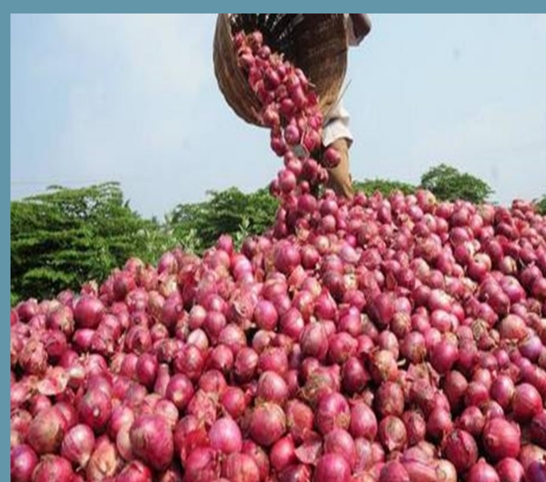
SSIGL 16

NATIONAL GUIDELINES

For Small Scale Irrigation Development in Ethiopia

Canals Related Structures Design



November 2018

Addis Ababa

MINISTRY OF AGRICULTURE

National Guidelines for Small Scale Irrigation Development in Ethiopia

SSIGL 16: Canals Related Structures Design

November 2018
Addis Ababa

National Guidelines for Small Scale Irrigation Development in Ethiopia

First Edition 2018

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

Financed by Agricultural Growth Program (AGP)

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.

LIST OF GUIDELINES

TABLE OF CONTENTS

LIST OF APPENDICES

LIST OF TABLES

LIST OF FIGURES

ANNOTATIONS

Q	Rate of Flow
A	Flow cross Sectional Area
P	Wetted Perimeter
R	Hydraulics Radius, A/P
n	Manning's Roughness Coefficient
v	Velocity of Flow
V	Volume of Water
q	Unit discharge, discharge per unit width
y_1	Depth of flow before jump
y_2	Depth of flow after jump
H_L	Head Loss between upstream and downstream reaches
g	Gravitational Acceleration, 9.81m/sec
L	Jump Length at Stilling Basin
b	Concrete member width
D	Concrete member Depth
d	Concrete member Effective Depth
c	Concrete Clear Cover
M	Moment in a structural member
Fcc	Resultant of Compression Force at compressed section of concrete member
F_{st}	Resultant of Tension Force at tension in the reinforcement bars
z	Moment arm in concrete section in bending
f_{ck}	Characteristics Compressive Strength of Concrete
f_{yk}	Characteristics Tensile Strength of Reinforcement bars
M_{bal}	Balanced Moment
v_c	Shear Resistance Capacity of Concrete
A_s	Area of Reinforcement Steel
γ_m	Load Partial Factor of Safety
Ø	Angle of friction of the soil
K_a	Active earth pressure coefficient
e	Eccentricity
F_r	Froude Number
γ_s	Unit Weight of Soil
LL	Live Load
DL	Dead Load
T.E	Total Energy
hfi	Inlet Loss
hf	Friction Loss
hfo	Outlet Loss
u/s	Upstream
d/s	Downstream
CBL	Canal bed level

ACRONYMS

AGP	Agricultural Growth Program
ERA	Ethiopian Road Authority
FAO	Food and Agricultural Organization
GIRDC	Generation Integrated Rural Development Consultant
JICA	Japan International Cooperation Agency
MoANR	Ministry of Agriculture and Natural Resource
MoWIE	Ministry of Water Irrigation and Electricity
OIDA	Oromia Irrigation Development Authority
OIDA	Oromia Irrigation Development Authority
SSID	Small Scale Irrigation Development
SSIGL	Small Scale Irrigation Guideline
SSIP	Small Scale Irrigation Project
SSIS	Small Scale Irrigation Scheme
USBR	United States Bureau of Reclamation

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 GENERAL

To accelerate irrigation development and at the same time increasing the effectiveness and efficiency of Small Scale Irrigation Scheme (SSIS) development, it is important to improve the subject matter knowledge and design capacity of the sector. Providing the sector with updated technical guidelines as well as with other inputs is seen as an important contribution to this objective.

1.2 OBJECTIVE AND SCOPE OF THE GUIDELINE

The objective of this guideline, Guideline for Design of Canal Related Structures for Small Scale Irrigation, is aim at to provide well organized and comprehensive user-friendly manual tailored with the current design practice exercised in the country.

As the purpose of the manual is to aid small-scale irrigation system design, the scope is also limited to the design of canal related structures for Small Scale Irrigation Schemes. The guideline is prepared to assist practice engineers to get easily in to the hydraulic and structural design aspects with the aid of basic concept demonstration, worked example and supplemented with design aid templates.

1.3 STRUCTURE OF THE GUIDELINE

The manual is divided in to five chapters, the first being this introductory part, the second chapter discussed the type and classification of canal structures, third chapter deal with the brief account of basic design consideration for the hydraulic and structural design of canal related structures. The fourth chapter is the main body of the manual, which incorporates the design of small canal related structures usually encountered especially in the context of our local current irrigation schemes deign. The structures categorized as conveyance, regulating, protective, energy dissipation and measuring structures. Worked examples are also incorporated for each of the structures type mentioned above. The fifth chapter contains Social Infrastructures like Campsite, Access road, Foot Bridge, Wash Basin and Cattle Trough. To facilitate the hydraulic and structural design, excel templates are also incorporated as an Appendices.

2 IRRIGATION STRUCTURES

Irrigation structures can be classified in to various forms depending on their purpose. According to USBR (1978) irrigation structures are classified in to five groups namely conveyance, regulating, protective, measurement and energy dissipation structures. The structures belongs to each group is presented in Table 2.1 below.

According to Ankum (1992) irrigation structures can also be classified in to three types; non-regulating type structure, regulating type structure and measuring structure. The conveyance and some protective structures in USBR (1978) are categorized as non-regulating according to Ankum (1992). On the other hand the energy dissipation structure is considered as an integral part of either non-regulating, regulating or measuring structure and hence ignored in the category. Due to its simplicity and unambiguousness, this classification is used in this training manual.

Table 2-1: Canal Structures Classification Table

Conveyance Structure	***Protective Structure***
• Siphon • Bench flume and elevated flume (aqueduct) • Drop • Chute	• Spillway • Side scape (escape) • Cross drainage (culvert, over chute)
	Energy Dissipation Structure • Baffle apron • Stilling basin, stilling well
Regulating Structure • Checks • Turnouts • Division structures	***Measurement Structure*** • Parshal flume • Weir • Open flow meter

2.1 NON-REGULATING STRUCTURE

Non-regulating structures are structures that transport the approaching flow over/ under an obstruction or when the design is interfered with the natural topography. They do not have the primary function of regulating the water level or the discharge. Non-regulating structures can be divided into two different groups depending on the head loss; structures with sub-critical flow, i.e. *conveyance structures*; and structures with super-critical flow, i.e. *drop structures*.

2.1.1 Conveyance structures

Conveyance structures are required at crossings of irrigation canals with roads, drainage channels and rivers. These structures are designed at a low head loss for all discharges. An example of such structure includes *culverts, aqueducts and (inverted) siphons.* But also, 'gated regulators' with low head loss are designed as 'Conveyance structures' during maximum discharge.

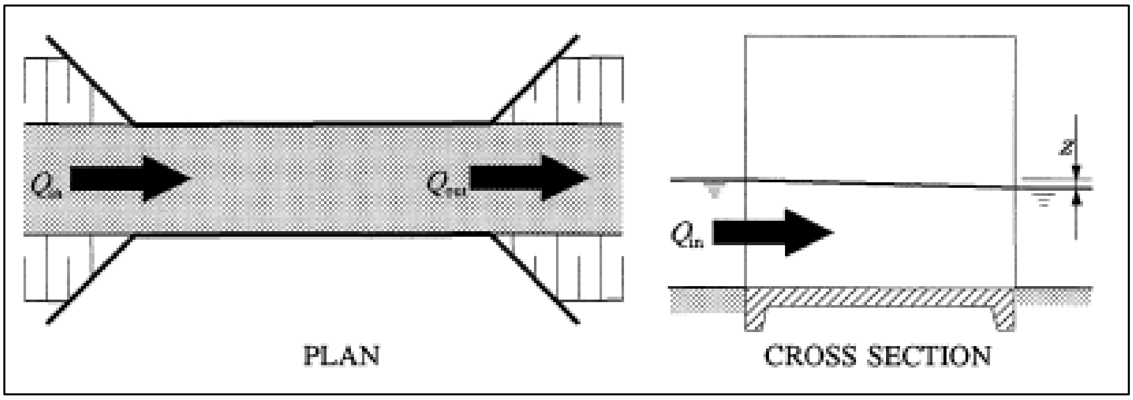


Figure 2-1: Conveyance structure with low head

2.1.2 Culverts

Culvert is a structure that conveys a drainage channel underneath a road or another watercourse. There are several possible flow conditions through a culvert, depending on the discharge, the cross section, the length of the culvert and the downstream flow conditions. Furthermore, the water in the culvert may flow as *free flow* i.e. called inlet control (requirement of large head losses, which is in contradictious to the low head loss requirement for a conveyance structure) or as *submerged flow* i.e. called tail water control. The design of culverts is essentially based on the head loss calculation as presented below that can be applied for tail water control. The velocity in the wet cross-sectional area of the culvert may range from 0.5 to 2.0 m/s and determines the head loss over the structure. The choice of a single or multi-barrel culvert depends on the bed width of the canal, the water depth of the canal invert and the available head loss for the culvert.

2.1.3 Aqueducts

Aqueducts are used for passing irrigation canal over gully, drainage or river course. The greatest part of the live load is the water in the aqueduct. Thus, the velocity in the wet cross-sectional area of the aqueduct should be kept high to minimize weight. On the other hand, the velocity should be kept as low as to minimize the head loss. The velocity should be at least twice the canal velocity to discourage deposition of sediment, but generally should not exceed 3.0 m/s. The box section of the aqueduct acts as the spanning beam.

2.1.4 Siphons

Siphon, also called inverted siphon, is designed at a low head loss. Siphons are mainly used in irrigation canals for river crossings where an aqueduct is unsuitable. The decision whether to use an aqueduct or a siphon in an irrigation canal depends on a number of factors, the main one being sediment transport. The siphon will trap all sediment at low flow. The reason is that the wet cross-sectional area in the canal will normally decrease during the lower flows, but the wet cross-sectional area in the siphon remains the same. Thus, the velocity in the siphon will become much lower than in the canal. Special actions may be required, such as wash-out facilities, or simply gates to close one of the barrels during low discharges. To prevent siltation and blockage, the velocity in the siphon should be kept high. However, a high velocity also leads to higher head loss. Normally, the velocity in the siphon should be at least twice the normal canal velocity and in any case do not less than 1.5 m/s. Maximum velocity in siphon should not exceed 3 m/s. The top of the siphon entrance is set below the normal water surface. This will minimize possible reduction in the siphon capacity caused by the entering of air into the siphon. The depth of submergence is known as the water seal. The required height of the seal depends on the slope and the size of the barrel, and can be taken at 1.5 times of the entrance loss, with a minimum of 0.15 m. Trash racks have to be installed at the entrance of siphons to avoid clogging by floating debris. However, the blocking of the trash rack will increase the entrance head loss. The inverted siphon will be under internal pressure and hence joints between the pipe elements should not be used. Instead a reinforced concrete collar may be used, preferably cast in-situ.

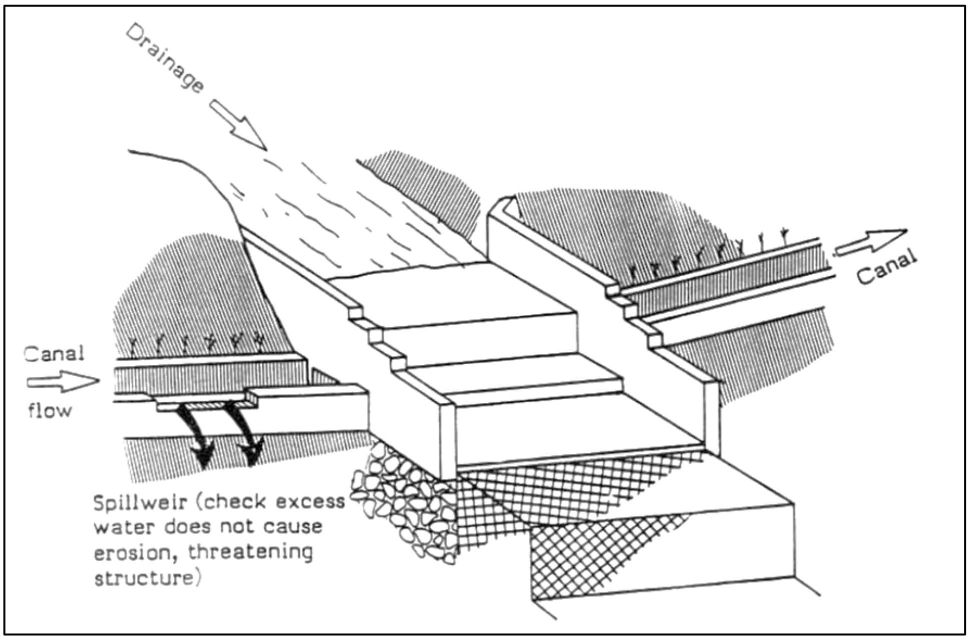


Figure 2-2: Canal siphon under a natural drainage

2.1.5 Drop or fall

Drop structures are required to dissipate the excessive energy at steep alignments to avoid erosion in unlined open-channels i.e. when the gradient of the terrain is steeper than the maximum permissible slope of the canal.

These structures are designed at a high head loss for all discharges. In broad, there are three types of drop structure; chute or flume, inclined drop structure (open channel or pipe drop), vertical drop structure.

Inclined drops are used where the drop in water surface is greater than that allowed in a vertical drop and where the distance between the sections having tranquil flow (sub-critical flow) is not limited.

A) Vertical drop

A vertical drop structure has a vertical wall between the control and the stilling basin. The small portion of energy dissipation occurs by impact of the jet on the floor and the major portion of energy dissipation occurs by turbulence in the stilling basin.

Common vertical drop structure characterized by aerated free-falling nappe (modular flow) hits the downstream basin floor, and with turbulent circulation in the pool beneath the nappe contributing to energy dissipation (shown in Figure 2.3 below).

A vertical drop causes an abrupt change in canal bottom level. For unlined canals, the water level drop should not exceed 1m. However, for lined canals, the drop height can be up to 2.5m. Drop height greater than 2.5m is not recommended as thick floor is required to counter balance the uplift pressure.

The choice of vertical drops depends up on the change in ground surface level, the need to have tranquil flow in the canal sections and drop in water surfaces by several meters.

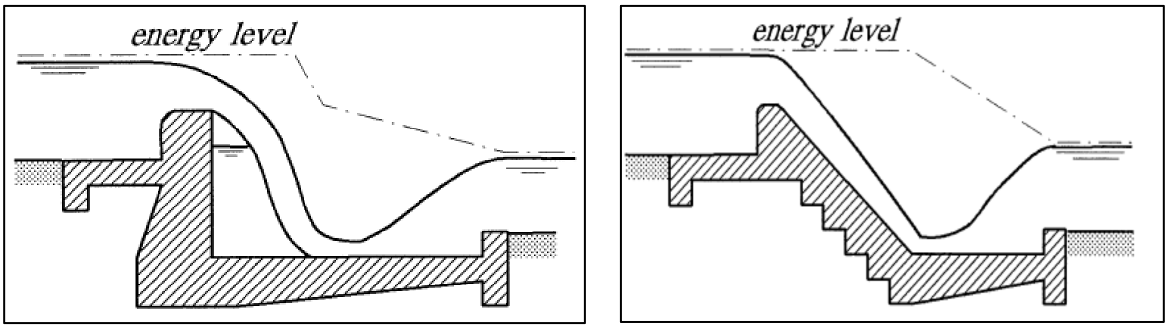


Figure 2-3: Vertical and inclined drop

B) Inclined drop

An inclined drop structure has a sloping glacis between the control and the stilling basin. The energy loss in the sloping glacis due to friction is small and is ignored. Therefore, the energy is dissipated in the stilling basin only. Inclined drops are used where the drop in water surface is greater than that allowed in a vertical drop and where the distance between the sections having tranquil flow is not limited.

The slope of the glacis can be as steep as 1V:0.5H but usually 1V:1H or 1V:2H is provided.

C) Chute Drop

A chute drops dissipates the major portion of the energy by friction over sloping glacis and the remainder portion by turbulence in the stilling basin. Chutes are normally provided when the drop in elevation is greater than 5 m and the water is conveyed over a very long distance. Chute drops has a very flat sloping downstream face compared to other drop but yet steep enough to maintain super- critical flow. Figure 2.4 below shows typical arrangement of rectangular chute plan and longitudinal section.

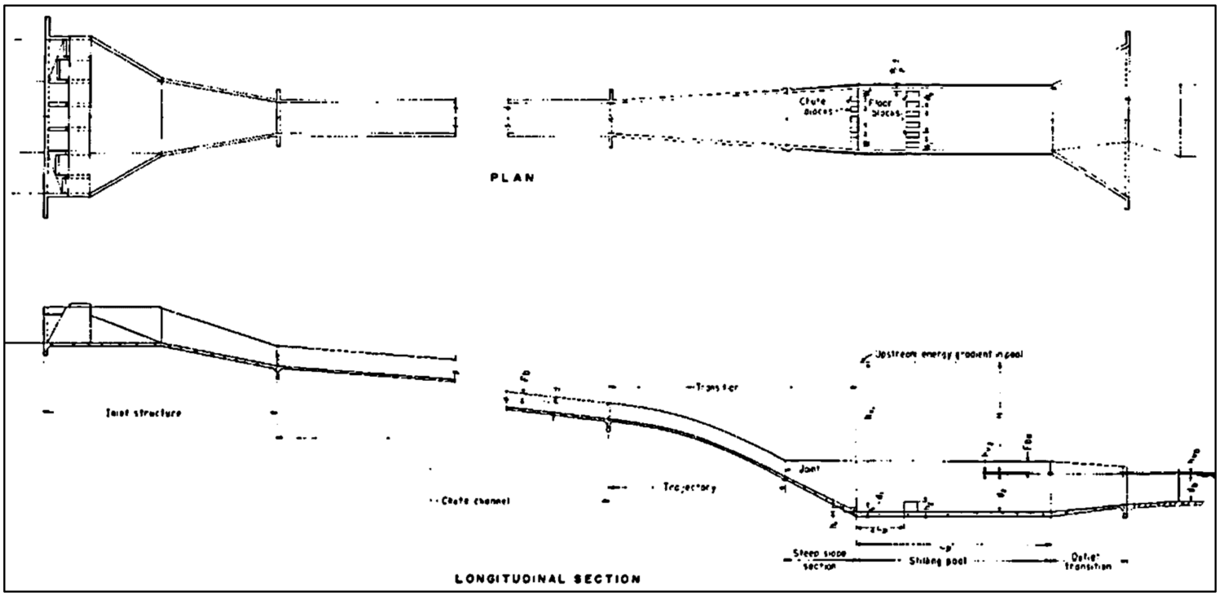


Figure 2-4: Typical Rectangular Chute Plan and Longitudinal Section

2.2 REGULATING STRUCTURE

Based on their function a Regulating structures can be classified as discharge regulators, or discharge regulators with measurement provisions, or water level regulators.

Discharge regulator structure is required at all canals intake and bifurcation points in order to regulate and control the discharge and levels. For instance, the structure to the off taking canal is a discharge regulator (usually called head regulator), and the structure in the continuing canal is a water level regulator (usually called cross regulator).

The function of water level regulators or cross regulator is to maintain a certain target water level, i.e. at set point for every discharge.

A special type of discharge regulator structures is passive regulator for dividing the flow from a supply channel among two or more channels. These structures are called flow divisors and can be sub-divided by the nature of proportional flow division as Adjustable and Fixed Proportion division structures.

Adjustable discharge regulator structures are those, which do not provide a strictly proportional division. The category includes mainly division or diversion boxes which have the additional function of alternating the flow between different offtake channels, using slide gates or flash boards.

Fixed proportion discharge regulator structures are those which are designed to give strictly accurate proportions, (using a control section across the supply channel - causing shooting flow or free fall; A significant feature of the former category is that the flow is divided by thin plated (splitting) walls.

The division structure may be a separate structure or it may be combined with outlet of tertiary canal, a drop structure, or a field canal/turnout from which further diversion is required. If measurement is not required at the point of division, the flow through the structure may be directed through the various outlets with vertical sliding gates or stop logs. If the flow must be measured regulating structure for discharge and water level may be provided.

Discharge regulation can be done by either an *on/off flow* or by an *adjustable flow*. The tertiary offtakes on the secondary canal has to supply the target discharge to the tertiary units. Similarly, the secondary offtakes on the primary canal has to supply the target discharge to the secondary unit.

The vertical gate also called vertical slide gate or slide gate is the most common used with regulating structures. It consists of a simple rectangular gate-leaf that is moved up- and downwards in grooves. Vertical gates are used only for the smaller openings to a maximum of 3 m height, as the hoisting device becomes otherwise too heavy to overcome the friction in the grooves.

2.3 MEASURING STRUCTURE

Discharge measuring structures are required in irrigation systems when an adjustable flow has to be regulated, while the discharge regulator has no provisions for discharge measurement. Normally, they are operated at a high head at all discharges. In fact, these structures belong to non- regulating structures with super-critical flow.

The most commonly used discharge measuring structures are rectangular broad crested weir, sharp crested weir and partial flume. Detail description and design for flow measuring structures are included in section 4.5 of this guideline.

3 DESIGN OF CANAL STRUCTURES FOR SMALL SCALE IRRIGATION PROJECTS

3.1 HYDRAULIC DESIGN CONSIDERATIONS

Predominantly open channel hydraulics will play the major role in the design of canal related structures but of course pressure flow hydraulics is also utilized in some structures, for instance for the design of inverted siphons and pressure flow pipes. The basic hydraulics principles utilized as a tool for the design of hydraulics of canal structures are the continuity, the energy and Momentum principles. A brief description of the basic hydraulic principles and important hydraulic formulae derived from the basic principle of open canal flow hydraulics is presented in the sections below.

3.1.1 The continuity principle

The rate of flow in any section of a channel is equal provided that there is no loss of water between the sections in consideration. Mathematically it is expressed as in equation 3.1 below.

$$Q = A_1 v_1 = A_2 v_2 \quad \text{.......} 3.1$$

Where,

Q = Rate of Flow

A_1 = Flow area at Section-1

v_1 = Flow Mean Velocity at Section-1

A_2 = Flow area at Section-2

v_2 = Flow Mean Velocity at Section-2

3.1.2 The energy principle

In elementary hydraulics the total energy, given in meter-newton per newton of water in any streamline passing through a channel section, may be expressed as the total head in meter of water, which is equal to the sum of the elevations above a datum, the pressure head, and the velocity head. For example, with respect to the datum plane the total head H, at a section O containing point A on a streamline of flow in a channel of large slope (Figure 3.1) can be written as:

$$H = z_A + d_A \cos\theta + \frac{v_A^2}{2g} \quad \text{.......} 3.2$$

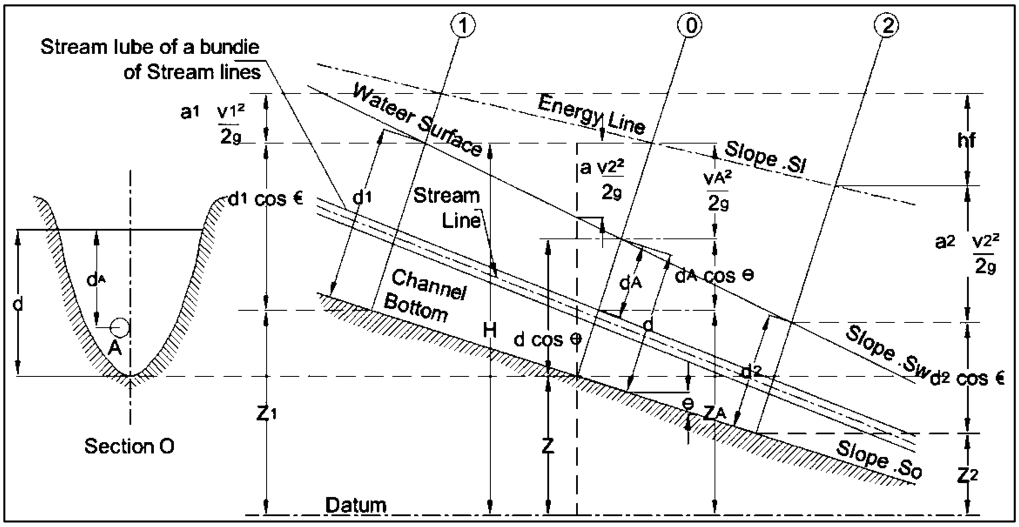


Figure 3-1: Energy in gradually varied open channel flow

For channel of small slope, $\theta \approx 0$, the total energy at the channel section is,

$$H = z + d + \frac{V^2}{2g} \qquad \text{...} \; 3.3$$

3.1.3 The momentum principle

In applying the momentum principle at a short horizontal reach of a prismatic channel, the external force of friction and the weight of the water can be ignored.

Thus, with $\theta = 0 \; and \; F_f = 0$

$$\frac{Qw}{g}(v_2 - v_1) = P_1 - P_2 \qquad \text{..} \; 3.4$$

Where, Q = Rate of Flow

v_1 = Flow Mean Velocity at Section-1

v_2 = Flow Mean Velocity at Section-2

w= Unit weight of water

P_1 = Hydrostatic Pressure Resultant Force at Section-1

P_2 = Hydrostatic Pressure Resultant Force at Section-2

g = Gravitational Acceleration

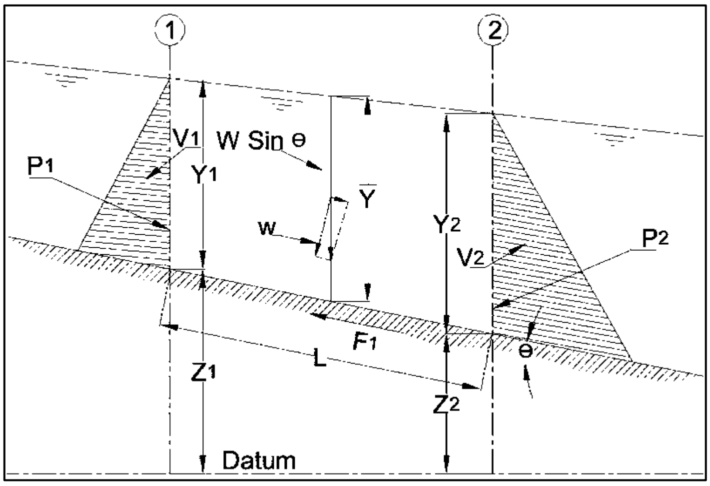


Figure 3-2: Application of the momentum principle

3.1.4 Important hydraulic equations

I) Manning's flow equation

$$v = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad \text{...} 3.5$$

Where,

v= Flow Mean Velocity

n = Manning's Roughness Coefficient

R= A/P (Hydraulic Radius)

A= Flow Cross sectional Area

P = Wetted Perimeter

S = Longitudinal Channel Slope

II) Hydraulic jump equations

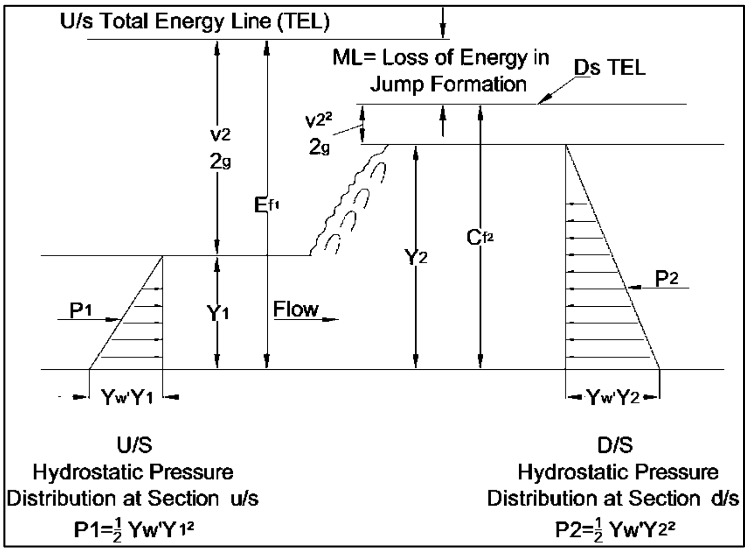


Figure 3-3: Hydraulic jump phenomena in a channel

For known value of y_1, the depth of y_2 after jump can be calculated from equation 3.6 below.

$$y_2 = \frac{y_1}{2}\left[-1 + \sqrt{1 + 8F_1^2}\right] \qquad \text{3.6}$$

Where,

$$F_1 = \frac{V_1}{\sqrt{gy_1}}$$

F_1 = Ratio of inertia force to gravity force (Froude Number) Just before jump.
v_1 & y_1 = Flow Mean Velocity and depth of flow just before jump respectively.

$$H_L = \frac{[y_2 - y_1]^3}{4y_1y_2} \qquad \text{3.7}$$

Usually in the study of hydraulic jump phenomena, the unit discharge (q) and head loss due to the jump (H_L) (i.e. the head difference between upstream reach and downstream tail water level) are known parameters. However, the depths y_1 and y_2 are the unknowns which can be determined graphical developed templates such as like the Blench Curves. Alternatively, equation 3.8 below can also be used for determining the flow depth y_1 through trial and error procedure, preferably the trial and error can be easily calculated with the aid of computer, in excel sheet by the goal seek technique.

$$\frac{8*q^2*H_L}{g} = \left(-1.5*y_1 + \left(\left(\frac{y_1^2}{4}\right) + \left(\frac{2*q^2}{g*y_1}\right)\right)^{0.5}\right)^3 * \left(0.5*y_1 + \left(\left(\frac{y_1^2}{4}\right) + \left(\frac{2*q^2}{g*y_1}\right)\right)^{0.5}\right) \qquad \text{3.8}$$

Where: q, y_1 & H_L are as defined above and g is gravitational acceleration.

Jump length L, is computed as,

$$L = 5\,(y_2 - y_1) \quad \text{or} \quad L = 4y_2 \qquad \text{3.9}$$

For stilling Basin Design Maximum of the two will be adopted.

III) Depth of Scour

The scour depends upon the type and size of bed materials. The scour in alluvial sand deposits shall be estimated by Lacey's formula indicated below.

$$R = 1.35(q^2/f)^{1/3} \quad \ldots\ldots 3.10$$

Where,

R = Normal scour depth in m below the maximum flood level.
q = Design flood discharge per unit width in m^3/sec. at the point of consideration.
f = Lacey's silt factor corresponding to the bed material at sites given by,
f = $1.76(d)^{1/2}$, where d is the mean diameter of the soil particle in millimeter up to possible scour depth.

The value of Lacey's silt factor of bed material are shown in table 3.1, where the bed materials are built up with the alluvial deposits up to 2mm size as there is limitation for adopting Lacey's silt values.

Table 3-1: Value of "f" for different soil

S.No.	Type of Soil	Size of Particle (mm)	Value of "f"
1	Very fine silt	0.052	0.4
2	Fine silt	0.120	0.6
3	Medium silt	0.233	0.85
4	Standard silt	0.323	1.00
5	Medium sand	0.508	1.25
6	Coarse sand	0.725	1.50
7	Fine bajra and sand	0.988	1.75
8	Heavy sand	1.290	2.00

To take care of the maximum scour, the following factors are to be adopted.

Table 3-2: Recommended value of scour depth (R)

Condition	Value
In a straight reach	*1.25R*
At a moderate bend, for example, along apron of guide bank	*1.50R*
At severe bend	*1.75R*
At right angle bends or at noses of piers	*2.00R*
In severe swirls, for example head of a guide bund	*2.50R*

3.2 STRUCTURAL DESIGN CONSIDERATIONS

All structures should be checked for the safety against stability and stress conditions. The major analysis and design steps involved in the structural design are:

- based on cost and easily constructability by local skilled laborers.
- Loads to be considered and factor of safeties for loads and materials.
- Structural configuration, which include modeling of the structure such as in beam column frame arrangement and placement of appropriate loading conditions such as imposed load, live load, water pressure, lateral earth pressure, wind load and earth quake loads.
- Analyze the structure for safety against overturning, sliding and bearing pressure.
- Analyze the structure to get the different actions such as bending moment, shear force and axial compression or tension forces at different location of the structural members.

- Design the structural member. If the structure is reinforced concrete one should define the section thickness and reinforcement bar requirement.
- Identifying construction materials to be used such as masonry, concrete, steel, timber etc.

Definition of some important terms in relation to structural design:

Dead Loads : Dead loads are those, which are normally permanent and constant during the structure's life.

Live Loads : Live load is also called imposed loads are variable in magnitude, as for example water, wind or to human occupants.

Bending Moment : The bending action at a cross section of the structural member due to the applied loads. The unit for bending moment is Force times distance i.e. Nm, KNm etc.

Shear Force : Shear force is the diagonal tension developed in a member section due to the applied load and is usually maximum near the support of a structural member. If the diagonal tension exceeds the limit tensile strength of the concrete, then shear reinforcement must be provided. This reinforcement is either in the form of (1) stirrups, or (2) inclined bars (used in conjunction with stirrups).

Axial Force : The compression or tension force on structural members due to the applied loads.

3.2.1 Loads on structures

The principal load which should be considered for structural design including canal structures are self-weight, earth pressure, lateral water pressure, uplift, live load such as live load at get operation platform, earth quake and wind load.

Operating decks for structures using stop logs shall be designed for a uniform live load of 7.2 KN/m^2.

For calculating various loads of the structure, the following unit weights of the basic construction materials shall be considered.

Table 3-3: Unit weight of basic materials

Dead Loads	Weight (KN/m^3)
Water	10
Stone masonry	21
Brick masonry	21
Mass concrete	23
Reinforced concrete	24
Steel	78.5
Timber (steel)	8
Wood (teak)	6
Dry backfill	16
Saturated backfill	20
Submerged backfill	10.2
Dry, compacted backfill	18.5
Saturated compacted backfill	21.5

Dead Loads	Weight (KN/m^3)
Submerged compacted backfill	11.7
Gabions	14

Table 3-4: Internal angle of friction (∅) of soil

Soil type	ø
Gravel	45° - 55°
Sandy - gravel	35° - 50°
Sandy - loose	28° - 34°
- dense	34° - 45°
Silt, silly sand - loose	20° - 22°
- dense	25° - 30°

Table 3-5: Allowable bearing pressure of soils

Soil Type	Allowable Bearing Pressure KN/m^2
Soft Clays And Silts	< 80
Firm Clays And Firm Sandy Clays	100
Stiff Clays And Stiff Sandy Clays	200
Very Stiff Boulder Clays	350
Loose Well Graded Sands And Gravel/Sand Mixtures	100
Compact Well Graded Sands And Gravel/Sand Mixture	200
Loose Uniform Sands	<100
Compact Uniform Sands	150

Note: - For dynamic loads, a 25% overstress may be allowed

3.2.2 Concrete grades

Grade of concrete is based on the characteristics compressive strength of concrete, which is defined as the 28 days strength below which not more than 5 percent of tested specimens, fails below the specified grade of concrete. For example, C15 represents a grade of concrete with a characteristics strength of 15N/mm^2. The letter "C" refers to the mix and the number to the specified characteristics compressive strength of 150mm cube at 28 days expressed in N/mm^2. Concrete Grade C10, C15, C20, C25 and C30 are recommended for use in irrigation structures as indicated in table 3.6.

Table 3-6: Concrete grades and use

Grade of Concrete	Mix	Specified Characteristics Compressive Strength at 28 days (N/mm^2)	Recommended use of different grades of concrete
C10	1:3:6	10	Use in foundation as lean concrete
C15	1:2:4	15	Use in mass concrete structure
C20	1:1.5:3	20	Use in reinforced Concrete structure
C25	1:1:2	25	Use in water retaining structures such as water tanks, flumes etc.

The recommended grade of concrete for various structures are as shown in table 3.7 below.

Table 3-7: Recommended grade of concrete for various structures

Structure	Recommended Concrete Grades to be used
Blinding concrete	C10
Base slab to covered canals	C10
Mass concrete abutments	C15
Reinforced concrete abutments	C20
Mass concrete piers	C15
Reinforced concrete piers	C20
Mass concrete walls	C15
Reinforced concrete walls	C20
Mass concrete retaining wall	C15
Reinforced concrete retaining wall	C20
Foot Bridge/Bridge slabs	C20
Aqueduct Flume	C20
Covered Canal slab (cast in site)	C15
Covered canal slab (Precast)	C20
Hume pipe	C20
Concrete surrounds to pipes	C15
Concrete block pitching	C15
Concrete lining (Cast in site)	C15
Water retaining structures	C25

3.2.3 Reinforcement steel

The characteristic tensile strength of reinforcement bar to be used shall have yield strength not less than 400MPa. (*fy = fck = 400MPa.* to be used for design in this manual).

The mean value of Modulus of Elasticity of reinforcement bar E_s can be assumed 200GPa. Minimum Reinforcement Provision is required to control the concrete crack during the immature age and the minimum reinforcement required shall be provided as per table 3.8 below.

Table 3-8: Minimum Re-bars required for crack control of immature Concrete

Structural Element	Thickness of Element (h)	Minimum Reinforcement (mm^2)*	
		Top Face	Bottom Face
Walls And Suspended Slabs	h Less Than 500 mm	3.25 X h/2	3.25 X h/2
	h Greater Than 500 mm	3.25 X 250	3.25 X 250
Ground Slabs	h Less Than 300 mm	3.25 X h/2	0
	h Between 300 mm And 500 mm	3.25 X h/2	3.25 X 100
	h Greater Than 500 mm	3.25 X 250	3.25x 100

* Minimum reinforcement per meter run.

3.2.4 Structural analysis

Structural Analysis is the process for the determination of the actions on the structure due to all possible applied loads as listed in section 3.2.1, load on structures. The actions obtained after the structural analysis are bending moment, Shear force and axial force. The analysis can be carried out manually with the help of equilibrium equations for simple determinate type structure, however for indeterminate type problems; the use of software application like SAP-2000 is preferred for accuracy and time saving. Following the completion of the analysis, the design of the member size and reinforcement requirement shall be carried out based on the limit state design.

3.2.5 Limit state design

In this manual, the limit state design will be in use as this method is the acceptable current practice by our local codes and other international codes. The limit state method multiplies the working load by partial factor of safety and also divides the materials ultimate strength by further partial factor of safety.

Table 3-9: Partial safety factor applied to material (γ_m)

Limit state	Material	
	Concrete	Steel
Ultimate		
Flexure	1.5	1.15
Shear	1.25	1.15
Bond	1.4	
Serviceability	1	1

Table 3-10: Partial factor of safety for loadings

Load combination	Ultimate				Serviceability All
	Dead	Imposed	Earth & Water	Wind	
Dead & Imposed (+ Earth & Water)	1.35 (Or 1)	1.5 (Or 0.0)	1.4	-	1.0
Dead & Wind (+ Earth & Water)	1.35 (Or 1)	-	1.35	1.35	1.0
Dead & Imposed & Wind (+ Earth & Water)	1.2	1.2	1.2	1.2	1.0

The lower values in brackets applied to dead or imposed loads at the ultimate limit state should be used when minimum loading is critical.

3.2.6 Flexural design of reinforced concrete member

The theory of bending for reinforced concrete assumes that the concrete will crack in the regions of tensile strains and that after cracking all the tension is carried by the reinforcement. It is also assumed that plane section of a structural member remains plane after straining, so that across the section there must be a linear distribution of strains.

Figure-3.4 below shows the cross section of a member subjected to bending and the resultant strain diagram together with 3 different types of stresses distribution in the concrete.

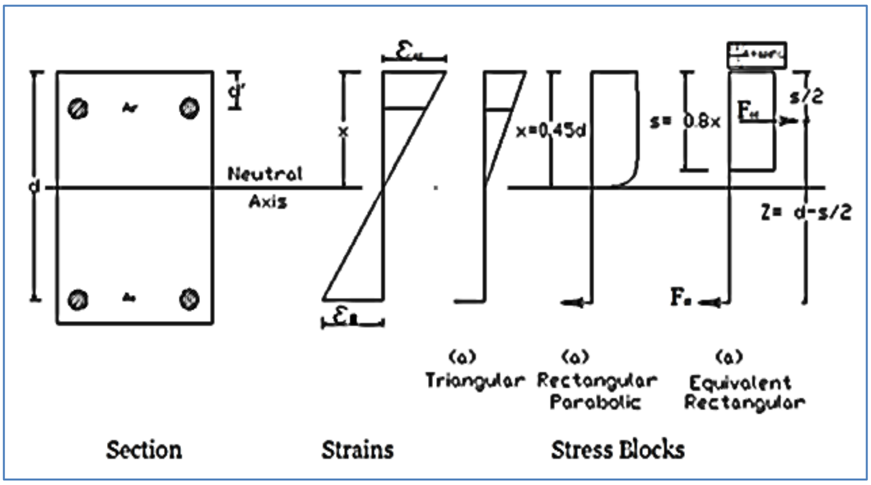


Figure 3-4: Section with stress diagram and stress block for singly reinforced section

(1) The triangular stress distribution applies when the stresses are very nearly proportional to the strains, which generally occurs at the loading levels encountered under working conditions and it is, therefore, used at serviceability limit state.
(2) The rectangular – parabolic stress block represents the distribution at failure when the compressive strains are within the plastic range and it is associated with the design for the ultimate limit state.
(3) The equivalent rectangular stress block is a simplified alternative to the rectangular – parabolic distribution.

