-0.50
Tee 90^o take off Straight run Coupling	1.50 0.30 0.30
Gate Valve (Open)	**0.30-0.40
With reducer and increaser	0.50
Globe	10.00
Angle	5.00
Swing Check	2.50
Meter venturi	0.30
Orifice	1.00

Source: INCID, Pipe distribution system for irrigation, 1998
* Varying with area ratio and ** varying with radius ratio

7.9 UNDERGROUND PIPELINE SYSTEM

7.9.1 Conditions to use different pipe materials

Both reinforced concrete pipes and PVC pipes are used for constructing water distribution systems in the command areas. PVC pipes are often preferred because of the ease of installation and ensure leak proof joints. Other factors favoring its use are speed of laying and greater resistance to internal friction, as compared to concrete pipes of a given diameter, to convey large quantities of water. However, skill and adherence to proper procedure in laying the pipes and accessories can be used more economically and with equal efficiency, as compared to PVC pipes on plain land.

PVC pipes have distinct advantages over concrete pipes. In undulation topography, however, they need to be buried to avoid UV degradation. HDPE pipe can be used above ground level.

7.9.2 Inlet Components of underground pipeline system

Water inlet components are required to carry water from the source in to low head underground pipelines. An inlet structure is required to develop adequate pressure and full flow capacity so as to distribute water at different points on the farm. Inlet components use a sand trap and trash screen to prevent entry of debris and heavy suspension of sand in the pipe lines.

7.9.3 Pump stand

A pump stand is located at the inlet end of underground pipeline system. Pump stand must be high enough to provide the pressure needed at all the pipe outlets. Pump stands size is larger than the diameter of pipe line, to dissipate high velocity stream and release of entrapped air before water enters pipeline. Sectional view of the pump stand is shown in Figure 7-1.

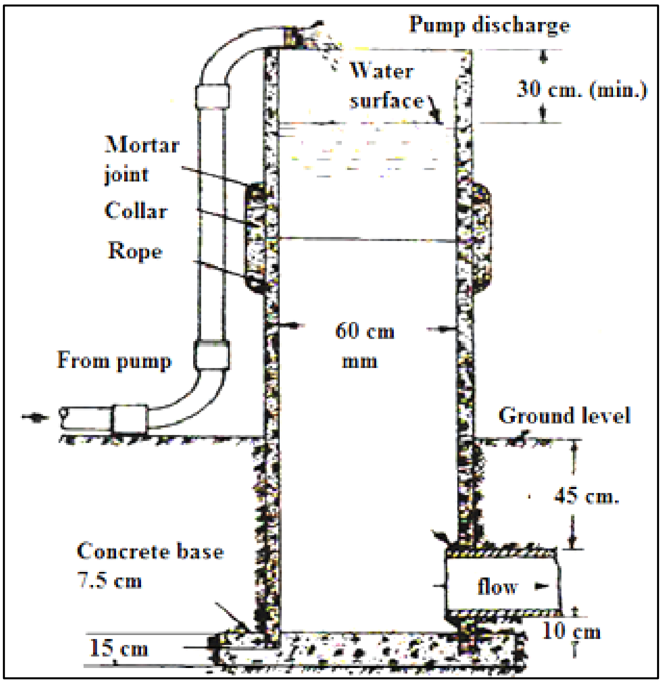


Figure 7-1: Pump stands for underground pipeline
Source: Michael, 2010

7.9.4 Gravity inlets

The gravity inlet is used when water surface elevation of the water source is sufficient to allow gravity flow into the pipeline and to provide the adequate pressure needed at every point of pipe line and outlet. The low head underground pipe line directly connected with water source can be used for delivering water from a minor canal as shown in Fig. 7-2.

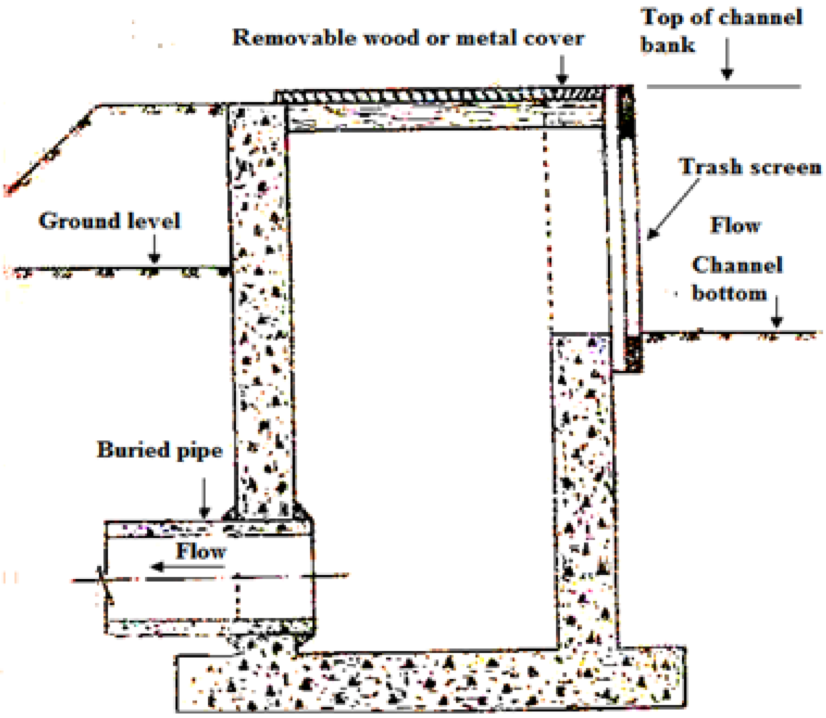


Figure 7-2: Cross-Section of an inlet taking water from canal into an underground pipeline
(Source: Michael, 2010)

7.9.5 Gate stands or Manholes

Gate stands or manholes are often circular, rectangular or dome in shape. They can be made of concrete or masonry. Some times in-situ molds are used to construct gate stands. Gate stands are installed to control flow into branch lines. These are installed where branch lines take off from main line. They also prevent high pressure and act as surge chamber. Each outlet of a gate stand is equipped with slide gate or gate valve to release water through a particular gate valve. Fig. 7-3 shows branching off water from main pipeline and gate stand.

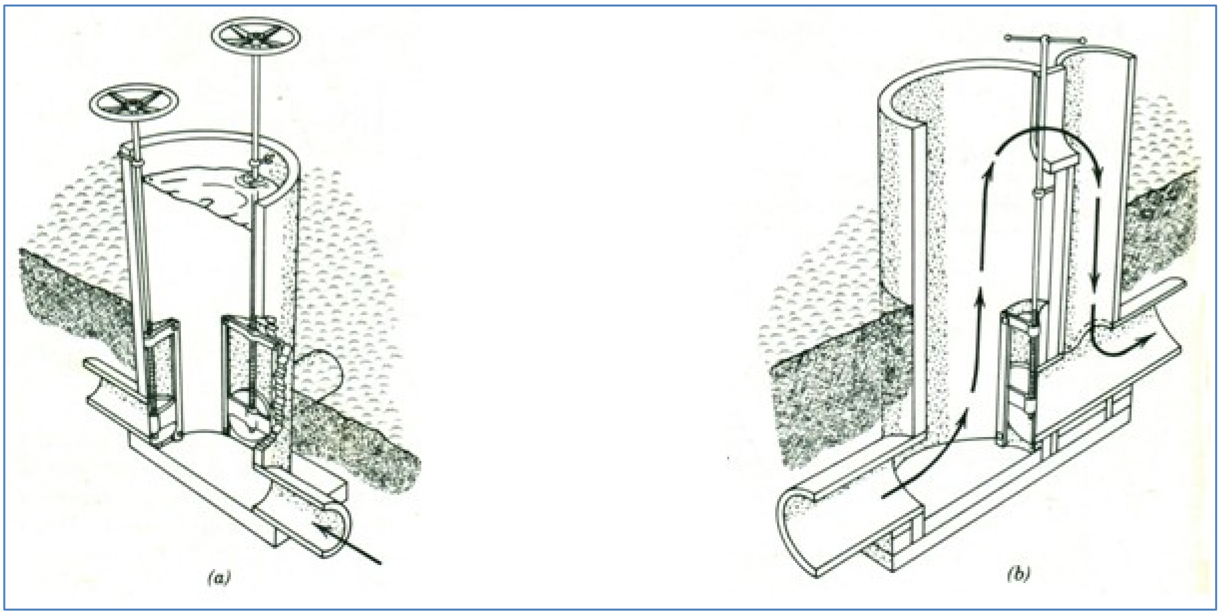


Figure 7-3: Gate stand (a) and overflow from gate stand (b)

Box 7-1:
Worked Example-8: *Design for a pipe line between two gate stands or manholes on straight and uniform gradient of given data below using modified Hazen William's method:*

Distance between two manholes is 200m
Design discharge is 200 liter/second
Required residual head at pipe outlet is 1m.
Assume pipe fitting, inlet and exit loss coefficient is in total 2.
Design for HDPE pipe
Design for steel pipe with design period of 30 years conveying corrosive water or water with negative Langelier's Index.

