SSIGL 10

NATIONAL GUIDELINES

For Small Scale Irrigation Development in Ethiopia

Diversion Weir Study and Design

November 2018

Addis Ababa

MINISTRY OF AGRICULTURE

National Guidelines for Small Scale Irrigation Development in Ethiopia

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First Edition 2018

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FORWARD

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

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

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

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

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

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

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

SMALL SCALE IRRIGATION DEVELOPMENT VISION

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

ACKNOWLEDGEMENTS

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

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

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

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

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

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

Ministry of Agriculture

DEDICATIONS

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

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ACRONYMS

AGP	Agricultural Growth Program
BCW	Broad Crested Weir
D	Scour depth below river bed
d1or y1	Depth of flow before jump
d2or y2	Depth of flow after jump
dcor yc	Critical depth of flow
DFL	Design Flood Level
ESRDF	Ethiopian Social Rehabilitation and Development Fund
Fr	Froude number
GIRDC	Generation Integrated Rural Development Consultant
Hav	Approach velocity head
Hd	Flow depth on the crest
HEC	Hydrologic Engineering Center
HFL	High Flood Level
JICA	Japan International Cooperation Agency
L	Length
m	meter
m3/s	Cubic meter per second
MoANR	Ministry of Agriculture and Natural Resource
MoWIE	Ministry of Water Irrigation and Electricity
MoWR	Ministry of Water Resource
OIDA	Oromia Irrigation Development Authority
q	Unit discharge per meter
Q	Discharge
Qd	Peak demand or discharge of canal
Qp	Design Flood for the Selected Return Period i.e. Q50 in this Manual
R	Hydraulic mean depth or scour depth or Hydraulic radius
RBL	River Bed Level
RCC	Reinforced concrete
RF	Reinforcement Bar
SB	Stilling Basin
SSID	Small Scale Irrigation Development
SSIGL	Small Scale Irrigation Guideline
SSIP	Small Scale Irrigation Project
SSIS	Small Scale Irrigation Scheme
TWD	Tail Water Depth

USBR	United States Bureau of Reclamation
WC	Weir Crest
WCL	Weir Crest Level
WES	Waterways Experiment Stations, US Army of Engineers
WL	Water Level

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 OBJECTIVES OF THE GUIDELINE

1.1.1 Main objective

To achieve at the above development interventions, the main objective of this guideline is to capacitate active participants of the field of irrigation development sector under the existing conditions to arrive at the most suitable and quality design and consequently increase agricultural production and productivity through developing traditional and modern small scale irrigation projects.

1.1.2 Specific objective

Specific objectives of this component of the guideline is to lay down study and design outlines, present the stepwise procedures and templates for designing geometry, hydraulic and structural aspects in terms of:

- Diversion weir structures,
- Energy dissipaters,
- Barrage structures,
- Wing walls,
- Flood protection structures,
- River training works around headworks, and
- Other related appurtenant components.

1.2 SCOPES OF THE GUIDELINE

This guideline gives a detailed design procedures including templates to be followed for the hydraulic and structural design of diversion headwork structures mainly broad crested weir, ogee weir, barrage and related components. The diversion structures considered in this guideline are those structures with a height not exceeding 3.0m. However, height in excess of this limit requires sound economic, social and other decisive justification. The structure to be considered here could be submerged or free over-fall and it may be established either on rocky or permeable foundation. Its longitudinal length could extend as long as to the requirement of piping and/or energy dissipation requirement, whichever govern. Its width also extends to the existing channel width and/or the requirement to bypass the expected return period flood without disturbing the surrounding ecosystem.

1.3 DEFINITIONS OF TECHNICAL TERMINOLOGIES USED IN THE GUIDELINE

For the purpose of easily understandability of this guideline, the following technical definitions for important terminologies as used in this guideline have been given:

- **Abutment**: That is part of a valley side against which the structure is constructed. Artificial abutments are sometimes constructed to take the thrust on the structure where there is no suitable natural abutment.
- **Afflux**: It is the rise in the high flood level of the river above normal level upstream of the weir (or the bridge in case of non-erodible soils), or barrage, as a result of its construction. It is the difference in water level at any point upstream of weir before and after its construction. Thus, maximum afflux is expected just upstream of the barrage/weir and declines gradually while moving upstream.

- **Apron**: Is the floor area at the upstream or downstream end of a diversion headwork structure to protect the floor against erosion and scouring by water.
- **Banks:** Are lateral boundaries of a channel or stream, as indicated by a scarp, or on the inside of bends, by the stream ward edge of permanent vegetal growth.
- **Barrage** is practically a low weir provided with an adjustable gate over it. Thus heading up of water is affected by gate. This is usually practiced in the case of wider channels so as to minimize cost of the solid structure and to provide a strong benefit where the sediment load of a river flow is enormous. In this case, if flow in the river is limited, pond level = Crest level + shutter height; but in case of weir, pond level ≤ Crest level.
- **Bed material**: Is deposit of materials in bed of a river consisting of particle sizes large enough to be found in appreciable quantities at the surface of a streambed.
- **Canal**: Is a long thin stretch of artificially made waterway for taking water from one area to another or allow movement of boats from one point to the other;
- **Capacity**: A measure of the capability of a channel or conduit to convey water;
- **Channel**: Is the bed and banks that confine the surface flow of a natural or artificial stream;
- **Cofferdam**: Is a temporary structure enclosing all or part of the diversion headwork construction area so that construction can proceed in dry conditions. A diversion cofferdam diverts a stream into a pipe or channel or tunnel (as in case of dam).
- **Control section**: Is a cross section, such as a bridge crossing, reach of channel, or dam, with limited flow capacity, and where the discharge is related to the upstream water-surface elevation. In an open channel, it is a cross-section where critical flow conditions take place. The concept of 'control' and 'control section' can be used with the same meaning/alternately.
- **Crest length**: Is that part of the diversion headwork on which the design flood is bypassed i.e. waterway. If there are piers at both ends or on one side or in between them, then the crest length which is accountable for bypassing this flood is the difference of total crest length and cumulative thickness of these piers, which we call it effective crest length.
- **Critical depth**: Is a depth at which water flows over a weir; this depth being attained automatically where no backwater forces are involved. It is the depth at which the energy content i.e. specific energy of flow is a minimum for the given discharge;
- **Cross-section**: is a hypothetical section line which defines the shape of a channel, stream, or valley as viewed across its axis. In watershed investigations it is determined by a line approximately perpendicular to the main path of water flow, along which measurements of distance and elevation are taken to define the cross-sectional area;
- **Cut-off wall**: Is a wall that extends from the end of a structure to below the expected scour depth or scour-resistant material for the whole crest length to control piping in the foundation.
- **Design discharge or flow**: Is the rate of flow for which a hydraulic structure is designed. Thus the structure is expected to accommodate it without exceeding the adopted design constraints. It is also called design flood and is defined as maximum flood selected/desired for certain return period that any structure can safely pass.
- **Diversion headwork** also called diversion weir or barrage is an overflow i.e. weir or underflow i.e. barrage structure with or without shutter. It is constructed at the head of main canals in order to divert river water toward the canal, so as to ensure a regulated continuous irrigation water supply which is mostly silt free water with certain minimum head into the canal. It can also be defined as a structure/facility/ies which is built at the head reach of conveyance canals for abstracting water from a river into a canal and/or pipe for irrigation purpose. It is one of the headwork structures, and is generally a solid obstruction/structure placed across the river to raise irrigation water to required head/level and allows it to turn away flow in to intake structures/ head regulators so as to irrigate the command area by gravity.

- **Divide wall:** is a long wall structure which is made as high as top level of wing walls or top of crest level depending on size of sluice gate in order to divide the river flow between the under sluice and the main over pass or underpass section of the headwork structure. It may be constructed with stone masonry or cement concrete.
- **Driving head:** is the energy head above top lip of the intake (soffit level) to the design water level or pond level on the upstream of the diversion headwork and is required to motivate flow in to head-regulator side as per the demand.
- **End Sill**: This is a solid structure situated at end of stilling basin for serving as energy dissipation arrangements.
- **Energy grade line**: Is a hypothetical line joining elevation of energy heads; a line drawn above the hydraulic grade line a distance equivalent to the velocity head of the flowing water at each section along a stream, channel, or conduit;
- **Exit Gradient** (EG): The slope (or gradient) of hydraulic grade line (for subsoil seepage flow), at the exit end of the structure where the seepage water comes out from subsoil. When the upward seepage force acting on soil at the exit end of the structure is exactly balanced by the submerged weight of the soil, the exit gradient is known as the Critical Exit Gradient, CEG.
- **Fish ladder** is provided just by the side of the divide wall for free movement of fishes. In general, fishes tend to move from upstream to downstream in rainy season and from downstream to upstream in dry season. Thus we should allow such essential movement of fishes for their survival (if any from Environmental study report). In the fish ladder, baffle walls are constructed in a zigzag manner so that velocity of flow within the ladder remain small or does not exceed 3 m/s. The width, length, and height of the fish ladder depend on the nature of the river and the type of the weir or barrage.
- **Floodplain**: Is the alluvial land bordering a stream or river bank, and is formed by stream processes and is subjected to inundation by floods;
- **Floor length:** is the total length of impervious floor provided upstream or downstream of a structure. It consists of upstream floor, upstream glacis, downstream glacis, downstream stilling basin and end sill.
- **Freeboard**: is the vertical distance between level of the water surface usually corresponding to design flow including afflux and a point of interest such as top of a wing-wall or flood protection dyke.
- **Froude number**: It is a dimensionless number proportional to the square root of the ratio of the inertial forces to gravitational force i.e. to the weight of fluid.
- **Gabion**: It is a cylindrical metal container made of wickerwork basket which is to be filled with rocks/stones and/or earth material for use in the construction and rerouting of waterways, diversion weir and in flood control.
- **Gate**: This is a valve or system for controlling/regulating the passage of water. In open channels the two most common types of gates are the underflow gates (e.g. scouring sluice gate and Intake gate) and the overflow gates (e.g. Shutter gate in barrages).
- **Guide bank:** When a barrage is constructed across a river which flows through the alluvial soil, the guide banks must be constructed on both the approaches to protect the structure from erosion. It is an earthen embankment with curved head on both the ends. It serves: (a) To protect the barrage from the effect of scouring and erosion. (b) To control tendency of changing the course of the river. (c) To control velocity of flow near the structure.
- **Head:** is the difference in water level between two reference points and is thus energy required to drive water from higher point to the lower point (for gravity flow).
- **Head-loss** is energy dissipated due to the resistance to flow from the material in which it is flowing, h_l = S*L= ΔH, for open channel flow, where ΔH-is head difference, S-is longitudinal slope, L-is length but for pipe flow, $h_l = hf+KV^2/2g$.
- **Headwork**: is any hydraulic structure located across the stream or on the lake, reservoir and/or ground water to collect, reserve or divert water for irrigating crops and/or hydropower use. Thus it includes Diversion Headwork/Weir, Free Intake

structure, Pump, Spring Protection or Development, Dam and Ground water extraction. In a storage system, such structure is called a 'Storage dam' and the main body of the structure is mostly 'Earth-dam' where-as for a diversion system, it is called a 'Weir', and the water pool is called a 'Pond'.

- **Hydraulic jump:** is a flow phenomenon in which flow transit from a rapid or supercritical flow to a slow flow motion i.e. subcritical flow;
- **Intake structure:** It is also called ahead regulator structure, is a structure on the headwork to divert or offtake flow. In a diversion headwork, it is situated at the immediate upstream end of a headwork consisting of a chamber, trash-rack, gate and sometimes provision for stop-logs. It is thus part of the structure in a weir through which water is drawn into a canal or pipe by extending to upstream end of a channel;
- **Irrigation project**: is a time bounded activities consisting of development of irrigation and related infrastructures like access road, bridge, camping, drainage, on-farm structures, social service structures, etc. for supporting artificial watering of land to sustain plant growth.
- **Marginal embankments or dykes:** are earthen embankments which are constructed parallel to and on either or both sides of the river bank based on prevailing condition to prevent the flood water from inundating the surrounding area and facilities.
- **Meandering channel**: is an alluvial stream characterized by a series of alternating bends (i.e. meanders) as a result of alluvial processes;
- **Normal depth**: Is a uniform equilibrium open channel flow depth. It is the depth of flow in a channel or culvert when the slope of the water surface and channel bottom is the same and the water depth remains constant. Normal depth occurs when gravitational force of the water is equal to the friction drag along the culvert and there is no acceleration of flow;
- **Piping** is a phenomenon which results when flownet beneath the structure having pressure at downstream end of the structure is more than critical exit gradient and thus brings soil particles with it on the downstream side. Such seepage, if uncontrolled causes springing at the downstream and finally results in hollows under the floor causing collapse of the floor and/or the structure as a whole.
- **Pond level**: The level of water, immediately upstream of the headwork. It is required to facilitate withdrawal into the canal or for any other purpose. In case of a diversion weir, the pool level is at the crest level of the weir;
- **Protection works**: These are protection mechanisms which are required both on the u/s and d/s of a weir to prevent possibility of a scour hole moving close to the u/s or d/s Cut-offs which otherwise undermine the structure. On the u/s side the need is due to higher velocities of flow near the structure due to draw down; whereas, on the d/s side the need is due to the turbulent nature of flow as it leaves stilling basin to guard against higher than expected exit gradients. When a weir is constructed on rock, such protection works are not required. They include block works like apron and inverted filter and launching apron;
- **Riprap**: It is a layer of rock, either dumped or hand-placed to prevent erosion, scour or erosion of river bed or sloughing of river bank.
- **River training**: is the stabilization of the channel in order to maintain the desired cross section and alignment. Its purpose is for maintaining sufficient channel conveyance capacity and depth so that the intended design flood may not overtop the bank and inundate the surrounding.
- **Scour**: Is a hydraulic phenomenon characterized by removal of bed material. It is caused by the eroding power of the flow.
- **Sediment**: is any material carried in suspension by the flow or as bed-load which would settle to the bottom of hydraulic structures in the absence of flow;
- **Silt factor, f** is a factor related to grain size and defines average particle size of the material forming bed of channels;
- **Specific energy**: is that part of the total energy measured above the bed level of channel i.e. the potential energy is ignored. It is quantity proportional to the energy per

unit mass, measured with the channel bottom as the elevation datum, and expressed in meters of water.

- **Stilling basin**: is a solid impervious floor or apron structure on the downstream of a main weir body, spillway, outlet work, chute or canal structure. It is required to dissipate excess energy of falling water in the form of hydraulic jump so as to prevent scour and undermining of structures and damage from waves;
- **Stop-log**: is a form of gate comprising a series of wooden planks, one above the other, and held at each end to be used when maintaining main gate.
- **Streamlines**: These are the lines drawn so that the velocity vector is always tangential to them i.e. no flow across a streamline. They are imaginary flow lines that are always parallel to the local direction of the flow, and that for steady flow are also the lines followed by individual fluid particles. When the streamlines converge it shows the velocity is increasing;
- **Subcritical flow**: is flow in open channel when the flow depth is larger than the critical flow depth or the Froude number, $F_r < 1$. In practice, subcritical flows are controlled by the downstream flow conditions. It is a flow where the predominant part of the specific energy is dictated by the depth of flow i.e. the gravitational force is in excess of the inertia force.
- **Supercritical flow**: is flow in open channel when flow depth is less than the critical flow depth. In a supercritical flow the Froude number, F_r is larger than one. Supercritical flows are controlled from upstream. It is a flow where the predominant part of the specific energy is dictated by the velocity head i.e. the inertia force is in excess of the gravitational force.
- **Submerged flow**: is flow condition which exists when a change in the downstream water surface elevation causes a change in the upstream water surface elevation. Thus, downstream energy influences flow rate at that specific control section.
- **Tail water depth**: This is the normal depth of flow immediately downstream of the structure;
- **Under-sluice**: is a flushing device in a diversion headwork structure constructed adjustment to the head regulator on one and/or two abutments to control entry of sediment in to intake structures by rejecting sediment during flooding.
- **Uplift**: is an upward pore water pressure or interstitial pressure in the pores of bed material under the base of hydraulic structures. It can led to the destruction of stilling basins and even to the failures of concrete dams, if not treated/managed, thus it dictates the thickness of floor of stilling basins;
- **Weir crest:** is part of the diversion weir structure over which excess flow passes from upstream to the downstream side. Its top level must be above the intake level so a s to enable required flow depth to the intake side.
- **Wing walls**: are also called retaining walls and are designed immediately on the u/s and/or d/s of main weir body to protect submergence of the structure as well as its environs during flooding. They are laid on an impervious concrete floor either on one and/or both sides of the weir depending on stability and nature of surrounding topography.

2 DIVERSION WEIR AND ITS APPURTENANCES

2.1 GENERAL ARRANGEMENT OF TYPICAL DIVERSION WEIR

Any design engineer must know functions of a diversion headwork and its appurtenances and visualize it so as to fix its size which best fits the existing conditions at the selected site. The following figure shows a typical arrangement of diversion weir with its main components.

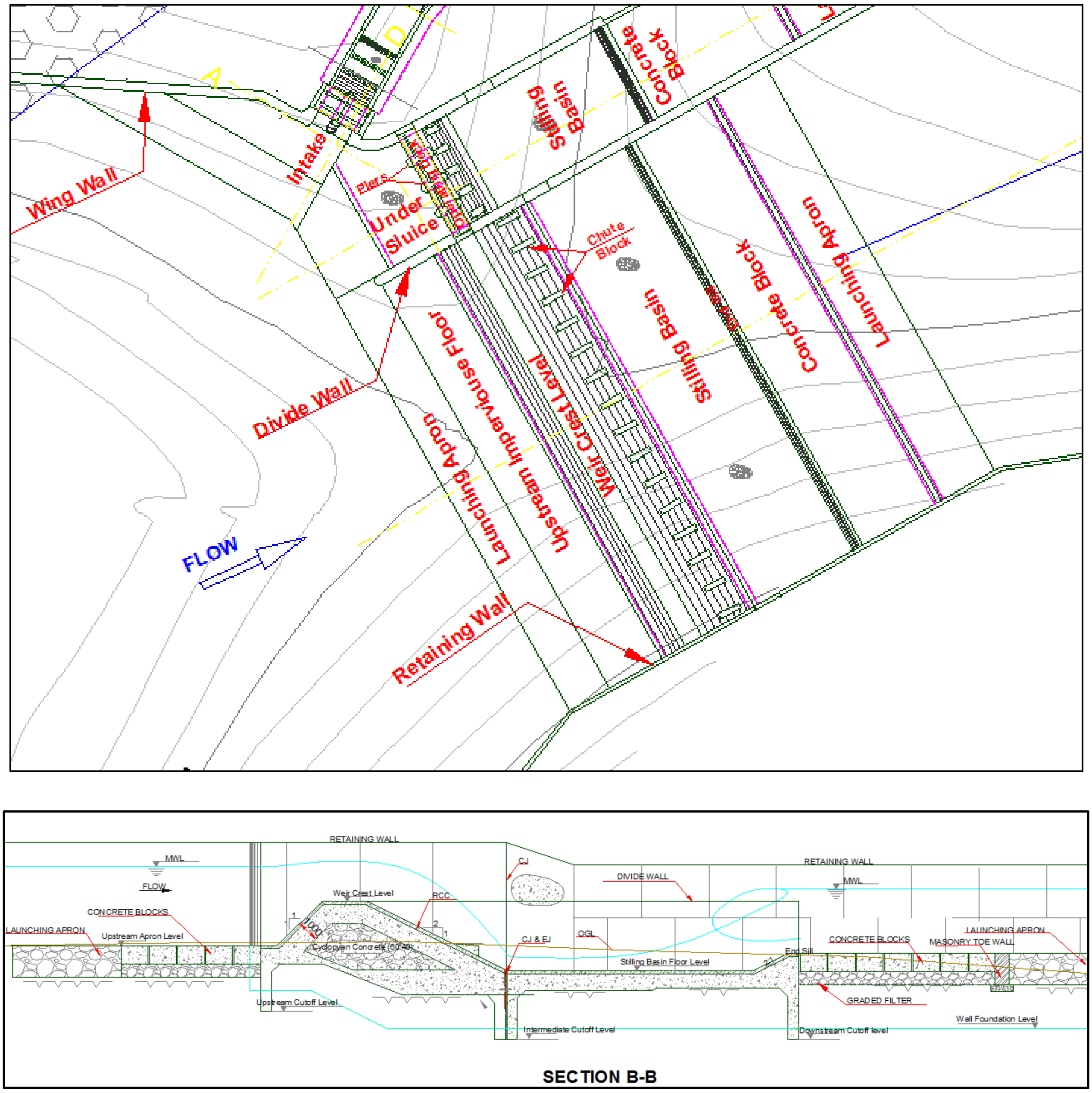


Figure 2-1: Main components of typical concrete diversion weir

2.2 FUNCTIONS OF DIVERSION HEADWORK AND ITS COMPONENTS

2.2.1 Functions of diversion headwork

The following are main functions of diversion headwork based on their purpose:

- To regulate water supplies in to the off-taking canal and river downstream,
- To raise water level in the river for maximizing the size of command area;
- To provide a stable riverbed level to abstract regulated flow from the river;
- To provide an impermeable cut-off to bed rock and drive sub-surface flow to the surface often a major requirement for weirs in arid climates;
- To sustain continuous flow towards the intake;
- To reduce the beneficiaries work load in constructing temporary diversion bunds after every flood season;
- To increase reliability of water supply.
- To form a temporary storage by construction of dykes on both side of banks of the river so that water is available throughout the year.
- To control entry of silt into the canal and to control the deposition of silt at the head of canal head regulator, and
- To control fluctuation of water level in the river during different seasons,
- To provide a positive cut-off to bed rock and thus drive sub-surface flow to the surface.

2.2.2 Components of a diversion headwork

To achieve at the aforementioned functions of diversion headwork, the structure need to comprise either part or all of the following components based on site specific conditions and intended purpose:

- Main weir body including part between sluices channels (Necessary)
- Intake Structure also called head regulator and that part embedded in wing walls (Necessary)
- Sediment exclusion structures, like Scour/Under sluice and/or sand trap (Necessary)
- Silt Ejector (Conditional, depending on transported bed material condition)
- Intake and Under-sluice gates, i.e. spindle or sliding depending on flood condition (Necessary)
- Stilling Basin and End Sill (depending on local bed material condition and magnitude of flood) (Conditional, based on d/s foundation condition)
- Protection Works (Conditional, depending on local bed material condition)
- Divide Wall (Conditional, depending on amount of lean flow availability)
- Wing walls (Conditional, depending on size of flood, bank stability and level of surrounding ground)
- River training bunds/marginal embankments or dykes (Conditional, depending on magnitude of flood and level of surrounding areas and facilities)
- Guide Bank (Conditional, depending on magnitude of flood and level of surrounding ground)
- Fish Ladder (Conditional, depending on availability of such resources)

2.3 CLASSIFICATION OF WEIR

2.3.1 General

Based on the availability of construction material, nature of flow and foundation condition at the diversion site, the design engineer is expected to select types of diversion weir that suit to the selected site. Consequently, understanding their types and characteristics is very crucial in order to select the appropriate one for the proposed headwork site. The following sections present classification of weir based on different considerations..

2.3.2 Classification of weir based on purposes

Based on their functions, weirs can be divided in to:

2.3.2.1 Diversion weirs

These are solid obstructions put across the river to raise or heading up its water level and divert the water into the conveyance canal. Thus they are weir types constructed across a river for the purpose of diverting water in to the canal. They are used for irrigation as well as hydropower purposes.

2.3.2.2 Pick up weirs

In this case, there is separate storage dam upstream and the reservoir water is then released in to the river through supply sluices or bottom outlet in a regulated manner. This released water is allowed to flow through its natural channel and then picked up by constructing a weir across the river where conveyance canal take off. A series of such pick up weirs may be constructed to utilize the available water. Such type of weir are usually used where the distance between the location of the reservoir (water storage) and its command is quite very far and uneconomical to construct a canal and the natural channel is expected to have very low water losses due to seepage. The pickup weir below Kessem dam is a typical example.

2.3.2.3 Storage weirs

These types of weir are also called low dams as they are used for storing water temporarily for tiding over small periods of short supplies. They are used to account for the case when the upstream farmers use large amounts of water and there is a risk of not getting the scheduled minimal flow for the downstream ones, e.g. Cherechera Weir at starting point of Abbay River is used as a storage/regulating weir.

2.3.2.4 Waste weirs

These are also called discharge weir and serve as discharge levelers and are generally used as spillway for reservoirs to protect the reservoir and the main storage dam.

2.3.2.5 Gauging/ measuring weir

Such weirs are used as a control structure for measurement of discharge on a river or canal.

2.3.3 Classification based on construction material

Diversion weirs can be subdivided in to three types based on material used for construction, design features and types of soil foundation. It can also be classified based on stabilizing factor though in this case, it can be made from any one or combinations of the aforementioned categories.

2.3.3.1 Masonry weir

This type of weir is made from locally available sound rock material. It is the most commonly constructed type for SSI Projects in our country. Here, a crest gate may or may not be provided to store more water during flood period. At the u/s and d/s ends of impervious floor, cut off piles are provided if foundation material is loose. Launching aprons are also provided both at u/s and d/s ends of floor to safeguard against scouring action. A graded filter is provided immediately at the d/s end of impervious floor to relieve uplift pressure. This type of weir is suitable for any type of foundation. It is of usually constructed with vertical face in the u/s and sloping in the d/s face.

2.3.3.2 Rock-fill/Gabion weir

Such a weir is also called 'Dry Stone Sloping Weir'. It consists of body wall. Dry stones are laid in the form of glacis with some intervening core wall on its upstream and downstream sides. It is suitable for fine sandy foundations like those in alluvial areas. Such a weir requires huge quantities of stone and is economical when stone is easily available. The stability of such a weir is not amenable to theoretical treatment. However, with the development of concrete glacis weirs, this type of weir is also becoming obsolete. They usually have sloping apron as shown below.

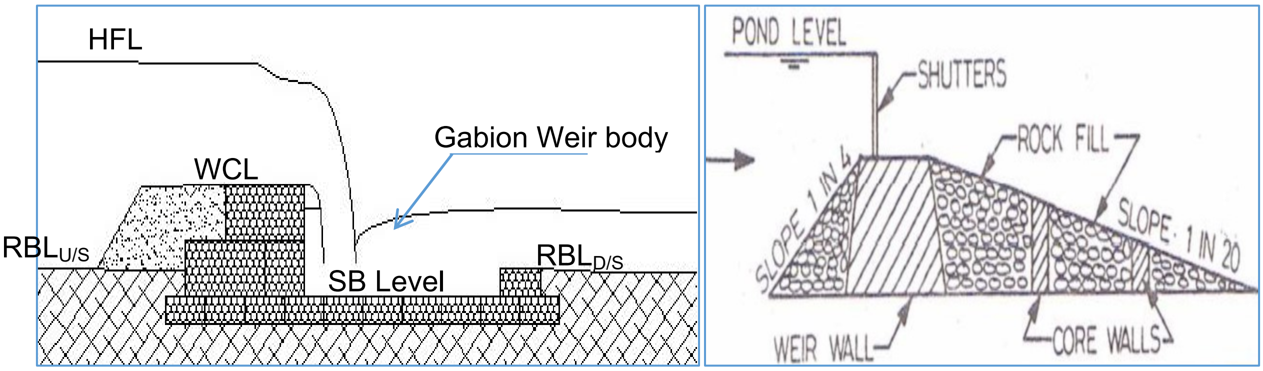


Figure 2-2: A Sketch of a typical gabion weir (l) and rock-fill weir (r)

Concrete glacis weirs type are now exclusively used especially on permeable foundations and are generally provided with a low crest with counter-balanced gates, thus, making it a barrage.

2.3.3.3 Concrete weirs

This type of weir is the strongest and it is constructed from the most expensive material, thus used when we don’t care for cost of the structure but try to manage incoming high flood magnitude and flood carrying large boulders. It is selected when the foundation material is soft and sandy and used where difference in weir crest and downstream riverbed is not more than 3 m. Hydraulic jump is formed when water passes over the sloping glacis. Weir of this type is of recent origin and commonly has sloping glacis.

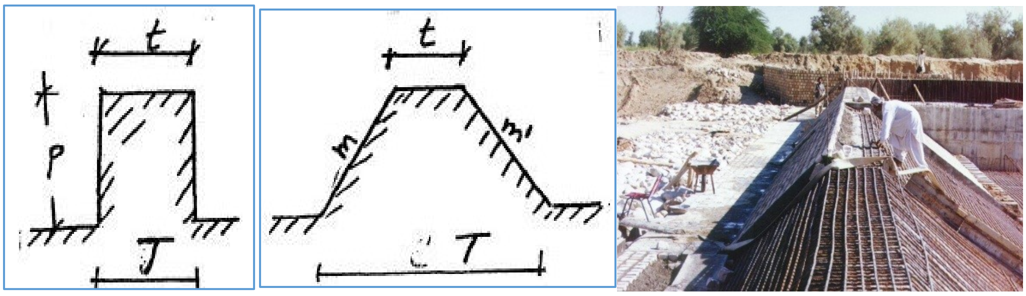


Figure 2-3: Sketches of typical rectangular (l) and trapezoidal/glacis type weirs (r)

In this family of weir types, there are two subgroups: Reinforced Concrete and Cyclopean concrete weirs. In Reinforced Concrete weir, all components aprons, crest, glacis, etc. are constructed out of reinforced concrete. Mostly sloping downstream glacis is provided to create hydraulic jump in a short distance. In case of cyclopean concrete, the major control sections of the weir is constructed out of cyclopean concrete i.e. a mix of rubble stone and cement concrete at the ratio of 1:1.9,as per MOWE Guide line, 2002.

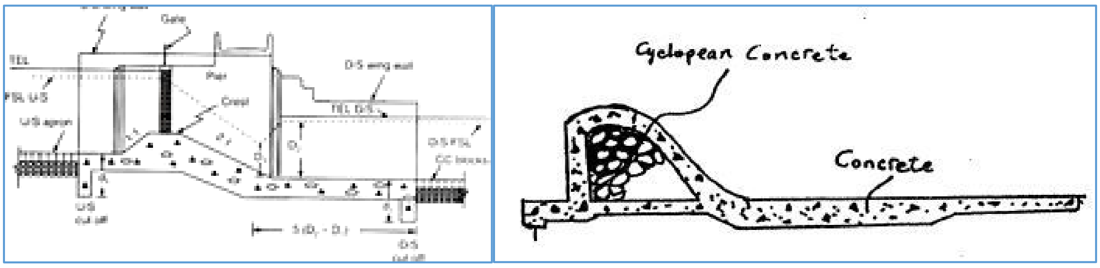


Figure 2-4: Typical longitudinal section of rcc (l) cyclopean concrete weir (r)

The sloping glacis weirs have the inherent advantage of stability. On rivers subject to high velocity flows carrying boulders, weirs should be made as low as possible and a shallow glacis weir would best transport boulders safely over the weir. Glacis weirs are used as diversion, head-regulator and sloping drop structure (when found high, usually when greater than 2m height).

2.3.4 Classification based on design features

Based on design features, diversion weirs can be classified in to three categories:

2.3.4.1 Parabolic/Ogee weir

A parabolic weir is almost similar to spillway section of a dam. It has a high hydraulic performance (C=2.2) than other weir types (e.g. C=1.7 for Broad Crested Weir). A weir with such high coefficient gives a more economical design because the crest length can be reduced for the same head and discharge. The weir body wall for this type is designed as low dam. A cistern/stilling basin is provided on its downstream side.

Such weir is commonly constructed from concrete and is preferred to allow higher discharge on rivers that transport relatively larger sized gravel and boulder. Its Disadvantages are the difficulty in constructing it accurately and soundly, particularly from reinforced concrete and the danger of reduction in coefficient of discharge if debris and sediment build up in front of the weir, thereby reducing discharge capacity. The portion of the ogee profile upstream of the origin is defined as a compound circular arc. The portion of ogee profile downstream of the origin is defined by the equation presented in equation 3-23.

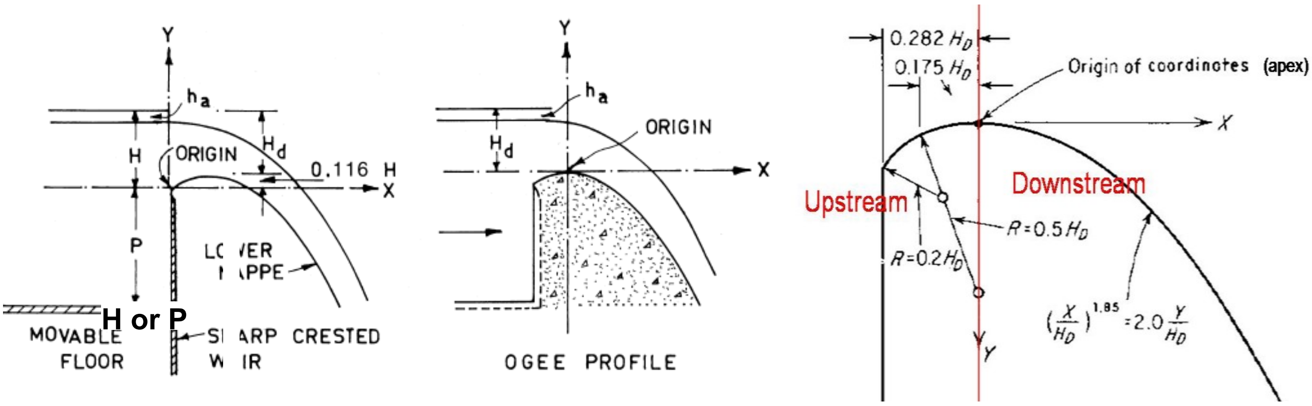


Figure 2-5: Ogee weir shape and corresponding parameters (Khatsuria, 2005)

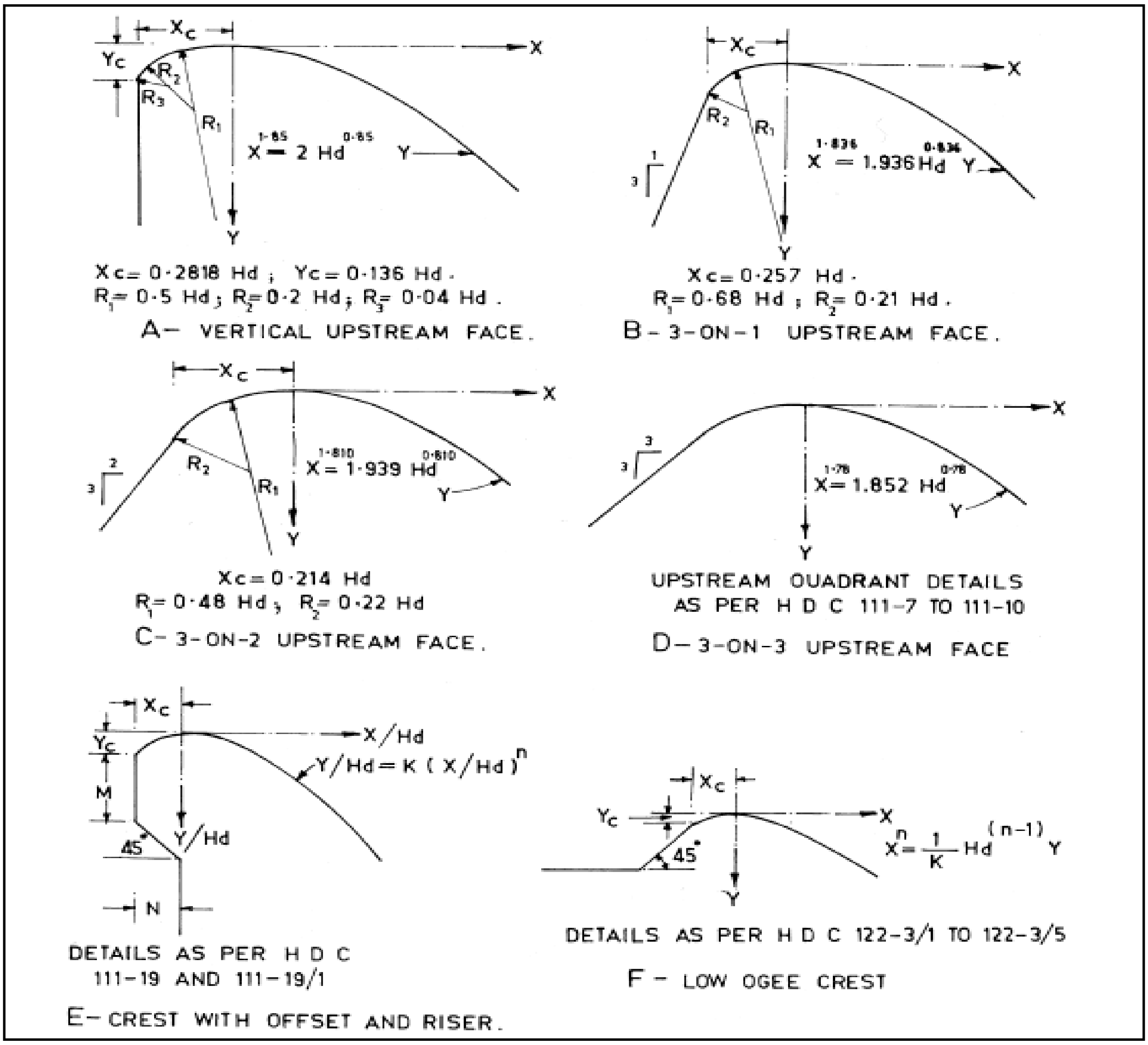


Figure 2-6: Different shapes of ogee weir (Khatsuria, 2005)

2.3.4.2 Broad crested weir

This is a robust flat-crested structure, with a long crest compared to flow thickness i.e. depth of flow. When the crest is "broad", the streamlines become parallel to the crest invert and the pressure distribution above the crest is hydrostatic. Practical experience have shown that the weir overflow is affected by the upstream flow conditions and the weir shape. This type of weir is the most commonly designed and constructed diversion structure in our country as it is simple to design and construct with locally available skilled manpower i.e. masons. If it is well constructed from sound hard rock and good workmanship, stone masonry weir resist abrasion by far higher than concrete. For a weir to be broad crested 0.5He < b < 2He, where b is crest width of the structure.

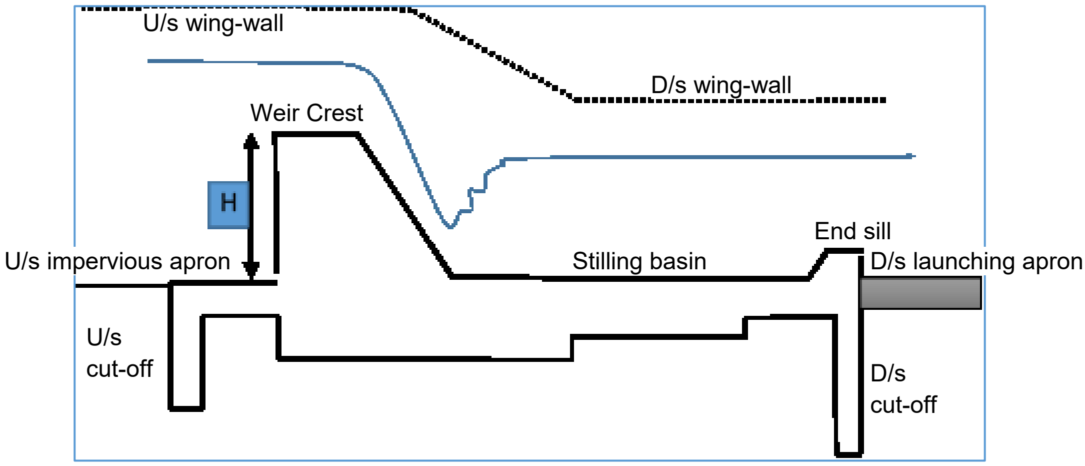


Figure 2-7: Typical longitudinal section through broad crested weir

2.3.4.3 Sharp crested weir

Sharp Crested weirs prevails when the energy depth *H* is greater than half of the crest width. They are usually used for flow measurement and thus come in many different shapes and styles, such as rectangular (with and without end contractions), V-notch and Cipoletti weirs. Under controlled conditions, sharp crested weirs can exhibit accuracies as good as ±2%, although under field conditions accuracies greater than ±5% should not be expected. The discharge equation in this case is given as:

$$Q = 1.84\,(L - 2t)\,H^{3/2} \quad \text{(2-1)}$$

Where, Q is design flood over the weir crest (m^3/s)

(L – 2t) is the structure width minus the two end contractions (m),

L is width of the channel at that particular section, (m)

t is end contraction (m).

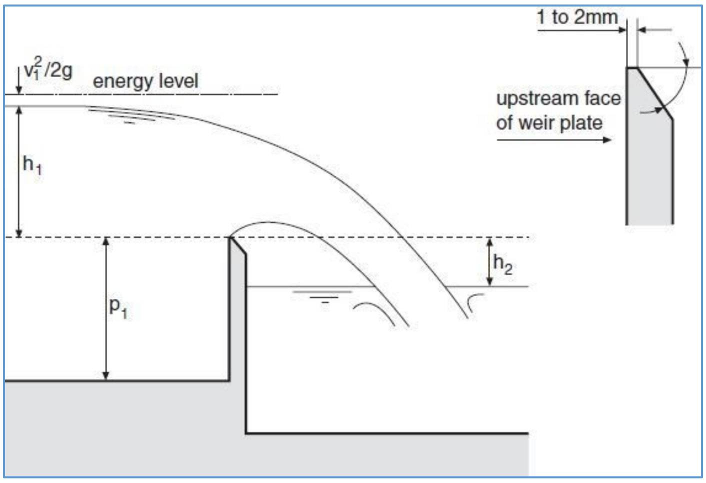


Figure 2-8: Typical section of sharp crested weir

The reduction in the coefficient for broad crested weirs as compared to ogee weirs is not very large. Some negative pressures can develop on the weir for heads exceeding 2m (unit flows of about 6 m^3/s/m of unit width). For flatter graded streams of slopes of about 0.001 or less, the weir is “drowned out” at higher heads and cavitation probably will not occur. For steeply graded streams with higher unit discharges corresponding to heads exceeding 2m, the impact of cavitation effects may be minimal if these events occur infrequently.

For weirs with design heads, Hd between 2 and 3m, the width of the flat crest width (L) shall be between 0.8 and 1.2m. This will maintain the Hd/L ratio at 2.5 to result in a reasonably high coefficient of discharge with only minimal negative pressures. For weir design heads of 1.5 to 3m the Froude number at the base of the weir will vary from about 3 to 4.5, which is in the transition zone for hydraulic jumps and only a weak jump will form. For typical weirs for small scale irrigation project diversions and for heads greater than 3m, the hydraulic jump is drowning out with only surface roller phenomena occurrence.

2.3.5 Classification based on position of d/s water level

2.3.5.1 Free-Overfall (Clear-overfall) weir

Condition of flow over a diversion weir can be either free flow (modular) or submerged flow. The condition of tail water level above the crest level of the weir shall not necessarily dictate submergence of a weir. The level of the tail-water surface in relation to the upstream depth usually justify whether a weir is submerged or not. During the free overfall or modular flow conditions, the discharge over the weir is independent of the tail-water elevation and the depth at the crest of the overflow structure is taken as the critical depth.

The depth of submergence is the criteria to distinguish between free overfall and submerged weir. The depth of submergence is the difference in elevation between the downstream water surface and the crest level of the overflow structure. When the depth of submergence is 70% of the critical depth or less, the discharge is usually taken as similar to the free flow discharge (USSCS, 1973). Or when the ratios of the energy depth or head over the weir crest in the downstream, H_2 to upstream H_1 does not exceed 70% (Halcrow, 1988; Novak et al., 2007) the weir become modular.

In this case, $Q = 2/3*C_d*L*(2g)^{0.5}*H^{1.5} = 1.7LH^{3/2}$……………………………………………… (2-2)

Where, Q is discharge m^3/s;

L is effective crest length (m);

C_d is discharge coefficient;

H is flow depth over the weir crest.

Here, downstream water level /WL/ is lower than crest level, thus Q is independent of D/S WL but Q α H. It is thus known as modular.

2.3.5.2 Submerged or drowned weir

Submerged flow exists when the tail water is sufficiently above the crest of the overflow structure so that the downstream energy head affects the flow rate. When the depth of submergence is more than 70% of the critical (USSCS, 1973) or the ratios of the energy head over the weir crest in the downstream, H_2 to upstream H_1 exceeds 70% (Halcrow, 1988; Novak et al., 2007) the weir can be considered become submerged.

In order to simplify the computation of discharge in submerged condition, a correction factor for submergence is usually introduced to the earlier weir discharge equation. The discharge capacity of a free overfall weir is reduced when it is submerged. This reduction factor is called a correction factor and is expressed as

In this case, D/S WL is higher than weir crest, thus non-modular and discharge over the free portion (i.e., upper portion) is given by:

$$Q_1 = \frac{2}{3} C_d * L * \sqrt{2g} * (H_1 - H_2)^{\frac{3}{2}} \quad \text{(2-3)}$$

And discharge over the submerged (i.e., lower) portion is given by:

$$Q_2 = C_d * L * H_2 * \sqrt{2g(H_1 - H_2)} \quad \text{(2-4)}$$

Therefore, total discharge expected to pass over the weir body: $Q = Q_1 + Q_2$ ……..…….. (2-5)

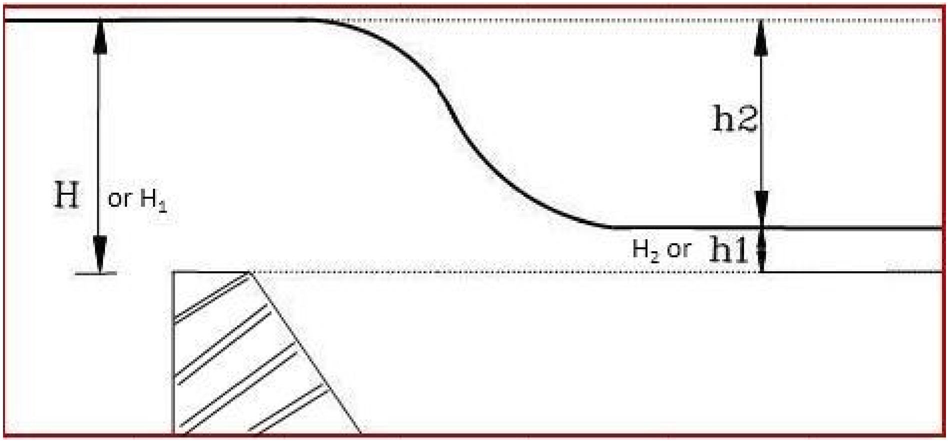


Figure 2-9: Position of water level relative to crest level (typical)

Note: In selecting any of the above weir types, its economic aspect, its stability and its practicality should be considered.

2.3.6 Classification based on stabilizing factor

Depending upon the criterion of the design of their floors, weirs can be classified in to:

2.3.6.1 Gravity weir

In gravity weir, the weight of the weir (i.e. its body and floor) balances the uplift pressure caused by the head of the seepage water below the weir. Thus, this type of weir depends on its self-weight for counteracting any unbalanced disturbing force in order to sustain stability of the structure.. Therefore, the unit weight of the construction materials and its volume is crucial for gravity weir. Such weir is used on permeable soil.

2.3.6.2 Non gravity weir

In this case, the weir floor is designed continuous with the divide piers as reinforced structure, such that the weight of concrete slab together with the weight of divide piers, keep the structure safe against the uplift. This type of weir rests on the piles (Cut-offs) and other pressure defusing mechanisms for its stability against uplift force from the subsurface flow. It needs careful design and reduces construction material cost. Stability of non-gravity weir is from its structural elements and support. The non-gravity weir is usually constructed with thin or slim surface plat or slab which is supported with pier or buttress. The structure is usually constructed in monolithic to enhanced stability strength. The non-gravity weir is usually constructed with reinforced concrete.

2.3.7 Classification based on provision of device on crest

Based on provision of control devices on weir crest, weir can be classified in to the following:

2.3.7.1 Diversion weir

This is a weir with the crest at its top level thus no extra gates are provided (as can be seen in figure below). Normally, a weir which is at its lower crest level and with some controlling devices is preferred to weirs with high crest level in the case of flatter river bed slope and wider channels. The reason is that the latter causes excessive afflux when the river is in floods. It’s another advantages is that when flood comes in, controlling devices lower the gates and makes extra space available to discharge the flood quickly.

The only difference between a weir and a barrage is of their gates that is the flow in barrage is regulated by gates and that in weirs, by its crest height. Barrages are costlier than weirs. Weirs and barrages are constructed mostly in plain areas. The heading up of water is thus affected by gates put across the river in barrages.

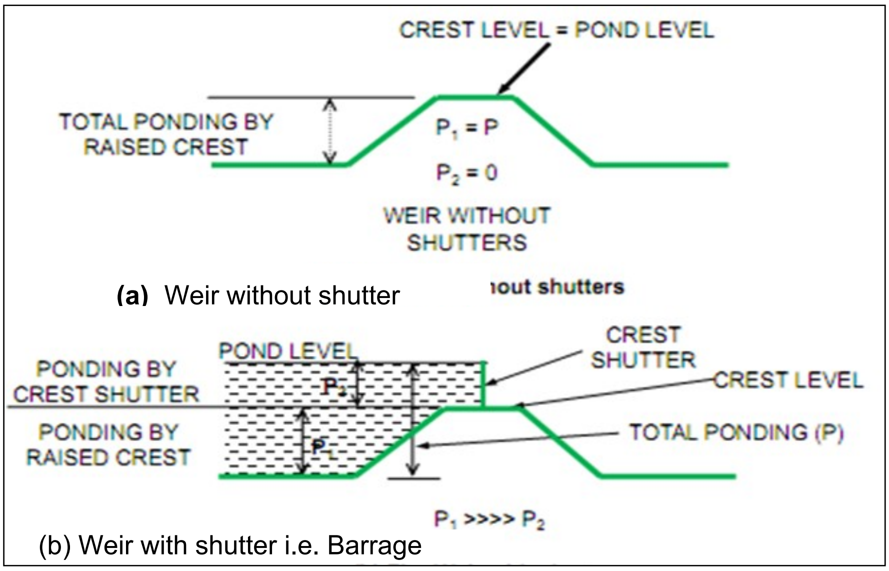


Figure 2-10: Schematic view of cross section through weir (a) and barrage (b)

2.3.7.2 Barrages

The function of a barrage is similar to that of weir, but the heading up of water is effected by the gates. Thus, in this case, gates are provided on the crest of the barrier. The gates are housed in the grooves made in the piers and abutments. The piers are constructed on the crest. They can also be used for supporting road and support the platform used for lifting and lowering of gates. Thus the head is controlled by the weir height and gates. The crest level in the barrage is kept at a low level, but during the floods, the gates are raised to clear off the high flood level, enabling the high flood to pass downstream with maximum afflux. When the flood recedes, the gates are lowered and the flow is obstructed, thus raising the water level to the upstream of the barrage for enabling water to divert in to conveyance canal. Due to this, there is less silting and better control over the levels.

Figure 2-11: Partial view of barrage weir on bilate river

Barrages are preferred over conventional broad crested diversion weirs when:

- The channel is very wide;
- When the bed material at the selected site is of alluvial in nature thus require deep Cut-off to support the large weir body;

- We are interested in partly to accommodate the floodplain discharge and partly to take advantage of the dispersion of the channel flow induced by the obstruction caused by the barrage itself;
- We need to control supply discharge based on demand at one center;
- We are interested in to control easily the rise in maximum flood level of the river upstream of the barrier structure;
- If the difference between the pond level and crest level is within 1.5m

2.4 CAUSES OF FAILURE OF WEIR/BARRAGE

2.4.1 General

Failure of hydraulic structures like a weir or a barrage is the combined effect of subsurface flow and surface flow at the site in addition to lack of quality of construction. Such causes include piping, uplift force, suction caused by standing wave and scouring on both upstream and downstream of the structures.

When hydraulic gradient or exit gradient exceeds the critical value of soil, surface soil at d/s end starts boiling first and is washed away by percolating water. This process of removal or washing out of soil continuous and eventually a channel in the form of pipe is formed by seepage water. This is called piping which may cause the failure of foundation. Similarly uplift force of percolating water is acting on the floor from bottom and if the weight of floor is not enough to resist this uplift force, the floor subjected to such condition may fail by cracking or bursting.

2.4.2 Failure due to subsurface flow

There are two fundamental causes for such failure: piping or undermining and Uplift Pressure.

2.4.2.1 Piping or undermining

This occurs when water from the upstream side continuously percolates through the bottom of the foundation and emerges at the downstream end of the weir or barrage floor. The force of this percolating water removes the soil particles by scouring at the point of emergence. As the process of removal of these soil particles goes on continuously, a depression is formed progressively which extends backwards towards the upstream through the bottom of the foundation. This process of erosion thus progressively works backwards towards the upstream and results in the formation of a channel or a pipe underneath the floor of the weir, causing its failure and a hollow like pipe formation develops under the foundation due to which the weir or barrage may fail by subsiding. This phenomenon is known as failure by piping or undermining,

2.4.2.2 Uplift pressure

This phenomenon occurs when the percolating water exerts an upward pressure on the foundation of the weir or barrage. If this uplift is not counterbalanced by the self-weight of the structure, it may fail by rapture. Its distribution is high at the upstream and minimized while moving to the downstream.

Figure 2-12: Collapsed weir as a result of piping (L) & scouring & absence of wings (R)

2.4.3 Failure by surface flow

2.4.3.1 By hydraulic jump

When the water flows with a very high velocity over the crust of the weir or over the gates of the barrage, then hydraulic jump develops. This hydraulic jump causes a suction pressure or negative pressure on the downstream side which acts in the direction of uplift pressure. If the thickness of the impervious floor is not sufficient, then the structure fails by rapture.

2.4.3.2 By scouring during floods

The gates of the barrage are kept open and the water flows with high velocity. The water may also flow with very high velocity over the crest of the weir. Both cases can result in scouring effect on the downstream and on the upstream side of the structure. Due to scouring effect on the downstream and on the upstream side of the structure, its stability gets endangered by shearing.

2.4.4 Failure due to silt (aggradations and degradation or retrogression)

Constructing a weir across the river causes progressive retrogression on the downstream part and aggradations in the upstream part of the structure. The upstream aggradations have the tendency of increasing the approach velocity in the upstream side of the weir since the initial flow area computed for the approach velocity in the upstream side encloses between the U/S high flood level and pond level.

The downstream retrogression on the other hand, causes lowering of the downstream river stages which thus need to be considered during the design. Lowering of the river water level due to retrogression on the downstream causes increased exit gradients and endanger the safety of the structure. So, as a rule of thumb, a retrogression of 0.3-0.5m may be assumed for calculating the design floor and exit gradient. However, if the downstream river course is strong rock, retrogression will not be assumed.