For singly reinforced section in equilibrium, the ultimate design moment, M, must be balanced by the moment of resistance of the section so that,

$$M = F_{cc} \times z = F_{st} z$$

Where, z is the lever arm between the resultant forces F_{cc} (Compression Force in concrete) and F_{st} (Tension Force in Steel).

$$F_{cc} = 0.567 f_{ck} bs$$

$$M = 0.567 f_{ck} bsz$$

$$z = d - \frac{s}{2}$$

$$M = 1.134 f_{ck} b(d - z)z$$

$$K = \frac{M}{bd^2 f_{ck}}$$

Therefore,

$$z = d\left[0.5 + \sqrt{0.25 - k/1.134}\right]$$

Hence,

$$A_s = \frac{M}{0.87 f_{yk} z} \quad \text{……….………………………3.11}$$

The lower limit for the lever arm can be determined from the limit depth of the neutral axis that is x=0.45d, Minimum lever arm limit is therefore, z= d-(0.8*0.45d/2)= 0.82d,

Hence, for balanced failure,

$$M_{bal} = 1.134 f_{ck} b(d - 0.82d) * 0.82d = 0.167 f_{ck} bd^2$$

Therefore,

$$\frac{M_{bal}}{bd^2 f_{ck}} = K_{bal} \quad \text{……….………………………3.12}$$

For section to be designed as single reinforcement and failure first to happen in yielding

$$K_{bal} < 0.167 \quad \text{……….………………………3.13}$$

3.2.7 Shear Resistance design of reinforced concrete member

It is inconvenient to use shear reinforcement in slabs because it is difficult to fix, impends placing of concrete, and is inefficient in the use of steel. The wall or base slab thickness therefore should be at least sufficient to allow the ultimate shear force to be resisted by the concrete in combination with the longitudinal reinforcement. Maximum ultimate shear carrying capacity of reinforced concrete slab is given by equation 3.14 below as per British Standard (BS 8110).

$$\upsilon_c = \left(0.79\left(\frac{f_{ck}}{25}\right)^{\frac{1}{3}}\left(\frac{100A_s}{bd}\right)^{\frac{1}{3}}\left(\frac{400}{d}\right)^{1/4}\right)/\gamma_m \ldots\ldots\ldots\ldots\ldots\ldots 3.14$$

The steel ration should not be taken as greater than 3. The value of effective depth (d) should not be taken more than 400mm and f_{ck} should not be taken as greater than 40N/mm^2. Partial safety factor (γ_m) shall be taken as 1.25.

For effective depth greater than 400mm thick, the following table shall be used for the ultimate shear carrying capacity according to the respective grade of concrete and percentage of longitudinal reinforcement steel.

Table 3-11: Ultimate shear capacity of concrete (N/mm^2) for depth greater than 400mm

Percentage of Reinforcement	Grade of Concrete			
100As/bd	C15	C20	C25	C30
≤0.15	0.29	0.32	0.34	0.36
0.25	0.33	0.37	0.40	0.42
0.5	0.42	0.46	0.50	0.53
0.75	0.49	0.53	0.58	0.61
1	0.53	0.59	0.63	0.67
1.5	0.60	0.67	0.72	0.76
2	0.68	0.74	0.80	0.85
≥3	0.77	0.85	0.92	0.97

3.2.8 Stone masonry design

i) General

In most cases of small scale irrigation infrastructures the use of stone masonry structural work is the common practice. Compared to other construction materials, masonry is relatively cheap and easy to work with. One major disadvantage of masonry work is that its capacity to with stand tension is very limited. Due to this, it will be necessary to check the magnitude of tension force at critical sections. The unit weight for Stone masonry and soil for design purpose can be taken from Table 3.3.

ii) Active earth pressure

Active earth pressure shall be calculated based on Equation 3.15 and 3.16 below.

$$p = k_a \gamma_s h \ldots\ldots\ldots\ldots\ldots\ldots 3.15$$

Where,

K_a is active earth pressure Coefficient calculated as, $K_a = \frac{1-sin\emptyset}{1+sin\emptyset}$

And, $\emptyset$ = Angle of friction of the soil

h = Soil depth from top surface

The resultant force of the triangular pressure distribution of the active earth pressure (F_s) is also calculated as,

$$F_s = \frac{1}{2} k_a \gamma_s h^2 \quad \text{..3.16}$$

iii) Design Assumptions

- When a surcharge load is to be considered, the value of surcharge load should be taken according to the nature of fill and slope of surcharge.
- For Hydraulic structures, 2/3 of the bottom soil is assumed to be saturated.
- The triangular wedge of the retained soil is assumed to assist the stabilizing effect.
- The passive earth pressure is assumed to be counter blocked by an equivalent active pressure.

iv) Stability Analysis

The following procedures shall be used in stability analysis of a retaining wall.

1. Consider unit length of the structure (Retaining Wall)
2. Work out the magnitude and direction of all the vertical forces acting on the structure and their algebraic sum i.e ∑V.
3. Similarly work out all the horizontal forces and their algebraic sum i.e. ∑ H.
4. Determine the liver arm of all these forces about the toe.
5. Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments i.e. ∑M.
6. Find out the location of the resultant forces by determining its distance from the toe.

$$z = \frac{\sum M}{\sum V}$$

Find out the eccentricity e of the resultant using.

$$e = \frac{B}{2} - z$$

7. Determine the vertical stresses at the toe and heel using:-

$$P_v = \frac{\sum V}{B}\left[1 \pm \frac{6e}{B}\right]$$

8. Determine the factor of safety against to overturning as equal to :-

$$F.O.S = \frac{\sum Stablising\ Moment\ (+)}{\sum Diturbing\ Moment\ (-)}$$

Minimum F.O.S shall be = 2.00

9. Determine the factor of safety against sliding, using sliding factor and sliding resistance force.

$$Sliding\ Factor = \frac{\tan\emptyset \sum V}{\sum H}$$

$\emptyset$ = Angle of friction of the soil

Minimum Sliding Factor shall be $= 1.5$

v) Grades of cement masonry for various structures

Table 3-12: Recommended grades of cement masonry for various structures

Structure	Recommended Cement and Sand Mix ratio for masonry work	Plastering Requirement
Abutment	1:4	No
Pier	1:4	No
Wall	1:4	No
Retaining Wall	1:4 or 1:5	No
Flume Wall	1:4	No
Chute wall	1:3	Yes
Stilling Basin	1:3	Yes

Allowable stresses for designs are:

Cement Masonry (1:4)

Compressive stress = 0.7N/mm2 for well-constructed new masonry
= 0.4N/mm2 for poorly constructed old masonry
Tensile stress = 0 for normal loading cases
= 0.1N/mm2 for extreme loading cases

Brick Masonry (1:4)

Compressive stress = 0.35N/mm2 for common building bricks
= 0.7N/mm2 for second class bricks
= 1.07N/mm2 for first class bricks

Tensile stress = 0 for normal loading cases

4 HYDRAULIC AND STRUCTURAL DESIGN FOR SSIPS

4.1 CANAL RELATED STRUCTURES

Many different types of canal structures are required in an irrigation system to effectively and efficiently convey, regulate, and measure the canal discharge and also to protect the canal from storm runoff damage. The design capacity of the conveyance, regulating, and water measurement structures discussed throughout this manual is limited to the scale of small-scale irrigation design to the context of our local practice. Canal related structures, which commonly provided by our current local practice are categorized as follows:

i) Canal Conveyance Structures,
ii) Canal Regulating Structures,
iii) Flow measurement structures,
iv) Night storages,
v) Drainage Crossing Structures

4.2 APPROPRIATE CANAL RELATED STRUCTURES SELECTION CRITERIA

The selection of suitable canal structure is based on the following basic criteria;

i) Proper Optimization of the available head to command the intended irrigable scheme.

Whenever any structure is introduced in a canal, there will be certain head loss associated with the structure. Some structures have small head loss contribution and others introduce more head loss. For the case where the available head is limited to command the target irrigation scheme, the designer should give more emphasis how to minimize the head losses while introducing structures in the canal system. For instance in a canal system of drainage crossing structure, one may thought of to provide either an inverted siphon or elevated flume structure. It is usually evident that inverted siphons require more head than elevated flume, and hence if the system is sensitive in utilizing the available head, it is wise decision to select elevated flume instead of an inverted siphon for such case of drainage crossing structure.

ii) Initial Construction Cost and operation Safety

If there are two or more alternative type of structures, which can equally serve for the intended purpose, and where available head limitation may not be the governing criteria, it is natural that one should compare the investment cost and operation easiness comparison to identify the most suitable structure. The case of inverted siphon and flume structure is also the best illustrative example for such case too.
Merit and Demerit of Inverted Siphon over Flume structure

- Inverted siphon usually require lower initial investment cost than Flume structure.
- Inverted siphon will be less efficient structure with respect to operational aspect as inverted siphon is clogged by entrance of gravel, rock, silt, floating materials, and therefore, trash racks and/or screens should always be provided at the inlet and require frequent maintenance work.

In spite of the lower initial investment cost for inverted siphon, due to the operation difficulty it is usually preferred to provide flume structure especially when the crossing is across wide depression or wide roadways.

iii) Easy of constructability and familiarity with the local skill laborers.

The brief description and design aspect for each of the canal related structures listed in section 4.1 above are presented in the sections below with the help of illustrative worked examples.

4.3 CANAL CONVEYANCE STRUCTURES

Conveyance structures, i.e. road crossings, inverted siphons, drops, chutes, flumes, and pipelines, are used to safely transport water from one location to another, traversing various natural and manmade topographic features along the way.

Such structures include:

(1) Elevated flumes to convey water over river drainage gullies or a roadway.
(2) Inverted siphons to convey canal water under natural channels or roadways.
(3) Road crossings to carry canal water under roadways.
(4) Bench flumes to convey the water along a steep hillside.
(5) Drops or chute structures to safely lower the canal water down a hillside.

4.3.1 Design of Elevated Flume

Box 4-1:
Worked Example-1: Design an elevated flume structure based on the following given data:

I – Hydraulic Design

A- Data for Design

Hydraulic Characteristics of the canal

Discharge, Q m^3/sec.	Bed Width B(m)	Water Depth d(m)	Side Slope	Velocity (m/s)
0.4	0.7	0.5	1.2H:1.0V	0.61

Length of Flume: 21.0m
Flume Cross sectional Shape: Rectangular
Upstream Canal Bed level = 1800.00m

Flume Section

Assume b/d = 1.5 to 2.0 and v = 1.0m/s
Roughness Coefficient, n = 0.014

Water Area, $A = \frac{Q}{v} = \frac{(0.40)}{(1.0)} = 0.4\text{m}^2$

Bed Width, $b = (0.90)$

Water Depth, $d = \frac{b}{2} = \frac{(0.90)}{2} = 0.45\text{m}$

Velocity, $v = \frac{Q}{A} = \frac{(0.400)}{(0.405)} = 0.987\text{m/s}$

Froude number, $F_r = \frac{v}{\sqrt{gd}} = \frac{0.987}{\sqrt{9.81x(0.45)}} = 0.470 < 0.7 \text{ ok.}$

$F_r \leq 0.7$ provides to be stable subcritical flow.

Wetted Perimeter, P = b+2d = 0.90+2x0.45 = 1.80m

Hydraulics Radius, $R = \frac{A}{P} = \frac{(0.40)}{(1.80)} = 0.222$

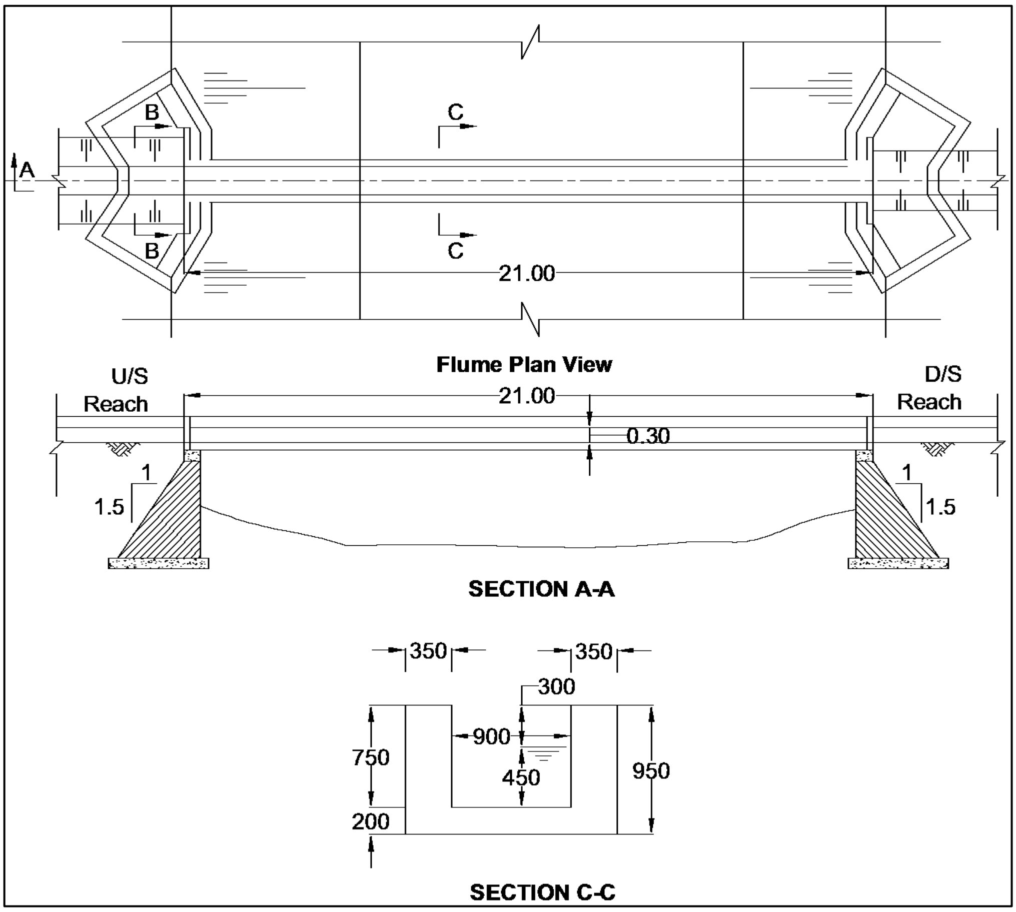


Figure 4-1: Flume plan and sectional view

B- Head Losses between upstream and downstream Canal Reach

The head loss components are Entrance Transition Loss, Frictional Loss in Flume and Outlet Transition Loss. Inlet and outlet transitions are made earth transitions with straight head wall as shown in Figure 4.1 and, the loss coefficient for the difference of the velocity head between canal and Flume is 0.50 for entrance loss and 0.7 for outlet loss.

Inlet Loss (h_{fi}), $h_{fi} = 0.5\left(\frac{v_{flume}^2}{2g} - \frac{v_{canal}^2}{2g}\right)$ $=0.5\left(\frac{0.987^2}{2x9.81} - \frac{0.61^2}{2x9.81}\right)$= 0.015m

Friction Loss (h_{fr}), $h_f = \frac{n^2 v_{flume}^2}{R^{4/3}} L$ $= \frac{.014^2 x 0.987^2}{0.222^{4/3}} x21m$ =0.03m

(Where, n=0.014, Manning's Roughness Coefficient for concrete lined channel)

Outlet Loss (h_{fo}),$h_{fo} = 0.7\left(\frac{v_{flume}^2}{2g} - \frac{v_{canal}^2}{2g}\right)$ $=0.7\left(\frac{0.987^2}{2x9.81} - \frac{0.61^2}{2x9.81}\right)$ =0.022m

Total Head Loss (h_f) = 0.015+0.03+0.022 = 0.067m

C- Elevations at different reaches of the flume

Elevations at various reach of the flume can be calculated using the energy equation. The total Energy (T.E) at any section is equal accounting the head losses in upstream reach.

Bed level at upstream (u/s) canal	= 1800.00	
Canal Flow Depth	= 0.50m	
Full Supply Level	= 1800.00 +0.50	= 1800.50m
Velocity Head at u/s canal	= $0.61^{2}/2*9.81$	= 0.019m
T.E at u/s canal	= 1800.5+0.019	= 1800.519m
Transition Loss between u/s canal and Flume inlet		= 0.015m
T.E at Flume inlet	= 1800.519-0.015	= 1800.504m
Velocity Head at Flume	= $0.987^{2}/2*9.81$	= 0.050m
Water Surface at Flume inlet	= 1800.504-0.050	= 1800.454
Water Depth at the Flume inlet		= 0.450m
Bed level at flume inlet	= 1800.454-0.45	= 1800.004m
Head Loss between flume inlet and outlet		= 0.030m
T.E. at flume outlet	= 1800.504-0.030	= 1800.474m
Velocity Head at Flume	= $0.987^{2}/2*9.81$	= 0.050m
Water Surface at Flume outlet	= 1800.474 – 0.050	= 1800.424m
Bed level of flume outlet	= 1800.43-0.45	= 1799.974m
Head Loss at outlet transition		= 0.022m
T.E. at downstream (d/s) canal	= 1800.474-0.022	= 1800.452m
Velocity Head at d/s canal	= $0.61^{2}/2*9.81$	= 0.019m
Water surface at d/s canal	= 1800.452-0.019m	= 1800.433
Bed level at d/s canal	= 1800.433-0.50	= 1799.933m

II – Structural Design

A- Design Data

Total Flume span Length		21.00m
Flume span c/c of support (Assuming 0.25mm less both side)		20.50m
Flume is to be constructed out of reinforced concrete Grade:		C-25
Reinforcement bar characteristics yield strength shall not be less than:		400Mpa
Unit Weight of Water		$10KN/m^3$
Unit weight of Reinforced Concrete:		$24KN/m^3$
Load Factor for limit state design for Dead Load (DL)		1.35
Load Factor for limit state design for Live Load (LL)		1.50
Flume wall and base slab thickness:		
Flume Wall Thickness	= Span/60 = 21,000/60	350mm
Flume Base slab Thickness	= Flume width/20 = (900+2*350)/20	80mm

However, for practical construction purpose, the thickness of wall and

Slab should not be less than 200mm, therefore 350mm and 200mm thicknesses for wall and base slab are adopted respectively.

B- Structural Design Procedure

The structural design is conducted in two steps, the first being design of the section to provide adequate resistance for the wall to withstand vertical hydrostatic pressure and the bottom slab to resist the water load. The second step is to design the flume in longitudinal direction to provide resistance capacity of the flume to carry the self-weight and the water inside the flume.

C- Flume Section Analysis and Design

To calculate the hydrostatic Pressure at bottom of the Flume channel, the water is assumed full up to flume wall top. And the computation is also carried out for unit length of the flume.

Wall height, h = Water depth + Base Slab thickness/2 = 0.75+0.2/2 = 0.85m

Pressure at Bottom = 10KN/m^3*0.85m = 8.5KN/m^2

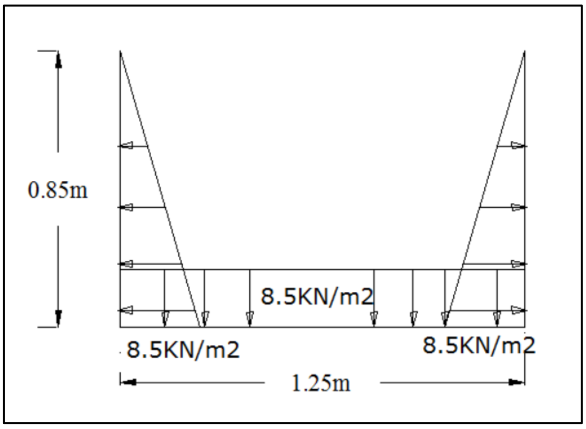


Figure 4-2: Hydrostatic load on flume wall and base slab

The analysis is preferred to be carried out in simplified static formula, as shown below;

Self-weight of the base slab = Thickness x Unit weight of concrete = 0.2x24 = 4.8KN/m

Water Load = 8.5KN/m

Load factors used are 1.35 for self-weight and 1.5 for water load which are Dead load and Live load factors respectively.

Bending Moment at middle of the base slab = $wl^2/8$, where l= 0.9m+0.35m = 1.25m

Thus, Factored Bending moment at mid span = $1.35*4.85*1.25^2/8+ 1.5*8.5*1.25^2/8$
= 3.8KNm

Bending Moment at Support of the base slab = $wl^2/6$,

Thus, Factored Bending moment at Support = $1.35*4.85*1.25^2/6+ 1.5*8.5*1.25^2/6$
= 5.0KNm

For the design of the flume section, the first step is to determine the minimum reinforcement requirement. From Table 3.8, for wall and suspended slab less than 500mm thick, minimum reinforcement required is calculated as:

$A_s = 3.25*200/2 = 325mm^2$ (Both Faces),

Where, A_s is the minimum area of steel required in each face of the slab

Using Dia. 10mm reinforcement bar, spacing = 1000/(325/(10*10/4*3.14) = 240mm
To calculate the flexural reinforcement for the maximum design bending moment value of 5.0KNm, flexural Reinforcement calculation equation provided in previous sections will be employed. To facilitate the computation, an excel sheet template is used as shown below.

Flexural design

Description	Value	Unit	
M(Design Moment), at Support	5	KNm	
Concrete Compressive Strength (Fck)	25	Mpa.	
Steel Tensile Strength (Fst)	400	Mpa.	
Section width (b)	1000	mm	
Section Height (h)	200	mm	
Clear Cover (c)	50	mm	
Assume Re-Bar Dia.	10	mm	
effective depth(d)	135	mm	
K	0.0110	<	0.167 Ok!
z	133.68	mm	
As	107.48	mm2	
No. Bars	1.37		
Spacing	730	mm	
Actually Provide	240	mm c/c	Min Requirement

D- Flume Analysis and Design in Longitudinal Direction

Load assessment
Self-Weight of Flume channel (DL) = Wall loads and Base slab load
= 2* 0.75*0.35*24 + 1.6 *0.2*24 = 20.28KN/m
Water Load, for full supply level (LL) = .75*0.9*10 = 6.75 KN/m
Factored DL = 1.35*20.28 = 27.38KN/m
Factored LL = 1.5*6.75 = 10.13KN/m
Total Factored Load DL+LL = 37.51KM/m
Max. BM will be at mid span Calculated as, $wl^2/8$ **=** $37.51*20.5^2/8$ = 1970.45KNm

Considering the two wall as deep beams the reinforcement bars required for each wall can be computed. The bending moment shared by one wall is equals 1970.45/2 = 985.23KNm

Flexural design

Description	Value	Unit	
M(Design Moment), at Support	985.23	KNm	
Concrete Compressive Strength (Fck)	25	Mpa.	
Steel Tensile Strength (Fst)	400	Mpa.	
Section width (b)	350	mm	
Section Height (h)	950	mm	
Clear Cover (c)	50	mm	
Assume Re-Bar Dia.	28	mm	
effective depth(d)	876	mm	
K	0.1467	<	0.167 ok!
z	742.22	mm	
As	3814.38	mm2	
No. Bars	6.20		
Total No. for the width of the flume = 2xNo. Bars	12.40	mm	
Actually Provide at bottom of flume	13	No.	

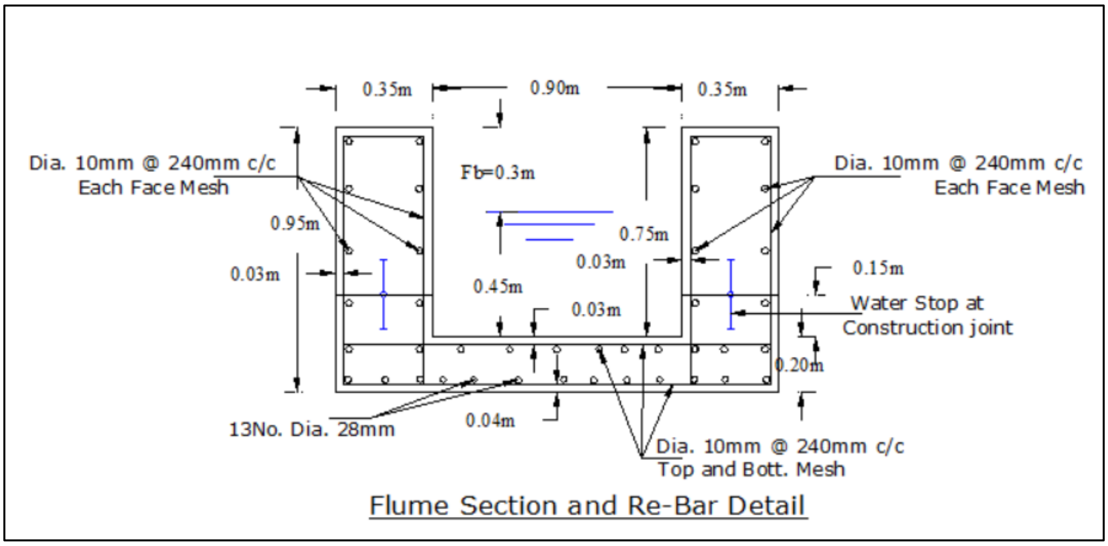


Figure 4-3: Flume reinforcement detail

E- Flume end support Abutment Wall Design

To check the stability of the Stone Masonry Abutment Retaining Wall, the following steps can be followed.

Step-1: Define the cross section of the abutment wall from the flume elevations data, site topography, geological formation, High Flood Mark and water table. Together with the acting forces magnitude and directions can be also incorporated as shown in sectional drawing of abutment wall below.

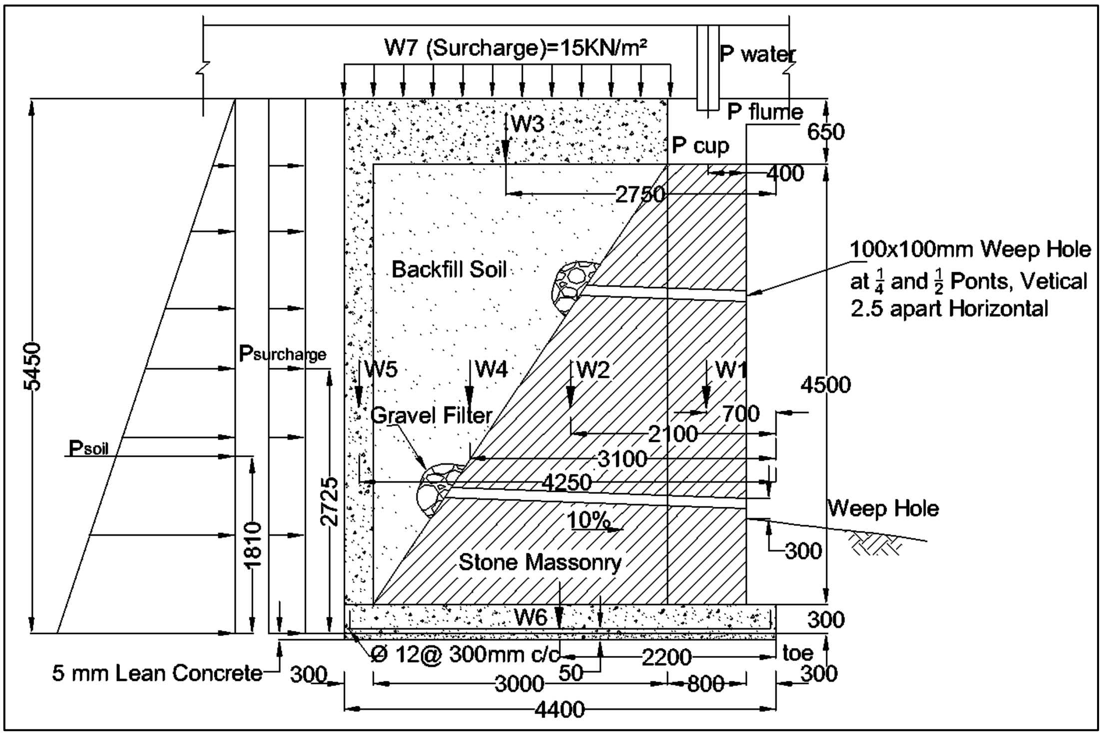


Figure 4-4 Abutment wall section and acting loads

Step-2: Define Engineering Properties of the materials

Unit weight of Saturated back fill soil = 20KN/m3

Unit Weight of Stone masonry Work = 21KN/m3

Unit Weight of Reinforced Concrete = 24KN/m3

Unit weight of Water = 10 KN/m3

Internal Angle of Friction (Ø) of Soil = 33^0

Active earth pressure Coefficient, $K_a = \frac{1-sin\emptyset}{1+sin\emptyset}$ = = 0.295

Coefficient of Friction of Foundation soil, μ = tan (Ø) = 0.65

Masonry wall side slope in the fill side = 1H: 1.5V

Hydrostatic pressure due to water is avoided by providing weep holes and drainage gravel pack filters as detailed in the sectional drawing above.

Step-3: Stability Analysis and Foundation Bearing Pressure Calculation

The following table below can be used to perform the stability analysis of the abutment wall of the flume and foundation bearing pressure which is brought from automated Excel Spread sheet for easy of arithmetic calculation. The procedure outlined in section 3.2.9 is in use for the stability analysis and foundation bearing pressure calculation.

i) **Stability check when flume channel is at full water level operation case**

Load Description	Load Magnitude in (KN) per meter width of abutment		Moment Arm Length of the Load from Toe of the abutment	Stabilizing Moment from Vertical Loads, (MS)	Destabilizing Moment from Horizontal Loads, (Mdis)	Factor of Safety Against Over Turning , F.O.S	Factor of Safety Against Sliding , F.O.S
	Vertical Loads	Horizontal Loads					
(A)	(B)	(C)	(D)	(E) =(B) X (D)	(F) =(C) X (D)	(G) =(∑(E)/∑(F))	(H) =μ∑(B)/∑(C)
P Water = Water Load in Flume for at one of the abutment wall	85.05		0.5	42.525		5.95	4.44
P Flume =Flume section Concrete Load at one of the abutment wall	162.75		0.5	81.375			
P cup =Abutment Concrete Cup Load	10.5		0.7	7.35			
W1 = Masonry Wall, Rectangular	75.6		0.7	52.92			
W2 = Masonry Wall, Triangular	141.75		2.1	297.675			
W3 = Back fill Soil, Rectangular	42.9		2.6	111.54			
W4 = Back fill Soil, Triangular	135		3.1	418.5			
W5 = Back fill Soil, Rectangular	27		4.25	114.75			
W6 = Base Slab	33		2.2	72.6			
W7 = Soil Surcharge Load	49.5		2.75	136.125			
P_{soil} = Soil Lateral Load		24.12	2.725	0	65.72		
$P_{surcharge}$= Surcharge soil Lateral Load		87.62	1.81	0	158.60		
Total = ∑ ( ) and Check for Safety	763.05	111.74		1335.36	224.31	Safe!	Safe!
∑ M = ∑ (E) - ∑ (F)	1111.05						
Resultant Force position from toe, z = ∑(M)/∑(B)	1.46						
Foundation Base Width, B in meter	4.40						
Eccentricity , e = B/2-z	0.74						
Check for Foundation Bearing Capacity	Pmax (KN/m2)	349					
$P_v = \frac{\sum V}{B}\left[1 \pm \frac{6e}{B}\right]$	Pmin (KN/m2)	-3					

ii) Stability check when flume channel is empty case

Load Description	Load Magnitude in (KN) per meter width of abutment		Moment Arm Length of the Load from Toe of the abutment	Stabilizing Moment from Vertical Loads, (MS)	Destabilizing Moment from Horizontal Loads, (Mdis)	Factor of Safety Against Over Turning , F.O.S	Factor of Safety Against Sliding , F.O.S
	Vertical Loads	Horizontal Loads					
(A)	(B)	(C)	(D)	(E) =(B) X (D)	(F) =(C) X (D)	(G) =(∑(E)/∑(F))	(H) =μ∑(B)/∑(C)
P Water = Water Load in Flume for at one of the abutment wall	0		0.5	0		5.76	3.94
P Flume =Flume section Concrete Load at one of the abutment wall	162.75		0.5	81.375			
P cup =Abutment Concrete Cup Load	10.5		0.7	7.35			
W1 = Masonry Wall, Rectangular	75.6		0.7	52.92			
W2 = Masonry Wall, Triangular	141.75		2.1	297.675			
W3 = Back fill Soil, Rectangular	42.9		2.6	111.54			
W4 = Back fill Soil, Triangular	135		3.1	418.5			
W5 = Back fill Soil, Rectangular	27		4.25	114.75			
W6 = Base Slab	33		2.2	72.6			
W7 = Soil Surcharge Load	49.5		2.75	136.125			
Psoil = Soil Lateral Load		24.12	2.725	0	65.72		
Psurcharge= Surcharge soil Lateral Load		87.62	1.81	0	158.60		
Total = ∑ ( ) and Check for Safety	678	111.74		1292.835	224.31	Safe!	Safe!
∑ M = ∑ (E) - ∑ (F)	1068.52						
Resultant Force position from toe, z = ∑(M)/∑(B)	1.58						
Foundation Base Width, B in meter	4.40						
Eccentricity , e = B/2-z	0.62						
Check for Foundation Bearing Capcity $P_v = \frac{\sum V}{B}\left[1 \pm \frac{6e}{B}\right]$	Pmax (KN/m2)	285					
	Pmin (KN/m2)	23					

Thus, from the above result of bearing pressure, the bearing capacity of the foundation of the abutment wall should not be less than 349KN/m^2.

F- Middle pier support

The above example has not include the case when there is middle pier support. Every hydraulic parameters are kept similar to the previous example but middle pier is introduced for this situation. Thus, the structural design calculation for the flume and middle pier will be worked out as follows.

Total Flume span Length	21.00m
No. Bays	2.0
Each Bay Width	10.50
Flume span c/c of support, from Abutment to middle pier	
(Assuming 0.25mm less both side)	10.00m

Flume wall and base slab thickness:

Flume Wall Thickness	= Span/60 = 10,500/60	175mm
Flume Base slab Thickness	= Flume width/20 = (900+2*175)/20	62.50mm

However, for practical construction purpose, the thickness of wall and Slab should not be less than 200mm, therefore thickness of 200mm is used for both wall and slab of the flume.

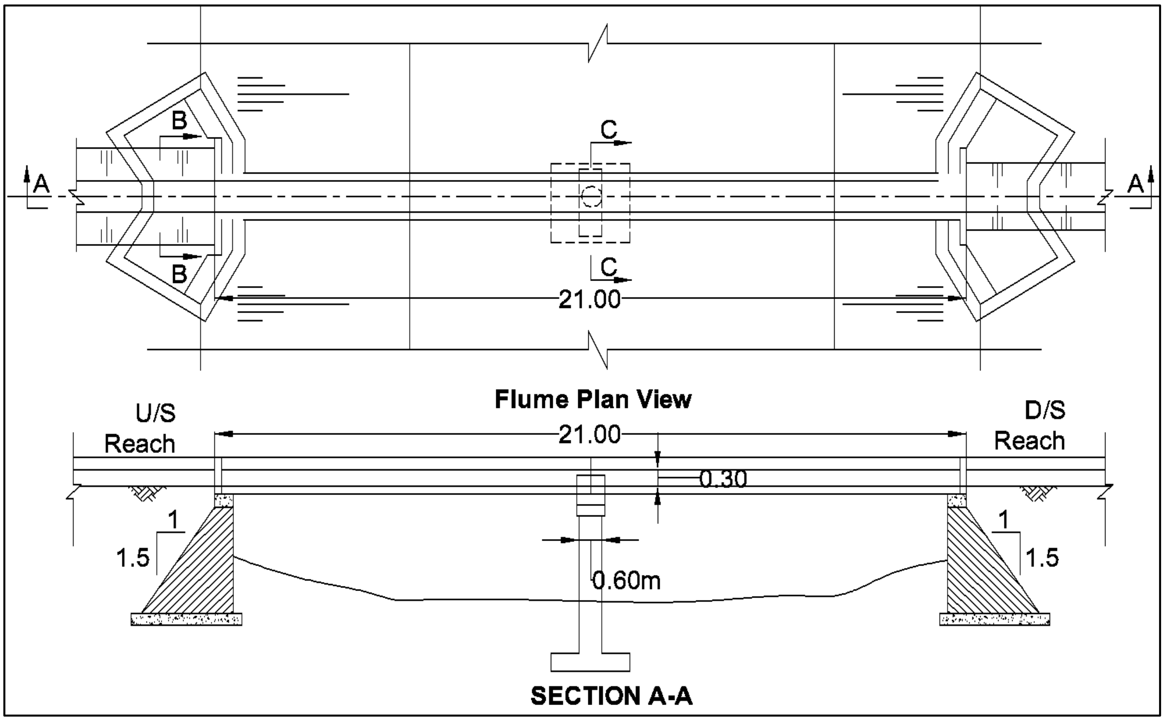


Figure 4-5: Flume Plan and sectional view (flume support Pier at mid span)

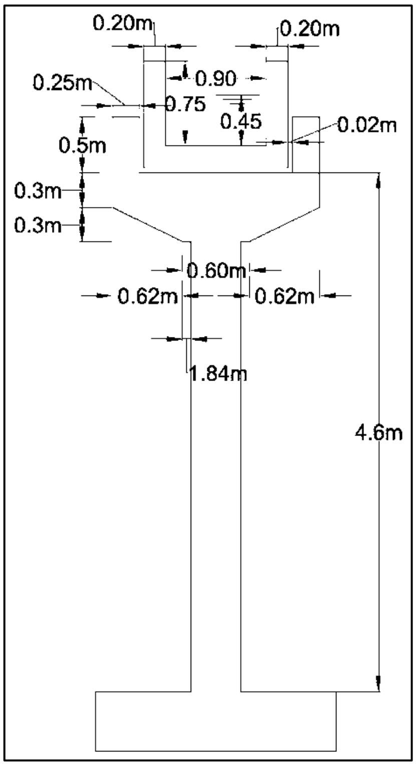


Figure 4-6: Middle pier sectional view

The flume channel reinforcement can be designed with similar procedure as done in previous section. The following section will be the design of pier components, i.e. Column cup (Bracket), Column and Footing.

Bracket design

Loads on the bracket,

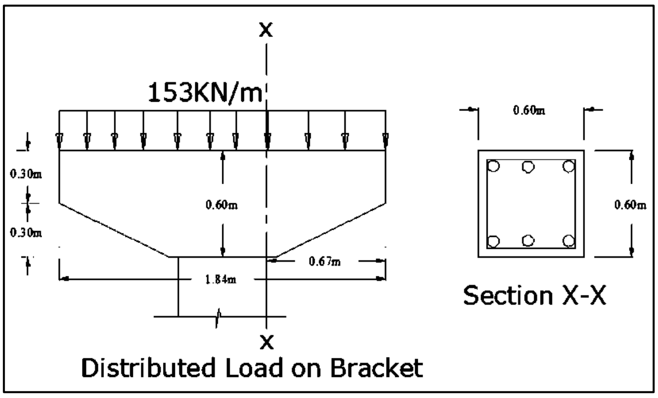
Flume Channel Load (DL) = 2*Wall + Base Slab
= 2*.2*.75*10.5*24+0.2*.9*10.5*24 = 121.0 KN

Bracket self-weight (DL) = 2*0.25*0.5*0.6*24+0.3*1.84*.6*24
+.3*.6*.6*24+.62*.3*.6*24 = 40.2KN

Total DL = 121.0+40.2 = 161.2KN

Water Load (LL) = 0.9*0.45*10.5*10 = 42.5KN

Factor Load on Bracket = 1.35*DL + 1.5 LL = 1.35*161.2+1.5*42.5 = 281.4KN
Factored distribution load on the width of bracket = 281.4/1.84 = 153KN/m



Bending Moment at face of column = $wl^2/2$= $153*0.67^2/2$ = 34.34KNm

Shear Force at face of column = wl/2 = 153*.67/2 = 51.26KN

Flexural design

Description	Value	Unit	
M(Design Moment), at Support	34.34	KNm	
Concrete Compressive Strength (Fck)	25	Mpa.	
Steel Tensile Strength (Fst)	400	Mpa.	
Section width (b)	600	mm	
Section Height (h)	600	mm	
Clear Cover (c)	50	mm	
Assume Re-Bar Dia.	12	mm	
effective depth(d)	534	mm	
K	0.008	<	0.167 ok!
z	530.74	mm	
As	186.12	mm2	
No. Bars	1.65		
Actually Provide for top bar	3	No.	