Solution:

(a) Design for HDPE pipe

Generally continuity equation governs the flow in the pipe system i.e., Q=V*A
Economic velocity range is given as 1.8-2.4m/s hence if 2m/s velocity is to be adopted then:
Area of pipe or flow will be $A=Q/V=0.2m^3/s/2m/s$; $A=0.10m^2=\pi D^2/4$, thus diameter, D=0.36m
Consider nominal pipe diameter of =0.35m then area of pipe is $0.0962m^2$ and flow velocity will be 2.08m/s.

Head loss due to friction in HDPE pipe,

$$\boldsymbol{h} = \frac{\mathbf{L}[\frac{\mathbf{Q}}{C_r}]^{\mathbf{1.81}}}{\mathbf{994.62}\ \boldsymbol{D}^{\mathbf{4.81}}} \quad \text{……………………………………………………………..……………….. (7-12)}$$

Cr = Pipe roughness coefficient, (1 for smooth pipes, < 1 for rough pipes) here for HDPE pipe Cr=1 is considered for HDPE; D = internal diameter of pipe in m, h = friction head loss in m
L = Length of pipe in m and Q = Flow in pipe, in m^3/s.
Value computed on Excel sheet is presented as follows:

Table 7-5: Friction head loss in HDPE pipe line

L	Q	Cr	D	Q/Cr	L(Q/Cr)^1.81	994.62D^4.81	h
200	0.2	1	0.4	0.2	10.86	12.12	0.896

Minor loses, $h_m = \frac{KV^2}{2g}$ ……………………………………………………………..…………..…. (7-13)

Where, h_m is minor head losses in m
V= velocity of flow in pipe line in m/s =2.08m/s
g = gravitational acceleration $=9.81m/s^2$
K= minor loss coefficient given as 2

Thus, minor head loss=0.44m
Total head loss in pipe line=1.337m
Residual head required is 1m
Thus level difference between the two manholes must be at least 1+1.337=2.337m.
Or slope of hydraulic gradient will be:
S = H/L= (2.337/200) x100 =1.17%
This indicates that 350mm diameter HDPE pipe can convey 200l/s in 200m pipe line laid with 1.17% slope with residual head of 1m at outlet.

(b) Design steel pipe conveying corrosive water (design period 30 years)

Same size of pipe can be considered that is 350mm steel pipe to convey 200l/s

Here the coefficient of friction Cr for design purpose =0.73 from table 6-6.

Table 7-6: Friction head loss in steel pipe conveying corrosive water

L	Q	Cr	D	Q/Cr	L(Q/Cr)^1.81	994.62D^4.81	h
200	0.2	0.73	0.4	0.273973	19.20	12.12	1.584

Here head loss due to friction would be 1.584m

Minor head loss= 0.44m

Residual head required 1m

Thus total head required=3.024m

Slope of hydraulic gradient will be: S = H/L ………………………………………………..…... (7-14)

= (3.024m/200m)*100=1.51%

Thus, for steel pipe of same diameter conveying corrosive water of 200l/s in 200m pipe line gradient shall be 1.51% to obtain residual head of 1m at outlet.

7.10 PRESSURE VARIATIONS IN IRRIGATION PIPE LINES

Pressure in the pipe line increases or decreases due to change in elevation (uphill or downhill conditions). Thus the pressure at each reach must be calculated accordingly and designed. The difference in pressure between two locations along a pipeline can be estimated using following equation.

$$H_d = H_u - 9.81(H_L \pm \Delta He) \qquad \text{(7-15)}$$

Where, H_d, H_u pressure at down- and upstream position, respectively (kPa);

H_L= energy loss in pipe between the up-and downstream positions (m);

ΔH_e = difference in elevation between up-and downstream positions (m);

When the change in elevation between the up-and downstream positions is uphill, the sign is plus (+) conversely, this sign is negative (-) when the elevation at the upstream location exceeds the elevation at the downstream location.

8 DESIGN OF DRAINAGE SYSTEMS

8.1 THE NEED FOR DRAINAGE IN IRRIGATION PROJECTS

Drainage works are inseparable from irrigation, as excess water should be drained for normal growth of the crops irrigated. Lack of drainage also results in failure of irrigation infrastructure. When irrigation or rainfall water cannot fully infiltrate into the soil or cannot move freely over the soil surface to an outlet or ground water rises over a certain period of time, then ponding or water logging occurs. Smoothening the land surface or establishing grids of drainage network along irrigation canals leading to natural drains so as to remove water from such low-lying areas in which water settles can partly solve such problem. Otherwise, lack of adequate drainage impairs both crop growth and farm operation.

This drainage guideline thus presents surface agricultural drainage systems relevant to small scale irrigation projects so as to enable to remove these water logging. It describes drainage water sources, when & why drainage is required, suggests relevant data required, where and how to obtain them and apply the data with a view to provide adequate drainage.

8.2 CLASSIFICATION OF DRAINS

Drains are broadly classified into surface, sub-surface or a combination of the two. In SSI projects, the commonly constructed drainage system is limited to surface drainage, which is further sub divided in to internal and external drains. A surface drainage system is applied when the waterlogging occurs on the soil surface, whereas a subsurface drainage system is applied when the waterlogging occurs in the soil. The followings are detailed categories of drains, according to the purpose for which they are designed.

- Field Drains: These are designed on the farm along field canals to drain the water from the irrigation plot,
- Collector or Link drains: These are branch draining sub-catchments into the out fall drain. They are aligned along subsidiary valley lines, as collector drains. These are designed to receive drain water from field drains;
- Out Fall Drains: These are the main drains out falling into a stream or river from and particular catchment. They are usually natural drains, collecting drainage from collector and/or minor ones depending on their location;
- Ditch drain: These are designed to drain runoff water by connecting borrow pits along roads, railway line, etc.
- Cunette: This is a small drain designed in the bed of main drain at a level lower than the normal bed levels of the main drain for carrying seepage discharge without allowing it to spread across the entire section of the drain,
- Seepage drains: These are designed to collect the seepage water from the embankment and to collect the seepage water from the canal embankments to drain it either directly into natural out fall or into a carrier drain.

Interceptor drains (Catch Drain): These are designed at the outer periphery of access road along main canal to protect it from flood damages.

8.3 DESIGN CRITERIA AND CONSIDERATIONS IN DRAINAGE SYSTEM

8.3.1 Drainage system layout

The alignment of open drains follows the paths of natural drainage and low contours. The drains are not aligned across a pond or marshy land. Every drain has an outlet, the elevation of which decides the bed and water surface elevations of the drain at maximum flow.

At the outset of irrigation and drainage system planning, identification of natural drainage network in the command area and possible location of escape down the secondary canal need to be carried out. If they are few in number then additional on-farm and collector drainage canals parallel to field and tertiary canals and as required in between two blocks need to be designed.

8.3.2 Drainage module or drainage coefficient

For designing of internal drains, a maximum one day precipitation corresponding to once in 5 years recurrence interval or drainage module is adopted.

The amount of water in the field drain can be estimated by different methods and set as drainage module which is also called drainage coefficient. This drainage coefficient is related to the characteristics of the catchment area and the magnitude of the storm against which the catchment area is to be protected. Because, within a particular catchment area, there may be sloping upland, flat bottom land, forest land, highly developed general cropland, or even some urban land. The characteristics of each distinct type of land and land use within the catchment area determine the coefficient to be used in design of improvements on that parcel of land and in computing the drainage flow from it (Leyva, 2010). The drainage coefficient is computed using the simple formula called the Cypress Creek equation (NRCS, 1998), which was developed by the Soil Conservation Service (now called the Natural Resource Conservation Service) of the United States Department of Agriculture.

$$q = 0.21 + 0.00744 * P_{24} \quad \text{(8-1)}$$

Where, q = drainage coefficient related to the drainage area and the magnitude of the storm (cubic meters per second per square kilometer) (Ochs and Bishay, 1992); P_{24} = 24 hr. maximum rainfall with different return period (5 year return period preferred).

8.3.3 Design procedures of interceptor/catch drain

Design procedures of such run off interceptor or catch drain is summarized as follows:

- Delineate the watershed that contributes to the flows across the main canal;
- From topography map along the main canal identify all drainage water ways;
- From the watershed identify the catchment area contributing directly to the runoff interceptor
- Determine directions of disposal (one/two directions);
- Compute the peak flood for the known catchment area and physical characteristics;
- Fix the drain section using Manning's (say 0.03) may be assumed for the roughness coefficient as it could be poorly maintained;
- The section can be varied for drains such that it is the least at the beginning and maximum size at the end where it discharges to the nearest drainage course;
- In case of interceptor drains meant to cutoff part of the subsurface flow lower bed width to depth ratio could be adopted otherwise shallow and wide drains are more acceptable;

- No major structures are expected along interceptor drain except cross drainage works which are on the adjacent main canal;
- Human & animal crossings can be made by providing level crossings such as Irish fords.

8.4 DESIGN OF DRAINAGE SYSTEMS

8.4.1 Layout of surface drainage system

Layout of surface drainage system is drawn in conjunction with that of irrigation system layout but with broken lines. There are four types of open drains which need to be considered as surface drainage in SSI Projects: interceptor/catch drains (ID) laid along the main canals which of-course is treated with access road running parallel to this canal, Collector Drains (CD), Tertiary Drains (TD) and Field Drains (FD).