Thus, silt deposition could hamper soundness of operation of the diversion headwork by clogging of gates (under sluice gate for weir and gate for barrage) and other water ways there by reducing its efficiency or leading to its collapse. Thus, following mechanisms are recommended:

- Introduce silt- controlling mechanism, (establish standard gate operation norm);

- Opening weir or barrage is recommended on Rivers with high sediment concentration usually those with poor catchments on upper reach of a river and seasonal rivers,
- Depending on the magnitude and type of sediment, an excluder in the River or extractor along the canal could be constructed,
- Usually on the lower reach, suspended silt is carried by the flood thus, an extractor in the canal would be appropriate. The decision is dictated by the pertinent condition of the silt and it is the designer who decide on either mechanism.

2.4.5 Failure due to seismic load

Failure due to earth quake is likely in seismic zones, especially on any elevated structure it is considerable. Therefore, an appropriate ground acceleration coefficient should be adopted in the design activity depending on the delineated seismic zones of our country. Seismic Risk Map showing 1:50 earthquake acceleration has been attached as appendix for this purpose (Appendix VI). In case of difficulty to get reasonable coefficient for site specific location it is advantageous to be based on local and international practice by consulting appropriate institution or organization.

2.4.6 Failure due to man-made activities

Man-made activities can also be a potential cause for the failure of a diversion headwork structure. Such activities like use of poor construction activity and construction material for the purpose of fraud and absence of skilled construction labor including intentional interference like looting of steel, cement and other materials and deliberate attack on the structure due to unhealthy attitude towards it can also cause danger to the structure. This requires strict follow-up of the construction activities and awareness creation for not only the project beneficiaries but also for the communities around the project area.

2.4.7 Remedies for failure of diversion headwork structure

Remedies to avoid failure due to piping and uplift pressure is to decrease the hydraulic gradient: this can be made by increasing the path of percolating or seeping flow through:

- Increasing path of percolation or creep length of seepage water either by providing sheet piles at upstream, downstream or at intermediate point to reduce the hydraulic gradient or increase length of the impervious layer itself;
- Increasing floor thickness to increase its self-weight to counterbalance the uplift force;
- Provision of energy dissipater blocks like friction blocks, impact blocks, i.e. select appropriate type of stilling basin;
- Provision of inverted filter with concrete blocks on the top so that the percolating water does not wash out the soil particles;

Remedies to avoid failure due to scour is

- to provide pile or curtain at greater depth in excess of the scour level
- to provide launching apron with sufficient length

Remedies to avoid failure due to man-made problem is

- to give sufficient time for study and investigation (hydrological study, geotechnical investigation) to properly understand the hydrologic and geological condition of the project
- to augment the analytical design with model simulation in order to increase the confidence of design product
- to enforce sufficient construction supervision team for monitoring the construction process (records of various tests should be retained)

- scaling down the maintenance norm to the ground and prepare feasible action plan (with accountability) for inspection and maintenance of structure

2.5 SELECTION OF SUITABLE WEIR/BARRAGE TYPE

2.5.1 Criteria to be considered in selecting weir type

Diversion weir structures differ in type and shape, however they all are designed and constructed to serve the same purposes i.e. diversion of water. Consequently, the following points shall be considered to select type of a weir that suits to a specific site:

- Suitability of the local construction materials;
- The hydrological characteristics of the river at that particular site;
- A weir with a shape that can easily be constructed by local manpower;
- The availability of skilled manpower for implementing it;
- The skill of the local masons to perform it as per design and specification;

The cross sectional shape of a weir has to be decided based on a combination of hydraulic and structural requirements conditions, as well as some consideration as to the nature of the weir. The four commonly adopted shapes are:

- An ogee crested weir (as it can bypass more Qd than other for the same crest length);
- A vertical drop weir;
- A steep glacis weir;
- A shallow glacis weir.

The often claimed advantage of ogee shaped weir is that it has a higher coefficient of discharge than other weir shapes and that the coefficient is more predictable than for other weirs. The main disadvantage of the curved shape is the difficulty in constructing it accurately and soundly.

The vertical drop weir appears, at first glance, both simple to construct and easy and economical to design. In truth, if all structural checks are undertaken and adequate factors of safety are applied at maximum levels of flow, it will be found that for all weirs apart from those with very small flows per unit width, the wall section becomes very thick to resist overturning and shear forces. The glacis weir is an improvement on the vertical drop weir to enhance its structural stability.

2.5.2 Selection of type of diversion weir

Type of diversion weirs need to be selected based on nature of flow in the river, cost of the structure, foundation condition at the selected site and function of the structure itself. The following table shows a glance at ways to select type of diversion weir.

Table 2-1: Recommended type of weir based on nature of river

SN	Site condition	Diversion Recommended			
		Function	Crest	Design	Construction
1	Perennial river less significant bed load and moderate up to good foundation bearing capacity	Storage/ diversion	Weir	Gravity	Masonry /cyclopean /concrete/rockfill with masonry/
2	Perennial or intermittent river highly significant bed load and moderate up to good foundation bearing capacity	Storage/ diversion	Barrage/ open slotted weir	Gravity	Masonry /cyclopean /concrete/
3	Same as above but bad foundation bearing capacity	Storage/ diversion	Barrage/ open slotted weir	Non-Gravity	Cyclopean concrete/ reinforced concrete/
4	River water released from storage dam and moderate up to good	Pick up	Weir	Gravity/Non-Gravity	Masonry /cyclopean concrete /rock fill with

SN	Site condition	Diversion Recommended			
		Function	Crest	Design	Construction
	foundation bearing capacity				masonry/
5	Same as above but bad foundation bearing capacity	Pick up	Weir	Gravity/Non-Gravity	Cyclopean concrete/ reinforced concrete/

Source: As adopted from MoWR, PART I-G Diversions, 2002

Table 2-2: Summary to select type of diversion weirs

Criteria\Weir Type	Broad Crested	Ogee	Barrage	Glacis
Nature of the river and flow condition	Preferred for narrow river section and When flow in the river is not dangerous and carries small boulders	Preferred for narrow river section and when flow in the river carries large flow and boulders	Preferred for the case of diversion from wide channel with large flow andwhen expected backwater effect is high	Preferred for the case of diversion from wide channel with large flow
Construction material availability	When quality stone is available in the nearby area of the project	When there is not enough quality stone is available in the site	When there is not enough quality stone is available in the site	When there is not enough quality stone is available in the site
Easiness of construction	This is the simplest and preferred where there is no skilled manpower is limited	This is a bit harder and preferred where there is enough skilled manpower	Needs huge number of skilled manpower	Needs huge number of skilled manpower
Foundation condition	Preferred when foundation condition is not complicated	Needs stronger foundation condition	Preferred in case of alluvial channel	Preferred when the channel bed is alluvial
Hydraulic performance	This has the lowest performance	The best performance	Better performance	Better performance
Investment Cost	The cheapest	Relatively expensive	Very expensive	Very expensive

Source: As compiled from own experience

2.6 TYPES OF STILLING BASIN

2.6.1 General

The stilling basin also called downstream impervious apron is an integral part of the diversion headwork structure to its downstream side. It is used for the purpose of resisting uplift pressures exerted from the bottom of the structure and to dissipate the incoming energy from over the diversion headwork. There are different types of stilling basins, but the most commonly used types are USBR Standard and Indian Standards. The USBR standard recommend lengths and depths of basin, based on calculated specific energy, velocity and Froude number.

2.6.2 USBR standard stilling basin

Types of stilling basins as per Design of Small Dam, USBR, 1987, are divided in to the following.

2.6.2.1 Stilling basin type-I

For Froude numbers ranging between 1.0 to about 1.7, the incoming flow is only slightly below critical depth, and the change from this low stage to the high stage flow is gradual and manifests itself only by a slightly ruffled water surface. As the Froude number approaches 1.7, a series of small rollers begins to develop on the surface, which intensifies as the value increases. Therefore, no special stilling basin as well as baffle or other jump enforcing structure is needed to stabilize the flow. However, the length of the stable channel (or simply basin) from the location where depth starts to change up to its end should not be less than 4 times the conjugate depth (USBR, 1987).

Relatively smooth flows prevail throughout the Froude number ranging between 1.7 and 2.5. The phenomena under this range designated as the pre-jump stage because flows are not attended by active turbulence. Therefore, baffles or sills are not required. This type of basin is used where hydraulic jump occurs on a horizontal apron thus energy dissipation is very low. Incoming velocity is less than 15m/s, thus it is the weak jump zone. Such flows are not appeared by active turbulence. Thus, in such case, the basin is plain horizontal and jump occurs on the floor with no chute blocks, baffle piers or end sill provided thus simple for construction. Usually this is not recommended because of need for excessive length, but discussed here since it provides a grounding in the basic hydraulics of all stilling basins.

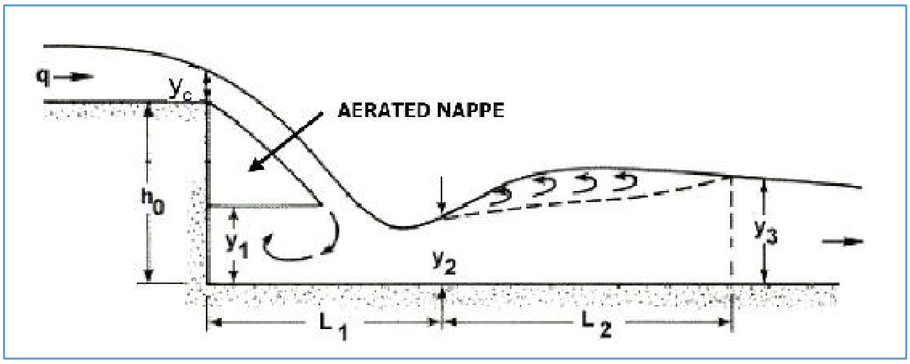


Figure 2-13: USBR Stilling Basin, Type-I
Source: S.K. Garg, 2006

2.6.2.2 Stilling basin of type-II

This type of basin is used for spillways of high concrete dam and embankment dams. Its incoming velocity exceeds 15m/s and Froude number, F_r varies from 4.5 to 9.0. It is a range of well-balanced jump and its performance is at its best and least affected by tail water variations. In this case length of the basin can be shortened by introducing devices like baffles and chute blocks. Such jump is called a steady jump.

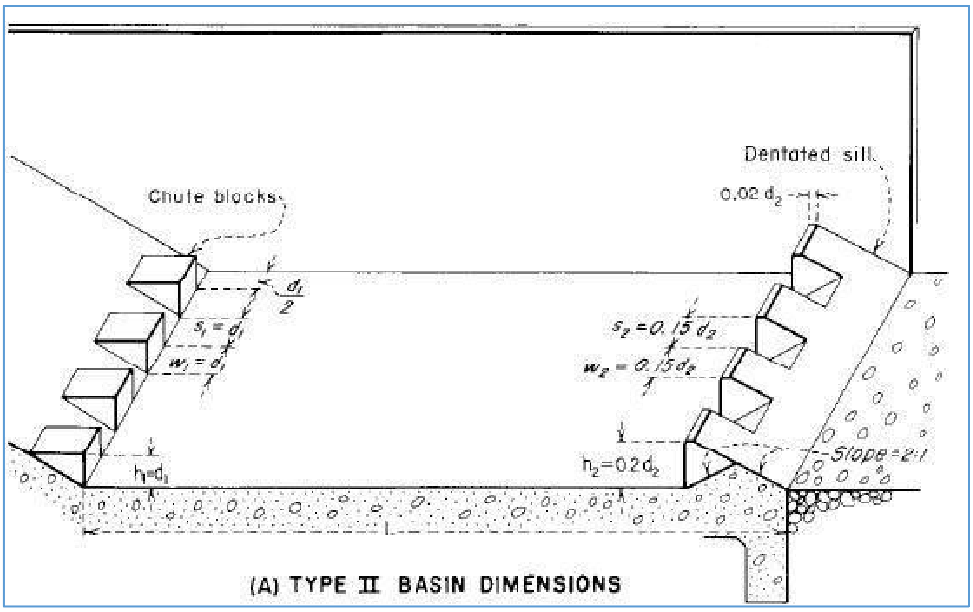


Figure 2-14: USBR Stilling Basin, Type-II
Source: S.K. Garg, 2006

2.6.2.3 Stilling basin of type-III

This can be more economical than basin II. Here a row of baffle piers is placed downstream from the chute blocks. This basin relies on dissipation of energy by the impact blocks and also on the turbulence of the jump phenomena for its effectiveness.

For Froude numbers range between 4.5 and 9, a stable and well-balanced jump occurs. Turbulence is confined to the main body of the jump, and the water surface downstream is comparatively smooth. Basin designated as USBR type III basin, can be adopted for this case if incoming velocities do not exceed 18.2 m/s and the specific discharge do not exceed 18.6 m3/s/m (USBR, 1987; Novak et al, 2007). The basin uses chute blocks, impact baffle blocks, and an end sill to shorten the jump length and to dissipate the high-velocity flow within the short basin length. This basin relies on dissipation of energy by the impact blocks and on the turbulence of the jump phenomena for its effectiveness.

The large impact forces to which the baffles are subjected to by the impingement of high incoming velocities creates the possibility of cavitation along the surfaces of the blocks and floor. There is a temptation to use this type of stilling basin outside of the velocity and specific discharge range because of reduced length. However, the danger of cavitations damage in these cases is substantial and great care must be exercised in the design and positioning of the blocks (Novak et al., 2007). The USBR type III basin is similar to the Saint Anthony Fall (SAF) basin but SAF is provided with a larger safety factor (French, 1985).

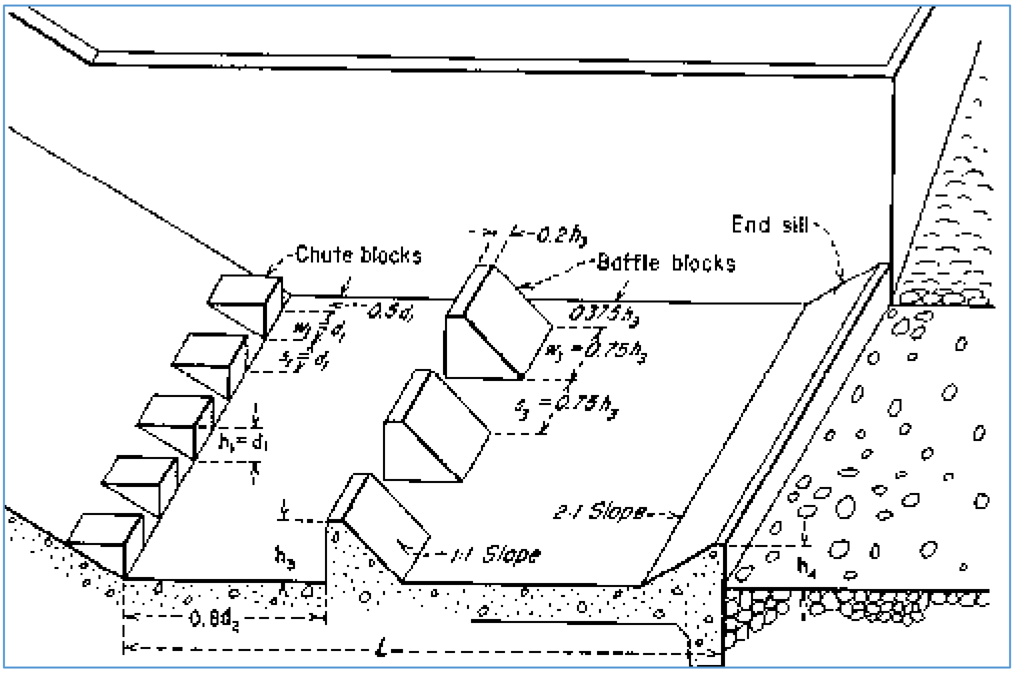


Figure 2-15: View of USBR stilling basin, type-III
Source: S.K. Garg, 2006

2.6.2.4 Stilling basin of type-IV

This type of basin is used for stilling basin design and wave suppressors for canal structures and low head diversion weirs. Its Fr varies from 2.5 to 4.5 and is in the transition zone with incoming velocity less than 15m/s. It has rough water surface with roller and oscillating jet. The jump is not stable and waves are thus generated. This basin is a short basin, but complicated by floor and chute blocks.

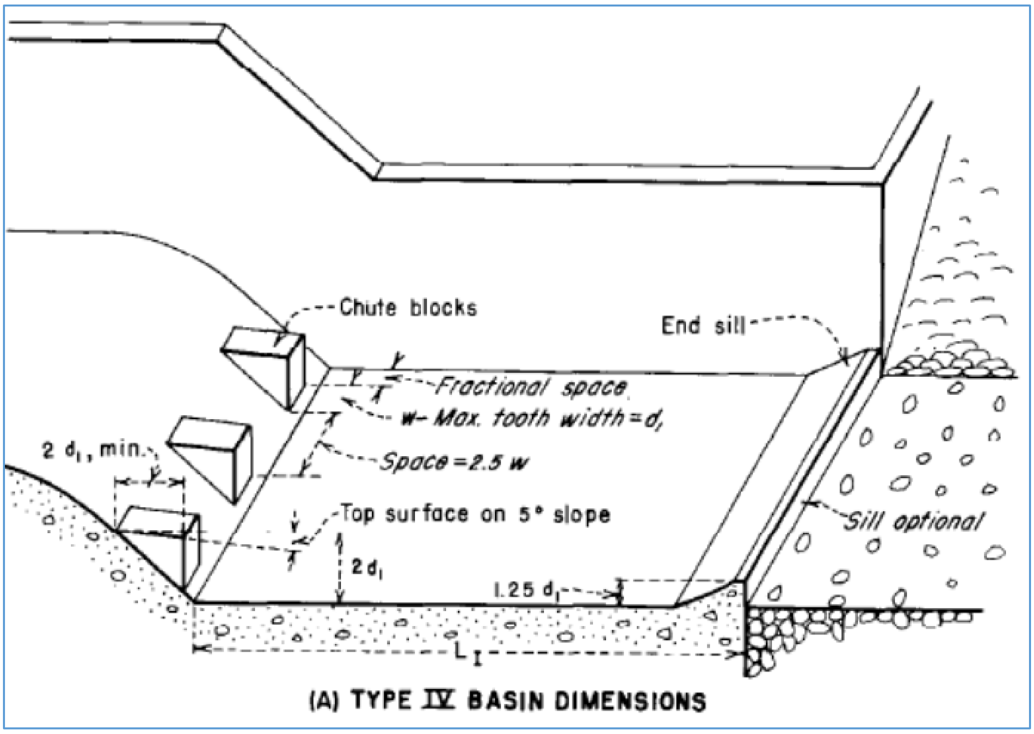


Figure 2-16: USBR stilling basin, type-IV
Source: S.K. Garg, 2006

2.6.3 Indian standard stilling basin

Indian Standard Institution has also standardized certain stilling basins for uses under different conditions in IS: 4997 - 1968. Stilling basin I and II (for Fr<4.5 and Fr> 4.5 respectively) with horizontal aprons and stilling basins III and IV with sloping .aprons are described in detail in standard. But the most commonly adopted basins are the above described USBR ones, if anyone is interested in IS, it can be referred in the mentioned standard.

2.6.4 Appurtenant structures in stilling basin

2.6.4.1 Chute blocks

Chute blocks are a kind of ragged wedges (i.e. row of small projections like teeth of saw) and provided at the entrance of the stilling basin i.e. foot of d/s sloping face. The incoming jet of water is creased and partly lifted from the floor, producing a shorter length of jump than what would have been without them. They also help in stabilizing the flow and thus improve the jump performance. The dimension of the chute block and its spacing depends on the basin type as per the recommendation.

Chute blocks: Height = d_1 or $d_2/9$, whichever is greater; and,
Width = spacing = 0.75 to 1.0 times height.

2.6.4.2 End sills and dentated siils

Sill or end-sill or more preferably dentated sill is generally provided at the end of the stilling basin. The dentated sill diffuses the residual portion of high velocity jet reaching the end of the basin. They, therefore, help in dissipating residual energy and to reduce the length of the jump or the basin. The dimension of the end-sill and its spacing depends on the basin type as per the recommendation.

Baffle/Basin blocks: Height = d_1 or $d_2/8$, whichever is greater;
Width and spacing are same as for the chute blocks, but staggered;
Location = $0.4*L_b$ from start of basin.
End Sill: Height = $d_2/10$.

Note: These dimensions are indicative only, as it is based on the type of the selected stilling basin.

2.6.4.3 Baffle block or friction piers

Baffle Block or Friction Piers are the blocks placed within the basin, i.e. across the basin floor. They help in breaking the flow and dissipate energy mostly by impact. These baffle piers, sometimes called friction blocks are very useful in small structures, such as low spillways and weirs, etc. They, however, give way due to cavitation, under the influence of high velocity jets, and hence are unsuitable for large works.

2.7 RETAINING WALLS AND THEIR TYPES

2.7.1 General

Retaining wall is any structure that holds or retains soil, rock or other materials behind it. Such walls are structures designed to restrain soil to unnatural slopes, thus provide a lateral support to vertical slopes of soil that would otherwise collapse into a more natural shape. In general they are used to bound soils between two different elevations often in areas of terrain possessing undesirable slopes or in areas where the landscape needs to be shaped severely and engineered for more specific purposes like hillside farming or roadway overpasses.

There are many types of materials that can be used to create retaining walls like concrete blocks, poured concrete, masonry, gabions, treated timbers, rocks, or boulders. Some are easy to use, others have a shorter life span, but all can retain soil.

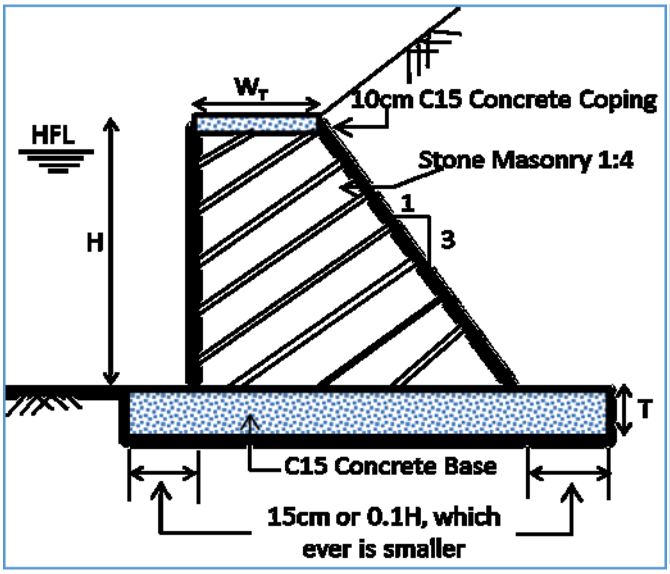
Wing walls are short retaining structures and are used as a retaining wall and to stabilize the abutment. They are short in section of walls used to guide a stream into an opening, such as at a culvert or bridge inlet and/or outlet or diversion weir. A wing wall also named as "wingwall" or "wing-wall" is a smaller wall attached or next to a larger wall or structure. In a bridge, the wing walls are adjacent to the abutments and act as retaining walls. They are generally constructed of the same material as those of abutments. The wing walls can either be attached to the abutment or be independent of it.

There are four common types of Retaining walls as described in the following sections:

2.7.2 Gravity retaining wall

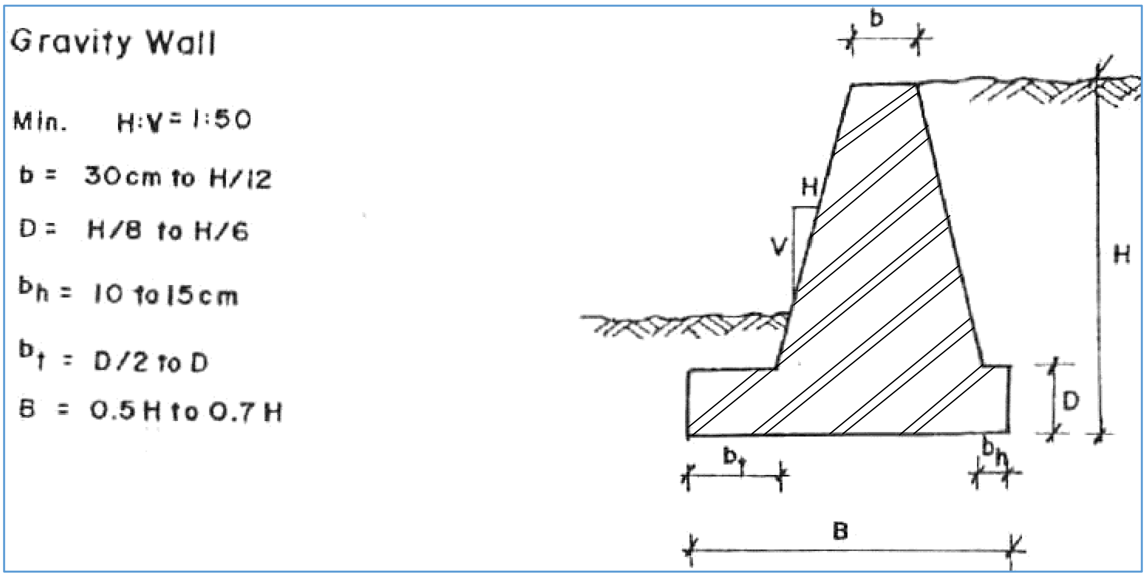
It is that type of retaining wall that relies on their huge weight to retain the material behind it and achieve stability against failures. Gravity Retaining Wall can be constructed from concrete, stone or even brick masonry. Gravity retaining walls are much thicker in section. Geometry of these walls also help them to maintain the stability by their weight. Mass concrete walls are suitable for retained wall heights of up to 3 m. For wall over 4.5m high the reinforced concrete cantilever wall offers a substantial cost advantage over gravity masonry wall. The cross section shape of the wall is affected by stability, the use of space in front of the wall, the required wall appearance and the method of construction.

Gravity retaining walls rely on the mass of the wall structure for their stability. Thus, the wall mass must be sufficient to counteract sliding and overturning forces from the retained soil. These systems can use masonry, stone, concrete or other heavy material as well as mechanically stabilized earth for stability. These are the most common type of retaining walls and include gabions, bin walls, and modular block concrete construction. In most cases the slope behind the wall needs to be temporarily removed during construction. For mechanically stabilized earth walls, the reinforcing often extends horizontally into the embankment about as far as the exposed wall face is tall.



Nr	H(m)	WT(m)	T(m)
1	H ≤ 0.6	0.30	0.15
2	0.6 ≤ H < 1.6	0.30	0.15
3	1.6 ≤ H < 2.2	0.40	0.20
4	2.2 ≤ H < 2.7	0.50	0.30
5	2.7 ≤ H < 3.0	0.60	0.30
6	3.0 ≤ H < 3.4	0.70	0.40
7	3.4 ≤ H < 3.8	0.80	0.40
8	3.8 ≤ H < 4.1	0.90	0.40
9	4.1 ≤ H < 4.5	1.00	0.40
10	4.5 ≤ H < 4.9	1.10	0.40
11	4.9 ≤ H < 5.3	1.20	0.50
12	5.3 ≤ H < 5.7	1.30	0.50
13	5.7 ≤ H < 6.0	1.40	0.50

Figure 2-17: Typical gravity masonry retaining wall (Halcrow-ULG, 1988)



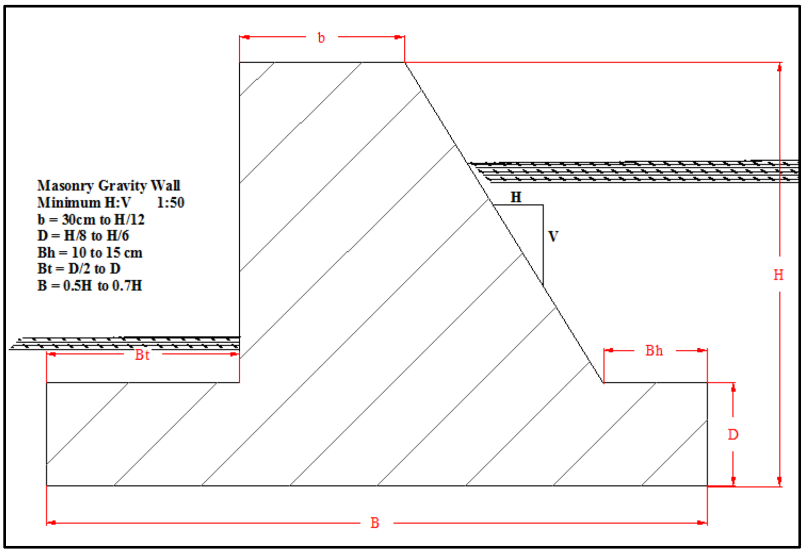


Figure 2-18: Alternative gravity masonry retaining wall arrangement

2.7.3 Cantilever wall

A cantilever retaining wall is one that consists of a wall which is connected to foundation. A cantilever wall carries a significant amount of soil, so it must be well engineered. Thus, cantilever walls are built of reinforced concrete and are typically composed of a horizontal footing (slab foundation) and a vertical stem wall. They are the most common type used as a cantilever wall rest on a slab foundation. This slab foundation is also loaded by back-fill and thus the back-fill and surcharge also stabilizes the wall against overturning and sliding. The weight of the soil mass above the heel helps keep wall stable. Cantilever walls are economical for heights up to 10 m.

Cantilever retaining walls thus have a large effective mass due to the soil placed over a horizontal section of the wall. These walls are typically constructed of cast-in-place, reinforced concrete. The horizontal (cantilevered) leg of the structure can either extend back into the retained soil or out away from the slope. The slope behind the wall typically needs to be temporarily removed during construction.

Cantilever walls are relatively expensive due to the work required to build concrete forms, install reinforcing, pour concrete, and provide joints between pours. The concrete needs ample time to cure before the soil can be replaced behind the wall.

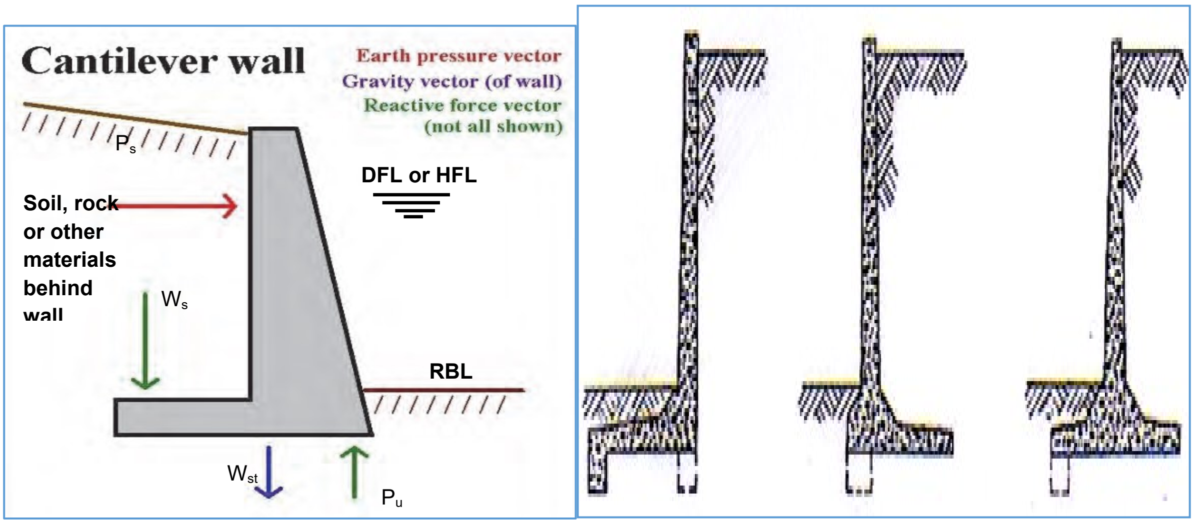


Figure 2-19: Cross sections of typical cantilever retaining walls

2.7.4 Counterfort wall

Counterfort or Counter-fort retaining walls are also known as buttressed retaining walls. Counterfort walls are cantilever walls strengthened with counter forts monolithic with the back of the base slab and base slab. The counter-forts act as tension stiffeners and connect the wall slab and the base to reduce bending and shearing stresses. To reduce the bending moments in vertical walls of great height, counterforts are used, spaced at distances from each other equal to or slightly larger than one-half of the height. Counter forts can be used for high walls with heights greater than 8 to 12 m.

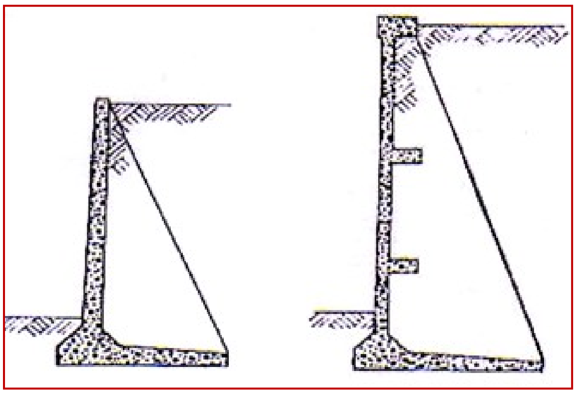


Figure 2-20: Cross sections of typical retaining walls with counterforts

Note: Cantilever and Counter fort retaining walls are families of Reinforced Retaining Walls.

2.7.5 Hybrid system of retaining wall

The type of retaining walls that use both factors that is their mass and reinforcement for stability are called hybrid systems or composite retaining wall systems.

2.8 SELECTION OF RETAINING WALLS

2.8.1 General

Below are important considerations that affect the decision to provide a retaining wall and which type of wall should be constructed. The decision to construct a retaining wall can be subjective and must balance the cost of installing a retaining wall with the overall impacts to utilities, wetlands, the environment, or adjacent properties. Impacts to wetlands often demand construction of retaining walls due to permitting requirements to avoid or minimize wetlands impacts if possible. Exact locations of retaining walls will require further refinement during the design phase.

2.8.2 Embankment impacts

Gravity and cantilever retaining walls typically require some of the soil behind the wall to be temporarily removed during construction. In some cases the slope can be cut to stand near vertical for short periods of time to reduce impacts but OSHA embankment safety guidelines for worker safety must be accounted for. Piling walls can minimize impacts to adjacent properties and structures.

2.8.3 Foundation soils

Gravity and cantilever retaining walls require a solid foundation to resist the forces of the wall and soil. Where foundation soils are weak, a piling or anchor wall should be considered or the weak soil replaced. Existence of bedrock encountered near the south end of the Golden View Drive project provides an excellent foundation but makes driving sheet piles very difficult.

2.8.4 Groundwater/drainage

Groundwater needs to be removed from behind the retaining wall to reduce hydrostatic forces. Many types of wall are inherently porous while other types, like reinforced concrete, require weep holes to be integrated into the design to relieve pressure from water behind the wall. In areas where substantial groundwater and glaciation is expected, a sub-drain should be considered to direct the runoff to drainage ditches or a piped storm drain collection system.

2.8.5 Utilities

Some types of retaining walls impact a considerable amount of soil behind the face of the wall. The fabric behind the wall can impact existing utilities or limit future placement of utilities. Anchor wall cables also extend into surrounding soils and affect utilities. Therefore, proper consideration should be made for utilities, if any.

2.8.6 Aesthetics

Retailing walls are often very noticeable along a roadway corridor and they should be selected to compliment the landscaping design and blend in with the surrounding zone. Sometimes retaining walls are chosen to match existing walls on adjacent properties. However, such consideration shall be taken in to consideration in urban areas.

2.8.7 Construction schedule

In some cases, the amount of time required to construct a retaining wall is very important since it can affect impacts to adjacent property owners or the environment. Reinforced concrete headwalls take a considerable amount of time to construct. They are typically not a good option for stream culvert headwalls that need to be completed quickly to reduce environmental impacts.

2.8.8 Maintenance

Retaining walls and associated fences or handrails should be designed to require little if any maintenance. Concrete surfaces can be provided with coatings to facilitate removal of spray paint vandalism.

2.8.9 Cost

The estimated installed cost for retaining walls varies considerably from nature of surrounding topography and height of the exposed vertical face and its stability. Some of the biggest factors include soil conditions, wall height, tiebacks, construction access, type of fence and the amount of soil to be removed behind the wall for construction.

As a generally guide, a stone masonry is commonly used for gravity retaining walls around diversion structures for a height not in excess of 4.5m high. However, concrete with nominal temperature and shrinkage reinforcing could be used. The base is typically constructed of reinforced concrete. The factor of safety for structure stability (the ratio between stabilizing to destabilizing forces) should be at least 1.5 to ensure long term sustainability. For gravity retaining walls constructed on sound bedrock and adequately interlocked to the bedrock, stability should not be a problem.

3 DESIGN OF DIVERSION WEIR

3.1 DESIGN CONSIDERATIONS OF DIVERSION WEIRS

Before deciding to build a weir, it is worth considering if it is really necessary at that location. Moreover, to design a stable and cost effective headwork structure with appropriate geometry, a surface and subsurface flow analysis shall be done as flow condition is determined by the geometry of a structure and in turn the geometry of such hydraulic structure affects its design and economics. Therefore optimization between the economics and the design of a stable structure is expected from the design engineer. The following successive sub-sections present design considerations and procedural approach to design of diversion weir and related appurtenant structures in this accord.

Setting design criteria and considerations ahead of design of any structure is imperative as it can help designers to know their scope of design and direction. In view of that, diversion weirs need to be designed from both hydraulic as well as structural design requirements points of view.To do this, basic data as stated under, are required for design of such diversion weir that need to be gathered before jumping to the next stage i.e. the design processes.

Weirs thus, should be designed to withstand recommended large flood events and last a reasonably long time. Hence expensive construction materials and hence robust designs are necessary. If the site is suited to a Free Intake then the high cost of such structure can be avoided. In both cases the command should be lower than intake level/the head regulators so as to irrigate the land without the need of pumping system. Pumping option shall be considered only where the level of intake is lower than the irrigable command level.

The followings are also areas that need to be considered while designing diversion weir.

- Weir axis should be laid at right angles to the river flow so as to take advantages of economy and for ensuring balanced flushing function of the scouring sluice along its section.
- Design levels of intake must be secured(from topographic and irrigation requirement),
- The site shall be such that it can give economic structure and ensure proper intake function,
- Weir height has to be designed to match the design water level in the conveyance canal.
- Weir length has to be designed to allow design flood to safely discharge over the weir.
- Design discharge of intake should be estimated or obtained from agronomy report plus some 10% for future need/expansion requirement (as flexibility factor),
- Determination of design discharge at the selected headwork site,
- Levels of flood flow under normal condition at the selected headwork site
- The height of the diversion weir structure should be fixed such that it is capable of supplying irrigation water to intake at peak demand;
- Sill of under-sluice pocket shall be kept at or slightly above deepest river bed and about 0.5 to 1.0 m below sill level of canal head regulator.
- Divide walls are to be designed as cantilever retaining walls subjected to silt pressure and water pressure from under-sluice side; and,
- Future level at which riverbed is expected to be stabilized must be studied and fixed in order to set appropriate design conditions;
- Intake takes-off alignment should provide smooth entrance of diverted irrigation water. According to several literatures an intake offtaking at an angle of 90° to the main river flow direction is the ***least desirable*** one from hydrodynamic performance point of view.

Despite the controversies among scholars the recommended range is to have intake aligned at an angle between 30°and 90° (Novak et al., 2007; Smith, 1995).
- Safety structure (like rejection spillway or sluice) and sediment control arrangement (like settling basin) shall be provided if deemed necessary,

3.2 SELECTION OF APPROPRIATE DIVERSION WEIR SITE

The design engineer in collaboration with engineering geologist need to identify possible options of diversion sites during the preliminary site identification stage. The most appropriate one can be selected after comparing all options with multi-criteria analysis (topography, command area location and potential, geological condition, environmental, social, cost, etc.). The selected site should provide better comparative advantage.

Headwork site selection has direct implication on cost and stability of the structure. Therefore, due attention has to be paid in selecting the best option among potential headwork sites.

The following points should be considered while selecting appropriate and ideal site for diversion headwork.

- It shall be located in the narrowest valley
- It shall be located in straight reach of the river i.e. not meandering channel at the site. However, in sediment laden river meandering might be considered as a potential site.
- It shall enable irrigation of maximum potential command area on both left and right sides with short idle canal length;
- It shall be such that elevation of the site is at higher level than the area to be irrigated for gravity flow.
- It shall have impervious and strong foundation condition, if not it requires especial foundation treatment;
- It shall have strong, well defined, stable and water tight abutments/banks;
- It shall irrigate the command area with reasonable or minimum structure height;
- It shall be selected in such a way that there is enough flow without adverse effect for downstream users. Site located downstream of confluence of rivers is the best;
- It shall be selected so that maximum water can be harvested, say d/s of confluence of rivers;
- It shall be possible to align offtaking canal in such a way that command area is obtained without excessive digging;
- It shall be environmentally friendly, socially and culturally acceptable, and economically feasible;
- It shall be situated in the nearby command area (to reduce cost of idle MC)
- It shall be such that valuable land upstream of the headwork structure like weir or barrage should not be submerged.
- It shall be at a location such that river width should be wide enough to accommodate expected flood but narrow enough to avoid construction of long crested weir and ovoid sluggish flow condition causing silt deposition, which could hinder the structure performance leading to high maintenance cost.
- It shall be selected at a site where the thalweg is stable and located near the river bank where it is easy to install the intake in the scheme and enable to set the scouring sluices in the thalweg.
- It shall be accessible;
- Thus an ideal diversion weir site would be one where the river is stable, not meandering and neither degrading or aggrading and capable of supplying maximum area with minimum weir height;

3.3 DATA REQUIRED FOR WEIR AND BARRAGE DESIGN

3.3.1 Physiographic data

Physiographic and river morphology /bed and banks/ data which are required for design of a diversion weir are data of the river from which irrigation water is expected to be supplied, such as:

3.3.1.1 Length/width across the river data

The shorter the crest length the more economical it is; but it results in higher flow depth over the crest of the weir thus thicker energy dissipater and extended stilling basin are required. On the other hand, the longer crest length results in lower/shallow flow depth over the crest of the weir thus thinner energy dissipater and shorter stilling basin are required. Thus maintaining existing natural channel width is preferred so long as it accommodates the incoming flood and requires minimal training of the channel.

3.3.1.2 Longitudinal profile around the weir site data

Such profile is required to estimate average slope of the river along its thalweg around weir site and hence flow of the river using Manning's equation or Velocity-Area method.

3.3.1.3 River banks data

Data is required related to river bank height and stability condition. Bank height is prerequisite to evaluate its susceptibility flood and hence overtopping; whereas, its stability condition is used to identify type of wing wall the need to be embedded in the abutments.

3.3.1.4 River flow regime data

This is concerned with the river geometry and nature of flow in river reaches such as discharge condition, relation of water level and discharge and sedimentation conditions. A channel is said to be in regime when, over a hydrological cycle, the channel shows no appreciable change in its width, depth or gradient. Regime theory postulates that for a stable channel there is a relationship between the channel parameters of width, depth, gradient and flow. Thus if any one of these four parameters is artificially (or naturally) changed, the channel will adjust itself so that regime conditions are re-established.

3.3.1.5 Water level or stage – discharge relationship data

It is also called rating curve or discharge curve. This curve is used to estimate different discharges of a stream for specific water levels of that stream at that particular cross section. Thus, it is important to estimate tail water depth for the expected design flood of certain return period.

3.3.1.6 River shape (meanders) data

River shape shall be straight at the diversion site so as to minimize unbalanced force distribution on the structure. However, from the point of view of sediment exclusion, an intake can be aligned in the outer (concave) bank of meandering river preferably located towards the downstream end of the bend. This is because bottom layers of the flow (i.e. sediment laden flow) around a bend are swept towards its inside (convex) bank. Therefore, river meander data should be provided.

3.3.1.7 Appropriate coffer dam site

Convenient site for temporary river diversion shall also be considered for its easy construction without affecting surrounding environs severely.

3.3.2 Topographical data

3.3.2.1 General

Topographical features data around the river/channel both upstream and downstream of the proposed weir site, including river cross-sections, profile between these cross sections, topographic data like contour map of the weir site covering 200m on the left and right sides of the river bank at weir site and a minimum of 300m on the upstream and downstream sides need to be acquired to understand features around it and hence ranges of the layout. In general, the following topographic data are required for the study and design of a diversion weir and appurtenant structures:

3.3.2.2 Index map

An index map of scale 1:50,000 showing catchment features of the entire river system upstream of the proposed site of diversion is paramount important for identifying appropriate headwork site and boundary of potential command area. In addition to this, recent imagery map of the study area is necessary to capture recent land use/ land cover of the catchment area and command area including settlement pattern.

3.3.2.3 Topographic survey of the proposed headwork site data

Topographic survey of the proposed headwork sites is essential data as it shows the physical appearance of the natural features of an area around headwork sites, especially the shape of its surface. It enables us to look for different options for headwork site and select the best one (in terms of technical, economic and environmental requirements).

3.3.2.4 Cross-sectional survey at the proposed site data

Once the headwork site has been selected, cross-sectional survey of the river and its flood-plain up to about 5m above the floodplain along axis of the headwork, 50-100m both on the u/s and d/s of weir axis need to be executed as they are required to enable to properly define the existing river channel and helps to know nature of carrying capacity of the channel. This cross section need to be taken from left to right banks while looking in the flow direction and corresponding map should also be plotted in the same manner.

3.3.2.5 Flood mark data

Level of historical or observed flood on the banks of the river need to be identified to understand the magnitude of incoming flood and decide on necessity precautions to be taken care of longitudinal survey between u/s and d/s cross sections i.e. profile survey through thalweg of a river is required to know average bed slope. The river to be used as input for discharge estimation at that site.

3.3.2.6 Existing infrastructures around the proposed site data

Identification of any existing infrastructures around the proposed site is required either to make use of it or take care of the necessary precaution in design. This infrastructure can be settlement area, road, bridge, other diversion structure, etc. They are usually associated to the social value of the community.

Details on topographic survey can be referred in "GL 4: Topographic and Irrigation Infrastructures Surveying Guideline for SSID ".

3.3.3 Hydrological data

3.3.3.1 Design flood

For the design of headwork structure the design flood of the river shall be provided with probability of exceedance. The probability of exceedance is usually expressed in terms of return period. Usually a design flood of 50 to 100 years return period is quite common. If the area is vulnerable to flood a design flood for flood protection dyke is also required to be provided.

Details on hydrological data analysis can be referred in "GL 3: Hydrology and Water Resources Planning Guideline for SSID” as it is directly related with cost of the structure.

3.3.3.2 Monthly low flows data

Irrigation using diversion structure relies on the monthly distribution of river flow. Therefore, understanding on the low flows of the river and associated water levels are important. The hydrology analysis should provide information in this regard.

3.3.3.3 Stage - discharge relationship data

Based on the data from topographic survey and hydrological analysis, the relationship between discharge and the corresponding water level are required to be established for the selected site. The stage-discharge relationship should be verified and confirm its validity for the historical flood level, if any.

3.3.3.4 Drainage module of the area

Drainage module is one of the most important parameter for the design of drainage system and cross drainage works. The capacity of the drains and cross drainage structure is determined by the drainage module and the area to be drained. The drainage module depends on soil type, climate and land-use/land/cover.

3.3.3.5 Sediment data

Sediment rate in the river is another hydrological data which need to be assessed in order to decide on the requirement of sediment excluding facilities. The sediment characteristics of the river (during normal flow and flood season) is required to be determined. Sediment sampling need to be carried out at least once during the normal flow.

3.3.3.6 Analyzed water quality data

Data on water quality of the source river water is also required. The chemical characteristics (acidity and Alkalinity) is required to be known. The chemical characteristics need to meet the standards set for irrigation water by various institution and organization like FAO. Water sample has to be collected during the normal as well as the dry season.

3.3.4 Geology and geotechnical data

3.3.4.1 Geotechnical Data

Geology and geotechnical data including riverbed material and features such as rock outcrops from foundation investigation are crucially required to know compressive strength of foundation material and its bearing capacity and study piping condition.

3.3.4.2 River banks data

Geologic conditions of river bank data are required for wing walls' stability analysis at the headwork site.

3.3.4.3 Foundation data

Depending on the identified foundation material of the river bed along weir axis as recommended by geologist, we need to fix depth of cut off trench to the required good foundation material.

3.3.4.4 Weir Site Geological Cross Section Data

A graph showing geological cross section along headwork axis should be presented for deciding depth of foundation and requirement for protection works. In addition to this, over burden information and their suitability are required to decide type of structure on permeable foundation or on hard foundation.

3.3.4.5 Laboratory test results data

Results of sampled soil and rock laboratory tests are essential as it indicate permeability condition, Angle of internal friction, cohesion, density, bearing capacity of the soil in the foundation and in the embankment soil. For details refer "Geology and Geotechnical guidelines on SSIP" component of the guideline.

3.3.4.6 On-site construction materials data

Location map of quarry site: Quarry site for a certain SSI project needs to be located and as is has a direct relationship with unit rate analysis of that project. Similarly, location and distance of sand, gravel, cement, RF bar, pipe, etc. need to be shown to do the same.

In addition to this, type, texture, properties/quality and quantity of construction material are important as they affect type of structure that need to be designed for a specific site.

3.3.5 Irrigation agronomy

Assessment of upstream and downstream irrigation water demands need to be obtained to account for some percentage of flow for seasonal downstream and peripheral water use while fixing abstraction amount.

To fix height of a diversion weir, net irrigable area, maximum elevation in the net command area and depth of water in the conveyance canal are necessary. For this purpose, agronomic inputs including the agricultural soil irrigation suitability or SMU based on waterlogging, salinity, etc. and soil characteristics like permeability, water holding capabilities, etc. are required. This is not only used for determining the irrigation area, but also for irrigation scheduling. Other factors that need to be considered for irrigation design include the effective precipitation, evapotranspiration, surface runoff, deep percolation, and overall crop water requirements.

3.3.6 Other data

In addition to the above basic data, the following additional data are required at the project site.

- Daily labourers' daily rate at the project site;
- Machinery rental rates at the project site;
- Local construction material costs (like cost of poles, rock, fill material, etc.) on site;
- Transportation cost to the project site;
- Loading and unloading rate at the project site, etc.

3.4 DESIGN PROCEDURE AND STEPS OF WEIR/BARRAGE DESIGN

The following flow chart shows design procedures and steps for design of a diversion headwork.

Steps for Designing Diversion Weirs for SSIP

Step-1: Assess potential weir sites on 1:50,000 map/s

Step-2: Visit the sites & select appropriate one in discussion with stakeholders

Step-3: Mobilize surveyors & get cross section, profile & topographic maps

Step-4: Relocate the weir site based on the newly produced topographic map

Step-5: Pick one of the highest irrigable command level, $Peak_{elv.}$ & compute head loss along the MC route up to this point, h_f

Step-6: Design layout & Compute monthly or decade based CWR from CROPWAT8 & select the highest irrigation requirement for actual area in l/s/ha among them & design water depth in the MC

Step-7: Estimate head loss at the head regulator, h_g & operational head losses, h_o

Step-8: Sum-up these head losses and the highest irrigable command level, i.e. $Peak_{elv.} + h_f + h_g + h_o$ (which gives Intake bottom level, IBL)

Step-9: If this level is ≤ existing RBL at selected weir site, go for design of only intake otherwise design a weir with a minimum of 1m height (to get silt clearance at intake)

Step-10: Compute weir crest level/WCL by adding IBL and Water depth in MC, d including a minimum of 15cm working head i.e. WCL = IBL + d + 15-30cm working head

Step-11: Now, compute weir height by subtracting WCL from the lowest RBL i.e. h = WCL - RBL

Step-12: Estimate effective bed width, b of the river at that river section & corresponding cross section area, A, wetted perimeter, P, velocity of flow, V and discharge, Q by Manning's eq^n & establish stage-discharge curve or rating curve of that particular river section.

Step-13: Compute TWD or d_3 corresponding to selected design discharge, Q_p from the established stage-discharge curve.

Step-14: Select *type* and *shape* of weir based on nature of river, flow, sediment, etc.

Step-15: Estimate energy head, H_e, from $Q = C_d L He^{3/2}$, where C_d=1.7 for broad crested & 2.2 for ogee shape; design discharge head, H_d (by trial-and-error or goal seek); & Velocity Head, Hav = He-Hd. For the case of submerged weir, $Q_1 = 2/3*C_d*L*\sqrt{2g}*(H_1-H_2)^{3/2}$ & $Q_2 = C_d*L*H_2*\sqrt{(2g*(H_1-H_2))}$ where H1&H2 are u/s & d/s head above WCL respectively.

Step-16: Estimate WL at u/s & d/s i.e. U/S WL = WCL + H_d & D/S WL = TWD + RBL and corresponding u/s & d/s top level of WW i.e. u/s WW = WL at u/s +0.5 & d/s WW = WL at d/s+0.4

Step-17: Assume floor level of the d/s apron i.e. stilling basin & check if $d=d_2-d_3$ lies b/n 0.2 to 0.4 by Bernoulli's eq^n. Note: If it is greater, the basin level should be depressed by some level.

Step-18: Get surface and subsurface geological & morphological conditions of river /banks & foundation/ data & cross section at headwork site

Step-19: Estimate u/s cutoff depth from Lacey's equation & d/s cutoff depth from piping & exit gradient requirements

Step-20: Fix top width of the weir wall, $b=H_d/(r-1)^{1/2}$ & Bottom width, B of the weir = $(H_b+h)/(\sqrt{(r-1)})$

Step-21: Fix thickness of impervious apron at any point by Lane's creep theory and check piping effect by computing G_E using Koslla's flow net analysis.

Step-22: Provide u/s and d/s protection mechanisms by computing apron length & stone size, if need be.

Step-23: Fix Head regulator/s size & gate (either pipe or box depending on amount of flow in intake)

Step-24: Fix under-sluice size both when u/s WL is at WCL & at its max i.e. HFL & its gate

Step-25: Carryout stability analysis both for high flood and no over flow conditions, for Sliding, Over-turning, Vertical stress/Tension & Contact Pressure or BC

Step-26: Carryout structural design of scouring & intake gates, operating deck and breast wall sizes /i.e. thickness, dia. & RF bars arrangement/

Step-27: Design & evaluate stability of wing-walls/retaining walls both for high flood and no over flow conditions, for Sliding, Over-turning, Vertical stress/Tension & Contact Pressure or BC

Step-28: Provide & design arrangement of weep holes in the body of the walls for pressure relief

Step-29: Analyze back-water surface profile on the upstream of the weir

Step-30: Diversion Weir Drawings, TOS, BOQ and Cost Estimation

Figure 3-1: Design procedures and steps for weir/barrage design

3.5 SELECTING DESIGN FLOOD OF APPROPRIATE RETURN PERIOD

After appropriate weir site is selected, the next step to consider for designing a weir is selecting design flood of certain return period for the flood which is to be adopted. Weirs are usually designed to withstand a flood event of 50 years return period (either flood level corresponding to this plus some free board or 100 years flood level). The selection however, is dependent on economic and structural requirements.

Upstream and downstream guide bunds and channel protection bunds are usually designed for a 1 in 25 or 1 in 50 year flood event level where overtopping and failure of the bunds will not result in catastrophic failure of the weir.