Shear Carrying capacity of the bracket at column face can be calculated from table-3.11,

Percentage of reinforcement = Total area of longitudinal bar/Gross area of bracket

= 6*12*12*3.14/4/(600*600)%= 0.19%

From Table 2.11, C-25 concrete Shear capacity is equal to 0.34N/mm2 for 0.15% of rebar and 0.40N/mm2 for 0.25% re-bar percentage.

Therefore, by interpolation the shear carrying capacity for 0.19% of re-bar percentage is 0.364N/mm2.

Thus, shear force capacity of the section = 0.364*600*600/1000= 131KN >51.26KN, hence the section is safe for shear carrying capacity.

Pier column design

Pier column should be designed for two possible cases, as when the flume construction site is free from earthquake zone or if the site is in earth quake zone.

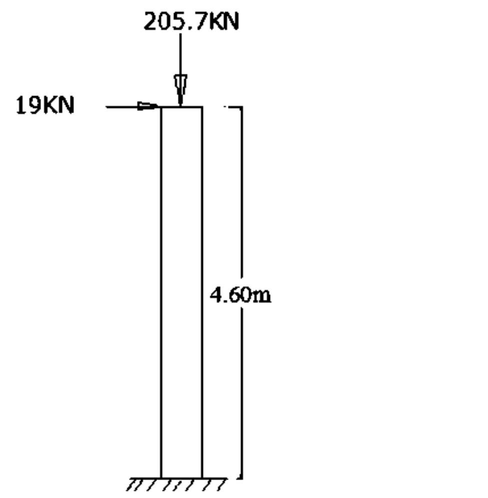
When the site is free from earthquake zone, the vertical load and wind load from the super structure and substructure will be considered for design of the column. Whereas when the flume site is in a seismic zone, the horizontal force generated due to the earthquake should also be considered.

Case 1: Column Design for none earth quake Zone.

Total Load of Structure on the Column = Flume channel Load + Pier Cup Load + Water Load
= 121 + 40.2 + 42.5 = 205.7KN

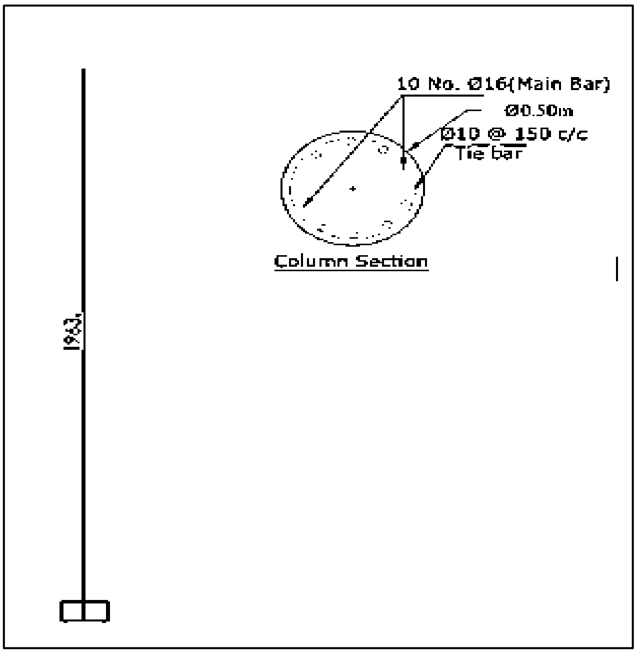
For earth quake free zone it is advisable to design the column to with stand lateral wind load. As per the Ethiopian Road Authority (ERA-2002) Bridge design manual, the base wind pressure for large flat surface is taken as 1.9KN/m^2. For low height flume, this base wind pressure value can be considered as design wind pressure.

Wind pressure load = 1.9*Flume area shared by the middle column = 1.9*10.5*.95 = 19KN



Forces acting on column
free body diagram, free of earth quake load

Using reinforced concrete Circular column with cross sectional diameter of 500mm and the loads arrangement as shown in the free body diagram, the SAP-2000 output for the design of the column main reinforcement bar in unit of mm^2 is as shown below.



Using 10 No. Dia. 16mm main bar, which will give gross re-bar area of 2010mm2 > 1963mm2 , provided bar is well above the required, thus provided bar is satisfactory.

Nominal stirrups/ties with dia. 10 @ 150mm will be sufficient.

Case 2: Column Design for earthquake Zone.

According to the Ethiopian Building Code of Standard (EBCS-1995), Ethiopia is categorized in four seismic zones depending on the bed rock acceleration ratio, α_0.

Bed Rock Acceleration Ratio, α_0

Zone	4	3	2	1
α_0	0.1	0.07	0.05	0.03

The seismic base shear force F_b for each main direction is determined from:

$$F_b = S_d(T_1)W$$

Where,

W is weight of dead load and live load on the Pier Column

$$S_d(T_1) = \propto \beta\gamma$$

$\propto = \propto_0 I$, I= importance factor, for public infrastructure can be taken as I= 1.2

The parameter β is the design response factor for the site and is given a by,
$\beta = \frac{1.2S}{T^{2/3}} \leq 2.5$, s= Site Coefficient, T=fundamental period of structure vibration,
For simple structure the value, $\beta = 2.5$, can be taken conservatively

The behavioral factor, $= 1.0$, can be taken for reinforced concrete framed structures.
Therefore from the assumptions made above the base shear force can be simplified to be dependent on only the bed rock acceleration coefficient as follows,
$F_b = S_d(T_1)W = \propto_0 \mathrm{I}\,\beta\gamma W = \propto_0 * 1.2 * 2.5 * 1 * W = 3 * \propto_0 * W$

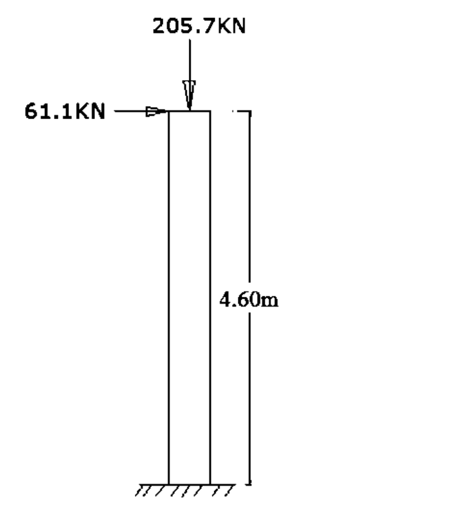
One more assumption to be made is about the distribution of the base shear on the overall height of the structure. The flume support column can be assumed as a lumped mas system at the top of the column, hence the base shear can be considered to act at the top of the pier cup.

Total Load of Structure on the Column = Flume channel Load + Pier Cup Load + Water Load

= 121 + 40.2 + 42.5 = 205.7KN

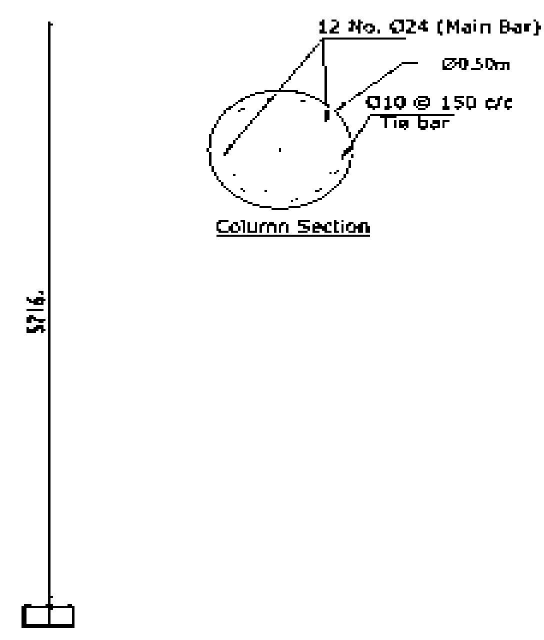
If the flume is to be constructed in Zone -4 (High seismic area) of seismicity, the base shear force will be,

$$F_b = 3*0.1*205.7 = 61.71KN$$



Forces acting on column
free body diagram with earth quake load

Using reinforced concrete Circular column with cross sectional diameter of 500mm and the loads arrangement as shown in the free body diagram, the SAP-2000 output for the design of the column main reinforcement bar is found to be as shown below,



Using 12 No., Dia. 24mm main bar, which will give gross re-bar area of 5426mm2 > 5216mm2 , provided bar is well above the required, thus provided bar is satisfactory.
Nominal stirrups/ties with dia. 10 @ 150mm will be quite sufficient.

Footing pad design

Case-1: Footing Design for none earth quake zone

In addition to the loads coming to the footing pad from super structure and substructure comports, it is also necessary to know the bearing capacity of the site for the structural design.

For the case of the example we may assume 300KN/m2 as safe bearing capacity of the foundation below the footing.

For earth quake free zone it is advisable to check the stability of the flume structure against wind load. The destabilizing wind load on the flume is calculated as,

Wind Destabilizing Moment = Wind Force * Moment arm length
From Free Body Diagram above,

Wind Force = 19KN,
Moment arm Length = 4.6+.5=5.1m
= 19*5.1 = 97KNm
Stabilizing Moment is generated by the weight of the structure, the critical situation is when the flume is in empty condition, and the vertical load to be considered is without water in the flume.

Considering also square footing with lateral dimensions of 1.5m wide and thickness of footing pad equal to 0.5m,

Total vertical load = 121 + 40.2 + 1.5*1.5*0.5*24= 188.2KN,

Stabilizing Moment = 188.2*1.5/2 = 141.15KNm

F.O.S = 141.15/97 = 1.5 < 2, need to increase the footing dimension to get F.O.S at least to be equal to 2.

Second trial for footing dimensions, 2.0m and thickness 0.5m,

Total vertical load = 121 + 40.2 + 2*2*0.5*24= 209.2KN,

Stabilizing Moment = 209.2*2/2 = 209.2KNm

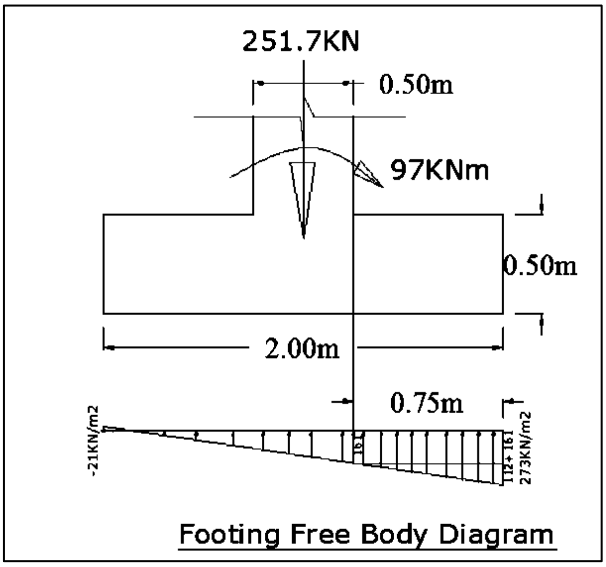
F.O.S = 209.2/97 = 2.2 >2, Satisfactory!

Stresses at Footing ends = P/A (1 + 6e/b)

For Footing Stress Calculation Water load should be added

Total Load = 209.2+42.5 =251.7KN
b = 2.0m
A= 2*2 = 4m2
e = M/P= 97/251.7 = 0.39m

Stresses at Footing ends = P/A (1 ± 6e/b) = 251.7/4 (1±6*0.39/2)
= 209KN/m2 (Max.) and -83KN/m2 (Min.)
209KN/m2 < 300KN/m2, i.e. Max. Stress is less than bearing capacity of foundation, therefore safe for bearing pressure.
Considering load factor equal to 1.35 and load distribution at footing base as shown below,



Max. Factored Bending Moment at Face of Col.=1.35*100*0.75^2/2+1.35*109*0.75^2/3
= 65.6KNm

Flexural design

Description	Value	Unit	
M(Design Moment), at Support	65.6	KNm	
Concrete Compressive Strength (Fck)	25	Mpa.	
Steel Tensile Strength (Fst)	400	Mpa.	
Section width (b)	1000	mm	
Section Height (h)	500	mm	
Clear Cover (c)	50	mm	
Assume Re-Bar Dia.	12	mm	
effective depth(d)	434	mm	
K	0.0139	<	0.167 ok!
z	428.60	mm	
As	439.82	mm2	
No. Bars	3.89		
Spacing of Bar	257	mm	
Actually Provided	200	mm	

Case-2: Footing Design for earth quake zone

Horizontal Load = 61.1KN

Moment arm at footing base = 4.6+.6 = 5.2m

Check for Stability against Over Turning.

Considering Footing Width, b = 3.7m, and thickness of footing, t= 0.6m

Total vertical load = 121 + 40.2 + 3.7*3.7*0.6*24 = 358.34KN,

Stabilizing Moment is generated by the weight of the structure = Vertical Load* b/2

= 358.34*3.7/2 = 662.92KNm

Destabilizing Moment = 61.1*5.2 = 317.72KNm

F.O.S = 662.92/317.72 = 2.1 >2.0, Satisfactory!

Stresses at Footing ends = P/A (1 + 6e/b)

b = 3.7m

A= 3.7*3.7 = 13.69m2

e = M/251.7 = 317.72/358.34 = 0.89m

For footing Stress calculation the water load should be also added

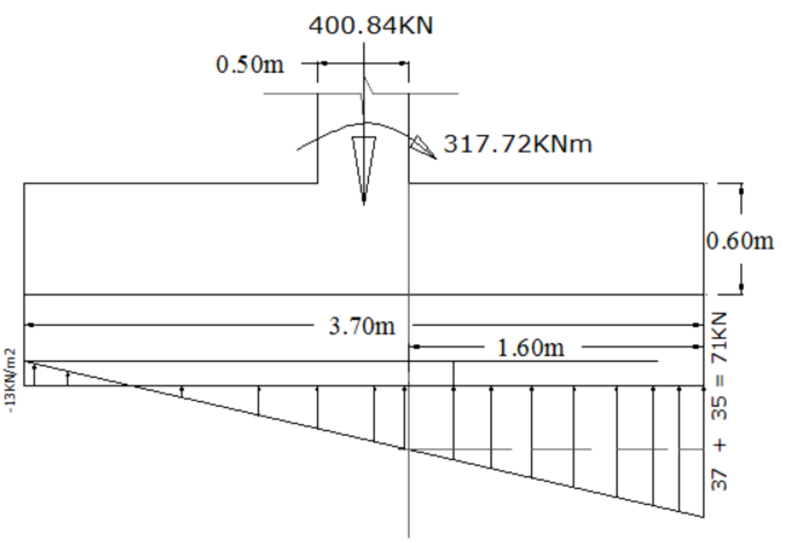
Total Vertical Load = 358.34+42.5(Water Load) = 400.84KN

Stresses at Footing ends = P/A (1 $\pm$ 6e/b) = 400.84/13.69 (1$\pm$6*0.88/3.7)

= 71.5KN/m2 & -13.0KN/m2

71.50KN/m2 < 300KN/m2, i.e. Max. Stress is less than bearing capacity of foundation, therefore safe for bearing pressure.

Considering load factor equal to 1.35 and load distribution at footing base as shown below,



Footing Free Body Diagram

Max. Factor Bending Moment at Face of Col.=1.35*35*1.6^2/2 + 1.35*37*1.6^2/3 = 103.14KNm.

Flexural design

Description	**Value**	**Unit**	
M(Design Moment), at Support	103.14	KNm	
Concrete Compressive Strength (Fck)	25	Mpa.	
Steel Tensile Strength (Fst)	400	Mpa.	
Section width (b)	1000	mm	
Section Height (h)	600	mm	
Clear Cover (c)	50	mm	
Assume Re-Bar Dia.	12	mm	
effective depth(d)	534	mm	
K	0.0145	<	0.167 ok!
z	527.10	mm	
As	562.29	mm2	
No. Bars	4.97		
Spacing	201	mm	
Actually Provide	200	mm c/c	

4.3.2 Design of bench flume

Flumes supported on a bench excavated into a hillside are called bench flumes. A bench flume is usually rectangular in shape and made of reinforced concrete (Figure 4.7) with inlet and outlet transitions to the adjoining canal. Excavation into the hillside to form the bench should be of sufficient width to provide for an access road along the downhill side unless other provisions have been made for a road.

Hydraulic design consideration

The hydraulic design of a bench flume is similar to the hydraulic design of elevated flume, however the structural design is different and discussed in the following section.

Structural design consideration for Stability

After backfill requirements have been determined, resistance to sliding from uphill backfill pressure must be provided. If the dead weight of the empty flume does not furnish the required resistance to backfill pressure on the uphill side, the base may be extended to engage additional earth weight. The ratio of the horizontal forces to the vertical forces should not exceed the coefficient of sliding. The friction coefficient for sliding of concrete on earth is taken as 0.35. Then $\frac{\sum H}{\sum V}$ should be equal to or less than 0.35. The resultant of all the forces considered should intersect the base of the flume within the middle third to provide bearing pressure over the entire base width.

Box 4-2:

Worked Example-2: Similar flume section which was used as worked example-1 for elevated flume in previous example is also used for bench flume example as shown in section drawing below including the hill cut, back fill, pipe and grating arrangements. The bench flume is to be seat at the hill side bench excavated and back filled with soil material with saturated unit weight of the soil, g_s = 20KN/m3 and coefficient of active earth pressure, K_a= 0.33.

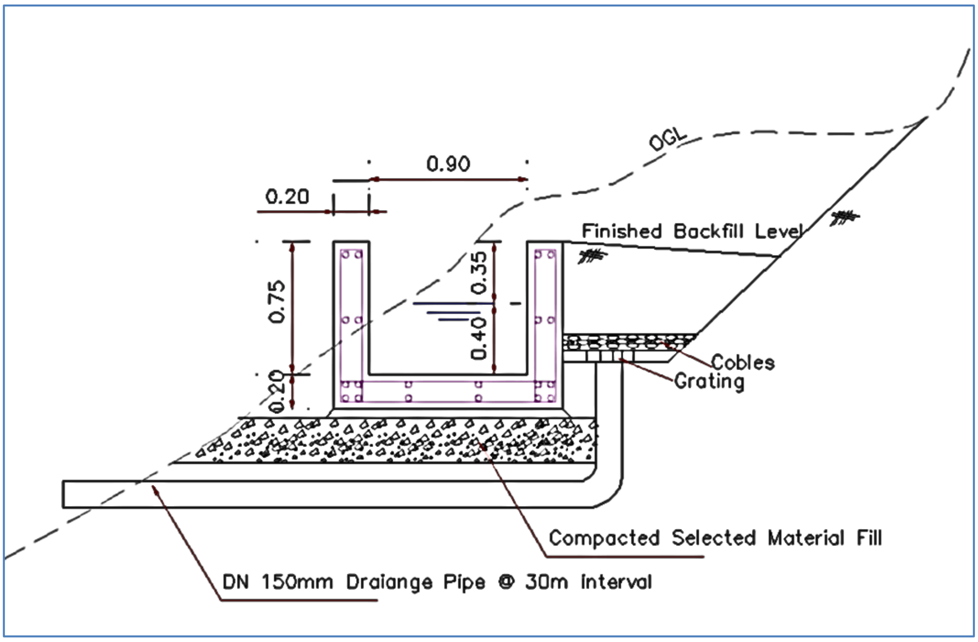


Figure 4-7: Bench flume typical cross section

Check for stability,

The lateral horizontal force is generated from the back fill soil and as there is no road facility at the upper hill side vehicular lateral load will not be considered.
Wall height subjected to lateral earth pressure, h = 0.75+0.2 = 0.95m

Soil lateral Load per meter width, $H = \frac{k_a * \gamma_s * h^2}{2}$ = 0.33*20*0.952/2 = 3KN.
Vertical Load of Empty Flume per unit width, $= A_c * \gamma_c$,

Where: Ac= Concrete cross sectional area of flume and,

Ac = 2*0.75*0.2+0.9*0.2 = 0.48m^2

γ_c = Unit weight of reinforced concrete= 24KN/m3.

$V = A_c * \gamma_c = 0.48 * 24 = 11.52$ KN

$$\frac{\sum H}{\sum V} = \frac{3}{11.52} = 0.26 < 0.35, Satisfactory!$$

Check for over turning,

The resultant should pass within middle third of the base width, the distance of the resultant from the center line of the flume is usually indicated as, e = M/V,

M=V*h/3 = 3*0.95/3 = 0.85KNm

e= M/V = 0.95/11.52 = 0.082m,

For the resultant to be within the middle third, the following condition, e $\leq \frac{b}{6}$, should be satisfied

b= 0.9+2*0.2 = 1.3m, b/6 = 1.3/6 = 0.22

Since, e (0.082m) < b/6 (0.22m), the flume is safe against overturning.

The flume section is already designed in the previous example for water pressure, and now the bending moment (M) due to the side soil lateral load, when the flume is empty condition will be checked as follows:

$$M = H * \frac{h^3}{6} = 3 * \frac{0.75^3}{6} = 0.21KNm$$

Multiplying the moment with load factor of 1.5, Design moment, M_d = 1.5*0.21 = 0.32KNm,
This moment is much more less than the capacity of the section to resist the design moment; hence the provided reinforcement for the case of elevated flume is quite sufficient.

4.3.3 Design of inverted siphon

I – Hydraulic design

A. Data for design

Hydraulic Characteristics of the canal

Discharge, Q m^3/sec.	Bed Width B(m)	Water Depth d(m)	Side Slope	Velocity (m/sec.)
0.4	0.7	0.5	1.2H:1.0V	0.61

Length of Pipe: =8.0m
Pipe Diameter: =600mm
Pipe Material: =PVC
Upstream Canal Bed level = 1800.00m

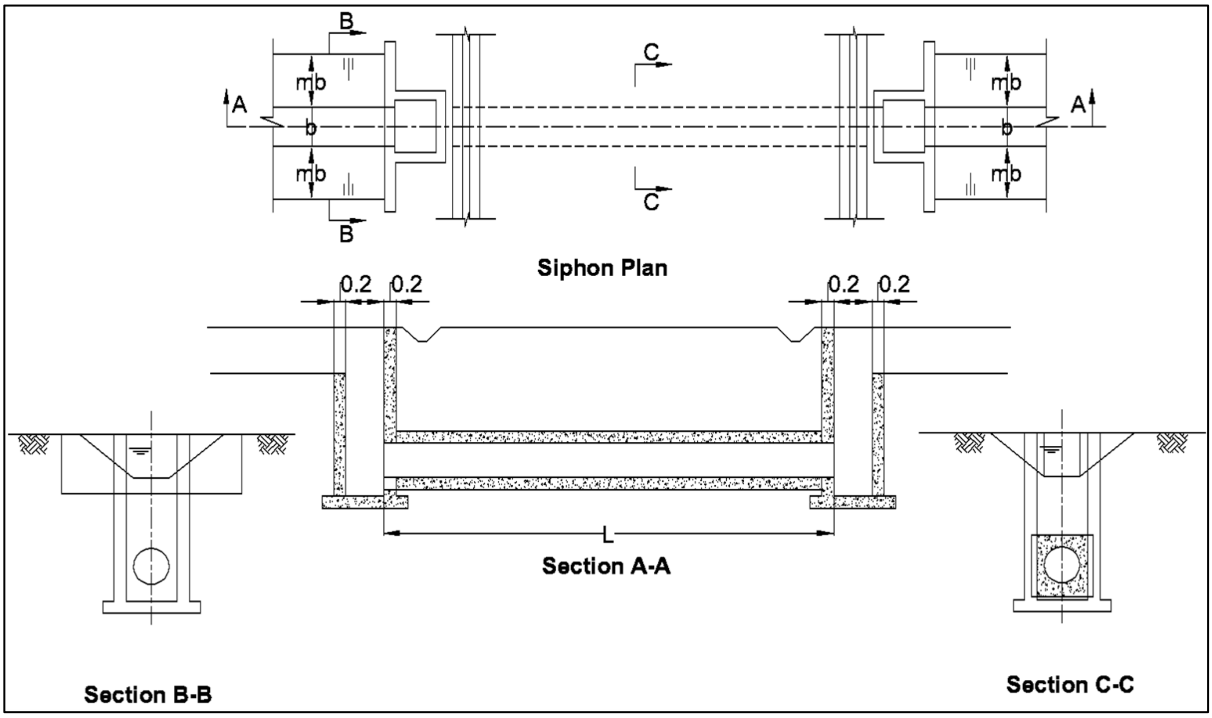


Figure 4-8: Inverted siphon plan and sections

B. Flow hydraulics of barrel and head losses

Cross sectional Area,	$A = \pi D^2/4$	= 3.14*0.6^2/4	= $0.283m^2$
Velocity of flow in Barrel,	v= Q/A	= 0.4/0.283	=1.413
Coefficient of Roughness,	n= 0.014		

Inlet loss coefficient = 0.5

Outlet loss Coefficient = 1.0

Inlet loss,	$h_i = 0.5(v^2/2g)$	=0.5(1.413)2/2*9.81	= 0.051m
Coefficient of Frictional Loss,	$f = 124.5\, n^2/D^{1/3}$	= $124.5*0.014^2/0.6^{1/3}$	= 0.029
Frictional Loss in Barrel,	$h_l = f * L/D\, (v^2/2g)$	= $0.029*8/0.6*(1.413^2/2*9.81$	= 0.040m
Outlet loss	$h_o = 1.0(v^2/2g)$	= $1.0(1.413)^2/2*9.81$	= 0.102m
Total Head Loss,	$h_T = h_i+h_l+h_o$	= 0.051+0.04+0.102	= 0.193m

Adding 10% extra head as contingency, the total head loss becomes = 1.1*0.193m = 0.212m

D/s Water Level =Upstream Water Level–Total Head Loss =1800+0.5-0.212 =1800.288m

D/s Canal Bed Level= 1800.288-0.5 = 1799.788m

II – Structural design

A. Design data

Inlet and outlet drop manholes are constructed out of reinforced concrete, Concrete Class C-25, and reinforcement bar f_y= 400Mpa.

The pipe used is PVC pipe incased with reinforced concrete to protect the pipe against traffic load. Minimum thickness of encasement concrete is 200mm.

The critical loading condition for design of the manhole is when inside is empty and lateral soil load is acting on the barrel wall side.

The bottom depth of the manhole from top ground level is = 3.0m

Taking unit weight of soil equal to 18KN/m2, and lateral pressure coefficient, 0.33,

The lateral soil pressure at bottom is therefore = 0.33*18*3 = 17.82KN/m2

Longer side wall width from wall center to center = 1.0m

Considering 1.5 load factor, the ultimate moment at wall face and mid span of the wall can be calculated as:

$M_{wall\ end}$ =1.5* $wl^2/12$ = 1.5*17.82*1^2/12 = 2.23KNm

$M_{wall\ span}$ = 1.5*wl^2/24 = 1.5*17.82*1^2/24 = 1.11KNm

Max Shear Force = 1.5*wl/2 = 1.5*17.82*1/2 = 13.37KN

The values of the actions calculated are so small that the sectional capacity is by far greater the acting moment and shear force. Hence minimum requirement will be sufficient. Thus use Dia. 10 @ 240mm center to center for wall and base slab.

The bending moment and shear force resistance capacity of the section with the minimum reinforcement bar provision are 17KNm and 108KN respectively.

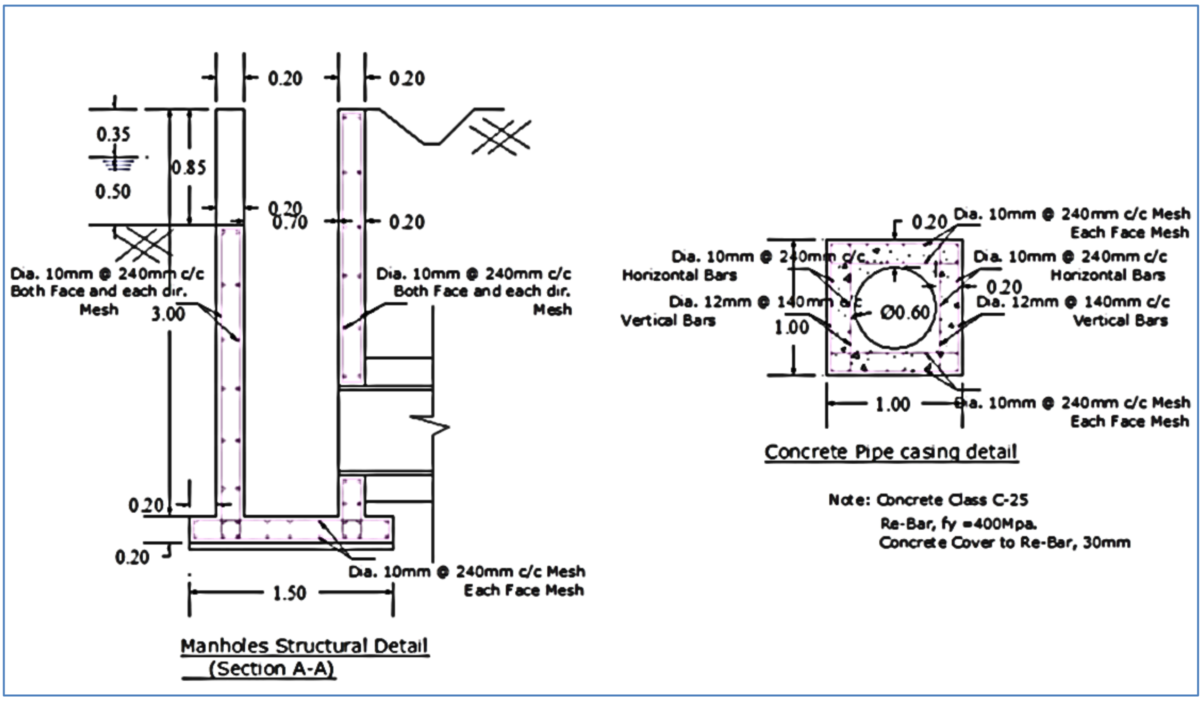


Figure 4-9: Inverted siphon inlet, outlet manholes and pipe casing structural detail

4.3.4 Design of drop structure

I – Hydraulic design

A. Data for design

Hydraulic Characteristics of the canal at upstream and downstream of drop

Discharge, Q M^3/sec.	Bed Width B(m)	Water Depth d(m)	Side Slope	Velocity (m/s)	Drop Height , Z (m)
0.4	0.7	0.5	1.2H:1.0V	0.61	1.0

Upstream Bed Level = 1800.00

Down Stream Bed Level = 1800.00-1.00 = 1799.00m

B. Design Calculation

Critical flow Hydraulics

B1. Width of drop, $b_c = \frac{0.734Q}{d_0^{3/2}} = \frac{0.734*0.4}{0.5^{3/2}}$ = 0.83m, use, b_c = 0.85m

B2. Unit discharge, q = Q/bc = 0.4/0.85 = = 0.471m3/sec./m

B3. Critical Depth, $d_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{0.471^2}{9.81}\right)^{1/3}$ = 0.283m

B4. Critical Velocity, Vc = q/dc = 0.471/0.261 = 1.664m/s

B5. Normal Velocity Head, $h_{vo} = \frac{v_0^2}{2g} = \frac{0.61\text{^}2}{2g}$ = 0.019m

B6. Critical Velocity Head, $h_{co} = \frac{v_c^2}{2g} = \frac{1.664\text{^}2}{2g}$ = 0.141m

B7. Total Head at u/s of drop, He = d+ h_{vo} = 0.5+0.019 = 0.519m

B8. Stilling Basin Length (L), $= 3\sqrt{H_e F}$, In general F= Z = 1.0m

$L = 3\sqrt{0.519 * 1}$ = 2.2

Take, L= = 2.2m

B9. Stilling Basin Width (B)

$B = \frac{18.46\sqrt{Q}}{Q+9.91} = \frac{18.46\sqrt{0.4}}{0.4+9.91}$ = 1.13, Take B = 1.20m

B10. Cutoff Wall

i) Upstream (u/s) Cut off Wall

Depth of upstream cut off wall = u/s FSD/3+0.60 = 0.5/3+0.6 = 0.77m
Provide u/s cutoff wall, 0.80m deep and, thickness of wall = 0.20m

ii) Downstream (d/s) Cut off Wall

Depth of d/s cut off wall = d/s FSD/2+0.60 = 0.5/2+0.6 = 0.85m
Provide d/s cutoff wall, 0.85m deep and, wall thickness = 0.20m

B11. Calculation for Total Apron Length and d/s floor thickness
Safe exit gradient (GE) = 1/5 = 0.20 (for coarse sand Material)
Maximum Static Head is exerted when water is stored up to crest level on u/s and no water on d/s, Maximum Static Head , H = (Crest level-Downstream cistern level)
Therefore, H= 1800.00-1799.00 = 1.00m
Depth of d/s cutoff wall, d= 0.85m

$GE = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}; \quad \frac{1}{\pi\sqrt{\lambda}} =$ GE*d/H= 0.20*0.85/1.0 = 0.17

Hence, $\lambda = 1/(3.14*0.17)^2$ = 3.51

$\alpha = \left((2\lambda - 1)^2 - 1\right)^{1/2}$ = $((2*3.51-1)^2-1)^{1/2}$ =5.936

Total Floor Length Required = αd = 5.936*0.85 = 5.00m
Upstream floor length required = Total length – d/s cistern Length
= 5.00-2.2 =2.8m
Unbalanced head at toe = (H/b)* d/s floor length= (1/5)*2.2 = 0.44m

The drop can be designed either constructed out of stone masonry wall which is common practice for small scale irrigation projects or alternatively can be construction with reinforced concrete structure. The two options will be separately discussed in this example.

i) Vertical Drop with Stone Masonry Construction

Unit weight of stone masonry is equal to 21KN/m^3 (from Table 3.3)
Floor thickness required at toe= Unbalanced Head at Toe/1.21 = 0.36m

Thus, provide masonry thickness of 0.4m at the stilling basin floor.

For the upstream apron, masonry thickness equal to 0.25m will be adequate.

Lip Height = dc/2 = 0.238/2 = 0.119m
Provide Lip Height = 0.15m

Canal protection and freeboard

Upstream Canal Protection length, L is calculated as,
L= 1.2 + 3/2 $Q^{1/2}$ = 1.2+3/2*$0.4^{1/2}$ = 2.15m
Provide, protection length in the upstream canal =2.20m
Provide, protection length for d/s canal in the proportion of cut of walls
i.e. (0.85/0.80 = 1.06), = 1.06*2.20 = 2.35m
Provide u/s and d/s walls Free board =0.40m

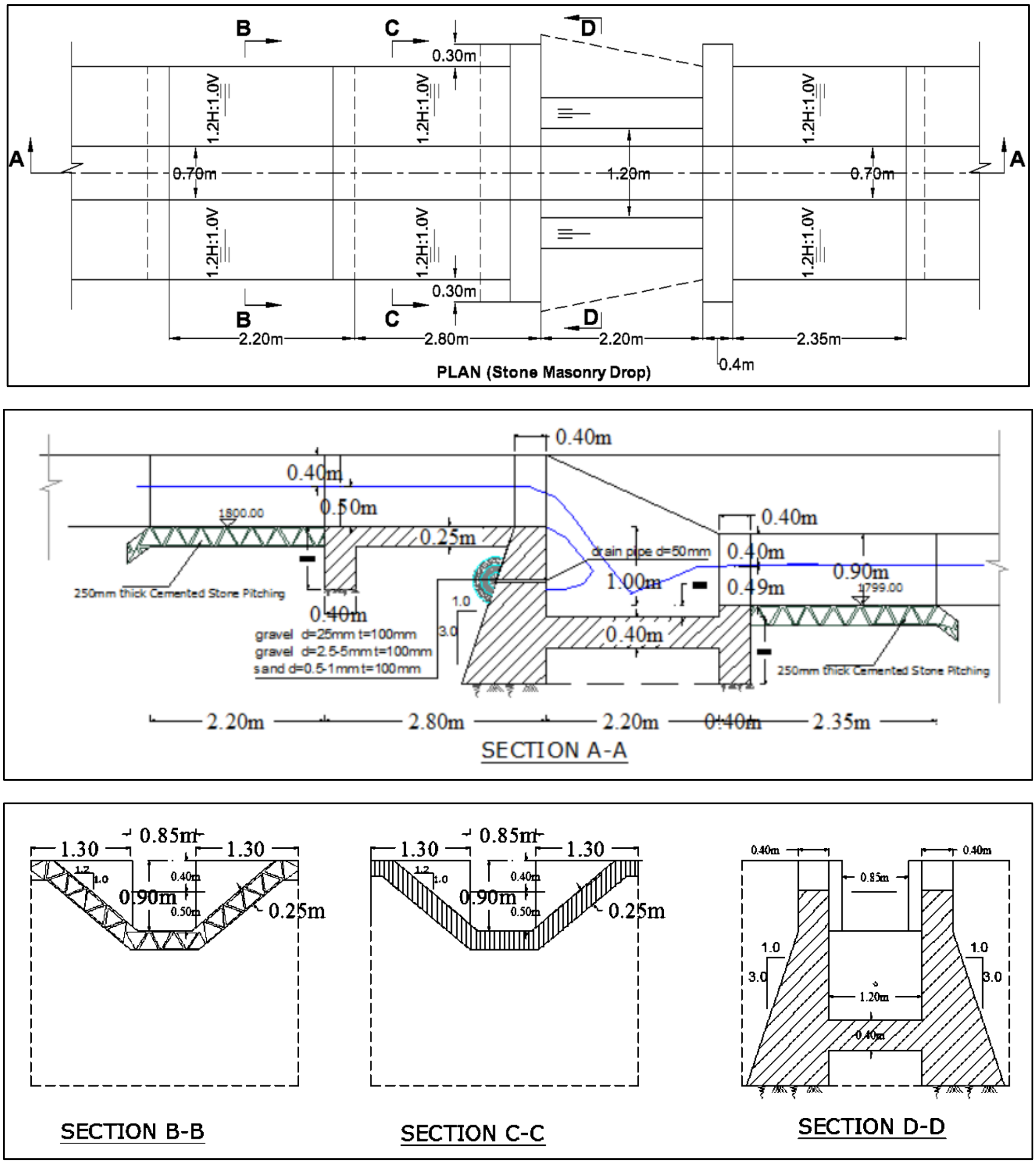


Figure 4-10: Vertical drop plan and sections (Masonry)

ii) Vertical Drop with Reinforced Concrete Construction

Unit weight of Reinforced concrete is equal to $24KN/m^3$ (from Table 3.3)
Floor thickness required at toe= Unbalanced Head at Toe/1.24 = 0.36m

For the purpose of adapting economical floor section, wall contribution for uplift force resistance shall be considered together with the floor slab.
If the thickness of the floor slab is 200mm, the total equivalent floor thickness including the side walls = 0.2+ 2*1.55*.2/1.6 = 0.79m > 0.36m, 200mm base slab thickness is adequate.

Provide also u/s apron thickness equal to 0.20m.

Provide similar dimensions of Canal protection, lip height and free board which is used for masonry construction option.