The interceptor and tertiary drains need to run nearly parallel to the contours, but the field and collector drains are laid to run across contours parallel to field and secondary canals in low-lying areas respectively. The interceptor drains are fully external drains while the field and tertiary drains are totally internal drains, but some collector drains collect both from external and internal, i.e. external from natural and interceptor drains and internally from the field and/or tertiary drains.

As a rule, prior to starting the design of drainage canals, the out fall and main collector drains layout need to be drawn out on maps or enlarged aerial photographs. A decision should then be made on the location of the inlet structures carrying the drainage flows from the collector drainage into the main collector and outfall drain.

8.4.2 Design discharge and cross section of drains

Normally, the cut sections of on-farm drains are provided to accommodate the design discharge where drains follow natural valley lines. In such cases, no embankments should be provided along the drain so as to allow free flow of water from the surrounding areas. Wherever embankments are necessary for accommodating a portion of the design discharge or where disposal of excavated soil will be very costly, wide gaps should be provided in the embankments on either sides so as to allow unrestricted flows and incase of incidence of discharges higher than the channel capacity. The water should spill over the area and return to the channel fully when the discharge in it recedes.

The design capacity of open ditches is influenced by:

- Precipitation,
- Size of the contributing area,
- Topography,
- Soil characteristics,
- Vegetation,
- Degree of protection warranted,
- Frequency and height of ridges and flood waters from rivers, lakes and other outlets, and
- Leaching requirements in irrigated areas.

In the forced on diversion reaches, embankments on both sides are, however, provided, as the design discharge cannot be accommodated within the cut section of the drain. However, even in such cases attempts should be made by selecting a proper alignment to keep the height of the embankment to the minimum. In such cases, inlets of adequate size should be provided in the embankments to allow entry of water from surrounding area.

Drainage canal discharge increase as we go from its initial reach to the tail unlike irrigation canal, because it receives and collects drains while moving downstream. Thus its capacity should accordingly increase to carry the expected design flow.

8.4.3 Water level

Design water level (WL) in field canals should preferably be kept at 0.25 meter below the ground level to work as a free board and also to drain away the soil moisture of the root zone. To facilitate gravitational flow from furrows into field ditches it should be adequate to maintain a minimum of 0.1m head in the field drains.

Water level and hence drain ditch should be below OGL on the field side so as to allow entrance of drainage water in to the drain, otherwise it may block and create stagnant water on this side. Typical section of earthen trapezoidal drain is shown in figure below.

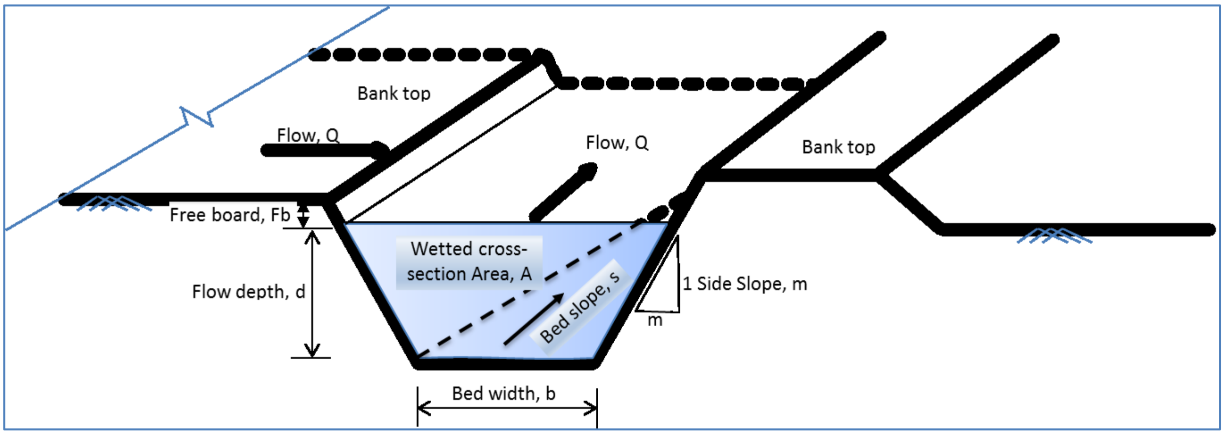


Figure 8-1: Typical cross section of earthen trapezoidal drain

8.4.4 Allowable velocities in drains

The permissible velocity of flow in a grassed canal is the velocity that will prevent severe erosion in the channel for reasonable length of time. Permissible velocities for different vegetal cover, channel slopes, and soil conditions, recommended on the basis of investigation by the U. S Soil conservation source, are shown in Table below.

Table 8-1: Permissible velocities for channels lined with grass

Cover	Slope Range (%)	Permissible Velocity, m/s	
		Erosion-resistant soil	Easily eroded Soils
Bermuda grass	0-5	2.4	1.8
	5-10	2.1	1.5
	>10	1.8	1.2
Buffalo grass, Kentucky bluegrass,	0-5	2.1	1.5
	5-10	1.8	1.2
	>10	1.5	0.9
Grass mixture	0-5	1.5	1.2
	5-10	1.2	0.9
	Do not use on slopes steeper than 10%		
Lespedeza sericea, weeping love grass, ischaemum (yellow blue stem), kudzu, alfalfa,	0-5	1.1	0.75
	Do not use slopes steeper than 5 % except for side slopes		

Cover	Slope Range (%)	Permissible Velocity, m/s	
		Erosion-resistant soil	Easily eroded Soils
crabgrass	in a combination channel		
Annuals-used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudan grass	0-5	1.1	0.75
	Use on slopes steeper than 5 % is not Recommended		

Source: Design Guideline on Drainage System Design, MoWR, 2002

8.4.5 Minimum permissible velocity in drains

The minimum permissible velocity also called the non-sitting velocity is the lowest velocity that will not start sedimentation and induce the growth of aquatic plant and mass. This velocity is uncertain and its exact value cannot be easily determined. For water carrying no silt load or for de-silted flow, this factor has little significance except for its effect on plant growth.

The ideal minimum gradient is one that would have sufficient velocity at low flows to prevent deposition and grow of aquatic plants. This velocity would be in the range of 0.23 to 0.30 m/sec for prevention of silt and fine sand deposits; 0.45 to 0.6 meter per second for the prevention of weeds and grasses and 0.75 meter per second or more to inhibit growth of aquatic plants. In areas where ideal velocities cannot be obtained, drain should be designed with a minimum velocity of about 0.30 m/sec for a normal flow.

8.4.6 Maximum permissible velocity in drains

The maximum permissible velocity, also called the non-erodible velocity, is the largest mean velocity of drain flow that will not cause erosion of the channel body. This velocity is very uncertain and variable and can be estimated from different reference materials once bed material is known. In general, following table shows the maximum permissible velocity and n (Manning coefficient) values of various materials.

Table 8-2: Maximum permissible velocity and tractive force

Material	Clear Water		Water transporting colloidal silt		Roughness, n
	V (m/s)	τ (N/m2)	V (m/s)	τ (N/m2)	
Fine sand, colloidal	0.45	1.3	0.75	3.7	0.020
Sandy loam, non-colloidal	0.50	1.8	0.75	3.7	0.020
Silt loam, non-colloidal	0.60	2.3	0.90	5.4	0.020
Alluvial silts, non-colloidal	0.60	2.3	1.07	7.3	0.020
Ordinary firm loam	0.75	3.7	1.07	7.3	0.020
Volcanic ash	0.75	3.7	1.07	7.3	0.020
Stiff clay, very colloidal	1.15	12.7	1.5	22.5	0.025
Alluvial silts, colloidal	1.15	12.7	1.5	22.5	0.025
Shells and hardpans	1.8	32.8	1.8	32.8	0.025
Fine gravel	0.75	3.7	1.5	15.6	0.020
Graded loam to cobbles When non colloidal	1.15	18.6	1.5	32.8	0.030
Graded silts to cobbles When colloidal	1.2	21.0	1.7	39.1	0.030
Coarse gravel, non-colloidal	1.2	14.7	1.8	32.8	0.025
Cobbles and shingles	1.5	44.5	1.7	53.9	0.035

Note: V- is Unit tractive force and τ is Max. Permissible average velocity
Source: Design Guideline on Drainage System Design, MoWR, 2002

8.4.7 Tractive force theory

Drainage channels are commonly designed from earthen cross section, thus they are subjected to erosion especially during first operation. Consequently, flow velocity in such channels need to be checked using tractive force theory. The theory is based on the fact that the stability of bed and bank material is a function of their ability to resist erosion resulting from the shear force exerted on them by the moving water.