For L service years or design life of structures, the risk r of flood exceeding the design flood of a given return period T years can be computed as (Novak et al., 2007).

$$r = 1 - \left(1 - \frac{1}{T}\right)^{L} \qquad (3\text{-}1)$$

In general, the selection of design flood depends on the risk anticipated as this implies a matter of economics i.e. higher return period leads to larger structure and hence higher cost but lower risk.

Box 3-1:

Descriptive example-1: Should design life of structures be the same as that of the return period for design conditions? If 25 years' service life of structure is considered, how much is the risk of flood exceeding the design flood for 50 year and 100 years return period? For which percentage of risk is the project ready to anticipate?

Solutions: Design life means the minimum duration a structure is expected to last. Thus, the longer is the design life; the higher is the cost of a project. Therefore, in choosing the design life for a structure, engineers should consider the design life which generates an economical project without sacrificing the required function.

In selection of return period of certain design conditions, winds, waves, etc., one should consider the consequences of exceedance. In fact, there are normally no extreme maximum values of these design conditions and its selection is based on the probability of exceedance which is related to return period. Therefore, design life may not be equal to return period of design conditions because their selections are based on different considerations.

Based on the equation given in (3-1), the risks of flood corresponding to the given exceedance of 50 and 100 years return period design floods are 40% and 22% respectively.

3.6 RATING CURVE CONSTRUCTION

Rating curve also known as stage-discharge curve is a plot of the water levels/gauge heights on the Y-axis against the corresponding discharges on the X-axis. In the absence of detailed local measured or recorded stage-discharge data, preliminary rating curve for existing un-retrograded condition of the river can be prepared by computing the flow depth for the corresponding discharge either analytically or using computer models (like HECRAS or others). For analytical method, the flow depth can be estimated preferably at 20-40cm interval and the corresponding discharge can be computed using the Manning's formula:

$$Q = {}^{1}/_{n} * A * R^{2/3} * S^{1/2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3-2)}$$

Where, Q = Flow rate in (m^3/s)

n = Manning's roughness coefficient for compound section, Refer appendix-1 for details

A =Cross-sectional area of flow (assumed composite trapezoidal channel, but need to be estimated by breaking it in to small segments of say 0.5m wide, as shown in table 3-2 and Figure 3-3), (m^2)

R = Hydraulic mean radius = A/P, (m),

P = Wetted perimeter of channel, (m)

S = Avg. friction/longitudinal slope of channel at headwork site, S=∆H/L, (%) …….. (3-3)

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

L = distance between reference points (m)

Then after, water level (assumed normal flow depth) corresponding to selected design flood magnitude should be read from the plotted curve to use it for fixing top level of downstream wing wall and thickness of downstream apron. For stage-discharge relationship using computer model refer "GL 21: Major Application Software's Guideline for SSID".

Table: TWD Computation at Wier Axis

Stage, H (m)	Mannin g's Roughn	Water Area, A (m2)	Wetted primeter, P(m)	Hydraulic Radius, R (m)	Longitudi nal Slope, S	Velocity, V (m/s)	Discharg e, Q (m3/s)
1432.475	0.04	-	4.60	0.00	0.0246	0.00	0.00
1432.675	0.04	0.98	7.11	0.14	0.0246	1.05	1.03
1432.875	0.04	2.59	9.03	0.29	0.0246	1.70	4.41
1433.075	0.04	4.57	10.95	0.42	0.0246	2.19	10.00
1433.275	0.04	6.93	12.98	0.53	0.0246	2.58	17.89
1433.475	0.04	9.71	15.89	0.61	0.0246	2.83	27.44
1433.675	0.04	12.91	17.21	0.75	0.0246	3.24	41.78
1433.875	0.04	16.47	19.13	0.86	0.0246	3.55	58.47
1434.075	0.04	20.40	21.04	0.97	0.0246	3.84	78.39
1434.275	0.04	24.80	23.14	1.07	0.0246	4.11	101.89
1434.475	0.04	30.23	26.21	1.15	0.0246	4.31	130.40
					V_{avg} =	2.67	

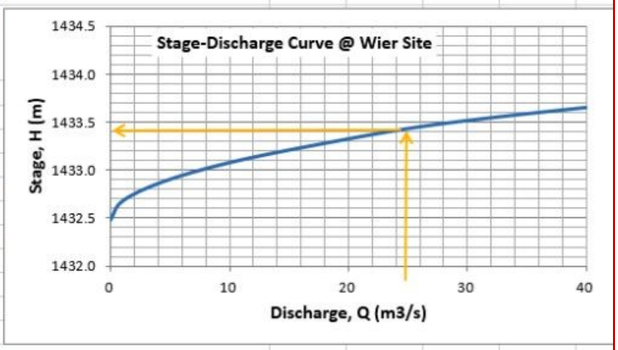


Figure 3-2: Stage-discharge-curve for *Petu SSI Project (SNNPR)*

Note: Indicated initial stage, H=1432.475 in table above is the lowest river bed level at selected weir site.

Cross section area and wetted perimeter can be estimated for each assumed raise in stage (say 0.2m for small channel or up to 1m for large channel) by either of AutoCAD or hydraulic tool box or HEC RAS software. The following steps are prepared for manual computation using AutoCAD:

Step-1: Extract river cross section from DEM or survey data along axis of the structure, if any;
Step-2: Arrange the data by concatenate in excel from cumulative distance and elevation;
Step-3: Open AutoCAD and click on polyline;
Step-4: Then paste the concatenated data;
Step-5: Then simply click enter and zoom it out;
Step-6: Now draw a horizontal polyline through the lowest point of the channel;
Step-7: Then offset it at the required interval specified above up to river banks top level;
Step-8: Starting from one end of the offset polyline, digitize along bottom horizontal bed up to another end of same line following river bank. This gives us the first wetted perimeter;
Step-9: On the same route draw another polyline and close it. This gives us the first cross section area;
Step-10: Then compute hydraulic radius from these two values;
Step-11: Compute bed slope of the river;

Step-12: Assume roughness coefficient, n and estimate flow velocity and corresponding discharge as shown in figure 3-2 above;

Step-13: Repeat same procedure and estimate flow velocity and corresponding discharge till we get a value little bit greater than the expected return period flood;

Step-14: Plot stage or head against discharge which we call rating curve

Step-15: Finally, read a stage or level that corresponds to the selected design discharge from the graph.

3.7 SUMMARY OF BASE FLOW OF SOURCE OF IRRIGATION WATER SUPPLY

Table 3-1: Estimated river flow by floating method for Petu SSI project (SNNPR)

Upstream Cross section, $b_{u/s}$	**3.5**											**Total/Avg**
Partial distance, b_{ui} (m)	0.5	0.5	0.5	0.5	0.5	0.5	0.5					3.5
Flow depth, d_{ui} (m)	0.02	0.05	0.06	0.06	0.07	0.09	0.09	0.07				0.064
Area of flow, A_{ui} (m2)	0.0175	0.0275	0.03	0.0325	0.04	0.045	0.04					0.23
Downstream Cross section, $b_{d/s}$	5.0											Total/ Avg
Partial distance, d_{di}	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5		5.0
Flow depth, d_{di} (m)	0.03	0.05	0.05	0.06	0.06	0.05	0.05	0.06	0.06	0.05	0.02	0.049
Area of flow, A_{di} (m2)	0.02	0.025	0.0275	0.03	0.0275	0.025	0.0275	0.03	0.0275	0.0175		0.26
Flow time b/n sections, t_i (sec)	13.8	11.8	12.6	12.73								
Length b/n sections (m)	9.3											
Velocity of flow, V_i (m/s)=L/t	0.73											
Actual velocity of flow, V_{act} (m/s)=(0.6*Vi +0.8*Vi)/2	0.51											
Average area of flow, A_{avg} (m^2)	0.25											
Estimated base flow, Q=V*A, m^3/s	0.125											

Note: Marginal segments are assumed triangular and others trapezoidal or rectangular as perceived.

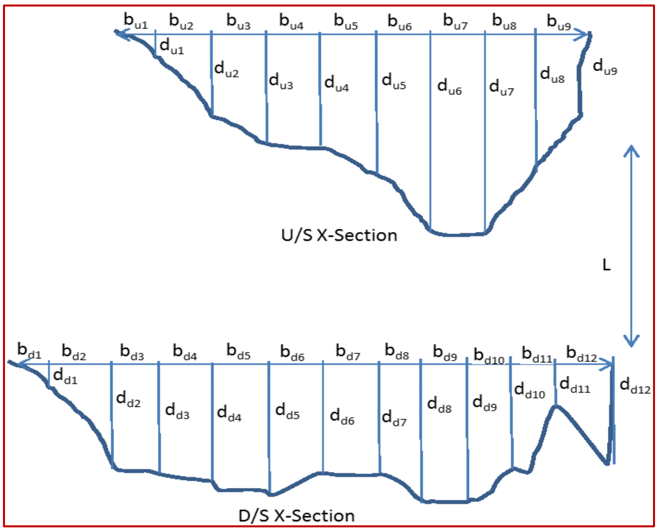


Figure 3-3: Channel cross sections at U/S and D/S ends of flow measurement site

3.8 HYDRAULIC DESIGN OF DIVERSION WEIR AND RELATED STRUCTURES

The followings are the widely practiced design procedures of a diversion weir. They can be found in various national manuals and guidelines.

3.8.1 Fixing weir/barrage crest level

3.8.1.1 WCL when there is no downstream existing structures

Weir crest level is mainly governed by the targeted peak level in the irrigable command area and water demand or depth of flow in the canal as well as head losses in the conveyance canal and losses at crossing structures (if any).

Design consideration for fixing weir crest level is that:

- The crest level should be set at desired height or level that enable to obtain the required driving head to safe delivery of the designed discharge to main canal offtake. This level has direct implication on height of a weir crest thus care should be taken as it can affect the discharge coefficient, water head over the crest, and backwater curve;
- Crest i.e. pond level, upstream of the canal head regulator shall generally be obtained by adding the working/driving head to the designed full supply level in the canal depending on the command topography;
- The weir crest should be set to allow a safely passage of maximum flood discharge within designed weir crest length;
- Provision of appropriate crest width in weir design is crucial for a smooth flow and moderate velocity.
- The bed level of the under sluice should be below sill level of canal head regulator. But level of the under sluice can be taken as equal to or greater than RBL
- The MC at the head reach should not be too deep in order to avoid large excavation work, to minimize construction cost and to reduce maintenance and side slopes stability problems;

Weir Crest Level is then given by:

WCL = (Water surface level at 1st off-take down the main canal) + (L x S) + hg; Where, L = length of main canal from the head regulator to the 1st off-take (m); S = Bed slope of the main canal; hg = Head loss at head regulator (m) (usually taken as 0.1-0.3m); ………………….. (3-4)

Water surface level at 1st off-take down the main canal = Peak irrigable land level in the command area + Water depth in the MC + Operational losses (usually 0.1m) ……….……. (3-5)

Weir Height, h = WCL – RBL ………………………………………………………………. (3-6)

(Note: RBL refers to minimum or thalweg at weir axis). This height should be high enough for allowing water in to the intake at peak demand case. Thus, a minimum of 0.15m driving head should be permitted to allow full supply to the intake.

In case of barrage, it is a controlled structure where the whole crest excluding piers is provided with a gate to control (regulate) the flow. It is costly structure but efficiently regulates the flood with low afflux. It is used in seasonal rivers where regulation is highly required.

Box 3-2:

Worked Example-2*: A design engineer has identified a peak point to be irrigated in Petu SSI Project (SNNPR) irrigable command area to be 1433.000 m a.s.l. If the length of conveyance canal up to junction to this block is 380m and bed slope of this canal, S = 0.01%, then how much is the required height of a diversion structure if a* RBL *at this site is of 1432.475 m a.s.l. Assume, Intake Level above min. RBL=0.5m; Water depth in this canal, d=0.35m (normally this is taken from MC design); and Operational head loss,* h_o*=0.10 m;*

Solution*: Head loss along the canal,* h_f *= S*L = 0.38m; Head loss at the head regulator,* h_g*=0.1m; Water surface level at 1st off-take down this canal=1433.000 +0.35++0.10=1433.450; Therefore, Weir Crest Level = 1433.450+0.38+0.1=1433.930 m a.s.l. and calculated Weir Height, h= 1.45m. Now, let's check if this height can supply the case of peak demand i.e. Bottom of Intake Level =Max. Command level +* H_f *along MC+* h_g*=1433.480m a.s.l. Its top level is thus =1433.480+0.35 =1433.830m a.s.l.*

Consequently, computed WCL-Top of Intake Level=0.10m, indicating it is less than the required minimum driving head i.e. 15cm for peak demand. Now, if we increase weir height, h=1.6m, updated WCL=1434.075m. Now, updated driving head for full supply discharge= 1434.075-1433.830=0.245m, which is greater than 15cm. Thus, adopt WCL=1434.075m a.s.l.

3.8.1.2 WCL when there is downstream existing structures

When there is a downstream existing structure along the conveyance canal, then weir crest level is mainly governed by the head required to cross such structure, rather than the targeted peak level in the irrigable command area. If there is a siphon crossing along this canal, then head losses in this structure comprises three components: trash rack losses, transition losses and pipe friction losses. Usually a head lose at such crossing is assumed 0.5m.

3.8.2 Fixing crest length of diversion headwork

3.8.2.1 General

There are four conditions to fix crest length of a weir/barrage based on flow condition as described in following sections.

3.8.2.2 Crest Length based on lacey's equation

As per Lacey, the weir crest length (P) should be adequate to pass the design flood safely. The minimum stable width of an alluvial channel is usually determined from the Lacey's wetted perimeter (P). Lacey calculated a series of regime flow equations. One of these describes the regime perimeter of a river in alluvial material in terms of its dominant flow. The equation, in metric units, is as follows (Sharma-1987):

$$P = 4.75\sqrt{Q} \quad \text{(3-7)}$$

Where, P is the wetted perimeter of the river (m), and
Q is the discharge (m^3/sec).

This equation is used as a first approximation of the likely width of the river channel and an appropriate maximum width of the weir for alluvial channels. Thus, the length of a weir shall be chosen somewhere between the regime width of the river and that which gives about $5m^3$/s/m width of weir, and depend on site conditions in each case. Where rivers are naturally constrained to less than the above values by rock banks, the weir length must be taken as the natural width of the river though it may require raised wings and hence expensive design.

In other reference books, this equation has the following form (Murthy, 2009)

$$B = 4.83\sqrt{Q} \quad \text{(3-8)}$$

Where, *B* is the waterway width (m)

It has been speculated that factors like consideration of the standard silt factor, unit conversion, etc. could have caused the variation in the coefficient. Regardless of the difference in the coefficient between the two forms of the regime equation, the width of waterway calculated is very wide in most of the cases. Therefore, the reduction in the width of the waterway is inevitable. As a result, a constant called a looseness factor, which is the ration of the actual linear waterway provided to waterway calculated by the regime formula, is being adopted. The recommended range of the looseness factor is between 0.5 and 1.0 (Murthy, 2009).

On the contrary, the minimum width of the weir can be calculated from estimate of the maximum level which the water can be allowed to back up against the weir and the height of embankment walls. An energy head of 3m over the weir crest is often considered as maximum allowable (Halcrow, 1988) which is associated with the limit of validity of the assumption for hydrostatic pressure over the weir crest during the derivation of modular weir formula given by;

$$Q_m = C_d B H^{3/2} \quad \text{(3-9)}$$

Where, Q_m is the modular flow rate in m^3/s,
Cd is the discharge coefficient (dimensionless),
B is the linear waterway in m and
H is the energy head over the weir crest in m.

For the most common type of diversion weir in small scale irrigation scheme in Ethiopia, i.e. broad crested weir, a 2m energy head will corresponds to a maximum discharge intensity of 58.8m3/s/m over the weir crest. The minimum linear waterway for the design discharge can be estimated from this relationship. This will give width of waterway much smaller than that obtained by regime equation.

As a rule of thumb, the adopted width of waterway (or simply weir crest length) is somewhere between the waterways calculated by the regime equation (maximum limit) and that which give a maximum discharge intensity of $5m^3/s/m$ (minimum limit). As much as possible the adopted width of waterway should be almost similar to the width of the natural river channel.

When there is pier to support crossing structure over the weir the effective width of waterway shall be modified due to flow contraction. The effect may be taken as:

$$L_e = L_t - 2(N K_p - K_a) H_d \qquad (3\text{-}10)$$

Where Le is effective length of crest (m),

Lt is total length of crest (m),

N is number of piers,

K_p is pier contraction coefficient,

K_a is abutment contraction coefficient, and

H_d is design head on crest.

The coefficients for the different types of shapes are given in figure 3-6 and tables 3-2 and 3-3.

Table 3-2: Pier contraction coefficient

Shape and characteristics of pier	K_p	Remark
Square-nosed piers with corners rounded the upstream approach wall and the axis of the flow, on a radius equal to about 0.1 of the pier thickness	0.02	
Round-nosed piers	0.01	
Pointed-nose piers	0.0	

Table 3-3: Abutment contraction coefficient

Shape and characteristics of abutment	K_a	Remark
Square abutments with headwall at 90^O to direction of flow	0.2	
Rounded abutments with headwall at 90^O to direction of flow when $0.5H_d \le r \le 0.15H_d$	0.1	r is radius abutment
Rounded abutments where $r > 0.5H_d$, and headwall is placed not more than 45^O to direction of flow	0.0	

3.8.2.3 Crest length for cohesive soils with velocity > 1.5m/s and <1.5m/s

The above Lacey's equation being valid has yet a limitation as to its applicability to all conditions and magnitude of rivers. It is rather appropriate to apply it for designing new river channel (artificial water way); and rivers expected to carry a flood more than their capacity due to river merging intensive soil and water conservation; urbanization or other factors. For small and self-contained rivers, the formula is not applicable as it yields highly exaggerated value.

Thus, the following formulas can be used depending on the nature of the River bed material:

For cohesive soils and rocks, with velocity >1.5 (MoWR, 2002)

$$P = \sqrt{\frac{Q * g^h}{V_p^3}} \qquad (3\text{-}11)$$

For non-cohesive soils with velocity <1.5

$$P = \frac{A\sqrt{Q}}{S^{0.2}} \quad \text{(3-12)}$$

$$A = \sqrt{\frac{S}{V_p}} \quad \text{(3-13)}$$

Where, Q is design discharge of the river
h is upstream water depth
S is Slope of the River
V_p is Permissible or non-scouring velocity. Refer table below.

Table 3-4: Permissible or non-scouring velocity of flow in rivers, (m/s)

Bed Material	Permissible Velocity of River Flow, (m/s)
Rock and gravel	1-5
Hard soil	1-1.1
Sandy loam black cotton	0.6-0.9
Very light loose sand to average sandy soil	0.3-0.6
Ordinary soil	0.6-0.9

Source: As adopted from MoWR, PART I-G Diversions, 2002

In case of meandering alluvial rivers for minimizing shoal Formation, the wetted perimeter P is further multiplied by a looseness factor. The looseness factor to be used in the above equations (3-11 and 3-12) is given by:

$$f = 1.76^* \sqrt{d_{50}} \quad \text{(3-14)}$$

Where, f is silt factor for 50% of particle distribution,
d_{50} is average particle size, mm (It is to be read from a gradation curve for the 50% of particle distribution).

Table 3-5: recommended looseness factor

Silt Factor	Looseness Factor
Rock and gravel	1.2 to 1.0
Hard soil	1.0 to 0.8

Source: MoWR, 2002

In case, the actual river cross-section is sufficient to carry the design flood, which should be ascertained by field assessment, the lacey formula may not be used. Rather, the effective width of a weir is determined by the following formula.

$$B_u = \varepsilon * B_{gross} \quad \text{(3-15)}$$

Coefficient of contraction, $\varepsilon = 1 - \frac{a * H_d}{(B + H_d)}$ ………………………………………….. (3-16)

Where a-coefficient/or shape factor is taken from sketches shown in Appendix-V.
H_d - head over crest
B - Crest width

3.8.2.4 Crest length based on actual river width

In practice, length of a weir crest is determined based on the physical characteristics of the selected weir site and the width of the existing waterway. Rationally, a weir with a long crest gives a small discharge per unit length i.e. intensity of discharge and hence, the required energy dissipater per meter of the crest width is smaller than what is needed for a shorter crest length. On the other hand, weir's crest longer than maximum wetted river width causes formation of islands at upstream side of the weir. The formation of such island upstream of the weir reduces effective length of the crest (part of the weir less effective in passing the flood).

In general, width of a weir could be the natural river channel width or it could be widened or contracted depending on the flood flow condition in the river. Usually weir crest length is the river width corresponding to the fixed crest height at that selected site. Thus, width of a river is usually kept to be width of the weir crest if the river flow is not too sluggish with a velocity of low quantum or if the flow doesn't spill over the banks. If it is too sluggish, to avoid silt accumulation we may reduce the section till we get minimum or critical silt-driving velocity. If the river flow is perennial with moderate fluctuation, the river width could be reduced to get the required commanding head in the place of constructing high head weir or in seasonal rivers, an open weir with a barrage could be cost effective and efficient.

On the other hand, the river channel width could be enlarged to reduce high head and high velocity that could damage (cause scouring) the riverbed, river banks and the structures. The high head over the crest could be reduced by elongated crest width without expanding the river channel.

Thus, as a general rule, the crest length of the weir including scouring sluice, should be taken as the average wetted width during the flood. If possible the flow per unit width should not exceed $15m^3/s/m$ so as to avoid a relatively costly energy dissipation arrangement. In such case, increasing the length of the weir crest to 1.2 times the river width is allowable. The designer need to observe that by limiting the waterway, the shoal formation upstream can be eliminated. However, it increases the intensity of discharge and consequently the section of the structure becomes heavier with excessive gate heights and cost increases, though the length of the structure is reduced.

3.8.2.5 Crest Length for Barrage

In case of barrage, the design flood usually passes through successive openings and hence the flow hydraulics is quite different to diversion weir. Therefore, the modular weir discharge formula does not work here. Therefore, the design assumes a standard gate opening size and determines the maximum number of gates required to allow the passage of design flood safely. And the size of gate opening of the barrage can be calculated based on submerged orifice formula given by (USBR, 1987; Ankum, 1991; Smith, 1995; Novak et al., 2007)

$$Q = CA\sqrt{2gh} \quad \text{...} \quad (3\text{-}17)$$

Where, Q is discharge in m^3/s,
C is coefficient for the gate,
A is area of gate opening in m^2,
g is gravitational acceleration in m/s^2,

h is the differential head across gate opening in m.

Determination of *h* requires detail information on both sides of the gate. Therefore, in practice *h* is taken as conservatively the difference in pool between upstream and downstream even if actual effective differential head across the gate opening is marginally greater than this. Therefore, for easy of work, the above formula is simplified as gate flow formula (Chow, 1986; Ankum, 1991).

$$Q = C_d Bw\sqrt{2g\,d_1} \quad \text{.......... (3-18)}$$

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

C_d is the discharge coefficient,
w is the height of gate opening (gate setting) in m,
d_1 is the average headwater depth above the intake crest level,
B the effective width or opening (water way) of the intake structure in m and
g is the gravitational acceleration in m/s2.

The discharge coefficient can be estimated by:

$$C_d = \frac{\mu}{\sqrt{1+\mu\frac{w}{d_1}}} \quad \text{.......... (3-19)}$$

Where, μ is the contraction coefficient of the jet, usually taken as 0.6.
Others as defined in previous section

Once the size of a single gate opening is determined, the number of gate shall be obtained from the magnitude of the design flood.

The flow through the gate can be either free or submerged. Free orifice flow prevails when the tail water level is below a certain level which can be interpreted in terms of anticipated head loss. Free orifice flow occurs with limit of gate opening and tail water level. In practice gate operates as free with in the following limits

$$\pm 0.15\,d_1 < w < 0.5\,d_1 \quad \text{.......... (3-20)}$$

And the associated minimum head loss is

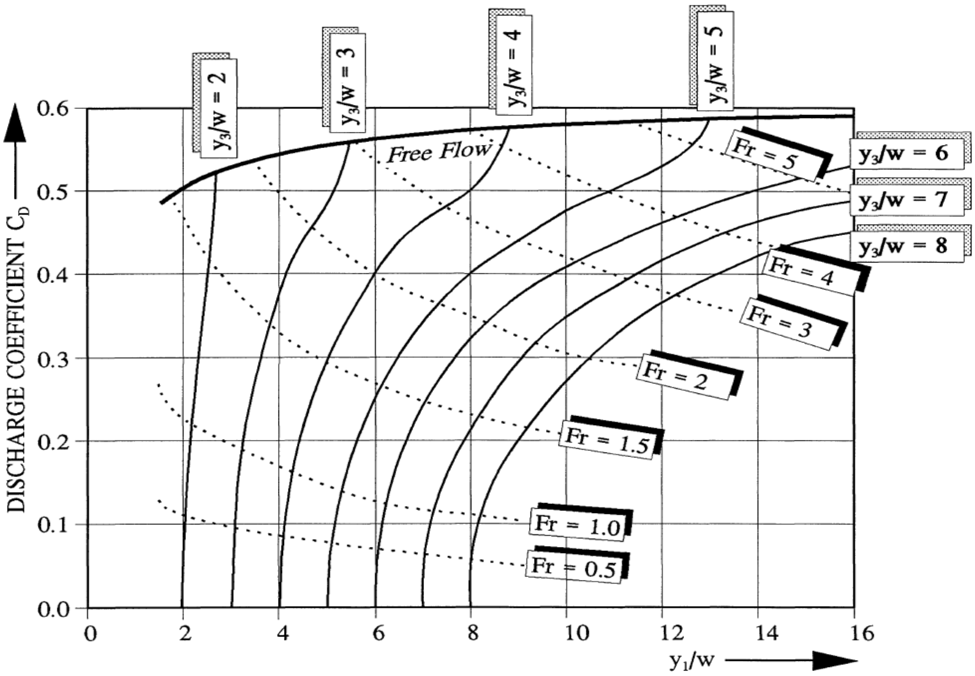
$$\frac{1}{2}d_1 < Z_{\min} < \frac{1}{4}d_1 \quad \text{.......... (3-21)}$$

Where z_{min} is the minimum head loss in m.

The above is valid if the flow is free underflow otherwise the computation will not yield correct magnitude of discharge. The free underflow at an adjustable vertical gate occurs when the tail-water level is below a critical value called d_3 both measured from the same reference level. The critical value is given by:

$$d_3 < 0.37\,d_1 + 0.75w \quad \text{.......... (3-22)}$$

If computation dictates the flow under the gate is submerged, the discharge coefficient formula cannot be used and is obtained from figure 3-4 using the ratio of headwater to gate setting and tail-water to gate setting.



When the flood level reaches a stage corresponding to the deign discharge, the entire gates of the barrage is required to be opened completely (lifted out of the water). It has to be understood that when gate opening reached more than two-third of headwater depth, i.e. w=2/3d1, the flow through the gate shifts from orifice to weir flow hydraulics and hence the weir flow formula applies.

In general, the barrage opening has to be checked to fulfill that the design flood pass through the gate openings while computing using the weir formula under submerged condition. In addition, the piers and abutment to cause side contractions have important influence in the case of barrage and hence a correction need to be made on the width of waterway. The effective width of the overflow section is less than the net width of the barrage opening. This can be represented by formula given in equation 3-10.

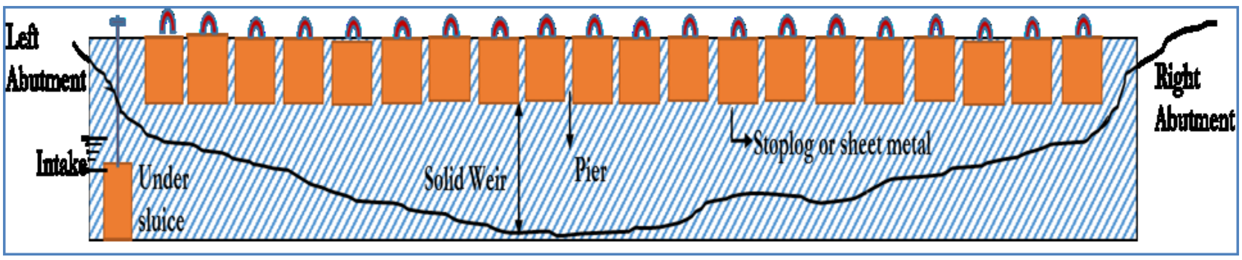


Figure 3-4: Typical Cross Section through and Arrangement of Barrage Diversion

3.8.3 Determination of flow depth over broad crested weir

The general formula for computing discharge, Q, passing over such weir crest is given by:

$Q_p = C*L_e*H_e^{3/2}$ .... (3-23)

Where, Q_p is flood discharge of a given return period (m^3/s)
C is coefficient of discharge equal to 1.7
L_e is effective crest length (m), $L_e = L_t - (2*(N*K_p + K_a)*H_e) - t$ .... (3-24)
L_t is Total crest length i.e. bank width at crest level (m)
N is Number of piers
K_p is pier contraction coefficient (as given in figure 3-6)
K_a is abutment contraction coefficient
t is thickness of each pier (m)
H_e is specific energy head (m)=$H_d + H_{av}$ (Refer the sketch shown below) .. (3-25)
H_d is flow depth on the crest (m)
H_{av} is approach velocity head (m)

Now, deriving equation from $H_e = H_d + H_{av}$, $H_{av} = V_a^2/2g$ and $V_a = Q/A$, we will have:
$V_a = Q/A = Q_d/(L_e*(h+H_d))$ and from eq. (3-13), $H_e = (Q_d/CL_e)^{2/3}$.

Thus, $H_e = H_d + (Q_p/(L_e*(h+H_d)))^{\wedge 2}/2g$.. (3-26)

Then substituting equation (3-26) in to equation (3-23), we can solve for H_d by goal seek (self-iterative or trial and error method on excel) i.e. compute and compare till the left side H_e from (3-23) equals the right side i.e. by inserting different values of H_d in $H_d + (Q_p/(L*(h+H_d)))^{\wedge 2}/2g$).

Note: Discharges through a barrage under free flow condition is determined by the same formula as above. But in case of submerged flow conditions, the flow over the weir is modular when it is independent of variations in downstream water level. For this to occur, the downstream energy head over crest (E_2) must not rise beyond 80% of the upstream energy head over crest (E_1). The ratio (E_2/E_1) is known as the "modular ratio" and the "modular limit" is thus the value (E_2/E_1 = 0.80) of the modular ratio at which flow ceases to be free.

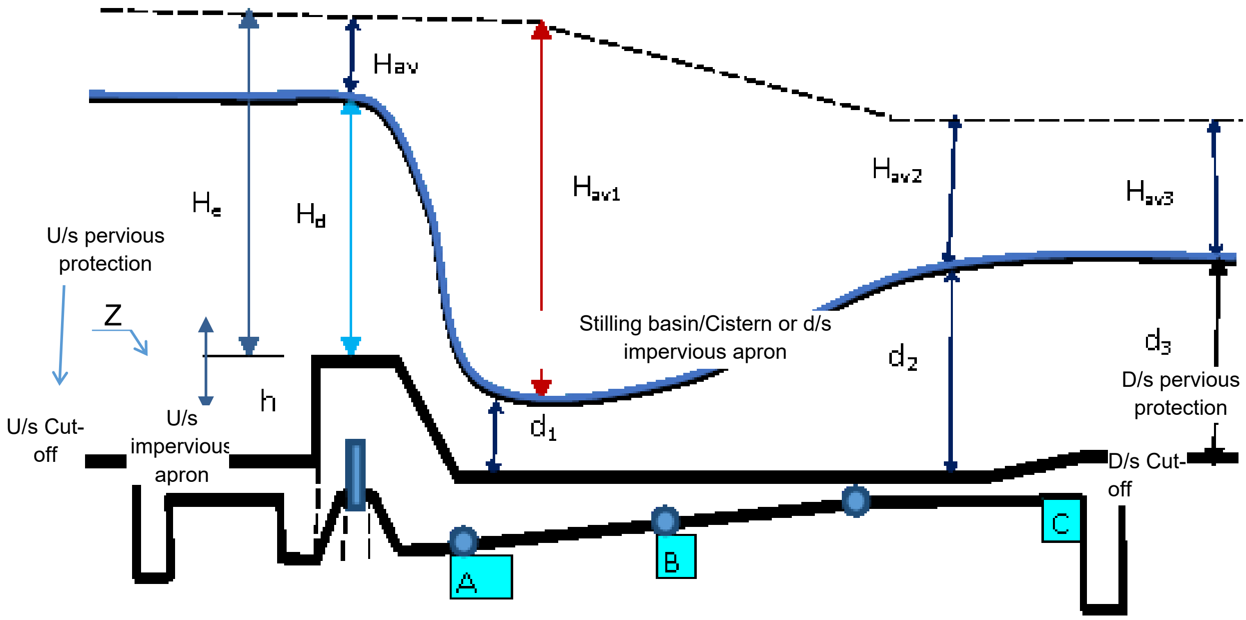


Figure 3-5: Schematic Arrangements of Flow Heads over a Weir (Typical)

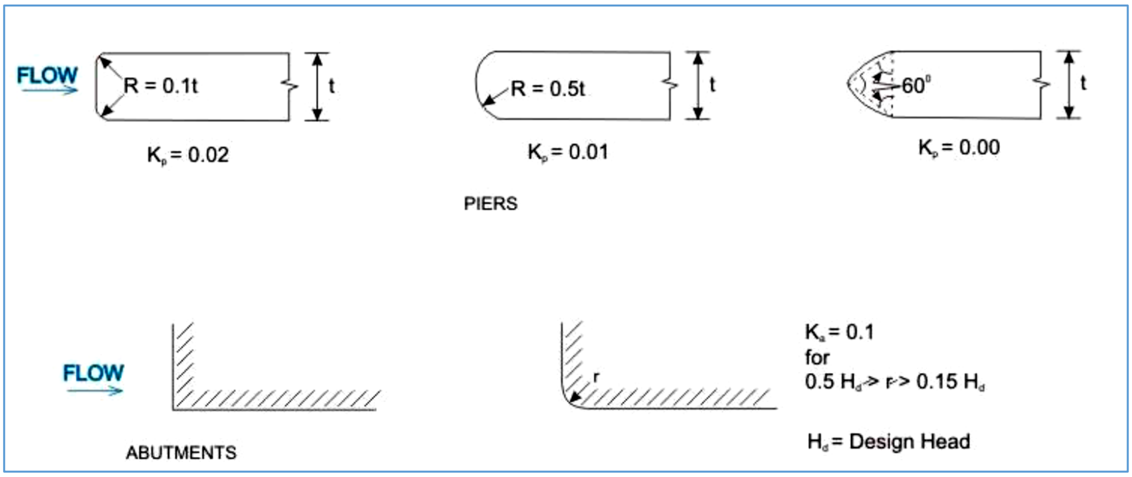


Figure 3-6: Recommended values of Kp and Ka

3.8.4 Determination of flow depth over glacis and ogee weirs

The same procedures which are described above for the determination of flow depth over broad crested weir can be applied here in this case too, but the only difference is, coefficient of discharge, C which is equal to 2.2 for ogee weir and 1.84 for Glacis and Sharp crested weir. Note: the difference in all the three cases is value of coefficient of discharge, C= (2/3)*C_d√(2g), where Cd= 0.58, 0.62 and 0.75 for broad crested, glacis and ogee shapes respectively.

3.8.5 Flow depth over barrage

A barrage diversion structure consists of two parts: the overflow part located between piers that acts as a broad crested weir and the shutter part which acts as a sharp crested weir. Thus, it behaves as both hydraulic structures either as a broad crested weir when head over the barrage bays crest, H_d is less than 1.5 times width of the crest and as a sharp crested weir when it is more than 1.5 times width of the crest (S.K. Garg, 2006). Even when the shutters are not removed completely, there is a condition when the barrage structure acts as an orifice flow, but this can happen in rare cases.

Normally, it is assumed that shutters will be removed during flood times thus the broad crested weir case can be considered however, in case these shutters are not removed due to different reasons, then the worst case can happen. Anyway, assuming that the barrage will be operated as per the design, flow over barrage structure can be considered for the first two scenarios:

$$Q = C*(L - K*n*H_e)*H_e^{3/2} \quad \text{...... (3-27)}$$

Where,

Q is discharge over the barrage, m^3/s

C is discharge coefficient =1.7 for BCW and 1.84 for sharp crested weir

L is clear waterway lengths i.e. crest lengths of the overflow (m)

K is Coefficient of end contraction (equals twice the number of gated bays and varies from 0.1 for thick blunt pier noses to 0.04 for thin pointed noses: generally taken as 0.1 in ordinary calculations.

n is number of end contractions

H_e is specific energy head (i.e. H_d + h_{av}), (m)

Consequently, flow depth over such structure, H_d needs to be considered for either of these cases which is found fulfilling the above stated conditions.

3.8.6 Checking modularity of a weir

Modularity or non-submergence of a weir is checked by computing the ratio H_2/H_1 (where H_1 and H_2 are the upstream and downstream head above the weir crest respectively, refer figure 2-9). The design should check modularity and if it is found that the weir is submerged during the design, the following table shall be used to correct the modular discharge in to submerged condition.

The discharge capacity of an overflow structure if submerged by tail water level is:

$$Q_s = fQ_m \quad \text{(3-28)}$$

Where Q_s is submerged flow in m^3/s,
Q_m is the modular flow in m^3/s, and
f is correction factor.

Table 3-6: Correction factor for discharge for submerged (non-modular) flow

Type of structure	H_2/H_1	f	Remark
Broad-crested weir	≤0.75	1	Upstream and downstream faces vertical or sloping
	0.80	0.95	Vertical faces
	0.85	0.88	
	0.90	0.75	
	0.95	0.57	
	0.80	≅1	Upstream face 1:5, downstream face 1:2
	0.85	0.95	
	0.90	0.82	
	0.95	0.62	
	0.80	≈1	Upstream face 1:1, downstream face 1:2
	0.85	0.98	
	0.90	0.90	
	0.95	0.73	

This value usually varies from 0.79 to 0.94 for broad crested weirs with a shallow sloping back i.e. downstream face, while for a vertical back face, the modular limit varies from 0.67 to 0.92, depending on the value of the ratio (H_2/H_1).

Box 3-3:

***Worked Example-3**: If the design flood of a 50 years return period in a river supposed to supply the command area mentioned in worked example-2 is found out to be 24.8m^3/s, how high is flow depth on the crest of this broad crested weir. Given: The headwork is designed to supply on both left and right flanks, operating pier thickness is 0.5m and sluice channel width is 0.70m. Average width of the river at the diversion site is 9m. Level corresponding to Q_d at this site =1433.450m (From rating curve).*

***Solution**: Since this headwork is intended to supply on both sides, we need to provide two sluice channels and hence we will have two piers. Consequently, effective width of the crest = 9-2*(0.7+0.5) = 6.6m. From* $Q=C*L*H_e^{3/2}$, $H_e= (Q/C*L)^{2/3}$ = 1.698m. Now, inserting different values of H_d in equation (3-16) till it equals 1.698, or computing for H_d by goal seek gives H_d=1.622m.
Now, check for modularity or submergence of the weir. U/S WL = WCL+H_d = 1435.697m and D/S WL = TWD + RBL = 1433.450m, where TWD=Level corresponding to Q_d – RBL= 0.98m. Thus, U/S water height above weir crest (WC), H_2 =1435.697-1434.075=1.622m and D/S water height above WC, H_1 = -0.62m, meaning it is not submerged, thus the flow is modular in this case.

Box 3-4:

Worked example-4: *If we plan to provide bridge over this structure, what are the advantages of assigning the central pier and the abutment as fixed piers?*

Solutions: *(i) For abutment pier to be assigned as fixed pier while the bridge is quite long, the longitudinal loads due to earthquake are quite large. As the earthquake loads are resisted by fixed piers, the size of fixed piers will be large and massive. In this connection, for better aesthetic appearance, the selection of abutment as fixed piers could accommodate the large size and massiveness of piers. Normally abutments are relatively short in height and for the same horizontal force; the bending moment induced is smaller. (ii) Secondly, for the central pier to be selected as a fixed pier, the bridge deck is allowed to move starting from the central pier to the end of the bridge. However, if the fixed pier is located at the abutment, the amount of movement to be incorporated in each bearing due to temperature variation, shrinkage, etc. is more than that when the fixed pier is located at central pier. Therefore, the size of movement joints can be reduced significantly if central pier (being chamfered at both ends) is provided.*

3.8.7 Determination of weir/barrage geometry

The following sections present the existing and widely practiced methods for determination of the section geometry of diversion weir.

3.8.7.1 Geometry of broad crested vertical drop weir

This section is concentrated on how to fix top and bottom width of weir body. The weir body is the main component of the diversion weir structures and its section is determined as follows.

If the weir is of broad crested (vertical drop weir and sloping glacis weir) type, the top width is the maximum value of the following four equations:

$$b = \frac{H_e}{\sqrt{\rho}} \quad \text{……(3-29)}$$

$$b = \frac{3 * H_e}{2 * \rho} \quad \text{……(3-30)}$$

$$b = S + 1 \quad \text{……(3-31)}$$

This top width can also be estimated using Etcheverry's Method as:

$$b = 0.552 * \left(\sqrt{h} + \sqrt{(H_d + H_{av})}\right) \quad \text{……(3-32)}$$

Where, b is the top width of a weir, (m)

He is the specific energy head over the weir crest during the design flood, (Ii is the sum of overflow depth and approaching velocity head), m

ρ is the specific gravity of the weir body material and

S is the shutter height if it is provided (in case of barrage).

h, H_d, and H_{av} are as described in figure 3-5.

The recommended top width for diversion weirs of SSI Project from practice/experience varies from 1.0 m to 1.5m with vertical upstream face and 1:1 downstream slope of the weir body (if the selected weir type is vertical drop weir).

The bottom width, B is given by:

$$B = \frac{H + H_e}{\sqrt{\rho - 1}} \qquad (3\text{-}33)$$

Where, H is height of weir, m

He and ρ are as described above

However, these formulae give preliminary sizes of the structure as the final one is governed by the stability requirements of the structure especially for no over flow condition and the downstream channel is dry, i.e. when the u/s water level is at crest level.

3.8.7.2 Geometry of ogee weir

If the weir is of ogee type, then the crest shape is determined based on the design head. The profile of ogee weir is thus fixed for the design head as it is generally chosen to give the maximum practical hydraulic efficiency, in keeping with the operational requirements, stability and economy of the structure. If the actual head is less than the design head, the pressure on the crest gives positive (i.e. above atmospheric) value. However, for the actual heads greater than the design head, the pressure on the crest is negative (i.e. less than the atmospheric pressure) thus it leads to cavitation (i.e. A hydraulic phenomenon of formation of vapor bubbles and vapor pockets within dropping water caused by excessive stress. Cavitation may occur in low-pressure regions where water has been accelerated). Thus, to avoid any possibility of negative pressures on the crest, the ogee crest shall be designed for a design head in the condition, when the under sluices are not in operation and whole of the design flood passes over the weir crest. Usually, the type of ogee selected for SSIP is vertical upstream face and downstream face sloping at 45 degrees to the vertical.

The downstream profile of the weir crest is represented by the equation:

$$X^n = K * H_e^{n-1} * Y \qquad (3\text{-}34)$$

Where, He is the design head including velocity head, (m)

X and Y are coordinates of the points on the crest profile with the origin at the highest point of the crest, called the apex.

K and n are constants depending on slope of u/s face (as shown below)

For vertical upstream faced ogee weir, the value of K and n are 0.5 and 1.85, respectively.

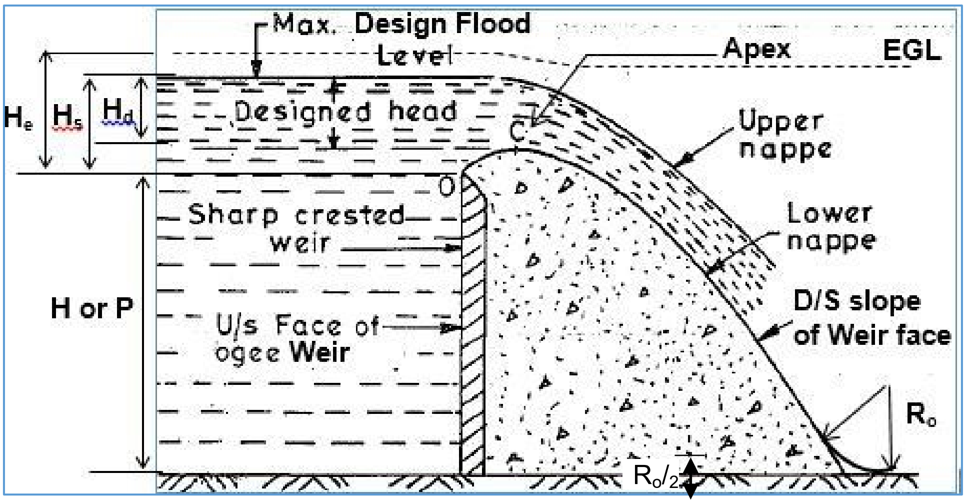


Figure 3-7: Components of ogee weir, (S.K. Garg, 2006)

Table 3-7: Values of the constants K and n (S.K. Garg, 2006)

Slope of the u/s face of the spillway weir	K	n
Vertical	2.0	1.85
1H:3V	1.936	1.836
1H:1.5H i.e. 2H:3V	1.939	1.810
2H:3V	1.873	1.776

Source: US Army of Engineers, WES

Determination of the discharge coefficient for ogee weir follows a series of trial and error method. Furthermore, it requires the use of charts and figures. Here is the procedure for determining the coefficient of discharge, C for ogee spillway:

i. Assume the anticipated design head over the weir for the given design flood and width of waterway;
ii. Read value of C based on the ratio of height of weir to design head i.e. $\frac{P}{H}$ (Figure 3-8)
iii. If head on crest is different from design head, a correction for C can be read from Figure 3-9.
iv. Using the value of C, determine all the losses and calculate the effective head i.e. H_e
v. Read value of C based on the ratio of height of weir to the effective head i.e. $\frac{P}{H_e}$ (Figure 3-8)
vi. Based on value of $\frac{C_i}{C_v}$ based on the ratio of height of weir to the effective head and for the appropriate upstream slope (Figure 3-10)
vii. Determine the value of C by multiplying the value of C and $\frac{C_i}{C_v}$
viii. Calculate the total head and read degree of submergence from Figure 3-11. The reading should be in the supercritical zone of the figure.
ix. Read the value of $\frac{C_s}{C_o}$ for the above value of degree of submergence from Figure 3-12
x. Determine the value of C by multiplying the value of C and $\frac{C_s}{C_o}$
xi. Using the above C value determine the velocity head by multiplying the above value with the effective head or H_e
xii. Determine the velocity of flow and calculate the velocity head. If the velocity head obtained above is almost similar to this one, stop the computation and take the C value for the design flood otherwise continue by changing width of waterway or design head over weir until you get similar result.

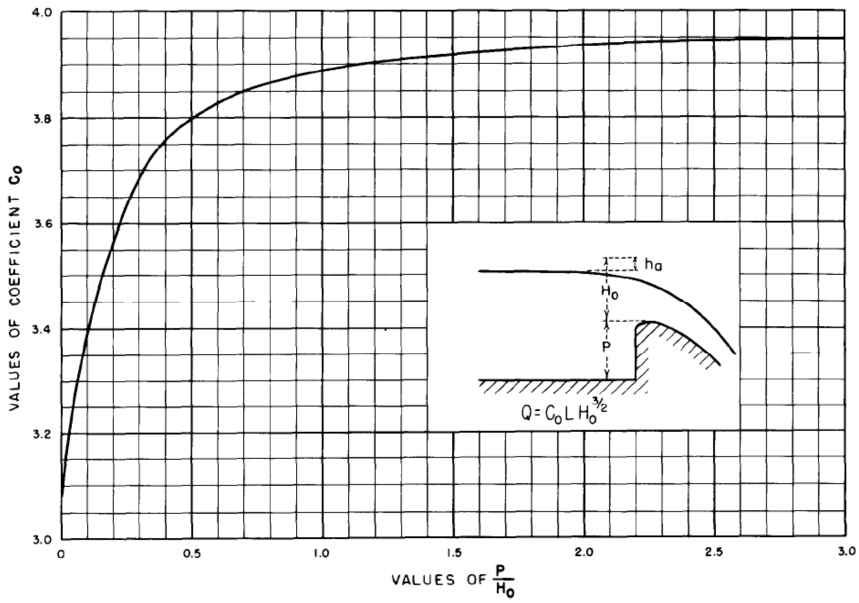


Figure 3-8: Discharge Coefficients for Vertical Faced Ogee weir (mowr, 2002)

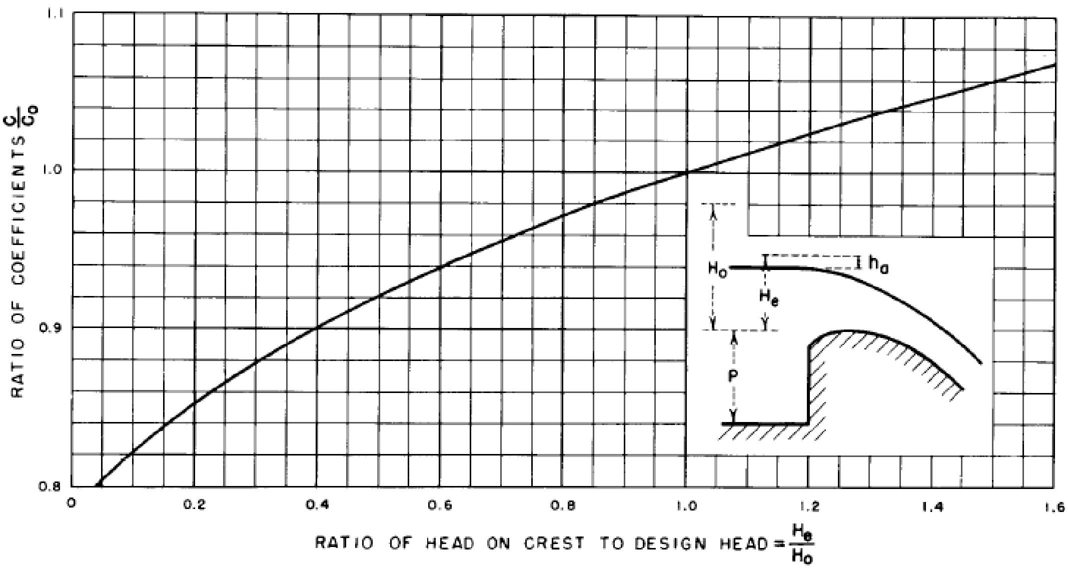


Figure 3-9: Discharge coefficients for other than the design head (MoWR, 2002)

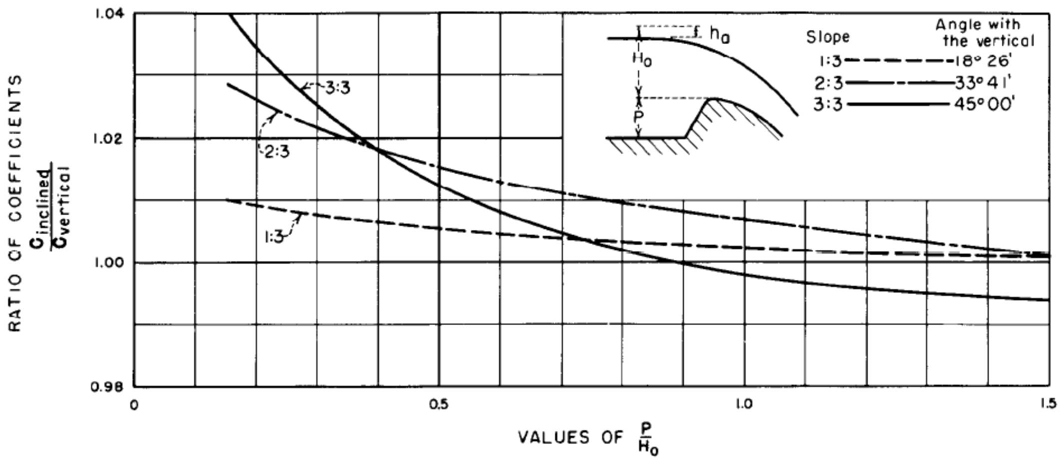


Figure 3-10: Discharge Coefficients for Ogee Shaped Crest with Sloping u/s Face

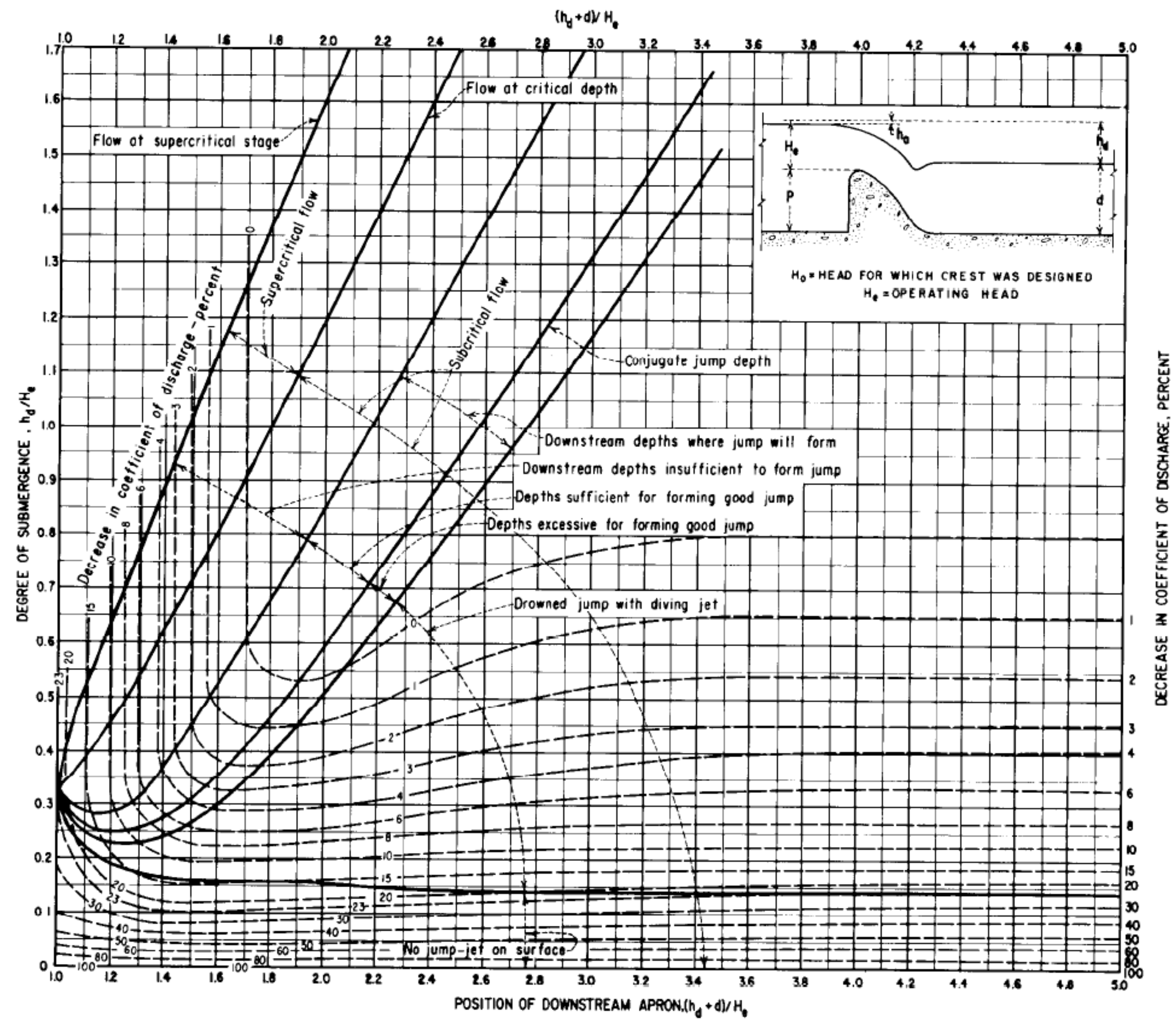


Figure 3-11: Effects of downstream influences on flow over weir crests

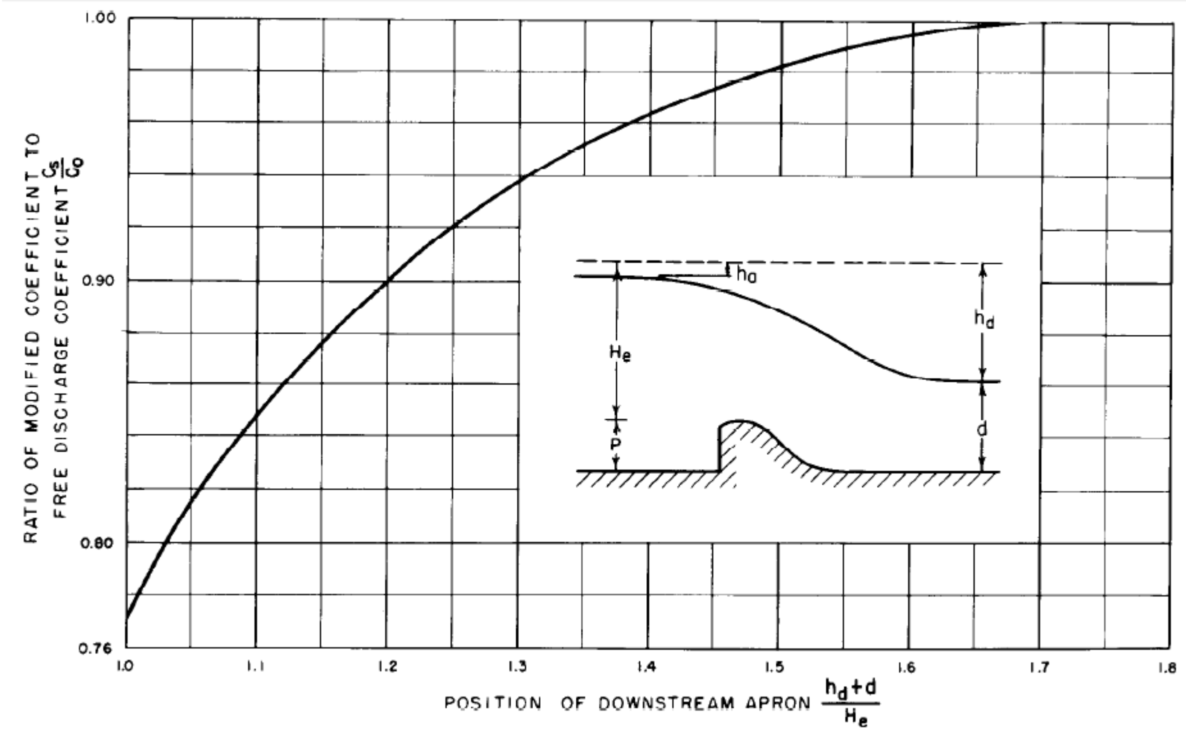


Figure 3-12: Ratio of discharge coefficients resulting from apron effects

From stability and construction point of view, a downstream slope of 1:1 is commonly provided following end of ogee shape and extends to a horizontal distance of (for vertical drop):

$$X = \left(\frac{2 * H_e{}^{0.85}}{S * 1.85} \right)^{\left(\frac{S}{0.85} \right)} \quad \text{......} \quad (3\text{-}35)$$

Where, X is horizontal distance (m)

S is slope of d/s face of weir (e.g. if it is 1:1, then S=1 and if 1:3, then S=3)

According to the latest studies of U.S. Army Corps, the u/s curve of the ogee spillway having a vertical *u/s* face, should have the following equation:

$$Y = \frac{0.724 * \left(X + 0.27 H_e \right)^{1.85}}{H_e^{0.85}} + 0.126 H_e - 0.4315 * H_e^{0.375} * \left(X + 0.27 * H_e \right)^{0.625} \quad \text{..} \quad (3\text{-}36)$$

Where, Y is vertical distance, (m)

He and X are as defined above at any point X, (m)

This profile extends from peak point (crest level or apex) back to $X = -0.27*He$ …………… (3-37)

The radius of the base bucket is calculated as follows (MoWR, 2002):

$$R_o = 0.305 * 10^k \quad \text{......} \quad (3\text{-}38)$$

$$K = \frac{V + 6.4 + h_a + 4.88}{3.6 * H_d + 19.5} \quad \text{......} \quad (3\text{-}39)$$

$$V = \sqrt{2g * \left(h / 0.5 * H_d \right)} \quad \text{......} \quad (3\text{-}40)$$

Where, V is Velocity at the toe of the weir (m/s)

The radius of base bucket, R_o can also be assumed to be ¼ of H or P, as per S.K. Garg, 2006.

ha is approach velocity head (m) and given by, $h_a = \frac{V_a{}^2}{2g}$ ……………………….. (3-41)

The discharge coefficient, C can be determined by the relation of P/h_a when:

P/h_a> 1.33, velocity head is negligible thus, C=2.225 ………………………......…. (3-42)

P/h_a< 1.33, velocity head is need to considered, thus C is read from the curve given below.