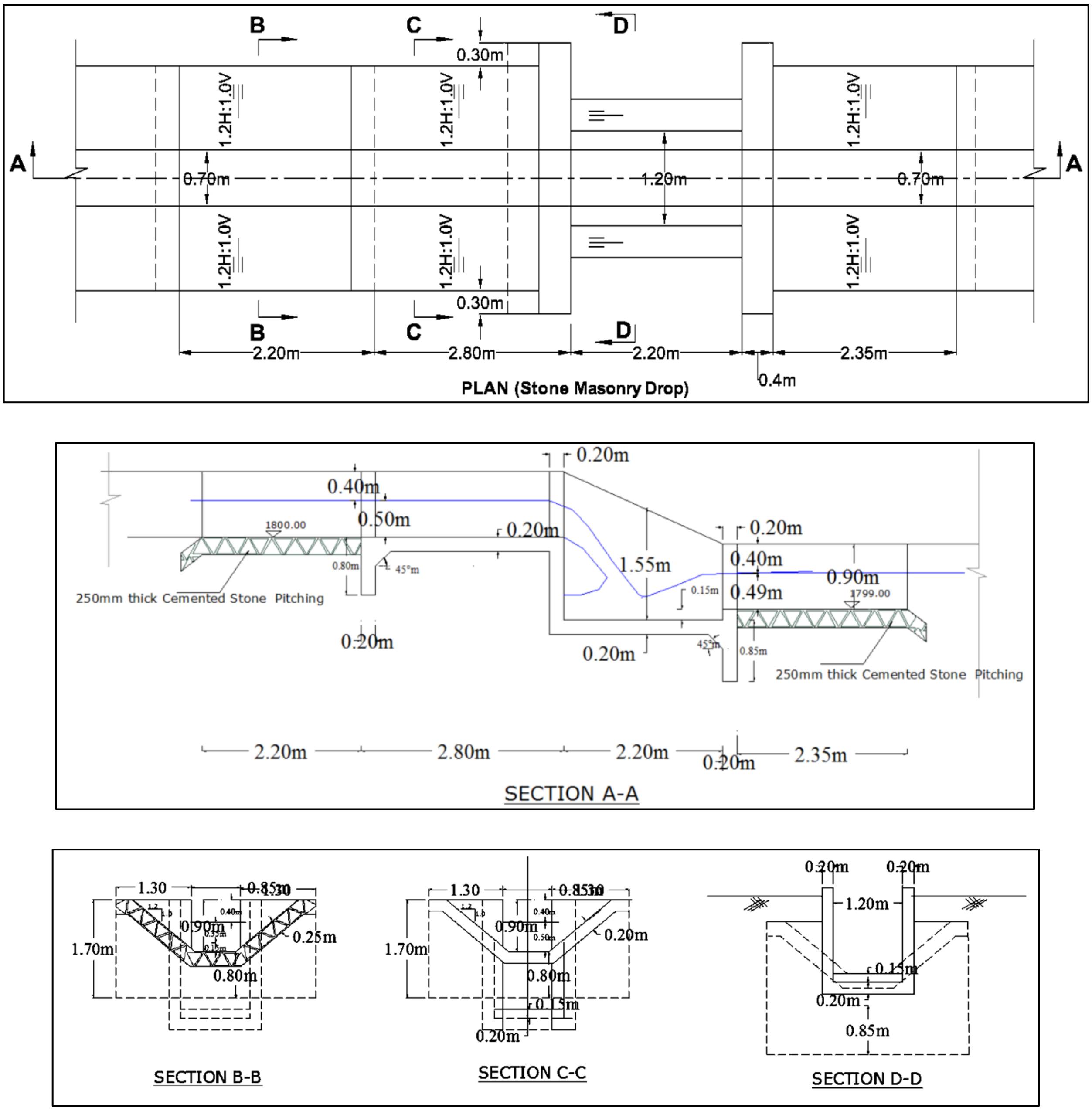


Figure 4-11: Vertical Drop Plan and sections (Concrete)

II – Structural design

The structural Design involves the determination of the reinforcement bar requirement for each structural members of the drop.

For the upstream apron, minimal requirement shall be provided as there is no major actions acting in the cistern. The minimum requirement is to be provided for top reinforcement as per table 3.8.

As = 3.25* 200/2 = 325mm2/m

Using dia. 10mm bar, spacing of bars is, 1000/(325/3.14*10^2/4) = 240mm c/c

For the cutoff wall provide Dia. 10mm bar at 240mm c/c both faces.

For the downstream cistern,

The unbalanced pressure is generated from the side walls, Thus, upward pressure = 2* h*t*24/B_b = 2*1.53*0.2*24/1.6 =9.18KN/m2/m

Where, h=wall height = 1.53m

t = Wall thickness = 0.20m

B_b= Base slab Width = 1.60m

BM at base slab end with 1.5 load factor =1.5* $wl^2/12$= 1.5*9.18*1.53^2/12 = 2.90m KNm

Flexural design

Description	**Value**	**Unit**
M(Design Moment), Wall Support	2.9	KNm
Concrete Compressive Strength (Fck)	25	Mpa
Steel Tensile Strength (Fst)	400	Mpa
Section Width (B)	1000	Mm
Section Height (H)	200	Mm
Clear Cover (C)	50	Mm
Assume Re-Bar Dia.	10	Mm
Effective Depth (D)	135	Mm
K	0.0064	<0.167
Z	134.24	Mm
As	62.08	Mm^2
No Bars/M	0.79	No
Bar Spacing	1264.52	
Actually Provided	240	C/C

OK! Ductility

Min. Requirement

For Vertical Wall design at the downstream stilling basin,

Consider the wall height at mid height, (h) = 1.65m,

d/s side Wall Span length, l = 3.6+2*.1 = 3.8m,

Unit weight of soil behind the wall, (γ_s) = 18KN/m3

Lateral Pressure Coefficient, (Ka) = 0.33

Lateral earth pressure at bottom of wall = Ka*γ_s*h=0.33*18*1.65 = 9.80KN/m2

Lateral Earth Pressure at 1.0m above wall base = 0.33*18*(1.65-1.0) =3.86KN/m2

Average Earth pressure between bottom wall and 1.0m above bot. Wall = (9.80+3.86)/2

= 6.83 KN/m2

Vertical Bending moment at Wall Base using load factor (1.5)=1.5*9.80*1.65^2/6 = 6.67KNm

Horizontal Bending M. near wall base at end = 1.5*$wl^2/12$ = 1.5*6.83*$2.4^2/12$ = 4.92KNm

Horizontal Bending M. near wall base at mid span = 1.5*$wl^2/24$=1.5*6.83*$2.4^2/24$= 2.46KNm

Vertical Shear force at Wall Base using load factor (1.5) = 1.5*9.80*1.65/2 = 12.13KN

Hor. Shear force at Wall Base using load factor (1.5) = 1.5*6.86*2.4/2 =12.35KN

Flexural design

Description	Value	Unit
M(Vertical Bending Moment), Wall Support	6.67	KNm
Concrete Compressive Strength (Fck)	25	Mpa
Steel Tensile Strength (Fst)	400	Mpa
Section Width (B)	1000	Mm
Section Height (H)	200	Mm
Clear Cover (C)	50	Mm
Assume Re-Bar Dia.	10	Mm
Effective Depth (D)	135	Mm
K	0.0146	<0.167
Z	133.23	Mm
As	143.86	Mm^2
No Bars/M	1.83	No
Bar Spacing = 1000/No Bars	545.68	mm c/c
Actually Provided	240	mm C/C

Flexural design

Description	Value	Unit
M(Design Moment), at Support	4.92	KNm
Concrete Copressive Strength (Fck)	25	Mpa.
Steel Tensile Strength (Fst)	400	Mpa.
Section width (b)	1000	mm
Section Height (h)	200	mm
Clear Cover (c)	50	mm
Assume Re-Bar Dia.	10	mm
effective depth(d)	135	mm
K	0.0108	<0.167
z	133.70	mm
As	105.74	mm2
No. Bars	1.35	
Spacing of Bar	742	mm
Actually Provided	240	mm

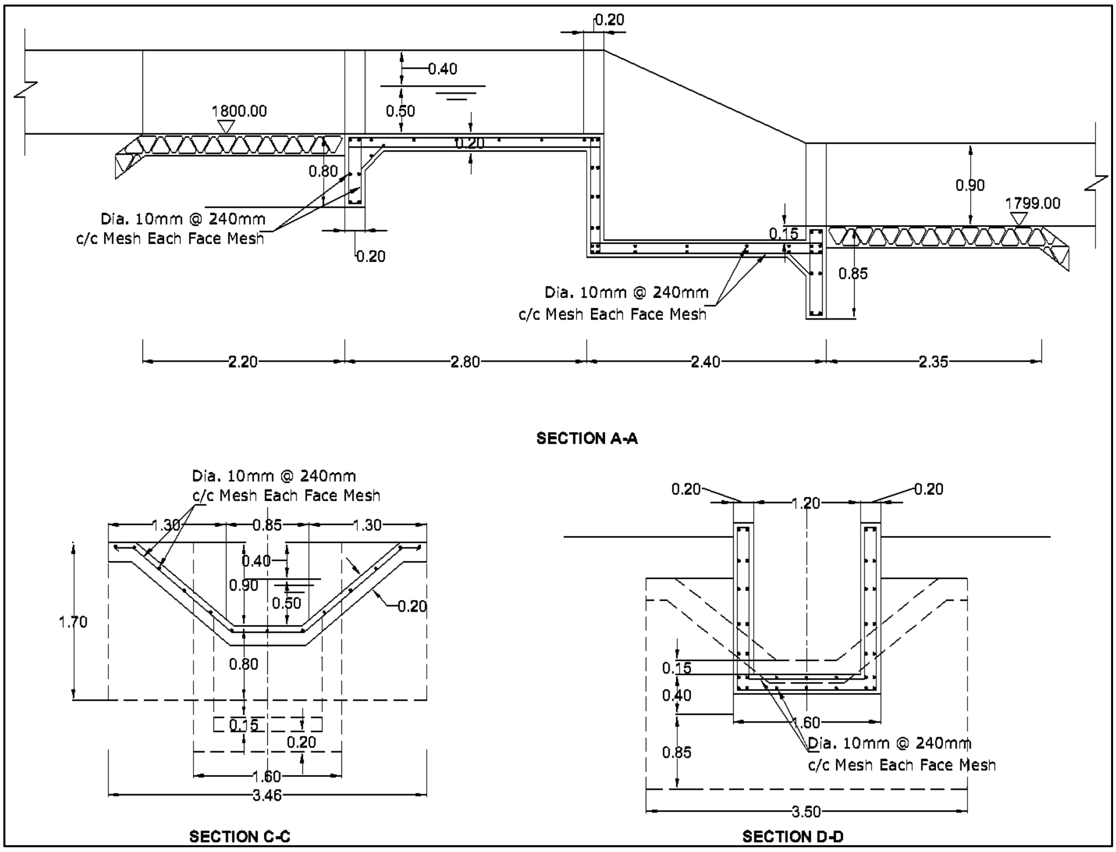


Figure 4-12: Vertical drop structural detail

Shear resistance capacity of the section is calculated based on Equation 2.14 as follows,

Acting shear on str.	Vs	12.35 KN
Concrete class	Fcu	25 N/mm^2
Long re-bar top	Dia. 10 at 240 mm	327.08mm^2
Long re-bars bottom	Dia. 10 at 240 mm	327.08 mm^2
Total long re-bar	As	654.17 mm^2
Total	Depth (D)	200 mm
Effective	Depth (d)	142 mm
	Width (b)	1000 mm
	Steel ratio (q) %	0.46
Shear stress	vc	0.63 N/mm^2
Shear force	Vc = vc * d*b/10^3	89.79 KN
		$V_c > V_s$, Safe for shear

4.3.5 Design of inclined drop structure (Drop height less than 4.5m)

I – Hydraulic design

A Data for design

Hydraulic Characteristics of the canal at upstream and downstream of drop

Discharge, Q M3/sec.	Bed Width B(m)	Water Depth d(m)	Side Slope	Velocity (m/sec.)	Drop Height (m)
0.4	0.7	0.5	1.2H:1.0V	0.61	4.00

Upstream Bed Level = 1800.00

Down Stream Bed Level = 1800.00- 4.00 = 1796.00m

B Design Calculation

Critical flow Hydraulics

B1. Width of drop, $b_c = \frac{0.734Q}{d_0^{3/2}} = \frac{0.734*0.4}{0.5^{3/2}}$ = 0.83m, use, b_c = 0.85m

B2. Unit discharge, q = Q/bc = 0.4/0.85 = = 0.471m^3/sec./m

B3. Critical Depth, $d_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{0.471^2}{9.81}\right)^{1/3}$ = 0.283m

B4. Critical Velocity, Vc = q/dc = 0.471/0.283 = 1.664m/sec.

B5. Normal Velocity Head, $h_{vo} = \frac{v_0^2}{2g} = \frac{0.61^\wedge 2}{2g}$ = 0.019m

B6. Critical Velocity Head, $h_{co} = \frac{v_c^2}{2g} = \frac{1.664^\wedge 2}{2g}$ = 0.141m

B7. H_L=4.00m, q= 0.471m3/sec./m, y1 can be calculated from the following Equation by trial and error or goal seek in excel spread sheet

$$\frac{8*q^2*H_L}{g} = \left(-1.5*y_1 + \left(\left(\frac{y_1^2}{4}\right)+\left(\frac{2*q^2}{g*y_1}\right)\right)^{0.5}\right)^3 * \left(0.5*y_1 + \left(\left(\frac{y_1^2}{4}\right)+\left(\frac{2*q^2}{g*y_1}\right)\right)^{0.5}\right)$$

y_1 = 0.048m, v1 = q/y1 = 0.471/0.048 = 9.81m/sec.

B8. $y_2 = \frac{y_1}{2}\left[-1+\sqrt{1+8F_1^2}\right]$,

$F_1 = V_1/(gy_1)^{1/2}$ = 9.81/ $(9.81*0.048)^{1/2}$ =14.3

y_2 = 0.942m

B9. Downstream Apron Length (L)

L= 4y2 or L= 5(y2-y1), Maximum of the two will be provided

L= 4*0.942 = 3.77m, or L= 5(0.942-0.048) = 4.47m

Provide, L= 5.0m

B10. Cutoff Depths

i) Upstream (u/s) Cut off Wall

Depth of upstream cut off wall = u/s FSD/3+0.60 = 0.5/3+0.6 = 0.77m
Provide u/s cutoff wall, 0.80m deep and wall thickness = 0.20m

ii) Downstream (d/s) Cut off Wall

Depth of d/s cutoff wall = FSD/2+0.60= 0.5/2+0.6 = 0.85m
Provide d/s cutoff wall, 0.85m deep will bring too long total apron length, it is therefore wise to provide deeper cutoff wall, so that the apron length will be made shorter. So provide d/s cutoff wall = 2.00m

B11. Exist Gradient and Total apron length

Safe exit gradient (GE) = 1/5 = 0.20 (for Coarse sand Material)
Maximum Static Head, H = (Crest level-d/s cistern level)
Therefore, H= 1800.00-1796.00 = 4.0m

$$GE = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}};$$

Depth of d/s cutoff wall, d= 2.00m

$$\frac{1}{\pi\sqrt{\lambda}} = \quad 0.20*2.0/4.0 = \quad = 0.10$$

λ = 1/(3.14*0.10)^2 = 10.14

$\alpha = \left((2\lambda - 1)^2 - 1\right)^{1/2}$ = ((2*10.14-1)^2-1)$^{1/2}$ =19.26

Total Floor Length Required = αd = 19.26*2.0 = 38.50m

Providing a sloped Apron, with 6H: 1.0V, Inclined Chute length

= 6*(Z+Lip Height)

= 6*(4.442) = 26.70m

(Z= 4.0m, Lip Height = y2- D/c FSL = 0.942-0.5 = 0.442m)

Upstream floor length = Total length – (Sloped Chute Length +d/s cistern Length)

= 38.5-(26.7+ 5.00) = 6.80m

B12. Floor Thickness

Pressure variation calculation

(i) At u/s cut off wall

Total Floor Length (b)	=	38.50m
Depth of u/s cut off wall (d)	=	0.80m
1/α = d/b	=	0.02
α = b/d	=	48.125
λ= (1+√(1+α^2))/2	=	24.568
$\phi_E = \frac{1}{\pi}cos^{-1}(\frac{\lambda-2}{\lambda})$	=	13
$\phi_D = \frac{1}{\pi}cos^{-1}(\frac{\lambda-1}{\lambda})$	=	9
$\phi_{C1} = 100 - \phi_E$	=	87
$\phi_{D1} = 100 - \phi_D$	=	91

Assuming floor thickness at u/s end as = 0.20m

Correction for floor thickness at u/s end = (91-87)*0.2/0.8 = 1.00%

ϕ_{C1}(Corrected) = 87.00+1.00 =88.00%

(ii) At d/s cut off wall

Total Floor Length (b)	=	38.50m
Depth of d/s cut off wall (d)	=	2.0m
1/α = d/b	=	0.052
α = b/d	=	19.250
λ= (1+√(1+α^2))/2	=	10.138
$\phi_E = \frac{1}{\pi}cos^{-1}(\frac{\lambda-2}{\lambda})$	=	20
$\phi_D = \frac{1}{\pi}cos^{-1}(\frac{\lambda-1}{\lambda})$	=	14

Assuming floor thickness at d/s end as = 0.40m
Correction for floor thickness at d/s end = (20-14)*0.4/2 =1.20%
$\emptyset_E$(Corrected) = 20.0– 1.2 =18.80%

Pressure variation = (88-18.80)/38.2 = 1.797%
Calculation for Floor Thickness

(i) Downstream Floor

Floor thickness from d/s face of d/s cut off wall at = 5.0m
ρ = 18.8 +5.00*1.797 = 27.785%
Floor thickness = (ρ*H)/((sp.gr. – 1)*100) = 0.896m
Provide Minimum as = 0.90m

Floor thickness from d/s face of d/s cut off wall at = 18.35m
ρ = 18.8 +5.00*1.797 = 51.775%
Floor thickness = (ρ*H)/((sp.gr. – 1)*100) = 1.670m
Provide Minimum as = 1.700m

Floor thickness from d/s face of d/s cut off wall at = 6.69m
ρ = 27.79 +6.69*0.896 = 30.822%
Floor thickness = (ρ*H)/((sp.gr. – 1)*100) = 0.994m
Provide Minimum as = 1.000m

Wall contribution for uplift force resistance shall be considered together with the floor slab.

a) Cistern Floor Thickness,

Floor thickness required = 0.90m
If the thickness of the floor slab is 450mm and wall thickness 250mm, wall height 1.55m, Base slab Width 1.60m,

The total equivalent floor thickness including the side walls = 0.45+ 1.55*2*.25/1.6 = 0.93m
0.93m > 0.90m, hence 450mm base slab thickness is adequate.

b) Chute Base Slab thickness at mid length,

Floor thickness required = 1.00m
If the thickness of the floor slab is 450mm and wall thickness 250mm, wall height 1.55m,
Base slab Width= 0.85+2*0.25 = 1.35m
The total equivalent floor thickness including the side walls = 0.45+ 1.55*2*.25/1.35 = 1.024m > 1.0m, 450mm base slab thickness is adequate.

II – Structural Design

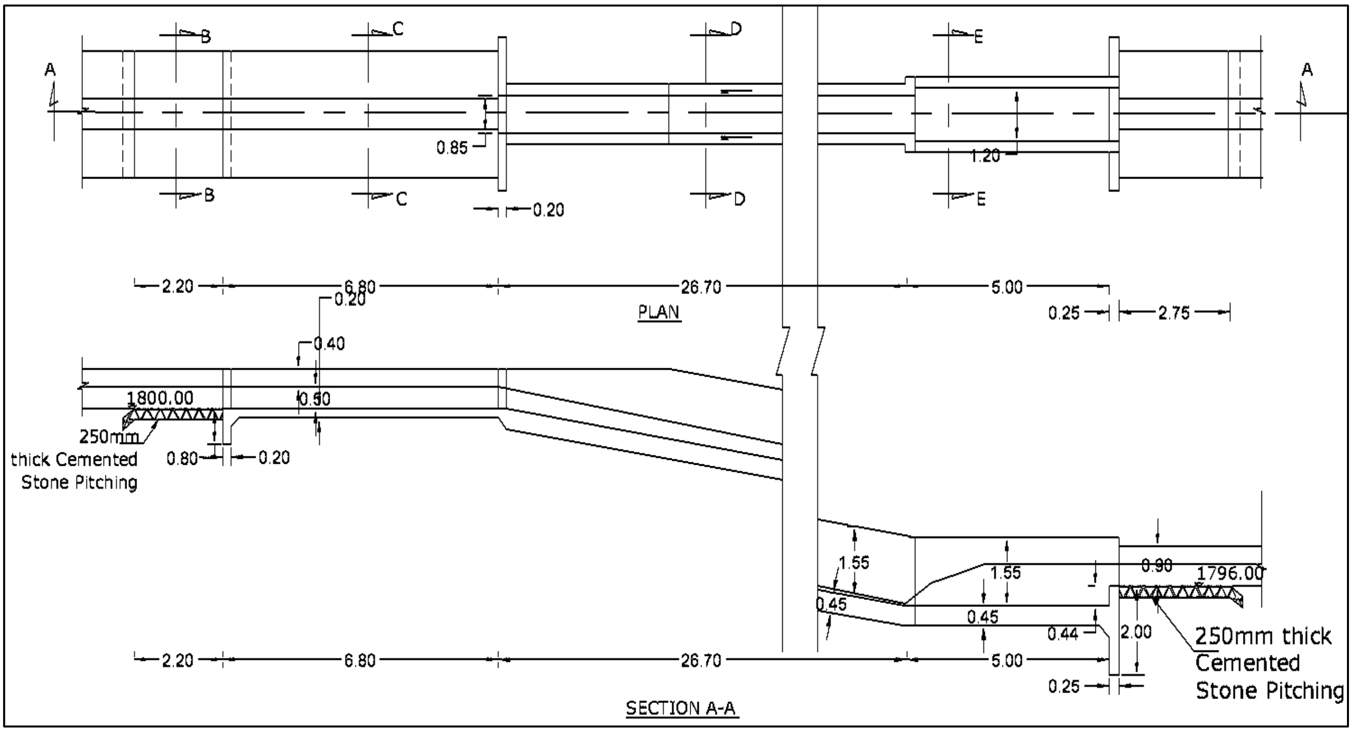
Minimum Reinforcement requirement for 200mm, 250mm and 450mm thick slabs.
For 200mm thick slab at drop inlet, Dia. 10 @ 240mm c/c (same as Previously calculated for Ver. Drop)
For 250mm thick Slab at wall of chute and stilling basin = 3.25*250/2 = 406.25mm2
Using Dia. 10 bar at 190mm the area of bar in 1000mm width, both face = 413.13mm2 (Adequate)

For 450mm thick base slab at chute and stilling basin base slab,

Top face = 3.25*450/2 = 731.25mm2

Bottom face = 3.25*100 = 325.00mm2

Using Dia. 12 @ 150mm c/c for Top face and Dia. 10 @ 240mm c/c for Bottom Face would be adequate.



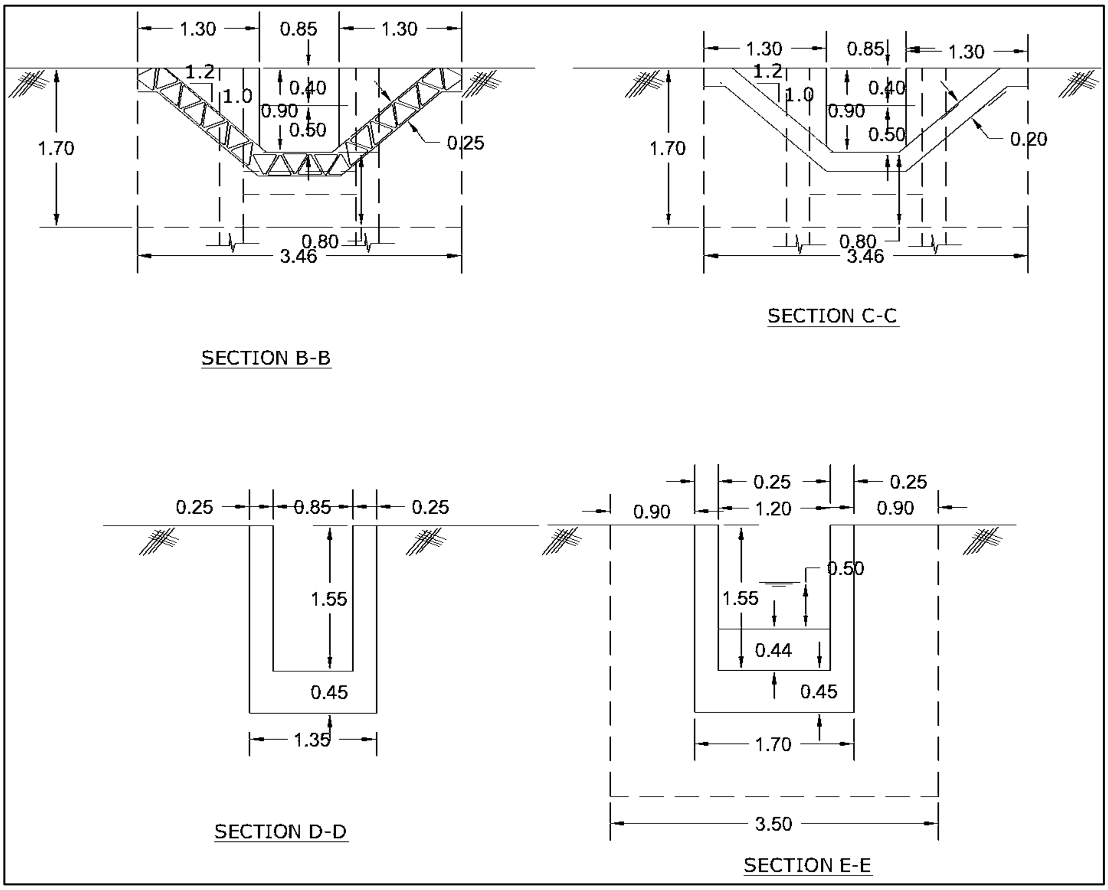


Figure 4-13: Inclined drop plan and sections according to the design example

Chute and stilling basin wall design,

Wall height, = 1.65m,

Unit weight of soil behind the wall, = 18KN/m3

Lateral earth pressure at bottom of wall = Ka*g_s*h=0.33*18*1.65 = 9.80KN/m2

(Lateral Pressure Coefficient, Ka= 0.33, g_s =18KN/m2, h = 1.65)

Lateral Earth Pressure at 1.0m above wall base = 0.33*18*(1.65-1.0) =3.86KN/m2

Average Earth pressure between bottom wall and 1.0m above bott. Wall = (9.80+3.86)/2 = 6.83 KN/m2

Vertical Bending moment at Wall Base using load factor, 1.5

= 1.5*9.80*1.65^2/6 = 6.67KNm

Horizontal Bending M. near wall base at end = 1.5*wl^2/12 = 1.5*6.83*5.25^2/12 = 23.53KNm

(Horizontal Moment is considered at stilling basin with l = 5.25 (effective wall width)

Horizontal Bending M. near wall base at mid span = 1.5*wl^2/24=1.5*6.83*5.25^2/24= 11.75KNm

Vertical Shear force at Wall Base using load factor, 1.5 = 1.5*9.80*1.65/2 =12.13KN Hor. Shear force at Wall Base using load factor, 1.5 = 1.5*6.86*5.25/2 =27.00KN

Flexural design

Description	Value	Unit
M(Design Moment), Wall Support	6.67	KNm
Concrete Compressive Strength (Fck)	25	Mpa
Steel Tensile Strength (Fst)	400	Mpa
Section Width (B)	1000	Mm
Section Height (H)	250	Mm
Clear Cover (C)	50	Mm
Assume Re-Bar Dia.	10	Mm
Effective Depth (D)	185	Mm
K	0.0078	<0.167
Z	183.72	Mm
As	104.33	Mm^2
No Bars/M	1.33	No
Bar Spacing = 1000/No Bars	752.45	mm c/c
Actually Provided	190	mm C/C

Flexural design

Description	Value	Unit
M(Design Moment), at Support	23.53	KNm
Concrete Compressive Strength (Fck)	25	Mpa.
Steel Tensile Strength (Fst)	400	Mpa.
Section width (b)	1000	mm
Section Height (h)	250	mm
Clear Cover (c)	50	mm
Assume Re-Bar Dia.	10	mm
effective depth(d)	185	mm
K	0.0275	<0.167
z	180.40	mm
As	374.81	mm2
No. Bars	4.77	
Spacing of Bar	209	mm
Actually Provided	180	mm

4.4 CANAL REGULATING STRUCTURES

A regulating structure in an open irrigation system is used either for regulating the flow passing through the structure or to control the elevation of the upstream canal water surface, or to do both. Structures which perform these functions include: checks, check-drops, turnouts, division structures, check inlets, and control inlets. Regulation is achieved with weirs, control inlets, stop logs, and slide gates.

Mainly for small irrigation schemes; division box, turn out and off-take structures are used to distribute balanced amount of water from parent canal to secondary canal, tertiary canal and field canal respectively. Thus, the hydraulic and structural design procedures for division box, turnout and off take remain the same.

4.4.1 Flow division Structures

Division structures are used for dividing the flow from a supply pipe or channel among two or more channels or pipes. The division structure may be a separate structure or it may be the outlet of a siphon, a drop, or a turnout from which further diversion is required. If measurement is not required at the point of division, the flow through the structure may be directed through the various outlets with gates or stop logs. If the flow must be measured and the required head is available, weirs may be used to proportion the flow. Figure 4.14 below shows typical plan of division box with the slots which could accept the stop logs, gates in frames or movable weirs.

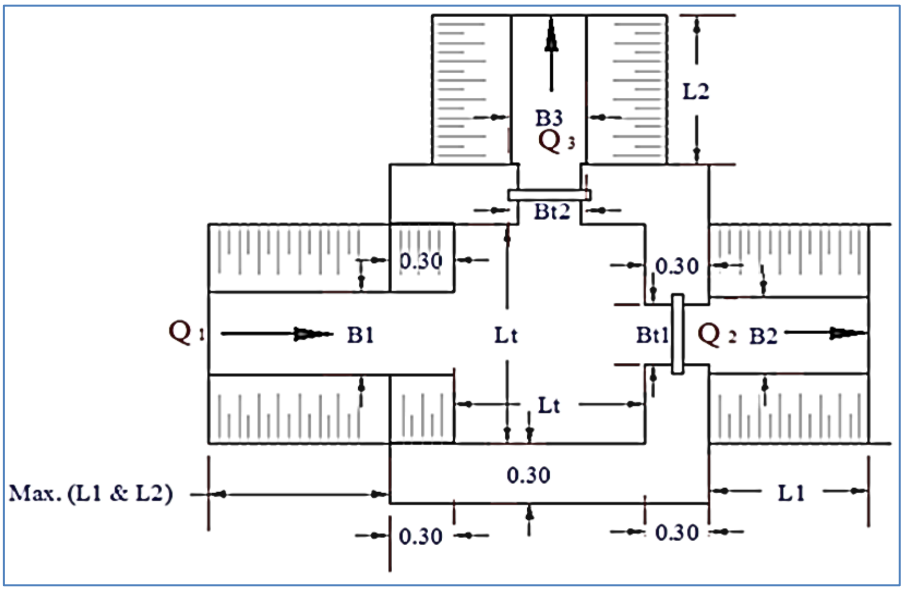


Figure 4-14 Typical division structure arrangement

Box 4-3: Worked Example-3: Design of Division Box based on the following given data

I – Hydraulic design

Parent Canal Data

Incoming Flow in Parent Canal (Q_1)	400 l/sec.
Full Supply Depth (d_1)	0.50m
u/s Canal Bed Level (CBL)	1800.00m
Bed Width (B1)	1.00m
Ground Level (GL)	1800.50m
Full Supply Level	1800.50m
Top Bank Level (T.B.L)	1800.90m
Free Board (FB)	0.40m
Side Slope	1.2:1.0

Branch Canal Data (0^0) (BC1)

Outgoing Flow (Q2)	150 l/sec.
Full Supply Depth (d2)	0.30m
d/s Canal Bed Level	1799.40m
Bed Width (B1)	0.50m
Ground Level (GL)	1800.50m
Full Supply Level	1799.70m
Top Bank Level (T.B.L)	1800.00m
Free Board (FB)	0.30m
Side Slope	1.2:1.0

Branch Canal Data (90^0) (BC2)

Outgoing Flow (Q2)	250 l/sec.
Full Supply Depth (d2)	0.40m
d/s Canal Bed Level	1799.40m
Bed Width (B2)	0.40m
Ground Level (GL)	1800.50m
Full Supply Level	1799.80m
Top Bank Level (T.B.L)	1800.10m
Free Board (FB)	0.30m
Side Slope	1.2:1.0

As the proportion of flow is between 1:1 to 1:10 with respect to branching canal and Parent canal respectively, therefore broad crested type weir is more suitable here.
Conditions for the design of Broad Crested Weir:

(a) The discharge is given by Q = 1.71*K*(Bt - 0.2 h)*(h)1.5

(Where, K= 1.00 for 0^0, K= 0.975 for 45^0, K=0.965 for 60^0, K= 0.95 for 90^0)

(b) The Abutments of the weir are vertical and parallel to each other.

(c) The hydraulic head loss (H_L) is normally 20 % of the driving head (h) i.e, H_L = 0.20h

However for safe design we take H_L = 0.25 h

(d) The throat length (L_t) in the direction of Flow $L_t \geq 2.0$ h

(e) The height of crest (P1) $\geq$ 0.20h

(f) The coefficient of discharge remains constant for the design discharge and below the design discharge.

(g) The distance of end abutment E $\geq$ 0. 20h

Design calculations:

Driving Head (h):

For maximum permissible head loss (H_L), the crest height P_1 should be > 0.2 Therefore, selecting P1 = 0.25h
As, h = d_1 - P1, or h = d_1 - 0.25h, or d1 =1.25 h, or, h = 0.8 d $_1$ = 0.40m
Throat width (B_t):

	Particulars	BC1	BC2	Remark
(a)	Discharge (Q) in m^3/sec.	0.15	0.25	
(b)	The value of 'K'	1.00	0.95	
(c)	The value of 'h' in m	0.40	0.40	
(d)	Throat width (B_t) in m B_t =(Q/(1.71*K*(h)$^{1.5}$)) +0.2*h	0.43	0.69	
(e)	Crest height above C.B.L (P_1)	0.10	0.10	
(f)	Check for (P_1/h)³ 0.20	0.25	0.25	Ok!
(g)	Head Loss (H_L) (U/S F.S.L-D/S F.S.L)	0.80	0.70	
(h)	Throat Length L_t $\geq$2h	0.80	0.80	
	Provide L_t =	0.95	0.95	Ok!
(i)	Crest Level (U/S C.B.L +P_1)	1800.10	1800.10	
(j)	U/S F.S.L (Crest R.L + h)	1800.50	1800.50	
(k)	D/S F.S.L (U/S F.S.L -HL)	1799.70	1799.80	
(l)	D/S C.B.L (D/S F.S.L-F.S.D)	1799.40	1799.40	

Dimensions Provided:

SI.N0.	Particulars	BC1	BC2	units
1	Crest height (P1)	0.100	0.100	m
2	Driving head (h)	0.400	0.400	m
3	Throat Length (Lt)	0.950	0.950	m
4	Throat Width (Bt)	0.427	0.688	m
5	End abutment (E)	0.080	0.080	m

Calculation of y_1, y_2, E_{f1} and E_{f2} for corresponding value of H_L & q by Iteration Method :-

Items		BC1	BC2	Units
As, $(8*q^2*H_L)/g = (-1.5*y_1+((y_1^2/4)+((2*q^2)/(g*y_1)))^{0.5})^3*(0.5*y_1+((y_1^2/4)+((2*q^2)/(g*y_1)))^{0.5})$				
Where,	H_L =	0.80	0.70	m
	q^2 =	0.12	0.13	
So, $(8*q^2*H_L)/g$ =		0.08	0.08	
Let , the Vaue of y_1 =		0.07	0.07	m
Therefore, $(8*q^2*H_L)/g$ =		0.08	0.08	
After iteration we get the value of y_1 =		0.07	0.07	m
y_2 =	$(-0.5*y_1+((y_1^2/4)+((2*q^2)/(g*y_1)))^{0.5})$	0.57	0.57	m
$E_{f2} = y_2 + \{ q^2 / (2*g*y_2^2) \}$		0.59	0.59	m
$E_{f1} = H_L + E_{f2}$ =		1.39	1.29	m

Calculation of y_1 , y_2 , E_{f1} and E_{f2} for corresponding value of H_L & q by Alternate Direct Method :-			
Items	In M/S-2	In M/S-3	Units
Critical y, yc =(q2/g)(1/3) =	0.23	0.24	m
$Z = H_L / y_c$ =	3.44	2.94	
$Y = y_2/y_c = 1+ (0.93556)* Z^{0.368}$ for Z < 1			m
$Y = y_2/y_c = 1+ (0.93556)* Z^{0.264}$ for Z > 1	2.30	2.24	
$y_2 = Y*y_c$ =	0.53	0.53	m
$y_1 = (-y_2/2) + ((y_2^2/4) + (2*q^2/(g*y_2)))^{0.5}$ =	0.08	0.08	m
$E_{f2} = y_2 + \{ q^2 / (2*g*y_2^2) \}$	0.56	0.56	m
$E_{f1} = H_L + E_{f2}$ =	1.36	1.26	m

Calculation of cistern level & length

Sl. No.	Item	BC1	BC2	units
1	Discharge intensity q in cumec/m	0.352	0.363	
2	Upstream water level	1800.500	1800.500	m
3	Downstream water level	1799.700	1799.800	m
4	Head loss H_L	0.800	0.700	m
5	Energy level at downstream E_{f2}	0.556	0.557	m
6	Level at which jump will form (d/s F.S.L.-E_{f2})	1799.144	1799.243	m
7	D/S Floor level provided	1799.400	1799.400	m
8	$E_{f1} = E_{f2} + H_L$	1.356	1.257	m
9	y_1 corresponding to E_{f1}	0.077	0.082	m
10	y_2 corresponding to E_{f2}	0.534	0.534	m
11	Length of cistern of d/s concrete floor required = $5(y_2-y_1)$	2.286	2.259	m
12	Provided d/s floor length (L1 & L2)	2.300	2.300	m
13	Froude No. $F_1 = q / (g*y_1^3)^{0.5}$	5.240	4.950	
14	F_1^2	27.457	24.499	
15	Normal depth of scour $R = 1.35*(q^2/f)^{1/3}$ =	0.672	0.687	m
16	R.L. of bottom of d/s cutoff required = d/s water level - 1.5 R =	1798.691	1798.769	m
17	R.L. of bottom of u/s cutoff required = u/s water level - 1.25 R =	1799.660	1799.641	m

Total Impervious floor Length, Exit gradient and floor thickness:

BC1

Safe exit gradient (GE)= 1/5= 0.20 (Assumed)
Maximum static head is exerted when water is stored up to crest level on u/s and no water on downstream. Maximum static head, H= (Crest Level – Downstream cistern level)

Therefore, H= 0.70m

Depth of downstream curtain wall, d = d/s floor level- R.L. d/s cut of wall
= 1799.40-1798.69= 0.71m (from above table)

$$GE = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}\ ;\ \frac{1}{\pi\sqrt{\lambda}} = \left(\frac{d}{H}\right) * GE = \left(\frac{0.71}{0.7}\right) * 0.2 = 0.2, and\ \lambda = \left(\frac{1}{\pi * 0.2}\right)^2 = 2.47$$

$$\alpha = ((2\lambda - 1)^2 - 1)^{1/2} = 3.82$$

Total floor length required 'b' = αd = 3.82*0.7 =2.70m
u/s floor length required = 2.7 – d/s floor length provided (2.3m from above table) = 0.4m
un balanced head at toe = (H/b)*(d/s floor length) = (0.7/2.7)*2.3 = 0.60m
Floor thickness at toe required =0.60/1.24 = 0.48m

BC2

Safe exit gradient (GE)= 1/5= 0.20 (Assumed)
Maximum static head is exerted when water is stored up to crest level on u/s and no water on downstream. Maximum static head, H= (Crest Level – Downstream cistern level)

Therefore, H= 0.70m

Depth of downstream curtain wall, d = d/s floor level- R.L. d/s cut of wall
= 1799.40-1798.769= 0.63m (from above table)

$$GE = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}\ ;\ \frac{1}{\pi\sqrt{\lambda}} = \left(\frac{d}{H}\right) * GE = \left(\frac{0.63}{0.7}\right) * 0.2 = 0.18, and\ \lambda = \left(\frac{1}{\pi * 0.18}\right)^2 = 3.12$$

$$\alpha = ((2\lambda - 1)^2 - 1)^{1/2} = 5.14$$

Total floor length required 'b' = αd = 5.14*0.63 =3.24m
u/s floor length required = 3.24 –d/s floor length provided (2.3m from above table) = 0.94m
un balanced head at toe = (H/b)*(d/s floor length) = (0.7/3.24)*2.3 = 0.50m
Floor thickness at toe required =0.50/1.24 = 0.40m

II– Structural design

The division box is constructed out of reinforced concrete, since the acting forces associated with the structure is small, minimum reinforcement requirement is adequate.

Area of steel for top reinforcement, As = 3.25*480/2 = 780mm2
Using Dia. 12 bar @ 140 c/c , area provided is, = 807mm2 > Required, ok!

Similar reinforcement for side short walls shall be provided.
No reinforcement is required for bottom side of the base slabs.

4.4.2 Turn out (offtake) structure

Turnouts or offtake structures are canal regulating structures used for taking proportioned amount of water either from secondary canal to tertiary canal which is referred as turnout structure or from tertiary canal to field canal which is referred as an offtake structure. Turnout and offtake have similar set up of structural components consist of inlet and outlet facilities, flow control gate and pipe. Like other similar canal structures, the hydraulic design is done to allow smooth flow from the feeding canal to the branching canal by computing the head losses at inlet, in the pipe and at the out let. Typical turnout arrangement to divert water from secondary canal to tertiary canal is as shown in figure 4.15 below.