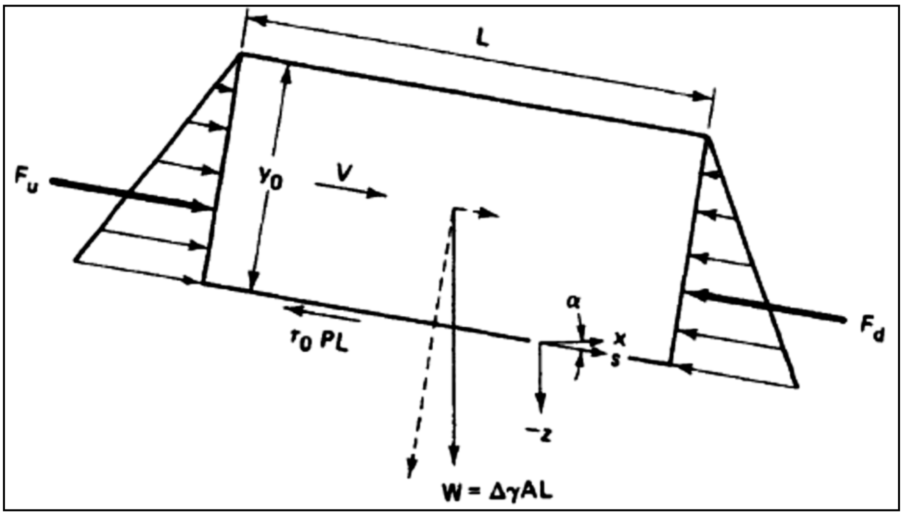


Figure 8-2: Distribution of forces acting on flowing water

This method is used to design stable channel of alluvial nature focusing primarily on defining a channel configuration that performs with an acceptable limit of stability. In this method, the canal is considered to be stable when its configuration is such that the tractive force between flowing water and the channel boundary is less than the permissible tractive force.

$F_u + W^*\sin\alpha = F_d + \tau^*P_w{}^*Lo$ ………………………………………………………….……. (8-2)

Where, Fu & Fd are u/s & d/s hydro-static forces (Fu, = Fd for uniform flow),
Pw is wetted perimeter,
W is weight of segment of fluid,
α is angle that a canal slope makes with the horizontal datum,
Lo is length of free-body segment, and
τ is the average boundary shear that retards flow.

Assuming that $F_u = F_d$, the above equation can be rearranged to solve for the boundary shear as:

$\tau = W^*\sin\alpha / P_w{}^*Lo$ ……………………………………………………………….………. (8-3)

But, $W = \gamma * A^*L$ ……………………………………………………………..…………….…….. (8-4)

By substituting equation (8-4) for the weight W and the slope S for sin α, equation (8-3) reduces to:

$\tau = \gamma^*A^*L^*S / P_w{}^*Lo = \gamma^*R^*S$ ……………………………………………………….……. (8-5)

The unit tractive force in canals, except for very wide channels, is not uniformly distributed along the wetted perimeter. This distribution varies with channel shape. In practice, the actual tractive force never quite reaches the theoretical value of γhS. The maximum value is generated along the center of the bottom and is approximately by:

$\tau = 0.97\ \gamma * h * S$. …………………………………………………………..…………….….… (8-6)

On the sides the maximum force occurs at about one-third of the water depth from the bottom and is approximately by:

$\tau = 0.75\ \gamma * h * S$. …………………………………………………………..…………….….… (8-7)

The stress is zero at both bottom corners as well as at the water surface

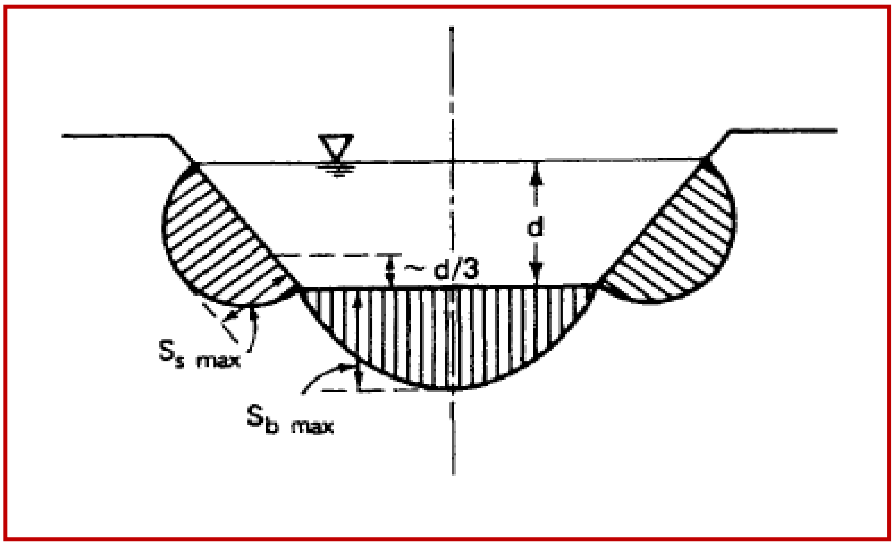


Figure 8-3: Tractive force distribution on canal surfaces

The critical tractive force is the maximum unit shear force that will not cause serious erosion of the material forming the channel bed on a level surface. Several equations are available to estimate the critical shear stress/ critical tractive force.

Leliavsky: $\tau_c = 166*D$ ………………………………………..……………………..…..….... (8-8)

Where, τ_c is in g/m^2

D is particle size/average diameter in mm

Shields: $\tau_c = S_p * (\gamma_s - \gamma) * D_{50}$ ….……..……………………………..……………………..……..… (8-9)

Where, S_p is Shields parameters

Box 8-1:
Worked example-7: *Design a trapezoidal drainage channel of side slopes 2H:1V to carry 25 m^3/s of clear water with a slope equal to 10^{-4}. The channel bed and banks comprise gravel with angle of repose estimated to 31° of size 3.0 mm. The kinematic viscosity of water can be taken as 10^{-6} m^2/s.*

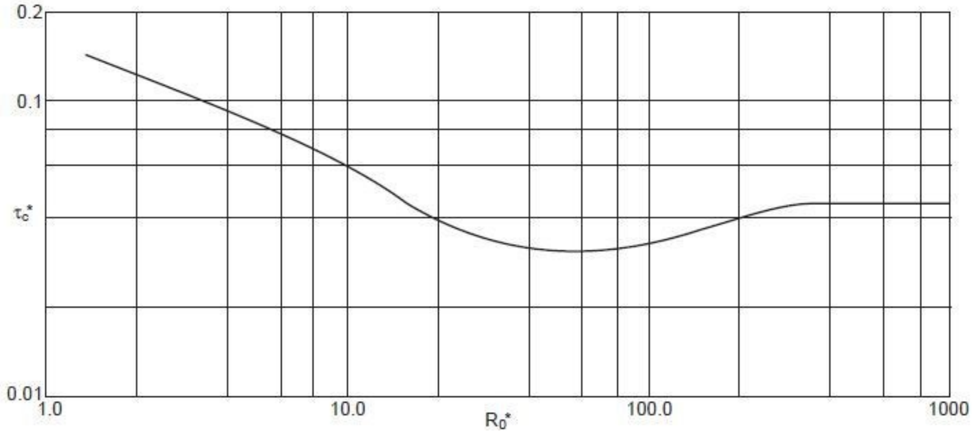
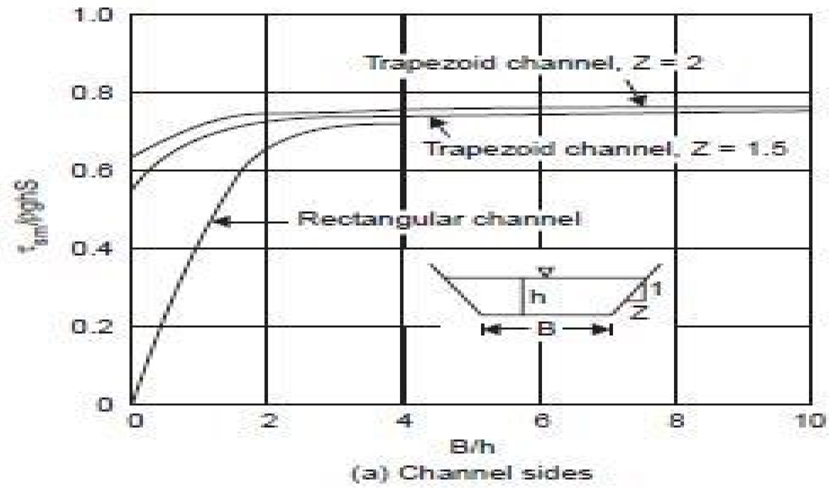


Figure 8-4: Shields' curve for the direct computation of τc



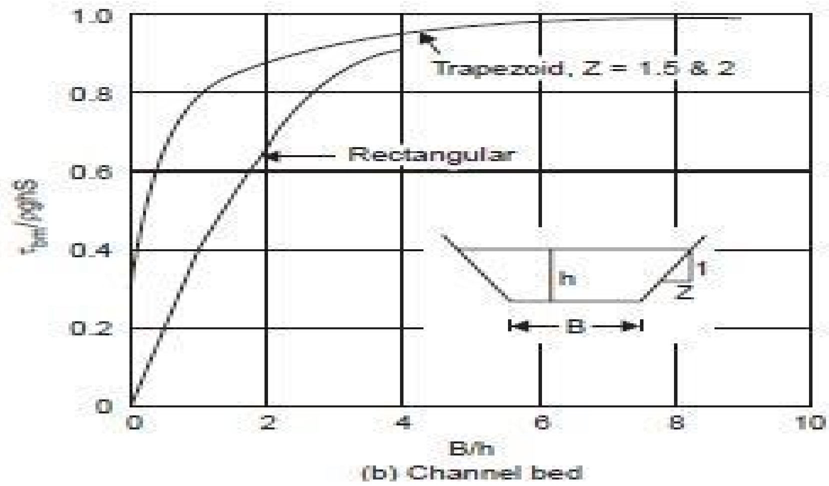


Figure 8-5: Maximum shear stress on (a) sides & (b) bed of smooth channels in uniform flow