Procedure for determining C:

i. Calculate Hc/Hd and P/Hd
ii. Then from the curve, read C/Cd
iii. Calculate actual discharge coefficient from $C = c/C_d * 2.225$……………….....……… (3-43)
iv. If the u/s face is not vertical correct the value of C calculated in step iii, multiplying it by the correction factor for given P/Hd. Repeat the procedure with the corrected value of C and fix Hc.

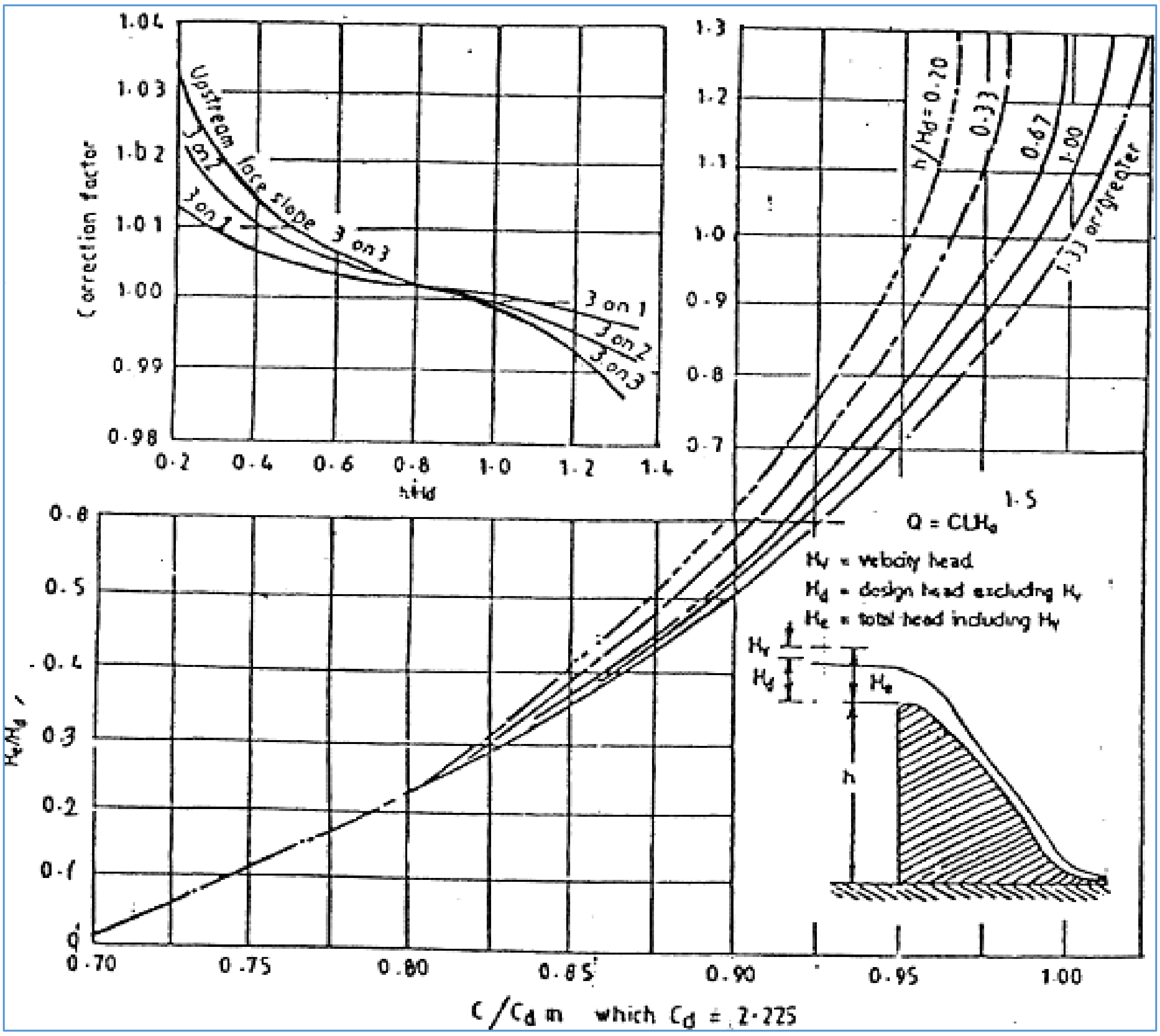


Figure 3-13: H-Q relation for Selecting Coefficient of Discharge, C (MoWR, 2002)

3.8.7.3 Geometry of sloping glacis weir

Such weir has sloping faces both on the upstream and downstream. The slope of the glacis u/s of the stilling basin has little effect on the jump as long as the distribution of velocity and depth of flow are reasonably uniform on entering the jump. The sloping glacis weirs have the inherent advantage of stability. On rivers subject to high velocity flows carrying boulders, weirs should be made as low as possible and a shallow glacis weir would best transport boulders safely over the weir. Glacis weirs with an upstream slope of 1V:1H to 1V:1.5H and a downstream slope of 1V:1.5H to 1V:2.0H shall be adopted as required.

The position of the start of the jump on a sloping glacis can be estimated by projecting the conjugate depth back from the tail water level to the incoming flow level.

3.8.8 Design of energy dissipater

3.8.8.1 General

Various types of structures have been developed for the dissipation of energy of flow. The dissipation is achieved by transforming super-critical flow into sub-critical flow through hydraulic jumps, turbulence, impacts, etc. Commonly used dissipaters in irrigation works are:

- (i) USBR hydraulic jump basins (Types I to IV)
- (ii) Vlugter basin;
- (iii) SAF Basin;
- (iv) Impact block (straight drop) type basin;
- (v) Slotted grating dissipater;
- (vi) Impact type stilling basin;
- (vii) Plunge pool;
- (viii) Stilling well;
- (ix) Baffled spillway and
- (x) Deflector bucket

Dissipaters' of type (i) to (v) and (x) are mostly used in spillways, whereas types (vi) to (ix) are used in outlets. Deflector buckets are suitable for use in headworks structures, but not generally used in canal works. In this manual only the hydraulic jump basins will be discussed.

3.8.8.2 Hydraulic jump computation

Hydraulic jump is one of the flow hydraulic characteristics which takes place when a super-critical flow changes into a sub-critical flow. This characteristics usually occurs in the stilling basin and is mainly a function of the Froude number, which also defines types of stilling basin. Froude number is generally represented by F_r and is computed by:

$$F_r = \frac{V}{\sqrt{g * D}} \quad \text{(3-44)}$$

Where, V is mean flow velocity (m/s) and
D is Hydraulic depth or Hydraulic mean depth (=A/T), thus it is same as flow depth, d in case of rectangular flow sections (m)
A is flow cross sectional area (m^2) and
T is the water surface width (m)

The Froude number is generally used for scaling free-surface flows, open channels and hydraulic structures.

For rectangular channels, The Froude number is given by:

$$F_r = \frac{V}{\sqrt{g * d}} \quad \text{(3-45)}$$

For a channel of irregular cross-sectional shape, the Froude number is defined as:

$$F_r = \frac{V}{\sqrt{g * \frac{A}{B}}} \quad \text{(3-46)}$$

Where, F_r is The Froude number, dimensionless;
V is the mean flow velocity (m/s);
d is the characteristic geometric dimension and represent internal diameter of pipe for pipe flows and flow depth for open channel flow in a rectangular channel (m);
A is the cross-sectional area (m^2); and
B is the free-surface flow width (m).

To ensure adequate hydraulic jump stilling action, it would be desirable to keep the Froude number of the incoming flow (F_{r1}) as high as possible. The Froude numbers shown in table 3-5 from Chow provides some guidance.

3.8.8.3 Determination of jump length

Length of jump is the distance that is generated when there is a transition from supercritical flow to subcritical flow condition. It is thus the main indicator and decisive parameter for estimating length of stilling basin. Hydraulic jump entering a stilling basin can be interpreted by a curve called specific energy curve. This curve is given by:

$$H_E = d + \frac{V^2}{2g} + \frac{q^2}{2g\,d^2} \quad \text{(3-47)}$$

Where, H_E is Specific energy, (m)
d is flow depth (m)
q is unit discharge or discharge per meter width (m^3/s/m)

A graph of specific energy plotted against depth generates u-shape curve with the specific energy a minimum at a turning depth called critical depth as shown in figure below. Here, the flow passes from subcritical to supercritical. At critical flow,

$$V^2 = g*D \quad \text{(3-48)}$$

In other words the Froude Number F_r = 1. At subcritical flows, the Froude Number is less than 1, because the velocity is low and the depth is large. At supercritical flows the Froude Number is greater than 1.

The specific energy curve shown in figure 3-14 illustrates that for any energy level there are two possible depths: one subcritical and one supercritical. These are known as conjugate depths, and if the flow state changes from one side of critical to the other it also changes from one depth to its conjugate depth. The most dramatic instance of such a transition is the hydraulic jump.

$$d_c=(q_c/\sqrt{g})^{2/3} = 2/3 * H_{Emin} \quad \text{(3-49)}$$

Where, V is flow velocity, (m/s)
d_c is critical depth of flow, (m)
q_c is discharge per unit critical width, (m^3/s/m)

At this point, the curve produces a minimum specific energy, H_{Emin}.

The depth before the jump is called the initial depth (y_1 or d_1) and the depth after the jump is called the sequent depth (y_2 or d_2). These depths are shown on specific energy curve shown below. These depths of flow must be differentiated from alternate depths y_1 and y_2'. y_2' is the depth that shall occur in a subcritical flow if there was no loss of energy in the jump formation; while y_2 is the actual depth that occurs after the jump, involving the energy loss H_l.

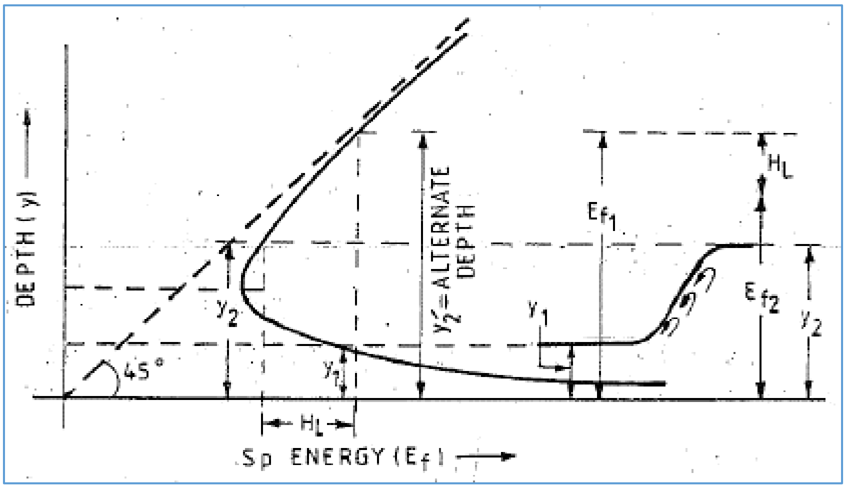


Figure 3-14: Hydraulic Jump Interpreted By Specific Energy Curve

$$Hl = Ef_1 - Ef_2 = \frac{(d_2 - d_1)^3}{4 * d_1 d_2} \quad \text{(3-50)}$$

$$Ef_1 = d_1 + \frac{V_1^2}{2g} \quad \text{(3-51)}$$

$$Ef_2 = d_2 + \frac{V_2^2}{2g} \quad \text{(3-52)}$$

$$V_1 = \frac{q}{d_1} \quad \text{(3-53)}$$

$$V_2 = \frac{q}{d_2} \quad \text{(3-54)}$$

Where, E_{f1} and E_{f2} are specific energies at section 1 and 2 respectively, (m)
V_1 and V_2 are corresponding flow velocities at same section, (m/s)

Pre- and post-jump flow depths are interrelated with the following equation:

$$d_2 = \frac{d_1}{2}\left(\sqrt{1+\left(8q^2/g\,d_1^3\right)}-1\right) \text{ or } d_2 = \frac{d_1}{2}\left(\sqrt{1+8Fr_1^2}-1\right) \quad \text{(3-55)}$$

Based on such computed flow depth values at pre- and post-jumps, one can estimate jump length as:

L_j = 5 to 6*(d_2-d_1) …………………………………………………………….. (3-56)

3.8.8.4 Determination of location of the jump

Hydraulic jump can occur only when the Froude number exceeds 1. When the Froude number is near one, the jump is very weak and somewhat gradual such that it might not even be able to see it is "jumping".

Thus, to figure out where the jump occurs it needs to know when the Froude Number transits from less than one to more than one. To help this, the following equation is extremely useful for modeling open channel flow:

$$\frac{dy}{dx} = S\frac{1-\left(\frac{y_n}{y}\right)^3}{1-\left(\frac{y_c}{y}\right)^3} \qquad \text{(3-57)}$$

Where, y represents the depth of flow in a riverbed at any point,

x is the distance along the flow,

S is slope of riverbed (positive when flowing downhill, negative, when flowing uphill),

y_c is the critical depth of flow corresponding to Fr =1, and

y_n is the normal depth, the depth of flow corresponding to flow that is balancing the frictional losses with gravitation gains.

As a general rule, it is easiest to work upstream i.e. from d_2 towards d_1. If we try to work downstream from an initial guess, we'll end up with non-physical flows unless we happen to guess perfectly.

In general, we can model the flow by going upstream from the subcritical section and downstream from the supercritical section. Then having modeled the supercritical section we can plot vs position of the depth of flow that would result from a hydraulic jump at that location. Any time that plot intersects the subcritical plot which is a location where a hydraulic jump could be stable.

3.8.9 Selection of appropriate stilling basin

3.8.9.1 General

The objective of all stilling basin designs is to make the highly turbulent region of flow that occur on the stilling basin at all levels of discharge and to ensure that the flow leaving the downstream end of the basin is subcritical and will not cause excessive scour downstream. The most important parameter required to set stilling basin is sequent depth of a hydraulic jump. There are numerous studies carried out on the study of shape, dimension and details of stilling basins. One of the latest and most comprehensive and being widely practiced investigations was conducted by USBR which is presented here briefly.

However, if it is supposed to accommodate the jump and match exit gradient criteria by such stilling basin alone, it results in costly and long massive structure. Although not adequate, the length of apron from exit gradient could be reduced by introducing a cut off walls as long as the jump remain within the apron. Such mechanism of controlling location of jump by introducing energy dissipater results in cost reduction. Thus, provision of additional structures need to be made within the stilling basin so as to optimize length and hence size of the structure by accommodating and hence dissipating incoming energy from hydraulic Jumps. Details of this have been discussed in section 2.6.2.

Table 3-8: Summary of types of hydraulic jump

Types of Jump	Froude number	Jump Characteristics	Energy dissipation, %
Strong jump	$F_r > 9$	Rough jump, lots of energy dissipation	5
Steady jump	$4.5 < F_r < 9$	Considerably energy losses	20
Oscillating jump	$2.5 < F_r < 4.5$	Unstable oscillating jump; production of large waves of irregular period	20-40
Weak jump	$1.7 < F_r < 2.5$	Little energy loss	45-70
Undular jump	$1.0 < F_r < 1.7$	Free-surface undulations d/s of the jump; negligible energy loss	70-85

Source: Types of Hydraulic Jumps (rough classification), Chow 1973

Table 3-9: Selection of energy dissipaters for different range of flows

Types of basin	Consideration/ Application limits
USBR Type I	Wide range of F_r, V< 15 m/s; Long basin, simple construction
USBR Type II	For V>15m/s; Long basins, with chute blocks
USBR Type III	For F_r> 4.5, V < 15 m/s; Short basin, but complicated by floor and chute blocks
USBR Type IV	For F_r between 2.5 - 4.5, V < 15m/s; short basin, but complicated by floor and chute blocks
Vlugter	For canal falls of drop<4m; Simple to design and construct; Suitable for masonry construction
SAF	For small low head structures; Short basin, but complicated by floor and chute blocks
Impact block	For drops<2m; Generally economical; Short basin, but complicated by floor and chute blocks
Slotted grating	For 2.5<Fr<4.5; for small structures; complicated by grating
Baffled spillway	No tail water requirement; Complicated by blocks; economical
Bucket	Needs more tail water than jump type basins; Good foundation and protection required, in view of scour hole formation

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

3.8.9.2 Fixing stilling basin floor level

The floor of the stilling basin must be set to a sufficient depth below the minimum tail-water to confine the hydraulic jump to the basin at all times. If the setting of aprons/stilling basins is too high to ensure hydraulic jump formation, instead of a hydraulic jump dissipating the energy over a relatively short reach, a gradual thickening of the flow results in dissipating the energy over a relatively long reach. Thus, the typical stilling basin/apron length selected shall be set shorter to provide protection from these gradually varying flows by depressing it and introducing additional structures like chute blocks, baffle blocks, end sill, etc.

Typical small scale diversion weir stilling basins usually have lower Fr (generally between 2.5 and 4.5), and the USBR “Design of Small Dams” recommends a type IV basin. Flows for these basins are considered to be in the transition flow stage because a true hydraulic jump does not fully develop. Stilling basins that accommodate these flows are the least effective in providing satisfactory dissipation because the attendant wave action ordinarily cannot be controlled by the usual basin devices. Waves generated by the flow phenomena will persist beyond the end of the basin and must often be dampened by means apart from the basin.

USBR thus recommends that “because of the tendency of the jump to sweep out, and as an aid in suppressing wave action, the water depths in the basin should be about 10 percent greater than the computed conjugate/sequent depth". Also they indicate that higher Fr stilling basins and better hydraulic jump performance can be facilitated by selecting wider structure/basin widths. The USBR chart for jump/basin length for type IV basins (generally with chute blocks which are probably not practical for typical small scale weir diversion projects) indicates basin lengths of about 5 to 6d_2 for Froude numbers between 2.5 and 4.5.

In general, to design the stilling basin we need to estimate the depth of flow, velocity of flow and Froude number at the upstream end of the stilling basin. This is done by undertaking an energy balance between the top of the weir, where flow conditions are governed by the weir equation, and the downstream toe of the weir where supercritical flow is occurring (neglecting losses due to friction and turbulence in between). This energy balance also called Bernoulli's equation is expressed as follows:

$$E=d_1+ Z +V_1^2/2g =d_2+V_2^2/2g \qquad (3\text{-}58)$$

Where, E is the total energy at the upstream, (m)

d_1 is flow depth at entrance to the jump, (m)

Z is datum level, i.e. reference elevation (in this case it is RBL, m);

V_1 is velocity of flow entering the jump, (m/s);

d_2 is sequent depth, (m)

V_2 is velocity of flow leaving the jump, (m/s).

The left hand side of the above energy balance equation can be solved at first hand since all parameters are known, the right hand side of the equation however has to be solved iteratively or by goal seek for d_2 by varying d_1 by assume floor level of the d/s apron/stilling basin (i.e. fist assume floor level then compute equation on LHS till it equals RHS by varying d_1).Thus the general procedure for designing stilling basin is as outlined below:

- Determine the pre-jump and post-jump water depth, d_1 and d_2,
- Determine the pre-jump velocity V_1 and Froude Number, Fr_1,
- Select suitable basin from the following types of Jump and recommended Stilling Basis.
- The floor of the basin should be set to give a tail water depth to be at least 10% greater than the sequent depth D_2 given by the equation (3-55).

And rearranging the above relationship yields:

$$d_1 = \frac{q}{\sqrt{2g(E-d_1)}} \qquad (3\text{-}59)$$

Primarily, a drop in the floor level between upstream and downstream is assumed. Then, using calculated E and q, the value of d_1 is determined by iteration. Once the value of d_1 determined, the Froude Number calculated and if it is within the acceptable range for the proposed basin type proceed to determination of d_2 using equation (3-55). The level of d_2 should be compared to the tail water level at design discharge and if it is above the tail water level, modify the design until d_2 is sufficiently below the tail water level.

3.8.9.3 Estimation of length of stilling basin

The length of the downstream impervious apron or stilling Basin is determined considering the hydraulic jump length and the available exit gradient at the end of the apron. The apron thickness varies along this length from a maximum at the start of the hydraulic jump in the stilling basin/apron to a minimum at its end. In practice the length requirement for jump is small compared to the length requirement for safe exit gradient especially there is only cutoff (no sheet pile).
To ensure safety against piping, Lane's Weighted Creep Theory states that "The sum of the vertical creep lengths, plus one third the sum of the horizontal creep lengths must be greater than the differential head across the structure times Lane's creep coefficient (C)" which is given in the table 3-7.

The downstream cistern i.e. solid apron should be long enough to accommodate the jump and is 5 to 6 times jump height, as given in equation (3-56).

Level of stilling basin is fixed either by energy method (or trial and error) or conventional method. Energy method involves assuming level of stilling basin and applying Bernoulli's/Energy equation shown in equation (3-58) at two sections: on the u/s and on the d/s of weir body and solve for initial depth of flow entering the jump such that the difference between tail water depth and sequent depth is within allowable range of 20 to 40%. Thus, data required for this method are:

i. **Data required on the upstream side:**
   - Approaching velocity head, H_{av};
   - River Bed Level, RBL;
   - Head of design discharge or Overflow depth, H_d; and
   - Weir height, h

ii. **Data required on the downstream side:**
   - Characteristics of entering hydraulic jump such as d_1 and H_{av1};
   - Assumed level of the basin.

This method also involves calculations based on approximation as well as trial and error methods. It also neglects losses between points/sections and considers similar datum.

The Conventional Method on the other hand, involves experimental formula as has been presented here under:

$$L = 3 * \sqrt{h * F} \quad \text{……………………………………………………………… (3-60)}$$

$$D = 0.5 * \sqrt{h * F} \quad \text{…………………………………………………………….....(3-61)}$$

The required criteria in this case is, $\frac{F}{2} \geq D \geq \frac{F}{3}$ ………………………………………..…..(3-62)

Where; L is length of the basin (horizontal length of the d/s slope of the weir + length of the d/s apron floor), (m)

D is Depth of the stilling basin below RBL, (m)

H_d is Overflow depth, (m)

F is the difference of (u/s Water Level + Velocity Head) – (d/s Water Level)…..… (3-63)

Finally, if both the conditions stated under the two methods are satisfied, then select the shorter length from economic point of view. For this purpose, and from practical experiences, the result of Conventional Method is lower than the Energy Method.

In general, the following steps can be used to fix length of the stilling basin.

i. For the design flood determine the scour depth and determine the cutoff depth at the downstream end of the floor *d* (use the appropriate equation and factors)
ii. Assume floor length *b* (covering the upstream apron, diversion headwork base width and stilling basin)
iii. Determine λ using the above formula
iv. Determine the exit gradient and compare it with the safe exit gradient recommended for the type of foundation materials. If it exceeds modify the cutoff depth and/or floor length until the calculated exit gradient is less than the safe exit gradient recommended for the foundation material.
v. This procedure should loop again and again in order to optimize the cutoff depth and floor length.

Note: The selected basin length should also satisfy piping and exit gradient requirements.

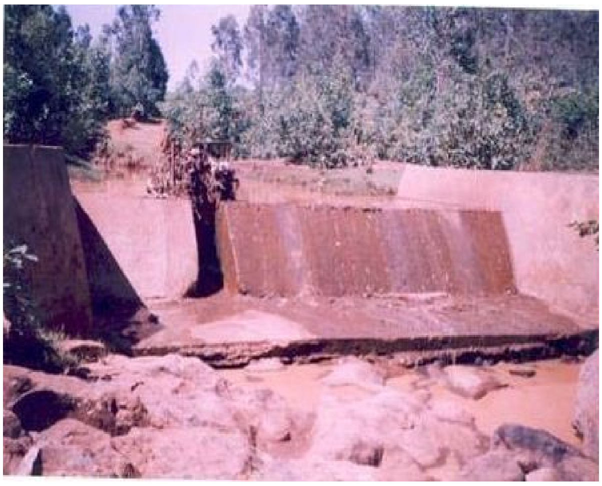


Figure 3-15: Insufficient Basin Length as Resulted in d/s Retrogression (SNNPR)

3.8.9.4 Determination of Thickness of Stilling Basin

The floor of a stilling basin is subjected to uplift pressures resisting the tail water loads and water loads whose magnitudes depend on hydraulic-jump depths. For articulated slabs, such uplift pressure must be resisted by the weight of the slab and the water inside the basin and by anchor bars. Consequently, to know effect of such subsurface flow or seepage which builds up an uplift pressure under the foundation of the structure, especially on pervious foundations, proper seepage analysis should be carried out to enable the designer to evaluate the threat and incorporate necessary measures to defuse it. Three methods of such seepage analysis are discussed in following section.

(a) Bligh's Creep Theory

For the safety of hydraulic structures on pervious foundation, he considered the following two design criteria that should be satisfied.

- The subsoil hydraulic gradient should be less than the permissible value to prevent piping failure.
- The floor should be sufficiently thick to prevent its rupture due to uplift pressure.

For this purposes, Bligh postulated that percolating water follows the outline of the base and foundations of a structure, i.e. water creeps along the bottom structural contour, According to his theory:

- Percolating water creeps along the base profile of the structure, which is in contact with the subsoil. The length of path traversed by the percolating water is called creep length;
- Head loss per unit length of creep called hydraulic gradient is proportional to the length of the creep, H_L/L;
- No distinction is made between horizontal and vertical creep, i.e. he assumed them as if they are equally effective.

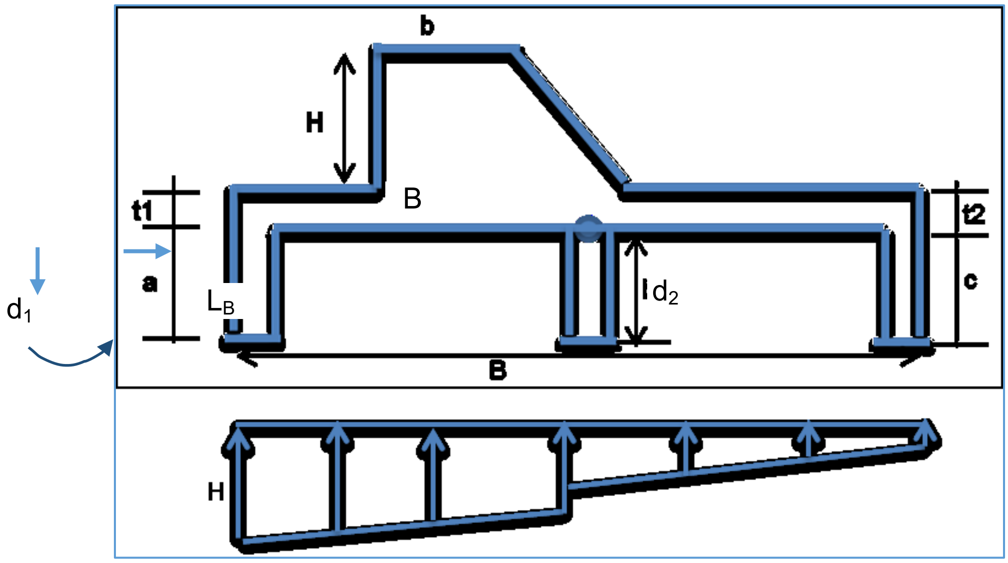


Figure 3-16: Assumed Pressure Distribution under the Base Profile of the Structure

Uplift pressure, h_B at any point B from above pressure diagram is determined as follows:

$$h_B = \frac{H(L_{eq} - L_B)}{L_{eq}} \quad \text{(3-64)}$$

Where, h_B = Uplift pressure along the base i.e. residual head up to point B, (m)
L_{eq} = equivalent creep length according to Bligh's Theory (m)
$= (t_1+2a)+L_1+2b+L_2+(t_2+2c)$ ……(3-65)
L_B = Creep length along the base up to point B, (m)
H = Actual water head, (m)

Table 3-10: Values of Lane's Creep Coefficient, C and Safe Hydraulic Gradient

Type of soil	Value of C	Safe exit gradient	
		Lane's Method (1/C)	Bligh's Method
Very fine sand or silt	8.5	1/8.5	1/18
Fine sand	7.0	1/7.0	1/15
Medium sand		1/6.0	-
Coarse sand	5.0	1/5.0	1/12
Fine gravel		1/4.0	-
Medium gravel		1/3.5	-
Gravel and sand	3.5 to 3.0	1/3.0	1/9
Coarse gravel including cobbles		1/3.0	-
Boulders, cobbles and gravels		1/2.5	-
Boulders, gravel and Sand	3.0 to 2.5	-	1/4-6
Soft clay		1/3.0	-
Medium clay	3.0 to 1.6	1/2.0	-
Hard clay		1/1.8	-
Very Hard clay and Hard pan		1/1.6	

Source: As adopted from MoWR, Garg and Halcrow

For final design of a non-piping structure at the toe of the foundation, the exit gradient, G_e should be less than the recommended safe hydraulic gradient shown in the table above.

$$L = C*H \qquad (3\text{-}66)$$

Where, L is required creep length, (m)
C is creep coefficient

Bligh, then calculated floor thickness from $t = h_B/(G\text{-}I)$ ……………………………..……….. (3-67)

Where, G is specific gravity of floor material;
With safety factor, $t = (4/3)* h_B/(G\text{-}I)$ …………………………………….……………. (3-68)

Required criteria in this case is:

$$\text{Exit gradient } Ge = H/L_{eq} < G_s \qquad (3\text{-}69)$$

This theory is simple but with its limitation in that its exaggerated safety factor results in high value and hence high construction cost.

The design will be economical if the greater part of the creep length (i.e. of the impervious floor) is provided upstream of the weir where nominal floor thickness would be sufficient. The downstream floor has to be thicker to resist the uplift pressure. However, a minimum floor length is always required to be provided on the downstream side from the consideration of surface flow to resist the action of fast flowing water whenever it is passed to the downstream side of the weir.

Bligh's Theory is simple but has the following limitations:

- Bligh made no distinction between horizontal and vertical creep.
- The theory holds well as long as horizontal distance between cut-offs or pile lines is greater than twice their depth.
- No distinction is made between the effectiveness of the outer and inner faces of sheet piles and short and long intermediate piles. However, investigations, later, have shown that the outer faces of the end piles are much more effective than the inner ones. Also intermediate piles of shorter length than the outer ones are ineffective except for local redistribution of pressure.
- No indication on the significance of exit gradient. Average value of hydraulic gradient gives idea about safety against piping. Exit gradient must be less than critical exit gradient (for safety).
- The assumption, loss of head is proportional to creep length is not true and actual uplift pressure distribution is not linear, but it follows a sine curve.
- Bligh did not specify the absolute necessity of providing a Cut-off at the downstream end of the floor, whereas it is absolutely essential to provide a deep vertical Cut-off at the downstream end of the floor to prevent undermining.

(b) Lane's Weighed Creep Theory:

Lane modified Bligh's Creep Theory, after analyzing the foundations of 200 dams worldwide, and stipulated that in computing the creep length, a weighting factor of one third should be applied to the horizontal creep as it is less effective in reducing uplift or differential head. Thus to ensure safety against piping:
“The sum of the vertical creep lengths, plus one third the sum of the horizontal creep lengths must be greater than the differential head across the structure times Lane's creep coefficient *(C)* which is given in the table below”. Thus:

L_c = 1/3*N +V……………………………………………………………..…………………(3-70)

Where, N= sum of all horizontal and all sloping contacts less than 45°.
V= Sum of all vertical contacts and all sloping contacts greater than 45°
L_c = equivalent creep length according to Lane's Theory (m)
= 1/3 * (L_1+L_2)+(t_1+2a+ 2b+2c)……………………………..…………………..… (3-71)

Lane’s principles are:

- Always, the exit gradient must be less than the safe gradient.
- The horizontal creep is less effective in reducing uplift pressure than vertical one, hence N/3 is taken.

To determine the thickness of the apron both dynamic and static case should be considered. The bottom parts of the apron will generally require larger thickness when static case is selected, but the top part of the apron (the toe section) will have larger thickness when dynamic case is considered. Therefore, the thickness at any point say A, B and C are calculated from:

$$t = \left[H_{\max}\left(1 - \frac{L_A}{L_c}\right) + \left(TWL - WL_A\right)\right] * \frac{f}{(\gamma_m - 1)}$$ ……………………………………………………………..……(3-72)

Where, t=Thickness of apron at any point A, B, and C where residual pressure head is computed (m)
f = Factor of safety = 1.3
H_{max} = Total seepage head i.e. U/S and D/S differential (maximum) head, m
L_c= Weighted creep length total (m), $L_c = \sum L_v + \frac{1}{3}\sum L_H \geq CH_{\max}$ ……………….(3-73)

L_V =Vertical creep length (m) and
L_H = Horizontal creep length (m)
C = Lane’s creep coefficient (From Table 3-7)

Note: Required condition is: $\boldsymbol{L_c \geq C*H_{max}}$; or $\boldsymbol{(H_{max}/L_c) \leq (1/C)}$ ……………..……………….. (3-74)

(c) Khosla's Theory

To assess the uplift pressure under composite weirs in any hydraulic structure, Khosla (1954) evolved the method of independent variables. In this method, the base of the structure is split up into a number of simple standard forms of known analytical solutions.

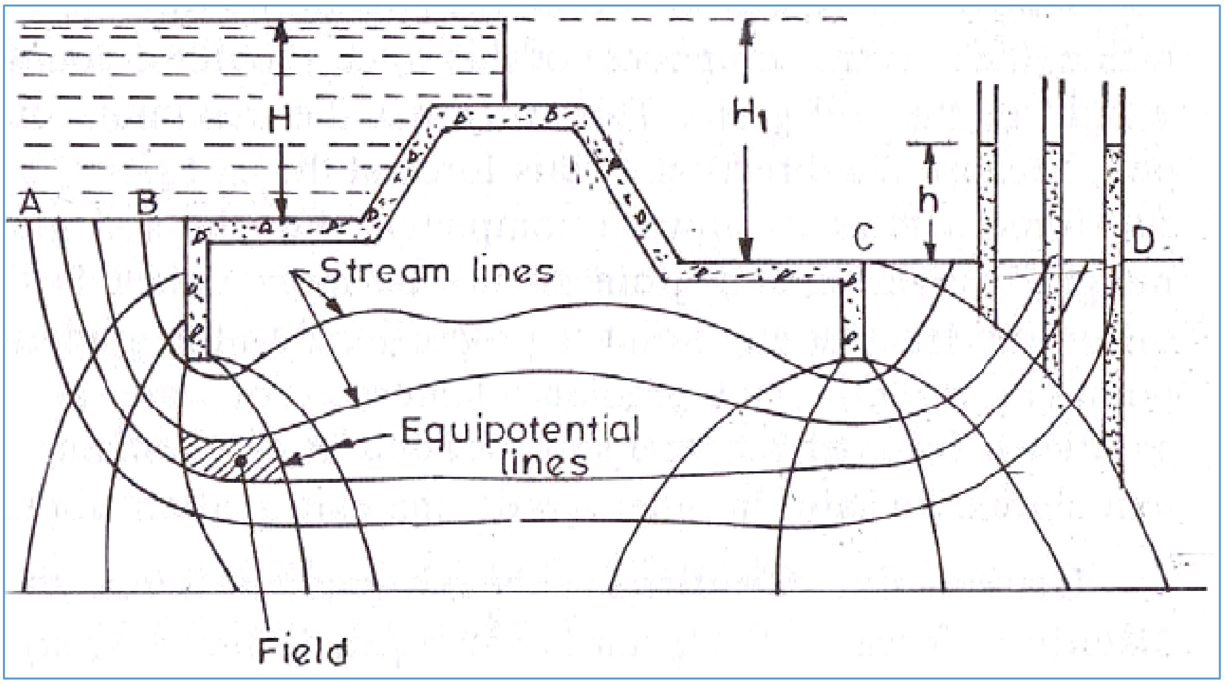


Figure 3-17: Schematic Representation of Khosla's Flow Net

This theory is used to analyze the uplift pressure under any hydraulic structure of regular, irregular or composite shaped foundation. The method takes in to account effects of their shape and thickness.

Base of the structure is divided in to a number of single standard forms of known analytical solutions i.e.:

- Straight horizontal floor of negligible thickness with a sheet pile at either ends;
- Straight horizontal floor of negligible thickness with a sheet pile line at some intermediate position;
- Straight horizontal floor depressed below the bed but with no vertical cut-offs.

The main principles of this theory are:

The seepage water does not creep along the bottom contour of pucca-floor as stated by Bligh, but moves along a set of streamlines. This steady seepage in a vertical plane for a homogeneous soil can be expressed by the 2-D partial differential equation called the Laplacian equation:

$$\frac{d^2\phi}{dx^2} + \frac{d^2\phi}{dy^2} = 0 \quad \text{.. (3-75)}$$

Where, ϕ = Flow potential = K*h; .... (3-76)
K = Coefficient of permeability of soil as defined by Darcy's law and
h = is the residual head at any point within the soil

This equation represents two sets of curves intersecting each other orthogonally. The resultant flow diagram showing both of the curves is called a Flow Net.

The streamlines represent paths along which water flows through the sub-soil. Every particle entering the soil at a given point upstream of the work will trace out its own path and will represent a streamline. The first streamline follows the bottom contour of the works and is the same as

Bligh's path of creep. The remaining streamlines follows smooth curves transiting slowly from the outline of the foundation to a semi-ellipse, as shown in Figure

Treating the downstream bed as datum and assuming no water on the downstream side, it can be easily stated that every streamline possesses a head equal to h_1 while entering the soil; and when it emerges at the downstream end into the atmosphere, its head is zero. Thus, the head h_1 is entirely lost during the passage of water along the streamlines.

Further, at every intermediate point in its path, there is certain residual head, h, still to be dissipated in the remaining length to be traversed to the downstream end. This fact is applicable to every streamline, and hence, there will be points on different streamlines having the same value of residual head h. If such points are joined together, the curve obtained is called an equipotential line.

Every water particle on line AB is having a residual head $h = h_1$, and on CD is having a residual head $h = 0$, and hence, AB and CD are equipotential lines.

The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines. This force (F) has an upward component from the point where the streamlines turns upward. For soil grains to remain stable, the upward component of this force should be counterbalanced by the submerged weight of the soil grain. This force has the maximum disturbing tendency at the exit end, because the direction of this force at the exit point is vertically upward, and hence full force acts as its upward component.

For the soil grain to remain stable, the submerged weight of soil grain should be more than this upward disturbing force. The disturbing force at any point is proportional to the gradient of pressure of water at that point. This gradient of pressure of water at the exit end is called the exit gradient. In order that the soil articles at exit remain stable, the upward pressure at exit should be safe. In other words, the exit gradient should be safe.

This exit gradient is said to be critical, when the upward disturbing force on the grain is just equal to the submerged weight of the grain at the exit. When a factor of safety equal to 4 to 5 is used, the exit gradient can then be taken as safe. In other words, an exit gradient equal to ¼ to 1/5 of the critical exit gradient is ensured, so as to keep the structure safe against piping.

Khosla *et al*. used the method of independent variables and obtained solutions of Laplace equation for a number of simple profiles. This solution is commonly known as Khosla's solution. The following forms of these simple profiles are very useful in the design of weirs and barrages on permeable foundations:

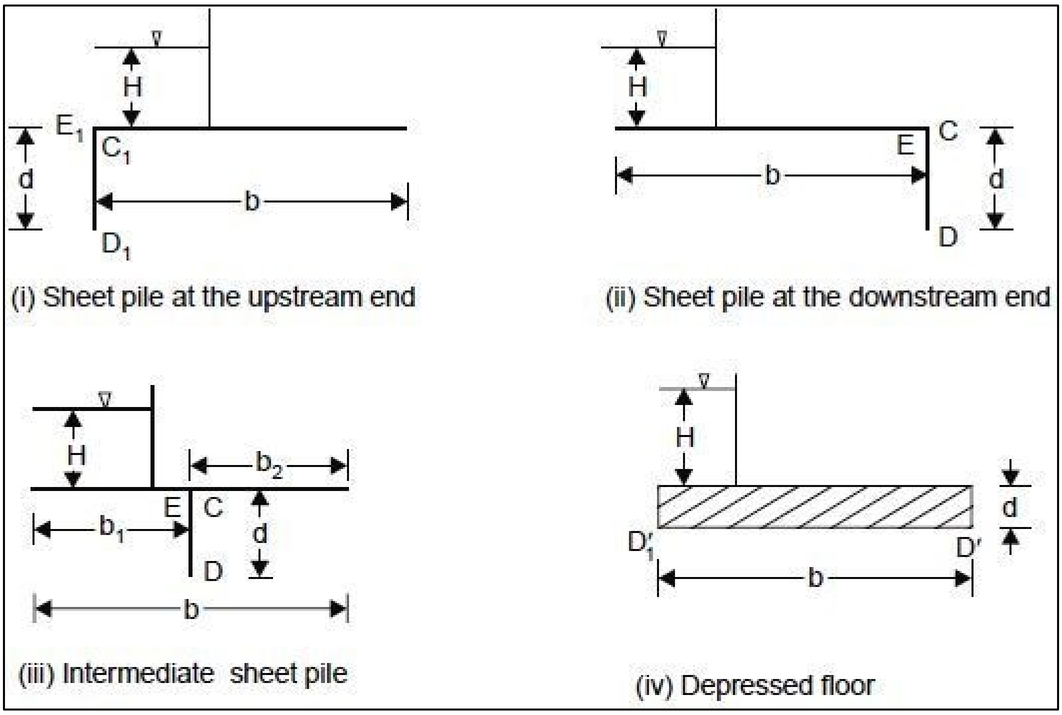


Figure 3-18: Simple Standard Profiles of Weir Floors of Khosla's proposals

For sheet piles at either upstream end or downstream end [as shown in Figure 3-13above]:

$$\phi_E = \frac{1}{\Pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) \quad \text{(3-77)}$$

$$\phi_D = \frac{1}{\Pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) \quad \text{(3-78)}$$

$$\phi_{C1} = 100 - \phi_E \quad \text{(3-79)}$$

$$\phi_{D1} = 100 - \phi_D \quad \text{(3-80)}$$

Where, $\lambda = \frac{1}{2}\left[1 + \sqrt{1 + \alpha^2}\right]$ .... (3-81)

And $\alpha = \frac{b}{d}$ .... (3-82)

For sheet piles at the intermediate point [as shown in Figure 3-13, above]:

$$\phi_E = \frac{1}{\Pi}\cos^{-1}\left(\frac{\lambda_1 - 2}{\lambda_2}\right) \quad \text{(3-83)}$$

$$\phi_D = \frac{1}{\Pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda_2}\right) \quad \text{(3-84)}$$

$$\phi_C = \frac{1}{\Pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda_2}\right) \quad \text{(3-85)}$$

Here, $\lambda_1 = \frac{1}{2}\left[\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}\right]$ .... (3-86)

And $\lambda_2 = \frac{1}{2}\left[\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}\right]$ .... (3-87)

Where, $\alpha 1 = \frac{b1}{d}$ and $\alpha 2 = \frac{b2}{d}$ ………………………………………………………….……..….(3-88)

In the case of a depressed floor [as shown in Figure 3-13 (iv), above]

$\phi_D' = \phi_D - \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$ ………………………………………………………… (3-89)

$\phi_D' = 100 - \phi_D'$ ……………………………………………………………………….…….. (3-90)

Where, ϕ_D and ϕ_E are as given above and $\alpha = \frac{b}{d}$,…………….……………………….……(3-91)

In general, sufficient thickness should be provided in order to counterbalance the uplift pressure. Usually an apron having thickness varies along its length from a maximum at the start of the hydraulic jump (or at location of initial depth) to a minimum at its end. Sometimes floor slabs, strengthened by anchor bars. The design will be considered economical if the greater part of the creep length (i.e. of the impervious floor) is provided upstream of the weir where nominal floor thickness would be sufficient. However, practical conditions do not allow for doing so. In earlier times it had been in use of the following method.

Suppose *h* is the pore water pressure head (ordinate of hydraulic gradient) measured above the top of the floor at a point and introducing a factor of safety of around 1.5, the thickness *t* of the concrete floor at that spot is;

$t = 1.2h$……………..……………………….…………………………… ..(3-92)

In another approach the thickness of the floor is designed for the worst condition anticipated. Here is the practice

The minimum floor thickness *t* follows a stability calculation, i.e. the uplift pressure at location x designated as P_x should be less than the weight of the floor plus the weight of the water above the floor.

$$\gamma_{flr} \times d + \gamma_w \times y_{flr} > P_x \text{……………..………………………….…………….. (3-93)}$$

Where γ_{flr} is the unit weight of the floor in N/m^3,

d is minimum floor thickness in m,

γ_w is the unit weight of water (equal to 9,810 N/m3) and

y_{flr} is the water depth above the floor in m, Px is the uplift pressure in N/m2.

Usually the weight of the water is ignored in computation in order to increase safety.

On the other hand, the maximum net uplift pressure at a location occurs with a condition when the water on the upstream is up to normal water level or crest elevation of the weir and no flow in the downstream (dry). This condition is usually considered as a governing during the design of safe floor thickness. However, the negative pressure developed in the hydraulic jump (standing wave) shall be checked under some circumstance.

Therefore, the floor thickness shall be worked out for the largest of the with-standing wave in the stilling basin and when downstream flood is dry. The thickness of the floor at a point is the ratio pressure and submerged unit weight of the floor material.

The following formula can be used at key location along the length of the floor (say at toe, midway, etc.) to design the floor thickness

$$t_i = \frac{H_i}{SG_f - SG_w} \qquad (3\text{-}94)$$

Where t_i is the minimum floor thickness required at location *i* in m,

H_i is the uplift pressure per unit weight at location *i* in m,

SG_f and SG_w are the specific gravity of floor material and water, respectively.

It is required to determine the floor thickness at several locations on the downstream floor using the available uplift pressure in order to decide on safety of floor thickness.The length of the downstream slab or floor depends on the floor length requirement calculated for energy dissipation and the anticipated exit gradient.

The determination of the seepage pressure can be worked out using a relationship established by Khosla et.al. (1955) as percentage of the available maximum pressure head at the upstream. The relationship is based on Figure 3-16.

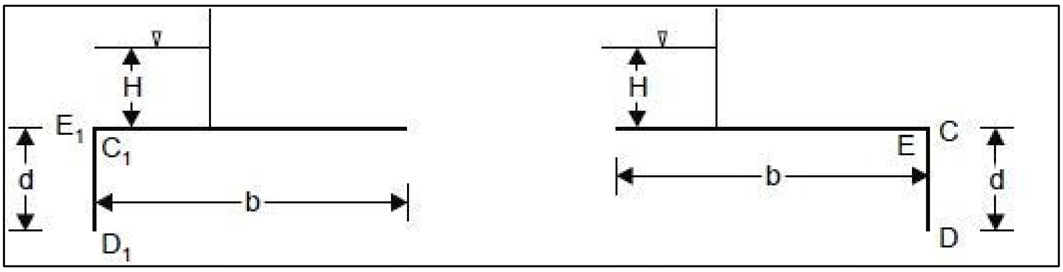


Figure 3-19: Simplified profile at upstream (left) & downstream (right) of diversion headwork

Referring to figure the percentage of the uplift pressure (in terms of head of water) can be computed at the key location below the floor at the downstream as:

$$\phi_C = 0 \qquad (3\text{-}95)$$

$$\phi_D = 100\left[(1/\pi)Cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)\right] \qquad (3\text{-}96)$$

$$\phi_E = 100\left[(1/\pi)Cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)\right] \qquad (3\text{-}97)$$

Where λ can be calculated using the above formula provided by Khosla *et al*. (1955).

The percentage of the uplift pressure (in terms of head of water) can be computed at the key location below the floor at the upstream as:

$$\phi_{C1} = 100 - \phi_E \qquad (3\text{-}98)$$

$$\phi_{D1} = 100 - \phi_D \qquad (3\text{-}99)$$

$$\phi_{E1} = 100 \qquad (3\text{-}100)$$

A correction for uplift pressure due to floor thickness might be required. The correction can be computed at key points on the floor and interpolated linearly over the entire length of the basin floor. The correction for percentage uplift pressure at the end of the floor is:

$$\phi'_E = \frac{(\phi_D - \phi_E)}{d}t \qquad (3\text{-}101)$$

The correction for percentage uplift pressure at the beginning of the floor is

$$\phi'_{c1} = \frac{(\phi_{D1}-\phi_{Ec1})}{d} t \qquad (3\text{-}102)$$

The correction reduces the percentage uplift pressure at the end of the floor by that amount while it increases at the beginning of the floor.

A correction for uplift pressure due to interference of intermediate cutoff can be computed at key points on the floor and interpolated linearly over the entire length of the basin floor using the above stated formula.

3.8.10 Determination of Exit Gradient (GE)

When the upward thrust exceeds a certain value at the exit, piping will occur. Exit gradient is a gradient of pressure of water at the exit end of structure. This exit gradient is said to be critical, when the upward disturbing force on the grain is just equal to the submerged weight of the grain at the exit. When a factor of safety equal to 4 to 5 is used, the exit gradient can then be taken as safe. In other words, an exit gradient equal to ¼ to 1/5 of the critical exit gradient shall be ensured so as to keep the structure safe against piping.