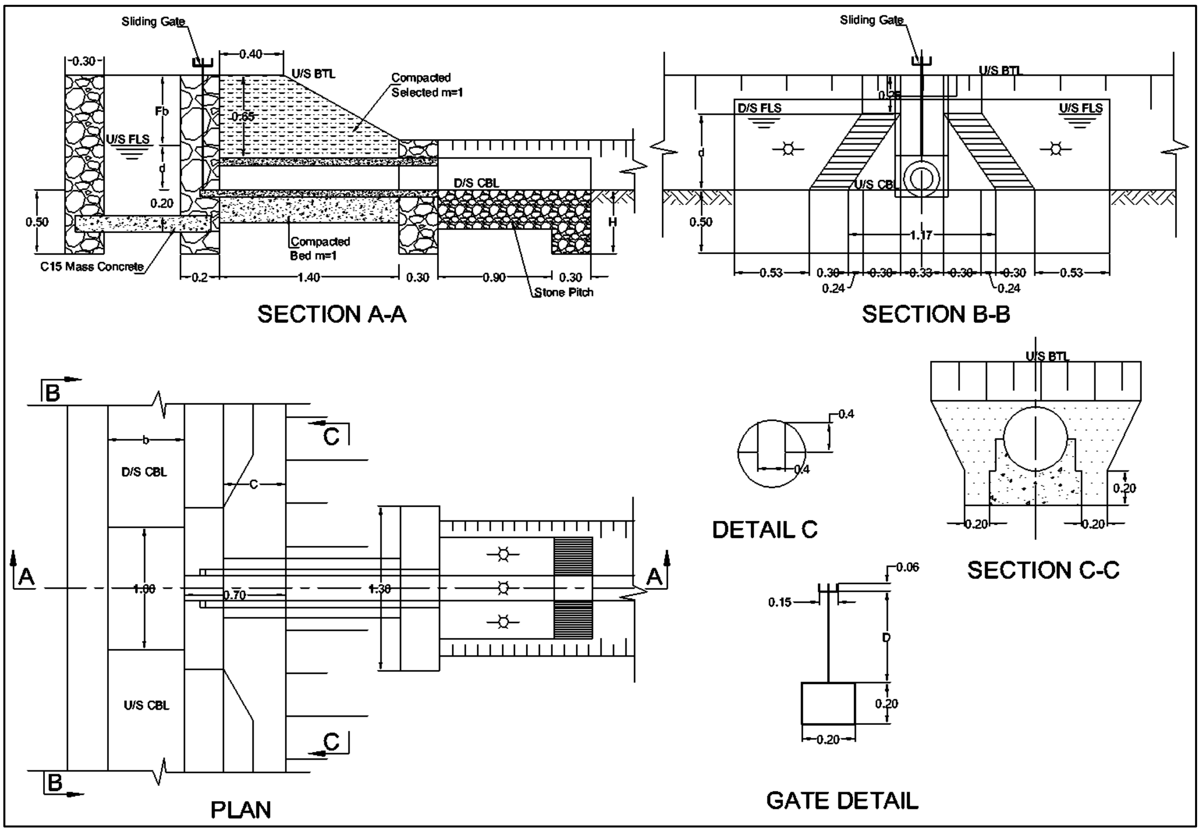


Figure 4-15 Turn out structure arrangement

4.4.3 Check structures

Check structures are used to regulate the canal water surface upstream of the structure and to control the downstream flow. When a canal is flowing at partial capacity, checks are operated to maintain the canal water surface elevation required for upstream water deliveries. The water surface that the check must maintain is the control water surface. It is generally the same as the normal water surface at the check.

In the event of a break in the canal bank, checks can be used to limit the volume of escaping water to that confined between check structures and prevent the entire canal from being emptied. The use of checks also allows reaches of the canal to be isolated and dewatered for repair or inspection. Figure 4.16 below shows a typical arrangement for a check structures.

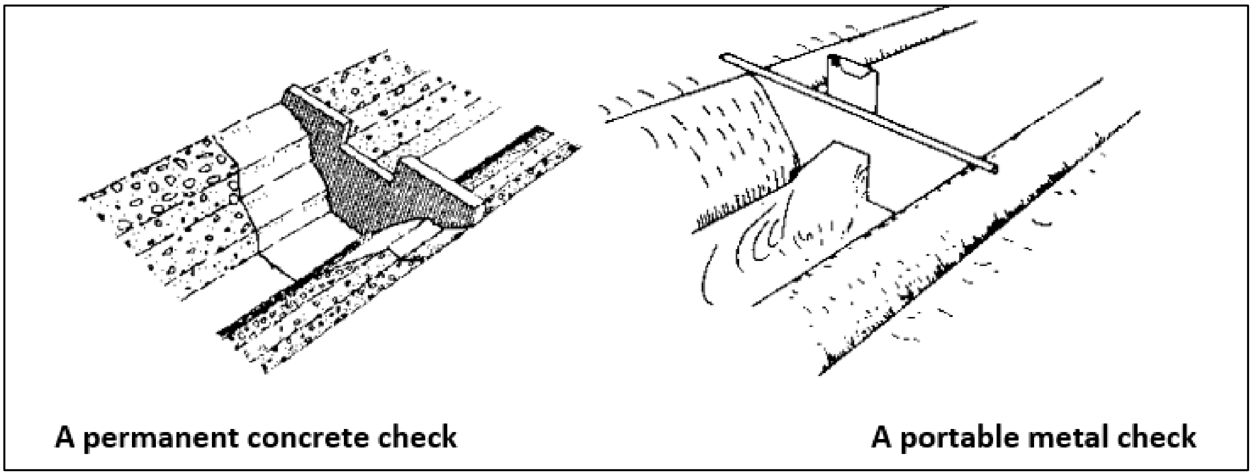


Figure 4-16: Typical check structures

Design considerations for Check structure

1. Stability

Checks are designed to withstand the hydraulic forces imposed by water ponded to the top of the check wall on the upstream side and no water on the downstream side. Adequate check length is provided to dampen water turbulence caused by: (i) the sheet of water flowing over stop logs, or (ii) the jet of water flowing through a partially opened gate. The length in conjunction with the cutoff walls also provides a percolation path with sufficient length to prevent the removal of foundation material by water moving adjacent to the structure.

2. Flow velocity

Velocity through check structures using stop loges should be limited to about 1.0m/sec. for efficient stop log operation. The velocity through checks using gates may be increased to about 1.5m/sec.

3. Head loss

The head loss through a check structure is computed as $0.5\Delta h_v$ (0.5 times the difference between the velocity head at the check opening and velocity head in the canal section upstream of the structure). A minimum loss of 3 centimeter should be allowed for isolated checks in small canals. Except for this head loss checks normally don't provide a drop in grade across the structure. At partial canal capacity, with the water upstream checked to the control water surface, a drop in water surface elevation will occur across the structure.

4.5 WATER MEASUREMENT STRUCTURES

There are many different types of water measurement structures in irrigation systems. The types most commonly used and incorporated in this manual are sharp crested weirs, broad crested weir and parshall flume.

4.5.1 Sharp crested weirs

Sharp crested weirs, also called thin plate weirs, consist of a smooth, vertical, flat plate installed across the channel and perpendicular to the flow (Figure 4.17). The plate obstructs flow, causing water to back up behind the weir plate and to flow over weir crest. The distance from the bottom of the canal to the weir crest, p, is the crest height. The depth of flow over the weir crest, measured at a specific distance upstream of the weir plate (about four times the maximum h1), is called head h1. The overflowing sheet of water is called the nappy. Thin plate weirs are most accurate when the nappy springs completely free of the upstream edge of the weir crest and air is able to pass freely around the nappy. The crest of a sharp crested weir can extend across the full width of channel or it can be notched. The most commonly used are rectangular contracted weir, Trapezoidal (Cipoletti) weir (Figure 4.18) and sharp sided 90^0 V-notched weir (Figure 4.19).

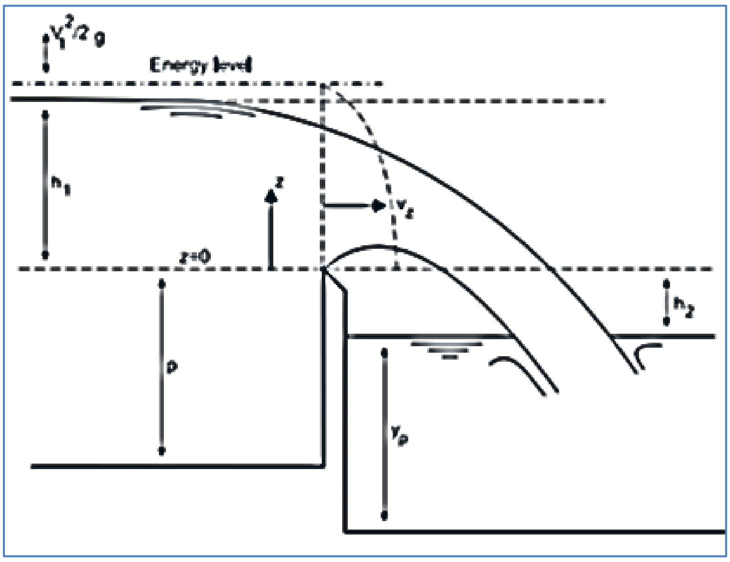


Figure 4-17: Parameters of a harp crested weir

The type and dimensions of the weir chosen are based on the expected discharge and the limit of its fluctuation. For example, a V-notch weir gives the most accurate results when measuring small discharges. Calibration curves and tables have been developed for standard weir types.

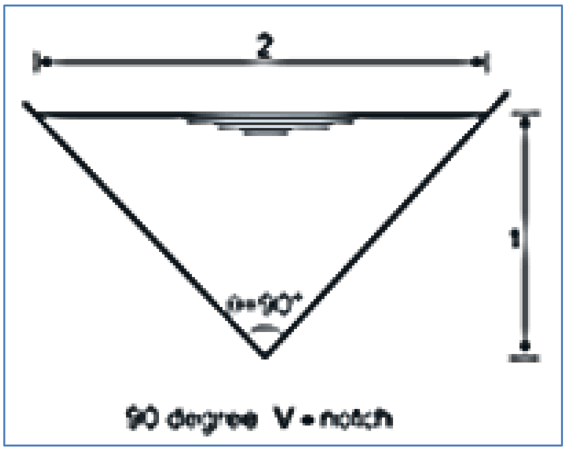


Figure 4-18: Trapezoidal (Cipoletti) weir

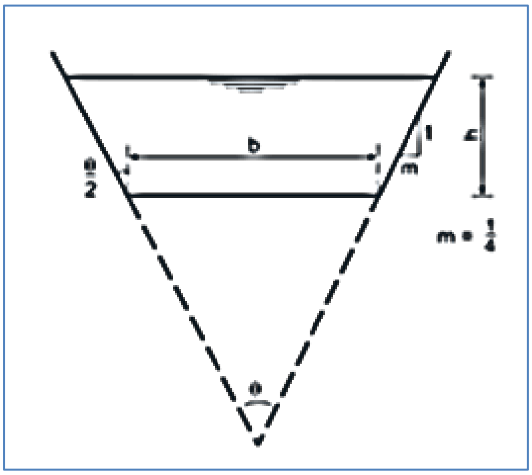


Figure 4-19: 90^0 V-notched weir

The conditions and settings for standard weirs are as follows:

i) The height of the crest from the bottom of the approach canal (p) should be at least twice the depth of water above the crest and should in no case less than 30cm. This will allow the water to fall freely, leaving an airspace under and around the jets.
ii) At a distance upstream of about four times the maximum head a staff gauge is installed on the crest with the zero placed at the crest elevation, to measure the head h1.
iii) For the expected discharge the head should not be less than 6cm and should not exceed 60cm.
iv) For rectangular and trapezoidal weirs, the head h1 should not exceed 1/3 of the weir length.
v) The weir length should be selected so that the head for the design discharge will be near the maximum, subject to the limitations given in (iii) & (iv).
vi) The thickness of the crest of sharp crested weirs should be between 1-2mm.

In sediment laden canals, a main disadvantage of using weirs is that silt is deposited against the upstream face of the weir, altering the discharge characteristics. Weirs also cannot be used in canals with almost no longitudinal slopes, since the required difference in elevation between the water levels upstream and downstream side of the weir is not available.

Discharge equations for weirs are derived by the application of continuity and energy equations. In each case, a discharge coefficient is used in order to adjust the theoretical discharge found by laboratory measurements.

4.5.2 Rectangular contracted weir

A rectangular contracted weir is a thin-plate weir of rectangular shape, located perpendicular to the flow. To allow full horizontal contraction of the nappy, the bed and sides of the canal must be sufficiently far from the weir crest and sides.

Many practical formulae have been developed for computing the discharge, amongst which are the Francis formula is handier and is as indicted below.

$$Q = 1.838(b - 0.2h)h^{3/2} \qquad \text{4.1}$$

Where:

Q = Design discharge over weir (m^3/sec.)
b = Length of Weir Crest (m)
h = Design Water depth measured from the top of the weir crest (m)

Box 4-4:
Worked Example 3-5a: A rectangular contracted weir has to be placed in a lined canal. The design discharge is 0.0783m3/sec and the maximum allowable water depth, h, at the measuring gauge can be 0.15m. What should be the minimum weir crest length, b, calculated using Francis formula.

$$Q = 0.0783 = 1.838(b-0.2*0.15)*0.15^{3/2} = 0.1068*b-0.032, \text{ hence } b = 0.76m$$

4.5.3 Trapezoidal (cipoletti) weir

The trapezoidal weir has a trapezoidal opening, the base being horizontal. The Cipoletti weir is a trapezoidal weir with sides having an outward sloping inclination of 1 horizontal to 4 vertical. The side slope is such that the water depth-discharge relationship is the same as that of fully width rectangular weir. The discharge equation for a Cipoletti weir is:

$$Q = 1.859bh^{3/2} \text{ ....4.2}$$

Box 4-5:
Worked Example 3.5b: A Cipoletti weir has to be placed in a lined canal. The design discharge is 0.0783m3/sec and the maximum allowable water depth, h, at the measuring gauge is 0.15m. What should be the minimum weir crest length, b?

*Q= 0.0783= 1.859*b*0.15$^{3/2}$, hence b= 0.73m*

4.5.4 V-notch weir

A V-notch weir has two edges that are symmetrically inclined to the vertical to form a notch in the plane perpendicular to the direction of flow. The most commonly used V-notch weir is the one with a 900 angle. The V-notch weir is an accurate discharge measuring device, particularly, discharge less than 30l/sec, and it is as accurate as other types of sharp crested weirs for discharge from 30 to 300 l/sec. To operate properly, the weir should be installed so that the minimum distance from the canal bank to the weir edge is at least twice the head on the weir. In addition, the distance from the bottom of the approach canal to the point of the weir notch should also be at least twice the head on the weir.

The general and simplified discharge equation for a V-notch weir is:

$$Q = 1.38 * \tan\left(\frac{\theta}{2}\right) * h^{5/2} \text{ ....4.3}$$

Where:

Q = Design discharge over weir (m^3/sec.)
θ = Angle included between the sides of the notch (degree)
h = Design Water depth (m)

Box 4-6:
Worked Example 3.5c: A design discharge of 0.0783m3/sec has to pass through a V-notch weir with an angle θ of 900. What will be the water depth over the weir?

*Q= 0.0783= 1.38*tan (90^0/2)*h$^{5/2}$, thus h= 0.317m*

4.5.5 Broad crested weir

A broad crested weir is a broad wall set across the canal bed. The way it functions is to lower the specific energy and thus induce a critical flow (Figure 4.20). One of the most commonly used broad crested weir for discharge measurements is the Romijn broad crested weir, which was developed in Indonesia for use in relatively flat areas and where the water demand is variable because of different requirements during the growing season. It is a weir with a rectangular control section, as shown in Figure 4.21 below.

The Romijn weir consists of two sliding blades and a movable weir crest. Which are mounted in one steel guide frame (Figure 4.22). The bottom blade, which is locked under operation conditions, acts as the bottom terminal for the movable weir. The upper blade, which is connected to the bottom blade by means of two steel strips placed in the frame grooves, acts as the top terminal for the movable weir. Two steel strips connect the movable weir to a horizontal lifting beam. The horizontal weir is perpendicular to the water flow and slopes 1:25 upward in the direction of the flow. Its upstream nose is rounded off in such a way that flow separation does not occur. The operating range of the weir equals the maximum upstream head (H_{crt}) which has been selected for dimensioning the regulating structure.

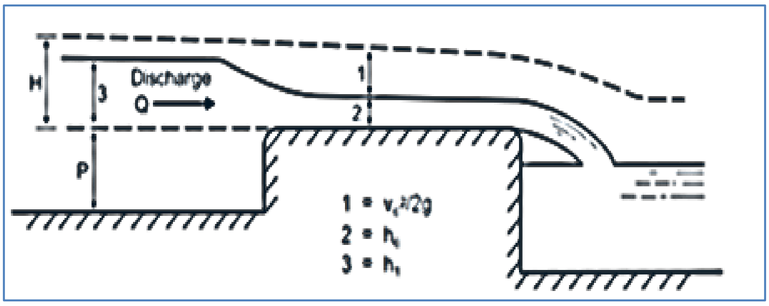


Figure 4-20: Broad crested weir longitudinal section

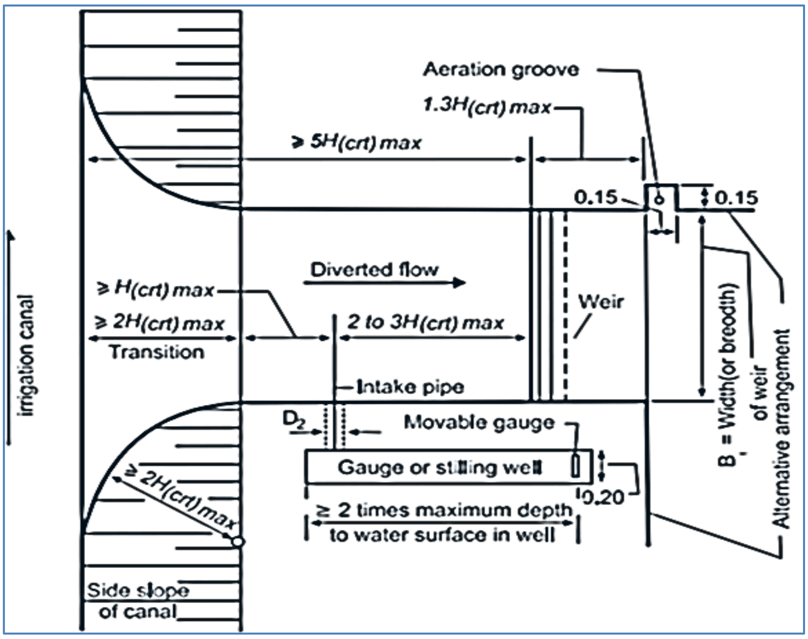


Figure 4-21: Romijn broad-crested weir hydraulic dimensions of weir abutments

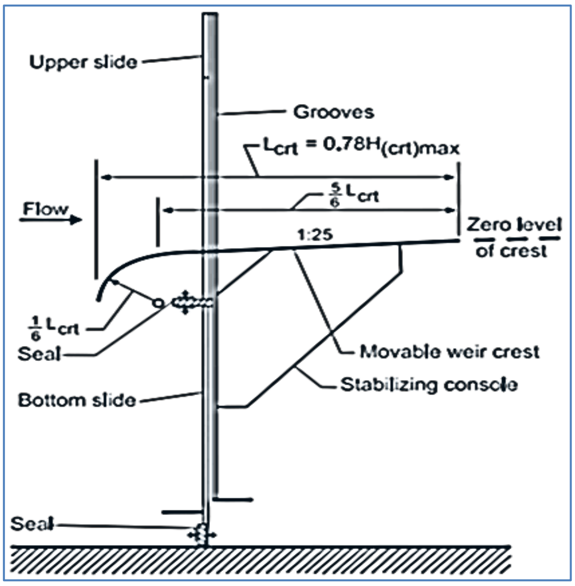


Figure 4-22: Romijn broad-crested weir, sliding blades and movable weir crest

The discharge equation for Romijn broad-crested weir is written as:

$$Q = \frac{2}{3} * C_d * C_v * \left[\frac{2}{3} * g\right]^{\frac{1}{2}} B_t * H_{crt}{}^{3/2} \quad \ldots\ldots \quad 4.4$$

Where:

Q = Design discharge over the weir (m^3/sec.)
C_d = Discharge Coefficient
C_v = Approach Velocity Coefficient
g = Acceleration due to gravity (9.81m/sec^2)
B_t = Width of the weir across the direction of flow (m)
H_{crt} = Design upstream water depth over the weir (m)

The value of the discharge coefficient, C_d, has been determined in a laboratory test. For field structures with concrete abutments, it is advisable to use an average value of C_d = 1.00. The value of the approach velocity coefficient, C_v, range between 1.00 and 1.18,depending on H_{crt} When both C_d and C_v are considered to be 1.00, substituting these values and the value for *g=9.81m/sec^2:*

$$Q = 1.7 * B_t * H_{crt} \quad \ldots\ldots 4.5$$

Box 4-7:
Worked Example 3.5d: A Romijn broad crested weir has to discharge 0.0783m3/sec. The maximum allowable water depth over the weir can be 0.15m. What should be the minimum width of the weir?

Considering a C_d value of 1.00 and an average C_v value of 1.04:

*Q= 0.0783= 2/3*1.00*1.04*(2/3*9.81)$^{1/2}$ *Bt*0.15$^{3/2}$, thus B_t = 0.76m*

Using the simplified equation, would give;

Q= 0.0783= 1.7 B_t *0.15$^{3/2}$, thus B_t = 0.79m*

4.5.6 Parshall flume

The parshall flume is widely used discharge measurement structure. Figure 3.23 shows its general form. The characteristics of parshall flumes are:

- Small Head Loss
- Free Passage of sediments
- Reliable measurements even when partial submerged
- Low sensitivity to velocity of approach

The parshall flume consists of a converging section with a level floor, a throat section with the downward sloping floor and a diverging section with an upward slopping floor. Flume sizes are known by their throat width. Care must be taken to construct the flumes accurately if the calibration curves have to be used. Each size has its own characteristics, as the flumes are not hydraulic scale models of each other. In other words, each flume is an entirely different device. Table 4.1 below is the standard size parshall flume dimension tables.

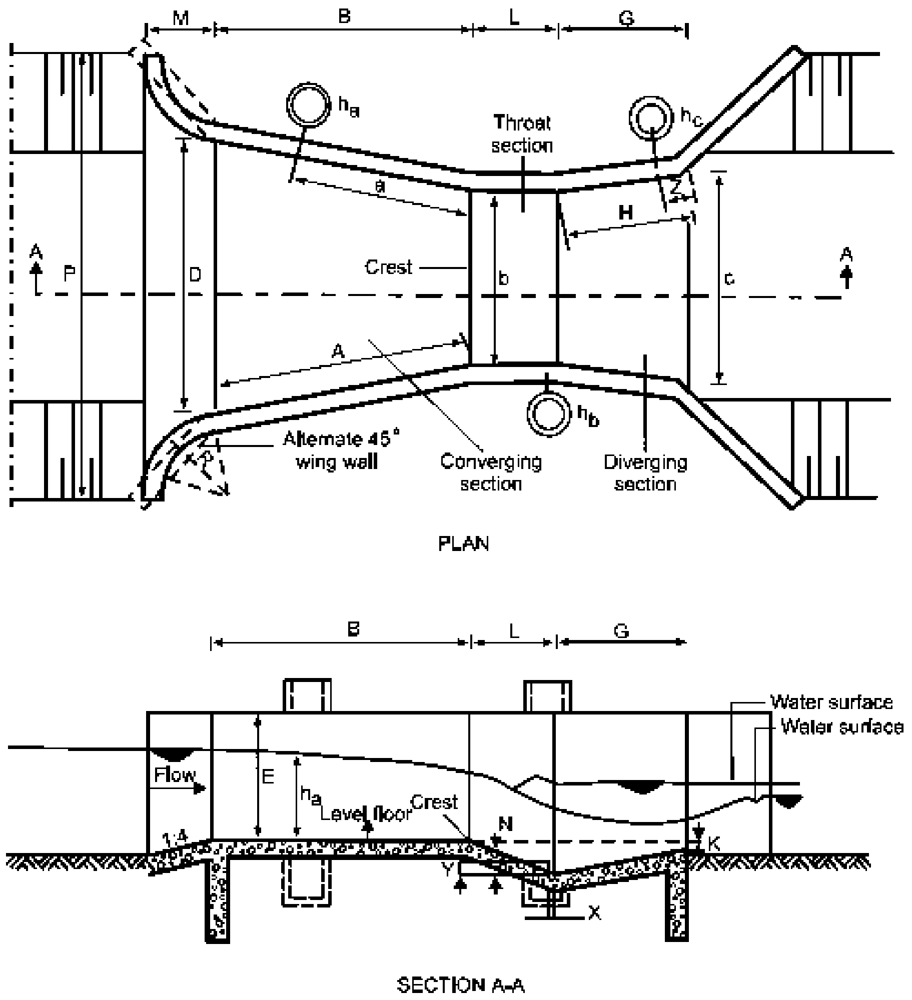


Figure 4-23: Standard parshall flume plan and sectional view

Table 4-1: Standard dimensions of parshall flume

Standard Flume Dimensions according to the Variables labeled in Figure 3.23 (mm)																	
b	**A**	**a**	**B**	**C**	**D**	**E**	**L**	**G**	**H**	**K**	**M**	**N**	**P**	**R**	**X**	**Y**	**Z**
25.4	363	242	356	93	167	229	76	203	206	19	-	29	-	-	8	13	3
50.8	414	276	406	135	214	254	114	254	257	22	-	43	-	-	16	25	6
76.2	467	311	457	178	259	457	152	305	309	25	-	57	-	-	25	38	9
152.4	621	414	610	394	397	610	305	610	-	76	305	114	902	406	51	76	-
228.6	879	587	864	381	575	762	305	-	-	76	305	114	1080	406	51	76	-
304.8	1372	914	134	610	845	914	610	914	-	76	381	229	1492	508	51	76	-
457.2	1448	965	1419	762	1026	914	610	914	-	76	381	229	1676	508	51	76	-
609.6	1524	1016	1495	914	1206	914	610	914	-	76	381	229	1854	508	51	76	-
914.4	1676	1118	1645	1219	1572	914	610	914	-	76	381	229	2222	508	51	76	-
1219.2	1829	1219	1794	1524	1937	914	610	914	-	76	457	229	2711	610	51	76	-
1524.0	1981	1321	1943	1829	2302	914	610	914	-	76	457	229	3080	610	51	76	-
1828.8	2134	1422	2092	2134	2667	914	610	914	-	76	457	229	3442	610	51	76	-
2133.6	2286	1524	2242	2438	3032	914	610	914	-	76	457	229	3810	610	51	76	-
2438.4	2438	1626	2391	2743	3397	914	610	914	-	76	457	229	4172	610	51	76	-
3048	-	1829	4267	3658	4756	1219	914	1829	-	76	-	343	-	-	305	76	-
3658	-	2032	4877	4470	5607	1542	914	2438	-	152	-	343	-	-	305	229	-
4572	-	2337	7620	5588	7620	1829	1219	3048	-	152	-	457	-	-	305	229	-
6096	-	2845	7620	7315	9144	2134	1829	3658	-	305	-	686	-	-	305	229	-
7620	-	3353	7620	8941	10668	2134	1829	3962	-	305	-	686	-	-	305	229	-
9144	-	3861	7925	10566	12313	2134	1829	4267	-	305	-	686	-	-	305	229	-
12192	-	4877	8230	13818	15481	2134	1829	4877	-	305	-	686	-	-	305	229	-
15240	-	5893	8230	17272	18529	2134	1829	6096	-	305	-	686	-	-	305	229	-

Source: FAO, 2002

The flow through the parshall flume can occur either under free flow or under submerged flow conditions. Under free flow the rate of discharge is solely dependent on the throat width and measured water depth, h_a. The water depth is measured at a fixed point in the converging section.

The upstream water depth-discharge relationship, according to empirical calibrations, has the following general form:

$$Q = Kh_a^u \quad \text{4.6}$$

Q = Discharge (m^3/sec.)
h_a = Water depth in converging section (m)
K = A fraction, which is a function of the throat width
u = Variable, laying between 1.522 and 1.60

Table 4.2 below gives the values for K and u for each flume size.

Table 4-2: Discharge Characteristics of Parshall Flume

Throat width B Feet + inches	Discharge range Minimum Maximum (m^3/sec x 10^{-3})		Equation $Q = K*h_a^u$ (m^3/sec)	Head range Minimum Maximum (m)		Modular limit h_b/h_a (m)
1’	0.09	5.4	$0.0604\ h_a^{1.55}$	0.015	0.21	0.5
2’	0.18	13.2	$0.1207\ h_a^{1.55}$	0.015	0.24	0.5
3’	0.77	32.1	$0.1771\ h_a^{1.55}$	0.03	0.33	0.5
6’	1.5	111	$0.3812\ h_a^{1.55}$	0.03	0.45	0.6
9’	2.5	251	$0.5354\ h_a^{1.53}$	0.03	0.61	0.6
1’	3.32	457	$0.6909\ h_a^{1.522}$	0.03	0.76	0.7
1’6’’	4.8	695	$1.056\ h_a^{1.535}$	0.03	0.76	0.7
2’	12.1	937	$1.428\ h_a^{1.550}$	0.046	0.76	0.7

Throat width B Feet + inches	Discharge range Minimum Maximum (m^3/sec x 10^{-3})		Equation $Q = K^*h_a^u$ (m^3/sec)	Head range Minimum Maximum (m)		Modular limit h_b/h_a (m)
3’	17.6	1427	2.184 $h_a^{1.588}$	0.046	0.76	0.7
4’	35.8	1923	2.953 $h_a^{1.578}$	0.06	0.76	0.7
5’	44.1	2424	3.732 $h_a^{1.587}$	0.06	0.76	0.7
6’	74.1	2929	4.519 $h_a^{1.595}$	0.076	0.76	0.7
7’	85.8	3438	5.312 $h_a^{1.601}$	0.076	0.76	0.7
8’	97.2	3949	6.112 $h_a^{1.607}$	0.076	0.76	0.7
	m^3/sec					
10’	0.16	8.28	7.463$h_a^{1.6}$	0.09	1.07	0.8
12’	0.19	14.68	8.859$h_a^{1.6}$	0.09	1.37	0.8
15’	0.23	25.04	10.96$h_a^{1.6}$	0.09	1.67	0.8
20’	0.31	37.97	14.45$h_a^{1.6}$	0.09	1.83	0.8
25’	0.38	47.14	17.94$h_a^{1.6}$	0.09	1.83	0.8
30’	0.46	56.33	21.44$h_a^{1.6}$	0.09	1.83	0.8
40’	0.60	74.70	28.43$h_a^{1.6}$	0.09	1.83	0.8
50’	0.75	93.04	35.41$h_a^{1.6}$	0.09	1.83	0.8

When the ratio of gauge reading h_b to h_a exceeds 60% for the flumes up to 9 inches (228.6mm), 70% for flumes between 9 inch (228.6mm) and 8 feet (2438.4mm) and 80% for larger flume sizes, the discharge is reduced due to submergence. The upper limit of submergence is 95%, after which the flume ceases to be an effective measuring device because the head difference between h_a and h_b becomes too small, such that a slight inaccuracy in either head reading result in a large discharge measurement error.

The discharge under the submerged condition is given as:

$$Q_s = Q - Q_c ,, 4.7$$

Where: Q_c = *Reduction of the modular discharge due to submergence*

The detailed procedure for determining the reduction of the modular discharge (Q_c), can be referred at FAO-Irrigation Manual-2002, Module -7 (Chapter-6).

Usually, the smallest practical size of flume is selected because of economic reasons. In general, the width should vary between 1/3 to 1/2 of the canal width. Often the head loss across the flume is the limiting factor.

The procedure for selecting the appropriate flume is as follows:

Step1: Collect site information: maximum and minimum canal discharge, corresponding normal flow depths and canal dimensions.

Step2: List flumes capable of taking the given discharge, using table 4.2.

Then, for free flow at the maximum canal discharge:

(i) List values of ha, for the maximum canal discharge passing through the flumes.
(ii) Apply the submergence limit appropriate to the flume to find the value of h_b and corresponding to the values of h_a (Table 4.2).
(iii) Subtract h_b from the normal flow depth at maximum discharge to give the vertical distance from the canal to the flume crest level. This assumes that at maximum

submergence the downstream stage is the same as that at h_b, and that the flow downstream of the flume is not affected by it.

(iv) Find the head loss across the flume at maximum discharge (Figure 4.24). Add this to the downstream water depth to obtain the water depth upstream of the flume.

(v) Select the smallest size of the flume for which the upstream stage is acceptable.

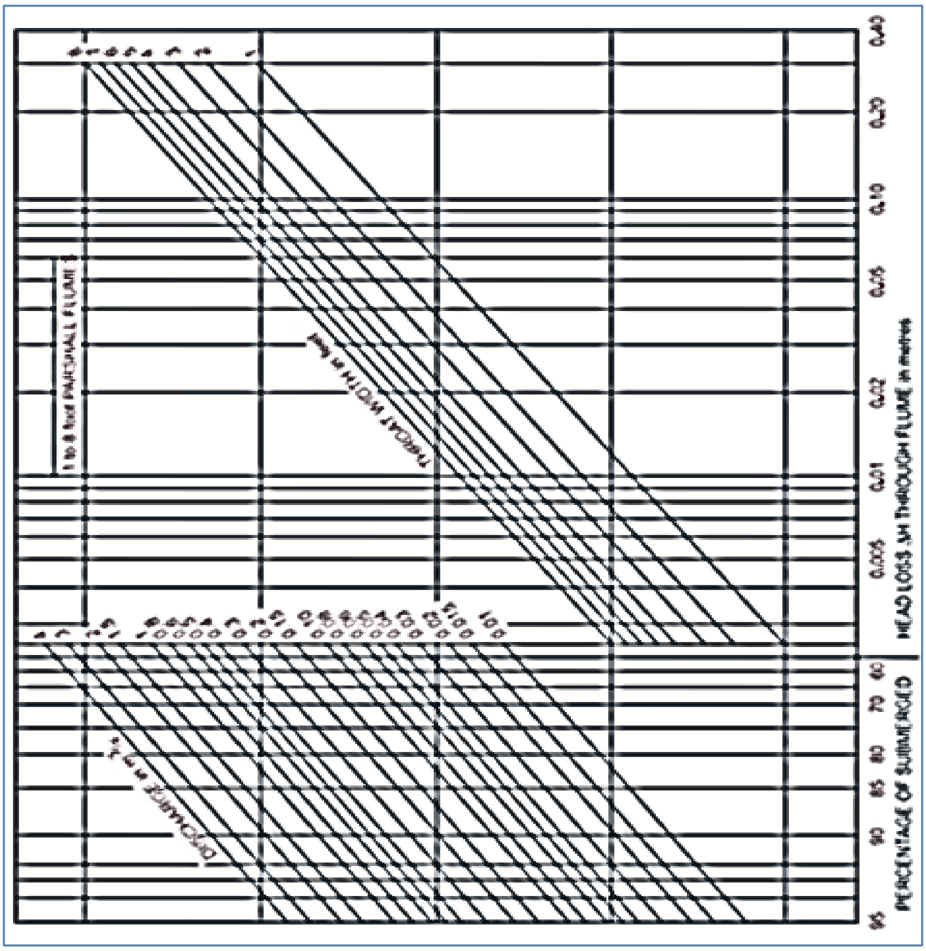


Figure 4-24: Head loss through parshall flume

Box 4-8: Worked Example 4.5: Select the most appropriate flume to be placed in a canal with the following characteristics:

Maximum discharge = 0.566m3/sec
Canal Water Depth = 0.77m
Canal banks at 30m apart
Free board of the canal = 0.15m

a. Consider the flumes with a throat width of 3 and 4 foot. Table 3.2 gives discharge equations for the different flume sizes.

The discharge equation for the 3 foot flume is:
$Q = 2.184 * h_a^{1.566}$, $0.566 = 2.184 * h_a^{1.578}$ thus, ha= 0.43m
The discharge equation for the 4 foot flume is:
$Q = 2.953 * h_a^{1.578}$, $0.566 = 2.953 * h_a^{1.578}$ thus, ha= 0.35m

b. From table 3.2 the submergence of 70% must not be exceeded for modular flow limit. This means that h_b= 0.70ha, Thus for the 3 foot flumes the water depth h_b= 0.3m and for the 4 foot flume h_b = 0.25m.
c. The elevation of the crest above the bottom of the canal (K in figure 4.23) equals the design water depth minus h_b. Thus K= 0.77m-0.3m= 0.47m for the 3 foot flume, and K= 0.77m – 0.25m = 0.52m for the 4 foot flume.
d. From Figure 4.24 it can be read that the head loss is 0.16m for the 3 foot flume and 0.13m for the 4 foot flume. Thus the upstream water depth becomes 0.77m + 0.16m = 0.93m and 0.77m+0.13m=0.90m for the 3 foot and 4 foot flume respectively.
e. The upstream water depth of 3 foot flume just exceeds the sum of the normal water depth and freeboard, thus overtopping would result. The 4 foot flume is just within the available limit of depth. Thus this flume could be selected for implementation. It there was sufficient freeboard available for either of the flumes, considering the rise in water level upstream of the flume, one should select the 3 foot flume because this is cheaper.

Figure 4-25: Example of Flumes: Crossing Stream & Local Depression

4.6 NIGHT STORAGE RESERVOIR

Night Storage Reservoirs (NSR) store water during times when there is an abstraction from the headwork but no irrigation. Depending on the size of the scheme one could construct either one reservoir, located at the top of the scheme or more than one to command sections of the scheme they are serving.

Night storage reservoirs could be incorporated in the design of a scheme when:

i. The distance from the water source to the field is very long, resulting in a long time lag between releasing water from the source and receiving it in the field, this type of Night storage is also referred as balancing reservoir.
ii. The costs of constructing the conveyance canal or pipeline are very high because of the large discharge it has to convey without a NSR. Incorporating a reservoir means that a smaller size conveyance system can be built.
iii. The discharge of the source of the water is smaller than would be required for the area without storing the water during times of no irrigation.

The need for a night storage reservoir should be carefully considered, weighing advantages, such as money saving in water delivery works, against disadvantages, such as the cost of reservoir construction, maintenance, seepage and evaporation losses and disease vector control costs.
The following examples demonstrate design aspects of night storage reservoirs.

Box 4-9:
Worked Example 4.6: A discharge of 78.3l/sec has to be delivered through a 7 Km long canal with a wetted cross-section of $0.19m^2$. When should the headwork gate be opened, if the water has to reach the field at 07:00 o'clock?

Water flow velocity (v) can be computed from continuity equation as:
v= Q/A = 0.0783/0.19 = 0.41m/s
The time (t) it takes for the water to reach the head of the field is given by:
t= Distance/Velocity = 7000/0.41 seconds = 4 hours and 45 minutes.
This would mean that the head gates should be opened at 02: 15 o'clock if irrigation is to start at 0700 o'clock. If this is unsuitable for proper management, one should incorporate a night or balancing reservoir.

Box 4-10:
Worked Example 4.7: Water is abstracted from a river with a base flow of less than 78.3 l/sec (required if the delivery period is 10 hours per day), but more than 32.6l/sec (required if the delivery period can take place 24 hours per day). If abstraction only takes place during daytime the area under irrigation would have to be reduced. Determine the size of the reservoir for the scheme in order to be able to irrigate the whole area.

With a night storage reservoir, one could collect the required discharge of 32.6l/sec from the water source. At any abstraction rate of 32.6l/sec, the volume of water accumulated during the 14 hours when there is no irrigation should be stored in the night storage reservoir. Thus the volume (V) to be stored is:

$$V= 32.6*3600*14/1000 = 1643m^3$$

If 20% is added to cater for evaporation and seepage losses, a night reservoir with a capacity of $1970m^3$ could be proposed.
If the reservoir provided is designed to have a depth of water equal to 2.0m, the diameter of circular type reservoir (D) required to cater the $1970m^3$ of water is calculated as:

$$1970 = \tfrac{1}{4}*D^2*3.14*2, \; D= 35.42m$$

Type of reservoirs

Reservoirs can be classified on the basis of:

- The material used in construction, such as bricks, concrete or earth
- Their shape, which can be circular, square or rectangular

Earthen reservoirs

Earthen reservoirs are the most common, as they are usually cheaper to construct. Figure 3.25 shows a design of a typical square earthen reservoir, including the inlet, the outlet and the spillway. The embankments should be well compacted. If the original soils are permeable, a core trench should be dug and filled up with less permeable soils.

Design Considerations for earthen Night Storage Reservoirs

i) Seepage control

Seepage from ponds often results in serious water losses. The amount of seepage loss in a pond depends on the soil in which the pond is constructed. Sandy soils are more permeable than clayey soils and permit seepage losses. The brief accounts for some of the methods for seepage control are presented as follows.