Solution:

Table 8-3: Detailed design & analysis of alluvial channel

Parameter	Value	Remark
Given:	**Trapezoidal channel**	
Side slopes	2	H:V
Clear water	25.0 m3/s	
Longitudinal slope (%)	0.0001	
Channel bed and bank types	gravely	
Thus, angle of repose f (^{0})=	31	
Size of particle	0.003 mm	
Kinematic viscosity of water, n (m^2/s)	0.000001	
Req'd		
Design a trapezoidal channel		
Solution		
Assume specific gravity of the material, Dr=	1.65	
Acceleration due to gravity, g =	9.81	
Using Shields' curve for the direct computation of τ_c		
Thus Ro = $R_o^* = \sqrt{\frac{\Delta \rho_s g d^3}{\rho \nu^2}}$	661.09	
Critical tractive force from graph, tc*=	0.0450	
Thus tc= tc*Δρs gd = (N/m^2)	2.19	
Taking τ_{bl} = 0.9 τ_c = (N/m^2)	1.97	
$\tau_{bm} = \tau_{bl}$ =	1.97	
Now, $k = \cos\alpha \sqrt{1 - \frac{\tan^2\alpha}{\tan^2\theta}}$	0.494	
Therefore, τ_{sl} = 0.494 × τ_{bl} (N/m^2)= τ_{sm}	0.972	
Remaining computations are to be done by trial & error as follows :		
Assume B/h =	10	
Consequently, from Fig. 8.4, $\tau_{sm}/\rho ghS$ =	0.78	
Thus, h =	1.27 m	
Also from Fig. 6.5, $\tau_{sm}/\rho ghS$ =	0.99	
Thus, h =	2.03 m	
Now choosing the lesser of the two values of h =	1.27 m	
B = 10h =	12.7 m	
A = Bh + mh^2	19.4 m	
P = B + 2h $\sqrt{(1+m^2)}$	18.4 m	
Thus, R = A/P =	1.05 m	
n = $d^{1/6}/25.6$	0.0148	
Q=1/n*$AR^{2/3}S^{1/2}$	13.5 m3/s	
Since this value of Q is less than the given value, another value of B/h, say, 20.0 shall be assumed.		
Using Fig. 6.5, it will be seen that h = 1.27 m.		
B = 20h =	25.4 m	
A = Bh + mh^2	35.5 m	
P = B + 2h $\sqrt{(1+m^2)}$	31.1 m	
Thus, R = A/P =	1.14 m	
Q=1/n*$AR^{2/3}S^{1/2}$	26.2 m3/s	

This value of Q is only slightly greater than the desired value 25.00 m3/s. Hence, B = 25.4 m & h = 1.27 m.

Table 8-4: Summary of calculations for each trial

B/h	$\tau_{sm}/\rho ghS$	$\tau_{bm}/\rho ghS$	h(m)	B(m)	A(m2)	P(m)	R(m)	Q(m3/s)
10.00	0.78	0.99	1.27	12.70	19.36	18.38	1.05	13.50
20.00	0.78	0.99	1.27	25.40	35.48	32.08	1.14	26.20

8.5 FLOOD PROTECTION WORKS AS DRAIN CONTROL MECHANISM

Drainage could also result from seepage or overtopping of river flow on the adjacent command area. In such cases, flood protection works can serve as a drain control mechanism. Still this can also be managed by increasing carrying capacity of channels or construction of embankments as in case of headwork structures.

Details of flood protection works have been covered under Guideline of diversion Headwork structure thus can be referred there.

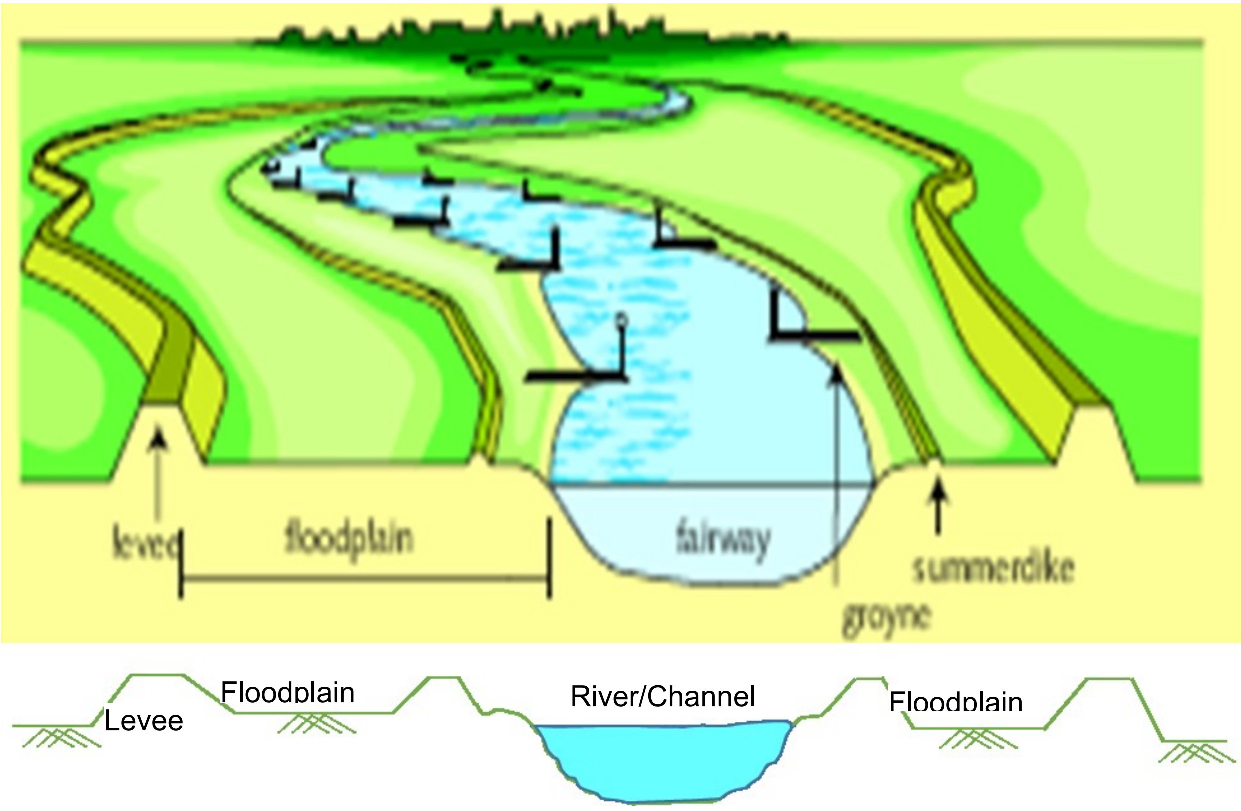


Figure 8-6: Plan and cross section of flood protection arrangements (Typical)

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APPENDICIES

APPENDIX I: Longitudinal Profile of SC2 (Generated data from Melka Lola SSIP)

Par. Dis (m)	Cum. Dis (m)	OGL (m)	Par. Dis (m)	Cum. Dis (m)	OGL (m)	Par. Dis (m)	Cum. Dis (m)	OGL (m)
0	0	1125.131	5	160	1122.993	5	320	1122.166
5	5	1124.995	5	165	1122.943	5	325	1122.141
5	10	1124.854	5	170	1122.912	5	330	1122.121
5	15	1124.708	5	175	1122.885	5	335	1122.101
5	20	1124.607	5	180	1122.86	5	340	1122.082
5	25	1124.529	5	185	1122.837	5	345	1122.063
5	30	1124.447	5	190	1122.814	5	350	1122.048
5	35	1124.367	5	195	1122.792	5	355	1122.034
5	40	1124.308	5	200	1122.77	5	360	1122.021
5	45	1124.251	5	205	1122.747	5	365	1122.007
5	50	1124.194	5	210	1122.723	5	370	1121.985
5	55	1124.137	5	215	1122.7	5	375	1121.965
5	60	1124.079	5	220	1122.677	5	380	1121.944
5	65	1124.022	5	225	1122.655	5	385	1121.925
5	70	1123.965	5	230	1122.63	5	390	1121.904
5	75	1123.908	5	235	1122.603	5	395	1121.883
5	80	1123.851	5	240	1122.577	5	400	1121.863
5	85	1123.794	5	245	1122.552	5	405	1121.842
5	90	1123.735	5	250	1122.526	5	410	1121.827
5	95	1123.678	5	255	1122.501	5	415	1121.812
5	100	1123.621	5	260	1122.475	5	420	1121.796
5	105	1123.564	5	265	1122.449	5	425	1121.781
5	110	1123.506	5	270	1122.423	5	430	1121.765
5	115	1123.453	5	275	1122.398	5	435	1121.749
5	120	1123.4	5	280	1122.372	5	440	1121.734
5	125	1123.349	5	285	1122.347	5	445	1121.718
5	130	1123.298	5	290	1122.32	5	450	1121.703
5	135	1123.247	5	295	1122.295	5	455	1121.688
5	140	1123.196	5	300	1122.269	5	460	1121.672
5	145	1123.145	5	305	1122.244	5	465	1121.656
5	150	1123.093	5	310	1122.218	1.23	466.23	1121.652
5	155	1123.043	5	315	1122.193			

Source: This raw data is generated from DEM which was created from actual surveyed data for use in design of canal profiles indicated in appendix-II.