When upward thrust of seepage flow passing beneath a structure is greater than submerged weight of the soil resisting the upward thrust on the d/s side of end cut off wall, piping will occur and bed material will be washed upwards and into river flow. Two popular methods for determining GE are: flow net which may be plotted by drawing or the use of electrical analogue methods and Khosla's approximation. But commonly Khosla's approximation is adopted for its easiness.

Khosla determined that for a standard form of structure with a floor length (b) and vertical cut-off (d), the exit gradient at the downstream side is given by:

$$G_E = \frac{H_{max}}{d} * \frac{1}{\pi\sqrt{\lambda}}, \qquad (3\text{-}103)$$

Where, $\alpha = \frac{b}{d}$ and $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ ..... (3-104)

G_E is exit gradient
H_{max} is maximum pressure head under Dynamic Case and/or Static Case
b is total length of impervious floor, m
d is depth of downstream Cut-off, m
λ is a number to be computed from equation above,
α is a number to be computed from equation above,

Critical Exit Gradient, CEG can be expressed mathematically as CEG = (S - 1) (1 - n) where S is the specific gravity of the soil and n is its porosity. When critical exit gradient is divided by factor of safety, it gives the safe exit gradient (SEG). For example, for ordinary alluvial type of soil, S = 2.6 and n = 0.4. If a factor of safety of 3 is adopted then SEG = CEG/SF = 0.32.

Note: To keep the exit gradient to a safe value, the depth of Cut-off (D) and the flow length are to be suitably adjusted. It may be required to increase the downstream floor length than needed, from hydraulic considerations. If the jump is not formed, this length can be adopted as 6 times the downstream water depth. Thus, the total floor length is the total of impervious floor consisting of upstream floor, upstream glacis (if any), downstream glacis, downstream stilling basin and end sill. It need to satisfy the requirements of exit gradient, scours as well as economy.

The above equation has been graphically established by Khosla (Refer appendix -III). A graph based on Khosla's theory is placed to indicate the correlation between the floor length, (b) Cut-off depth and (d) i.e. α and $1/(\pi\sqrt{\lambda})$ to determine exit gradient figuratively. By adopting suitable ratio of b and d, i.e. α, safe value of exit gradient can be achieved and ready from the graph.

Table 3-11: Recommended values of Khosla's safe exit gradient

Type of soil	Values of Khosla's Safe Exit Gradient
Shingle	0.25 to 0.20
Coarse sand	0.20 to 0.17
Fine sand	0.17 to 0.14

In general, Khosla's procedure for analyzing uplift pressure under a structure consists:

- Splitting up the foundation into standard forms
- Determining the pressure as a percentage of the water head at the key points. (Junctions of the floor and the pile; bottom points of pile; and bottom corners in case of depressed floor).

The profiles are then corrected for:

- Mutual interference of pile
- Floor thickness
- Slope of floor

Box 3-5:

Worked Example-5*: Referring to the previous worked examples 1-3, design bed level, length and thickness of the stilling basin floor for the specified headwork site.*

Solution*: From Bernoulli's equation, equating energy at sections '1' on the u/s and '2' on the d/s of weir body, we can develop the equation: $RBL+H_d+h+H_{av1} = Z+d_1+H_{av1}$, i.e. $E_{f1}=E_{f2}$, where Z is level of the stilling basin floor.*

Now, assume floor level of the d/s apron i.e. stilling basin is to be 0.2m below RBL. From $H_{av1}=V_1^2/(2g)$ and $V_1=q/d_1$; $d_1+q^2/d_1^2/2g=3.498$. Thus by goal seek or trial and error, varying value of d_1 and solving this equation till RHS and LHS equals gives d_1= 0.49m and consequently, V1=7.72m/s; H_{av1}= 3.04m; F_{r1}=3.53; d_2=1.446m from equation (3-24) but deducting it from assumed drop of 0.2m gives updated d_2=1.25m; $V_2=q/d_2$=2.6m/s; H_{av2}= 0.34m; F_{r2}=0.69; d_3=TWD=0.98m from rating curve; $V_3=q/d_3$=3.86m/s and H_{av3}=0.76m. Now, difference between TWD and d_2=0.3m, thus it lies within allowed range of 0.2 to 0.4 and hence the assumed stilling basin level is acceptable.

Length of the stilling basin is given by L_j =5 to 6(d2-d1) =6*(1.446-0.49) =5.76m, say 6m.*

Box 3-6:

Worked Example-6*: Determine the percentage pressures at various key points in Figure below. Also determine the exit gradient and plot the hydraulic gradient line for pond level on u/s and no flow on downstream.*

Table 3-12: Values of Khosla's corrections for standard slopes

Slope (Horizontal :Vertical)	Proposed Correction factor
1:1	11.2
2:1	6.5
3:1	4.5
4:1	3.3
5:1	2.8
6:1	2.5
7:1	2.3
8:1	2.0

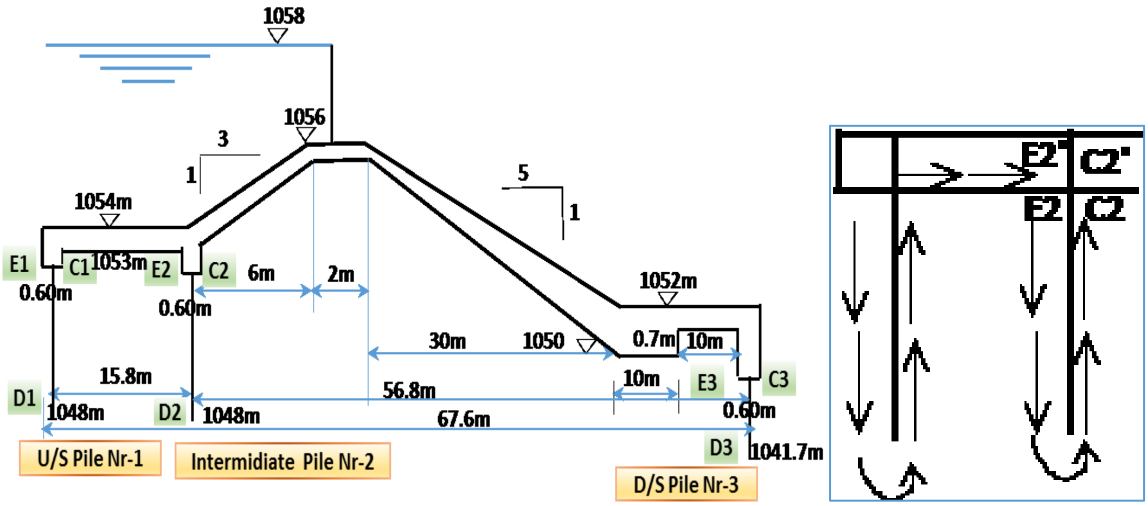


Figure 3-20: Schematic Cross Section of Glacis Weir for Khosla's Flow Net Analysis

***Solution**: These analysis have been calculated and presented in detail as follow.*

(1) For Upstream Pile Line No. (1)

Total length of the floor = b = 68 m

Depth of u/s pile line =d= 6.0

α=b/d= 11.27

$1/\alpha$= 0.089

From the curve Plate, ϕ_{c1} = 71%

From same curve, ϕ_{d1} = 80%

These values of ϕ_{c1}andϕ_{d1} must be corrected for three corrections as follow

Corrections for ϕc1

(a) Correction at C1 for Mutual Interference of Piles. ϕc1 is affected by intermediate pile Nr-2.

$$\boxed{\text{Correction} = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)}$$

Where, D = Depth of pile Nr-2= 5.0

d=Depth of pile Nr-1 = 5.0

b' = Distance b/n two piles = 10.8 m

b = Total floor length = 68 m

Thus, Correction 1.91%

Note: Since the point C1 is in the rear of direction of flow, the correction is + ve.

:. Correction due to pile interference on C1 = 1.88% (+ve)

(b) Correction at C1 due to thickness of floor.

Pressure calculated from curve is at C1', but we want pressure at C1. Pressure at C1 shall be more than at C1' as direction of flow is from C1 to C1' as shown; and hence, the correction will be +veand

Thus, Correction 1.50% (+ve)

(c) Correction due to slope at C1 is nil, as this point is neither situated at start nor at end of a slope.

:. Corrected ϕc1 = 71 % + 1.88% + 1.5% = 74.41%

Hence, corrected ϕc1 = **74.41% Ans**

and ϕD1 = 80%

(2) For Intermediate Pile Line Nr-2

d = 154.00-148.00	6.0 m
b =	57.0 m
α= b/d =	9.5
Using curves of Plate shown in Appendix IV, we have b1 in this case	11.4
α= b/d =	0.200
Thus, 100%-b1/b=	0.800
ϕc for a, base ratio of 0.712 and α = 9.5	30%
Thus, ϕE1 = 100%-30% =	70%
For a base ratio 0.298 and α = 9.5; ϕc2	56%
ϕd2 for a base ratio of 0.702 andα = 9.5; =	37%
Thus, ϕD2 = 100%-37%	63%

Corrections

for ϕE2

(a) Correction at E2 for sheet pile lines Nr-1 will affect pressure at E2and since E2 is in the forward direction of flow, this correction shall be -ve. The amount of this correction is given as :

$$\boxed{\text{Correction} = 19\sqrt{\frac{D}{b}}\left(\frac{d+D}{b}\right)}$$

Where, values of these parameters are computed as above

But in this case correction is negative 1.91%

$$\boxed{\text{Correction} = \left(\frac{Obs\phi_{E2} - Obs\phi_{D2}}{Dis\,between\,E2D2}\right) * Thicknes}$$

Thus, Correction at E2 due to floor thickness .17%

Since the pressure observed is at E2' and not at E2, and by looking at direction of flow, it can be stated easily that the pressure at E2 shall be less than that at E2', hence, this correction is negative.

:. Correction at E2 due to floor thickness 1.17% (-ve)

(c) Correction at E2 due to slope is nil, as the point E2 is neither situated at start of a slope nor at end of a slope.

Hence, corrected percentage pressure at E2 **66.92%** **Ans**

Corrections for $\Phi c2$

(a) Correction at C2 due to pile interference. Pressure at C2 is affected by pile Nr-3 and since point C2 is in the back water in the direction of flow, this correction is +ve. The amount of this correction is given as:

$$\text{Correction} = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

Where, D = Depth of pile Nr-3= the effect of which is considered below the level at which interference is desired 11.3

d=Depth of pile Nr-2, effect on which is considered = 5.0

b' = Distance between pile-2 and pile-3= 56.8 m

b = Total floor length = 57 m

Thus, Correction 2.42 % (+ve)

(b) Correction at C2 due to floor thickness. From Fig. above, it can be easily stated that the pressure at C2 shall be more than that at C2' and since observed pressure is at C2',
this correction shall be +ve and its amount is same as was calculated for the point E2, thus =1.17%(+ve)

(c) Correction at C2 due to slope. Since the point C2 is situated at the start of a slope of 3:1, i.e. an up slope in the direction of flow; the correction is negative.

Correction factor for 3:1 slope from Table= 4.5

Horizontal length of the slope = 6.0

Distance b/n two pile lines b/n which sloping floor is located 56.8 m

:. Actual correction = 0.48% (-ve)

Hence, corrected $\phi c2$ **59.11% Ans**

(3) Downstream Pile Line

d= 152.0-141.7 = 10.3

b = Total floor length = 57 m

$1/\alpha$=1/ b/d = 0.181 m

Then from curves of Plate shown in Appendix IV, we get:

$\phi D3$ = 32%

$\phi E3$ = 38%

Corrections for α_{E3} =

(a) Correction due to piles. Point E3 is affected by pile Nr-2, and since E3 is in the forward direction of flow from pile Nr-3, this correction is negative and Its amount is given by:

$$\text{Correction} = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

Where, D=Depth of pile Nr-2, effect of w/h is considered 2.7 m

d= Depth of pile Nr-3, effect on which is considered 9.0
b' = Distance between piles = 56.8 m
b = Total floor length = 57 m
Thus, correction = 0.85% (-ve)

(b) Correction due to floor thickness
It can be stated easily that pressure at E3 shall be less than at E3' since pressure observed from curves is at E3' this correction shall be -ve
and equals: 0.76% (-ve)
(c) Correction due to slope at E3 is nil, as the point E3 is neither situated at start nor at end of any slope 0%
Hence, corrected $\phi E3$ = **36.39% Ans**

Table 3-13: Summary of corrected pressures at various key points

Upstream Pile Nr-1		Intermediate Pile Nr-2		Downstream Pile Nr-3	
$\phi E1=$	100%	$\phi E2=$	66.92%	$\phi E3=$	36.39%
$\phi D1=$	80.0%	$\phi D2=$	63.0%	$\phi D3=$	32%
$\phi C1=$	74.41%	$\phi C2=$	59.11%	$\phi C3=$	0%

3.8.11 Fixing upstream and downstream cut off

Cut-off is a wall used for controlling piping in pervious foundation material. The required depth of this wall to satisfy the safe exit gradient can be calculated once foundation type and dimensions of the structure are known. If the required depth cut-off is greater than can be constructed, two or more Cut-offs of reduced depth may be used. However the space between the two Cut-offs should not be less than 6/5 times the Cut-off depth, otherwise the full benefit of the multiple Cut-off will not be obtained. The factor 6/5 originates from the requirement that the weighted creep length along the short path between the tips of the Cut-offs should at least equal the weighted creep length along the contact line. In practice, a Cut-off spacing of two or more times the Cut-off depth is generally used, but for most typical small scale diversion weirs this is not a problem.

A downstream erosion Cut-off is usually required to protect the structure from downstream retrogression. When a reverse filter with a drain pipe is not included at the end of the downstream hydraulic jump stilling basin/apron, this will also act as a seepage Cut-off to increase the seepage path length and assist in protecting the structure from a piping failure. An upstream Cut-off may also be necessary to increase the seepage path and protect against a piping failure.

For a diversion weir not constructed on bedrock, the Cut-offs will not only be required to resist seepage and piping, but will probably also be required to ensure that the structure will not slide. As a general rule, a typical small scale diversion weir with upstream and downstream Cut-off depths at least 50 % of the weir height will usually be adequate with respect to sliding resistance.

Bottom level of a weir body is fully dependent on condition of bed material on which the structure rests. If it rests on rocky bed material, then only key wall is required to tie or create bondage between the structure and the bed rock. However, if the bed material is other than bed rock, then bottom level of a weir body is fixed based on the requirement of cutting off piping effects under that structure.

To determine bottom level of the weir body, we therefore need to determine vertical cut-offs that need to be fixed at the upstream and downstream ends of the weir to safeguard against scouring and piping effects. Intermediate cut-offs are usually provided at the ends of upstream and/or the downstream slopes of the impervious floor based on the length required to dissipate subsurface hydraulic/pressure head and are useful in protecting the main structure against sliding too.

The depth of cut-offs should therefore be such that the structure's bottom level is lower than the level of possible flood scour at that section. In addition, the downstream cut-off should also be sufficient to reduce the exit gradient within safe limits, which is decided by the sub-surface conditions.

At the outflow from the stilling basin, there remains a certain proportion of energy in the flow that scours the downstream of the basin. The scour holes so formed may progress towards the structure end and results in structural failure. Such failures can be prevented by providing piles or cut-off at u/s and d/s ends of the impervious floor, by extending below the calculated scour level. This normal scour depth below High/Design Flood Level (HFL), R i.e. depth of normal scour is given by the Regime scour depth method developed by Lacey's equation. Accordingly, if the waterway provided is less than the regime width, the regime scour depth can be:

(R), $R = 1.35 \times \left(\frac{q^2}{f}\right)^{1/3}$ ……………………………………………………………… (3-105)

Where, R = Hydraulic mean depth or scour depth below HFL, m

q = Unit discharge or discharge per meter length, = Q/b, m ……………… (3-106)

f = Lacey's silt factor, for the stated bed material = $1.76\sqrt{d_{50}}$ …………….. (3-107)

d = Average river bed material particle/grain size, mm

To fix bottom levels of cut-offs, the scour depth, R is multiplied by a factor of safety ranging from 1.25 to 1.5 and 1.75 to 2.0 for upstream cut-off and downstream cut-off depths respectively.

Upstream Cut-off level = Upstream HFL - I.5R……………………………………… (3-108)

Downstream Cut-off level = Downstream HFL – 1.75R…………………………… (3-109)

Note: Hydraulic mean depth (=A/T) is different from Hydraulic mean radius (=A/P) and mean flow depth (d) as indicated in following sketch (Where T is top width of water surface, and P is wetted perimeter). Both hydraulic mean depth and hydraulic mean radius are the same for rectangular cross section. However, for other types of sections, they are completely different.

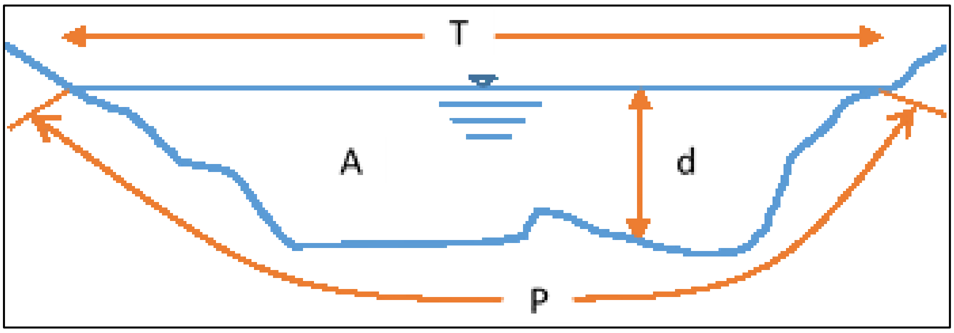


Figure 3-21: Hydraulic mean depth, hydraulic mean radius and mean flow depth

Table 3-14: Lacey's silt factor "f"

Type of Reach	Mean value of "f"
Large boulders and shingle	20
Boulders and shingle	15
Boulders and gravel	12.5
Medium boulders, shingle and sand	10.0
Gravel and bajri	9.0
Coarse gravel	4.75
Coarse bajri and sand	2.75
Heavy sand	2.0
Fine bajri and sand	1.75
Coarse sand	1.5
Medium sand	1.25
Standard silt	1.0
Medium silt	0.85
Very fine silt	0.6
Fine silt	0.4
Clay	5.0

Source: Weir Design Manual, Halcrow-ULG, 1988

Box 3-7:

Worked Example-7*: If investigation of the river bed material mentioned in above examples indicate coarse gravel sand of average particle size 7.26mm (i.e. diameter of 50% of particle size distribution), then how much is the hydraulic mean depth of the river at the selected site and* bottom levels of upstream and downstream cut-offs*?*

Solution*: Unit discharge, q = Q/b = 24.84/6.6= 3.76; Lacey's silt factor for coarse gravel material from table above is selected to be 4.75 or f=1.76*SQRT(7.26)=4.75. Accordingly, R = 1.94m; assuming a* factor of safety of 1.5 for upstream and 1.75 for downstream cut-offs respectively, bottom level of u/s cut-off = u/s HFL – 1.5R= 1432.782m, but this gives u/s $d_{Cut\text{-}off}$ of 1432.782-1433.475= (+) 0.3m, +ve value implies it is not required but for the sake of structural stability take u/s $d_{Cut\text{-}off}$ = 1.0m. Similarly, bottom level of d/s cut-off = d/s HFL – 1.75R= 1430.050m implying 1430.050-1433.475= (-) 2.425m, thus take 3m for bottom level of d/s cut-off i.e. 1429.475m.

3.8.12 Design of protection works

3.8.12.1 General

To maintain normal function of a weir for its intended design period, additional protection works are necessary based on stability conditions prevailing around the structure. These protective works are required both on the u/s and d/s of a weir depending on nature of foundation condition to prevent possibility of a scour hole moving close to the u/s or d/s Cut-offs and undermining the structure. These works are required in addition to impervious floors. Thus, provision of sufficient Cut-offs are enough for small structures, and remaining works shall be provided only in cases where the nature of the river is erosive as well as the river bed material is of loose/alluvial.

These protection work could be in the form of concrete block or stone riprap. Sometime, both the concrete block and the stone riprap might be used and usually the concrete block protection is provided next to the impervious floor followed by the stone riprap. The protection work should be provided throughout the whole contact surface of the river or channel. . A toe wall of nominal dimension is often provided in between and at the end of launching apron and concrete blocks so that stability of the protection work improved.

3.8.12.2 Upstream protection works

On the u/s side, the need for such protection works is owing to higher velocities of flow near the structure as a result of draw down. Thus, upstream protection works are required to keep the upstream apron and loose river channel from scouring. Such protection commonly include provision of combinations and/or either of cut off piles, block protection, rip-raps and launching apron as found necessary. The non-launching apron prevents the scour hole travel close to the floor or sheet pile line; whereas launching apron is designed to launch along the slope of the scour hole to prevent further scooping out of the underlying river bed material.

The most commonly used protection is cut-off and riprap in a form of dumped rock or precast concrete block or rock filled gabion (wire container or welded bar enlargement). Cut-off design has been presented in preceding section. However, the minimum length of stone rip-rap on upstream can be estimated by:

$$L_{pu} = 1.25 \text{ to } 1.5 * D_{u/s} \qquad (3\text{-}110)$$

Where, L_{pu} is length of upstream protection work (m),
$D_{u/s}$ scour depth below the river/channel bed on the u/s (m),

3.8.12.3 Downstream protection works

On the d/s side, the need for protection works is due to the turbulent nature of flow as it leaves stilling basin. The downstream impervious floor shall be safe guarded against exit gradient higher than expected and retrogression effects. The minimum length of stone rip-rap on the d/s side is given by:

$$L_{pd} = 1.5 \text{ to } 2.0 * D_{d/s} \qquad (3\text{-}111)$$

Where, L_{pd} is length of downstream protection work (m),
$D_{d/s}$ scour depth below the river/channel bed on the d/s (m),

The protection work usually consists of dry stone pitching whose sizes are fixed based on the anticipated velocity of flow. According to USBR (1987) the relation between velocity of flow and mean stone diameter is expressed as;

$$D = \left(\frac{V}{4.915}\right)^2 \qquad (3\text{-}112)$$

Where, D is Stone size/diameter (m),
V is average flow velocity at a cross-section under consideration, m/s

In actual practice only stones up to 40-50kg can be handled. Generally it is assumed that the stones launch at a slope of 2:1 (H:V).

According to Bligh theory, the total combined length of the downstream impervious floor and protection works (L) is computed under two conditions:

(i) For weir with shutters, it is given by:

$$L = 18 * C * \sqrt{\frac{H_s}{13}} * \sqrt{\frac{q}{75}} \quad \text{(3-113)}$$

(ii) For weir with no shutters, it is given by:

$$L = 18 * C * \sqrt{\frac{H_s}{10}} * \sqrt{\frac{q}{75}} \quad \text{(3-114)}$$

Where, H_s is the seepage head, the difference in water levels u/s and d/s of the weir,
C is coefficient of creep,
q is unit discharge over the weir, $m^3/s/m$

The upstream and downstream apron lengths, can also be computed from (MoWR, 2002):

$$L_u = 2.25 * q^{2/3} - 1.5 * d_{u/s} \quad \text{(3-115)}$$

Where, L_u is apron length on the u/s portion of the weir
q is discharge per meter width of channel;
$d_{u/s}$ is water depth at upstream corresponding to design discharge.

$$L_d = 3.0 * q^{2/3} - 1.5 * d_{d/s} \quad \text{(3-116)}$$

Where, q is as defined above and $d_{d/s}$ is water depth at downstream corresponding to design discharge, i.e. d_3 or TWD.

The length of the downstream impervious floor, L_d is also given by Bligh as follow:

For weir with shutters, it is given by:

$$L_d = 2.21 * C * \sqrt{\frac{HS}{13}} \quad \text{(3-117)}$$

For weir with no shutters, it is given by:

$$L_d = 2.21 * C * \sqrt{\frac{HS}{10}} \quad \text{(3-118)}$$

And the length of the upstream impervious floor, L_u is as:

$$L_u = L - (L_d + B + 2d_1 + 2d_2) \quad \text{(3-119)}$$

Where, B, d_1 and d_2 are as defined in figure 3-16.

In general, the value that can give safe exit gradient, taking economic aspects in to view shall be selected.

The downstream length could be reduced by providing a stilling basin with different energy defusing elements, i.e. chute blocks, friction block, arrow, dented sill, deflector, biff wall ribbed pitch, baffle wall, etc. (as described under section 2.6.4).

Note: When a weir is constructed on rocky foundation, such apron and protection works are not required.

Box 3-8:

***Worked Example-8**: Check safety of a horizontal impervious floor of typical weir cross section shown below from the exit gradient point of view.*

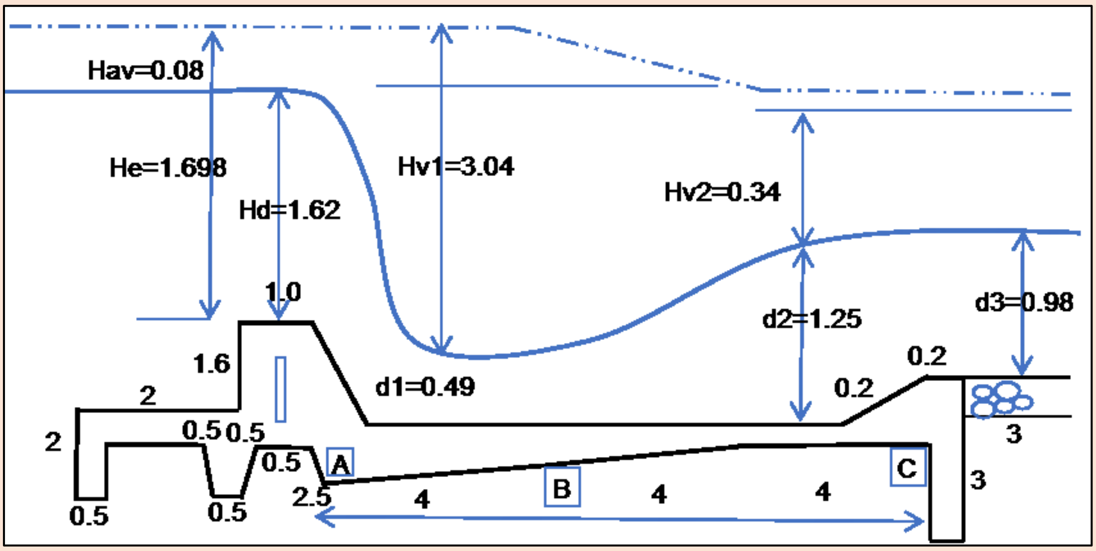


Figure 3-22: Typical weir cross section (for Design Exercise)

***Solution**: From this graph and the above exit gradient equations, the total length of floor, b=18m and depth of downstream Cut-off, d=3m; α = b/d= 6.0; λ = 3.54. Lane's creep coefficient for coarse gravel sand material, C=3.2 (from table 3-7);*

Thus, maximum pressure head under dynamic case H_{max} = U/S water level above WC, H2 - D/S water level above WC, H1 = 2.25m and/or static case H_{max}= Updated WCL - Stilling basin level =1.80m and consequently, CH_{max} =7.19 dynamic case and C*H_{max} = 5.76 for static case.*

3.8.12.4 Launching Apron

Launching apron also called pervious apron or floor is a protection work provided after block protection and inverted filters. It is that part of the protection work laid horizontally over the riverbed on both the upstream and downstream sides of weir, as shown in figure 3-23. It is also provided as continuation of the pitching whenever its sloping face needs to be protected by stone pitching against scour by extending it beyond the toe on the bed in the form of packed stones especially in river training works, as shown in figure 3-23. The launching apron consists of dry stone pitching whose sizes are based on the anticipated velocity of flow and hence the size is determined as indicated above. The stone is layered in either dumped or hand-placed or plated placement method. The stone in hand-placed riprap is aid carefully by hand following a definite pattern, with the voids between the larger stones filled with smaller stones and the surface kept relatively even. The launching apron can be provided in the downstream or both upstream and downstream depending on the condition of the river at the headwork site.

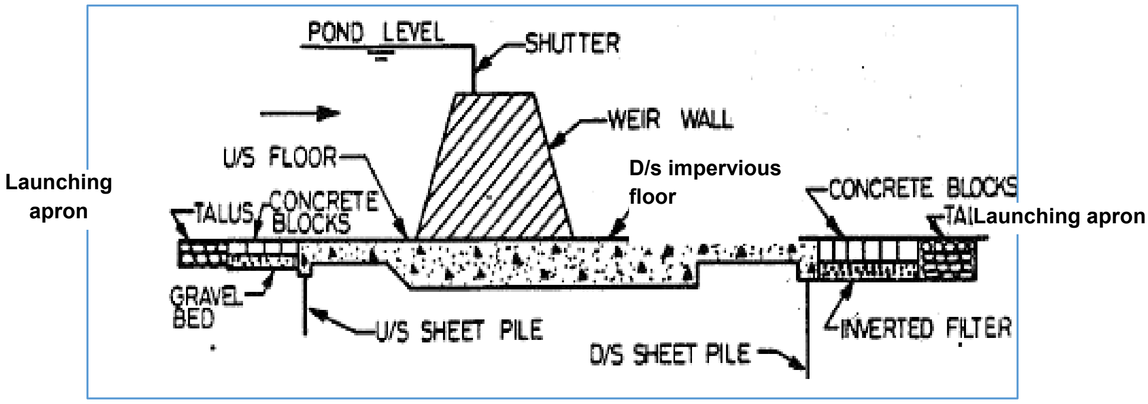


Figure 3-23: Arrangement of launching apron on horizontal floor (S.K. Garg, 2006)

The different aspects to be looked into in the design of launching apron are:

- Size of the stones (from equation 3-94),
- Depth of scour (from Lacey's scour equation),
- Thickness of launching apron,
- slope of launching apron,
- Shape and size of launching apron.

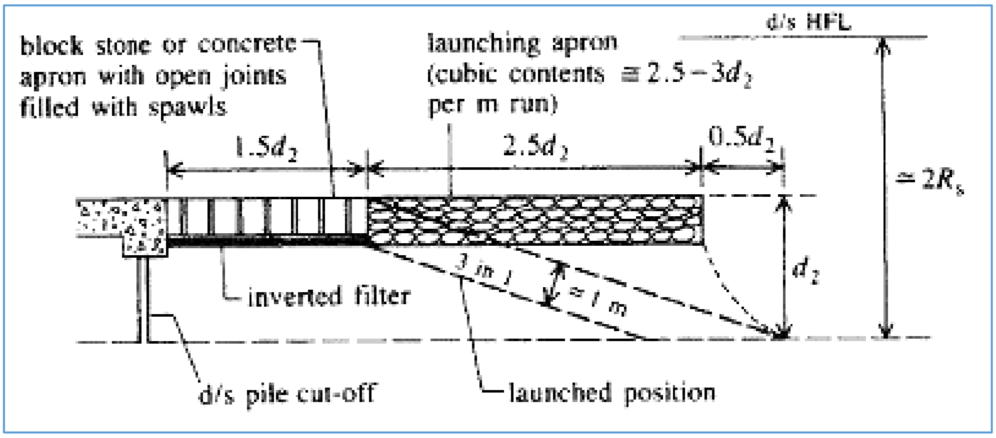


Figure 3-24: Inverted filter and flexible (launching/talus) apron (P. Novak, 2007)

The required size of stone for the apron can be obtained from the curves. In case of non-availability of required size of stones, cement concrete blocks or stone sausages, prepared with 4 mm GI wire in double knots and closely knit and securely tied, may be used.

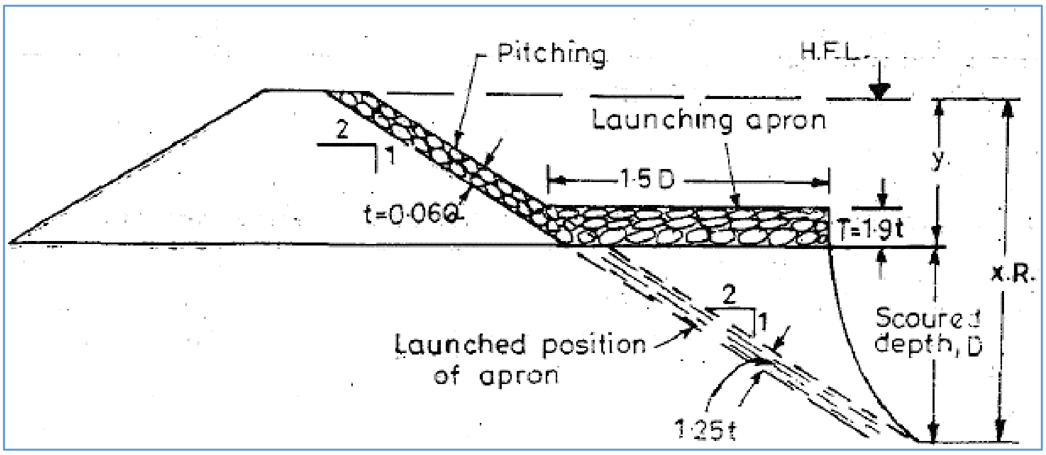


Figure 3-25: Typical arrangement of launching apron at an angle (S.K. Garg, 2006)

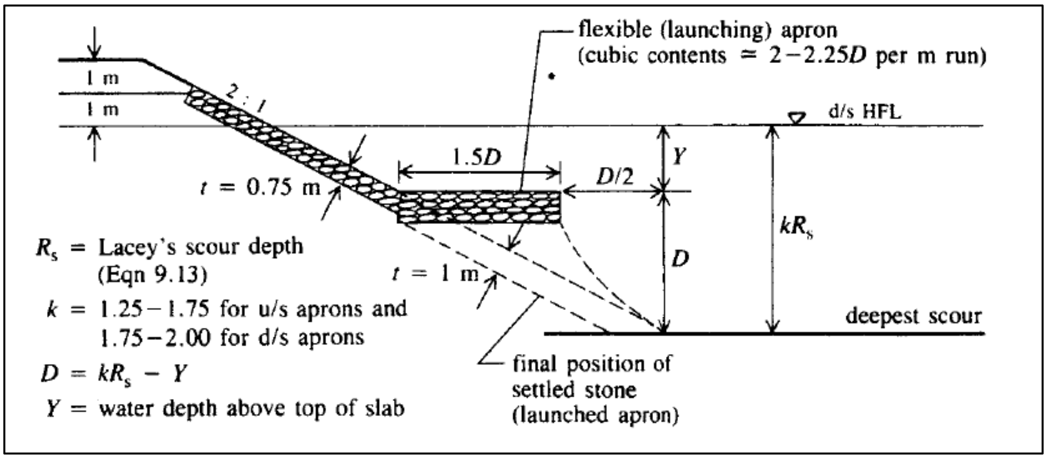


Figure 3-26: Typical arrangement of launching apron on channel bed (Novak et al., 2007)

3.8.12.5 Inverted filter

This protection mechanism consists of layers of selected materials of increasing permeability from bottom to top. The thickness of the inverted filter varies from 0.5 to 1.25 m. To prevent the filter material from dislocation by surface flow, they are weighted down with large size stones or concrete blocks. The blocks are usually 0.9 to 1.2 m thick and are placed with open joints filled with river sand or filter material. It is provided immediately at the d/s end of the impervious floor to relieve the uplift pressure. The length depends on the scour depth R below the river bed and it usually varies from 1.5R to 2R as given by equation:

$$D = XR - Y \qquad (3\text{-}120)$$

Where, R = depth of the deepest scour level below HFL (m),
X = a multiplying factor (varying from 1.25 for u/s to 2 for d/s)
Y = HFL-RBL is depth of river bed or impervious floor below HFL (m)

For details of filter design refer “GL 12: Small Embankment Dams Study and Design Guideline for SSID, section 3.18 and 3.19”

3.8.12.6 Block protection

Block protection is made of concrete blocks and usually provided immediately at the upstream and downstream ends of the impervious floor. It consists of 0.6 to 1.0 m thick stone or concrete blocks laid on 0.4 to 0.6 m thick loosely packed stone. The length of the block protection is usually equal to 1 to 2 times the depth of scour, D. The upstream concrete block protection is laid without open joints between blocks so that uplift pressure relief is not required. However, the downstream concrete block is laid with 70–100 mm open joints filled with sprawls (broken stones) so that the uplift pressure is relieved through the openings. An inverted filter of well-graded gravel and sand is placed under the concrete block in order to prevent the loss of soil through the joints to avoid particle migration.

3.8.12.7 Riprap protection

Riprap protection can be rock/rubble, broken concrete slabs, and preformed concrete shapes. However, rock riprap is the most widely used and most desirable type of revetment. It is compatible with most environmental settings. The term "riprap" alone is most often used to refer to rock riprap.

The rock is layered in either dumped or hand-placed or plated placement method. Dumped riprap is graded stone dumped on a prepared slope by mechanized means, such as crane and skip, dragline, or some form of bucket in such a manner that segregation will not take place. Hand-placed riprap on the other-hand is stone laid carefully by hand following a definite pattern, with the voids between the larger stones filled with smaller stones and the surface kept relatively even. Plated or keyed riprap is stone placement on the bank with a skip and then tamped into place using a steel plate, thus forming a regular, well organized surface. During the plating operation, the larger stones are fractured, producing smaller rock sizes to fill voids in the riprap blanket.

Figure 3-27: Dumped or hand-placed or plated riprap, HEC-11, 1997

The basic premise underlying riprap design based on tractive force theory is that the flow induced unit tractive force should not exceed the permissible tractive force or critical shear stress of the riprap.

Shapes of stones affect how well the stones are interlocked and offer resistance to movement. General design consideration:

- Riprap stone should be block in shape rather than elongated.
- It should be predominately angular and sub-angular in shape.
- Less than 30% of the stone should have a/c < 2.5
- Less than 15% of the stone should have a/c < 3.
- An approximate guide to stone shape is that neither the breadth nor thickness of a single stone should be less than one-third its length.
- It should not be steeper that 1V:1.5H; Recommended 1V:2H to 1V:3H though Ideal Side slope equals angle of repose of the material.
- Stone for riprap should be hard, durable field or quarry materials. They should be angular and not subject to breaking down when exposed to water or weathering. The specific gravity should be at least 2.5.
- The minimum layer thickness should be 1.5 times the maximum stone diameter, but in no case less than 150mm. It should not be less than the spherical diameter of the D_{100} stone, or less than 1.5 times the spherical diameter of the D_{50} stone, whichever results in the greater thickness. For practical placement, usually 300 mm is adopted. The thickness determined by either method should be increased by 50 percent when the riprap is placed underwater to provide for uncertainties associated with this type of placement. An increase in thickness of 150-300 mm, accompanied by an appropriate

increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or waves from wind, or bed-forms.

- Placement of Stone for riprap should follow immediately after placement of the filter. Place riprap so that it forms dense, well-graded mass of stone with a minimum of voids.
- Stones should be shaped so that the least dimension of the stone fragment is not less than one-third of the greatest dimension of the fragment. Flat rocks should not be used for riprap. Blocky and angular shaped rocks with sharp clean edges and relatively flat faces are good. If rounded stones are used, they should be placed on flatter slopes (not exceeding 2.5:1 horizontal to vertical) and the recommended median rock diameter should be increased by 25% with a comparable increase in the thickness of the revetment.

Note: The size of individual rock is usually expressed by the dimensions of their three axes. The long axis, a, is the maximum length of the stone. The intermediate axis, b, is the maximum width, perpendicular to the long axis. The short axis, c, is the thickness of the stone perpendicular to the plane of axes, a and b. The size of an individual rock is usually expressed as its b-axis dimension. The use of rock or stone size is preferred for riprap dimensions; however, weight is commonly used. The relationship of size to weight depends on stone shape and also on the specific weight or density of the rock. Typically, the space that the rock used for riprap is not spherical and its shape lies between that of a sphere and a cube.

Theoretical shear stress on channel bed is given by:

$$\tau_b = K_b * \gamma * R * S \qquad (3\text{-}121)$$

On channel sides:

$$\tau_s = K_s * \gamma * R * S \qquad (3\text{-}122)$$

Where, τ_b is theoretical shear stress on bed

τ_s is theoretical shear stress on sides

γ is the unit weight of water;

R is the hydraulic radius; and

S is the energy grade line slope;

K_b is 0.97 but usually taken as 1;

K_s is a function of stone size but it can be taken to be equal to 0.75

The riprap materials' resistance to movement i.e. its permissible unit tractive force is known as critical shear stress, (τ_c) and given by the following relationship the form of which was first proposed by Shields:

$$\tau_c = K * S_p * (\gamma_s - \gamma) * D_{50} \qquad (3\text{-}123)$$

Where, S_P is the Shields parameter;

γ_s is the unit weight of the riprap material;

γ is as defined above;

D_{50} is the median riprap particle size; and

K is the tractive force ratio defined as:

$$K = \left(1 - \frac{\sin^2\alpha}{\sin^2\phi}\right)^{1/2} \quad \text{.......... (3-124)}$$

Where, α is the bank angle with the horizontal; and

ϕ is the riprap material's angle of repose.

The ratio of the riprap's critical shear stress and the tractive force exerted by the flow is defined as the stability factor.

$$SF = \frac{\tau_c}{\tau_s} \quad \text{.......... (3-125)}$$

As long as the SF is greater than 1, the critical shear stress of the material is greater than the tractive stress induced by flow, thus the riprap is considered stable.

Dividing the critical shear stress by the tractive force due to the flow, rearranging terms, and replacing the hydraulic radius (R) with the average flow depth (h_{avg}) yields the following relationship:

$$\frac{D50}{h_{avg}} = \frac{SF}{S_p}\left(\frac{K_s S}{K(S_s - 1)}\right) \quad \text{.......... (3-126)}$$

Where, SF = the stability factor

S_s = the specific gravity of the rock riprap.

This equation represents the basic form of the tractive stress relationship and here, the median riprap size is primarily a function of flow depth and slope.

Table 3-15: Allowable ranges of stability factor

Factor	Range of SF
Uniform flow; straight or mildly curving reach (curve radius/channel width >30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0-1.2
Gradually varying flow; Moderate bend curvature (30 > curve radius/channel width > 10); Impact from waves or floating debris moderate.	1.3-1.6
Approaching rapidly varying flow; Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (0.30 - 0.61 m)); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6-2.0

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the structure. The filter

- Prevents the migration of the fine soil particles through voids in the structure,
- Distributes the weight of the armor units to provide more uniform settlement, and
- Permits relief of hydrostatic pressures within the soils.
- For areas above the water line, filters also prevent surface water from causing erosion (gullies) beneath the riprap.

The proper design of granular and fabric filters is critical to the stability of riprap installations on channel banks. If openings in the filter are too large, excessive piping through the filter can cause erosion and failure of the bank material below the filter. On the other hand, if the openings in the filter are too small, the build-up of hydrostatic pressures behind the filter can cause a slip plane to form along the filter resulting in massive translational slide failure. Thus, for rock riprap, a filter ratio of 5 or less between layers will usually result in a stable condition (HEC-11). The filter ratio is defined as the ratio of the 15 percent particle size (D15) of the coarser layer to the 85 percent particle size (D85) of the finer layer. An additional requirement for stability is that the ratio of the 15 percent particle size of the coarser material to the 15 percent article size of the finer material should exceed 5 but be less than 40. These requirements can be stated as:

- $$\frac{D_{15}(CoarserLayer)}{D_{85}(FinerLayer)} < 5 < \frac{D_{15}(CoarserLayer)}{D_{15}(FinerLayer)} < 40 \text{ .... (3-127)}$$

The left side of this equation is intended to prevent piping through the filter, the center portion provides for adequate permeability for structural bedding layers, and the right portion provides a uniformity criterion.

The thickness of the filter blanket should range:

- From 150 mm to 380 mm for a single layer, or
- From 100 mm to 200 mm for individual layers of a multiple layer blanket.

3.8.12.8 Gabions and Mattress Protection

Wire-enclosed rock revetments consist of rectangular wire mesh baskets filled with rock. They are formed by filling pre-assembled wire baskets with rock, and anchoring to the channel bottom or bank. Wire enclosed revetments are applicable for conditions that are similar to those of other revetments. However, their economic use is limited to locations where the only rock available economically is too small for use as rock riprap slope protection.

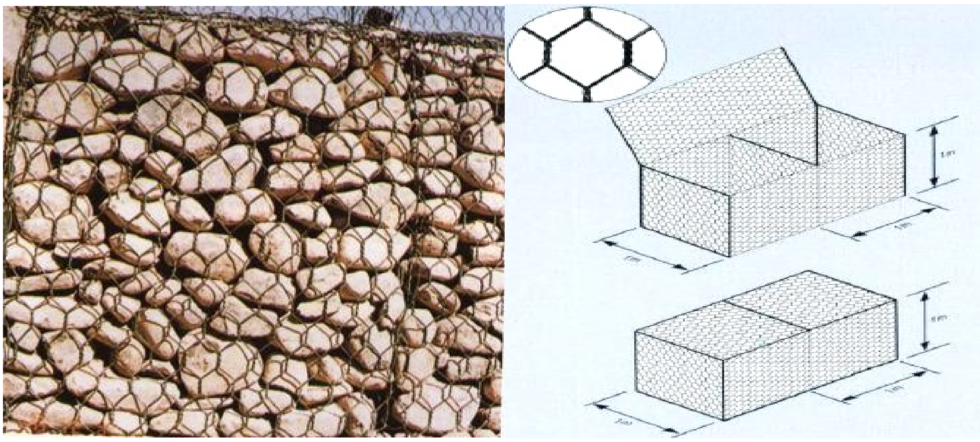


Figure 3-28: Rock filled gabion (L) and under preparation (R)

General design consideration:

- Subgrade should be compacted and levelled to receive first layer of gabions.
- The first row should be keyed into the existing grade at the toe, a minimum of 0.5 m.
- Gabions should be placed according to the manufacturers recommendations.
- Gabions should be filled with stone or crushed rock from 100 to 200mm in diameter.
- In corrosive environments, gabion wire should be coated with Poly Vinyl Chloride (PVC).

Wire-enclosed rock revetments are generally of two types distinguished by shape or geometry:

- Mattresses: consist of flat wire baskets having a depth dimension which is much smaller than their width or length. The individual mattress sections are laid end to end and side to side on a prepared channel bed or bank to form a continuous mattress layer. The individual basket units are attached to each other and anchored to the base material.

- Block gabions: consist of rectangular wire baskets having depths that are approximately the same as their widths and of the same order of magnitude as their lengths. The baskets are stacked in a stepped-back fashion to form the revetment surface. They are typically rectangular or trapezoidal in shape.

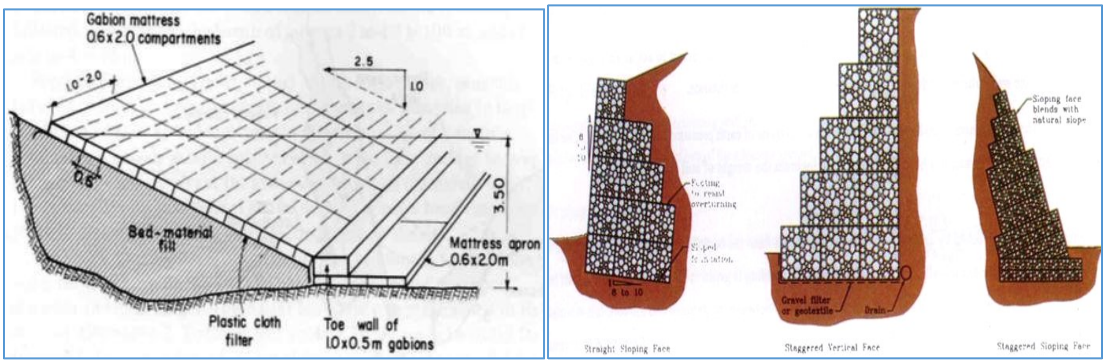


Figure 3-29: Arrangement of mattresses (L) block gabions (R)

3.8.12.9 Indirect protection

This includes works that are not done directly on the bank but in front of them with the view of reducing the erosive force of the current by deflecting the current away from the banks or by inducing deposition. Vegetation can function as either revetment or indirect protection, and in some applications, can function as both simultaneously.

3.9 ANALYSIS OF UPSTREAM WATER SURFACE PROFILE/AFFLUX/

3.9.1 Upstream water surface profile determination

When a barrier structure is constructed across a river, water surface propagates upstream and consequently the water surface profile changes from the normal water surface to a level that is detained by the barrier plus depth of water over the crest. Afflux, though confined in the beginning to a short length of the river above the barrier, extends gradually very far up till the final slope of the river upstream of the barrier is established due to sediment deposition in-front of the structure. Thus, afflux is maximum at location close to the barrier structure and reduces gradually while moving towards the upstream until it vanish at far upstream. The difference between afflux level and weir crest level shall be the depth of water over crest. Afflux shall also govern the top levels of protection embankment, their sections, length and all top levels of the components of structure like abutments, operating platform, divide walls, etc.

Determination of the upstream water surface profile (afflux) is mostly not required, unless there are some upstream concerns with the raised river water levels (e.g. shallow upstream bank, incidence of flooding, land control, etc.) for the case of shallow u/s/ banks. With lesser upstream velocities, generally there should not be additional erosion, except possibly some isolated bank erosion concerns because of the raised water levels.

The value of afflux at the design flood permitted to bypass the structure is one of the factors that the width of a weir is governed by, in addition to the existing stream width and the proposed crest levels. Moreover, the upstream water surface profile helps us to determine depth of water upstream of the weir and to find out whether water has sufficient head to feed the off taking canal in addition to its use for the weir stability analysis. The amount of afflux determines the top levels of guide banks and their lengths, and the top levels and cross-sections of flood protection bunds. It also governs the dynamic action, as the greater the afflux or fall of levels from upstream to downstream, the greater is the action. It also controls the depth and location of the standing wave. By providing a high afflux, the width of the barrier can be narrowed but the cost of training works can go up and the risk of failure by out flanking will increase. Thus, selection and adoption of a realistic medium value is imperative, which is commonly 1-1.2m.

There are different methods for determining this profile but the most common ones are stated as follow.

3.9.2 Approximate method

This is applicable method for determining the upstream water profile for preliminary design purpose by considering a channel with uniform cross section and constant hydraulic properties.

$$Y = \frac{(XS - 2*\Delta_0)^2}{4*\Delta_0} \quad \text{.......... (3-128)}$$

Where, Y - Water rise at distance X upstream of the weir above the normal water depth.
X - Distance from the crest to the point where Y is required to be determined.
S - Slope of the riverbed
Δ_0= (h+H_d)-TWD, is rise of water above normal flow depth at X=0.................. (3-129)

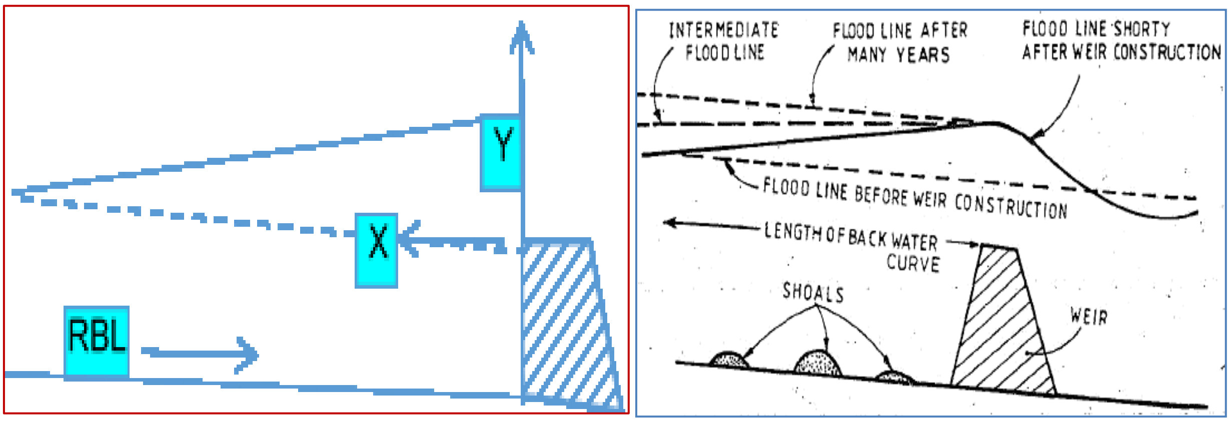


Figure 3-30: Schematic view of backwater profile

Box 3-9:
Worked Example-9: *If coordinates of the lowest three points along thalweg of upstream of weir site, along weir axis and downstream of weir site are identified to be* (793197.715, 789102.648, 1398.107); (793156.162, 789105.566, 1396.686) and (793117.610, 789121.653, 1396.219) Adindan UTM Zone-36 *respectively, then how far does water level corresponding to the given design flood* Q_{50} *will be back?*

Solution*: First the average longitudinal slope of upstream and downstream reaches of the river bed,* (S=ΔH/L) *need to be estimated. Then using equation* (3-105)*, we need to estimate water rise and corresponding distance, X upstream of the weir above the normal depth.*

Table 3-16: Estimation of average longitudinal slope of reaches of the river

Description	X	Y	Part. Dis.	Cummu. Dis.	OGL
RBL @ u/s	793197.715	789102.648	0.000	0.000	1398.107
RBL @ weir site	793156.162	789105.566	41.66	41.66	1396.686
RBL @ d/s	793117.610	789121.653	41.77	83.43	1396.219

From this table, Slope on u/s side, S_u = (1398.107-1396.686)/41.66 = 3.4%; Slope on d/s side, S_d= (1396.686-1396.219)/41.77=1.1%; thus average slope, S_{avg}=2.26%. $\Delta_0=(h+H_d)-TWD$= (1.60+1.622)-0.98=2.25m. Consequently, by goal seek or trial and error, Y=0 at X=214m. This means back water effect on the water profile during peak flood due to weir ceases at 214 m from the axis back.