Soil compaction: Compaction is a particularly effective technique of reducing seepage, especially where the soil being compacted is well graded (that is, it contains all particle sized from small gravel to fine sands, silts and clays). This is also an inexpensive method of pond sealing.

Clay blanket: Pond areas containing a high percentage of coarse-grained soils but lacking enough clay to prevent excessive seepage can be sealed by clay blanketing. The success of the lining depends on the clay content of the borrowed earth.

Bentonite: Bentonite or drillers mud is highly plastic clay prepared from volcanic ash. Adding bentonite is another method of reducing excessive seepage in soils containing a high percentage of coarse-grained particles and lacking enough clay. At complete saturation, bentonite will swell 8-15 times its original volume, disperses and fills spaces between the soil particles and decreasing soil permeability. It is best added dry and worked in to the soil surface before filling the dugout. It can also be added to the water after filling. The second method usually has less chance of success.

Chemical additives: There is a common misconception that soils with adequate amounts of clay will not leak. In fact, not all clays are alike, and the arrangement of clay particles in a soil greatly influences permeability. Dispersants can be added to soil to reduce aggregation and permeability. There are various kinds of chemicals designed to break down aggregates and disperse clay particles, thus sealing the ponds. Such treatments are effective primarily for medium to fine – textured soils. The most commonly used dispersing agents are sodium polyphosphate and sodium chloride (common salt). Soda ash, technical grade, 99 to 100% sodium carbonate, can also be used.

Polyethylene liner: In coarse textured soils, flexible membranes of polyethylene, vinyl and butyl rubber can be used to protect excessive seepage losses. The polyethylene membrane should have a minimum thickness of 0.162mm. Individual membranes should be joined together by overlapping and connecting with two–sided tape or joined using a mildly hot iron box. Membrane lining should be supplied in sections as large as can be handled by the available equipment. Field splices should use a minimum overlap of 5cm. Splices on the side slopes should be oriented perpendicular to the water surface whenever practical. This orientation reduces stress on the joint.

ii) Top width

For embankment less than 3.0m high, a conservative minimum top width is 1.8m. As the height of the embankment increases, increase the top width. The recommended minimum top width for embankments of various heights is:

Table 4-3: Recommended top width of embankment

Height of Embankment(m)	Minimum Top Width (m)
≤ 3.0	1.8
3.1 to 4.3	2.4
4.4 to 5.8	3.0
5.9 to 7.3	3.7
7.4 to 10.4	4.3

iii) Side Slopes

The side slopes of an embankment depend primarily on the suitability of the fill and on the strength and stability of the foundation material. The more stable the fill material, the steeper the side slopes. Unstable materials require flatter side slopes. Recommended slopes for the upstream and downstream faces of various materials are shown in table 4.4 below.

Table 4-4: Recommended side slopes for embankment

Fill Material	Slopes	
	Upstream Face	Downstream Face
Clayey sand, Clayey gravel, sandy clay, silty sand, silty gravel	3:1	2:1
Silty clay, clayey silt	3:1	3:1

iv) Freeboard

Freeboard is the additional height of the embankment provided as a safety factor to prevent overtopping by wave action or other causes. It is the vertical distance between the elevations of the water surface in the pond when the spillway is discharging at designed depth and the elevation of the top of the dam after all settlement. If the embankment is less than 200m long, a freeboard not less than 0.30m is provided. The minimum freeboard is 0.45m for ponds between 200m to 400m long, and is 0.60m for embankments up to 800m long. For longer embankment an engineer should determine the freeboard.

v) Embankment Reservoir Capacity

The dimensions selected for the embankment depend on the required capacity. The capacity should satisfy the amount of the water requirement at a specific period. The volume of the pond can be calculated using prismoidal formula. This formula can be applied for ponds, which have a shape of circular and rectangular.

$$V = D\frac{(A+4B+C)}{6} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.8$$

Where,

V = Volume of the reservoir, m^3
D = Depth of the reservoir
C = Reservoir bottom area, m^2
A = Reservoir top Area, m^2
B = mid depth area (0.5D), m^2

For reservoirs, which do not have a definite shape, or for ponds that will be constructed on water way or at foot of the hill, the following formula can be applied.

$$V = \frac{A\ x\ D}{3} \qquad \text{..4.9}$$

Where,

A = Top area, m^2
D = Maximum depth, m

Circular Reservoirs

A circular reservoir is the common shape of a concrete or brick reservoir. It is the most economical, as the perimeter of a circle is smaller than the perimeter of a square or rectangle for the same area. It also does not need heavy corner reinforcement to resist the water pressure, as do square or rectangular reservoirs.

vi) Inlet and outlet structures design

The inlet of the earthen reservoir is similar to the design of inclined drop incorporating the inlet approach apron, control gate, sloped apron and stilling basin. The design procedure is the same as the design of inclined drop discussed in 4.3.4. The outlet structure is also designed in a similar manner as turnout structure but also shall incorporate energy dissipating system at the outlet downstream reach to dissipate the excess energy of water when the operation is at the full or partly full condition of the reservoir.

Circular Reservoir Components

A foundation of 450-600mm in width and 250-300mm in depth for a 250mm wall thickness should be adequate for circular reservoirs on firm, solid ground. Normally, foundations do not need reinforcement, except when placed on unstable soils.

A floor thickness of 100mm should be adequate. Often a reinforcement grid with 200-300mm interval is placed in the concrete floor. Joints, meant to control cracking, are placed in the reservoir floor and the reinforcement grid should not be cut across these joints. The concrete panels should not exceed 6m in either length or width. The long narrow panel should be avoided.

Reinforcement

The pressure of the water in a circular reservoir produces tension in the wall. The pressure exerted by the water is directly proportional to the water head (depth) from the surface to the depth considered. The tension produced in the wall of a circular reservoir is directly proportional to the water depth and diameter of the reservoir. The tension is taken up by, to some degree, by the material of the wall. However, Concrete and bricks are weak in tension, therefore reinforcement should be provided.

Tanks to the advancement of structural analysis software in the current days, the tension developed in the circular reservoir at a different level of the reservoir can be generated. SAP-2000 software can easily manage such analysis tasks. Once the tensions developed at each level is known, the required reinforcement can be provide to overcome the developed tension force.

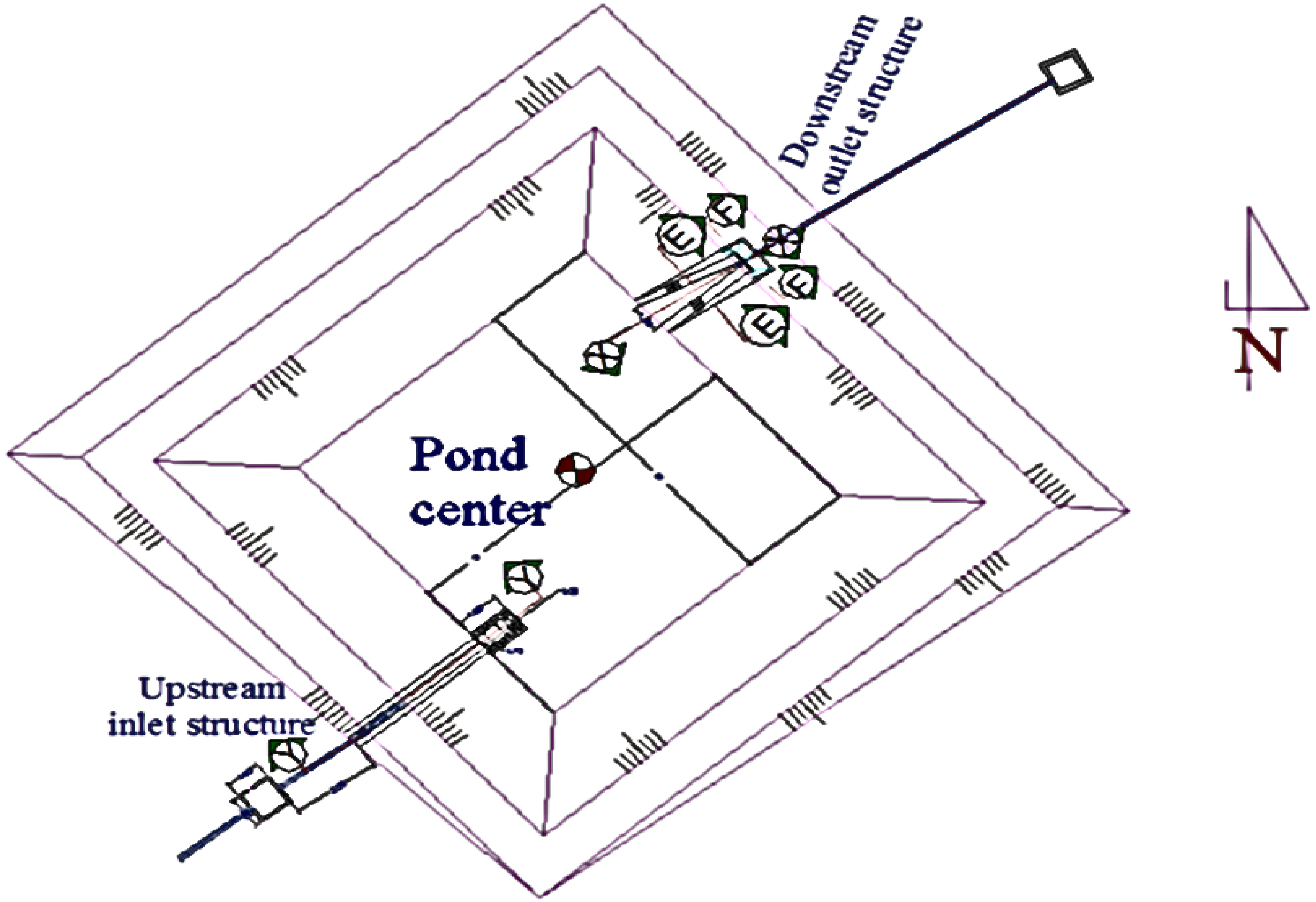


Figure 4-26: Typical earthen night storage reservoir, plan view

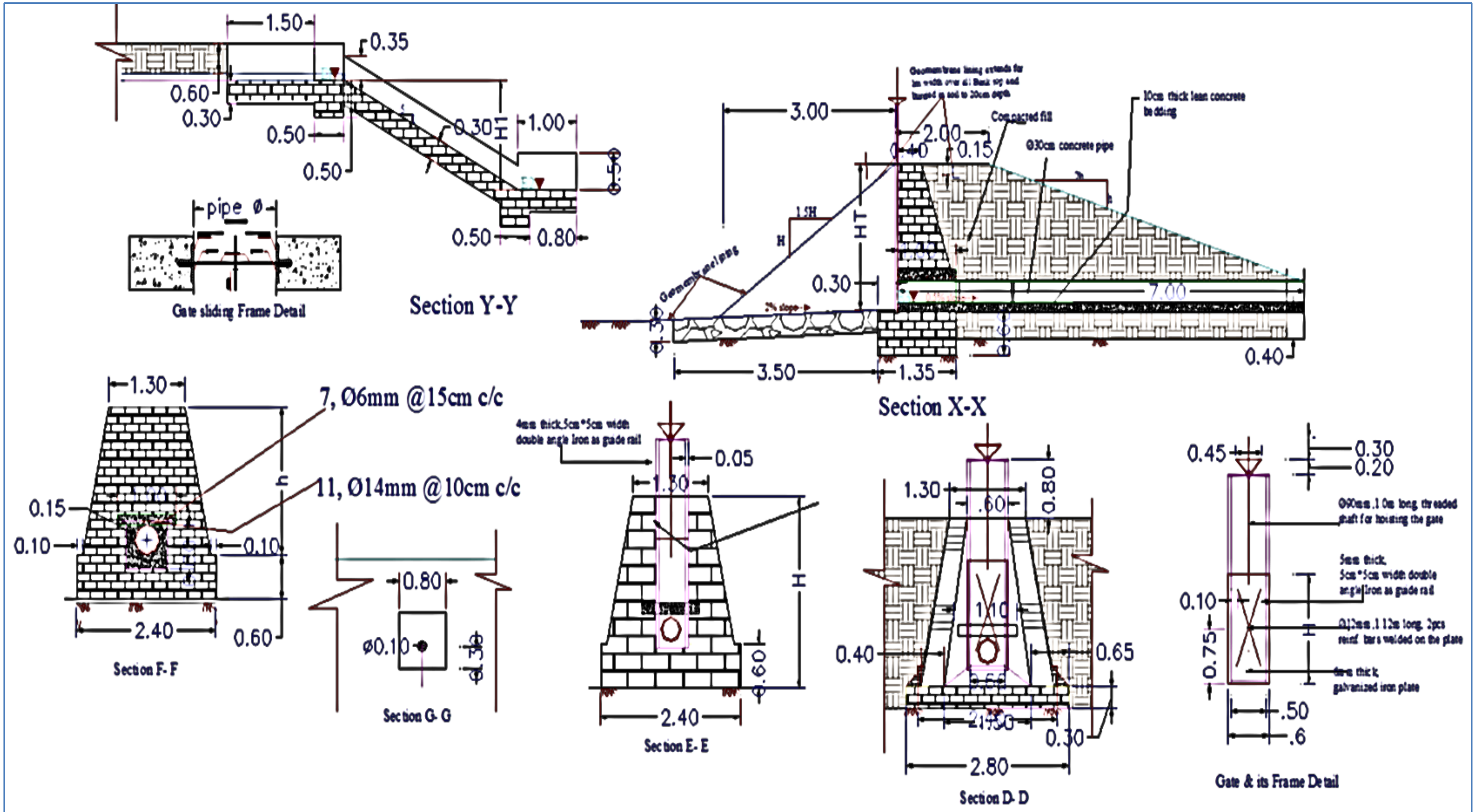


Figure 4-27: Typical earthen night storage reservoir, sections of inlet, outlet & gates

Figure 4-28: Example of night storage pond with Inlet and outlet

4.7 CROSS DRAINAGE STRUCTURES

The need for cross drainage structures results from the high side of the canal to the low side. To protect the canal from such flows, cross drainage structures are provided at locations best suited for handling the runoff.

In crossing natural drainage channels, the canal flows may go under the drainage channels in a siphon, or the channel flows may go under the canal in a culvert or drop pipe. Where natural channels are not available, or where economy dictates, the cross drainage flows may be carried over the canal in an over chute, or small flows may be taken into the canal through a drain inlet.

4.7.1 Types of cross drainage structures

Drainage crossing structures can be classified according to its function and hydraulic design consideration as listed herein below:

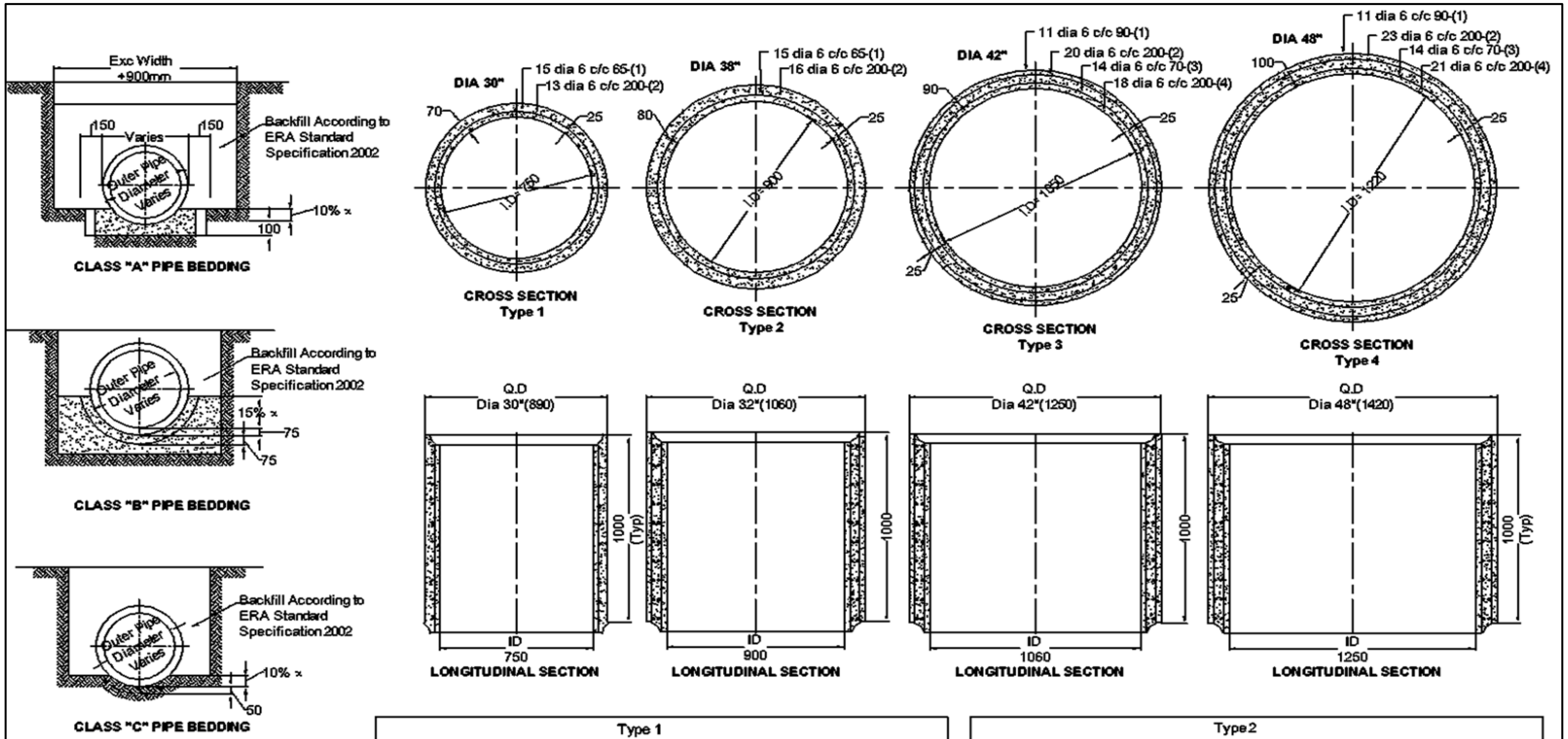
- Siphon Crossing
- Culverts
- Foot Bridges
- Over Chutes
- Pipe Drops
- Super passage

i) Siphon crossing

Where a small canal crosses a large drainage channel, it is usually more economical to carry the canal water under the drainage channel in an inverted siphon than to carry the drainage water under the canal through a culvert. However, the use of a siphon is contingent upon available of the head for siphon losses. Other factors which will affect the results of a cost comparison are the width and depth of the drainage channel. Operational aspects should also be considered as inverted siphons are prone to clogging if debris and laden sediment bed load are anticipated in the canal.

ii) Culverts

Where the canal section is primarily in fill as it crosses a drainage channel, a culvert is a logical structure to carry the drainage water under the canal. Small culverts may become plugged with trash, particularly if the drainage area has a cover of the bush. To overcome such problem and another possible clogging of pipes it is usually advisable to fix the smallest pipe size to be used for pipe culvert construction. The Ethiopian Roads Authority 2002 Manual (ERA-2002) provided the minimum internal reinforced concrete pipe diameter for road crossing is 750mm. Besides, pipe diameter of 900mm, 1060mm and 1220mm are also included in the standard drawing. Even if the concrete pipe culverts are designed for the road project, it can also be equally used for the purpose of this guideline. Figure 4.27 below is the standard reinforced concrete pipes as per ERA-2002 Standard drawing. In addition, ERA- 2002 manual also contains box culverts standard, which can also be used for this guideline. Table 4.5 below shows the list of cross drainage structures available in ERA-2002 Standard Drawing.



GENERAL NOTES:
. Pipe shall conform to ERA Technical Specification 2002
. Concrete class
Minimum cylinder compressive strength at 28 days of 4000 PSI (28Mpa)
- Minimum cement content 360kg/m3
- Maximum water/cement ration 0:53
- Maximum aggregate size 20mm
Revision to comply with AASHTO, specification Part I 17th edition 1996.
Reinforcement:
Imtermediate grade (Grade 40) allowable strength 20,000PSI (140N/mm2
LAPS: 20xBar dia. for deformed bars
40xBar dia. for plain bars
Cover: Minimum 25 mm.

Type 1

I.D mm	(IN)	O.D	T.W	Bars 1 No	Size	Shape	Spaceing	Quant/m Con(m³)	Quant/m Rs(kg)
750	30	890	70	15	6	○	65	0.19	12.13
				15	6	—	200		

BAR AND CONCRETE SCHEDULE

Type 2

I.D mm	(IN)	O.D	T.W	Bars 1 No	Size	Shape	Spaceing	Quant/m Con(m³)	Quant/m Rs(kg)
900	32	1060	70	15	6	○	65	0.27	23.40
				15	6	—	200		

BAR AND CONCRETE SCHEDULE

Type 3

I.D (IN) mm	(IN)	O.D	T.W	Bars 1 No	Size	Shape	Spaceing	Quant/m Con(m³)	Quant/m Rs(kg)
1060	42	1250	90	11	6	○	90	0.35	30.30
				20	6	—	200		
				14	6	○	70		
				16	6	—	200		

BAR AND CONCRETE SCHEDULE

Type 3

I.D (IN) mm	(IN)	O.D	T.W	Bars 1 No	Size	Shape	Spaceing	Quant/m Con(m³)	Quant/m Rs(kg)
1220	48	1420	100	11	6	○	90	0.42	51.82
				23	6	—	200		
				14	6	○	70		
				21	6	—	200		

BAR AND CONCRETE SCHEDULE

Figure 4-29: Reinforced concrete pipe as per ERA-2002

Table 4-5: List of ERA's standard pipe, box culverts, slab bridge & pedestrian foot bridge

Drawing No.	Structure Description	Cross Drainage Structure Size			Material Type
		Pipe internal Dia. (mm)	Box Culvert Size (m)		
			Span	Height	
D-01	Pipe Culvert	750			Reinforced Concrete
D-01	Pipe Culvert	900			Reinforced Concrete
D-01	Pipe Culvert	1060			Reinforced Concrete
D-01	Pipe Culvert	1220			Reinforced Concrete
C-02	Box Culvert (Single)		3.0	Var. from 2.0 to 4.0	Reinforced Concrete
C-03	Box Culvert (Single)		4.0	Var. from 2.0 to 4.0	Reinforced Concrete
C-04	Box Culvert (Double)		3.0	Var. from 2.0 to 4.0	Reinforced Concrete
C-05	Box Culvert (Double)		4.0	Var. from 2.0 to 4.0	Reinforced Concrete
C-06	Slab Culvert (Single)		Var. 1.0 to 6.0		Reinforced Concrete slab and Stone masonry wall Abutment
P-01	Pedestrian Foot Bridge		4.0		Reinforced Concrete slab and Stone masonry wall abutment
P-02	Pedestrian Foot Bridge		6.0		Reinforced Concrete slab and Stone masonry wall abutment
P-03	Pedestrian Foot Bridge		8.0		Reinforced Concrete slab and Stone masonry wall abutment
P-04	Pedestrian Foot Bridge		10.0		Reinforced Concrete slab and Stone masonry wall abutment
P-05 & P-06	Pedestrian Foot Bridge		28.0		Steel Truss Bridge

However, the use of the above concrete pipe for canal water pass way especially for small scale irrigation projects will not be practical as well as economical. In such cases, it is advisable to use lower size pipe crossing such as concrete encased Pvc pipe, steel pipe or concrete pipes of smaller size which are available in the local market.

iii) Foot bridges

Foot bridge facility is frequently required in small irrigation scheme to cross either natural drainage gully or sometimes to arrange over pass over the canal for the smooth operation of the system. Similar to pipe and box culverts, the Ethiopian Road Authority (ERA-2002) has also provided slab type foot bridges for the clear width of 3.0m and steel truss bridge with clear span 2.0m. There are five types of foot bridges with respect to span length included in the standard. The spans of slab type foot bridges are 4.0m, 6.0m, 8.0m and 10.0m. The steel truss foot bridge is provided for a span of 28.0m (see Table 4.5). Figure 4.28 below shows a partial view of the detail drawing for the 4.0m span Foot Bridge.

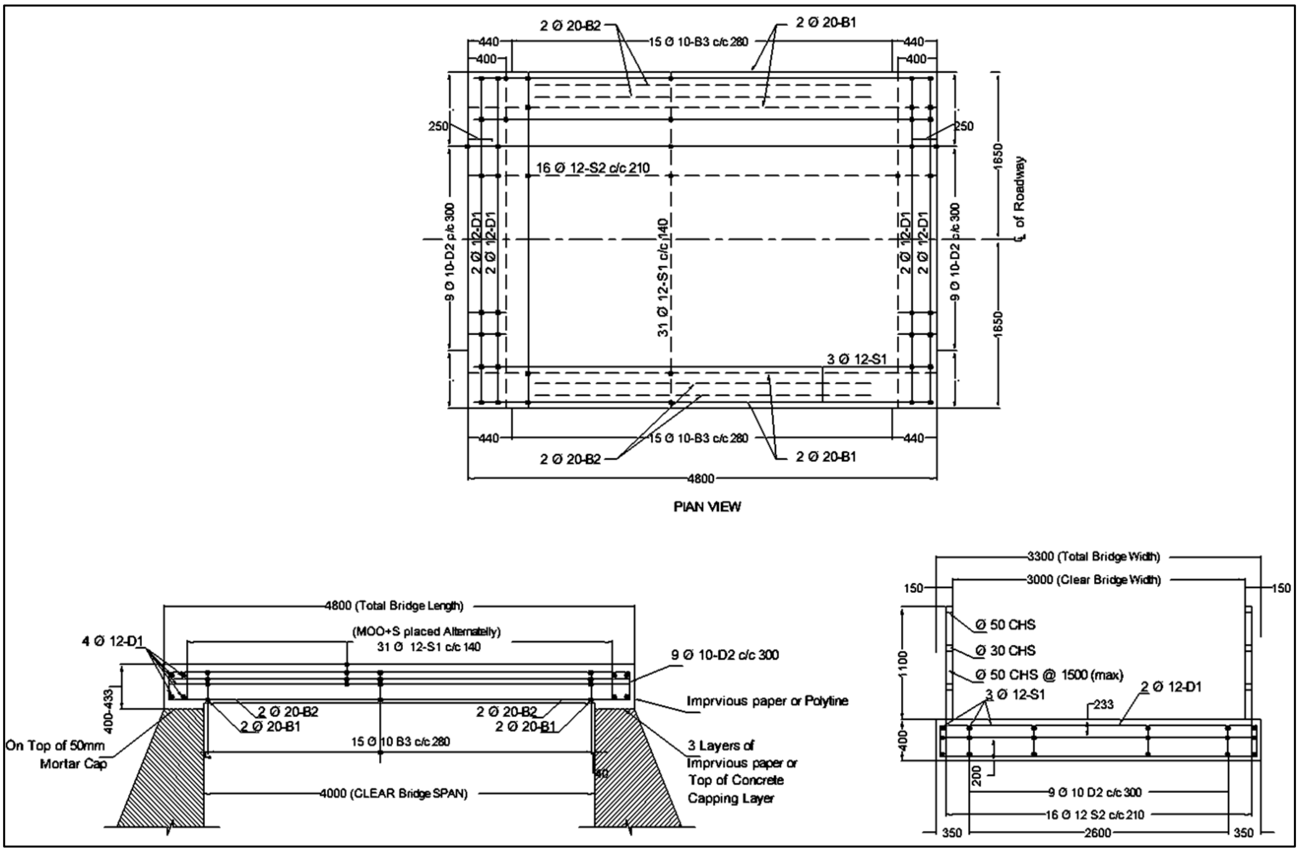


Figure 4-30: Slab foot bridge for 4.0m span

iv) Over chutes

Where the drainage channel invert on the uphill side of the canal is higher than the normal water surface of the canal, an over chute may be the logical choice of structure, carrying the drainage water over the canal. The rectangular concrete over chute is able to pass a considerable amount of debris, whereas the pipe over chute is usually prone to clogging.

v) Pipe drops

A pipe drop conveys water from a higher elevation to a lower elevation. This drop in elevation may be any amount between 1.0m and about 4.5m. A pipe drop not only conveys water but it must also dissipate the excess energy and still the water after it has reached the lower elevation.

vi) Super passage

The super passage is similar with an over chute in its functionality as it is used for the passage of drainage flow over a canal. The difference between the two structures is that flow transition structures are provided for both the drainage channel as well as for the irrigation canal for super passages, whereas for the case of an over chute, the transition is provided only for the drainage channel. Super passage drainage crossing is therefore usually appropriate for medium to large scale irrigation project as it is possible to make the structure smaller and minimize drainage structure construction cost by providing transitions for the irrigation canal. However for the case of small scale irrigation project there is no such economic advantage of super passage structure and usually an over chute structure is preferable. Figure 4.29 depicts the photograph of super passage in Irrigation Project.

(a) Super passage under construction Viewed along the Irrigation Canal

(b) Super passage under construction Viewed along the Drainage Channel

Figure 4-31: Super passage under construction for irrigation project

4.7.2 Cross drainage capacity

For the determination of the required design capacity for drainage structures, based upon expected storm run off for small irrigation systems, cross drainage structures are sized on the basis of storm run off for a 25 years floor frequency.

4.7.3 Design of culverts

As discussed in previous sections, culverts carry storm runoff or drainage water under the canal. Thorough consideration should be given to the culvert alignment, profile, conduit, inlet, and outlet, with special attention given to the hydraulic design.

i) Alignment

A primary rule in locating a culvert is to utilize the natural channel with as little disturbance as possible to the natural runoff pattern. Thus, if a canal crosses a natural channel on a skew, it is usually better to locate the culvert on a skew with the canal, rather than to realign the inlet and outlet of the culvert.

ii) Profile

The profile of the culvert is usually determined by the invert profile of the natural channel and the cross section of the canal. The inlet invert should be located near the existing ground surface, or at the bottom grade elevation of the channel, this will have the advantage to prevent upstream degradation.

Where the conduit is on a uniform grade, it should be steep enough to prevent sedimentation in the conduit but not steep enough to require an energy dissipater. In practice it has been found satisfactory to use a minimum slope of 0.005, and a maximum slope slightly steeper than the critical slope, s_c.

Where a uniform slope would greatly exceed the critical slope, requiring an energy dissipater, it is usually preferable to include a vertical bend and two invert slopes, s_1 and s_2 as shown in figure 4.30 below. The upstream slope s_1 should be much steeper than critical, resulting in free flow at the inlet and inlet control.

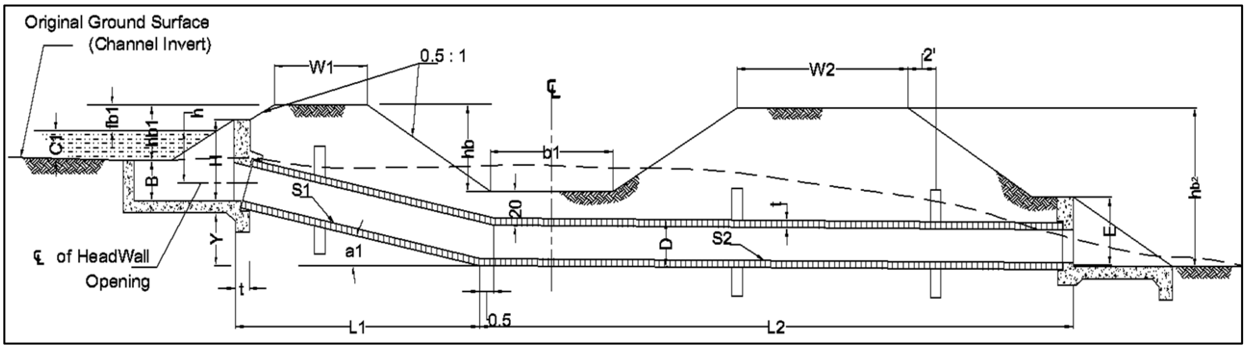


Figure 4-32: Culvert, typical pipe profile

The downstream slope, s_2, is usually set on the flat slope of 0.005 to facilitate dissipation of excess energy by a hydraulic jump in the pipe, without being flat enough to permit sedimentation in the pipe. To obtain a downstream leg of sufficient length to insure a hydraulic jump in the pipe, or to reduce the velocity in the pipe, it may be necessary to steepen the upstream slope, s_1. Two slopes, s_1 and s_2, are also required where the culvert inlet is relatively high with respect to the canal invert elevation. The pipe should cross under the canal prism with at least 600mm of clearance below the invert of an earth section canal or 300 below the lining of concrete lined canal.

iii) Inlet

Several types of transitions are used as culvert inlets. The best choice for any particular situation is dependent upon the hydraulics, the topographic character of the site, and the relative elevations of the canal and the drainage channel. Suitability of each of the basic types of transitions to use under various conditions is given as follows:

a) Type 1: The broken-back transition, as shown in figures 4.31 & 4.32 is best suited to well define inlet drainage channel, where the channel banks can be shaped to conform to the sides of the transition.

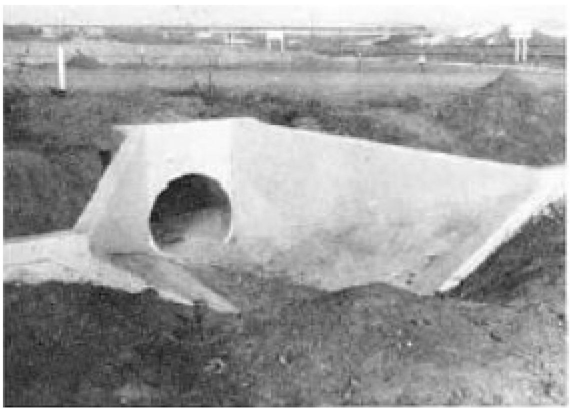


Figure 4-33: Photo graphic view of broken back concrete transition (Type-I) inlet & outlet

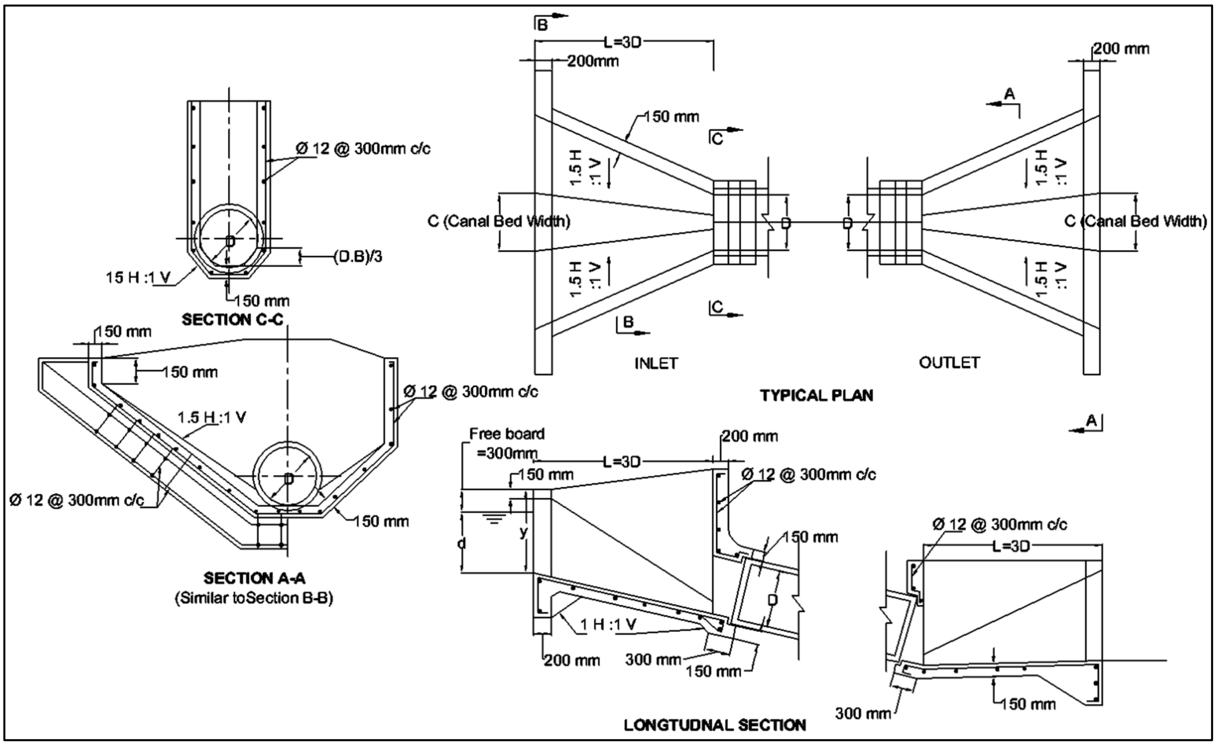


Figure 4-34: Drawings of broken back concrete transition (Type-I) inlet & outlet

b) Type 2: The type 2 transitions is well suited to use in a wide, poorly defined channel, and combines the economy of simple lines with good flow characteristics, see Figures 4.33 and 4.34 below.

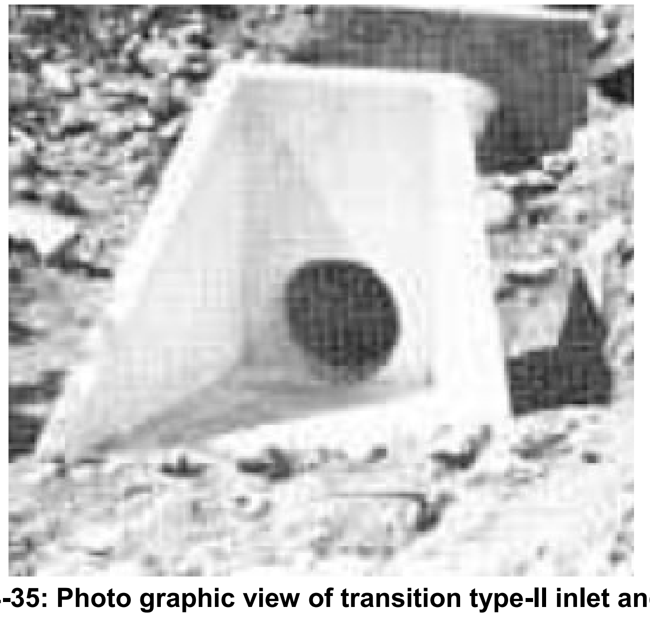
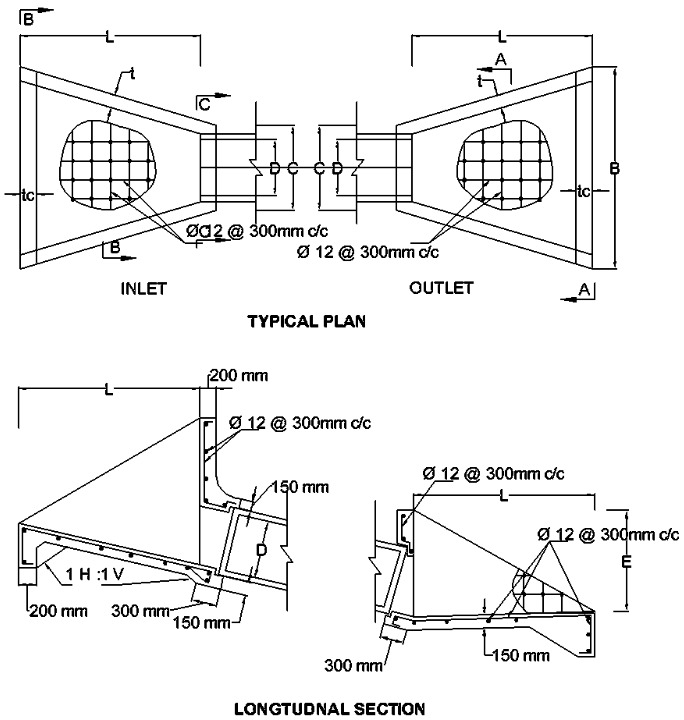
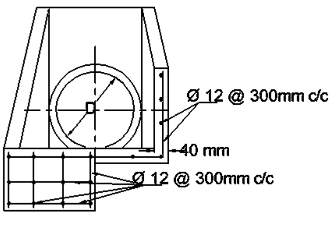


Figure 4-35: Photo graphic view of transition type-II inlet and outlet





D	E	e	L	B	C	t	tc
750	1400	600	2100	2000	1300	150	150
900	1500	800	2300	2300	1400	150	200
1050	1700	800	2500	2700	1600	150	200
1220	1800	800	2700	3200	1800	200	200

TABLE OF DIMENTION IN UNIT OF MILIMETER (mm)

Figure 4-36: Drawings of transition Type-II inlet & outlet

c) Type 3: The type 3 transition, like type 2, is suitable to use in a poorly defied channel. By extending the pool beyond the end of the sloping sidewalls, a longer crest results. By lowering the invert, the headwall opening is lowered, permitting a lower water surface elevation for the inlet pool, provided the flow control is upstream, see Figures 4.35 & 4.36 below.