APPENDIX II: Designed longitudinal profile of LSC2 (Melka Lola SSIP)

Cum. Dis (m)	OGL (m)	b (m)	m	d (m)	S (m/m)	FB (m)	Drop ht. (m)	CBL (m)	FSL (m)	EML (m)	Cut Dep. (m)	Fill Dep. (m)	B (m)	D (m)	T (m)	A_{cut} m2	A_{fill} m2	V_{cut} (m3)	V_{fill} (m3)	Lining thickness, m	Total lining length, L (m)	lining Volume, (m3)	Remark
0	1125.131	0.35	0	0.18	0.0040	0.2		1124.931	1125.111	1125.311	0.50	0.18	0.40	0.38	0.95	0.65	0.32	0.00	0.00	0.30	1.11	0.00	Start of canal
5	1124.995	0.35	0	0.18	0.0040	0.2	1	1124.911	1125.091	1125.291	0.38	0.30	0.40	0.38	0.95	0.50	0.52	2.50	2.59	0.30	1.11	1.67	
5	1124.995	0.35	0	0.18	0.0040	0.20		1123.911	1124.091	1124.291	1.38	0.00	0.40	0.38	0.95	1.80	0.00	0.00	0.00	0.30	1.11	0.00	
10	1124.854	0.35	0	0.18	0.0040	0.2		1123.891	1124.071	1124.271	1.26	0.00	0.40	0.38	0.95	1.64	0.00	8.21	0.00	0.30	1.11	1.67	
15	1124.708	0.35	0	0.18	0.0040	0.2		1123.871	1124.051	1124.251	1.14	0.00	0.40	0.38	0.95	1.48	0.00	7.39	0.0	0.30	1.11	1.67	
20	1124.607	0.35	0	0.18	0.0040	0.2		1123.851	1124.031	1124.231	1.06	0.00	0.40	0.38	0.95	1.37	0.00	6.86	0.0	0.30	1.11	1.67	
25	1124.529	0.35	0	0.18	0.0040	0.2		1123.831	1124.011	1124.211	1.00	0.00	0.40	0.38	0.95	1.30	0.00	6.49	0.0	0.30	1.11	1.67	
30	1124.447	0.35	0	0.18	0.0040	0.2		1123.811	1123.991	1124.191	0.94	0.00	0.40	0.38	0.95	1.22	0.00	6.08	0.0	0.30	1.11	1.67	
35	1124.367	0.35	0	0.18	0.0040	0.2		1123.791	1123.971	1124.171	0.88	0.00	0.40	0.38	0.95	1.14	0.00	5.69	0.0	0.30	1.11	1.67	
40	1124.308	0.35	0	0.18	0.0040	0.2		1123.771	1123.951	1124.151	0.84	0.00	0.40	0.38	0.95	1.09	0.00	5.44	0.0	0.30	1.11	1.67	
45	1124.251	0.35	0	0.18	0.0040	0.2		1123.751	1123.931	1124.131	0.80	0.00	0.40	0.38	0.95	1.04	0.00	5.20	0.0	0.30	1.11	1.67	
50	1124.194	0.35	0	0.18	0.0040	0.2		1123.731	1123.911	1124.111	0.76	0.00	0.40	0.38	0.95	0.99	0.00	4.96	0.0	0.30	1.11	1.67	
55	1124.137	0.35	0	0.18	0.0040	0.2		1123.711	1123.891	1124.091	0.73	0.00	0.40	0.38	0.95	0.94	0.00	4.72	0.0	0.30	1.11	1.67	
60	1124.079	0.35	0	0.18	0.0040	0.2		1123.691	1123.871	1124.071	0.69	0.00	0.40	0.38	0.95	0.89	0.00	4.47	0.0	0.30	1.11	1.67	
65	1124.022	0.35	0	0.18	0.0040	0.2		1123.671	1123.851	1124.051	0.65	0.03	0.40	0.38	0.95	0.85	0.05	4.23	0.3	0.30	1.11	1.67	
70	1123.965	0.35	0	0.18	0.0040	0.2		1123.651	1123.831	1124.031	0.61	0.07	0.40	0.38	0.95	0.80	0.12	3.99	0.6	0.30	1.11	1.67	
75	1123.908	0.35	0	0.18	0.0040	0.2		1123.631	1123.811	1124.011	0.58	0.10	0.40	0.38	0.95	0.75	0.18	3.75	0.9	0.30	1.11	1.67	
80	1123.851	0.35	0	0.18	0.0040	0.2		1123.611	1123.791	1123.991	0.54	0.14	0.40	0.38	0.95	0.70	0.25	3.51	1.2	0.30	1.11	1.67	
85	1123.794	0.35	0	0.18	0.0040	0.2		1123.591	1123.771	1123.971	0.50	0.18	0.40	0.38	0.95	0.65	0.31	3.27	1.5	0.30	1.11	1.67	
90	1123.735	0.35	0	0.18	0.0040	0.2		1123.571	1123.751	1123.951	0.46	0.22	0.40	0.38	0.95	0.60	0.38	3.02	1.9	0.30	1.11	1.67	
95	1123.678	0.35	0	0.18	0.0040	0.2		1123.551	1123.731	1123.931	0.43	0.25	0.40	0.38	0.95	0.56	0.44	2.78	2.2	0.30	1.11	1.67	
100	1123.621	0.35	0	0.18	0.0040	0.2		1123.531	1123.711	1123.911	0.39	0.29	0.40	0.38	0.95	0.51	0.51	2.53	2.5	0.30	1.11	1.67	
105	1123.564	0.35	0	0.18	0.0040	0.2		1123.511	1123.691	1123.891	0.35	0.33	0.40	0.38	0.95	0.46	0.57	2.29	2.9	0.30	1.11	1.67	
110	1123.506	0.35	0	0.18	0.0040	0.2	0.5	1123.491	1123.671	1123.871	0.31	0.37	0.40	0.38	0.95	0.41	0.64	2.05	3.2	0.30	1.11	1.67	
110	1123.506	0.25	0	0.15	0.0045	0.20		1122.991	1123.141	1123.341	0.81	0.00	0.40	0.35	0.85	0.90	0.00	0.00	0.0	0.30	0.95	0.00	
115	1123.453	0.25	0	0.15	0.0045	0.2		1122.969	1123.119	1123.319	0.78	0.00	0.40	0.35	0.85	0.86	0.00	4.31	0.0	0.30	0.95	1.43	
120	1123.40	0.25	0	0.15	0.0045	0.2		1122.946	1123.096	1123.296	0.75	0.00	0.40	0.35	0.85	0.83	0.00	4.15	0.0	0.30	0.95	1.43	
125	1123.349	0.25	0	0.15	0.0045	0.2		1122.924	1123.074	1123.274	0.73	0.00	0.40	0.35	0.85	0.80	0.00	3.99	0.0	0.30	0.95	1.43	
130	1123.298	0.25	0	0.15	0.0045	0.2		1122.901	1123.051	1123.251	0.70	0.00	0.40	0.35	0.85	0.77	0.00	3.83	0.0	0.30	0.95	1.43	
135	1123.247	0.25	0	0.15	0.0045	0.2		1122.879	1123.029	1123.229	0.67	0.00	0.40	0.35	0.85	0.74	0.00	3.68	0.0	0.30	0.95	1.43	
140	1123.196	0.25	0	0.15	0.0045	0.2		1122.856	1123.006	1123.206	0.64	0.01	0.40	0.35	0.85	0.70	0.02	3.52	0.1	0.30	0.95	1.43	
145	1123.145	0.25	0	0.15	0.0045	0.2		1122.834	1122.984	1123.184	0.61	0.04	0.40	0.35	0.85	0.67	0.06	3.36	0.3	0.30	0.95	1.43	
150	1123.093	0.25	0	0.15	0.0045	0.2		1122.811	1122.961	1123.161	0.58	0.07	0.40	0.35	0.85	0.64	0.11	3.20	0.6	0.30	0.95	1.43	