3.9.3 Standard step method

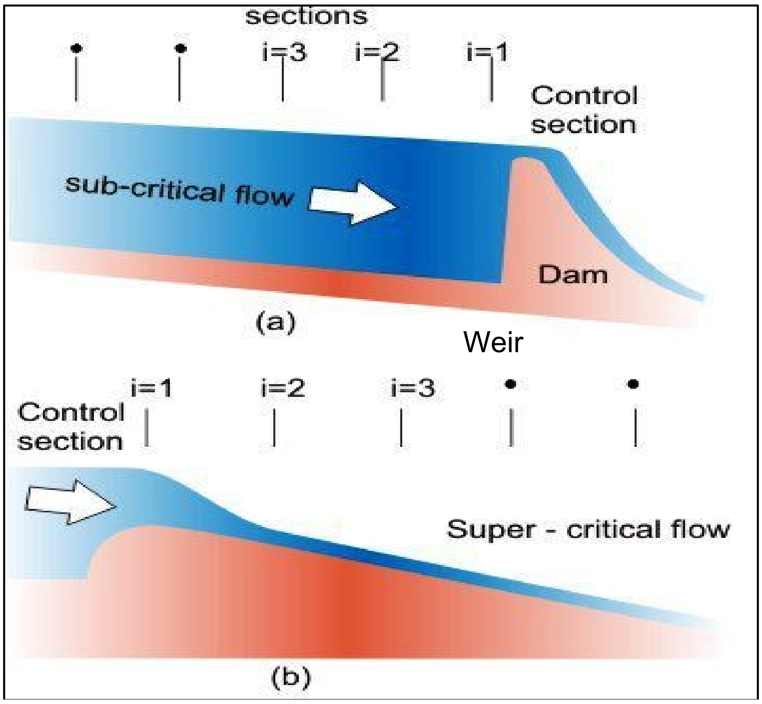
This method is used to determine both the upstream and downstream water profile. It is preferred if the water stretches long distance back and the friction loss is considerable. Steps to follow in this method are:

- Identify cross sections of the river at points where geometrical and hydraulically characteristics are expected to show a change.
- Determine cross sectional area of each section, A
- Determine wetted perimeter of each cross sections, P
- Determine hydraulic radius of each cross sections, R
- Prepare a table
- Start computation by assessing water level if not known at the 1stsection

$$S_f = \left(\frac{\sum Q_i}{\sum K_i}\right)^2 \qquad \text{(3-130)}$$

Where, Sf is friction slope of each sub areas
Qi is flow in each sub areas
Ki is conveyance parameters within each sub areas

Equation above can be used by starting from one end of the channel where the flow depth and velocity are known and working backward or forward in steps as required. Here, two, methods are used of which we shall discuss one, called the standard step method. Avery popular computer program called HEC-2 developed by hydrologic engineering center of the US Army Corps of Engineers is based on this method. In this method, for any given discharge the depth of flow would be known at the control section. It is then required to calculate the depth of flow at the section immediately next to the control section as illustrated in Figure below.



Weir

Figure 3-31: Representative computation grids by standard step method

(a) Profile behind the structure: Calculation proceeds upstream from control section
(b) Profile in a steep channel: Calculation proceeds downstream from control section

3.10 DESIGN OF HEAD REGULATOR

3.10.1 Location and functions of head regulator structures

Head regulator is a structure located at the off-take from a diversion headwork structure by atleast 0.5 to 1m on the upstream side of the scouring sluice to allow irrigation water to conveyor or main canal.

The functions of head regulator structures are:

- Serve for regulation of flow in to the canal for all river stages;
- Stilling basin and transition into the downstream canal;
- A room for measuring device such as Crump weir or cut-throat flume;
- Allow integration of canal crossing bridge in order to access the headwork structure and river banks.

The first requisite for the intake site is that the river channel, in other words the thalweg, should be stable. Stability of river banks and condition of sediment are also the most important factors governing the site selection and design of intake structure. Sediment inflow to the canal would takes place in the following cases:

- If the intake site is not correctly sited in relation to meandering of the river;
- If water intake is required even during flood season;
- If the intake is located too close to the riverbed elevation and the intake flow velocity is large;
- Flow velocity through the intake is high.

As a rule of thumb, flow velocity through the intake gate opening in the order of 1.5 to 2 m/s is recommended at design discharge and minimum pool level to keep the head loss through the structure less than 0.3m.

Intake structure can be situated on the upstream face of the wing wall or in the middle or on the outer side. However, each and every option has its own advantage and disadvantage as shown below.

Table 3-17: Comparison of possible arrangement of intake gate

Possible position of Intake	Advantage	Disadvantage
On the upstream face of the wing wall	- In this case the intake can get water without any problem	- Trash racks and the gate itself can easily be susceptible to destruction/deflection by flooding
In the middle of the wing wall	- No problem of flood hazard - Easily manageable	- Difficult to maintain
On the outer side of the wing wall	- The most easily operable - Easy to maintain	- Pressurized water can dismantle the system - Subject to clogging/sedimentation - Susceptible to stolen

Thus, the middle arrangement may be the most suitable though its selection is left to the designer based on site condition.

3.10.2 Types and selection of intake on diversion weir

Types of intake for off-taking water from the diversion headwork of SSIP can be:

- Rectangular opening (box), or, and
- Circular opening (concrete pipe).

Box type could be rectangular or square in shape and is used when required flow in the conveyance canal is larger. For smaller flow, piped system is preferred as it is more efficient in discharging than the box intake. If the adjacent topography is also steeper, then piped system is preferred to bury it and extend till it emerges from the rugged surface.

When considering intake of water on both banks, the intake levels of both sides should be equal, otherwise available low flow may be imbalanced thus slanted to the intake with lower bottom level. Their intake capacity shall rather be managed by water level passing through the intake by corresponding gate installed in the wings. Their location may not necessarily be parallel to each other, rather governed by adjoining geology and topography.

3.10.3 Consideration in fixing intake level and its type

The following points need to be considered while selecting type of and fixing intake level of Intake or head-regulator:

- Capacity of intake should be such that it allows the peak discharge or the maximum irrigation water demand plus some 20-30% allowance for future demand.
- Bed level of an intake structure should be high enough to prevent entrance of bed load in to the canal and the same time, it should be low enough to harvest available low flow of the river.
- Intake should be such that it enables regulation of supply of irrigation water based on the irrigation schedule or demand and the availability of water in the river.
- If the level of the intake at the headwork site is higher than the existing riverbed level then a weir with seepage under it, even with a deep vertical cut off and/or an upstream

seepage apron shall be considerable. In addition, if there is considerable evaporation losses from the pond created upstream of the weir, moving the weir's location further upstream should be considered.

- Perennial flow available in the river at the intake location limits the design discharge capacity. Thus the design discharge at the intake should be less than the 1:5 year low flow in the river for every month of the year.
- For most irrigation schemes it is usually sufficient to provide a simple gated orifice intake in the weir abutment wall just upstream of the scour sluice;
- The intake to the irrigation canal should be gated so that the canal can be closed off during floods;
- If not closed off sediment may enter the canal, requiring (labor intensive) cleaning or settling basin along the MC;
- The gate should be capable of being operated under high pressure of water during floods. So, should it be located in the wall or on the outer side of the wall?
- If gates are not accurately manufactured, during floods they may be jammed against their (rubber) gate seals rather than moving freely on their bronze track bars;
- The bed level of the intake canal should be below the main weir but above the invert of the scour sluice;
- The intake should be sufficiently below the main weir level to allow the full design flow to enter the intake for a river water level equal to the main weir crest level;
- Reinforced concrete breast walls are usually provided to the head regulator so that orifice flow occurs during floods;
- This makes the flow through the regulator proportional to the square root of the flow depth (i.e. $\propto h^{0.5}$), rather than proportional to the flow depth to the power 1.5 (i.e. $\propto h^{1.5}$).

In general, the inlet elevation is preferred to be 1.0m higher than scouring sluice sill and also preferred to be more than 1/6 of maximum flood depth of the river from the riverbed for prevention of sand in case of alluvial channel. But in case of small head diversion weirs, a minimum inlet elevation is at least 0.5m higher than scouring sluice sill. If the height from scouring sluice sill to inlet elevation is lower than 1.0m, settling basin should be considered especially if the river is sediment laden type.

3.10.4 Intake sizing

Intake also called Head Regulator can be box /rectangular or square/ type or concrete piped system. In case of box type, it acts as a submerged orifice. Thus, same formula as that shown in equation 3-17 is adopted:

$$Q = CA\sqrt{2gh} \qquad \text{(3-131)}$$

Where, Q = Discharge through the opening (m^3/s)
C = Coefficient of discharge, usually = 0.62
g = Acceleration due to gravity (m/s^2)
A = Actual water area, (m^2) =b*d, if rectangular and $=\pi d^2/4$, if pipe …................... (3-132)
h=Working head (=$d/_2$+driving head i.e. water head above center of opening) .... (3-133)
d = Depth of flow in conveyance canal/MC

The size of opening of the intake structure is determined based on the amount of irrigation water required if the available flow in the river is greater than the quantity of irrigation water demand; otherwise it is based on the available base flow in the river with considerations of some downstream release. Computed discharge through this opening should not in any case be less than the design discharge of MC, rather this capacity shall be considered in excess of the design discharge by at least 10 to 20% for the purpose of unaccounted and future demand.

Due to the difficulty to measure *h*, a simplified vertical gate flow formula was derived as:

$$Q = C_d A\sqrt{2gy_1} \quad (3\text{-}134)$$

Where C_d is the discharge coefficient and

y_1 is the average headwater depth above the intake crest level.

If the intake is a box:

$$Q = C_d Bw\sqrt{2gy_1} \quad (3\text{-}135)$$

Where w is the height of gate opening (gate setting) in m,

B the effective width or opening (water way) of the intake structure in m.

It is important to understand that:

- There is no problem with the supply during the rainy or flood season.
- The peak irrigation demand is usually synchronized with the low river flow (or simply dry season).

It seems that the governing condition for the design of the intake is satisfying the peak irrigation demand during the dry season i.e. low river flow and peak canal discharge. Therefore, it is unlikely to have submerged flow under such condition and hence the designer is required to check the condition of flow.

The other important factor to be considered while designing the intake is the requirement for sediment cleaning. If there is a culvert connecting the intake to the irrigation canal the condition for cleaning of the culvert should be considered. It is usually recommended to have at least two bays of intake in order to get prepared for unexpected accident that block or clog or damage one of the bays of the intake.

The following procedure can provide guidance on the sizing of intake:

i. Get the necessary input data i.e. design canal discharge (peak), normal pool level at the diversion headwork, under-sluice sluice bed level, canal full supply level (optional), selected type of intake (box or circular);
ii. Set the design criteria i.e. maximum gate setting and head-loss (as percentage of headwater depth);
iii. Assign value for width and depth (box) or diameter (circular);
iv. Assign level for intake gate crest level i.e. 0.5 to 1m higher than under-sluice level
v. Determine y_3 and compare with the critical value of y_3 *and understand the condition of flow (submerged or free flow);*
vi. Determine the value of C_D using the formula of free flow or using Figure 3-8;
vii. Calculate the discharge and compare with the design canal discharge; calculate the head-loss and compare it with the criteria. If enough accept the assigned width and depth (box) or diameter (circular) otherwise modify it;
viii. If there is a culvert integrated with the intake, refer a separate guideline for the design of the culvert.

When the pool level is at minimum, energy dissipation in the head regulator at design discharge is not a problem because the velocity through the gate is very low. However, when the canal operates at discharge less than design discharge, the canal water surface will be lower and significant differential head may exist across the head regulator. In order to control the discharge and for dissipation of the extra head, the gates must be partially closed. It is to be expected that the worst condition for design will occur when maximum pool level developed upstream of headwork and the head regulator is operated at design discharge, since both the head and discharge are maximum.

The head regulator stilling basin must be designed to accommodate a free or submerged hydraulic jump for this condition. Floor baffles and an end sill may be used. The design procedure is similar to that used for the stilling basin of diversion structure here in above. It is interesting to note that the gates are sized for the minimum pond level, whereas the basin floor must be sized for the maximum pond level.

For the case of concrete pipe head regulator system, head losses through the pipe shall first be estimated based on design discharge of MC from the following equations.

Intake loss, $h_l = (V^2/2g)*F_T$ ………………………………………………………….. (3-136)

Where, F_T= (Entry loss + Friction loss (F_L) + Exit loss) coefficients……………………….…. (3-137)

Entry and exit losses are taken 0.5m and 1m respectively.

Head loss due to friction, $F_L = f*L/D$ …………………………………………………. (3-138)

Where, factor $f=124.5*n^2/D^{1/3}$ ………………………………………………………….……....... (3-139)

V = Velocity of flow through the opening (m/s)
D = Internal pipe diameter (m)
L = Length of pipe (m)
n =Roughness coeff. of concrete pipe=0.014 and 0.012 for plain and reinforced resp.

Note: To simplify operation of such intake pipes, a minimum pipe diameter of 0.5m is usually adopted. For details, refer additionally accompanied excel template.

3.10.5 Trash racks arrangement at intake structure

These are simple structure provisions that need to be installed on intake structures and need to be mounted to the inlet of any structure that could suck floating debris, or any other animals that could cause damage or blockage to the intake. The requirement for trash rack is inevitable when the river flow is loaded with various forms and type of debris and rubbish. Therefore, a trash rack is required in front of an intake on the river side. The trash rack consists of stationary parallel row of steel bars that allows flow of water through it with the required flow velocity. It is recommended that the steel bars are required to be placed at 7.5 to 15.5cm spacing (USBR, 1987). Usually a minimum inclination of 10 degree from the vertical is in practice for trash rack.

Design considerations of trash racks for intake or head regulators are:

- Trash racks are bar screens, made from steel bars spaced at 5 to 15 cm center to center (in both directions) depending upon the maximum size of the debris required to be excluded from entering the conduit. Thus, they should be installed at the entrance to intakes, tunnels and inverted siphons as a matter of course.
- The velocity of flow through the trash rack is kept low (generally less than 0.62 m/s), so as to minimize losses. This is sometimes accomplished by constructing the trash rack in the form of a half cylinder.
- Rack Inclination: 60 degrees from horizontal;
- The racks shall be identical and interchangeable between intake bays;

- Rack Height: The rack shall extend above the elevation of the service bridge a distance considered necessary by the manufacturer to ensure the debris falls freely onto the conveyor. The rack shall be designed to facilitate the movement of the debris by the raking equipment to the conveyor by means of metal guides, sloping end bar sections, etc.

Design of these structures should consider that such trash rack shall be installed at the entrance of intake, tunnel and siphon. Flow velocity through the trash racks shall be kept low to minimize losses (generally less than 0.62 m/s) and the rack extends sufficiently above the soffit level of the entrance or opening.

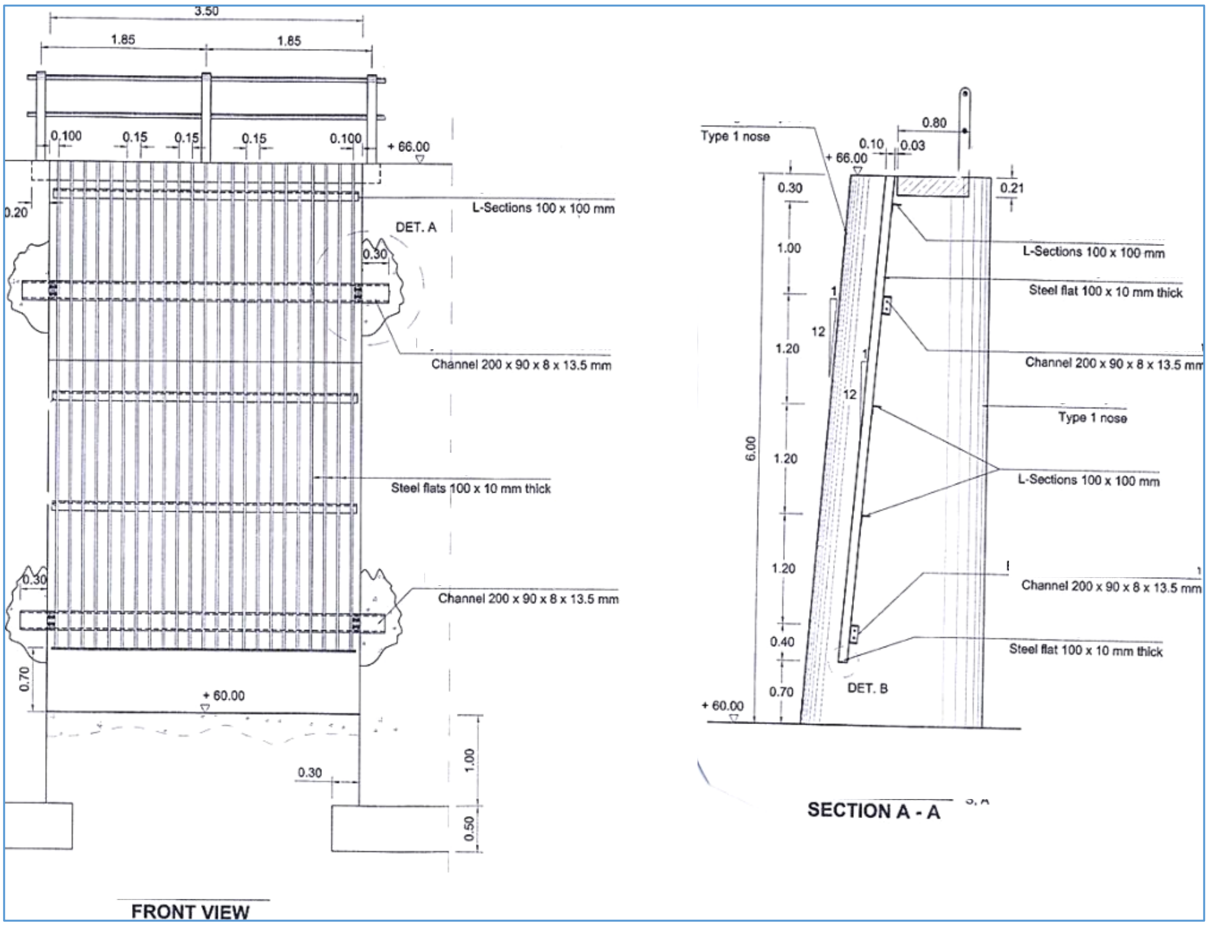


Figure 3-32: Typical trash rack arrangement at intake structure

Table 3-18: Head loss through trash racks

Velocity through Trash Rack, m/s	Head loss, m
0.15	0.006
0.30	0.03
0.45	0.09
0.62	0.15

Source: SK Garg 2006

Box 3-10:

Worked Example-10: *If a plain* concrete *pipe length of 5m is required for taking off 56.5 l/s* discharge *for the aforementioned worked examples, how much shall the size of this pipe be?*

Solution: *Assuming diameter of pipe (to start with), entry and exit losses to be 0.5m and 1m respectively and roughness coefficient of concrete pipe=0.014; factor* $f=124.5*n^2/D^{1/3} = 0.03074$; *Thus head loss due to friction,* $F_L = f*L/D = 0.30745m$ *giving total loss* $F_T = 1.80745m$. *Area of flow,* $A = \Pi D^2/4 = 0.19625m^2$ *and Velocity of flow through the opening =Q/A=0.29m/s. This consequently gives a velocity head,* $h_V = V^2/2g*F_T = 0.007634m$.

Now, check this capacity of the designed pipe using orifice formula: $Q_{intake}=C*A*(2gh)^{1/2}$, *Where: C is orifice coefficient=0.62 for circular intake,* $Q_{intake}=C*A*(2gh)^{1/2} = 0.0471m^3/s$ *which is less than the required discharge of 56.5l/s thus, revise it by assuming another diameter, say D=15cm which gives* $Q_{intake}=C*A*(2gh)^{1/2} = 0.6098m^3/s$. *But from operation point of view,adopt, d=50cm i.e. so as to allow operators to move and clean silt in the pipe.*

If it is of box type, $A=Q/(C*(2g*h)^{1/2}) = 0.032m^2$, *where h= (1/2)*0.35 +0.24= 0.42m. Thus, from A=b*w; w=0.11m, say w=20cm.*

3.11 DESIGN OF SCOURING-SLUICE

3.11.1 General

Scouring-sluices are also called under-sluices as well as sluiceways. A scouring sluice is one of the parts of the headwork structure that must be provided at the intake side so as to stabilize flow in to the intake. It is also designed to prevent sediment inflow into the canal as much as possible when drawing water.

3.11.2 Functions of scouring-sluice

Scouring-sluice is used to evacuate the accumulated sediment deposit upstream nearby the head regulator or intake during normal operation condition, to allow passage of flood during flood period, and to maintaining a channel towards the head regulator or intake (Smith, 1995; Novak et al., 2007).

By operating the sluiceway at full capacity during floods a current is maintained and the channel is kept clear. During normal operating period when most of the flow is being flowing through the sluiceway, a dune may buildup progressively towards the head gate and eventually sediment could pass in to the canal. This can be prevented if the sluiceway is opened alternatively and periodically to discharge the accumulated deposit in front of the head regulator or intake. In river having sufficient seasonal discharge a sluiceway may be set to operate continuously while a diversion is being made. Under-sluice should be left fully open during the rainy season/flood flows/ to drain as much sediment as possible through the sluice channel.

3.11.3 Locations of scouring-sluice

Scouring-sluices need to be located adjacent to the irrigation inlet or intake that is in-front of the head regulator or off-taking canal. They should be left open during the rainy season/flood flows/ to drain as much sediment as possible through the sluice channel. Thus they need to be set at a lower elevation than the intake.

The main weir portion is separated from the under-sluices portion by a long structure called divide wall. The arrangement is aimed at keeping approach channel to the intake or canal head regulator comparatively clear of silt and to minimize the effect of main river current on the flow conditions in the regulator. Hence the purpose of under-sluice is for flushing out the lower bed loads and comparatively enable drawing of silt free water by the intake. This structure should therefore be located adjacent to the intake structure so as to minimize entrance of silt in to intake and hence main conveyance.

By operating the sluiceway at full capacity during floods, flow current is maintained and the channel is kept clear. During normal operating periods when the flow is being diverted, a dune may build up progressively toward the head-gate and eventually sediment could pass into the canal. This can be prevented if the sluiceway is opened periodically to discharge the accumulated deposit from in-front of the head-gate.

The bay before the gate serves as a settling basin as the turbulent water rushing in to it is forced to calm down thereby sediment-load is also settled down and relatively clear water is picked up by intake canal through the head regulator, which is usually raised by a meter or above from crest of the sluice way.

3.11.4 Design Consideration

In order to accomplish the foregoing function and objectives of under-sluice the design must meet the following requirements (Halcrow, 1988; Smith, 1995):

- The sluiceway must be located adjacent to the head gate;
- The width and opening size of the under-sluice portion of a weir shall be determined on the basis of considerations that it should not be too wide to keep the approach velocities to cause maximum settlement of suspended silt load within the pond and should be manageable size that can be operable by a single operator;
- The invert level of the sluiceway and its approach apron and gates should be about 1m below the head gate invert level. Normally, it is kept equal to the deepest bed level of the river. As the crest of the under sluice pocket is at a low level, a deep channel develop towards this pocket, which helps in bringing low dry weather discharge towards this pocket, thereby, ensuring easy diversion of water into the intake;
- The under-sluice bay floor level is generally kept as low as possible to create pool conditions for silt settlement and its exclusion later. Thus, its crest level is fixed by considering the workability of the river bed and the required head below the canal sill level to scour the deposited sediment in front of the canal off take.

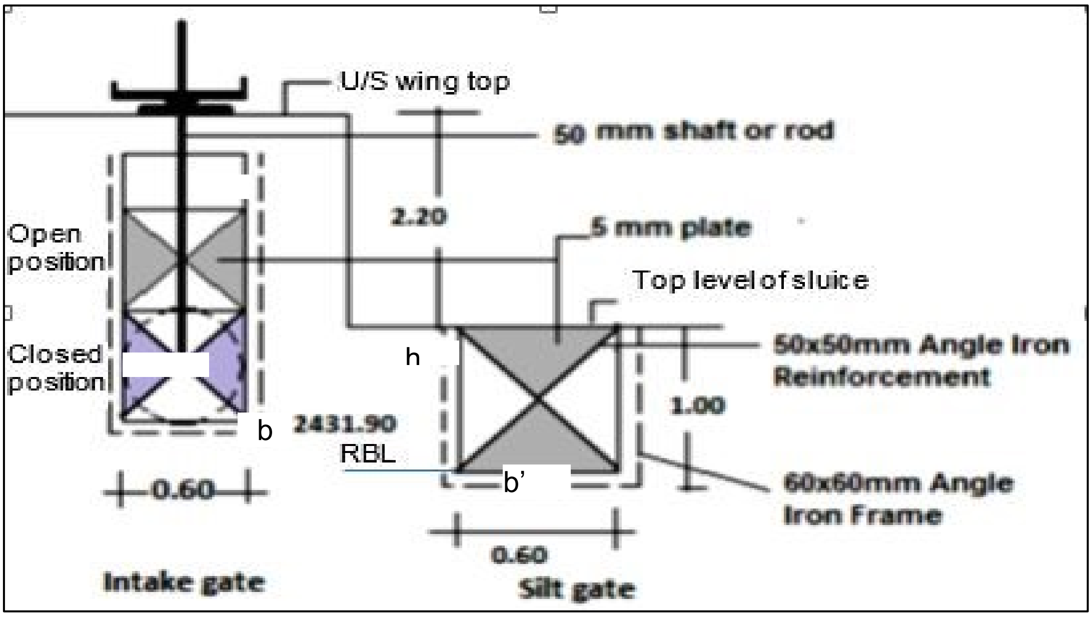


Figure 3-33: Relative Arrangements of Typical Intake and Silt Gates

- Normally, the crest level of the under-sluices is kept equal to the deepest bed level of the river during non-monsoon season. As the crest of the under sluice pocket is at a low level, a deep channel develop towards this pocket, which helps in bringing low dry weather discharge towards this pocket, thereby, ensuring easy diversion of water into the canal through the canal head.
- If the waterway is very wide, then the whole length is divided into bays each being separated with piers and each bay is provided with a gate so as to act as an opening of controlled height for flushing during flood times;
- The under-sluiced length of weir is then divided into a number of bays by piers, and separate gates are installed on these bays. Each bay can thus, be opened to any desired height by lifting its gate and hence, act as a gate controlled opening, and will help in bypassing the excess supplies to the down-stream side of the river. These openings will also help in scouring and removing the deposited silt from the under sluiced pocket; and hence are also called the scouring sluices;
- A substantial stilling basin must be designed for a sluiceway because it may have to operate when the discharge over the diversion dam is low. The tail water depth in the river channel will be correspondingly low, much lower than at flood stage, yet the sluiceway may have to be operated at or near full capacity in order to effectively flush out silt deposits. The slab elevation for the hydraulic jump stilling basin will have to be much lower than the basin slab for the diversion structure.

Discharging capacity of under-sluices is carefully chosen based on the following criterion:

- They should be able to ensure sufficient scouring capacity, for which discharging capacity should be at least twice the full supply discharge of MC at its head.
- They should be able to pass dry weather-flow and low floods during the months excluding rainy season without necessity of dropping weir shutters,
- They should be able to dispose of 5 to 20% of design flood discharge during expected design floods so as to reduce flood height over the structure and hence wing wall height, i.e. it should be designed to ensure sufficient scouring capacity to dispose off the above range of the peak flood. This value shall at least be greater than 2 times the intake capacity;
- Bottom Level of Under Sluices should be fixed such that it allows easy flushing of sediment (usually kept on river bed level);

The simplified vertical gate flow formula is still used for the under sluice. The usual opening in under-sluice is box type. The design of the opening size shall be made for three cases of scenarios. The size of opening fixed by the design of the sluiceway should fulfill the requirement of all the three cases. However, it should be understood that the governing case is when the river is at design flood and the under sluice is totally submerged because it is during this case the under sluice operates at maximum capacity.

Case 1: When the headwater depth in upstream is at normal pool level or crest level and no tail water in the downstream, then the opening of under-sluices work as free flowing orifice. The discharge through free flowing orifice is calculated by similar formula with box type opening.

Case 2: When the headwater depth upstream is at highest flood level and tail water level in the downstream is below the soffit level of the sluiceway, then above formula still holds true. However, check for submergence might be necessary as it is most likely to prevail.

Case 3: When the headwater level upstream is at highest flood level and tail water level in the downstream is above the soffit of the sluiceway (or above sluiceway opening), the sluiceway will acts as submerged orifice.

3.11.5 Hydraulic design of scouring sluices

The hydraulic design of scouring sluice is sizing of its opening and floor related works. The sizing of the opening use the simplified vertical gate formula given above but with box type of opening. The design procedure for sizing the under-sluice opening is almost similar to the procedure given for intake gate, however,

- The opening is box type (width and height) and the flow condition is submerged. The design criteria shall be the gate setting equivalent to the height of the gate.
- The flow velocity through the opening of the under sluice should be greater than the critical velocity. This will enable easy flushing of sediment deposited upstream of the sluice.

The following has also been in practice during the design of under sluice.

$$Q = C_d \; x \; A\sqrt{2gh} = C_d \; x \; L_{eff} \, x d x \sqrt{2gh} \quad \text{... (3-140)}$$

Velocity of flow through these openings is also given:

$$V = C_d \; x \sqrt{2gh} \quad \text{.. (3-141)}$$

Where, Q = Discharge through the openings (m^3/s)
A = Area of flow through the openings, (m^2)
g = Acceleration due to gravity (m/s^2)
h = Head above center of orifice for worst case, i.e. when it is at pool level, (m)
C_d = Coefficient of discharge, usually = 0.60
L_{eff}= effective length of the openings (m)
d = Depth of opening, (m)

This velocity of flow through each sluice channel should be greater than critical velocity, Vc, so as to enable easy flushing of sediment through the openings, i.e. flow within the scouring sluice should be in supercritical condition to remove sediment deposited in-front of intake, but that through the intake should give a critical flow condition so as to allow flow to the canal.

Design of these sluices should therefore be made so as to allow transportation of the maximum particle sizes of the riverbed materials with this critical flow velocity, V_c which is given by the following formula (JICA-OIDA, 2014).

$$V_c = \sqrt{20 * d_m} \quad \text{(3-142)}$$

$$h_c = \frac{20 * d_m}{g} \quad \text{(3-143)}$$

$$q_c = \sqrt{\frac{(20 * d_m)^3}{g^2}} \quad \text{(3-144)}$$

Where, V_c is Critical flow velocity through the openings (m/s)

d_m is maximum particle size through the openings, 90 % passing by weight, (m)

h_c is Critical depth of flow through the openings, (m)

q_c is flow per unit critical width, (m^3/s/m)

g is acceleration of gravity, (m/s^2)

The height of the guide wall H required to form a channel for the scouring sluice is made 1.5hc at the point of intake.

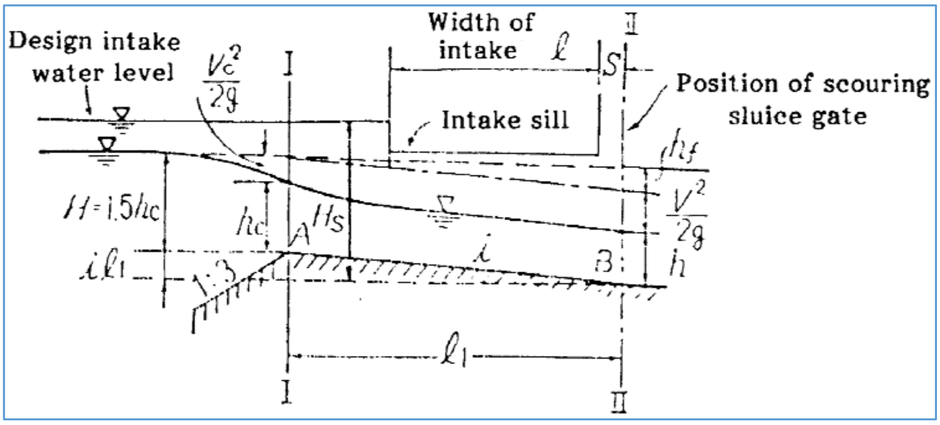


Figure 3-34: Flow Profile in u/s of Scouring Sluice Channel, (JICA-OIDA, 2014)

Note: River bed grain size distribution need to be sampled, tested in laboratory and analyzed so as to use in these equations. However, in case there is no data, the following indicative ranges of particle size distribution and corresponding permeability coefficients can be used (though not recommended unless it is compulsory).

Table 3-19: Particle Size Distribution and Corresponding Permeability Coefficient

Classification	Clay	Silty Clay	Silty sand	Fine sand	Medium sand	Course sand	Gravel
d (mm)	0~0.01	0.01~0.05	0.05~0.10	0.10~0.25	0.25~0.50	0.50~1.0	1.0~5.0
K (cm/s)	0.000003	0.00045	0.0035	0.015	0.085	0.35	3.0

Note: d is average or 50% of grain size distribution, mm.

Table 3-20: Average Particle Size, d (mm) for Various Types of Materials

Type a/material (soil)		Average grain size, d (mm)
Silt	Very fine	0.05 to 0.08
	Fine	0.12
	Medium	0.16
	Standard (i.e. f=1.0)	0.32
Sand	Medium	0.51
	Course	0.73
Bajri and sand	Fine	0.89
	Medium	1.29
	Course	2.42

Type a/material (soil)		Average grain size, d (mm)
Gravel	Medium	7.28
	Heavy/Course	26.1
Boulders	Small	50.1
	Medium	72.5
	Large	188.8

Source: S.K. Garg, 2006

Box 3-11:

Worked Example-11: *Suppose the designed diversion weir in the previous worked examples has two scour channels. Thus, how much shall the size of these openings be to allow recommended design flood? If 10% of this design flood is allowed to pass through these sluices compute appropriate size of these channels (Note: This exercise illustrates either assuming sizes and check for the capacity or fix capacity of channels and check for the sizes).*

Solution*: First assume size of sluice channels and check for allowable flow capacity in these channels.*

Case-1*: **When U/s WL is at WCL**:*

Let, length of the openings, L_{eff} = 0.7m each, C_d=0.6and Depth of opening, d =*0.7m; Sluice bottom Level, SBL=RBL=1432.475m, thus its top level = 1433.175m.* Head above center of orifice for worst case, h = weir height – (driving head + ½ * Depth of opening) = 1.01m. Now, Discharge through the openings, Qs= 2*(0.6*0.7*0.7*SQRT(2*9.81*1.01))=2.62m^3/s, which is 10.5% of design flood discharge thus ok as it is within allowable range. Velocity of flow, V=0.6*(SQRT(2*9.81*1.01))=2.66m/s and Critical velocity, Vc = q_s/d_c, Where q_s is unit discharge and dc is critical depth of flow; q_s= Qs/b = 1.31/0.7=1.87m3/s/m and*dc =(q_s^2/g)^(1/3)= 0.71m; Thus, Vc=2.64m/s;* implying V >Vc. Thus, take *0.7m wide by 0.7m high sluice channel openings.*

Case-2*: **When U/s WL is at its maximum i.e. HFL**:*

In this case head above center of orifice opening, h= weir height + flow depth over crest - ½ * Depth of opening=2.87m. Based on above procedure, this gives 17.8% for the above sizes, which thus satisfies condition of 5-20%, thus ok in both cases.

Case-3*: For the case when 10% of this design flood is allowed, discharge of each sluice, Qs=10%(24.8)/2= 1.24m3/s; Thus, critical width of sluice channel, L= bc =$Q_d/(C*H^{3/2})$, Where H = h+Hd (for the case of design flood)=24.8/(1.7*(1.6+1.622)^3/2)=0.13m, taking a practical value of multiple of 0.5, L=0.5m; With this width of channel, unit discharge, q in the sluice= Qs/b = 1.24/0.5 = 2.48m^3/s/m; Consequently, critical depth, dc =(q_s^2/g)^(1/3)= 0.86m; and corresponding critical velocity, Vc = q_s/d_c = 2.89m/s; In this case head above center of orifice opening, h=2.79m; Thus, Vc=qs/dc = 2.89m/s, but Velocity through each sluice, V_{sluice} =0.6* (SQRT(2*9.81*2.79)=4.44m/s,* implying V_{sluice}>Vc; But, sluice channel width required here i.e. 0.5 by 0.9 is not proportional.

For course gravel bed material, d_m=26.1mm (from table above), thus, critical unit discharge in sluice channel, q_c= 0.038m^3/s/m; 1and corresponding critical depth, dc =0.05m; andVc= 0.72m/s, but Velocity through each sluice, V_{sluice} =4.50m/s, implying V_{sluice}>Vc; Thus, take *0.7m wide by 0.7m high sluice channel openings on both left and right sides as they satisfied all required conditions.*

3.12 DESIGN OF DIVIDE WALLS

3.12.1 Location and functions of the divide walls

Divide walls are structures provided at right angles to the axis of the diversion headwork on the upstream side of a weir and extends from crest to a bit longer than intake structure or up to end of upstream impervious apron. Sometimes these walls also extend to the foot of downstream sloping face of the weir i.e. the beginning of launching apron when there is a bridge over the weir or even it extends down to the basin end sill when we are interested to provide a barrier between the stilling basin and scouring bay, so as to avoid cross-currents.

These walls are provided between two portions i.e. under sluices portion and weir portion to minimize flow current. Thus, the main functions of divide walls are:

- To form a still water pocket in front of the canal head so that the suspended silt can be settled down which then later can be cleared through the scouring sluices from time to time,
- To control the eddy current or cross current in front of the canal head,
- To provide a straight approach in front of the canal head,
- To resist the overturning effect on the weir or barrage caused by the pressure of the impounding water,
- To separate the turbulent floodwater from the pocket in front of canal sluice,
- To check parallel flow that would be caused by the formation of deep channels leading from the river to the pocket in front of the sluices, and,
- To support gate operation slab, if the gate is rotary/spindle type (if it is sliding type, no slab is required of-course).

3.12.2 Design consideration of the divide walls

- The length of these walls should extended atleast beyond the upstream end of the head regulators on the upstream side, just to cover the canal head regulator and on the downstream side, it is extended up to the launching apron;
- The dividing wall is built at right angles to the axis of the weir, separating the weir and the under-sluices;
- The dividing wall can be designed from masonry or concrete walls. However, the choice is dependent on availability of construction materials around the project area;
- Their height is as high as that of wing walls to serve as support for RCC operating desk thus extends from river bed up to the top level of upstream wing walls. However beyond of the operating slab deck, the height can be reduced to the pool level or crest level for economic reason;
- It should be designed at a right angle to the axis of a weir or a barrage to maintain stable streamlines of flow to the pocket/groyne of sluices;
- The foundation of the divide wall is to be kept well below the raft or floor and have their own foundation, and shall depend upon the scour depth.

3.12.3 Fixing section of a divide wall

Its length range lies between 0.5 to 2m depending on the magnitude of flood. The divide Wall height can be fixed using the most governing parameters of upstream high flood level and the weir bank condition. Therefore the height of the divide wall is the difference between the HFL and the river bed level plus some free board on the u/s or to the top of the weir in case when there is no rotary gate but operated by sliding.

Also downstream wall height can be fixed by subtracting the foundation or river bed level from the downstream HFL and adding free board of that range. Divide wall thickness is fixed by considering the wall height, the load's acting on the wall and the materials proposed for wall construction (i.e. concrete or masonry wall).

3.13 COFFERDAM DESIGN ASPECTS

3.13.1 Functions and location of a cofferdam

A cofferdam is a temporary retaining embankment (as in case of diversion weirs) or permanent structure (as in case of dams) that has to be designed and built in a river both on the upstream and downstream side of construction zone (for weirs as shown in figure below, not to let water come back in to construction zone) or on the upstream side alone (based on slope of the river) or to enclose an area during construction to withstand consecutive flood seasons (i.e., for the expected years of completion of construction of the main structure) and offer protection against expected return period flood.

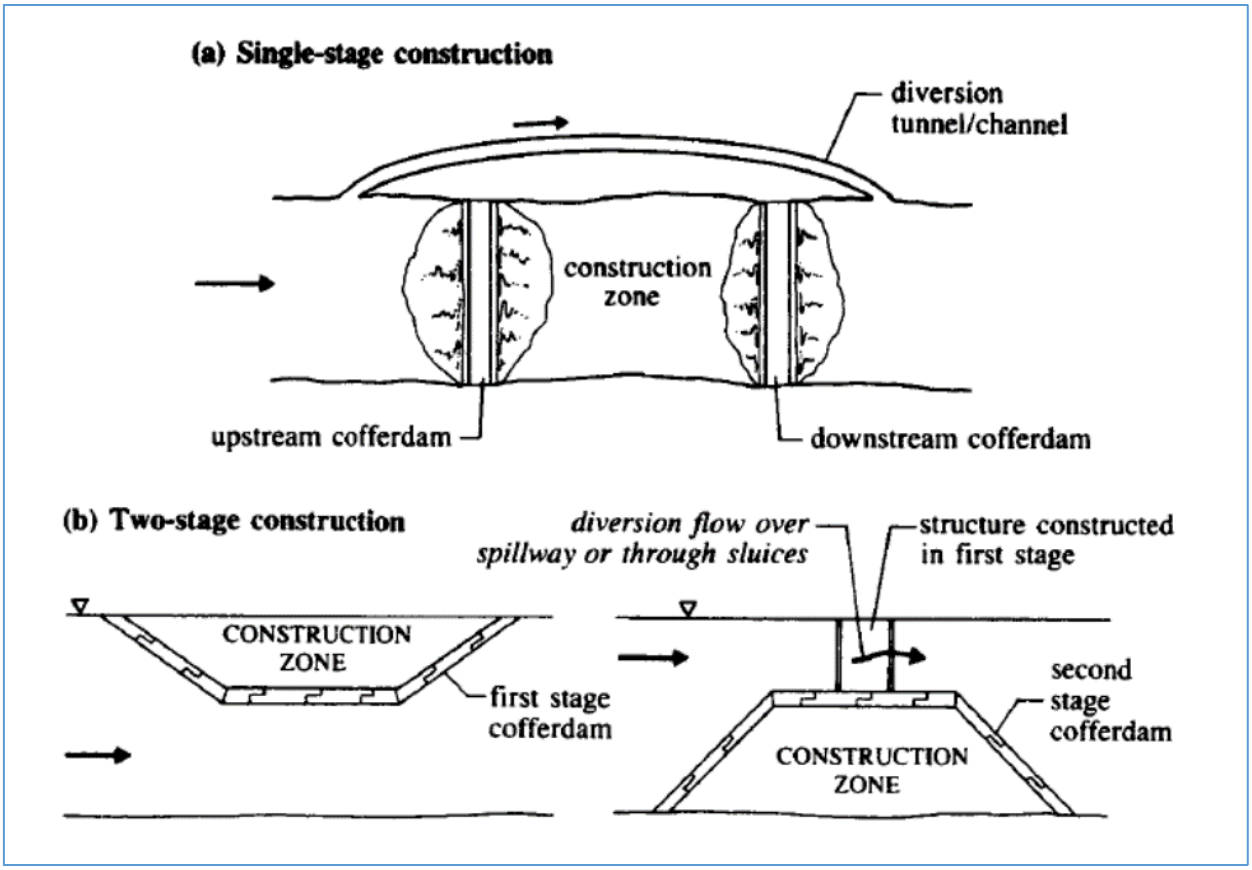


Figure 3-35: Typical arrangement of cofferdam around construction zone

3.13.2 Construction materials of a cofferdam

Generally, cofferdams must be as watertight as practicable, relatively cheap and, if possible, constructed of locally available materials. It is constructed from earth (rolled), rock or steel sheet materials. Metal sheet piling coffer dams which are usually reinforced with timber or earthfill is also common.

3.13.3 Design considerations of a cofferdam

Dewatering could prove to be impossible during excavation for the foundation because of water path in the trail of the thalweg where stone and gravel layers exist, and/or the lower layer below the cofferdam is broken by piping. In such case, riverbed deposits should be investigated based on geological data and information in order to prevent construction failure.

3.13.4 Design considerations of a cofferdam

Dewatering is less effective during excavation for foundation of the diversion headwork. The riverbed deposits should be properly investigated in order to get enough geological data and information so that construction failure is prevented.

Design of an adequate cofferdam involves concerns with economics. When the construction is planned so that foundation work can be executed and completed during the low river water season or dry season, the cost of cofferdams can be to the minimum by providing smaller height cofferdam. However, where the stream flow characteristics do not allow to do so or construction work period exceed the dry season period, construction of smaller cofferdam is not possible option because it is not safe. The construction of high cofferdam due to fear of risk of flooding is not economical. Therefore, an optimum cofferdam height should be determined and constructed based on safety and economic factor. This involves hydrological analysis to determine the magnitude of the flood with probability of exceedance.

Usually 50 years return period flood magnitude is considered for fixing the crest level of the coffer dam and temporary river diversion channel. However, depending on the prevailing site condition the design flood magnitude for the cofferdam can be minimized.

Depending on the prevailing condition at the selected diversion headwork site, the cofferdam can be either single stage or two stage construction type (Figure 3-35).

In the case of single stage construction the construction zone can be cleared from river flow using the followings components of temporary work

- upstream coffer dam
- downstream cofferdam
- river diversion channel
- rock riprap protection works at the outlet of diversion channel

In the case of a two stage construction the construction zone can be cleared from river flow using the followings components of temporary work

- first stage cofferdam isolating either the left or right half of the river channel with enough working space to carry out construction of part of the diversion headwork
- second stage cofferdam isolating the remaining half of the river channel with enough working space to carry out construction of remaining part of the diversion headwork
- river diversion channel is not required

The design of the cross section (geometry) of the cofferdam shall consider safety against piping and uplift.

3.14 HYDRAULIC DESIGN ASPECTS OF RETAINING/WING WALLS

3.14.1 Arrangements and functions of retaining/wing walls

Retaining walls are structures provided with wing walls flanking the diversion headwork at the upstream and supporting the adjoining earth bunds.

Wing walls have at least the following functions:

- As flood protection walls: To constrict and control design flood levels within the river banks so that there may not be submergence and inundation on the back of the diversion and crossing structures as a result of construction of such burrier structure across the river,
- As retaining structures: To restrain/hold or retain soil, rock or other materials behind it so that the river bank slopes get stabilized thus may not be collapsed as a result of the scouring created by hydraulic jump,
- They can also be used for protecting conveyance canals running on steep landscape from being eroded and sliding.

Wing walls are arranged both on the u/s and d/s of the weir body and both on the left and right banks of the river right from end of u/s impervious apron up to the end of stilling basin but being keyed to the natural ground such that, these keys should find design flood levels on all directions.

The wing walls and existing abutment walls may coincide or not depending on required intensity of flow we are interested in. if incoming design flood is high, the abutments need to be widened and if smaller, then the abutments need to be constricted so that allowable intensity of design flood can pass safely.

3.14.2 Design considerations for wing walls

Wing walls are retaining structures designed on river banks/abutments along with diversion structure such that there will not be overtopping of flood level for the expected design discharge of known return period, which is usually Q_{50} for SSI Projects as a result of introduction of such barrier structure. Thus, the flood protection wall height is determined based on the high design flood levels with some free boards. It is commonly designed from masonry walls (refer section 2-7 for details of more retaining structures).

Hydraulic design aspects of this structure should consider free board on top of Q_{50} flood level. In addition to this, wing walls are also required on downstream side of the crest so as to protect the scouring of the banks due to formation of jumps. Usually a free board on upstream is taken 0.4-0.5m and downstream one is 0.5-0.6m but at least 1.05 d_2 is recommended due to jump.

Retaining walls shall be designed to withstand lateral earth and water pressures, the effects of surcharge loads, the self-weight of the wall and in special cases, earthquake loads. If the retained earth is not allowed to drain, the effect of hydrostatic water pressure shall be added to that of earth pressure and hence a weep hole is provided for draining (for detailed stability analysis of wing walls, refer section 3-15-12).

Note: Provisions of wing walls both on the u/s and d/s may not be required if geology of the abutment on both sides of the bank is rocky, stable and deep enough for accommodating incoming design flood level.

3.14.3 Selection of wall type

Selection of appropriate wall type is based on:

- An assessment of the design loading,
- Depth to adequate foundation support,
- Presence of deleterious environmental factors,
- Physical constraints of the site,
- Cross-sectional geometry of the site both existing and planned,
- Settlement potential,
- Desired aesthetics,
- Constructability,
- Maintenance, and
- Cost.

3.14.4 Data required

Data required for hydraulic design of wing walls are:

- Weir wall height, h
- Head over the weir, h_d
- Sequent depth, D_2
- U/s river bed level,
- D/S High flood level, (D/S HFL)
- U/S High flood level, (U/S HFL)
- Depth of depression
- Stability of the abutments on both banks of the river and its foundation
- Topographic conditions (Map and coverage conditions)

Box 3-12:
Worked Example-12: *Determine top levels of upstream and downstream wing walls for the structure designed in preceding worked examples.*

Solution*: From worked example-3, for the upstream wing walls H_e=1.698m; Weir Crest Level has been fixed to 1434.075m a.s.l.; flow* depth on the crest, H_d is 1.622m *and* approach velocity head, $H_{av}=H_e-H_d$ = 0.08m. Thus taking free board of 0.4m, u/s wall top level= 1434.075+1.622+0.08+0.4 =1436.173m

For the downstream wing walls, top level of the downstream wing walls = downstream river bed level + TWD + d/s energy head ($hv_3=V_3^2/2g=(q/D_3)^2/2g=(q/TWD)^2/2g$)+ freeboard =1432.475+0.98+0.76+0.5=1434.709m.

3.15 STRUCTURAL DESIGN OF DIVERSION WEIR

3.15.1 Background

The objective of this section is to lay basis for consideration in structural design of weir/barrage, wing-wall, breast wall, operating deck and gate structures for irrigation projects, such as flat or sliding gates, spindle and radial gates.

3.15.2 Structural design considerations for weir and wing-wall

It is inevitable that, irrigation structures are subjected to different types of loading with varying magnitudes. These structures should thus withstand such loads, with tolerable damage and deliver the services requirements for which they are designed for. Usually, the ultimate limit state design approach has been followed to design such structures. Under limit state design approach, structures are designed for service as well extreme loads. For the service loads, especial attention has to be paid to limiting the crack width of the structures because the structures are exposed to water with high pressure.

This section thus gives considerations in structural design aspects of a weir/barrage and wing-wall sections which involves stability and sizing of the structure based on hydraulic requirements stated in previous sections and structural requirements. The structural design of weirs is mainly related with the nature of flow and foundation conditions.

Such analysis is made for a masonry and/or concrete wall imbedded in cement mortar like for retaining/wing wall, guide wall, divide wall, piers, river training and other related purposes. Gravity walls are usually stabilized by their own weight. But checking for tension in critical sections of masonry walls are necessary as it is weak intension. In case of concrete it can be alleviated by introduction of reinforcement bars.

All structures should be checked for the safety against stability and stress conditions. The major factors involved in the structural design aspects are construction materials, various loads to be considered and factor of safety to be adopted. According to our current construction industry practice, for small scale irrigation project construction, stone masonry and reinforced concrete are the dominant structural units used for weir type head work design. Steel structures are also in use for gates, ladders and railing purposes.

3.15.3 Structural design aspects of stone masonry

3.15.3.1 General

In most cases of small scale irrigation scheme diversion headwork structure construction, the use of stone masonry structural as side soil retaining work is quite common practice. Because, compared to other construction materials, masonry is relatively cheap and easy to work with and can be constructed with locally available materials. One major disadvantage of masonry work is however, that its capacity to withstand 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 3-24 above. Active earth pressure on such structure shall be calculated based on Equation (3-146) below.

3.15.3.2 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.

3.15.3.3 Stability analysis of masonry wall

This has been presented in detail under section 3.15.12.

3.15.4 Types of gates used in irrigation

The followings are types of gates which are commonly used in irrigation projects:

- Vertical Lift Gates: such as Fixed wheel/roller type gate, Rotary/Spindle gate, Slide gate, Double leaf gate and Stop-log;
- Hinged Type Gates: such as Radial gate, Sector gate, Mitre gate, Flap gate and Swing gate. These types are typically used in large scale irrigation projects.

Out of these different types of gates, the most commonly used gates for small scale irrigation projects are vertical lift gate type. On account of their low cost, small slide gates are used everywhere for passing discharges up to a few cubic meter per second. The problem with them are however, they are prone to frequent jamming thus require careful operation.

Type of gate is decided after considering its purpose, installation location, cases of operation, safety, dependability and economy of water intake function, especially from the view point of effective usage of the water resources, appropriate style and operation method to reduce over diversion.

Fixed wheel type gate is mechanically and structurally simple. Hoisting load is also lighter than for slide gate and is more dependable. It is the most frequently used gate for barrages. In this type of gate, hydraulic load is transmitted to a horizontal main girder through the skin plate (sheet metal) and its supporting girder. The load is finally transmitted to the guide frame by way of vertical end girders at each side of the gate leaf and wheel.

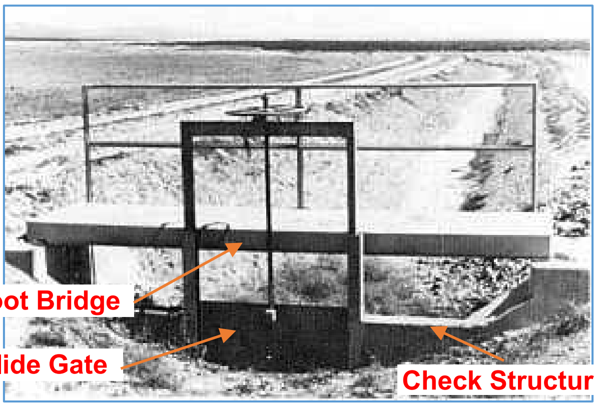
Slide gates can be used both for diversion weir and on-farm structures like division box and turnouts. This type of gate is suitable for a relatively small span and water level difference. The mechanism is simple as a metal plate can be used for guide frame. Operation under hydraulic load causes a large load for hoisting since the gate leaf has to slide on the guide frame. Thus, this type is not suitable for large gate leaves unless operated under balanced water pressure.

For the on-farm structures, it is used as undershot or overshot and are a common form of structure used for turnouts. Such gates should be fixed in a concrete structure which can be precast to ensure high quality. They do not normally have a regulating function other than diverting the flow in an on/off situation. Indeed, there is little point in using them as variable flow structures, because farmers will generally use them either fully open in order to maximize the flow, or else shut when they do not require irrigation water.

Radial gates are typically used in sizes up to 5 m wide with a capacity of up to 40 cubic meter per second each.

Stop-logs are not commonly self-stand gates but provided as reserve gates to be used during maintenance of main gate. Stop-logs are less expensive than gates and are more quickly adjusted but do not control the flow as closely.

Flap gate is a simple hydraulic automatic upstream water level control gate. Its simplicity is derived from ease of construction and maintenance-construction only requires flat plate and tubing fabrication, rather than curved surfaces as for other types of hydraulic automatic gates. Once installed and proper operation is verified, flap gate only requires lubrication of its bearings and occasional painting for maintenance.



		Upstream water height of gate (m)								
		1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
Skin plate (Height x Width)	1.0 x 1.0	6	6	7	8	9	10	10	11	11
	1.5 x 1.0	-	7	8	9	10	11	12	12	13
	2.0 x 1.0	-	-	8	9	10	11	12	12	13
	2.5 x 1.0	-	-	-	9	10	11	12	12	13
	3.0 x 1.0	-	-	-	-	10	11	12	12	13
	1.0 x 2.0	7	8	9	11	12	12	13	14	16
	1.5 x 2.0	-	11	13	14	16	17	18	19	20
	2.0 x 2.0	-	-	16	16	18	19	21	22	23
	2.5 x 2.0	-	-	-	18	20	21	23	24	26
	3.0 x 2.0	-	-	-	-	21	22	24	25	27

Note: Inside of □ is thickness of skin plate (mm)

Figure 3-36: Check structure with slide gate (Thicknesses are in mm)

Figure 3-37: Washout gate operated by Spindle (L) and sluice gate by chain block (R)

3.15.5 Design considerations of gates

3.15.5.1 Materials of gate structures and precaution to be taken

Such water control gate structures are designed with heavy brass thrust nuts on the rising spindle gates set in to concrete or plastered stone masonry. Steel gates invariably leak around the sides and base, contributing to seepage flows and leakage losses. The main problem is that the close tolerances required for gate seals can rarely be achieved during construction in the field. Thus, precaution should be taken care of it so as to obtain the desired design flow.