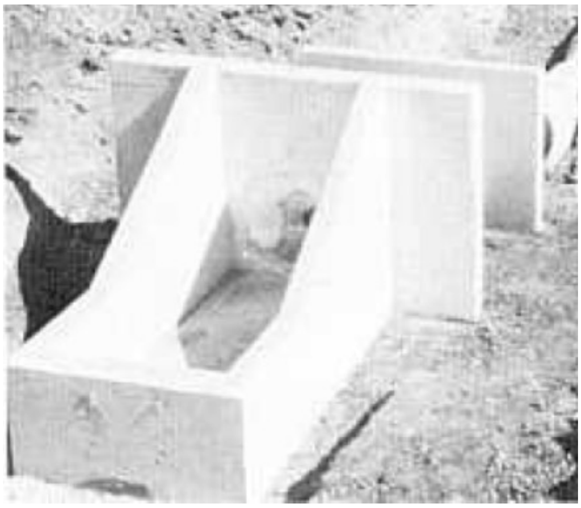
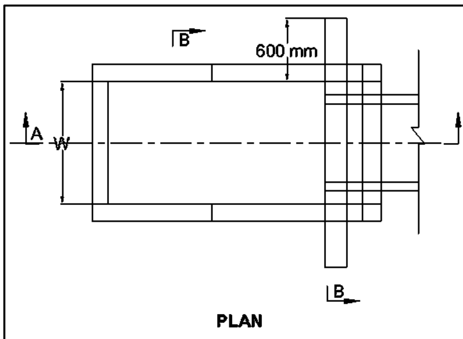


Figure 4-37: Photo graphic view of transition Type-III Inlet and outlet



Max Qm m^3/sec	D	L	W	H	A	B
1.4	750	2300	990	1980	900	1080
2	900	2750	1150	2133.6	900	1235

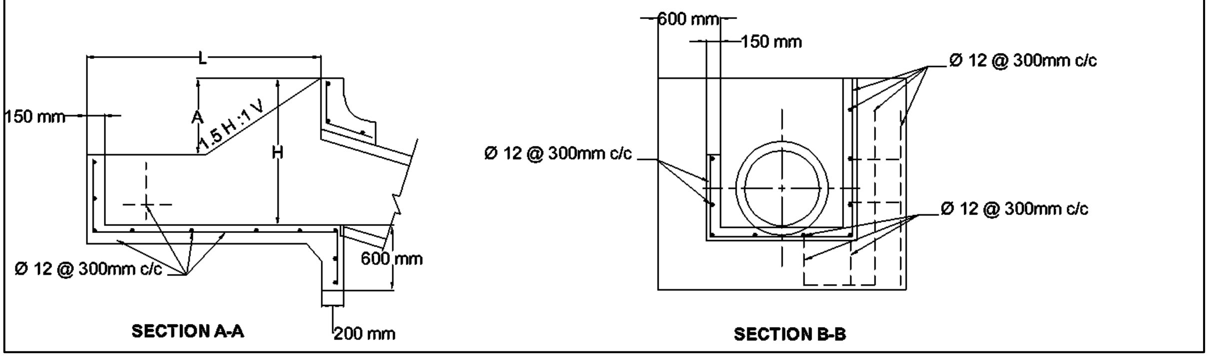
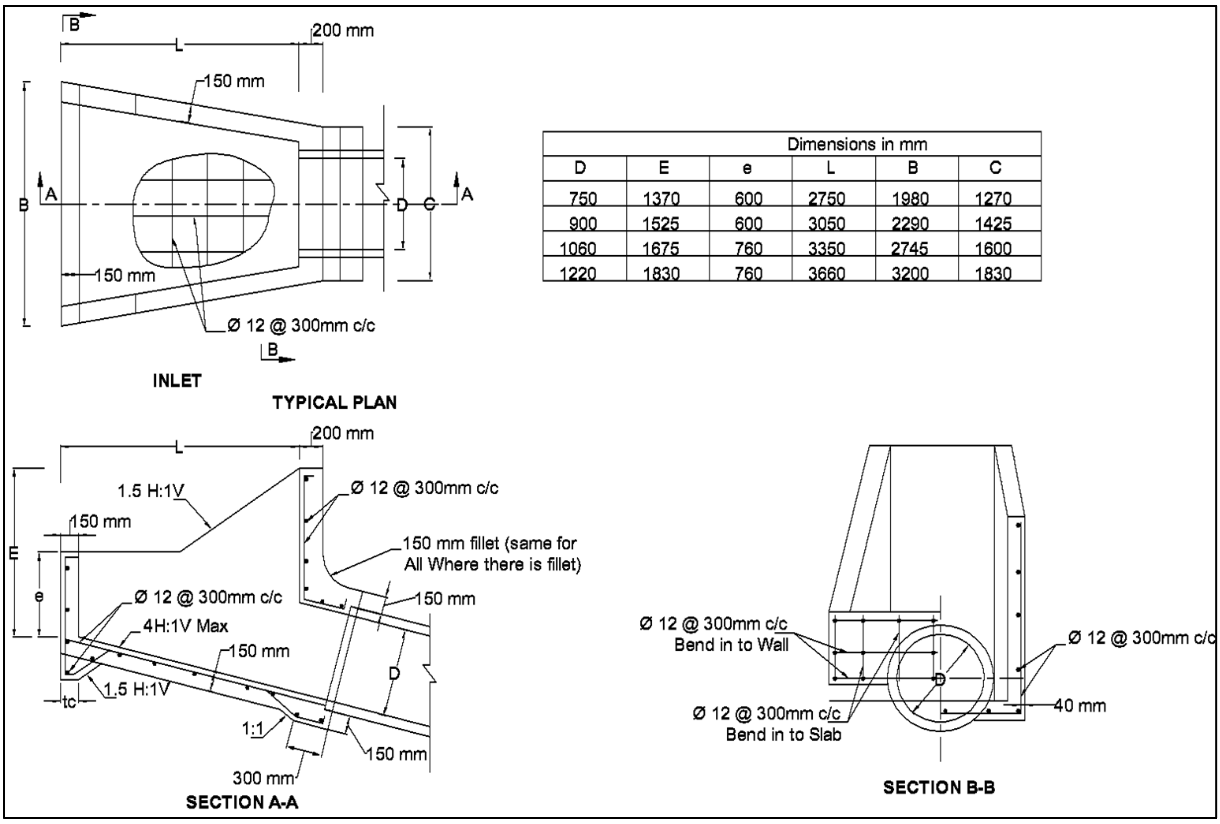


Figure 4-38: Drawings of transition Type-III inlet and outlet

d) Type 4: Except for its sloping floor, and the omission of the headwall cutoff, the type 4 transition, as shown in Figures 4.37 & 4.38. The sloping floor permits a lower pipe invert at the inlet headwall with the type 4 transition.

Figure 4-39: Photo graphic view of transition Type-IV inlet and outlet



Dimensions in mm					
D	E	e	L	B	C
750	1370	600	2750	1980	1270
900	1525	600	3050	2290	1425
1060	1675	760	3350	2745	1600
1220	1830	760	3660	3200	1830

Figure 4-40: Drawing of Transition Type-IV inlet and outlet

e) Earth Transitions: The designer may find that a particular culvert does not require a concrete transition due to the character of the earth or rock material in which it is constructed. An earth transition is permitted if the design flow can be carried with full pipe velocity of 1.5m/sec.

iv) Outlet

The culvert outlet performs the basic function of releasing water to the outlet without excessive erosion. Depending upon the amount of excess energy to be dissipated, this function may be performed by a concrete transition, like Type-I out let transition, Type-II outlet transition, baffled outlet (Figure 4.39) or baffled apron drop (Figure 4.40) used as energy dissipater outlet.

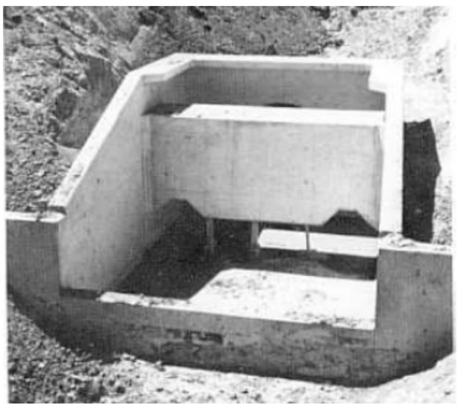


Figure 4-41: Baffled outlet at end of a culvert

Figure 4-42: Baffled apron drop at concrete box culvert outlet

v) Culvert Hydraulics

a) Design Capacity

For determining of the required design capacity, see the descriptions in section 4.7.2 above.

b) Pipe Velocity

The culvert should be designed for a maximum full pipe velocity of 3m/sec if a concrete transition used at the outlet, and a maximum full pipe velocity of 4 m/sec if an energy dissipater is used. In the rare cases that concrete inlets and outlets are not considered necessary, the pipe should be designed for a maximum full pipe velocity of 1.5m/sec.

c) Pipe Diameter

The diameter of the pipe is determined from the basic equation, Q= AV when related to a pipe full. This reduces to $D = 1.13\sqrt{Q}/V$.

d) Hydraulic Control

The upstream water surface will be controlled by the head that is required to satisfy inlet conditions or outlet conditions (such as a high tail water, or pipe losses). To insure the validity of other hydraulic computations, it must be determined whether the hydraulic control is located at the inlet or outlet.

e) Inlet Control

If the upstream water surface is not influenced by flow conditions downstream from the inlet, it is said to have inlet control. Inlet control results from the following conditions;

- A downstream water surface that is low enough with respect to the inlet, that it does not influence the upstream water surface, combined with an upstream pipe slope that is stepper than the critical slope.

f) Outlet Control

If the upstream water surface is influenced by downstream conditions, it is said to have outlet control. Outlet control results from the following conditions:

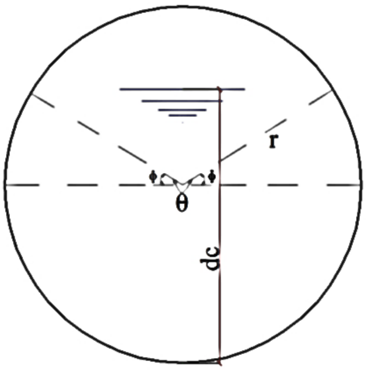
- A downstream water surface that is high enough with respect to the inlet, that it influences the upstream water surface or pipe losses that causes pipe flow at a depth greater than the critical depth.

g) Determination of inlet or outlet control

Examination of Profile: The type of control can be sometimes be determined by examination of the culvert profile. For example, inlet control is suggested if the downstream is wide, poorly defied, and considerable lower than the inlet. Outlet control is suggested if a high tail water (relative to the inlet invert) is assured by a well defied channel with a flat slope. Such conclusions should be supported by performing minimal hydraulic computations. The critical slope will be determined by calculating the critical depth (d_c) and then using the Manning's equation. The critical depth can be approximated with the formula developed for the rectangular section (Equation 4.10) with internal width same as the pipe internal diameter.

$$d_c = \left(\frac{q^2}{g}\right)^{1/3} \qquad \text{...4.10}$$

Where, $q = Q/A$, is unit discharge, d_c is critical depth and g is gravitational acceleration.
The subtending angle, ϴ is required to calculate Area and perimeter of the flow at critical depth, hence the subtended angle should be determined and can be calculated from the figure below as:



$$\theta = 180^0 + 2x\mathrm{Sin}^{-1}((d_c - r)/r)$$

$= 180^0 + 2x\mathrm{Sin}^{-1}(\Phi)$ $= 180^0 + 2*\mathrm{Sin}^{-1}((d_c-r)/r)$ (ϴ in degree)

Among other requirements, inlet control cannot be assured unless the upstream pipe slope is greater than critical slope.

h) Bernoulli Theory

Where the location of the hydraulic control cannot be definitely established by examination of the profile, the Bernoulli equation should be used. By the Bernoulli process, it should be determined if an energy balance exists throughout the length of the culvert as shown in figure 4.30. That is, the specific energy, E_{s2} at one point, plus the drop in invert elevation to a downstream point is equal to the specific energy, E_{s3} (at the downstream point), plus intermediate hydraulic loss,

or $E_{s2} + \triangle El. = E_{s3} + \text{loss}$

Beginning with the water surface and energy established for the outlet, a balance of energy, if obtainable, should be verified between pairs of points proceeding upstream. If a balance of energy results between each pair of points, outlet control is assured. If a balance cannot be achieved between any two points, this indicates that a hydraulic jump occurs between the two points, resulting in an additional head loss. This inability to achieve a balance of energy confirms inlet control of the upstream water surface, except that such imbalance occurring between the outlet channel and a point just inside the pipe indicates outlet control if pipe friction and bend losses produce subcritical flow in the pipe, but a low tail water in the channel allows the flow to pass through critical depth as it emerges from the pipe.

i) Inlet Control Hydraulics

Where inlet control exists, the head required at the culvert inlet is computed from the orifice equation, $Q = CA\sqrt{2gh}$, where *C* is the coefficient of discharge equal to 0.6, *A* is the area of pipe, and *h* is the head of water. Under inlet control, the head, h, is measured from the upstream water surface to the centerline of the pipe at the headwall. To determine the required head, the orifice equation may be written,

$$h = \frac{Q^2}{2gC^2A^2}$$

With C=0.6, and substituting V^2 for Q^2/A^2

$$h = 0.0433V^2$$

j) Outlet Control Hydraulics

Where outlet control exist, the head required to produce the design discharge is a function of the losses in the system as follows:

1) Inlet losses

$h_i = K_i \Delta h_v$

2) Pipe losses

The pipe losses consist of friction and bend losses. The friction loss should be computed from the Manning equation,

$$Q = \frac{AR^{2/3}s^{1/2}}{n}$$

Where *s* is the friction slope of the pipe, and n is the roughness coefficient, equal to 0.013 for the precast concrete pipe.

3) Outlet losses

$h_0 = K_0 \Delta h_v$

Box 4-11:
Worked Example: A concrete precast pipe is required for the design of a culvert to convey storm runoff water under a canal at its intersection with a natural drainage channel.

I) Design input Data

1) Canal Dimensions,

   b = 2.4m, h_b = 1.8m, ss = 1.5H: 1V, d_n = 1.2m, W_i = 1.8m, and W_2= 3.6m, to accommodate operating road.

2) The outlet bank heights, measured at the toe of the outside bank slopes are:

   h_{b1} = 1.2m, and h_{b2} = 2.7m, as shown in Figure 3.30.

3) The cross drainage channel is wide, shallow, and poorly defined, both at the inlet and outlet. It is estimated that the outlet channel will support a depth of 0.30m and a velocity of 0.3m/sec for the design flow.
4) It has been determined that the 25-year flood could yield a discharge of 1.27m3/sec.

II) Hydraulic Design

1) *Pipe velocity*: to use a standard outlet transition, the pipe diameter is determined on the basis of a maximum full pipe velocity of 3m/sec.
2) *Pipe diameter and class*:

$$D = 1.13\sqrt{Q}/V = 1.13\sqrt{1.27}/3 = 0.735\text{m (min.)}$$

   Using precast concrete pipe of ERA-2002 Design manual with internal diameter of 750mm,

   The velocity of flow will come up equal to v=Q/A = 1.27/(3.14*0.75^2/4) = 2.88m/sec.

3) *Hydraulic Control*: From a preliminary profile, using, a type 3 inlet transition and type 2 outlet transition for the poorly defined channel, it is seen that the culvert can clear the canal invert by the required 0.6m, and emerge beyond the lower canal bank with an outlet channel of minimal depth, requiring only minor excavation only.

   As the depth of cut in the poorly defined outlet channel is minimal, the tail water will spread as permitted by the natural topography, and will probably not influence the upstream water surface. Further, to provide 0.6m of clearance beneath the canal invert, the pipe must descend on a slope which is presumed to be steeper than the critical slope, resulting in free flow at the pipe inlet, assuring inlet control. This will be verified by hydraulic calculation too.

4) *Inlet type*: As the drainage channel is not well defied, a type I transition is not suitable. With the upper canal bank only 1.2m higher than the channel invert, a type 3 or 4 inlet transition would seem be better than type 2, as it requires less water depth at the inlet. Type 3 inlet will be used, provided the head is adequate to discharge the design flow without encroaching on the required bank free board.
5) *Pipe friction slope (for pipe flowing full):* Where inlet control is certain, determination of the frictional slope is not necessary. However, a better understanding of the culvert hydraulics is gained by an awareness of the relative slope. From the Manning equation, known flow velocity and known cross section of the pipe, s_f can be computed as,

$$s_f = \left[\frac{Vn}{R^{2/3}}\right]^2$$

   Where, V= 2.88m/sec, n= 0.013, R=A/P = D/4 = 0.750/4 = 0.188

$$= \left[\frac{2.88*0.013}{0.188^{2/3}}\right]^2 = 0.013$$

6) Critical Slope

Determine the critical slope (for comparison with the invert slope, s_1) for Q= 1.27m3/sec and D= 750mm, critical depth of flow can be calculated as,

$$d_c = \left(\frac{q^2}{g}\right)^{1/3}$$

Where, *q* = *Q/A*, is unit discharge, d_c is critical depth and *g* is gravitational acceleration.

q= Q/D = 1.27/0.75 = 1.693m^2/sec

$$d_c = \left(\frac{1.693^2}{9.81}\right)^{1/3} = 0.664$$

Subtended angle calculation,

$\theta = 180^0 + 2x\sin^{-1}((d_c - r)/r)$

$= 180^0 + 2^* \sin^{-1}(0.664-0.375)/0.375) = 280.30^0$

Area of flow at d_c depth,

A = $\pi D^2/4*(\Theta/360) + \sin(90-\Phi)*\cos(90-\Phi)*r^2$, $\Phi = (360-280.3)/2 = 50.15^0$

$= 3.14*0.75^2/4*(280.3/360)+\sin(90-50.15)*\cos(90-50.15)*0.375^2$

= 0.413m^2

Wetted perimeter, P = $\pi D*(280.3/360)$

= 1.507m

R = A/P = 0.413/1.833

= 0.225

Flow velocity at critical depth, V_c = Q/A = 1.27/0.413

= 3.075m/sec.

The critical Slope, s_c, therefore computed from manning equation:

$$s_c = \left[\frac{V_c n}{R^{2/3}}\right]^2$$

Where, n= 0.013, R & V as calculated above.

$$s_c = \left[\frac{3.075*0.013}{0.225^{2/3}}\right]^2 = 0.012$$

7) Inver slope, s_1

Invert slope is determined from the culvert profile drawing. A minimum of 0.6m of clearance between the canal invert and the concrete pipe is provided. Wall thickness of the pipe, t= 70mm = 0.070m,

Thus,

$y_1 = D+t+0.6+h_b-h_{b1}-B$

$= 0.75+0.07+0.6+1.8-1.2-0.9 = 1.12m$

$L_1 = 1.5h_b + w_1 + 1.5(h_{b1}+B-H) + t_w$

$= 1.5*1.8+1.8+1.5*(1.2+.9-1.8) +.20 = 5.15m$

$s_1 = y_1/L_1 = 1.12/5.15 = 0.22$

$\alpha_1 = tan^{-1}(0.22) = 12.4^0$

Therefore, $s_1 > s_c$, permitting free flow at the pipe inlet, with inlet control; providing the downstream water surface does not control.

8) Invert slope, s_2: Since s_1 is too steep to be extended to the outlet, without an outlet channel of excessive depth and length, a bend and a second slope, s_2 are required. Let s_2 = 0.005, which is the minimum slope required for clearing sediment at the pipe floor.

9) Length , L2

The length L2 must be sufficient to locate the outlet headwall beyond the canal bank at its intersection with the top elevation of head wall. L2 is determined by scaling the preliminary profile. L2= 12.50m.

10) Outlet Type

As the outlet is poorly defined, the type 1 transition would not be suitable. Therefore the Type-2 transition is selected. From Figure 3.34, transition type-II and for pipe width, D= 0.75m, dimensions like L=2100mm and E=1400mm can be read from the table of dimension.

11) Inlet Hydraulics

Proceeding with the assumption that inlet control exists, the ability to discharge the design flow is a function of the inlet hydraulics, without regard to the culvert pipe and outlet conditions. The type 3 inlet transition dimensions provide hydraulic control at the headwall opening. Therefore the head required to discharge 1.27m3/sec can be determined from the orifice equation with the coefficient C= 0.6 as,

$$h = \frac{Q^2}{2gC^2A^2} = \frac{1.27^2}{2g * 0.6^2 * (3.14/4 * 0.75^2)^2} = 1.17\text{m}$$

12) Inlet freeboard

Referring to Figure 3.30, the type 3 transition would require ponding the channel water surface to a depth, d_1:

$$d_1 = h - \left[B - \frac{D'}{2}\right]$$

$$D' = \frac{D}{\text{Cos}\,\alpha_1} = \frac{0.75}{\text{Cos}\,(12.4)} = 0.77\text{m}$$

$$d_1 = 1.17 - \left[0.9 - \frac{0.77}{2}\right] = 0.655\text{m}$$

The bank freeboard, f_b, is equal to the bank height above the channel invert minus the required ponding depth, or

$f_{b1} = h_{b1} - d_1 = 1.2 – 0.655 = 0.6$ (minimum requirement, it is thus sufficient)

III) Verification of Hydraulics by Bernoulli Method

1) Energy balance at the outlet

The specific energy in the outlet channel is determined as follows:

$$E_{s4} = d_4 + h_{v4} = d_4 + \frac{V^2}{2g} = 0.3 + \frac{0.3^2}{2 * 9.81} = 0.305\text{m}$$

The specific energy in the outlet end of the pipe cannot be less than Esc, as minimum energy accompanies critical flow. From previous steps of hydraulic design, d_c= 0.664m, V_c = 3.075m/sec

Then,

$$E_{sc} = d_c + h_{vc} = d_c + \frac{{V_c}^2}{2g} = 0.664 + \frac{3.075^2}{2 * 9.81} = 1.15\text{m}$$

As $E_{sc} > E_{s4}$, and E_{sc} represents minimum energy, a balance of energy is unobtainable at the outlet. Therefore, the tail water cannot provide outlet control.

2) Specific energy at point 1 (just inside the pipe at the inlet).

As s_1 is steeper than s_c and the tail water does not control, the supercritical flow will occur from point 1 to point 2, and critical flow will occur at point 1, where Esc is already calculated in the energy balance at the outlet ,

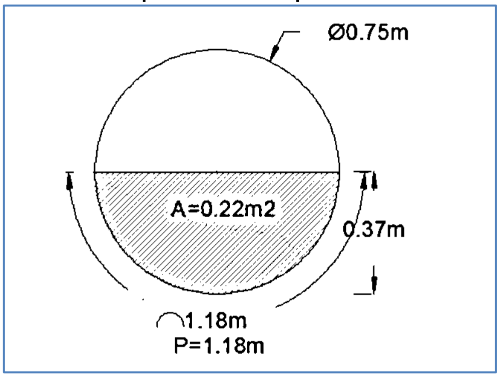
Esc = 1.15m

3) Specific energy at point 2

Find d_2 and h_{v2} yielding a specific energy, E_{s2} such that,

$$E_{s2} = E_{sc} + y_1 - h_{f1}$$

Where, h_{f1} is pipe frictional loss between point 1 and point 2.



Assume a trial flow depth of, d_2= 0.37m,
Accordingly, A= 0.22m^2, P= 1.18m, R= A/P = 0.186m
(Flow Area (A) and Wetted perimeter (P) for any trial depth of flow can be obtained easily with the aid of Auto-Cad Drawing software as shown in figure above).

v_2 = Q/A = 1.27/0.22 = 5.77m/sec.

$$E_{s2} = d_2 + \frac{{v_2}^2}{2g} = 0.37 + \frac{5.77^2}{2 * 9.81} = 2.07\text{m}$$

Frictional slope, sf2 at point 2 can be computed from manning equation as,

$$s_{f2} = \left[\frac{v_2 n}{R^{2/3}}\right]^2 = \left[\frac{5.77 * 0.013}{0.186^{2/3}}\right]^2 = 0.053$$

The frictional loss in the length, L_1, is

$$h_{f1} = \left[\frac{s_c + s_{f2}}{2}\right] L_1 = \left[\frac{0.012 + 0.053}{2}\right] 5.15 = 0.167\text{m}$$

Finally, the assumption that d2 = 0.37m is correct if,

$$E_{s2} = E_{sc} + y_1 - h_{f1}$$

2.07 = 1.15 + 1.12 - 0.167

2.07 ≠ 2.10 (instead of going for second trial depth, for practical Application this can be consider satisfactory)

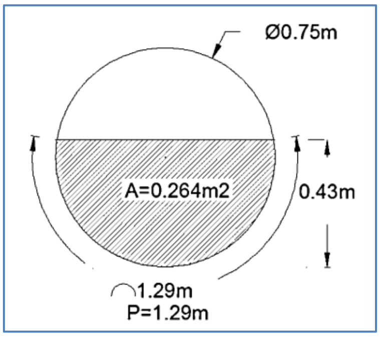
4) Specific energy at point 3

Finding d_3 and h_{v3} yielding a specific energy, E_{s3} such that

$$E_{s3} = E_{s2} + y_2 - h_{f2}$$

Where, $y_2 = s_2L_2 = 0.005*12.50 = 0.0625m$

Try $d_3 = 0.43m$



Accordingly, A= 0.264m^2, P= 1.29m, R= A/P =0.205m

$V_3 = Q/A = 1.27/0.264 = 4.81m/sec.$

$$E_{s3} = d_3 + \frac{v_3{}^2}{2g} = 0.43 + \frac{4.81^2}{2*9.81} = 1.61m$$

Frictional slope, s_{f3} at point 3 can be computed from manning equation as,

$$s_{f3} = \left[\frac{v_2 n}{R^{2/3}}\right]^2 = \left[\frac{4.81*0.013}{0.205^{2/3}}\right]^2 = 0.032$$

The frictional loss in the length, L_1, is

$$h_{f1} = \left[\frac{s_{f2} + s_{f3}}{2}\right] L_2 = \left[\frac{0.053 + 0.032}{2}\right] 12.5 = 0.531m$$

Finally, the assumption that d_2 = 0.43m is correct if,

$$E_{s3} = E_{s2} + y_2 - h_{f2}$$

1.61 = 2.10 + 0.0625 - 0.531

1.61 ≠ 1.60 (but quite enough for design purpose)

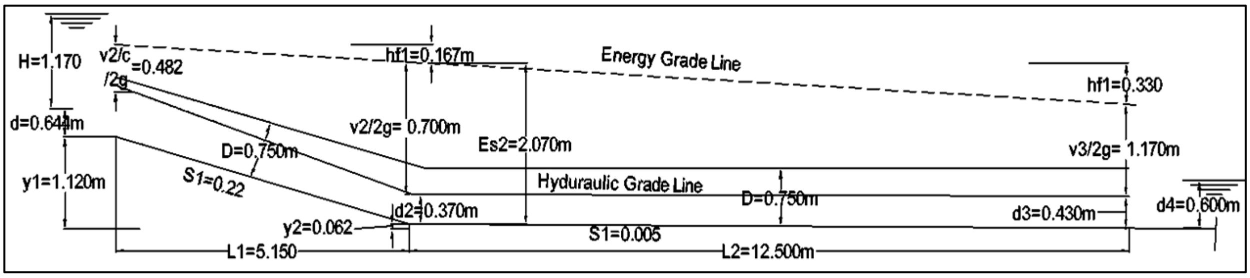


Figure 4-43: Culvert hydraulics as per the worked example

4.7.4 Design of an over chute

A canal is crossing a natural drainage channel, has insufficient head to accommodate a siphon under the channel. An over chute is selected, as it can carry the drainage flow over the canal with little channelization required. The drainage channel is intercepted deep gully after crossing the canal, appropriate drop structure has to be designed to dissipate the excess out flow from the over chute.

This example is based on over chute drainage crossing design at Eja-Kersa stream crossing for Tibila Irrigation Project in Oromia Regional State, the project site is near to Adama Town.

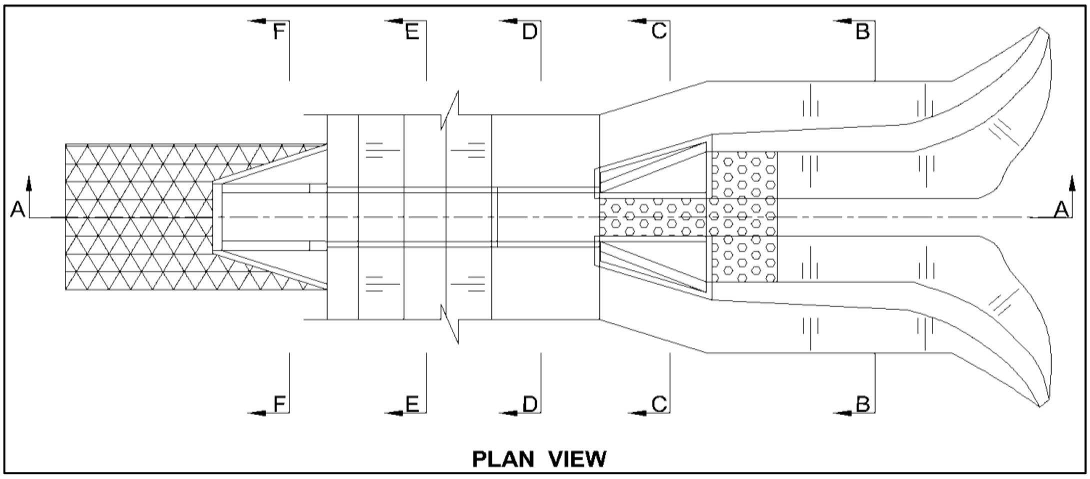
I) Design Input data

(1) Canal Parameters

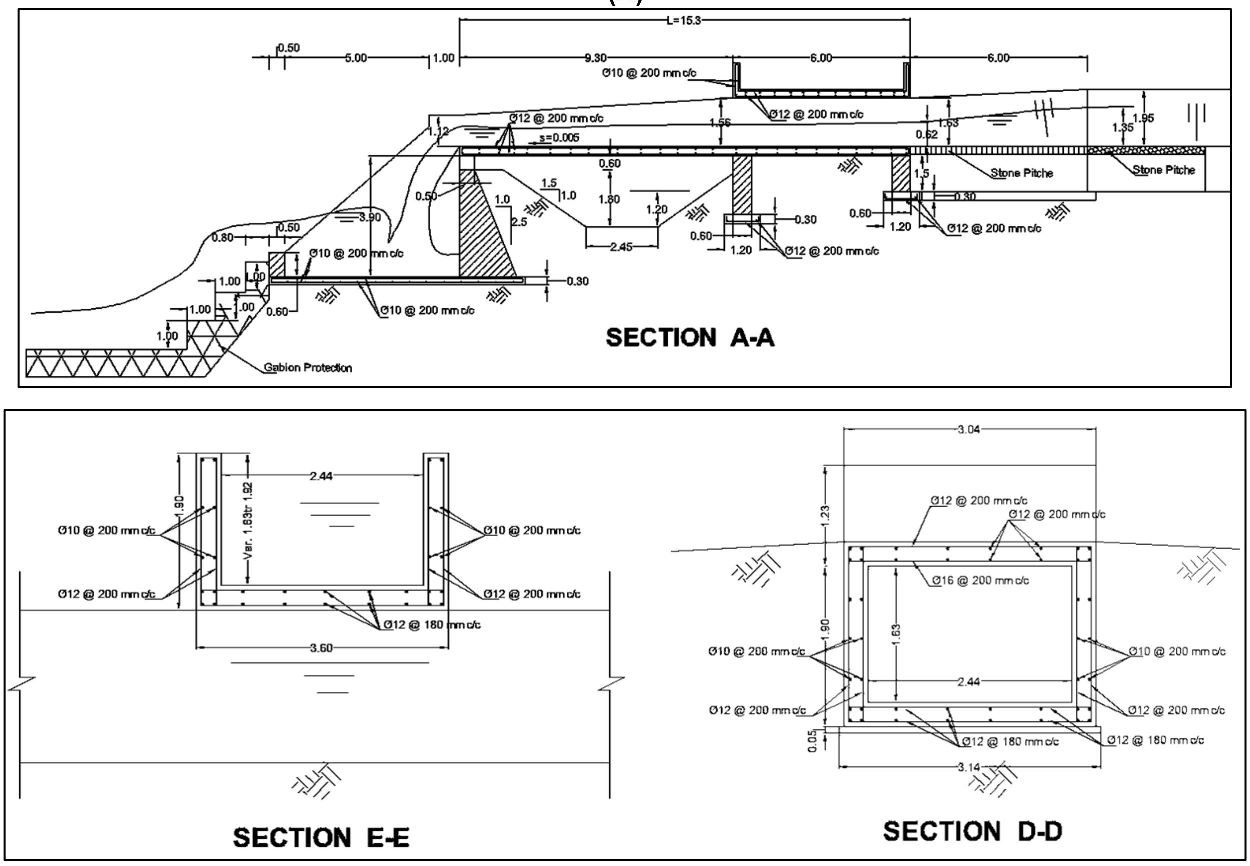
Full supply depth (d_n) = 1.20m, Bed width (b) = 2.45, h_L= d_n+Fb. = 1.80m, Side Slope (ss) = 1.5H:1V

The drainage channel has a well-defined section at the inlet, having properties as follows:

Channel Width (b)= 2.44m, Channel Depth (ho) = 1.95m, Side Slope (ss) = 1.5H:1V, and upstream channel bed slop, s- 0.0002, with n= 0.03.
25-Years frequency flood use for the design (Q) = 5.66m3/sec.



(A)



(B)

Figure 4-44: Over Chute (A) Plan View and (B) Sections and Structural Detail

II) Hydraulic Design

(1) Type of over chute

The techno-economical comparison has indicated that a rectangular concrete over chute is competitive over a pipe over chute.

(2) Over chute width made equal to drainage channel width, equal to 2.44m.
(3) Critical flow hydraulics

By setting the chute invert on a supercritical slope, critical depth, dc, will occur at the upstream end of the chute as shown in Figure 3.42(B).

The discharge per meter of width,

q = Q/b = 5.66/2.44 = 2.32m2/sec.

from equation, $d_c = (q^2/g)^{1/3} = (2.32^2/9.81)^{1/3} = 0.82$m

$v_c = q/d_c = 2.32/0.82 = 2.83$ m/sec.

$h_{vc} = d_c + v_c^2/2g = 0.82 + 2.83^2/2g = 0.82 + 0.41 = 1.23$m

Flow area , $A_c = b*d_c = 2.44*0.82 = 2.00$m2

Weted perimeter, $P_c = b+2d_c = 2.44+ 2*0.82 = 4.08$m

Hydraulic Radius, $R= A_c/P_c = 2.00/4.08 = 0.49$

From the manning formula, with a roughness coefficient of the drainage channel, n= 0.014,

$$s_c = \left[\frac{v_c n}{R^{2/3}}\right]^2$$

$$s_c = \left[\frac{2.83*0.014}{0.49^{2/3}}\right]^2 = 0.0041$$

(4) Invert Slope

To insure super critical flow in the flume, the invert is set on a slope of 0.005, which is steeper than the critical slope of 0.0041 by about 20 percent.

(5) Inlet Depth

The required inlet depth, d_o, is equal to the specific energy at the beginning of the chute, plus transition losses.

Thus,

$$d_0 = E_{sc} + 0.3\Delta h_v$$

$$d_0 = d_c + \frac{v_c^2}{2g} + 0.3\,(0.41 - 0) = 0.82 + 0.41 + 0.3 * 0.41$$
$$= 1.353\text{m}$$

(6) Inlet Bank Height

Providing 0.6m of freeboard above the inlet pool water surface, the upper bank of the drainage channel should be raised to an elevation equal to

d_o+0.6m = 1.353 +.6 = 1.953m

(7) Inlet transition

To transit the flow from the well defined natural channel to 2.44m wide rectangular flume section, a brawken-back transition, simillar to the type-I transition is ideal. Similarly, warped type transition constructed out of stone masonry wall can also be used, as such facility is provided for this particular design.

(8) Chute wall height, h_1 (at inlet)

The wall height, h1, should include a minimum of 0.3m above the upstream water surface, As the invert of the inlet transition is level,

$h_1 = d_o + 0.3m = 1.353+0.3 = 1.633m$

(9) Chute wall height, h_2 (chute channel end)

The height of the chute wall should be equal to the maximum water depth in the chute plus a free board of 0.3m minimum. As the invert slope is steeper than critical, the maximum depth in the chute is dc, which occurs at the inlet end.

$h_2 =$ dmax. + 0.3 = dc +0.3 = 0.82 + 0.3 = 1.12m

(10) Chute Length (L)

L= 6+9.3 = 15.3m

(11) Depth , d_L

To determine the depth, d_L at adistance L from the inlet transition , using the energy equation,

$$E_{sL} + h_f = E_{sc} + s_0 L$$

s_c is previously calculated using n=0.014 from manning formula as, s_c = 0.0041

Assuming d_L= 0.75m

A= b*d_L = 2.44*0.75 = 1.83m2
P= b+2 d_L = 2.44 + 2*0.75 = 3.94m
R=A/P = 1.83/3.94 = 0.465
V_L = Q/A =5.66/1.83 = 3.093m/sec.

$$s_L = \left[\frac{v_L n}{R^{2/3}}\right]^2 = \left[\frac{3.093*0.013}{0.465^{2/3}}\right]^2 = 0.0045$$

$$h_f = \left(\frac{s_c + s_L}{2}\right) * L = \left(\frac{0.0041 + 0.0045}{2}\right) * 15.3 = 0.066m =$$

$$E_{sc} + s_0 L = 0.82+0.41 + 0.005*15.3 = 1.306m$$

$$E_{sL} + h_f = d_L + \frac{v_L^2}{2g} + s_f L = 0.75 + \frac{3.093^2}{2*9.81} + 0.066 = 1.304m \approx 1.306, \text{good trial ok!}$$

4.7.5 Design of Pipe Drops

A pipe drop conveys water from a higher elevation to a lower elevation. This drop in elevation may be any amount between 1.0m and about 4.5m. A pipe drop not only conveys water but it must also dissipate the excess energy and still the water after it has reached the lower elevation.

Pipe Drop Componenets

Pipe drops consist of inlet transition, inlet (control and pipe inlet or check and pipe inlet), pipe and outlet transitions.

Pipe Drop Design Considerations

The hydraulic design consideration of pipe drop includes selecting a pipe diameter for a given capacity that will result in a velocity of 1.0m/sec. or less for a pipe with only an earth transition at one or both ends or 1.5m/sec. or less for a pipe with a concrete structure at each end.

The inclined portion of the pipe should be on a slope of 0.500 or flatter. To dissipate the excess energy from the supercritical velocity in the sloped part of the pipe profile, stilling is accomplished by providing a depression (sumped) sectionof pipe near the outlet end. The depth at which the bottom of the pipe is to depresses below the downstream water surface can be determined from the equation P1+M1 = P2+M2. This is the pressure momentum equation for a hydraulic jump derived from Newtons second law of motion. See figure 4.43 and the explanations and procedures following this figure for the determination of the elevation of the pipe invert at the beginning of the jump. The sumped section of pipe is set with the upstream end at this elevation and with a minimum slope 0.005 towards the downstream end. The length of the sumped section must be at least $4d_2$ (1.8m, min.) for a drop with an earth outlet transition. A drop with a concrete outlet transition should have at least a 5D length for the sumped section and also have $4d_2$ (1.8m min.) length of the pipe between the upstream sloped pipe and the transition but part of this distance can be sloped pipe as shown in Figure 4.43 below.

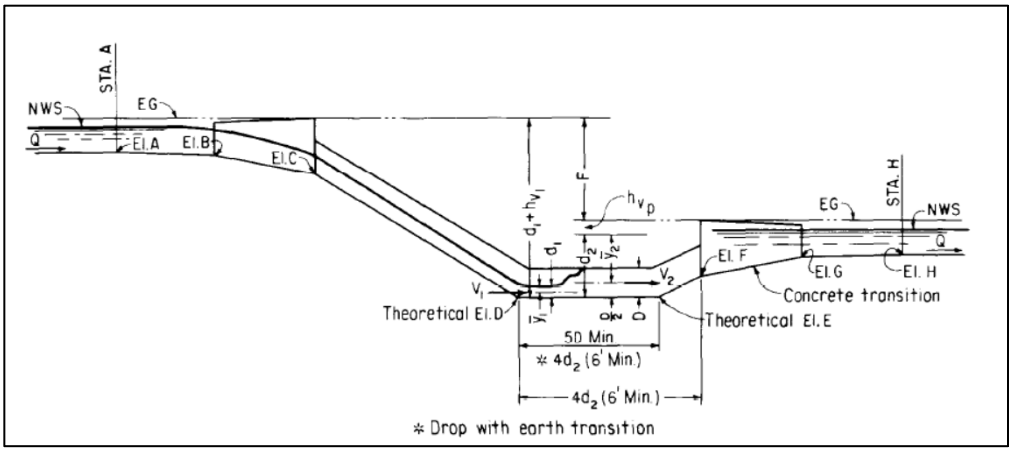


Figure 4-45: Typical section and hydraulic design steps of pipe drop

PROCEDURE: Use hydraulic table in Reference No. 16, to determine cross sectrion are A1 and $\bar{y}_1$ of pipe flowing partially full.