Cum. Dis (m)	OGL (m)	b (m)	m	d (m)	S (m/m)	FB (m)	Drop ht. (m)	CBL (m)	FSL (m)	EML (m)	Cut Dep. (m)	Fill Dep. (m)	B (m)	D (m)	T (m)	A_{cut} m2	A_{fill} m2	V_{cut} (m3)	V_{fill} (m3)	Lining thickness, m	Total lining length, L (m)	lining Volume, (m3)	Remark
155	1123.043	0.25	0	0.15	0.0045	0.2		1122.789	1122.939	1123.139	0.55	0.10	0.40	0.35	0.85	0.61	0.16	3.05	0.8	0.30	0.95	1.43	
160	1122.993	0.25	0	0.15	0.0045	0.2		1122.766	1122.916	1123.116	0.53	0.12	0.40	0.35	0.85	0.58	0.20	2.90	1.0	0.30	0.95	1.43	
165	1122.943	0.25	0	0.15	0.0045	0.2		1122.744	1122.894	1123.094	0.50	0.15	0.40	0.35	0.85	0.55	0.25	2.75	1.2	0.30	0.95	1.43	
170	1122.912	0.25	0	0.15	0.0045	0.2		1122.721	1122.871	1123.071	0.49	0.16	0.40	0.35	0.85	0.54	0.26	2.70	1.3	0.30	0.95	1.43	
175	1122.885	0.25	0	0.15	0.0045	0.2		1122.699	1122.849	1123.049	0.49	0.16	0.40	0.35	0.85	0.54	0.27	2.68	1.3	0.30	0.95	1.43	
180	1122.86	0.25	0	0.15	0.0045	0.2		1122.676	1122.826	1123.026	0.48	0.17	0.40	0.35	0.85	0.53	0.27	2.66	1.4	0.30	0.95	1.43	
185	1122.837	0.25	0	0.15	0.0045	0.2		1122.654	1122.804	1123.004	0.48	0.17	0.40	0.35	0.85	0.53	0.27	2.66	1.4	0.30	0.95	1.43	
190	1122.814	0.25	0	0.15	0.0045	0.2		1122.631	1122.781	1122.981	0.48	0.17	0.40	0.35	0.85	0.53	0.28	2.66	1.4	0.30	0.95	1.43	
195	1122.792	0.25	0	0.15	0.0045	0.2		1122.609	1122.759	1122.959	0.48	0.17	0.40	0.35	0.85	0.53	0.27	2.66	1.4	0.30	0.95	1.43	
200	1122.77	0.25	0	0.15	0.0045	0.2		1122.586	1122.736	1122.936	0.48	0.17	0.40	0.35	0.85	0.53	0.27	2.66	1.4	0.30	0.95	1.43	
205	1122.747	0.25	0	0.15	0.0045	0.2		1122.564	1122.714	1122.914	0.48	0.17	0.40	0.35	0.85	0.53	0.27	2.66	1.4	0.30	0.95	1.43	
210	1122.723	0.25	0	0.15	0.0045	0.2		1122.541	1122.691	1122.891	0.48	0.17	0.40	0.35	0.85	0.53	0.28	2.65	1.4	0.30	0.95	1.43	
215	1122.7	0.25	0	0.15	0.0045	0.2		1122.519	1122.669	1122.869	0.48	0.17	0.40	0.35	0.85	0.53	0.28	2.65	1.4	0.30	0.95	1.43	
220	1122.677	0.25	0	0.15	0.0045	0.2		1122.496	1122.646	1122.846	0.48	0.17	0.40	0.35	0.85	0.53	0.28	2.65	1.4	0.30	0.95	1.43	
225	1122.655	0.25	0	0.15	0.0045	0.2		1122.474	1122.624	1122.824	0.48	0.17	0.40	0.35	0.85	0.53	0.28	2.65	1.4	0.30	0.95	1.43	
230	1122.63	0.25	0	0.15	0.0045	0.2		1122.451	1122.601	1122.801	0.48	0.17	0.40	0.35	0.85	0.53	0.28	2.63	1.4	0.30	0.95	1.43	
235	1122.603	0.25	0	0.15	0.0045	0.2		1122.429	1122.579	1122.779	0.47	0.18	0.40	0.35	0.85	0.52	0.29	2.61	1.4	0.30	0.95	1.43	
240	1122.577	0.25	0	0.15	0.0045	0.2		1122.406	1122.556	1122.756	0.47	0.18	0.40	0.35	0.85	0.52	0.30	2.59	1.5	0.30	0.95	1.43	
245	1122.552	0.25	0	0.15	0.0045	0.2		1122.384	1122.534	1122.734	0.47	0.18	0.40	0.35	0.85	0.52	0.30	2.58	1.5	0.30	0.95	1.43	
250	1122.526	0.25	0	0.15	0.0045	0.2		1122.361	1122.511	1122.711	0.47	0.18	0.40	0.35	0.85	0.51	0.31	2.56	1.5	0.30	0.95	1.43	
255	1122.501	0.25	0	0.15	0.0045	0.2		1122.339	1122.489	1122.689	0.46	0.19	0.40	0.35	0.85	0.51	0.31	2.54	1.5	0.30	0.95	1.43	
260	1122.475	0.25	0	0.15	0.0045	0.2		1122.316	1122.466	1122.666	0.46	0.19	0.40	0.35	0.85	0.50	0.32	2.52	1.6	0.30	0.95	1.43	
265	1122.449	0.25	0	0.15	0.0045	0.2		1122.294	1122.444	1122.644	0.46	0.19	0.40	0.35	0.85	0.50	0.32	2.51	1.6	0.30	0.95	1.43	
270	1122.423	0.25	0	0.15	0.0045	0.2		1122.271	1122.421	1122.621	0.45	0.20	0.40	0.35	0.85	0.50	0.33	2.49	1.6	0.30	0.95	1.43	
275	1122.398	0.25	0	0.15	0.0045	0.2		1122.249	1122.399	1122.599	0.45	0.20	0.40	0.35	0.85	0.49	0.33	2.47	1.7	0.30	0.95	1.43	
280	1122.372	0.25	0	0.15	0.0045	0.2		1122.226	1122.376	1122.576	0.45	0.20	0.40	0.35	0.85	0.49	0.34	2.45	1.7	0.30	0.95	1.43	
285	1122.347	0.25	0	0.15	0.0045	0.2		1122.204	1122.354	1122.554	0.44	0.21	0.40	0.35	0.85	0.49	0.34	2.44	1.7	0.30	0.95	1.43	
290	1122.32	0.25	0	0.15	0.0045	0.2		1122.181	1122.331	1122.531	0.44	0.21	0.40	0.35	0.85	0.48	0.35	2.41	1.7	0.30	0.95	1.43	
295	1122.295	0.25	0	0.15	0.0045	0.2		1122.159	1122.309	1122.509	0.44	0.21	0.40	0.35	0.85	0.48	0.35	2.40	1.8	0.30	0.95	1.43	
300	1122.269	0.25	0	0.15	0.0045	0.2		1122.136	1122.286	1122.486	0.43	0.22	0.40	0.35	0.85	0.48	0.36	2.38	1.8	0.30	0.95	1.43	
305	1122.244	0.25	0	0.15	0.0045	0.2		1122.114	1122.264	1122.464	0.43	0.22	0.40	0.35	0.85	0.47	0.36	2.37	1.8	0.30	0.95	1.43	
310	1122.218	0.25	0	0.15	0.0045	0.2		1122.091	1122.241	1122.441	0.43	0.22	0.40	0.35	0.85	0.47	0.37	2.35	1.8	0.30	0.95	1.43	
315	1122.193	0.25	0	0.15	0.0045	0.2		1122.069	1122.219	1122.419	0.42	0.23	0.40	0.35	0.85	0.47	0.37	2.33	1.9	0.30	0.95	1.43	
320	1122.166	0.25	0	0.15	0.0045	0.2		1122.046	1122.196	1122.396	0.42	0.23	0.40	0.35	0.85	0.46	0.38	2.31	1.9	0.30	0.95	1.43	