Materials used for gate structures comprise steel, aluminum, stainless steel, rubber and FRP (fiberglass reinforced plastic). The selection is based on different flow conditions approaching to gate. When using materials other than steel for gate, the characteristic features of the material for gate should be studied carefully.

Gate structures consist of a gate leaf, guide frame, pivot and hoisting equipment. The gate leaf is the part which receives the hydraulic load and conveys it to pivot. The guide frame is the embedded part in concrete and adjacent to the sealing part of the leaf to prevent water leakage.

The pivot is part of a hinged type gate which transmit the external force (load) to the concrete. 'The guide frame covers this function for a vertical lift type gate. The hoist is the equipment which operates a gate leaf.

Slide gates are designed and fabricated from special shaped extrusions or structural angles, flats, and plates are assembled by welds and bolts. Since there are no machined parts or wedging devices in such gate itself, the gate depends upon water pressure and seal design to seat the fabricated slide. Fabricated slide gates are usually furnished with rubber seals to improve water-tightness. Head capacity is dependent on opening size and availability of structural members.

The gates recommended for SSI project should meet the following requirements:

- Should be reasonably watertight. Leakage if any unless otherwise specified shall not exceed 5 to 15 litres per minute per meter length of periphery of the sealing surface.
- Shall be rigid, smooth, straight and without offset at joints.
- Bottom shape of the gate shall be suitably designed to minimize down pull and to provide a converging fluid way.
- Slot shall be as narrow as possible in conformity with structural safety of the gate leaf.
- The gate as a whole shall be capable of being raised or lowered by the hoisting· mechanism provided.

3.15.5.2 Structural design aspects of sliding gates

The structural analysis of slide gates is part of electromechanically design work, however the structural design shall provide the provisions and structural detail such as get sliding steel channel embedding concrete groves, appropriate thickness of grove sides and reinforcement detail designed to withstand the hydrostatic and hydrodynamic pressure plus force induced during gate operation activity.

With respect to the gate sizing for both sluiceways and canal intakes, it is preferable to keep the gate width narrower, and use narrower gates than less wider gates. Gates (and the associated lift) wider than about 0.6 to 1 m are more expensive to construct, and can present operation and maintenance difficulties. Generally each gate is operated by a centrally located single stem screw lift. The sill/invert of the canal intake gate(s) should be as high as possible and at least 0.5 to 1 m higher than the sluiceway sill/invert. A number of relatively narrow/small canal intake gates with weir flow (as opposed to orifice flow) may be required. This will decrease the potential for future canal intake gate siltation problems, and reduce maintenance costs.

3.15.6 Design considerations of breast wall

3.15.6.1 Arrangement of breast wall

Breast wall is an RCC thin wall structure arranged vertically and designed to avoid spilling of water over the under sluice structure during HFL and canal regulator gate. It is provided from top level of the gate up to the HFL. It is also used as a runner for the under sluice spindle gate.

3.15.6.2 Structural aspects of breast wall

Breast wall is constructed out of reinforced concrete. The minimum thickness required for head work and scouring sluice breast walls are indicated in Table 3-29 above. The critical loading condition for the beast wall is during design flood flow condition. The hydrostatic force is a triangular force distribution nil at wall top that is at high flood mark and unit weight of water times breast wall height at beast wall bottom.

The hydrodynamic force can be computed as unit weight of water multiplied with approach flow velocity and hypothetical discharge over the breast wall area. Once the wall dimensions, end fixity condition with the side piers and the imposed loads and are known, the analysis can be easily managed by SAP-2000 software.

For the design, the critical actions to be investigated are the bending moments at span and at wall ends and the shear force at wall ends. Reinforcement therefore, can be designed with the limit state design for flexural and shear resistance design equations provided in previous sections. The reinforcement provided finally shall be checked if it is well above the minimum requirement, if otherwise the minimum reinforcement requirement will govern.

3.15.7 Design considerations of gate operating deck

3.15.7.1 Arrangement of operating deck

Operating deck is an RCC thin slab structure arranged horizontally and designed to serve as a bridge for operation of gates.

3.15.7.2 Structural aspects of operating deck

Operation slab is made to have dimensions of commonly 1m long by 3m wide for sluice and intake operation or as found necessary to make the operator free from risk during operation by enclosing it within hand rails drilled on both of its edges.

Thicknesses of the slab and breast wall are simply determined from practical recommendations of:

$L/t = 20$ to 35 ………………………………………………………………………… (3-145)

Where, L = length of greater span = 3.0m
t = thickness (m)

However, t =0.10 to 0.20m thicknesses are provided for the slab and breast wall work. And provide the reinforcement bar of φ12mm diameter spread out @ 150mm c/c spacing in all directions with reinforcement cover of 20mm.

The minimum slab thickness provided for gate operation slab is 200mm as indicated in table 3-29 above. However the thickness shall be checked to be satisfactory by conducting the analysis and design of the deck slab. In addition to the thickness of the deck slab the other geometrical data required for structural analysis is the lateral dimensions i.e. slab span and width over the supporting piers and the end restraint conditions shall be also defined based on the actual fixity condition provided in the design. The imposed operation live load recommended is $7.2KN/m^2$ which is presented in previous section, load on operation deck. The structural analysis and design will then be carried out in similar manner as explained for breast wall design.

3.15.8 Bedding under structures

Bedding, which are normally made of granular materials or concrete, serves four main functions:

- To enhance a uniform support under pipes in order to reduce the bending moment longitudinally;
- To increase the load-supporting strength of the pipes;
- For pipes with spigot and socket joints, it enables pipes to be supported along pipe lengths instead of pipe sockets. Otherwise, uneven stress may be induced and it may damage the pipes;
- To provide a platform for achieving correct alignment and level during and after construction.

3.15.9 Common loads on irrigation structures

The principal load which should be considered for structural design of components of a diversion headwork are self-weight, earth pressure, water pressure including uplift, imposed live load such as live load at get operation platform, earth quake load and wind load. For small scale irrigation scheme the effect of earth quake load and wind load are usually negligible and are not considered, if outside of seismic zone.

Operating decks for the head regulator and scouring sluice area of the weir part shall be designed for a uniform live load of 7.2 KN/m^2 (Design of Small Canal Structures, 1978).

Stability of body of the diversion weir wall above foundation slab is checked for two conditions: one is for condition of high flood level on the upstream and the other is for condition when water level is at crest level, i.e. no over flow condition for the downstream.

Foundation could be permeable or impermeable. The structural analysis design for both foundation conditions is the same except that weirs on permeable foundation need additional analysis regarding piping and uplift pressure. Therefore, these aspects have to be properly analyzed.

3.15.10 Forces acting on a weir and wing wall body

Before going to analyze stability of hydraulic structures, it is essential to identify the forces acting on such structures. Accordingly, expected forces which act on a weir body both on the surface and from subsurface direction can be categorized in to: Static water pressure, Uplift water pressure, Deposited Silt Pressure; Soil reaction at the weir base (on foundation), Friction forces at the base, Self-weight and water wedge, Dynamic force unburden, and Seismic force (if the structure is situated in the seismic zone).

These Loads can be classified in terms of applicability/relative importance in to three:

a) Primary Loads: are identified as those of major importance to all weirs, irrespective of type, e.g. water and related seepage loads, and self-weight loads.
b) Secondary Loads: are universally applicable although of lesser magnitude (e.g. sediment load) or, alternatively, are of major importance only to certain types of weirs (e.g. thermal effects within concrete weirs).
c) Exceptional Loads: are so designed on the basis of limited general applicability or having a low probability of occurrence (e.g. tectonic effects, or the inertia loads associated with seismic activity).

Dynamic force: This is a water reaction acting on the weir but supposed negligible as water behind the weir builds up gradually and filled with water and/or silt up to crest, however it could be considered if the designer found it necessary.

Silt brought by runoff commonly gets deposited against the upstream face of a weir just after construction. If h_s is the height of silt deposited, then the force exerted by this silt in addition to external water pressure, can be represented by Rankine's formula as:

$$P_S = \frac{1}{2} * K_a * \gamma_s * h_s^2 \quad \text{(3-146)}$$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} \quad \text{(3-147)}$$

Where, P_s is force exerted by deposited silt and it acts at $1/3h_s$........................ (3-148)
Ka is the coefficient of active earth pressure, as shown below
h_s is height of deposited silt, m
γ_s is submerged unit weight of silt material, KN/m^3
ϕ = Angle of repose or angle of internal friction of soil

Table 3-21: Summary of common forces acting on weir & wing-wall at different condition

Structure	Load\Flow Condition	No Flow Condition	Static Condition	Dynamic Condition	Remark
Weir critical section(Refer figure 3-31)	Water pressure on u/s face, P_{h1}	No	Yes	Yes	
	Water pressure on top, P_{h2}	No	No	Yes	Force on weir crest
	Water pressure on d/s face, P_{h3}	No	No	Yes	
	Self-weight	Yes	Yes	Yes	Force
	Silt/sediment load	No	Yes	Yes	
	Uplift	No	Yes	Yes	
	Seismic load	Yes	Yes	Yes	
U/s Wing-wall (Refer figure 3-32)	Water pressure on river side, P_{h1}	No	Yes	Yes	
	Self-weight	Yes	Yes	Yes	
	Earth pressure from back	Yes	Yes	Yes	As per shown figure
	Uplift pressure	No	Yes	Yes	
	Seismic load	Yes	Yes	Yes	
D/s Wing-wall (Refer figure 3-32)	Water pressure on river side, P_{h1}	No	No	Yes	
	Self-weight	Yes	Yes	Yes	
	Earth pressure from back	No	Yes	Yes	Similar to u/s wall but differ in height and base
	Uplift pressure	No	No	Yes	
	Seismic load	Yes	Yes	Yes	

Note: - No Flow Condition is when there is no flow in the river; Static Condition is when there is no-overflow i.e. water is at WCL; Dynamic condition is when flow is at its design capacity.
- Refer figure 3-31 for forces acting on weir and figure 3-32 for those on Wing-walls.
- Seismic impact should be considered based on the delineated seismic zones in Appendix-VI.

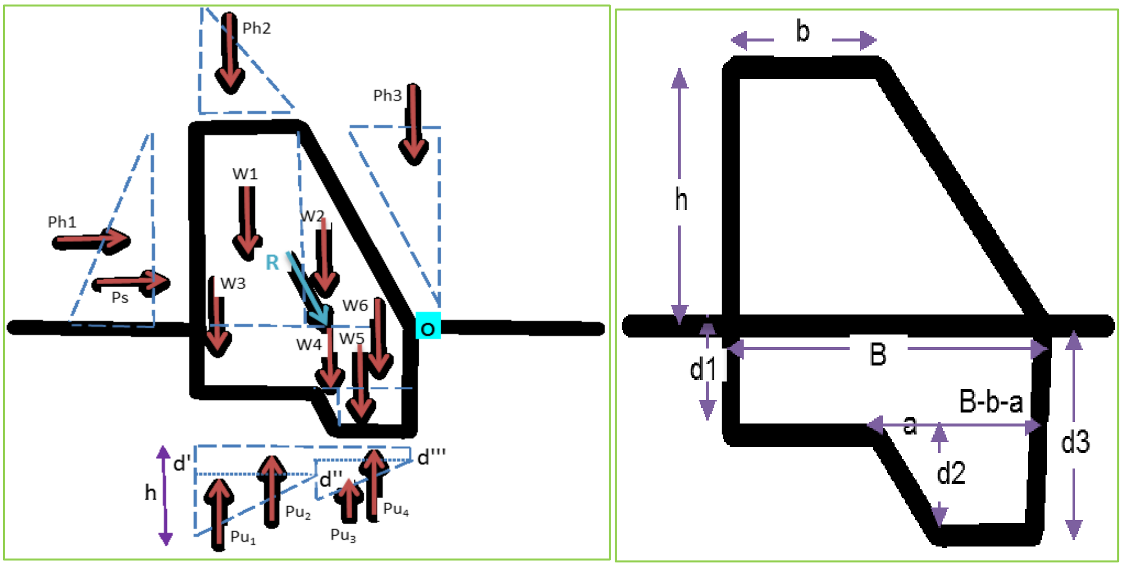


Figure 3-38: Forces acting on a Critical Section of a Weir

These different forces acting on weir body can act together simultaneously thus can be combined into a single resultant force, R, that can ultimately lie either inside or outside of weir body. However, for a stable structure it has to remain within the middle third of base of the structure. These forces are presented for full flow condition; however, for no overflow condition, Ph_2 and Ph_3 and corresponding moments will not exist.

3.15.11 Stability analysis of the weir

3.15.11.1 General

The objective of stability analysis of a diversion weir is to keep the compressive stresses within the allowable limit, and prevent the development of any tension stresses in the concrete. Unreinforced concrete or possibly stone masonry is weak in tension, the ultimate tension strength being only one-tenth of the ultimate compressive strength. Thus it is considered unwise to permit any tension stresses whatsoever. To achieve this, the “middle third rule” is adhered to, where the resultant of all the forces is maintained in the middle third of the structure. If the reinforced concrete apron is connected to the weir, which is required to resist apron uplift, overturning will generally not be a problem for the typical diversion weirs for small scale irrigation projects. Such analysis is made by considering forces acting on the structure and possible moment resulting from these forces which is taken about external toe of critical section of a weir as shown above bit internal toe for wing walls.

The objective of a stability analysis in general is to maintain horizontal, vertical, and rotational equilibrium of the structure (USACE, 2005). For sliding and bearing, the stability requirements have been expressed deterministically in terms of an explicit factor of safety that sets the minimum acceptable ratio of foundation strength along the most critical failure plane to the design loads applied to the failure plane. The analysis for determination of the resultant location in prior guidance has been termed an overturning stability analysis. This is a misnomer since a foundation bearing, crushing of the structure toe, and/or a sliding failure will occur before the structure overturns. The section of the weir shall satisfy the requirements of no overturning, no sliding and no settlement. A minimum factor of safety need to be set in stability and structural analyses because of the potential variability in loads and material strengths. The factor of safety required to be higher when site information is so limited.

When analyzing stability conditions of structures, we need to consider economic aspects as well. Thus, we need to design a structure which is economical and stable. For the diversion structure to be kept under stability condition, the following conditions must be fulfilled:

- The structure must be safe against sliding, on any plane or combination of planes within the structure, at the foundation or within the foundation;
- The structure must be safe against overturning at any plane within the structure, at the base, or at any plane below the base;
- The resultant force acting on the structure must lie within the middle third of the base;
- There should not be tension under the base of the structure;
- The maximum toe and heel pressures on foundations should not exceed the prescribed safe limits i.e. the safe unit stresses in the body of the structure or in the foundation material shall not be exceeded.

3.15.11.2 Stability against sliding

A weir should be stable against sliding at the base for different load conditions. The stability against sliding is a function of the shear strength of the construction materials. It is given by:

$$F_S = \frac{\eta \sum F_V}{\sum F_H} > 1.5 \text{ or } F_S = \frac{\sum F_H}{\sum F_V} < 0.75 \qquad (3\text{-}149)$$

Where, Fs is Factor of safety against Sliding and it should be greater than or equal to 1.5.

η is coefficient of friction b/n the material and the horizontal section and its value varies b/n 0.65 to 0.75 up on the materials used.

$\sum F_V$ is summation of vertical forces

$\sum F_H$ is summation of horizontal forces

3.15.11.3 Stability against overturning

Factor of safety against overturning, F_0, in terms of moments about the downstream toe of the weir is given by:

$$F_o = \frac{\sum M(+)}{\sum M(-)} > 1.5 \qquad (3\text{-}150)$$

Where, F_o is Factor of safety against overturning

$\sum M^{(+)}$ is summation of stabilizing moment

$\sum M^{(-)}$ is summation of overturning moment

F_0 should be greater than or equal to 1.5.

3.15.11.4 Safety against Tension

For no tension on the base of the head work structure, for critical section, the resultant force, R should act at the middle third part of the critical section. This implies that the eccentricity (e) should be less than or equal to one-sixth (1/6) of the base width (b) of the weir at the critical section.

$$R = SQRT((\sum F_H)^2 + (\sum F_V)^2) \qquad (3\text{-}151)$$

$$\bar{x} = \sum M / \sum Fv \qquad (3\text{-}152)$$

$$e = \bar{x} - B/2 \qquad (3\text{-}153)$$

Where, R is the resultant force, (KN)

$\bar{x}$ is arm length of resultant force from toe, i.e. centroidal distance, (m)

e is eccentricity, (m):

For the structure to be safe, the eccentricity 'e' should satisfy the following condition otherwise overturning may occur if the resultant R fell outside of the base:

$$e = \left| \frac{\sum M}{\sum F_v} - \frac{B}{2} \right| < \frac{B}{6} \quad \text{.......... (3-154)}$$

Where, $\sum M = \sum M^+ - \sum M^-$ .... (3-155)

$\sum M^+$ and $\sum M^-$ as defined above

B is bottom width of the structure (m)

3.15.11.5 Safety against bearing capacity of foundation

Also called safety for contact pressure or vertical stress. Here it is assumed that the distribution of vertical stress between foundation and bed of structure is linear. The required condition in this case is, if the magnitude of contact pressure at the base of the structure is less than allowable foundation material, is safe against settlement this is it is within safe limit of the crushing strength of the masonry or concrete. Thus, compressive stress or vertical stress, $P_{max/min}$ can be estimated with the trapezoidal law, shown by the following equation:

$$P_{max/min} = \frac{\Sigma F_v}{B}\left(1 \pm \frac{6e}{B}\right) \quad \text{.......... (3-156)}$$

Where, P_{max} is the maximum compressive stress or vertical stress, KN/m^2

P_{min} is the lower limit compressive stress or vertical stress, KN/m^2

3.15.11.6 Safety against buoyancy

This is the ability that weight of the material can resist the uplift pressure exerting from the bottom of the structure. It is given by the equation:

$$\frac{\Sigma W_m}{\Sigma P_x} > 1.2 \quad \text{.......... (3-157')}$$

Where, W_m is weight of the material

P_x is the uplift pressure

3.15.11.7 Safety against seismicity

Seismic impact especially on any elevated structure is high, therefore a seismic coefficient should be adopted in the design activities depending on the delineated seismic zones. An earthquake, which is a violent shaking of the earth's crust, may be treated as a reversing horizontal acceleration. Due to the inertia of the weir, it tends to resist the motion, and the stresses in the weir and foundation may increase momentarily. In the static loading method of analysis, the motion is replaced by the equivalent inertia force S_e applied at the center of gravity of the weir, and is given by:

$S_e = W*a$ …………………………………………………………………………….. (3-157)

Where, W- is the weight of the weir and
a- is earthquake intensity factor

Earthquake intensity is expressed in terms of the factor “a” which is the ratio of earthquake acceleration to the acceleration due to gravity. Design values for “a” ranges from 0.0 to 2.0 for Ethiopia, the largest being in the rift-valley (for details refer Appendix-VI).

Table 3-22: List of possible forces and moments acting on weir section (full flow condition)

Description	Vertical Forces	Lever Arm	Moments	Remark
Weight of structure				
Weight of top rectangular part of weir, W1	$W_1=b*h*\gamma_m$	$L_1=b/2+(B-b)$	$M_1 = W_1*L_1$	(+) Moment about external toe, O
Weight of top triangular part of weir, W2	$W_2=1/2*(B-b)*h*\gamma_m$	$L_2=B/3$	$M_2 = W_2*L_2$	(+)
Weight of bottom rectangular part of weir, W3	$W_3=(b+a)*d_1*\gamma_m$	$L_3= b/2+(B-b)$	$M_3 = W_3*L_3$	(+)
Weight of bottom rectangular part of weir, W4	$W_4=1/2*a*d_2*\gamma_c$	$L_4=B-b-a+a/3$	$M_4 = W_4*L_4$	(+)
Weight of bottom rectangular part of weir, W5	$W_5=(B-b-a)*d_2*\gamma_c$	$L_5=(B-b-a)/2$	$M_5 = W_5*L_5$	(+)
Weight of bottom rectangular part of weir, W6	$W6=(B-b-a)*d_1*\gamma_c$	$L_6=(B-b-a)/2$	$M_6 = W_6*L_6$	(+)
Static water pressure				
u/s water pressure, Ph1	$Ph_1 = 1/2* h*h*\gamma_w$	$L_7=1/3*h$	$M_7 = Ph_1*L_7$	(-)
Top water pressure, Ph2	$Ph_2=1/2* h_d*h_d*\gamma_w$	$L_8=((B-b)+2/3b)$	$M8 = Ph_2*L_8$	(+)
d/s water pressure, Ph3	$Ph_3=1/2*h*(B-b)*\gamma_w$	$L_9=1/3*(B-b)$	$M9 = Ph_2*L_9$	(+)
Uplift pressure				
Uplift pressure, Pu1	$Pu_1 =1/2*b*(h-d')*\gamma_w$	$L_{10}=2/3*b+(B-b)$	$M_{10} = Ph_1*L_{10}$	(-)
Uplift pressure, Pu2	$Pu_2 =b*d'*\gamma_w$	$L_{11}=1/2*b+(B-b)$	$M_{11} = Ph_1*L_{11}$	(-)
Uplift pressure, Pu3	$Pu_3 =1/2*(B-b)*d''$	$L_{12}=2/3*(B-b)$	$M_{12} = Ph_1*L_{12}$	(-)
Uplift pressure, Pu4	$Pu_4 =(B-b)*d'''$	$L_{13}=1/2*(B-b)$	$M_{12} = Ph_1*L_{13}$	(-)
Silt pressure				
u/s silt pressure, Ps	$Ps =1/2* h_s*h_s*\gamma_s$	$L_{14}=1/3*h_s$	$M_{14} = Ph_1*L_{14}$	(-)

Note: - Parameters a, b and B are as indicated in figure 3-31and others have their usual meaning; h_s=silt height.
- Based on the these formulae and the identified common forces acting on weir and wing-walls under different flow conditions in table 3-18, stability of the structure can be checked for all the conditions i.e. for no flow condition, static i.e. no-overflow condition or when water is at WCL and dynamic condition. (For details refer the accompanied excel templates.

3.15.12 Stability analysis of retaining and flood protection walls

In the same way to stability of weir, gravity retaining walls should also be designed to provide adequate stability against sliding, overturning, foundation bearing failure and if applicable deep foundation failure due to seepage. The forces acting on the wall must be identified at first and then moments on the wall shall be determined with reference to front toe of the wall (unlike weir body).

Forces acting on such walls are the lateral earth and water pressure, weight of wall and any soil on the wall; surcharge loading to account for items such as heavy equipment next to the wall (if any); and, in some instances uplift pressures and horizontal seismic loading if applicable with respect to diversion weir loads. The following figures show the generalized forces on gravity retaining walls.

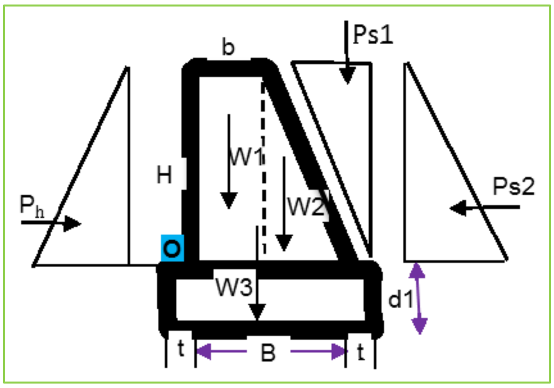


Figure 3-39: Forces acting on wing wall or retaining wall

Table 3-23: Possible Forces & moments acting on wing wall Section (full flow condition)

Description	Vertical Forces	Lever Arm	Moments	Remark
Weight of structure				
Weight of rectangular part of wing wall above RBL, W_1	$W_1=b*H*\gamma_m$	$L_1=b/2$	$M_1 = W_1*L_1$	(+) Moment about internal toe, O
Weight of triangular part of wing wall above RBL, W_2	$W_2=1/2*(B-b)*H*\gamma_m$	$L_2=b+1/3*B$	$M_2 = W_2*L_2$	(+)
Weight of rectangular part of wing wall below RBL, W3	$W_3=(B+t)*d_1*\gamma_c$	$L_3=½*(B+t)$	$M_3 = W_3*L_3$	(+)
Static water pressure				
U/s water pressure, Ph_1	$Ph = 1/2* H*H*\gamma_w$	$L_4=1/3*H$	$M_4 = Ph_1*L_4$	(+)
Silt pressure				
Inclined overburden soil pressure, Ps_1	$Ps1 =1/2* H*(B-b)*\gamma_s$	$L_5=b+1/3*(B-b)$	$M_5 = Ph_1*L_5$	(+)
Vertical overburden soil pressure, Ps_2	$Ps2 =1/2* H*H*\gamma_s$	$L_6=1/3*h_s$	$M_6 = Ph_2*L_6$	(-)

Retaining wall design is said safe, when the average compression does not exceed the allowable compressive stress, otherwise reinforcement need to be provided in concrete at base of the walls.

These forces are presented for full flow condition; however, for no overflow condition, P_h and corresponding moment will not exist. Uplift pressure is assumed negligible so long as we provide weep holes in the body of the walls. Moreover, overburden soil pressures, P_{s1} and P_{s2} are treated separately as they are opposite in reaction i.e. P_{s1} (+) and Ps_2 is (-).

The base is usually constructed of reinforced concrete. Generally the factor of safety for structure stability (i.e. the ratio between stabilizing to destabilizing forces) should be at least 1.3 to ensure long term sustainability. For gravity retaining walls constructed on sound bedrock and adequately interlocked to the bedrock, stability is not a problem.

For a quick stability analysis on a retaining wall, Retwall software (that can be obtained online by own cost) can give us soil bearing loads, analyze the wall stability, calculate all wall moments, design our rebar arrangement and give us design sketches. The program handles flowing water and multi-layers of backfill. We can adjust all allowable values to meet whatever code requirements we are tied to.

The retaining walls need to be backfilled with granular backfill material. Its foundation needs to be filled and compacted from this material; however, this is often not possible as impervious backfill is required to prevent seepage. Clay backfill material of swelling nature should be avoided. If granular backfill is used, it is good practice to place an impervious soil layer at the top of the backfill to reduce the amount of infiltration.

3.15.13 Basic engineering property of materials

Basic construction materials to be used in the structural analysis of structures have their own unit weight. Such unit weights of common engineering materials are presented as follow.

Table 3-24: Unit weights of basic materials

SN	Dead Load	Weight (KN/m3)
1	Water	9.8
2	Stone masonry	23.0
3	Brick masonry	21.0
4	Mass concrete	24.0
5	Reinforced concrete	25.0
6	Steel	78.5
7	Timber(steel)	8.0
8	Wood(teak)	6.0
9	Dry backfill	16.0
10	Saturated backfill	20.0
11	Submerged compacted backfill	10.2
12	Dry compacted backfill	18.5
13	Saturated compacted backfill	21.5
14	Submerged compacted backfill	11.7
15	Gabions	14.0

Table 3-25: Internal angle of friction (∅) of Soil

SN	Soil Type	Angle of internal friction, ϕ
	Gravel	40^0-55^0
	Sand-Gravel	35^0-50^0
	Sand-Loose	28^0-34^0
	Sand-Dense	34^0-45^0
	Silt, silty sand- Loose	20^0-22^0
	Silt, silty sand- Dense	25^0-30^0

Note: For small structures a conservative value of $\phi=25^0$ is commonly used

Table 3-26: Allowable bearing pressure of soils

SN	Soil Type	Allowable Bearing Pressure (KN/m^2)
1	Soft clays and silts	< 80
2	Firm clays and firm sandy clays	100
3	Stiff clays and stiff sandy clays	200
4	Very stiff boulder clays	350
5	Loose well graded sands and gravel/sand mixtures	100
6	Compact well graded sands and gravel/sand mixtures	200
7	Loose uniform sands	< 100
	Compact uniform sands	150

Note: that for dynamic loads a 25% overstress may be allowed

3.15.14 Commonly used standard grades of concrete

Concrete is graded in terms of its characteristics strength. Compressive strength of concrete is determined from tests on 150mm cubes at the age of 28 days in accordance with standard issued or approved by Ethiopian Standard. Table below gives the permissible grades of concrete for the two classes of concrete works commonly used in our country. The number in the grade designation denotes the specified characteristics compressive strength in MPa.

Table 3-27: Commonly used Standard Grades of Concrete

	Class I	Class II	f_{ck}	Nature of Concrete	Mix ratio	Remark
Permissible grades of concrete	C5	C5		Blinding/lean concrete	1:5:10	Used under structure
	C7	C7		Mass/Plain concrete of roughest type	1:4:8	
	C10	C10		Mass/Plain concrete	1:4:8	Where more stabilization is req'd
	C15	C15	12	Unreinforced concrete	1:3:6	
	C20	C20	16	Standard-grade Reinforced concrete	1:2:4	
	C25		20	High-grade Reinforced concrete	1:2:3	
	C30		24	High Strength Reinforced concrete	$1:1^{1/2}:3$	
	C40		32	High Strength Reinforced concrete	1:1:3	
	C50		40	Higher Strength Reinforced concrete	1:1:2.5	High-rise structures and buildings
	C60		48	Supper High Strength Reinforced concrete	1:1:2	High-rise structures and buildings.

Note: Given ratios are dry-volume ratios of cement, sand, and coarse aggregates respectively, Civil Engineering Hand book

The amount of water added to these mixtures is about 1 to 1.5 times the volume of the cement. For high-strength concrete, the water content is kept low, with just enough water added to wet the entire mixture. In general, the more water in a concrete mix, the easier it is to work with, but the weaker the hardened concrete becomes.

Table 3-28: Standard mixes for ordinary structural concrete per 50kg of cement

Concrete grade	Nominal max size of Aggregate	40		20		14		10		cement per m^3 of concrete	
	Workability	Medium	high	Medium	high	Medium	high	Medium	high		
	Limit of slump that may be expected	30 to 60	60-120	20-50	50-100	10 to 30	30-60	10to 25	25-50	40	20
C5	Total aggregate (kg) Fine Aggregate (%) Vol. of finished con.(m3)	640 30-45 0.312	550 30-45 0.275	540 35-50 0.277	4800 35-50 0.252					1.70	1.89
C15	Total aggregate (kg) Fine Aggregate (%) Vol. of finished con.(m3)	370 30-45 0.200	330 30-45 0.183	320 35-50 0.277	280 35-50 0.252					2.61	1.89
C20	Total aggregate (kg) Fine Aggregate (%) Vol. of finished con.(m3)	305 30-35 0.165	270 30-45 0.155	280 30-40 0.156	250 35-45 0.143	255 35-45 0.146	220 40-50 0.130	240 40-50 0.137	200 45-55 0.121	3.13	3.34
C25	Total aggregate (kg) Fine Aggregate (%) Vol. of finished con.(m3)	265 30-35 0.147	240 30-40 0.137	240 30-40 0.137	215 35-45 0.127	220 35-45 0.130	195 40-50 0.118	210 40-50 0.124	175 45-55 0.110	3.52	3.89
C30	Total aggregate (kg) Fine Aggregate (%) Vol. of finished con.(m3)	235 30-35 0.134	215 30-40 0.127	210 30-40 0.124	190 35-45 0.115	195 35-45 0.115	170 40-50 0.106	180 40-50 0.109	150 45-55 0.097	3.83	4.18

Note: Compressive strength of concrete at 28 days in Mpa is given by the following formula from 7 day strength
$fc_{28}=fc_{7}+2.491fc_{7}$^0.5Mpa; Source: “The Civil Engineering Hand book”.

3.15.15 Reinforcement steel

The characteristic tensile strength of reinforcement bar to be used shall have yield strength not less than 400MPa ($f_y = f_{ck}$ = 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 below.

Table 3-29: Minimum RF-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× h/2 in both faces	
	h greater than 500 mm	3.25× 250 in both faces	
Ground slabs	h less than 300 mm	3.25× h/2	0
	h between 300 mm and 500 mm	3.25× h/2	3.25× 100
	h greater than 500	3.25× 250	3.25× 100

Note: * Minimum reinforcement per metre run.

3.15.16 Structural analysis

Structural Analysis is the process for the determination of the actions on the structure due to all the possible applied loads as mentioned in section above: load on structures. The main 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 problem, 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.

Table 3-30: Permissible shear in concrete (N/mm^2)

100Ast/bd	Grade of Concrete		
	M-15	M-20	M-25
0.25	0.22	0.22	0.23
0.50	0.29	0.30	0.31
0.75	0.34	0.35	0.36
1.00	0.37	0.39	0.40
1.25	0.40	0.42	0.44
1.50	0.42	0.45	0.46
1.75	0.44	0.47	0.49
2.00	0.44	0.49	0.51
2.25	0.44	0.51	0.53
2.50	0.44	0.51	0.55
2.75	0.44	0.51	0.56
3.00	0.44	0.51	0.57

Source: Hydraulic Structures Design, A.E. 2009

3.15.17 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 divide the materials ultimate strength by further partial factor of safety.

Table 3-31: 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	1.0	1.0

Table 3-32: Partial factor of safety for loadings

Load combination	Ultimate				Serviceability all (γG, γQ, γW)
	Dead, γ_G	Imposed, γ_Q	Earth and Water, γ_Q	Wind, γ_W	
Dead and Imposed (+Earth and Water)	1.4 (or 1.0)	1.6 (or 0.0)	1.4	-	1.0
Dead and Wind (+Earth and Water)	1.4 (or 1.0)	-	1.4	1.4	1.0
Dead, Imposed and Wind (+Earth and Water)	1.2	1.2	1.2	1.2	1.0

For small scale diversion weir structure design, the structural strength requirement needs to check for bending moment (flexure), shear force and axial stress are quite sufficient.

3.15.18 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 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.

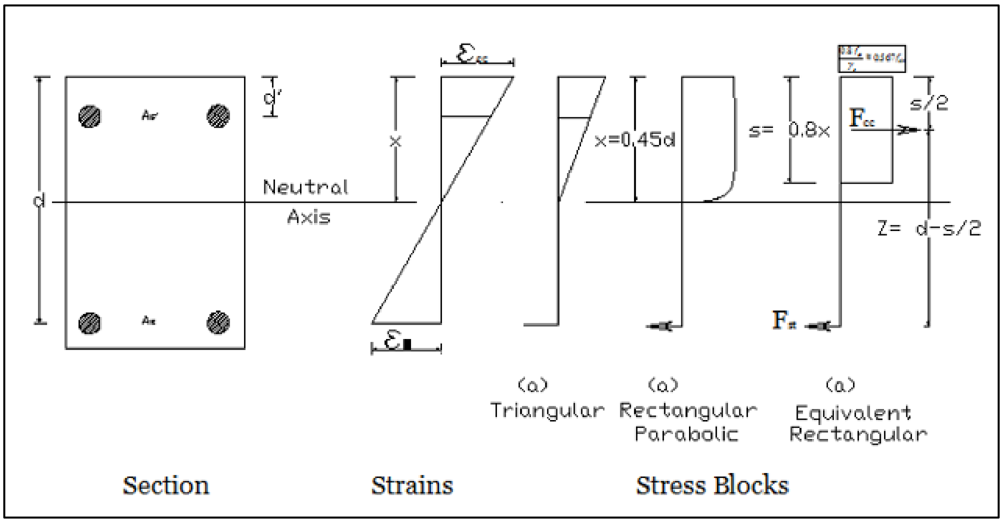


Figure 3-40: Section with stress diagram and stress block for singly reinforced section

Case-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.

Case-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.

Case-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:

$M = F_{cc}{*}\ z = F_{st}{*}z$ ………………………………………………………………………….. (3-158)

Where, z is the lever arm between the resultant forces F_{cc} and F_{st}

f_{st} is design strength of steel reinforcement (N/mm^2)

$F_{cc} = 0.567 * f_{ck}{*}bs$ ………………………………………………………….……………. (3-159)

$M = 0.567 * f_{ck}{*}bs{*}z$………………………………………………………..…………. (3-160)

Where, $z = d - s/2$ ……………………………..……………………………..………………. (3-161)

$M = 1.134 * f_{ck}b(d-z)z$…………………………………………………………….…. (3-162)

Where, $K = M/(bd^2 f_{ck})$ …………………………………………………………………. (3-163)

Therefore, $z = d\left[0.5 + \sqrt{0.25 - k/1.134}\right]$……………………………………………….….. (3-164)

Hence, $A_s = \frac{M}{0.87 f_{yk} z}$…………………………………………..……………………….…… (3-165)

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 ..…………… (3-166)

Hence, for balanced failure,

$M_{bal} = 1.134 f_{ck} b{*}(d-0.82d){*}0.82d = 0.167 f_{ck} bd^2$………………………………..…….. (3-167)

Therefore,

$$\frac{M_{bal}}{bd^2 f_{ck}} = K_{bal}$$ ……………………………………………………………..………... (3-168)

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

$K_{bal} < 0.167$……………………………………………………………..……………. (3-169)

3.15.19 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 below as per British Standard (BS 8110).

$$v_c = \left(0.79 * \left(\frac{f_{ck}}{25} \right)^{1/3} * \left(\frac{100\, A_s}{b * d} \right)^{1/3} * \left(\frac{400}{d} \right)^{1/4} \right) / \gamma_m \quad \text{.................................. (3-170)}$$

Where, ν_c is Maximum ultimate shear carrying capacity of RCC

b is width of rectangular section (mm)

d is effective depth of the section (mm), its value of should not be taken > 400mm

f_{ck} is characteristic tensile strength of RF bar and it should not be taken as > 40N/mm^2

A_s is area of reinforcement steel

γ_m is unit weight of material and shall be taken as 1.25

Note: The steel ratio should not be taken as greater than 3.

3.15.20 Water quality for concrete mix

Large amount of water is usually required in every construction site. Water is required for concrete mixing, curing, compaction for embankments, foundation moistening before placement, etc. Water for the above purposes can be obtained from river, spring, sea, lake, pond, groundwater, rain, etc. Water to be used in masonry and concrete works shall have the following properties:

- It shall be free from injurious amounts of oils, acids, alkalies, organic and inorganic impurities and
- It shall be free from mud, iron, vegetable matter or any substance which is likely to have adverse effect on concrete, masonry or reinforcement.

pH values between 6 and9 are usually don't need special precautions, but out of these ranges special protective measures such as increasing the cement proportion in the mix, increasing the dimensions of the section or corrosive resisting cement types shall be used. Similarly, sulphate attack ($S0_3$) is small if the water contains a concentration of sulphate less than 300mg/l, otherwise similar remedial measures have to be taken on the concrete works.

Some of the tests required in water for construction purposes include impurities and suspended material determination and chemical tests such as pH, sulphate, chloride, etc. The standard on material specifications (Part III.A) gives detail information on construction materials requirements.

3.15.21 Minimum structural member thickness requirement

The following table presents provision for the minimum concrete member and masonry wall thickness required for weir construction in small scale irrigation scheme context.

Table 3-33: Minimum structure thickness required for weir component

Structure Description	Recommended Minimum Requirement
Reinforced Concrete Structural Members	
Head Regulator, Scouring Sluice Gate and Operation Deck thickness	200mm
Head Regulator Side and Middle Piers thickness	800mm
Scouring Sluice Side and Middle Piers	1000mm
Breast Wall for Head Regulator	300mm
Breast Wall for Scouring Sluice	400mm
Stone Masonry Retaining Wall	
Masonry Wall top Cup Concrete Thickness	150mm
Masonry Wall Base Slab Thickness	200mm
Masonry Wall Top Width	500mm
Front Side Wall slope, Water Side	Vertical
Rare Side Wall Slope, Soil Side (Horizontal :Vertical)	1:2.5

Table 3-34: Recommended Slump for varies types of constructions

Type of construction	Slump (m)*	
	Max.	Min.
Reinforced foundations, walls and footings	175	50
Plain footings, caissons and sub structure walls	10	25
Slabs, beams, and reinforced walls	150	75
Building columns	150	75
Pavements	75	50
Heavy mass construction	75	50

*When high frequency vibrators are used, the values given above should be reduced by about one third.

Table 3-35: Maximum size of aggregate recommend for varies types of construction

Minimum Dimension of Section (mm)	Maximum Size of Aggregate (mm)			
	Reinforced Walls, Beams and Columns	Un-reinforced walls	Heavily Reinforced slabs	Lightly Reinforced Un-reinforced Slabs
62.5 - 125	12.5 - 20	20	20 - 25	20-40
150 - 275	20-40	40	40	40- 80
300 - 275	40- 80	80	40- 80	80
750 or more	40- 80	160	40- 80	80-160

Box 3-13:

Worked Example-13: *Working stress and limit state design approaches have been stated as design approaches for structural aspects of engineering activities. But what are the basic differences between these two approaches?*

Answer: *For working stress approach, service loads are used in the whole design and the strength of material is not utilized in the full extent. In this method of design, stresses acting on structural members are calculated based on elastic method and they are designed not to exceed certain allowable values. In fact, the whole structure during the lifespan may only experience loading stresses far below the ultimate state and that is the reason why this method is called working stress approach. Under such scenario, the most economical design can hardly be obtained by using working stress approach which is now commonly used in the design of temporary works.*

For limit state approach, for each material and load, a partial safety factor is assigned individually depending on the material properties and load properties. Therefore, each element of load and material properties is accurately assessed resulting in a more refined and accurate analysis of the structure. In this connection, the material strength can be utilized to its maximum value during its lifespan and loads can be assessed with reasonable probability of occurrence. Limit state approach is commonly used for the majority of reinforced concrete design because it ensures the utilization of material strength with the lowest construction cost input.

4 EXPANSION/CONTRACTION JOINTS IN STRUCTURES

4.1 NEED FOR INTRODUCTION OF CONTRACTION/EXPANSION JOINT

This part of the guideline is aimed to enable designers' to select appropriate contraction joints. Contraction joints are used mainly to control locations of cracks caused by shrinkage of concrete after it has hardened. If the concrete, while shrinking, is restrained from moving, by friction or attachment to more rigid construction, cracks are likely to occur at points of weakness. Expansion joints accommodate volumetric increase due to rise in temperature besides preventing transfer of stress between different exposed units of the structure. Contraction and expansion joints are constructed in such a manner that there is no bond between the adjacent units of the structure.

A need for introduction of expansion joint in concrete structures normally contains the following components: joint sealant, joint filler, dowel bar, PVC dowel sleeve, bond breaker tape and cradle bent.

4.2 JOINT SEALANT

This seals the joint width and prevents water and dirt from entering the joint and causing dowel bar corrosion and unexpected joint stress resulting from restrained movement.

Joint sealants are typically fluid, gel, or solid agents used to seal construction gaps in masonry, asphalt, timber, or steel structures. The use of a joint sealant serves the dual purpose of creating a physical barrier to exclude water, air, or dirt while creating an aesthetically pleasing finish to otherwise unsightly joints. The two most common types of joint sealant are fluid/gel and preformed solid seals. These joint sealants are specifically engineered with specific physical attributes and may also have additives such as flame or bacterial retardants included in their formulations.

Joints in construction elements are an unavoidable and often essential feature of most industries. Whatever their function, most joints require sealing to keep out moisture or air and exclude plant and dirt intrusion. In the case of decorative structures such as walkways, patios, decks, and pool paving, the joint sealant should also make for a visually pleasing finish. Joints in high traffic or stress applications such as sidewalks, roads, and bridges need to exhibit superior abrasion and shock resistant qualities along with their general sealing characteristics.

Figure 4-1: Sealant as seen in joints of concrete slabs

4.3 JOINT FILLER

A filler is a rigid material that supports the edge of the joint when heavy traffic crosses. This type of material is only effective with saw-cut joints; rounded tooled edges can't support the filler. It is compressible rubber type filler so that the joint can expand freely without constraint. Someone may doubt that even without its presence, the joint can still expand freely. In fact, its presence is necessary because it serves the purpose of space occupation such that even if dirt and rubbish are intruded in the joint, there is no space left for their accommodation.

Figure 4-2: Joint fillers in concrete works

Note: Both sealers and fillers should only be installed after the slab has had a chance to shrink as much as possible. Fillers are only effective if installed after the concrete has gone through most of its shrinkage, although that can take a year or more. Fillers and sealers should be checked at the end of the first year of service and repaired or replaced as needed. Effective sealant materials must bond to the concrete, be impermeable, and be able to handle the expansion and contraction. Before installing a sealant, the joint must be dry and free of dust and debris. Vacuum it thoroughly before sealing. Carefully follow the sealant manufacturer's installation instructions.

There are two types of fillers: Concrete Control Joint Filling and Concrete Expansion Joint Filling. Concrete control joints are intended to be cut the first or second day following placement of the slab at either 25% or 33% percent depth (depending on the day cut). Their purpose is to “control” stress cracking in the slab as the concrete expands and contracts with changes in moisture and temperature. If desired, control joint filler can be installed. Polyurea joint filler is intended to give the concrete joints protection under weight and traffic. This is a 2-part semi–rigid product that cures quickly, reaches high compressive strength, and forms a 3-sided bond.

The concrete expansion joint is visibly larger than the control joints and they allow for movement of the concrete slab due to vibration, settling, or temperature changes. The most common type of concrete expansion joint filler is caulk. Expansion joint caulk (or expansion joint sealant) is typically installed over backer rod or other foam insert, it forms a bond on 2 sides only.

4.4 DOWEL BAR

This is a major component of the joint. It serves to guide the direction of movement of concrete expansion. Therefore, incorrect direction of placement of dowel bar will induce stresses in the joint during thermal expansion. On the other hand, it links the two adjacent structures by transferring loads across the joints.

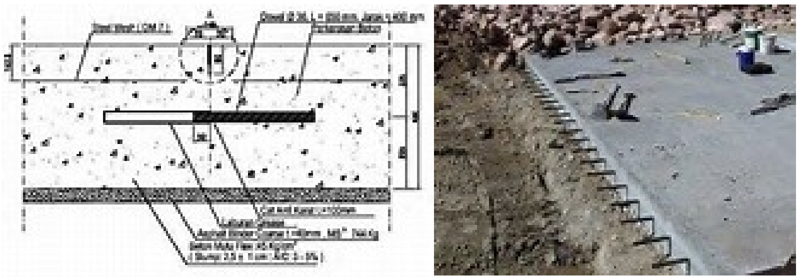


Figure 4-3: Dowel Bar Arrangement in Concrete Works

Dowel bars are typically placed at the mid-depth of the slab and should be parallel to the pavement surface and parallel to the direction of travel. The center of the dowel bar should be below the joint.

4.5 PVC DOWEL SLEEVE

It serves to facilitate the movement of dowel bar. On one side of the joint, the dowel bar is encased in concrete. On the other side, however, the PVC dowel sleeve is bonded directly to concrete so that movement of dowel bar can take place. One may notice that the detailing of normal expansion joints in structures standard drawing is in such a way that part of PVC dowel sleeve is also extended to the other part of the joint where the dowel bar is directly adhered to concrete. In this case, it appears that this arrangement prevents the movement of joint. If this is the case, why should designers purposely put up such arrangement? In fact, the rationale behind this is to avoid water from getting into contact with dowel bar in case the joint sealant fails. As PVC is a flexible material, it only minutely hinders the movement of joint only under this design.

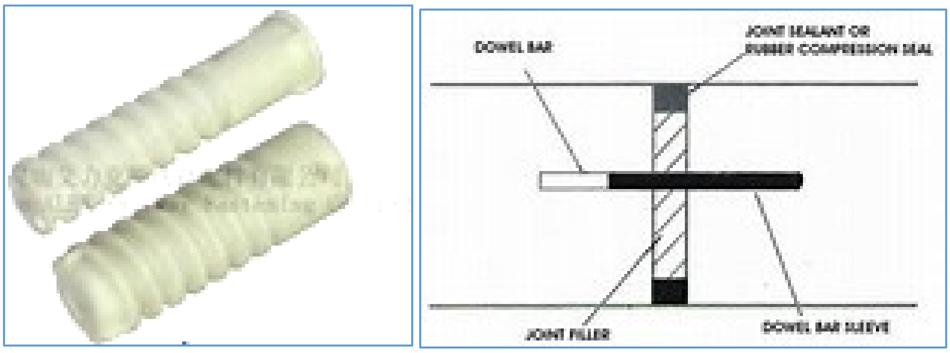


Figure 4-4: Plastic screw dowel and dowel bar sleeves

4.6 BOND BREAKER TAPE

A bond breaker is a substance applied over concrete surfaces to ensure that there is permanent bonding between the surfaces. Bond breakers are used normally on tilt-up walls and precast segments to ensure the right pieces are cast together. Bond breakers allow lifting and moving of precast pieces after stripping them from the form. As the majority of joint sealant is applied in liquid form during construction, the bond breaker tape helps to prevent flowing of sealant liquid inside the joint. Cradle bar: It helps to uphold the dowel bar in position during construction.

Bond breakers are used over concrete surfaces to eliminate or reduce the cracking of slabs due to temperature and moisture fluctuations. Bond breakers also avoid shrinkage cracks on tilt-up walls. Bond breakers are applied on surfaces that will be eventually joined together, since once the concrete is poured, they will be inseparable. Bond breakers are available in different forms, such as: Liquid, Spray, Rods, and Tape. Bond breakers are engineered products that form a membrane, allowing the surfaces to be separated easily. In some countries wax, petroleum-based substances or grease are used as bond breakers but these chemicals will change characteristics of the surfaces on which they are applied.

Bond breakers are classified into two major groups: membrane forming and non-membrane forming. They can also be divided into water based or non-water based products. Membrane forming bond breakers hold water in the casting slab and they are formulated to meet ASTM C-309, the standard specification for liquid-membrane-forming compounds. These types of bond breakers are made of crude resins to form the thin film. Non-membrane bond breakers are subdivided into reactive and non-reactive. The reactive bond breakers react forming a crude soap. The non-reactive bind breakers interact with the concrete surface and generate a waterproof surface.

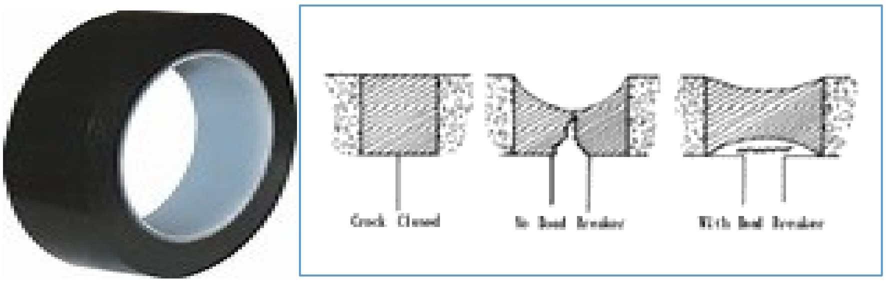


Figure 4-5: Bond breaker and its effect on concrete crack

4.7 WATER-STOPS

The principal function of waterstops is to prevent liquids (e.g. water), water-borne materials and solids to pass through concrete joints. In essence, it aims at providing water tightness to the drainage channel. Besides, waterstops in drainage channels or box culverts can also serve two other purposes:

- To avoid water contacting joints' dowel bars and causing corrosion.
- To avoid water seeping in from the underside of drainage channels or box culverts, thereby washing in soil particles and causing voids underneath these structures and finally leading to their failure.

To serve the second purpose, obviously only one waterstop is required at any depth location. To serve the first purpose, a waterstop has to be installed on top of dowel bars to prevent water from drainage channels from leaking through. On the other hand, a water stop has to be provided below dowel bars to avoid underground water from surging upwards. In fact, the other way out to serve the first purpose is by using corrosion resistant bars.

Water stops can be types: PVC type, Rubber water stop and Bentonite strip water stops.

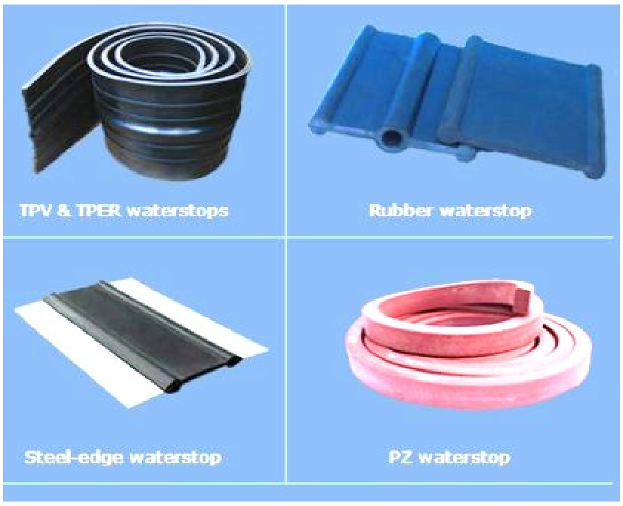


Figure 4-6: Different types of water stops

4.8 WEEP HOLES

Usually, the weep holes of 2 inch PVC at every 1m interval for high guide walls and additional expansion joints for long masonry work at headwork sites and retaining structures are provided.

"Standard Specifications for Highway Bridges."(American Association of State Highway and Transportation Officials) requires that contraction joints be provided at intervals not exceeding 9m. Alternate horizontal bars should be cut at these joints for crack control. Expansion joints should be located at intervals of up to 27m.

5 RIVER TRAINING WORKS AROUND HEADWORKS

5.1 NEEDS FOR RIVER TRAINING WORKS

Natural processes and human interference may disturb the equilibrium between the sediment load contributed to the channel and the transport capacity of the flow. Seasonal variations in the flow, dredging of the river, construction of a reservoir, construction of a weir and deforestation in the catchment area are all examples of causes of such disturbance. Training structures are thus necessary in order to protect the channel against the changes that occur due to this disturbance.

This guideline, is focused to river training works that need to be considered so as to protect overtopping of the channels as a result of introduction of hydraulic structures such as diversion weir, cross drainage structures, bridges as well as channel stabilization within the project boundary. Thus, river training here refers to the structural measures which are taken to improve a river and its banks. Such training is an important component in the prevention and mitigation of flash floods and general flood control, as well as in other activities such as ensuring safe passage of a flood over a weir and under or over cross drainage structures.

5.2 OBJECTIVES OF RIVER TRAINING OR IMPROVEMENT WORKS

The objectives of river training or improvement works are to prevent overtopping and hence flooding, to reclaim or protect land around river banks or to provide protection against conveyance canal for irrigation water supply, hydropower development or domestic and industrial use and/or to aid navigation.

River training or improvement works are mainly implemented for two purposes:

- Maintain normal flow depth of design flood within the existing reach of rivers,
- Reduce effect of erosive nature of rivers around structures under consideration.

Specifically, the objectives of river training are summarized as:

- To increase safety against flooding by accommodating the flood flow,
- To improve the efficiency of the sediment transport,
- To minimize bank erosion by stabilizing the course of flow,
- To direct the flow to a desired river stretch/reach,
- To reduce the probability of meandering, and
- In most cases, the primary objective of river training is to improve navigation by maintaining channel depth (though it is not the objective of this guideline).