1. Assume value of $\frac{d_1}{D}$
2. Compute d_1 and A_1 from $\frac{d_1}{D}$ and $\frac{A}{D^2}$ found in hydraulic table Reference No. 16
3. $v_1 = \frac{Q}{A_1}$
4. A_2 = Area of full pipe
5. $v_2 = \frac{Q}{A_2}$
6. Compute $\bar{y}_{1\,1}$ from assumed $\frac{d1}{D}$ by using hydraulic tables in Reference No. 16
7. Solve for d_2
8. Solve for F
9. By trial and error, repeat untill computed F= Actual F
10. Elevation of sump= El. Downstream EG- h_{vp} – $1.1d_2$ (10% safety factor)

Hydraulic head losses such as entrance, friction, bend, and exit are omitted because they are small.

Theoretical equation for d_2

$P_1+M_1 = P_2+M_2$

Note: Wt. of water is omitted from all values.

$$\boldsymbol{A_1\overline{y_1} + \frac{Qv_1}{g} = A_2\overline{y_2} + \frac{Qv_2}{g}} \quad , \quad \boldsymbol{\overline{y_2} = d_2 - \frac{D}{2}}$$

$$\boldsymbol{\frac{Q\Delta v}{g} + A_1y_1 = A_2\overline{y_2}} \quad , \quad \boldsymbol{\Delta v = v_1 - v_2}$$

$$\boldsymbol{\frac{Q\Delta v}{g} + A_1y_1 + \frac{A_2D}{2} = A_2d_2}$$

$$\boldsymbol{d_2 = \frac{Q\Delta v}{A_2g} + \frac{A_1}{A_2}y_1 + \frac{D}{2}}$$

$$F = d_1 + h_{v1} - d_2 - h_{vp} \quad , \quad h_{v1} = \frac{v_1^2}{2g}$$

5 SOCIAL INFRASTRUCTURES

5.1 THE NEED FOR THE STRUCTURES

Whenever an irrigation scheme is developed, the community will also seek to get some facilities in addition to the irrigation purpose. The services mainly comprise of water supply for community, cattle and washing closes and structures such as canal crossing bridge to provide smooth movement of the community at the vicinity of the irrigation scheme. The locations and the number of the facilities should be in general fixed with the consultation of the community and stakeholders. However, from the technical requirement point of view, it is recommendable to provide water supply outlets from the main canal to avoid the possibility of introducing public use system within the irrigation scheme. Besides for easy of water abstraction, the location of the water point should be at a lower level from the outlet point. The common type of community use structures drawing which was also used to be provided in previous similar small scale projects is presented here in below as a reference.

5.2 CAMPING SITE

A camping station for the construction crew such as the contractor and supervisor on the project site is indispensable for efficient implementation of the project. Consequently, consultants and contractor's residence and/or office which are made from G-32 corrugated iron sheet /CIS/ has to be designed. It has to be internally partitioned with chipwood wall & ceiling and founded on cemented floor. The rooms are designed such that they are well ventilated as they are equipped with window and door of the same material as shown on the drawing.

The station has to also comprise of 5m*5m store which is constructed from G-32 CIS wall and roof as well as, shower and toilet rooms, Cafeteria and kitchen facility, guard house and Fence work all around the camp of the area.

The camping sites have to be constructed with the acceptable quality workmanship and material, as proposed since it will later handed to "the, would be established" or already existing WUA to serve as office and stores.

5.3 ACCESS ROAD

To carryout operation and maintenance activities of irrigation system effectively and efficiently; access roads into the scheme and within the scheme are critically required. For this purpose, the size and type of access and service/farm roads which are appropriate for the project has to be selected and designed.

Accordingly, as the project is of small scale in its nature there will not be as such a dense network of farm roads along secondary, tertiary and field canals. As discussed in the above chapter only access roads along main canals have to be designed which also has to be connected to the nearby road out let.

Therefore, access roads along the MC of the total length of the MC, with a width of 4.5 m have to be considered and shown on infrastructure lay out. Its typical cross section has to also show and presented in working in the drawing. *Typical drawings of Access road for SSI construction is presented in Appendix F*

5.4 FOOT BRIDGE

These are structures proposed on at footpath crossing sites to allow easy movement of inhabitants in the project area. These have to be designed with reinforced concrete. The design has to be made for lined and unlined canals (as appropriate) depending on their locations which need to be fixed in discussion with beneficiaries during construction

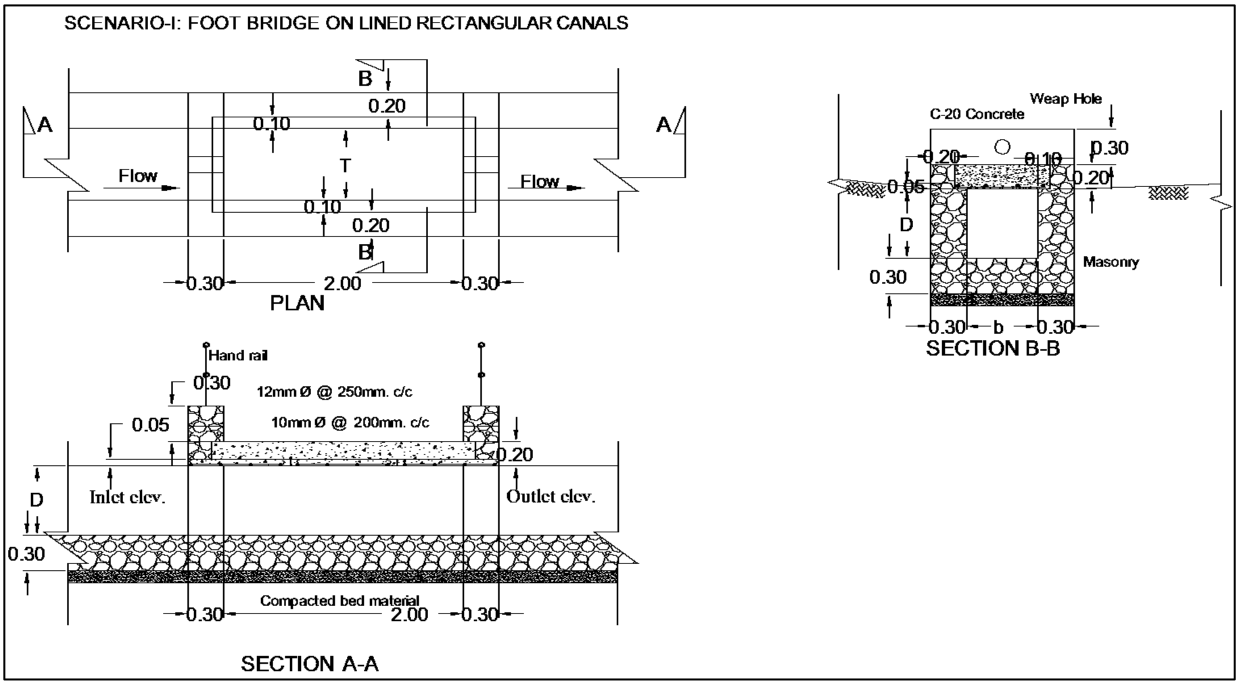
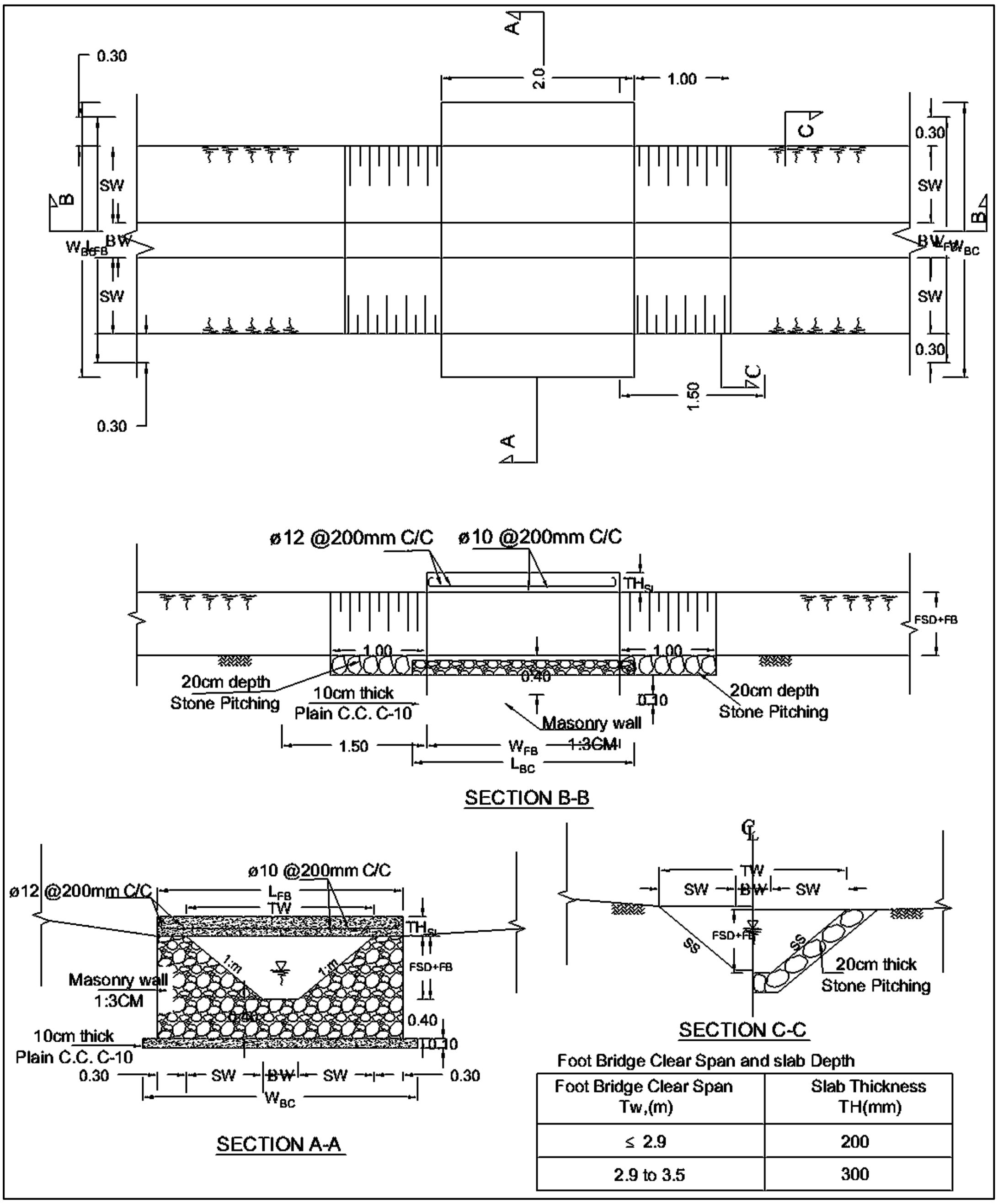


Figure 5-1: Foot bridge on lined rectangular canal



Foot Bridge Clear Span Tw,(m)	Slab Thickness TH(mm)
≤ 2.9	200
2.9 to 3.5	300

Figure 5-2: Foot bridge on earthen trapezoidal canals

5.5 WASHING BASIN

In order to protect the quality of irrigation water from being contaminated by polluted water from washed clothes, provision of facilities for this purpose is essential. The number, location and size has to be decided in consultation with the communities depending on the number of beneficiaries and distribution of villages.

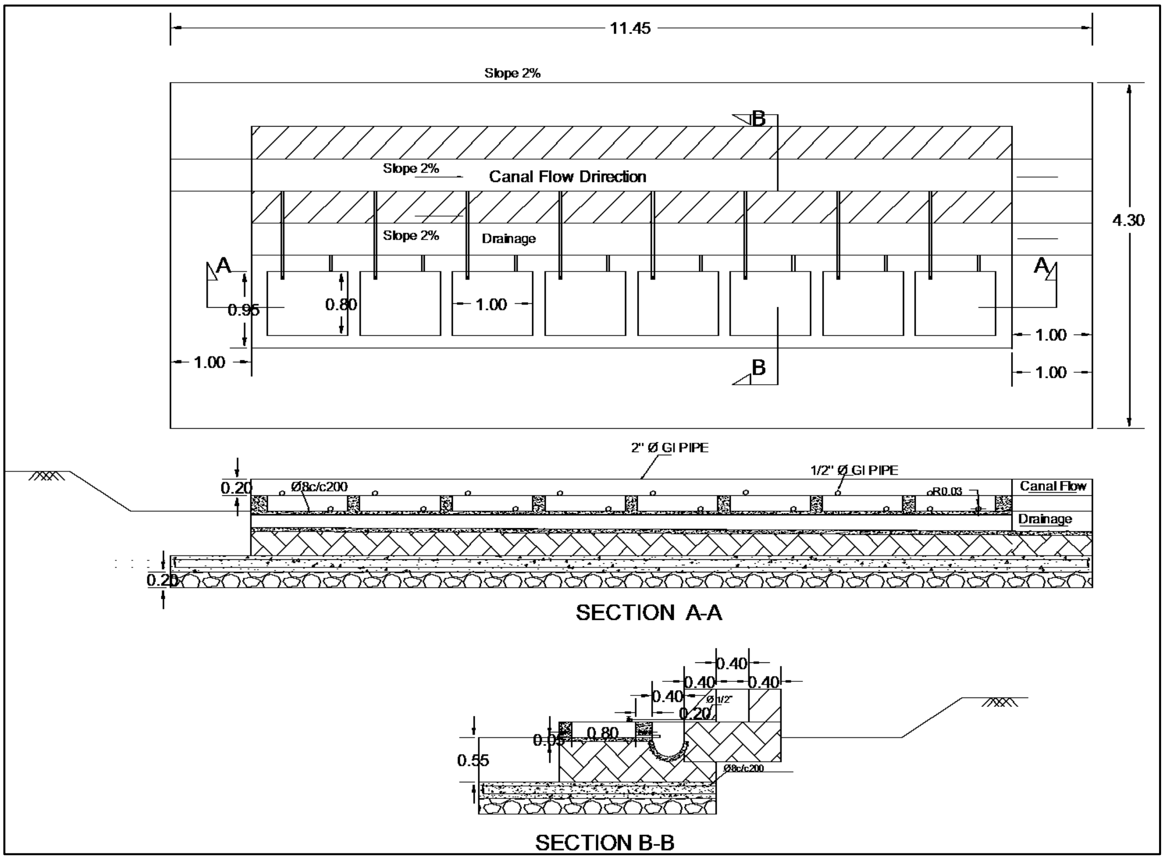
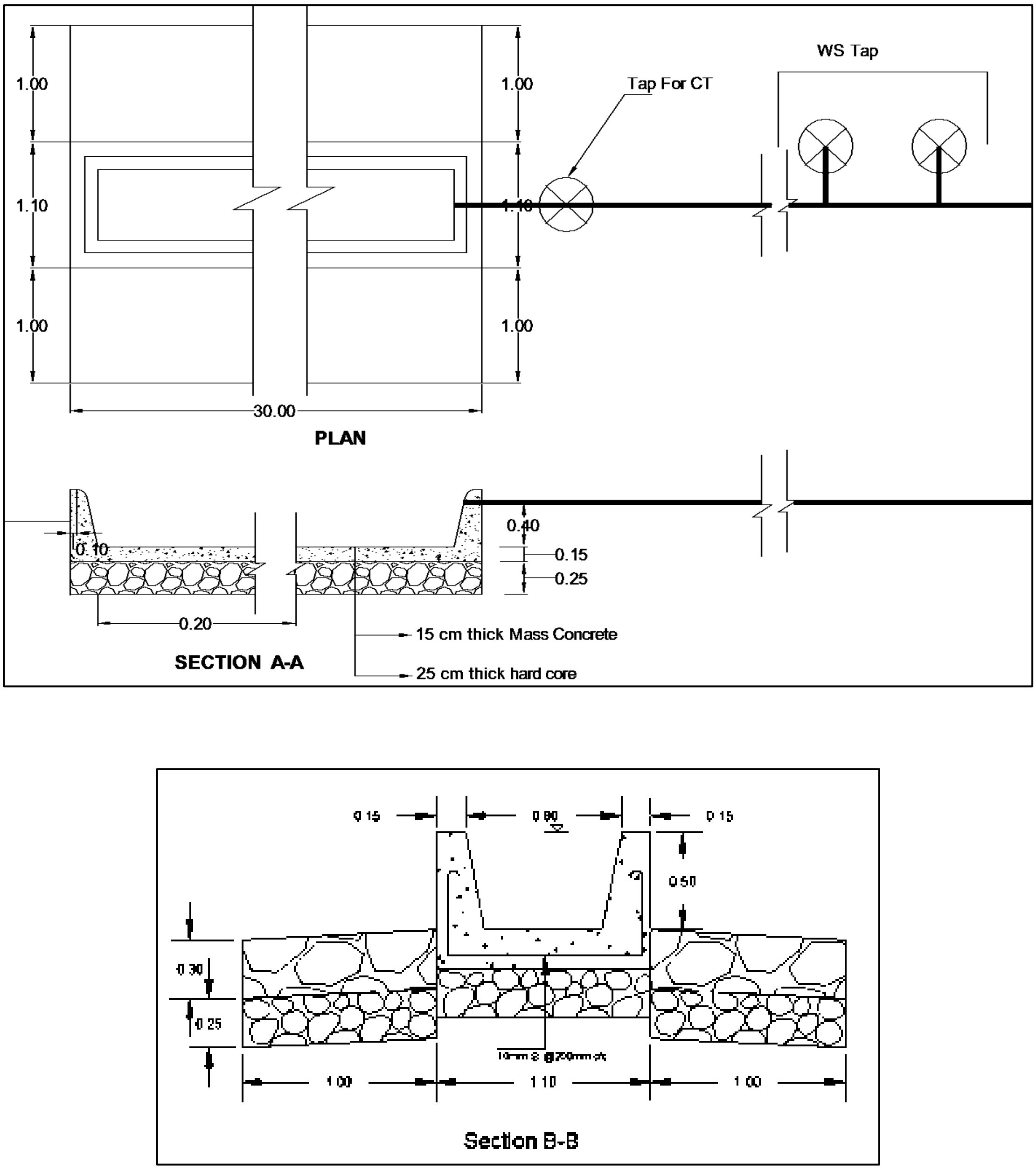


Figure 5-3: Wash basin plan and sections

5.6 CATTLE TROUGH / ANIMAL WATER POINT

Cattle troughs have to be designed so that animals should not disturb or interfere with irrigation operation. Their number, size and locations are to be fixed in discussion with beneficiaries during construction but they need to be placed outside of the canal boundary.



Note: All dimensions in Meter

Figure 5-4: Cattle trough and water supply point plan and sections

REFERENCES

- Anchor, R.D. "Design of Liquid Retaining Concrete Structures", Second Edition 1992.
- Chow V.T., 1950 " Open Channel Hydraulics" McGraw-Hill Book Company, Inc.
- Commission of Sustainable Agricultural and Env. Rehabilitation in Amhara Region "Standard Guideline for study and design work"
- ESRDF, 1997 Small Scale Irrigation Project “Technical Hand book Component IVB-4, Irrigation Structures " 1997.
- Ethiopian Road Authority, ERA-2002 Design Manual, “Standard Drawings”
- Ethiopian Building Code of Standard (EBCS), 1995 " EBCS-2: Structural Use of Concrete".
- FAO, 1975 " FAO 26/1- Small Hydraulic Structures"
- FAO, 1975 " FAO 26/2-Small Hydraulic Structures"
- FAO, 2002 " Surface irrigation systems: planning, design and maintenance, Module 7: Chapter 6: Hydraulic Structures"
- Garg, S.K., 2005 " Irrigation Engineering and Hydraulic Structures" Ninth Revised Edition
- GoV of Nepal, 2006 Guideline for Planning, Design and Monituring & Evaluation of Irrigation Schemes.
- JICA-OIDA, 2010 “Irrigation Master Plan Study Preparation on Surface Water Resources".
- MoANRD, 2000 Irrigation Development Department (IDD) “Irrigation Engineering Practice's Guidline".
- Mosley W.H. and Bungey J.H., 1990 “Reinforced Concrete Design" Fourth Edition
- MoWIE,2002 " Ministry of water Irrigation and Electricity's Standard design of Small and Medium scale irrigation in Ethiopia" 2002.
- United States Department of the Interior Bureau of Reclamation, 1978 “Design of Small Canal Structures"
- 17 United States Department of Agriculture, Agriculture handbook number 590 “Ponds-Planning, Design, Construction”.

APPENDICES

APPENDIX I: Excel Template for Hydraulic Design of Elevated Flume

A- Data for Design

Cells to fill for input data

Hydraulic Characteristics of the canal

Discharge, Q	0.4m^3/sec.
Bed width, B	0.7m
Water Depth, d	0.5m
Side Slope	1.2H:1.0V
Velocity,	0.61m/sec.
Flume Cross sectional shape	Rectangular
Upstream Canal Bed Level (a.m.s.l)	1800m

B- Flume Section

Assume: b (Flume width)	0.9m	
b/d ratio shall be in the ranges between	1.5 to 2.0	
Roughness Coefficient, n	0.014	
Taking b/d ratio equal to 2.0, d= b/2 =	0.45m	
Flow area in flume, A= b*d	0.405m^2	
Velocity, v= Q/A =	0.988 m/sec.	
Froude number, Fr = v/□√gd	0.47 < 0.7	OK!
Fr $\leq$ 0.7, flow is stable subcritical flow.		
Wetted perimeter, P= b+2d	1.8m	
Hydraulic Radius, R= A/P	0.225	

C- Head Loss between upstream and downstream Canal Reach

The Head loss coefficient for the difference of the velocity head between canal and flume is 0.50 for entrance loss and 0.70 for outlet loss. Frictional loss computed from Manning's Equation.

Inlet Loss (h_{fi}), $h_{fi} = 0.5\left(\frac{v_{flume}^2}{2g} - \frac{v_{canal}^2}{2g}\right)$ 0.0154m

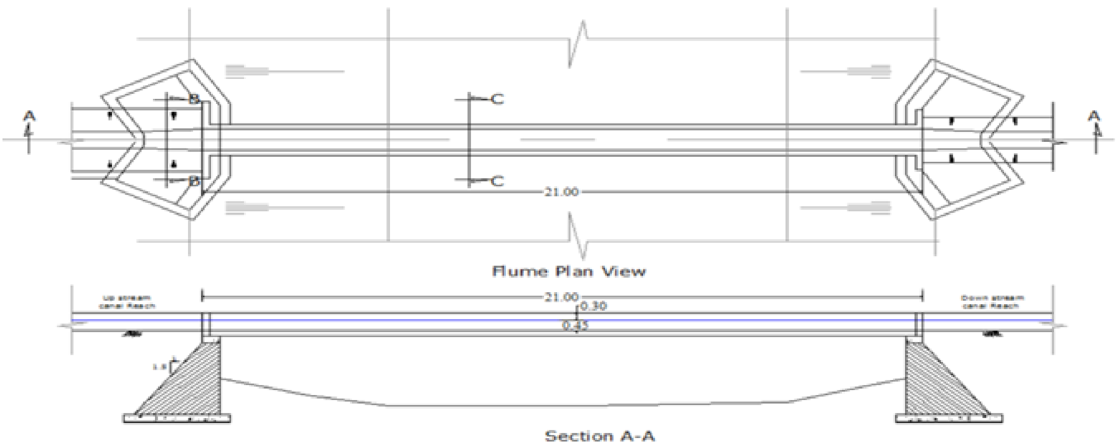
Friction Loss (h_{fr}), $h_f = \frac{n^2 v_{flume}^2}{R^{2/3}} L$ 0.029m

Outlet Loss (h_{fi}), $h_{fi} = 0.7\left(\frac{v_{flume}^2}{2g} - \frac{v_{canal}^2}{2g}\right)$ 0.022m

Total Head Loss (h_f) 0.0662m

D- Elevations at Different Reaches of the Flume

Bed level at upstream (u/s) canal	1800m
Canal Flow Depth	0.5m
Full supply Level, bed level + flow depth	1800.5m
Velocity head at u/s canal, $\frac{v_{canal}^2}{2g}$	0.019m
Total Energy (T.E) at u/s canal, full supply + velocity head	1800.519m
Transition loss b/n u/s canal and flume inlet, as calculated above	0.0154m
T.E at Flume inlet, T.E at u/s canal – Transition Loss	1800.504m
Velocity head in flume, $\frac{v_{flume}^2}{2g}$	0.050m
Water Surface at Flume inlet = 1800.504m - 0.050m =	1800.454m
Water depth at the flume inlet	0.454m
Bed level at flume inlet	1800.00m
Head loss between flume inlet and outlet	0.029m
T.E at flume outlet	1800.47
Water Surface at flume outlet	1800.42m
Bed level of flume outlet	1799.97m
Head loss at downstream (d/s) canal	0.022m
T.E at downstream (d/s) canal	1800.43m
Bed level at d/s canal	1799.93m



APPENDIX II: Excel Template for Hydraulic Design of Inverted Siphon

A- Data for Design

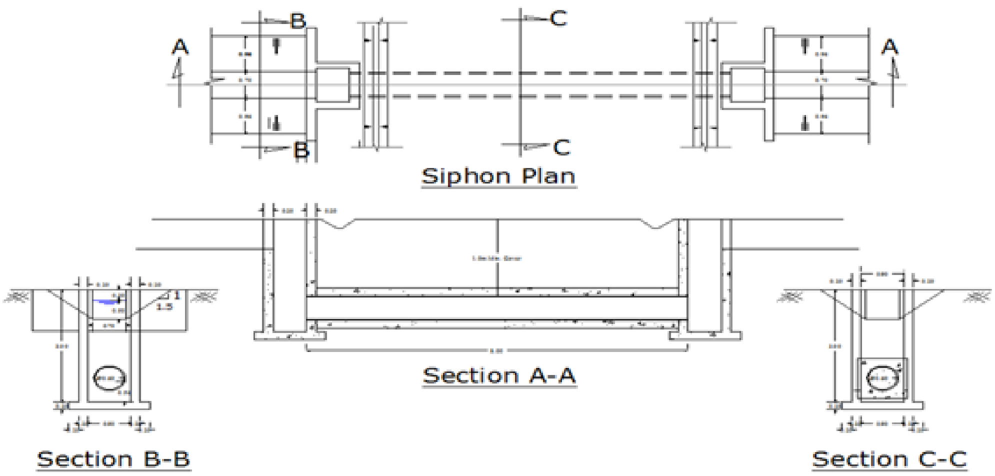
Cells to fill for input data

Hydraulic Characteristics of the canal

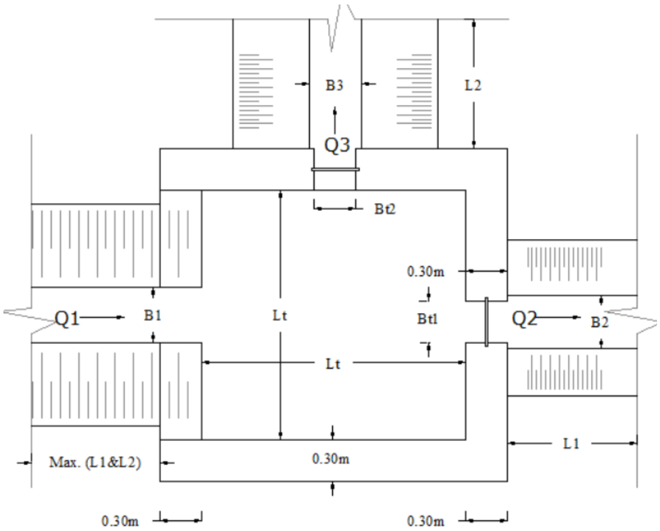
Discharge, Q	$0.4m^3/sec.$
Bed width, B	0.7m
Water Depth, d	0.5m
Side Slope	1.2H:1.0V
Velocity,	0.61m/sec.
Length of Pipe	8.0m
Pipe Diameter	0.60m
Upstream Canal bed level	1800m
Pipe Material	Pvc

B- Flow Hydraulics of Barrel and head Losses

Cross Sectional Area, $A= pD^2/4$	$0.283m^2$
Velocity of Flow in Barrel, $v= Q/A$	1.415m/sec.
Coefficient of Roughness	0.014
Inlet loss coefficient	0.50
Outlet loss coefficient	1.0
Inlet loss , Inlet loss = $0.5(v^2/2g)$	0.051m
Coefficient of frictional loss, $f= 124.5n^2/d^{1/3}$	0.029
Frictional loss in Barrel, $hf = f (L/D)(v^2/2g)$	0.039m
outlet loss , outlet loss = $1.0(v^2/2g)$	0.102m
Total Head Loss	0.192m
Adding 10% extra head as contingency, the total head loss becomes	0.212m
Downstream water level , u/s water level –Total head loss	1800.29
Downstream canal bed level, 1800.29 - 0.5	1799.79m



APPENDIX III: Excel Template for Design of Proportion Flow Divider



Turnout Plan

Parent Canal Data

Incoming Flow in Parent Canal (Q_1)	400 l/sec.
Full Supply Depth (d_1)	0.50m
u/s Canal Bed Level (CBL)	1800.00m
Bed Width (B1)	1.00m
Ground Level (GL)	1800.50m
Full Supply Level	1800.50m
Top Bank Level (T.B.L)	1800.90m
Free Board (FB)	0.40m
Side Slope	1.2:1.0

Branch Canal Data (0^0) (BC1)

Outgoing Flow (Q2)	150 l/sec.
Full Supply Depth (d_2)	0.30m
d/s Canal Bed Level	1799.40m
Bed Width (B1)	0.50m
Ground Level (GL)	1800.50m
Full Supply Level	1799.70m
Top Bank Level (T.B.L)	1800.00m
Free Board (FB)	0.30m
Side Slope	1.2:1.0

Branch Canal Data (90^0) (BC2)

Outgoing Flow (Q2)	250 l/sec.
Full Supply Depth (d_2)	0.40m
d/s Canal Bed Level	1799.40m
Bed Width (B2)	0.40m
Ground Level (GL)	1800.50m
Full Supply Level	1799.80m
Top Bank Level (T.B.L)	1800.10m
Free Board (FB)	0.30m
Side Slope	1.2:1.0

As the proportion of flow is between 1:1 to 1:10 with respect to branching canal and Parent canal respectively, therefore broad crested type weir is more suitable here.
Conditions for the design of Broad Crested Weir :-

(a) The discharge is given by Q = 1.71*K*(Bt - 0.2 h)*(h)1.5
(Where, K= 1.00 for 0^0, K= 0.975 for 45^0, K=0.965 for 60^0, K= 0.95 for 90^0)

(b) The Abutments of the weir are vertical and parallel to each other.

(c) The hydraulic head loss (H_L) is normally 20 % of the driving head (h) i.e, H_L = 0.20h

However for safe design we take H_L = 0.25 h

(d) The throat length (L_t) in the direction of Flow $L_t \geq 2.0$ h

(e) The height of crest (P1) $\geq$ 0.20h

(f) The coefficient of discharge remains constant for the design discharge and below the design discharge.

(g) The distance of end abutment E $\geq$ 0. 20h

Design Calculations :-

Driving Head (h):-

For maximum permissible head loss (H_L), the crest height P_1 should be $\geq$ 0.2 Therefore, selecting P1 = 0.25h
As, h = d_1 - P_1, or h = d_1 - 0.25h, or d_1 =1.25 h, or, h = 0.8 d_1 = 0.40m

Throat width (B_t) :-

	Particulars	BC1	BC2	Remark
(a)	Discharge (Q) in m^3/sec.	0.15	0.25	
(b)	The value of 'K'	1.00	0.95	
(c)	The value of 'h' in m	0.40	0.40	
(d)	Throat width (B_t) in m B_t =(Q/(1.71*K*$(h)^{1.5}$)) +0.2*h	0.43	0.69	
(e)	Crest height above C.B.L (P_1)	0.10	0.10	
(f)	Check for $(P_1/h)^3$ 0.20	0.25	0.25	Ok!
(g)	Head Loss (H_L) (U/S F.S.L-D/S F.S.L)	0.80	0.70	
(h)	Throat Length L_t $\geq$2h	0.80	0.80	
	Provide L_t =	0.95	0.95	Ok!
(i)	Crest Level (U/S C.B.L +P_1)	1800.10	1800.10	
(j)	U/S F.S.L (Crest R.L + h)	1800.50	1800.50	
(k)	D/S F.S.L (U/S F.S.L -HL)	1799.70	1799.80	
(l)	D/S C.B.L (D/S F.S.L-F.S.D)	1799.40	1799.40	

Dimensions Provided:

SI.N0.	Particulars	BC1	BC2	units
1	Crest height (P1)	0.100	0.100	m
2	Driving head (h)	0.400	0.400	m
3	Throat Length (Lt)	0.950	0.950	m
4	Throat Width (Bt)	0.427	0.688	m
5	End abutment (E)	0.080	0.080	m

Calculation of y_1 , y_2 , E_{f1} and E_{f2} for corresponding value of H_L & q by Iteration Method :-

Items	BC1	BC2	Units
As, $(8*q^2*H_L)/g = (-1.5*y_1+((y_1^2/4)+((2*q^2)/(g*y_1)))^{0.5})^3*(0.5*y_1+((y_1^2/4)+((2*q^2)/(g*y_1)))^{0.5})$			
Where, H_L =	0.80	0.70	m
q^2 =	0.12	0.13	
So, $(8*q^2*H_L)/g$ =	0.08	0.08	
Let , the Vaue of y_1 =	0.07	0.07	m
Therefore, $(8*q^2*H_L)/g$ =	0.08	0.08	
After iteration we get the value of y_1 =	0.07	0.07	m
y_2 = $(-0.5*y_1+((y_1^2/4)+((2*q^2)/(g*y_1)))^{0.5})$	0.57	0.57	m
$E_{f2} = y_2 + \{ q^2 / (2*g*y_2^2) \}$	0.59	0.59	m
$E_{f1} = H_L + E_{f2}$ =	1.39	1.29	m

Calculation of y_1 , y_2 , E_{f1} and E_{f2} for corresponding value of H_L & q by Alternate Direct Method :-			
Items	In M/S-2	In M/S-3	Units
Critical y, yc =(q2/g)(1/3) =	0.23	0.24	m
$Z = H_L / y_c$ =	3.44	2.94	
$Y = y_2/y_c = 1+ (0.93556)* Z^{0.368}$ for Z < 1			m
$Y = y_2/y_c = 1+ (0.93556)* Z^{0.264}$ for Z > 1	2.30	2.24	
$y_2 = Y*y_c$ =	0.53	0.53	m
$y_1 = (-y_2/2) + ((y_2^2/4) + (2*q^2/(g*y_2)))^{0.5}$ =	0.08	0.08	m
$E_{f2} = y_2 + \{ q^2 / (2*g*y_2^2) \}$	0.56	0.56	m
$E_{f1} = H_L + E_{f2}$ =	1.36	1.26	m

Calculation of cistern level & length

Sl. No.	Item	BC1	BC2	units
1	Discharge intensity q in cumec/m	0.352	0.363	
2	Upstream water level	1800.500	1800.500	m
3	Downstream water level	1799.700	1799.800	m
4	Head loss H_L	0.800	0.700	m
5	Energy level at downstream E_{f2}	0.556	0.557	m
6	Level at which jump will form (d/s F.S.L.-E_{f2})	1799.144	1799.243	m
7	D/S Floor level provided	1799.400	1799.400	m
8	$E_{f1} = E_{f2} + H_L$	1.356	1.257	m
9	y_1 corresponding to E_{f1}	0.077	0.082	m
10	y_2 corresponding to E_{f2}	0.534	0.534	m
11	Length of cistern of d/s concrete floor required = $5(y_2-y_1)$	2.286	2.259	m
12	Provided d/s floor length (L1 & L2)	2.300	2.300	m
13	Froude No. $F_1 = q / (g*y_1^3)^{0.5}$	5.240	4.950	
14	F_1^2	27.457	24.499	
15	Normal depth of scour $R =1.35*(q^2/f)^{1/3}$ =	0.672	0.687	m
16	R.L. of bottom of d/s cutoff required = d/s water level - 1.5 R =	1798.691	1798.769	m
17	R.L. of bottom of u/s cutoff required = u/s water level - 1.25 R =	1799.660	1799.641	m

Total Impervious floor Length, Exit gradient and floor thickness:

BC1

Safe exit gradient (GE)= 1/5= 0.20 (Assumed)
Maximum static head is exerted when water is stored up to crest level on u/s and no water on downstream.
Maximum static head, H= (Crest Level – Downstream cistern level)

Therefore, H= 0.70m

Depth of downstream curtain wall, d = d/s floor level- R.L. d/s cut of wall
= 1799.40-1798.69= 0.71m (from above table)

$$GE = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}\ ;\ \frac{1}{\pi\sqrt{\lambda}} = \left(\frac{d}{H}\right) * GE = \left(\frac{0.71}{0.7}\right) * 0.2 = 0.2, and\ \lambda = \left(\frac{1}{\pi * 0.2}\right)^2 = 2.47$$

$$\alpha = ((2\lambda - 1)^2 - 1)^{1/2} = 3.82$$

Total floor length required 'b' = ad = 3.82*0.7 =2.70m
u/s floor length required = 2.7 – d/s floor length provided (2.3m from above table) = 0.4m
un balanced head at toe = (H/b)*(d/s floor length) = (0.7/2.7)*2.3 = 0.60m
Floor thickness at toe required =0.60/1.24 = 0.48m

BC2

Safe exit gradient (GE)= 1/5= 0.20 (Assumed)
Maximum static head is exerted when water is stored up to crest level on u/s and no water on downstream.
Maximum static head, H= (Crest Level – Downstream cistern level)

Therefore, H= 0.70m

Depth of downstream curtain wall, d = d/s floor level- R.L. d/s cut of wall
= 1799.40-1798.769= 0.63m (from above table)

$$GE = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}\ ;\ \frac{1}{\pi\sqrt{\lambda}} = \left(\frac{d}{H}\right) * GE = \left(\frac{0.63}{0.7}\right) * 0.2 = 0.18, and\ \lambda = \left(\frac{1}{\pi * 0.18}\right)^2 = 3.12$$

$$\alpha = ((2\lambda - 1)^2 - 1)^{1/2} = 5.14$$

Total floor length required 'b' = ad = 5.14*0.63 =3.24m
u/s floor length required = 3.24 –d/s floor length provided (2.3m from above table) = 0.94m
un balanced head at toe = (H/b)*(d/s floor length) = (0.7/3.24)*2.3 = 0.50m
Floor thickness at toe required =0.50/1.24 = 0.40m

APPENDIX IV: Flexural Design Template for Reinforced Concrete Members

Description	Value	Unit	Remark
M(Design Moment)	50	KNm	
Concrete Compressive Strength (F_{ck})	30	Mpa.	
Steel Tensile Strength (F_{st})	400	Mpa.	
Section width (b)	1000	mm	
Section Height (h)	300	mm	
Clear Cover (c)	30	mm	
Assume Re-Bar Dia.	10	mm	
effective depth(d)	255	mm	
K	0.0256	<	0.167 ok!
z	249.10	mm	
As	576.79	mm2	
No. Bars, in 1000mm width	7.35		
Spacing	136	mm	
Actually Provide	130	mm	c/c

APPENDIX V: Shear Resistance Design Template for Reinforced Concrete Members

Shear resistance for concrete members for depth not greater than 400mm.

$$v_c = 0.79 * \left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} * \left(\frac{100A_s}{b*d}\right)^{\frac{1}{3}} * \left(\frac{400}{d}\right)^{\frac{1}{4}} / \gamma_m$$

- The steel ratio should not be taken as greater than 3,
- The value of effective depth should not be taken greater than 400mm,
- f_{cu} should not be taken as greater than 40N/mm^2,
- γ_m is taken as 1.25.

Sample Shear resistance Calculation

Acting Shear on Str.	Vs	203.58	KN	
Concrete Class	fcu	25	N/mm^2	
Long. Re-Bar Top	Dia 16 at 180mm	1256	mm^2	
Long. Re-Bar Bott.	Dia 16 at 180mm	1256	mm^2	
Total Long. Re-Bar	As	2512	mm^2	
Total	Depth (D)	400	mm	
Effective	Depth (d)	362	mm	
	Width (b)	1000	mm	
	Steel Ratio(ρ) %	0.693923		
Shear stress	vc =	0.573665	N/mm^2	
				ok!
Shear Force	Vc =vc*d*b/10^3	207.6669	KN > Vs	

SSIGL 16

Prepared by

GIRDC