Cum. Dis (m)	OGL (m)	b (m)	m	d (m)	S (m/m)	FB (m)	Drop ht. (m)	CBL (m)	FSL (m)	EML (m)	Cut Dep. (m)	Fill Dep. (m)	B (m)	D (m)	T (m)	A_{cut} m2	A_{fill} m2	V_{cut} (m3)	V_{fill} (m3)	Lining thickness, m	Total lining length, L (m)	lining Volume, (m3)	Remark
325	1122.141	0.25	0	0.15	0.0045	0.2		1122.024	1122.174	1122.374	0.42	0.23	0.40	0.35	0.85	0.46	0.38	2.30	1.9	0.30	0.95	1.43	
330	1122.121	0.25	0	0.15	0.0045	0.2		1122.001	1122.151	1122.351	0.42	0.23	0.40	0.35	0.85	0.46	0.38	2.31	1.9	0.30	0.95	1.43	
335	1122.101	0.25	0	0.15	0.0045	0.2		1121.979	1122.129	1122.329	0.42	0.23	0.40	0.35	0.85	0.46	0.38	2.32	1.9	0.30	0.95	1.43	
340	1122.082	0.25	0	0.15	0.0045	0.2		1121.956	1122.106	1122.306	0.43	0.22	0.40	0.35	0.85	0.47	0.37	2.34	1.8	0.30	0.95	1.43	
345	1122.063	0.25	0	0.15	0.0045	0.2		1121.934	1122.084	1122.284	0.43	0.22	0.40	0.35	0.85	0.47	0.36	2.36	1.8	0.30	0.95	1.43	
350	1122.048	0.25	0	0.15	0.0045	0.2		1121.911	1122.061	1122.261	0.44	0.21	0.40	0.35	0.85	0.48	0.35	2.40	1.8	0.30	0.95	1.43	
355	1122.034	0.25	0	0.15	0.0045	0.2		1121.889	1122.039	1122.239	0.45	0.20	0.40	0.35	0.85	0.49	0.34	2.45	1.7	0.30	0.95	1.43	
360	1122.021	0.25	0	0.15	0.0045	0.2		1121.866	1122.016	1122.216	0.46	0.19	0.40	0.35	0.85	0.50	0.32	2.50	1.6	0.30	0.95	1.43	
365	1122.007	0.25	0	0.15	0.0045	0.2		1121.844	1121.994	1122.194	0.46	0.19	0.40	0.35	0.85	0.51	0.31	2.55	1.5	0.30	0.95	1.43	
370	1121.985	0.25	0	0.15	0.0045	0.2		1121.821	1121.971	1122.171	0.46	0.19	0.40	0.35	0.85	0.51	0.31	2.55	1.5	0.30	0.95	1.43	
375	1121.965	0.25	0	0.15	0.0045	0.2		1121.799	1121.949	1122.149	0.47	0.18	0.40	0.35	0.85	0.51	0.30	2.57	1.5	0.30	0.95	1.43	
380	1121.944	0.25	0	0.15	0.0045	0.2		1121.776	1121.926	1122.126	0.47	0.18	0.40	0.35	0.85	0.51	0.30	2.57	1.5	0.30	0.95	1.43	
385	1121.925	0.25	0	0.15	0.0045	0.2		1121.754	1121.904	1122.104	0.47	0.18	0.40	0.35	0.85	0.52	0.29	2.59	1.5	0.30	0.95	1.43	
390	1121.904	0.25	0	0.15	0.0045	0.2		1121.731	1121.881	1122.081	0.47	0.18	0.40	0.35	0.85	0.52	0.29	2.60	1.5	0.30	0.95	1.43	
395	1121.883	0.25	0	0.15	0.0045	0.2		1121.709	1121.859	1122.059	0.47	0.18	0.40	0.35	0.85	0.52	0.29	2.61	1.4	0.30	0.95	1.43	
400	1121.863	0.25	0	0.15	0.0045	0.2		1121.686	1121.836	1122.036	0.48	0.17	0.40	0.35	0.85	0.52	0.29	2.62	1.4	0.30	0.95	1.43	
405	1121.842	0.25	0	0.15	0.0045	0.2		1121.664	1121.814	1122.014	0.48	0.17	0.40	0.35	0.85	0.53	0.28	2.63	1.4	0.30	0.95	1.43	
410	1121.827	0.25	0	0.15	0.0045	0.2		1121.641	1121.791	1121.991	0.49	0.16	0.40	0.35	0.85	0.53	0.27	2.67	1.4	0.30	0.95	1.43	
415	1121.812	0.25	0	0.15	0.0045	0.2		1121.619	1121.769	1121.969	0.49	0.16	0.40	0.35	0.85	0.54	0.26	2.71	1.3	0.30	0.95	1.43	
420	1121.796	0.25	0	0.15	0.0045	0.2		1121.596	1121.746	1121.946	0.50	0.15	0.40	0.35	0.85	0.55	0.25	2.75	1.2	0.30	0.95	1.43	
425	1121.781	0.25	0	0.15	0.0045	0.2		1121.574	1121.724	1121.924	0.51	0.14	0.40	0.35	0.85	0.56	0.24	2.79	1.2	0.30	0.95	1.43	
430	1121.765	0.25	0	0.15	0.0045	0.2		1121.551	1121.701	1121.901	0.51	0.14	0.40	0.35	0.85	0.57	0.22	2.83	1.1	0.30	0.95	1.43	
435	1121.749	0.25	0	0.15	0.0045	0.2		1121.529	1121.679	1121.879	0.52	0.13	0.40	0.35	0.85	0.57	0.21	2.86	1.1	0.30	0.95	1.43	
440	1121.734	0.25	0	0.15	0.0045	0.2		1121.506	1121.656	1121.856	0.53	0.12	0.40	0.35	0.85	0.58	0.20	2.90	1.0	0.30	0.95	1.43	
445	1121.718	0.25	0	0.15	0.0045	0.2		1121.484	1121.634	1121.834	0.53	0.12	0.40	0.35	0.85	0.59	0.19	2.94	1.0	0.30	0.95	1.43	
450	1121.703	0.25	0	0.15	0.0045	0.2		1121.461	1121.611	1121.811	0.54	0.11	0.40	0.35	0.85	0.60	0.18	2.98	0.9	0.30	0.95	1.43	
455	1121.688	0.25	0	0.15	0.0045	0.2		1121.439	1121.589	1121.789	0.55	0.10	0.40	0.35	0.85	0.60	0.17	3.02	0.8	0.30	0.95	1.43	
460	1121.672	0.25	0	0.15	0.0045	0.2		1121.416	1121.566	1121.766	0.56	0.09	0.40	0.35	0.85	0.61	0.16	3.06	0.8	0.30	0.95	1.43	
465	1121.656	0.25	0	0.15	0.0045	0.2		1121.394	1121.544	1121.744	0.56	0.09	0.40	0.35	0.85	0.62	0.14	3.09	0.7	0.30	0.95	1.43	
466.2	1121.652	0.25	0	0.15	0.0045	0.2		1121.388	1121.538	1121.738	0.56	0.09	0.40	0.35	0.85	0.62	0.14	0.76	0.2	0.30	0.95	0.35	End of canal
																	Total	295	112			138	

Source: As computed by excel, 2016

APPENDIX III: Crop response factors where yield reduction is proportionally < relative evapotranspiration deficit

Crop	Specific growth stage	ky	Irrigation method	Reference
Common bean	Vegetative;	0.57	Furrow	Calvache and Reichardt (1999)
	Yield formation	0.87		
	Whole season	0.99	Sprinkler	
Cotton	Flowering & yield formation	0.99	Sprinkler	Bastug (1987)
	Whole season	0.86	Drip	Yavuz (1993)
	Bud formation;	0.75	Check	Prieto and Angueira (1999)
	Flowering	0.48	Furrow	
	Boll formation;	0.46	Furrow	Anac et al. (1999)
	Flowering;	0.67		
	Vegetation	0.88		
Groundnut	Flowering	0.74	Furrow	Ahmad (1999)
Maize	Whole season	0.74	Sprinkler	Craciun and Craciun (1999)
Soybean	Vegetative	0.58	Furrow	Kirda et al. (1999a)
Sunflower	Whole season	0.91	Furrow	Karaata (1991)
	Vegetative & yielding	0.83	Furrow	
Sugar beet	Whole season;	0.86	Furrow	Bazza and Tayaa(1999)
	Yield formation and ripening;	0.74	Furrow	
	Vegetative and yield formation	0.64		
Sugar cane	Tillering	0.40	Furrow	Pene and Edi (1999)
Potato	Vegetative;	0.40	Furrow	Iqbal et al. (1999)
	Flowering;	0.33		
	Tuber formation	0.46		
	Whole season	0.83	Drip	Kovacs et al. (1999)
Wheat	Whole season;	0.76	Sprinkler	Kovacs et al. (1999)
	Whole season;	0.93	Basin	Madanoglu (1977)
	Flowering and grain filling	0.39	Basin	Waheed et al. (1999)

Source: Water Reports #22; Deficit Irrigation Practices, FAO, 2002

APPENDIX IV: Calculation procedures for response to water stress yield

1. Determine maximum yield (Ym) of adapted crop variety, dictated by climate, assuming other growth factors (e. g. water, fertilizer, pests and diseases) are not limiting.
2. Calculate maximum evapotranspiration (ETm) when crop water requirements .are fully met by available water supply.
3. Determine actual crop evapotranspiration (ETa) based on factors concerned with available water supply to the crop.
4. Evaluate factors concerned with the interaction between water supply, crop water requirement$ and actual yield (Ya); through:
5. Selection of yield response factor (ky) to evaluate relative yield decrease as related to relative evapotranspiration deficit, or (1 - Ya/Ym) = ky (I - ETa/ETm), and obtain actual yield (Ya).

APPENDIX V: Standard drainage coefficients for agricultural areas

Soil conditions	Water management	Drainage coefficient (mm/day)
Less pervious soils	Internal drainage restricted	<1.5
Pervious soils	According to the internal drainage and crop intensity	1.5-3.0
Pervious soils	Poor irrigation or leaching for salinity control	3.0-4.5
Very pervious soil	Irrigation of paddy fields	>4.5

Source: Adopted from FAO Irrigation and Drainage paper no. 38 (1980)
Note: These data shall be adopted only where there is no sufficient meteorology data for estimation of drainage module.

APPENDIX VI: Templates for Design of Furrow, Border & Basin Irrigation Application
(This have been separately attached)

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Prepared by

GIRDC