5.3 CONSIDERATIONS IN DESIGN OF RIVER TRAINING

5.3.1 Peak flood level

River training works are required particularly when a weir does not extend over the existing width of a river and/or backwater effect is significant. These comprise embankments and spurs, which may be stone-protected earthen embankments or gabion or masonry structures. The embankments need to be sufficiently high and robust to convey flood flows over the weir without overtopping or failing. They should be designed for a 1 in 25 year flood event where overtopping and failure of the bunds will not result in catastrophic failure of the weir and other costly structures. Otherwise they should be designed for a 1 in 50 year flood event.

5.3.2 Environmental impact assessment and socioeconomic considerations

The design of river improvements works in general is dependent upon fluvial geomorphology and wider river engineering aims and river mechanics. In any design of river training works, it is extremely important to consider a holistic approach and to incorporate environmental impact assessment and socioeconomic considerations, as it directly affects land adjacent to river banks and hence conflict of interest may arise. The proposed structures need to be environmentally friendly to maintain the natural riverine environment and ecology.

5.3.3 Freeboard requirement for levee

The minimum vertical distance between the maximum flood level and the top of the levee (the crown or crest) is generally taken to be 1.5 times the height of the wave (h_w), which is calculated from the following (Physical Methods for River Training, 2017):

$$h_W = 0.032 * \sqrt{V * F} + 0.763 - 0.271 * F^{1/4} \text{, for F< 32km} \quad (5\text{-}1)$$

$$h_W = 0.032 * \sqrt{V * F} \text{, for F< 32km} \quad (5\text{-}2)$$

Where, V is velocity of wind km/hr, and
F is fetch length, km.

5.3.4 Widths of levee

The top width of the embankment should be sufficient to keep the seepage line well within the levee. For a small levee, this top width is generally governed by the minimum roadway width requirements. The minimum top width (A) of an earthen levee can be calculated as follows:

$$W = \frac{H}{5} + 3 \text{, for a very low levee} \quad (5\text{-}3)$$

$$W = 0.55 * \sqrt{H} + 0.2 * H \text{, for a levee lower than 30 m} \quad (5\text{-}4)$$

$$W = 1.65 * \left(H + 1.5\right)^{1/3} \text{, for a levee higher than 30 m} \quad (5\text{-}5)$$

Where, H is the height of the levee.

Its bottom width is dependent on the expected height of dyke and pore-water pressure is safe on all the faces and phreatic line remains within the body of the dyke (refer Figure 5-6 for its cross section and profile).

5.3.5 Cost–benefit consideration

Flood protection schemes require a careful cost–benefit analysis to determine a suitable design discharge which depends on the type and necessity of land to be protected, type of structures and property to be protected and the processes involved. The return period of this discharge may vary from 1 to 100 years and in very special cases (large settlements, ancient historic monuments, nuclear installations, etc.) may be substantially higher. But for design of diversion weirs and cross drainage structures most commonly 50 years design flood level is adopted to design such protection structures. For flood protection at small crossings structures 25 years design flood level is enough.

When structures are designed for a flood less than the maximum probable value, there exists a certain amount of flood risk to the structures, nor is it economical to design for 100% flood protection. But protection against the highest rare floods is uneconomical because of the large investment and infrequent flood occurrence.

5.3.6 River cross sections

For designing flood protection structures river cross sections should be surveyed at upstream and downstream of the river reach requiring protection and longitudinal slope of this reach need to be determined.

The approximate river flood level for the design flood event can then be estimated using Manning's equation taking into account the surveyed cross sections, the slope of the river and an appropriate Manning's roughness coefficient "n" (refer appendix-I for ranges of Manning's roughness values). Where possible the river levels should be checked against trash marks left by floods, which should be noted when river cross sections are surveyed.

5.4 PRINCIPAL METHODS OF RIVER TRAINING

Flood-protection works include high-water river training (mainly by dykes), diversion and flood-relief channels with or without control structures, and flood-control reservoirs. The principal methods used to improve river channels are categorized in to two: River Regulation and Dredging/ Scouring.

5.4.1 River regulation

In river regulation method of training, the river is encouraged to follow its natural course or it may be straightened. But, the latter approach requires great sensitivity and should be used only with caution and due regard to environmental constraints is given. In the upstream reaches, the main problem is the short-term and seasonal variation of flow, high velocity, channel instability and shoal/raised area/ formation. In the middle and lower reaches, it is often necessary to raise river banks and carry out works reducing the channel width. To do this, there are a number of types of river training structures. The selection and design of the most appropriate structure depends largely on the project site conditions. River regulation training structures can be classified into two main categories: transversal protection structures and longitudinal protection structures.

5.4.1.1 Transverse protection structures

Transversal protection structures are installed perpendicular to the water course, such as around cross drainage structures. They are used to lower the river gradient in order to reduce the water velocity and protect the river bed and banks from erosion. Most of the rivers from highlands originate in the high mountains, where they have steep gradients giving the flow a massive erosive power. Moreover, intense rainfall and breakout events can accelerate the river flow to such an extent that the water has a significant impact on the watercourses and surrounding areas. Transversal protection structures are effective for controlling the velocity of such rivers and streams and reducing the development of flash floods.

Spurs: A spur or spur dyke, or groyne is a structure employed as an indirect way for protection of banks. They are in general cheaper than direct protection measures. They are made to project flow from a river bank into a stream or river with the aim of deflecting the flow away from the side of the river on which the groyne is built. Two to five structures are typically placed in series along straight or convex bank lines where the flow lines are roughly parallel to the bank (McCullah and Gray 2005). Such structure train a river to flow along a desired course by preventing erosion of the bank and encouraging flow along a channel with a more desirable width and alignment.

Spurs can be made from many materials including stone, for example in the form of gabions or in bamboo 'cages'; tree trunks and branches; concrete; or any material that is not easily detached by the river and is strong enough to withstand the flow and the impacts of debris.

Figure 5-1: Arrangement of groynes along river bank, gabion (L) Bamboo (R)

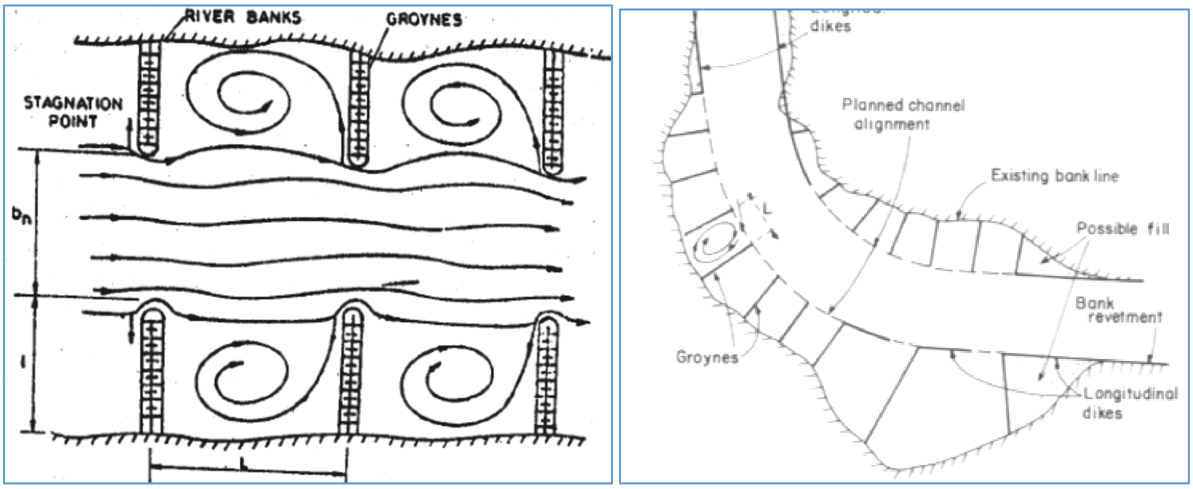


Figure 5-2: Schematic layout of groynes along river bank

Groynes can be categorized in to three: Attracting, Repelling and Deflecting Groynes. Attracting Groyne is designed obliquely to the bank by making an angle of 60^0 to 75^0toward the downstream/flow direction. Thus, flow of water is attracted towards the bank and the velocity of the flow is reduced to such an extent that it cannot cause any erosion to the bank. However, a bank protection of stone pitching is provided for safety (refer figure below).

In case of repelling groyne, the alignment is towards the upstream i.e. against flow direction at an angle of 60^0 to 75^0 with the river bank. Here a still water pocket is formed on the upstream where silting takes place. Thus, the bank protection is not necessary because the flow of water does not touch the bank and there is no effect of erosion on the bank. But still pitching should be provided for safety.

Deflecting groyne is designed perpendicular to the river bank and are also called ordinary groyne or normal groyne. Here flow of water is deflected from the bank by the perpendicular obstruction called Deflecting groyne. The flow of water follows an undulating path just outside the head of the groyne. An eddy current is formed on the upstream side of the structure. This eddy current will not affect the river bank. But a bank protection of stone pitching is provided for safety.

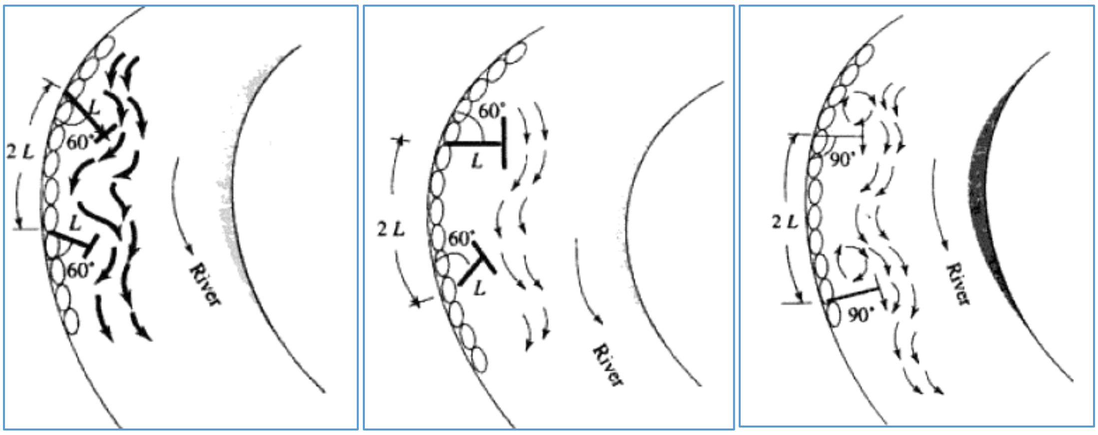


Figure 5-3: Attracting, repelling and deflecting Groynes (L to R resp.)

Table 5-1: Comparison between Spur and groyne

Spur	Groyne
It is a temporary structure	It is a permanent structure
It is permeable	It is impermeable
It requires repair works	It does not require any repair work
It is recommended for small rivers	It is recommended for large rivers
It is useful for low or medium velocity of flow	It is useful for high velocity of flow
It is constructed with bamboo pile, timber pile, sand bag, boulders, etc.	It is constructed with rubble masonry with cement mortar.

Source: Irrigation Engineering, by N. N.Basak, 2007

Check dams: These are weir like structures / low dams that are built across a stream bed to facilitate the bed-slope reduction. They can be made of gabions, concrete, logs, bamboo, and many other materials so as to decrease the morphological gradient of the torrent bed and reduce the water velocity during a flood event by increasing the time of concentration of the hydrographic basins and reducing the flood peak and solid transportation capacity of the river. They also help to reduce erosion and debris flow. The main purpose of check dams on rivers is to stabilize the riverbed over a long distance. Check dams generally require additional protection structures in the bed or on the banks to hinder undermining.

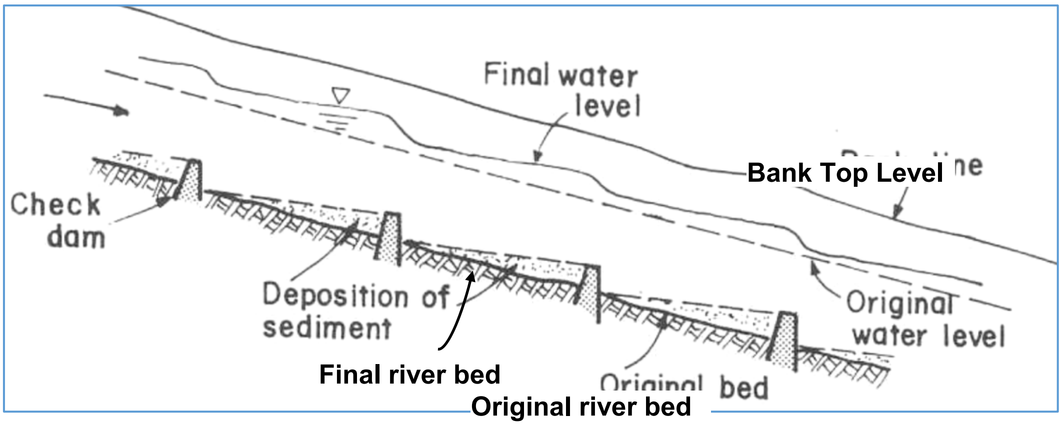


Figure 5-4: Arrangement of check Dams along River Reach

Check dams are usually designed such that the crest of the u/s dam is in line with the toe of the next check dam (if any) upslope. These structures should be used when it is not feasible or practical to line the channel or implement flow control practices.

Sills: A sill (also called a bed sill or ground sill) is a transverse gradient control structure built across the bed of a river or stream to reduce bed or head-ward erosion. Sills are installed along river stretches with a medium to low morphological gradient. They are used when small check-dams are not acceptable.

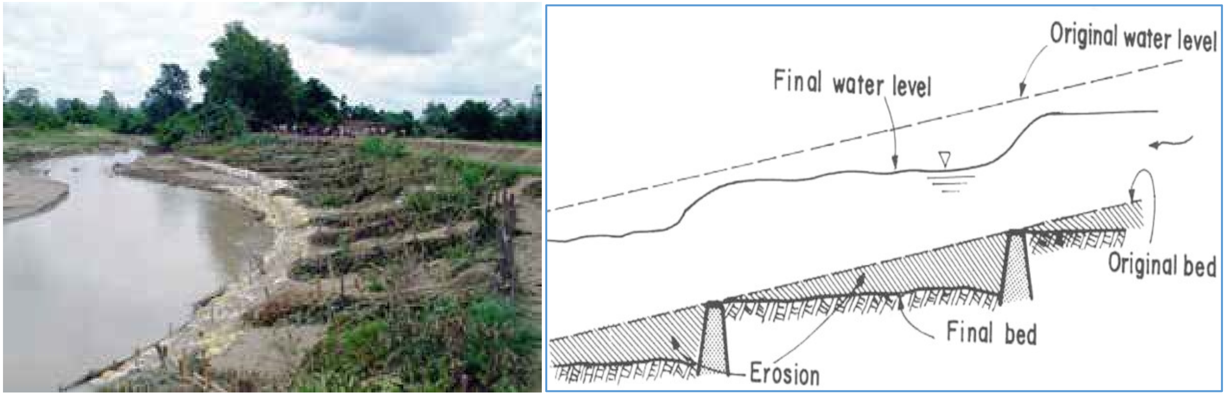


Figure 5-5: Arrangement of sill/Bed Sill along river bank (from Tree Trunks and Branches)

5.4.1.2 Longitudinal protection structures

Longitudinal protection structures are installed on river banks parallel to the river course, generally with the aim of protecting adjoining areas from inundation, erosion, and river meandering, typical example is protection around diversion weirs. They are usually constructed on natural banks and extend for a considerable distance. The most common structures are embankments or levees in the form of guide bunds or banks, afflux bunds, and approach embankments. Very often, spurs are constructed together with longitudinal structures to protect the latter.

Levees or Earth Fill Embankments/Dykes: Levees, or marginal embankments, are dam-like earthen structures constructed along a river in order to protect the surrounding countryside from flooding and/or to confine the course of a river to provide higher and faster water flow. They are usually constructed for long stretches along a river in low lying areas with an extended floodplain.

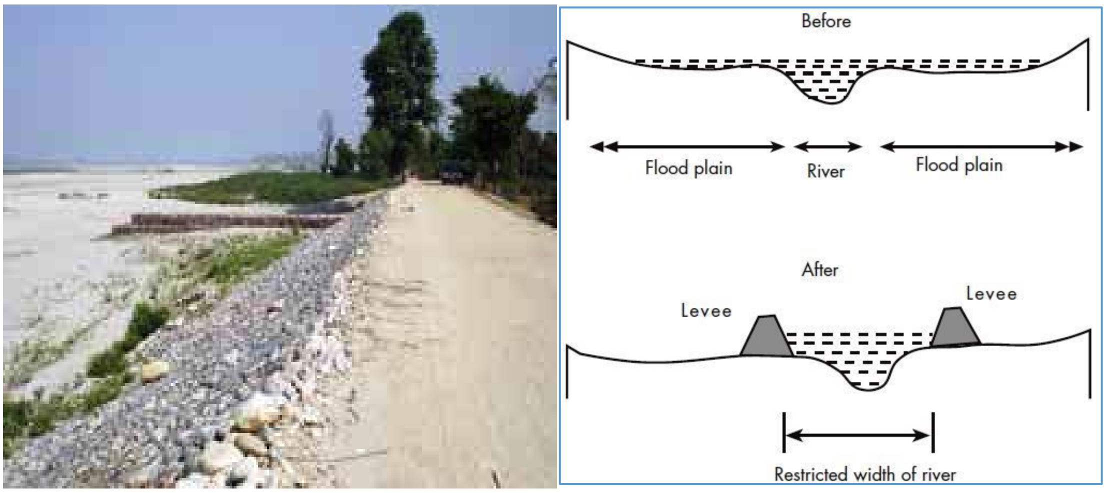


Figure 5-6: Levee/dykes arrangement along river bank and its section

The top width is generally 3 to 4 m and side slope is generally 1½: 1 to 2:1 (H:V). The height of the embankment depends on the level of the selected design flood. A suitable margin is provided between the toe of the embankment and the bank of the river. To resist the effect of erosion on the embankment, wooden piles are driven along the river banks throughout the length of dyke. The length of the dyke is protected by boulders pitching with cement grouting and the downstream side is protected by planting turf. They are built (a) To prevent the flood water or storage water from entering the surrounding area. (b) To retain the flood water or storage water within a specified section. (c) To protect the towns and village from devastation during heavy flooding. (d) To protect valuable agricultural lands from submergence.

Guide Banks and other Approach Embankments: Guide banks are extension of wing-wall structures built to guide a stream or river through a bridge opening or towards other hydraulic structures such as weirs, especially when river flow level is markedly higher than usual. The aim is to confine the river within a reasonable waterway and direct the flow in a manner that ensures its safe and expeditious passage. They also reduce or eliminate local scour at the embankment and adjacent piers. In a wide river lined by levees, a series of diversion structures may be used to guide and narrow the water course and protect the levee or highway embankment, where a highway or other bridge crosses the river. These consist of an afflux embankment or bund, an approach embankment, and the guide banks themselves.

Guide banks are constructed in a river in order to:

- Confine the flow to a single channel,
- Improve the distribution of discharge across the width of a river thus controlling the angle of attack by a flash flood,
- Protect weirs, barrages, or other hydraulic structures constructed in the river such as intakes from flash floods,
- Control the meander pattern of a river,
- Control overtopping of natural embankments in a flash flood and protect adjacent land from flooding,
- Reduce erosion of banks by the water current,
- Prevent sliding of soil as a result of the draw down effect of the flood water level,
- Facilitate smooth transportation of water, and
- Prevent piping of water through the banks.

Concrete Embankments: Concrete embankments are made from cemented bricks, stones, or concrete. These are thin but strong embankments usually installed in urban reaches of water courses where there is not enough space to build more massive structures. They can also be combined with earth fill structures. The construction cost of concrete embankments is higher than that of earth fill embankments and such an embankment has a significant impact on the environment and often destroys the ecology of riparian areas.

Revetments and rock riprap: Revetment refers to a continuous artificial surface on a river bank or embankment slope and part of the river bed, which is designed to absorb the energy of the incoming water and protect against erosion by the river current. Revetments are usually placed along the concave side of a river bend where river velocities are high. Upstream from barrages, revetments may be used to hold approaching river banks in their existing positions. Revetments can be flexible or rigid. They can be constructed from various materials including rock, stones, stone-filled gabions, concrete slabs, timber piles, bamboo piles, old tyres, and sandbags. If there is a potential for scour at the toe, the revetment must be extended down to the expected level of the scour and sufficient material added in the form of a thickened toe or horizontal apron such that the toe material will launch to a stable slope as the bed scour develops (For detailed design aspects of riprap, refer section 3.8.12).

Figure 5-7: Rock riprap (L) and protection by gabion at u/s of river crossing (R)

5.4.2 Dredging

Dredging is implemented using mechanical or suction dredgers and is the most effective means of estuarine or of confluence river regulation, but its impact is often only temporary, thus not considered in this guideline.

5.5 DESIGN OF RIVER TRAINING/FLOOD-PROTECTION WORKS

5.5.1 Required data

Data required for design of flood protection works are:

- The design flood levels established according to the economic value of the protected area or structure that need to be protected;
- The design freeboards against overtopping and wave attacks;
- The duration of the flood levels for calculating seepage and hydraulic gradients in earthen dykes and underground; and
- The probability of silt deposit and consequent backwater.

5.5.2 Alignment of flood protection works

Alignment of flood protection structures, (such as wing walls) need to be considered carefully, taking into consideration the following:

- Topography: for example island, outcrops, etc.
- The existing (and historical) river alignments;
- Farmers wishes, land ownership, location of other infrastructures such as houses, etc.
- The regime river width (for alluvial river beds);
- The effect of the proposed works on “others” outside the scheme area, such as on the opposite bank of the river to that being protected;
- The stability of the river bed, location of any rock outcrops, etc.

5.5.3 Cross section design of guide bank structures

5.5.3.1 General

Design of flood protection structures as related to diversion weir such as wing walls, dykes/levees and launching apron has been covered partly under hydraulic design aspect of this guideline and phreatic line analysis is in Small Dam Design Guideline, thus can be referred there.

Such flood protection structures can be designed and constructed from masonry, gabion or compacted clay materials thus selection of these materials depend on their availability, volume/quantity and quality in the vicinity of the project, cost, environmental impact assessment and socioeconomic considerations, etc., the most commonly used in relation to irrigation structures being longitudinal flood protection structures such as Levees or earth fill embankments or dykes, guide banks and revetments and rock riprap. Consequently, design of cross section of guide bank structures are presented as follow.

5.5.3.2 Length of the guide bank

Generally for shifting alluvial rivers, if any, the length depends on the distance necessary to secure a straight run for the river, and the distance necessary to prevent the formation of a bend in the river so as to avoid the angle of attack of the anticipated flash flood. In case of guide banks around diversion weir their length should be fixed such that they find and keyed to the natural ground corresponding to high flood levels shortly both on the upstream and downstream sides of the weir.

5.5.3.3 Plan shape of the guide bank

Ideally, the guide bank should have a converging curved shape forming a bell mouth entry to the waterway. The axis should be parallel to the principal direction of flood flow through the opening at crossing structures but governed by the shortest distance to HFL in case of weir. This shape is particularly suitable where the direction of flow can vary. In most cases, the main sections of the two banks are constructed parallel to each other, but other forms are possible, for example curved or converging. Refer typical sketch shown below.

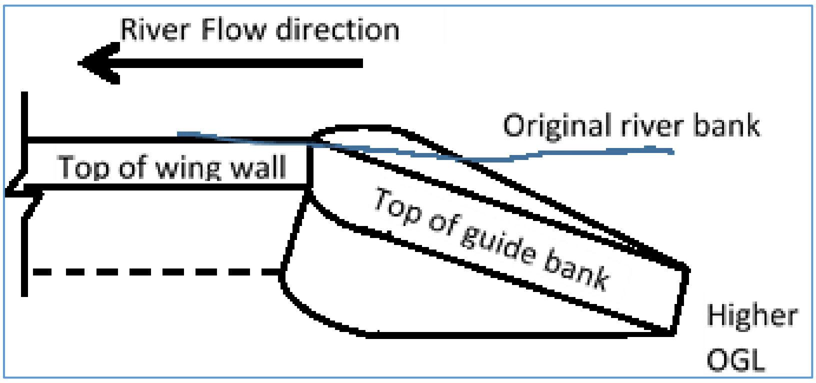


Figure 5-8: Schematic Plan of Guide Bank extended from Wing Wall

5.5.3.4 Embankment/guide bank section

The angle of the embankment slope is calculated according to:

- The subsoil conditions,
- the angle of repose of the embankment material, and
- The type of slope revetment provided, usually the slope should have a horizontal to vertical ratio of between 2:1 and 3:1.

In general, the top of the embankment is made wide enough to accommodate vehicles for construction and maintenance purposes. Guide banks should normally extend above the design high water level with a freeboard allowance of 1–1.5 m depending upon the discharge condition (Singh 1980). Lower guide banks that can be overtopped under high flood discharge condition may be preferred in some cases for economic purpose. However, under these conditions, the top of the bank and outside slope must be protected against erosion.

5.5.3.5 Spacing between the guide banks

The layout of the guide banks should be such as to guide the flood smoothly throughout the guide bank length. Generally, the guide banks are constructed to form a symmetrical pair. They should confine the river within a reasonable channel that can ensure safe and rapid passage of water during a flash flood. The confined width of the river between the guide banks in an alluvial river can be calculated using the Lacey's formula presented in (3-6).

5.5.3.6 Pitching

The inside slope of the embankment is subjected to erosion from the river flow, particularly during floods and flash floods. The continuous movement of water saturates the embankment material as a result of pore water pressure. Sudden increases and decreases in the water level can change the water inflow and outflow in the embankment material and damage the embankment. Hence, the inside slope should be protected by stone pitching. The usual thickness of the pitching varies from 40–60 cm. The thickness can be determined from the formula (Varshney et al. 1983):

$$t = 0.60\, Q^{1/3} \qquad \text{(5-6)}$$

Where, t is the thickness in meters and

Q is the maximum river water discharge in m^3/s

Thickness of stone riprap determination has also been given on Figure 3-19: arrangement of launching apron, thus can adopt either but the one that gives safe condition should be selected.

Stone pitching protects the face of the bank. However, floods can induce scouring at the toe which would undermine the pitching and cause its collapse. To prevent this, a stone cover or launching apron has to be laid beyond the toe of the bank on the horizontal river bed (Figure 3-19). As the scour undermines the apron starting at its farther end and working back towards the slope, the apron falls to cover the face of the scour, with the stones forming a continuous carpet below the permanent slope of the guide bank. The apron must have sufficient stone to ensure complete protection of the whole of the scour face. The length of the scoured face should extend to the anticipated scour depth below the apron and is given by:

$L_a = 5 * \sqrt{D}$ or 1.5*D …………….……………………………………………………….. (5-7)

Where, L_a is launching apron length, (m)

D is scour depth below the apron, (m)

The scouring effect is a function of the gradation of the silt available in the river bed and the discharge of the flowing water. It can be calculated using equation (3-81) or the following formula:

$R = 0.47 * 3\sqrt{\frac{Q}{f}}$ …………….……………………………………………………….. (5-8)

Where, scour depth below HFL, (m)

Q is the maximum discharge the river for 50 years, m^3/s,

f is Lacey's silt factor as defined in equation (3-83)

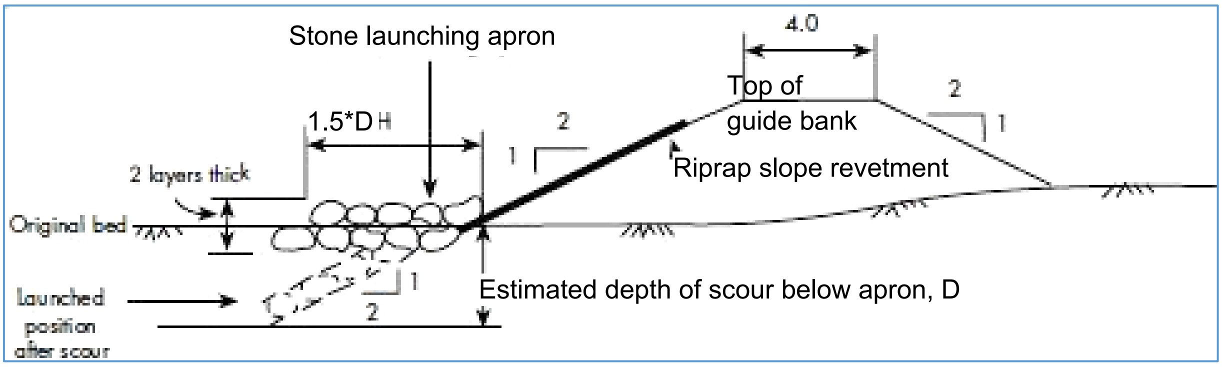


Figure 5-9: Typical Cross-Section through Guide Bank

Note: Numbers indicate relative values for any given size distribution

5.5.4 Design aspects of groynes

5.5.4.1 Length of groynes

Length between two successive Groynes in SSI projects is dictated by the intended channel alignment around the diversion structures and downstream reaches within the project command boundary. If the banks are deep enough to accommodate incoming flood such structures are not required,

5.5.4.2 Crest levels of groynes

For alluvial channels, the crest levels of the groynes are kept above the water level corresponding to the selected design discharge / dominant discharge selected for the intended return period plus 1-1.5m free board or as given under section 5.3.3,

The crest levels are either horizontal or inclined upward to meet the bank requirements. This helps to reduce the risk of scouring at higher floods especially at the bank-groyne junction.

5.5.4.3 Angle of groynes

No definite recommendation as to the angle between the groynes and the channel bank exist. In fact, experimental studies show some conflicting results

- Some studies show that groynes pointing upstream experience less scouring,
- Other studies show that groynes at 90^0 rather have less scouring,
- Other studies show that groynes pointing downstream experience less scouring.

In general, recent practices tend to favour an upstream inclination making 15-30 degree with a line normal to the flow.

5.5.4.4 Distance between groynes

So far no theoretical way to determine the distance between groynes. However, too short distance leads to expensive structure and too long distance leads to erosion of the banks. Thus, either of the following empirical rules of thumb are recommended:

- Twice the channel width;
- 1-5 times the length of the groynes;
- The distance is such that one strong vortex/eddy is formed.

5.5.4.5 Embankment materials

Embankment materials for the construction of the groynes is best taken from the stream bed material, if this proves unsatisfactory, rockfill may be used. Gabions can also be used

5.5.4.6 Scour Protection

The heads of groynes may be subjected to scouring and thus require protection such as mattress revetment or gabion can be used.

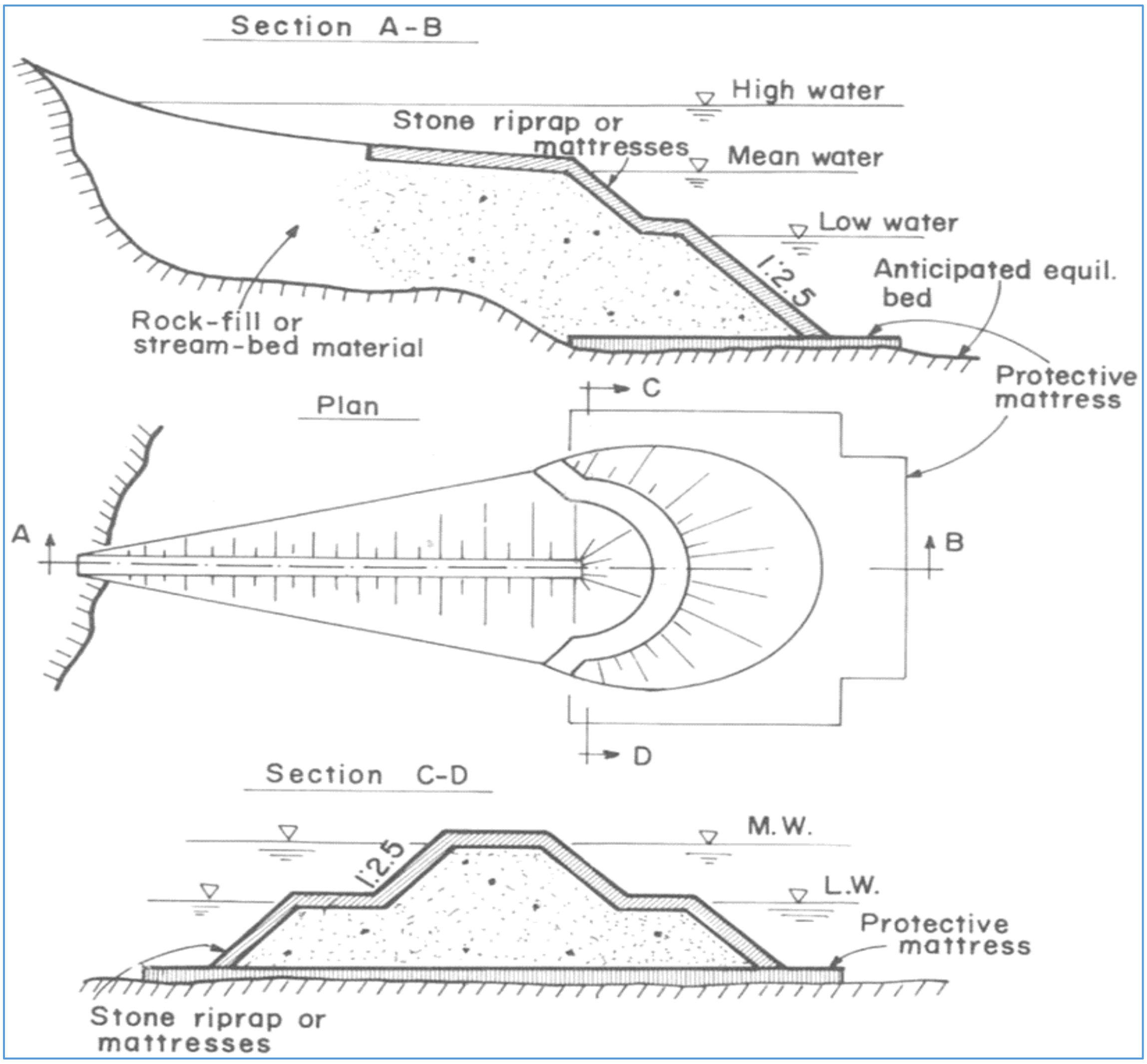


Figure 5-10: Typical Plan and Cross Sections of Groynes along River Bank

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APPENDICES

APPENDIX I: Typical Values of Manning's Roughness Coefficient of Channels

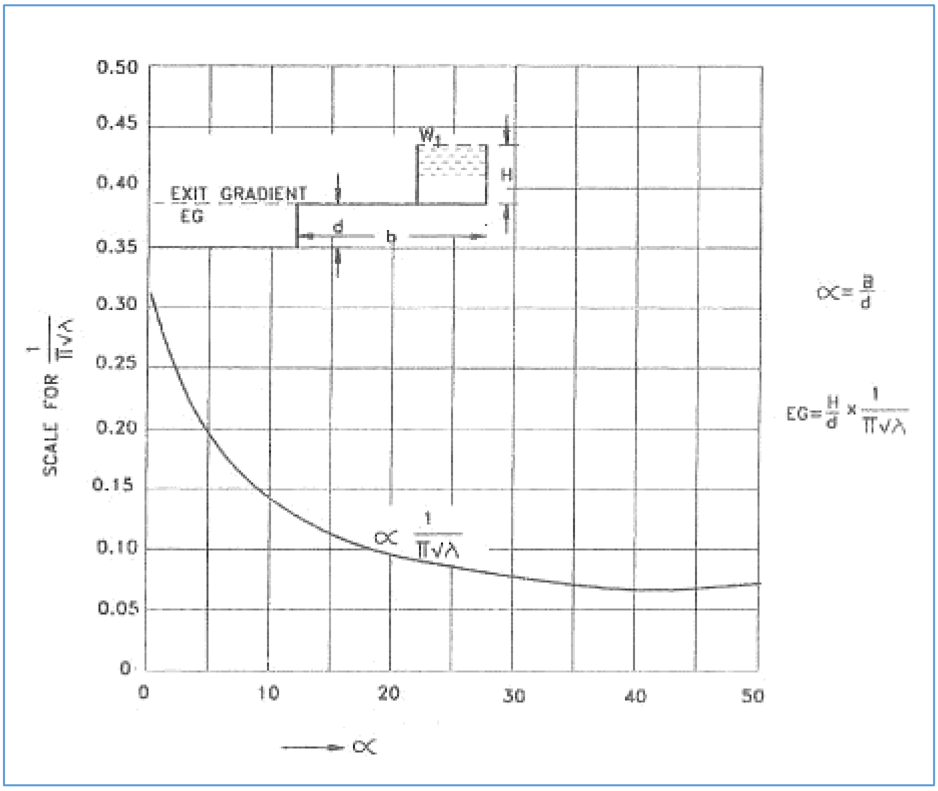
SN	Channel type	Manning's Values	Commonly Used Value
1	Concrete lined canals	0.012–0.017	0.012 for RCC and0.014 for lean Concrete
2	Rough masonry	0.017–0.030	0.018
3	Brick-lined channel	0.012–0.018	0.015
4	Roughly dug earth canals	0.025–0.033	0.025
5	Smooth earth canals	0.017–0.025	0.018
6	Earth channel: very overgrown with weeds, etc.	0.050-0.120	0.080
7	Floodplain	0.025–0.033	0.030
8	Natural river in gravel	0.040–0.070	0.040

Source: Practical Hydraulics, By Melvyn Kay, 2008

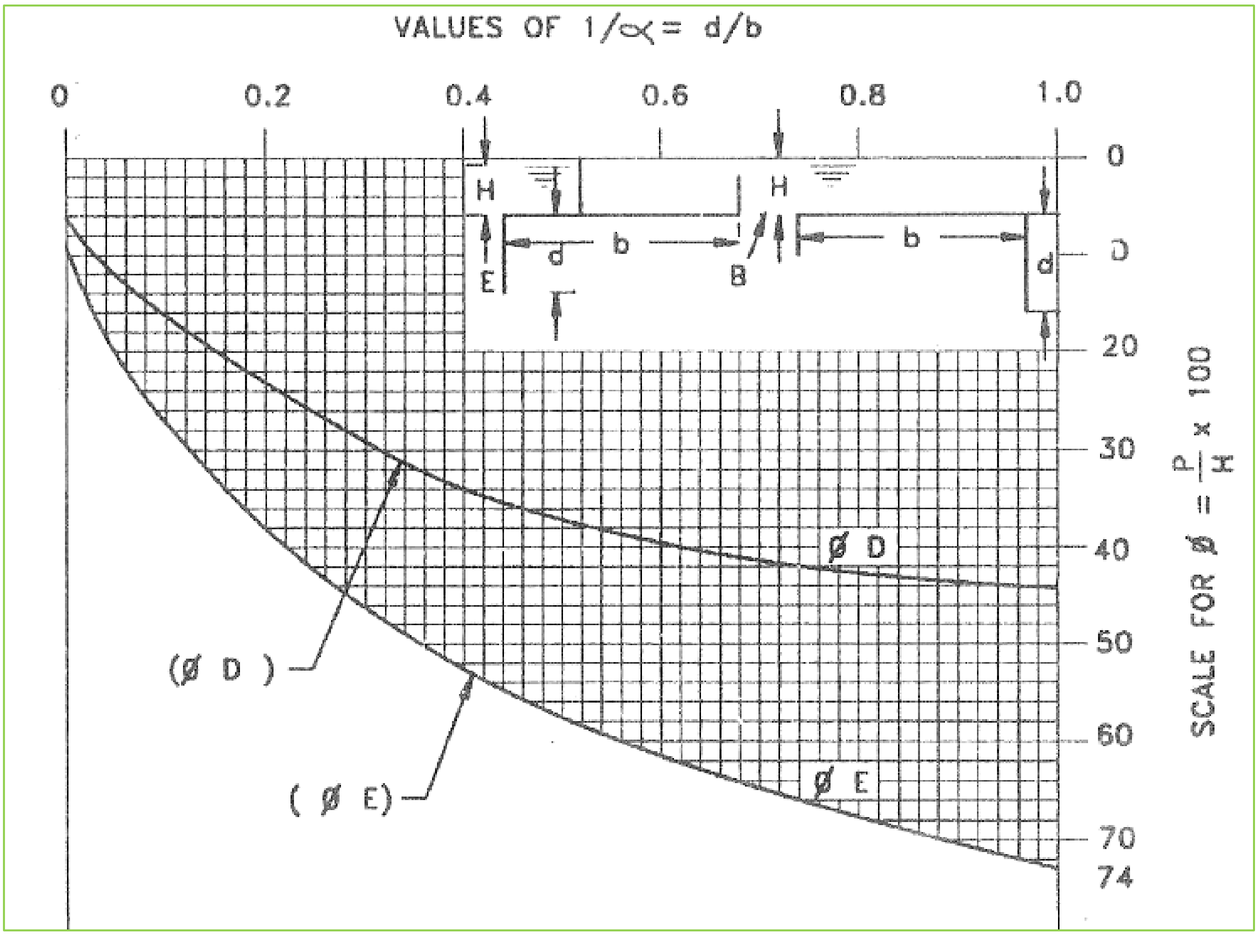
APPENDIX II: Allowable bearing capacity

Supporting Ground Type	Description	Compactness or Consistency	Presumed Bearing Value (t/m2)
Rocks	Basalt	Hard and sound	400
	Slate, schist	Medium Hard	200
	Sandstone, limestone	Medium Hard	200
	Soft limestone	Soft	100
	Soft shale	Soft	60
Non-cohesive soil	Gravel, sand and Gravel	Dense	40
		Medium dense	30
		Loose	20
	Sand	Dense	30
		Medium dense	20
		Loose	10
Cohesive soil	Silt	Hard	20
		Stiff	15
		Medium stiff	10
		Soft	5
	Clay	Hard	30
		Stiff	20
		Medium stiff	10
		Soft	5

APPENDIX III: Graphical Determination of Exit Gradient (Khoslas theory)



APPENDIX IV: Khosla's Graph to Determine Uplift Pressure at Cut Off Ends



APPENDIX V: Open Weir, Weir Crest and Contraction Factors

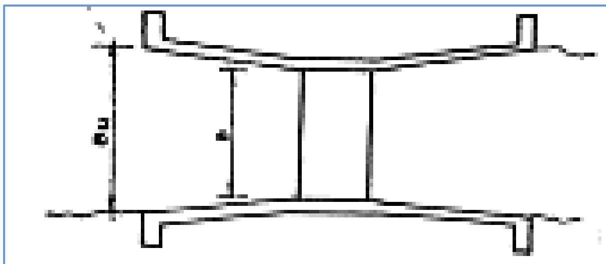


Fig 2.1.A Open weir on perennial rivers

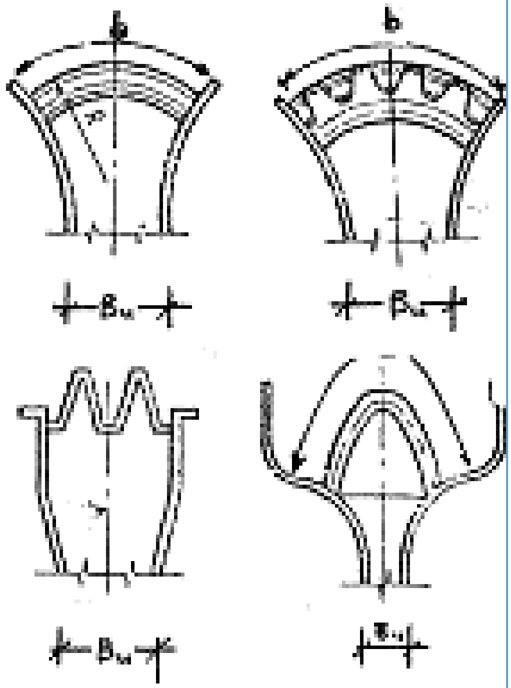


Fig 2.1.B Elongated weir crest

Basic design formulas for open weir

$Q=2.95CLh^{3/2}$ $\quad C_d=0.602+0.075(h/p_u)$

$D_r=\frac{q^2}{gP_u^3}$, $\quad L_j=L_d+8d_c$ $\quad L_d=4.3D_r^{0.27}P_u$

$Lc=\sqrt[3]{\frac{q^2}{g}}$, $\quad L_j>=2.55d_c$ $\quad Y_b=0.8d_c$

$W_b=0.4d_c$ $\quad S_b=0.4d_c$ $\quad Y_s=0.4d_c$

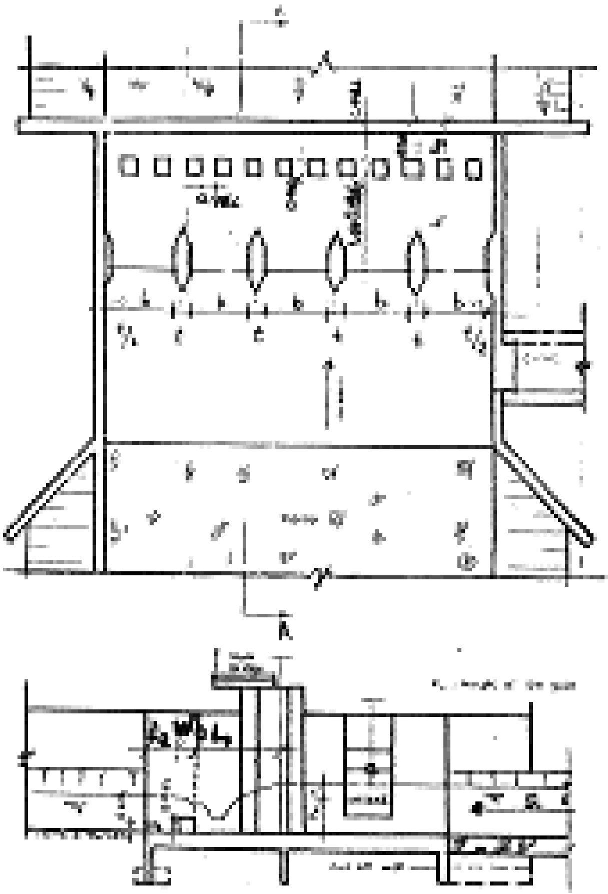


Fig 2.1.A Open weir on seasonal or regulation-oriented river

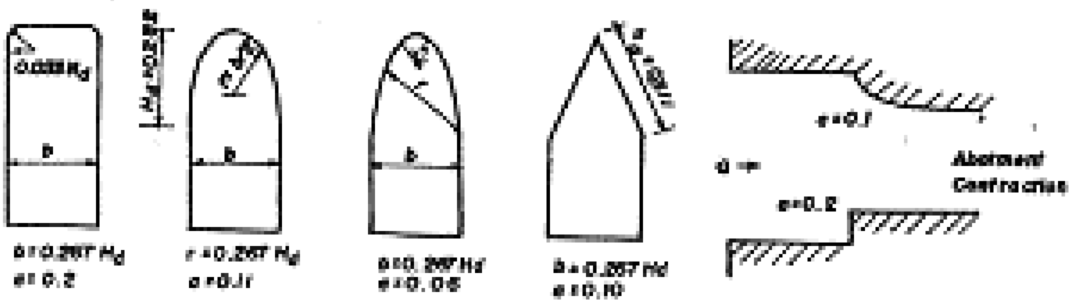
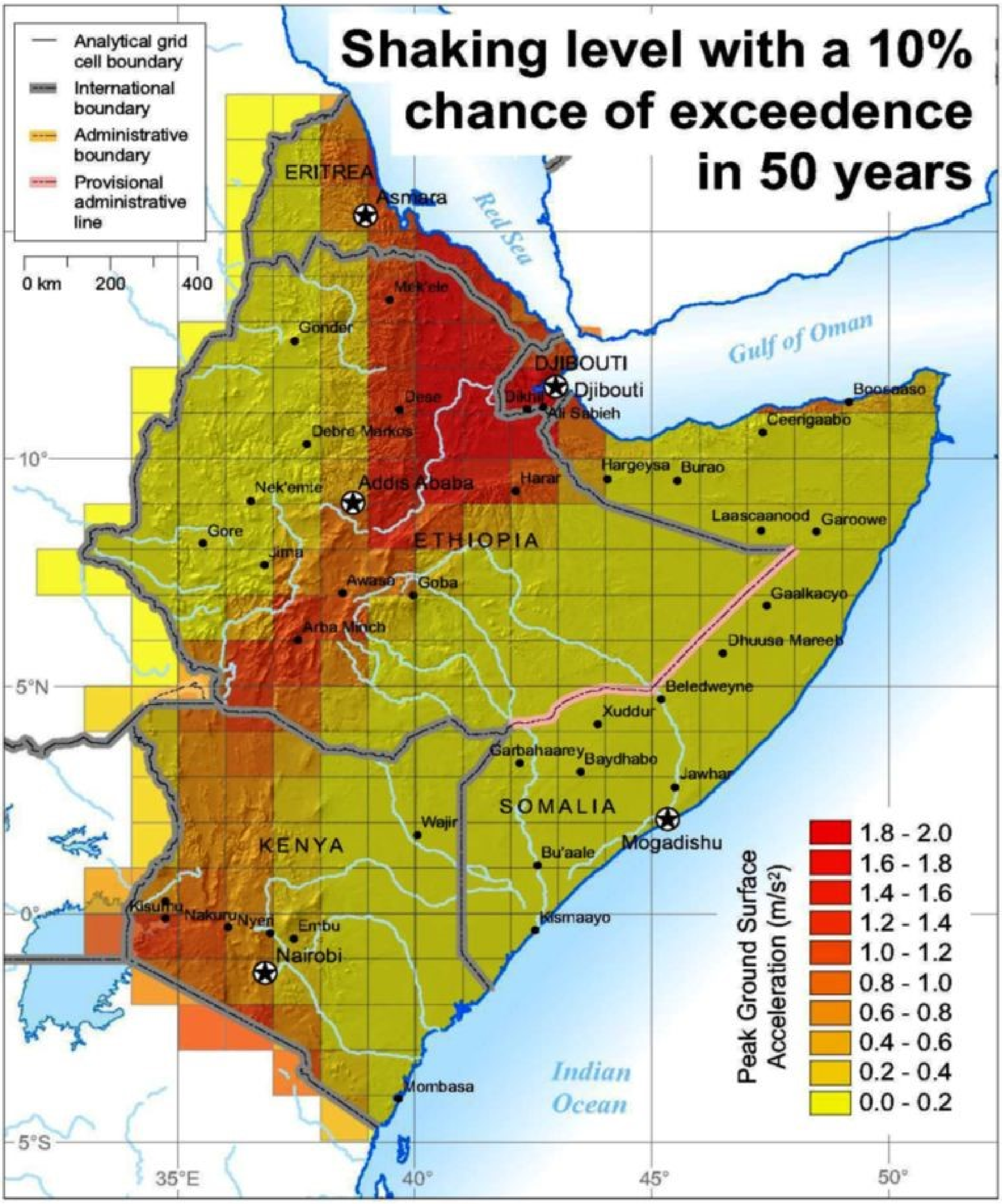


Fig 2.1.C pier & abutment contraction, pier dimension and coefficient of contraction

Source: As adopted from MoWR, PART I-G Diversions, 2002

APPENDIX VI: Seismic Risk Map showing 1:50 earthquake acceleration



Source: Adopted from Hydraulic Structures Design Guideline for SSI Projects in Amhara, 2009

APPENDIX VII: Maintenance Activities and Procedures for Diversion Headwork

(i) Removing dumped and/or anchored wood

- Check the river flow and bank stability condition. If the river condition is safe, remove the drifting wood. If the river condition is dangerous, do not try to remove the drifting wood as it may harm you.
- Remove drifting wood. If the drifting wood is big and difficult to remove, handsaw/axe has to be prepared to cut the wood in to pieces.
- Check the gate condition after removing drifting wood. If the gate can be operated smoothly, there is no problem. If the gate cannot be operated smoothly, the gate itself needs maintenance.

(ii) Painting

- Remove rust by wire brush and sand paper.
- Remove dust from the surface of iron
- Paint by brush three times. Paint should be rust preventive.

(iii) Greasing

- Remove dust from the spindle
- Put grease on the spindle
- Operate gate up and down to put grease

(iv) Chain block

- If there is problem on the chain block, it is necessary to bring it to maintenance workshop.

(v) Chain

- The chain on the beam should be bolted by spanner. If the bolt is loose, the bolt should be tightened immediately.
- When problem (e.g. elongation) is found on the chain, the chain must be replaced.

(vi) Dredging/desilting from main canal

- Open the under sluice/s to stop water flowing into the main canal.
- Wait for some days till soil in the main canal dry.
- Remove soil from the canal by using hoe, shovel, Spindle

(vii) Repairing crack/s

- Clean the surface of canal structure to find crack clearly
- Put chalk to make clear crack line
- Chisel crack part to be U shape ditch (Width: 10-50mm, Depth: 10-50mm) by chisel and hummer
- Remove dust from the U shape ditch
- Mix mortar and put it into the U shape ditch. Materials required are Cement, Sand, Bucket to keep water for mixing mortar, trowel, and shovel. Mixing ratio of mortar is commonly 1 (cement): 3 (sand) for structures subject to water.
- Cure the mortar at least 2 days after casting mortar it.

(viii) Concrete work

- Prepare platform to mix concrete on it.
- Put sand and cement according to the mix ratio (1 (cement):2 (sand):4 or as required (aggregate) for plain concrete (C-15), 1: 2: 3 for reinforced concrete(C-20)) on the platform. Measurement box (L: 50cm, W: 40cm, D: 18cm) should be prepared ahead.
- Spread the sand evenly on the platform.
- Dump cement on sand and spread evenly.
- Mix the sand and cement thoroughly with shovel/hoe until it is of even color and free from streaks.
- Spread aggregate in another place of the platform by shovel/hoe and spread the sand-cement mixer on it.
- Thoroughly mix the whole mass by shovel/hoe and turning over by twist from centre to side then back to the centre and again to the sides.
- Make a hollow in the middle of the mixed materials to add water. Water-cement ratio should be 50% in weight.

- Add required water gradually and turn the materials from side to centre with spade.
- Continue mixing until all material is mixed uniformly.
- Cast concrete to the place where repair is needed and compact concrete by vibrator/wooden stick. Buckets are needed to bring concrete to the casting site, if there is some distance from the mixing place.
- Cure the casted concrete at least 5 days after casting concrete.
- Finish surface by trowel

(ix) Mortar work

- Prepare platform (like. CIS) to mix mortar on it.
- Put sand and cement according to the mix ratio (1 (cement):3 (sand)) on the platform. Measurement box (L: 50cm, W: 40cm, D: 18cm) should be prepared. Water-cement ratio should be 50% in weight.
- Mix sand and cement until it is of even color and free from streaks by shovels/hoes.
- Make hallow on the surface of the mixed material to add water. Bucket should be prepared.
- Add water gradually and mix.
- Cast mortar by trowel where the mortal is needed.
- Cure the mortar at least 2 days after the casting it.

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Prepared by

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